



Internship Report

Master in Civil Engineering an “Building Construction”

***Design of the confined masonry building in Quito,  
Ecuador***

**Anna Zhgun**

*Leiria, September of 2018*

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

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# Dedication

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To my mother, who with great humility, efforts and sacrifices, supported me in this hard journey, and that in the difficult moments she always calmed me down and encouraged to keep going, thank you for teaching me to dream and that everything is possible if it is done with effort and dedication.

To the teachers who contributed to my education as a student.

To my relatives, friends and colleagues, who always supported me on this long road.

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# Acknowledgements

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I would like to thank my supervisor, Professor Hugo Rodrigues, for the patience, motivation, encouragement and advice he has provided throughout my time as his student. His guidance helped me in all the time of my study. His experience and knowledge that he is sharing with his students are priceless.

My sincere thanks also goes to Dario Bermudez who supported me throughout the study process, for always being willing to listen to me and help me at any time. I have been extremely lucky to have this opportunity to go to Ecuador for internship and to get acquainted with the culture and traditions of this amazing country.

I would like to express my sincerest thanks to all my family for their encouragement and that they believed in me.

Furthermore, I would like to thank all my friends in Ecuador, whom I knew before during our study at IPL and also new people that I met there. They were extremely kind and supportive.

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# Resumo

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Este relatório refere-se ao trabalho realizado durante o meu estágio na empresa IDC Proyectos Integrales que aconteceu no Equador, Quito, de janeiro de 2018 a julho de 2018.

O Equador é um país que tem um alto risco sísmico, instabilidade dos solos, a grande probabilidade de deslizamentos de terra, subsidência, erupções vulcânicas, tsunamis. Portanto, este é um lugar muito desafiador para se trabalhar em Engenharia de Estruturas. Uma das formas de mitigar o risco sísmico é projetar e construir edifícios com elementos estruturalmente resistentes a sismos e também a reabilitação e reconstrução de edifícios já existentes que foram projetados antes da implementação dos códigos e regulamentos relevantes.

O estágio teve como foco uma experiência com uma análise sísmica de diferentes tipos de estruturas, tais como estruturas metálicas, estruturas de betão armado e edifícios de alvenaria. No entanto, para um desenvolvimento mais detalhado neste relatório, foi escolhido o prédio de alvenaria confinada, uma vez que é o tipo de construção mais popular de construção na América Latina. Devido ao recente sismo que aconteceu no Equador em 16 de abril de 2016 com muitas vítimas e grande número de casas destruídas, os edifícios em alvenaria confinada poderiam ser uma boa alternativa que atenda à necessidade de construção de moradias e reconstrução devido ao baixo custo e baixa complexidade técnica de construção.

Este tipo de construção tem muitas vantagens, como isolamento acústico, térmico, disponibilidade do material, mas da mesma forma é muito vulnerável ao mencionado antes dos perigos da natureza e tem uma limitação de altura para a construção dependendo do risco sísmico.

Este trabalho apresenta a visão geral das estruturas de alvenaria e alvenaria confinada, suas características e comportamento durante os terremotos. E como um exemplo da experiência mundial e da aplicação de códigos locais, apresento neste relatório uma análise detalhada do edifício multifamiliar de alvenaria confinada de dois andares usando regulamentações equatorianas, colombianas e americanas. Além disso, o projeto é realizado com a ajuda do programa computacional ETABS 2016.

*Palavras-chave: alvenaria confinada, design sísmico, método DBF*

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# Abstract

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This report refers to work completed during my internship with the company IDC Proyectos Integrales that took place in Ecuador, Quito from January, 2018 to July 2018.

Ecuador is a country that has a high seismic risk, instability of soils, the great probability of landslides, subsidence, volcanic eruptions, tsunamis and etc. So this is a very challenging place to work for structural engineers. The only way to mitigate the seismic hazard is to design and construct buildings with elements structurally earthquake resistant and also rehabilitation and reconstruction of already existing buildings that were designed before implementing the relevant codes and regulations.

During the internship I had an experience with a seismic analysis of different types of structures such as steel frames, arch trusses, concrete frames, masonry buildings. However, for the further development I have chosen the confined masonry building as the most popular type of the construction in Latin America. Due to the recent destructive earthquake that happened in Ecuador on April 16, 2016 with a lot of victims and great number of houses destroyed, buildings in confined masonry could be a good alternative that meets the need for housing construction and reconstruction because of the low cost and low technical complexity of construction.

This type of the construction has many advantages, like acoustic, thermal insulation, availability of the material, but at the same way it is very vulnerable to mentioned before nature hazards and it has a height limitation for the construction depending of the seismic risk.

This work presents the overview of the masonry and confined masonry structures, its characteristics and behavior during the earthquakes. And as an example of the worldwide experience and the local codes application I present in this report a detailed analysis for the 2-storey confined masonry multifamily building using Ecuadorian, Columbian and American regulations. Moreover the design is performed with a help of the computational program ETABS 2016.

Keywords: confined masonry, seismic design, DBF method

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# 1 Introduction

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For six months from January 2018 till July 2018, I did an internship at IDC Proyectos Integrales, a construction company which is registered in Quito, Ecuador. This internship project is a part of my 2-years master program which I conduct at Polytechnic Institute of Leiria.

This work is aimed to study the seismic behavior of different type of structures located in the area with high level of seismic activity, but the detailed analysis will be presented in this report for confined masonry structures.

One year after the Pedernales catastrophe, construction mistakes due to no strict regulation in this area have been revealed throughout Ecuador. Since January 2015, the Ecuadorian Construction Standard "NEC 2015" has been implemented in Ecuador with the purpose of regulating the processes that allow to comply with the basic requirements of safety, design and quality in all types of buildings. This document has several chapters according to the type of work and in 2016 it was complemented with the presentation of guidelines to facilitate the application of the standard.

Those responsible for understanding and applying the concepts of standards in construction are the engineers and architects, who must be present from the design to the completion of the work. Due to social and economic reasons in Ecuador the construction is not solely in charge of architects or engineers also involved masons, teachers and professionals from other areas, who mostly ignore the principles of design and safety when planning and building a building.

In Ecuador, most structures are constructed of reinforced concrete frame structures. This constructive system requires an extensive design of each foundation, column, beam and slab to guarantee a safe and ductile behavior before seismic events. The demand and participation of professionals and highly skilled labor for this construction system is more essential while increasing the size of the structure.

In Mexico after the earthquake of 8.1 Mw of 1985 about 90% of single-family homes are built in confined masonry(Treviño, 2014). The results have been positive in terms of mastering the construction system and it has become a tradition to build houses in confined masonry. The Ministry of Urban Development and Housing (MIDUVI) together with the United Nations Program for Development (UNDP) launched the educational calendar in

Ecuador "Manuel and María present: Build Better with Confined Masonry"(COSUDE, 2017)to disseminate knowledge and techniques of this constructive method.

Another factor that exists and influences the constructive practices is the lack of planning and poor management of the work budget. The majority of builders seek to reduce costs in order to have higher profits, majority cases are affected by the quality of the materials and the quality of the construction.

The buildings in confined masonry are an alternative for the construction of houses that could be beneficial because of their low costs, simplicity in the configuration of the design, constructive ease, and for not requiring highly qualified labor. To implement a project in confined masonry it is necessary to understand the functioning of the structural system, know the construction process, and know the quality criteria for the use of the materials.

For the present work I designed a 2-storey multi-family dwelling and carried out an analysis of its behavior under seismic loading.

## 1.1 Objectives

---

The general objectives of my internship are analyzing and designing the structures in the region with high seismic activity. In order to achieve this goal first is necessary to learn and understand the local building codes; secondly obtain information about the type of structures, construction processes that are common in Ecuador; and third analyze the seismic risk which the buildings are subjected to.

More detailed attention I put to the confined masonry structures. So the specific objectives are analysis of this type of structure, materials that are used for the construction, behavior under different types of loading and modes of failure. The detailed design of the confined masonry has to be done according the Ecuadorian building codes (NEC-SE-MP, 2015) and (NEC-SE-Vivienda, 2015). Also Ecuadorian codes allow to use building codes of other countries like Colombia, Mexico, United States. So it is recommended to consider all the requirements for the confined masonry structures using experience different countries.

## 1.2 Internship report framework

---

The report consists of 13 chapters. In the first and second chapter I present the information about my internship in IDC Proyectos Integrales, the short review of the projects that I completed.

The third chapter talks about the masonry as material itself, about the types of constructions using this material, the behavior of the masonry structures and the performance during earthquakes.

In the fourth chapter I analyze the confined masonry structures and comparing it with other building technologies, seismic behavior of this type of structures, analysis of the connection between the masonry and confining elements and rules for reinforcement of the tie beams and columns.

The fifth chapter the analysis of lateral and vertical forces of the structure is presented. What is it horizontal and vertical diaphragms, deflexions in the diaphragms. The processes that are occurring in the structure during the earthquake like torsion or overturning moment.

In the sixth chapter I presented the building that I will analyze further in the report and in the eighth chapter I explain the design using computational software ETABS 2016. The seventh chapter gets information about seismic activity in Ecuador and the parameters regarding the project area.

The ninth chapter gives information about the method that is proposed by seismic codes for the design of structures, general objectives of this method and procedure.

In the tenth chapter I put a detailed design of the confined masonry structure using the results of the calculation obtained with ETABS 2016 software.

In the eleventh and twelfth chapters I do a calculation of the waffle slab and strip foundation.

In the thirteenth the conclusions and recommendations concerning the report are carried out. At the end of all chapters references are cited that served to write it.

In the Appendix A I put the results of the analysis of the structure using ETABS 2016 software and in the Appendix B - detailed analysis of the structure according

Ecuadorian building code based on the results obtained in the program. Appendix B consists from the general drawings of the design building created in Autocad.

## 2 Internship overview

---

IDC Proyectos Integrales is a company specializing in Civil Engineering, design, construction and inspection of projects.

The services that the company provides are following:

- 1) Works and services of construction, repowering and maintenance of civil and metal-mechanic infrastructure.
  - Construction of works in general
  - Maintenance and repowering of works in general
  - Structural reinforcements
  - Construction finishes (painting, ceramics, gypsum, floating floor, among others)
  - Plumbing.
- 2) Structural and architectural design
  - Elaboration of structural and architectural designs
  - Evaluation and rehabilitation of existing structures
  - Evaluation and rehabilitation of built heritage
  - Design of networks for drinking water and sewerage
  - Preparation of detail plans for works in general.
- 3) Consulting and electronic and computer services
  - Installation of video surveillance systems, motion sensors; telephone alerts, control from fixed and mobile devices
  - Design and installation of communication systems and networks
  - Graphic design and web pages.

The various projects have been executed in recent times, such as:

- Civil and electrical maintenance of several educational institutions belonging to the District Address 17D09 rural parishes from Tumbaco to Tababela
- Remodeling of Dr. Guillermo Park dental office, Plaza Kendo Building, Quito
- Civil and electrical maintenance work in dealers of Proauto, Quito
- Construction of Contact Center, Proauto Matriz, Quito
- Structural design of Edificio Torre Floreana, Quito

- Maintenance works for the cover of the Víctor Manuel Peñaherrera Educational Unit, belonging to the Archdiocese of Quito
- Foundation and drainage construction for a centralized CIP structure, stage 1 and stage 2, Nestlé - Ecuajalisco, Cayambe
- Construction of silos for storage of materials, Ecuadorian Institute of Cement and Concrete INECYC, Quito
- Construction of earth retaining walls for construction of Galpón UHT, Nestlé - Ecuajalisco, Cayambe
- Construction of the network towers owned by Balesia Towers, Guayaquil, Tarifa and Esmeraldas
- Construction of Mens Lockers in Communal Zones of the Nestlé Plant - Ecuajalisco, Cayambe
- Construction of chamber for curing of materials, Ecuadorian Institute of Cement and Concrete INECYC, Quito
- Structural design for the roof of the main courtyard of the Yaruquí Technical Educational Unit, belonging to the District Address 17D09, Yaruquí - Quito
- Construction of foundations of the MYMILL project in the Laceda Cía. Industrial Plant, belonging to the OMYA Group, Quito.

During my internship in this company I was working on a different kind of projects for residential and commercial use. I give a description below for some of them.

## **2.1 Steel frames with arch trusses for sports facilities in the schools**

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The structures considered is a truss types with tube profiles, and the area and height available on site are taken into account. The type of connections between the profiles are bolted and welded.

The seismic spectrum of the site was considered for the calculations and the sizing of the profiles and their yields were obtained through the Robot Structural Analysis and Etabs computer programs.

For the design in the program, the following steps were taken into account in order to obtain results, which are observed in the annexes of this document:

- ✓ Design Loads
- ✓ Calculating the base shear
- ✓ Verification of the results obtained in the calculation through the Robot Structural Analysis program with the efforts given by each profile according to the steel codes and the performance of the same for the load to be supported
- ✓ Amount of work

For the analysis work one model of 1-storey steel frame structure is made to know the realistic behavior of the building during earthquake.

The length of the building is 18.2m and the width is 27.3m in axis dimensions. Height of the building is 6-7.9m. Building is symmetrical about x and y-axis. Material steel - STEEL A36 is used. Analytical modeling that includes all components which influence the mass, strength and stiffness. The non-structural components that do not influence significantly the building behavior were not modeled. The effect of soil interaction is ignored in analysis. The columns are assumed to be fixed at the ground level.

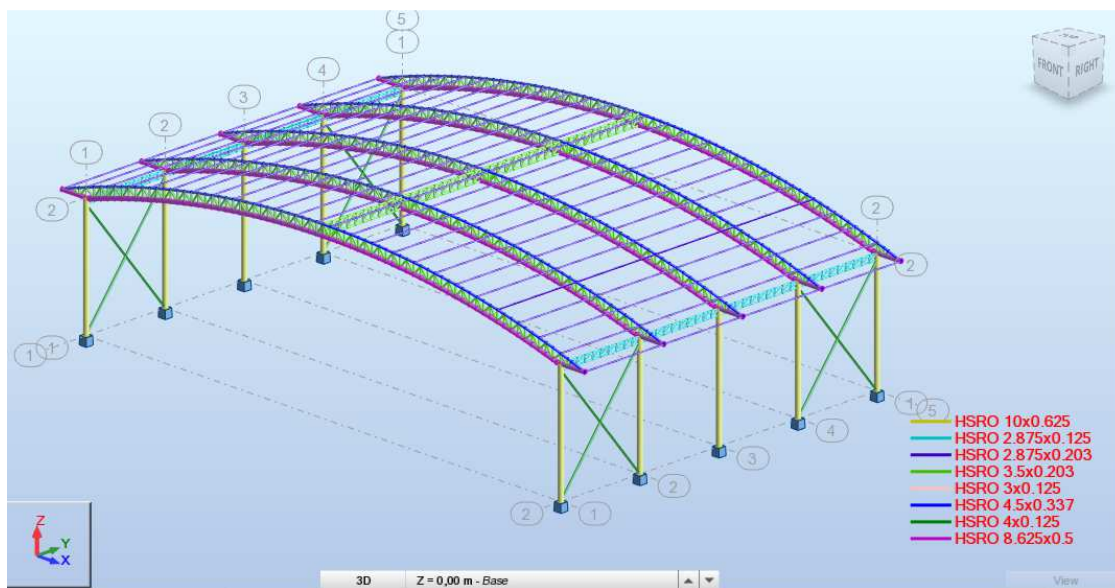


Figure 1 3D View of the structure

## 2.2 Scout project

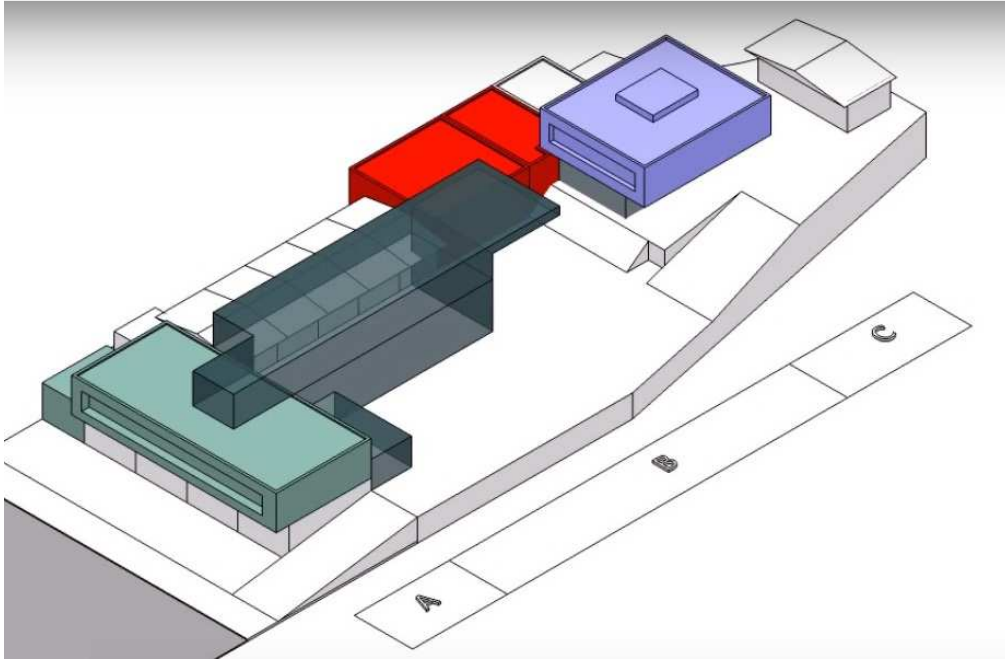
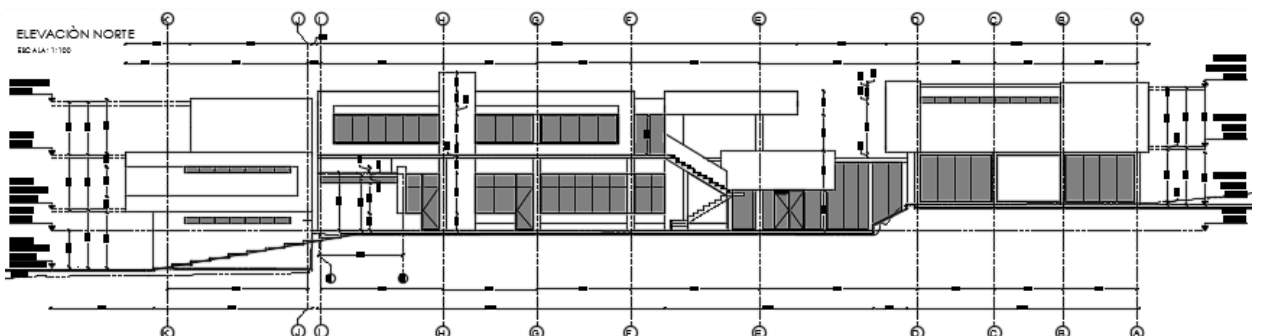


Figure 2 Conceptual model of the project

The architectural project, the basis for the required engineering studies, contemplates the intervention in existing structure and the expansion in the new part by requirements of the Scouts Association of Ecuador according to the following areas:

Table 1 Parameters of the Bloques

BLOQUE	NIVEL	EXISTENTE	AMPLIACIÓN
A	-1,97	169,13	32,65
	+1,10		198,34
B	0,00	240,16	169,52
	+4,00		155,33
C	+1,36		130,47
	+4,41		187,60
		409,29	873,91



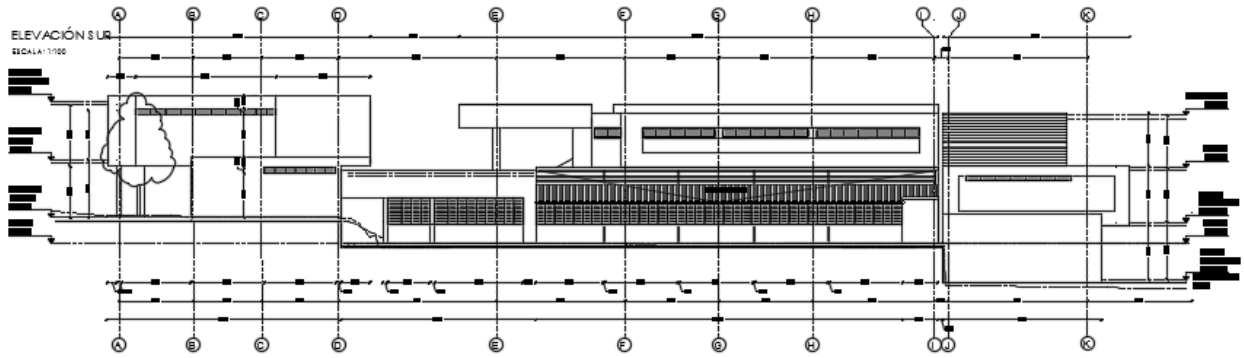


Figure 3 North and South Facades

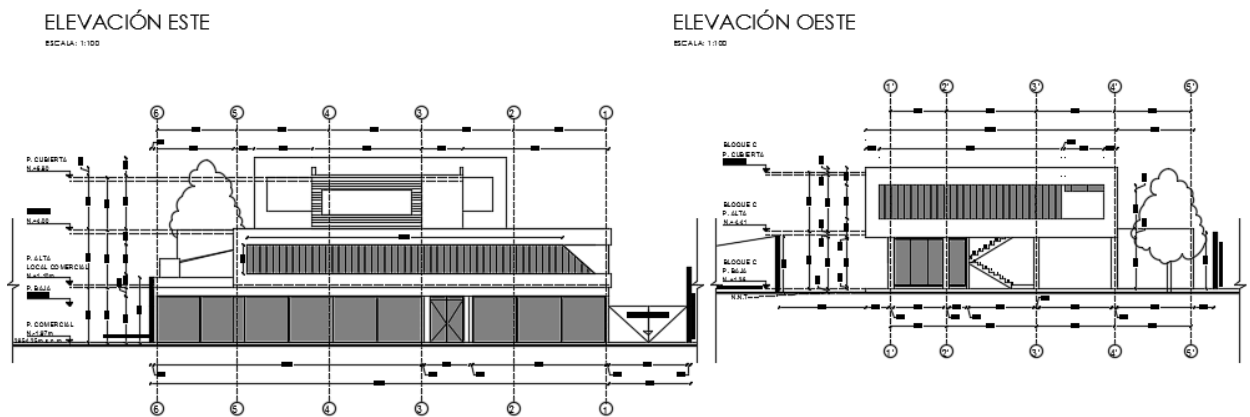


Figure 4 East and West Facades

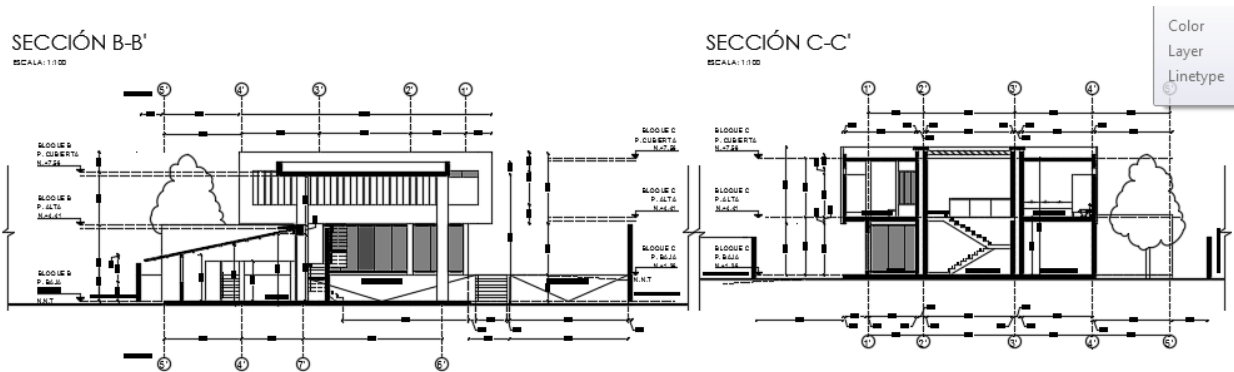


Figure 5 Sections B-B' and C-C'

Block A, opposite Av. America, includes commercial premises and offices, followed by block B, which includes the auditorium, offices, cafeteria and scout shop. Block C comprises residence, offices and storage. The quantity, dimensions and location of sanitary batteries, circulations, accesses, parking lots and other architectural components respond to the municipal requirements according to current regulations.

Main duties:

- Structural draft and technical and economic optimization of the structure

Defined the architectural characteristics of the project and the materials and construction systems to be used, the general planning of the structural systems of the building will be carried out. At the preliminary project level, seismic-resistant structural elements will be defined, columns, beams, slabs, walls, ramps, roofs, stairs, shear walls and foundations are pre-dimensioned. In order to get to the final dimensioning of the building structure, a process of optimization of its technical characteristics was followed, in order to define the structure that has the best economic conditions and the most suitable constructive characteristics, in such a way that they allow the execution of the project in the shortest possible time, at the lowest cost and in the best technical and safety conditions.

- Structural analysis

Defined the geometric characteristics of the structure, the mathematical models of structural type were elaborated, to perform the static and dynamic evaluation of the behavior of all the structural systems of the buildings, under the action of the permanent and occasional loads that will act on the building during its useful life. For its analysis, the structure of the building was considered spatially and the evaluation of its behavior was carried out through the use of computational software. All the frames were analyzed in two orthogonal directions, taking into consideration the action of the vertical, horizontal loads and the effects of torsion in the plant.

- Loads that will be used in the structural analysis

In the structural analysis of the building, two types of loads will be considered: permanent loads and occasional loads.

The permanent loads that will be considered are those produced by: the weight of the structure, the weight of the non-structural elements and the soil pressures.

The occasional loads that were considered are: the overload or live load that gravitates on the structure when it is in operation, the loads that will be generated during the construction process, the effects produced by the action of the earthquakes or by the action of the wind, any of the two load states that is most critical to the structure of the building. There were also taken into consideration, when the situation justifies it, the effects that are introduced into the structure due to temperature changes and because of the shrinkage produced during the concrete setting process. Within the occasional loads the one of greater importance and importance, by its magnitude and special characteristics of dynamic action on the structure, is the one produced by the seismic effects.

- Earthquake-resistant analysis

Due to the geotectonic characteristics and high seismicity in which the project is located, special attention was put to the analysis and design of the structure of the buildings to resist medium and high intensity earthquakes. For this purpose, the natural frequencies of the structure were determined, using dynamic analysis methods, the design spectra were defined for the seismic analysis and the responses of the structural systems to the most probable seismic actions were calculated.

- Structural Design: dimensioning and definitive design of structural elements

With the results obtained in the structural analysis processes for each one of the different permanent and occasional load states, according to the design codes used and for each of the cross sections of the structural elements of the building, the more critical combinations of the different effects: axial load, bending moment and cut, were used in the designs. With the knowledge of these maximum actions for each of the structural elements, I performed dimensioning and final design. This design includes: foundations, columns, diaphragms, walls, beams, slabs, ramps, deck, stairs, expansion joints and other minor structural parts. The designs were made in those materials that constitute the best technical, economic and constructive solution for the project.

- Drawing of structural drawings

The drawing of the structural plans was done on a computer using Autodesk AutoCAD Software. The structural plans include all the construction details, the technical specifications for the construction of the structure, slab casting areas, iron sheets for each of the structural elements, number of lightening, metal profiles, details of joints.

- Structural work volumes

For each of the structural elements of the buildings (foundations, walls, columns, slabs, beams, roof) were calculated the quantities of materials that are required for its construction: iron, concrete, lightening, metal profiles, etc.

The summaries with the quantities of materials were be included in each of the structural plans and were also be presented in an independent document as part of the final report.

- Standards and Codes were used

The analysis and structural design were carried out in accordance with the current regulations of the "Ecuadorian Construction Standard" (NEC). All the analysis and design were carried out within the best and most updated engineering practices. The following design codes and standards will also be used as reference: for designs in reinforced concrete and prestressed concrete those established by the "American Concrete Institute" (ACI 318-99); and for the "Pre-stress Concrete Institute", PCI, for designs in steel those established by the "American Institute of Steel Construction" (AISC-99), the American Iron and Steel Institute (AISI) and the American Welding Society (AWS). ); for wood design those established in the "Wood Construction Primer" (PAD- REFORT / JUNAC); for general building design those established in the "Uniform Building Code" (UBC) and for the definition of the quality of the materials used in the construction established by the "American Society of Testing and Materials" (ASTM).

## 2.3 Two-stories residential house

The project includes a two-story single-family building with an area of 115.17 m<sup>2</sup> for the 1st floor and 110.14 m<sup>2</sup> for the second floor in the city of Quito with a total construction area of 225.31 m<sup>2</sup>. The structural type of a building - reinforced concrete frame with columns of size 30x30 cm, beams - 20x30 cm, slab - 20 cm.



*Figure 6 Facades of the building*

### 3 Masonry construction

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In the field of Civil Engineering, the system of load-bearing walls has been widely used for many years in Ecuador. This type of construction has great qualities of acoustic and thermal insulation and has addressed the problems of affordable housing.

However, if these types of buildings do not have a proper reinforcement, they are vulnerable to natural agents such as wind that is very little or not taken into account in the design of these buildings. The wind factor was not taken into account for the structural calculation because the regulations did not ask for it, but in the NEC 2011(NEC-SE-CG, 2011) regulation it is essential to take this factor into account. A construction material, is any product processed or manufactured to be incorporated permanently in any work, whether building or civil engineering. In general, construction materials must meet these requirements:

- ✓ Mechanical resistances according to the use they will receive.
- ✓ Chemical stability (resistance to aggressive agents)
- ✓ Physical stability (dimensional).
- ✓ Security for its management and use.
- ✓ Protection of the hygiene and health of workers and users.
- ✓ Do not conspire against the environment.
- ✓ Thermal and acoustic insulation (collaborate in saving energy).
- ✓ Stability and protection in case of fire (resistance to fire).
- ✓ Convenience of use, aesthetics and economy (NEC-SE-MP, 2015)

This type of traditional construction without reinforcement has a very low response to earthquakes causing several damages throughout its structure and in many cases causing significant structural problems since the reinforcement of the walls cannot be placed inside the bricks. In recent years, other types of problems have been observed more frequently, such as: cracks in the masonry caused not only by telluric movements but also by differential settlements, considerable deflections on the structure, and many other problems that arise during and after the construction process, which brings with it additional costs for its repair and loss of the functionality of the structure.

## 3.1 Types and construction processes

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### NON-REINFORCED MASONRY

It is the construction based on pieces of masonry joined by means of mortar that does not meet the minimum amounts of reinforcement established for the partially reinforced masonry. This structural system is classified, for effects of earthquake-resistant design, as one of the systems with minimum energy dissipation capacity in the inelastic range.

### SIMPLE MASONRY

It is the type of structural masonry without reinforcement formed by pieces of masonry joined by means of mortar and that do not meet the minimum amounts of reinforcement established for partially reinforced masonry. The dominant efforts are compression which must counteract the stresses produced by the horizontal forces (NEC-SE-MP, 2015). However, this type of masonry lacks external and internal reinforcement and it is only the masonry that absorbs the requested efforts.

### CONFINED MASONRY

Widely used in our environment, this type of confined masonry is reinforced throughout its perimeter by reinforced concrete elements, ie beams and columns, which will absorb the tractions produced by the lateral loads in the wall planes (NEC-SE-MP, 2015). In this type of masonry, the wall is first raised and then the confining elements, columns, chains and beams are all made of reinforced concrete.

### BEARING MASONRY

It is the type of masonry built with clay bricks or blocks of concrete and mortar paste, with a reinforcement of electro-welded mesh on the faces of the wall, which is not composed of columns or beams in its perimeter, so the masonry forms the entire superstructure and is responsible for the transmission of the loads to the substructure (NEC-SE-MP, 2015).

### REINFORCED MASONRY

It is brick masonry or reinforced concrete block by using steel rods distributed horizontally, vertically or both, embedded in mortar or concrete, forming a monolithic system, so that the system works together to withstand the stresses. The columns and beams will be reinforced concrete. This reinforcement will mainly resist tensile stresses

and occasionally when the masonry is not able to withstand compression and cutting forces this reinforcement will be able to absorb these stresses (NEC-SE-MP, 2015).

### 3.2 Elements that make up the masonry

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#### BRICK

The brick is a piece in the shape of rectangular parallelepiped, clay or clay earth properly fired, in order to give it hardness. It can also be called brick as that unit whose size and weight allows it to be manipulated with one hand, the brick that will be used will be of the handcrafted type made of clay. In the case of the concrete block, a module unit of 20 cm in thickness x 20 cm in height and 40 cm in length is usually used (NEC-SE-Vivienda, 2015).

#### *Compressive strength*

The compressive strength of the bricks,  $f_m$ , shows a fairly wide range of variation, depending on the material and the type of unit. Table below shows the variation intervals of the compressive strength of various materials. All these values have been obtained by tests in isolated units, however, these values can suffer a drastic decrease when they are tested as part of a masonry panel. This effect is illustrated in Figure 7.

*Table 2 Compression resistance of different masonry units*

Material	Compression resistance
Stone	400 ... 1000
Concrete solid	150 ... 250
Clay	50 ... 200
Light concrete	40 ... 140
Adobe	10 ... 15

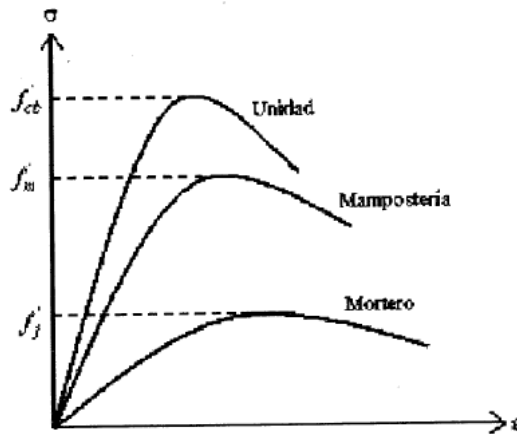


Figure 7 Stress-strain relationships for mortar, separated units and masonry panels (Pauley et al., 1992)

There is not a very broad knowledge about the stress-strain relationship of the masonry units under compression. The results of experiments in clay bricks indicate that this relationship is almost linear until failure, at which point a rapid decrease in resistance follows.

The effect of perforations on compressive strength has been studied by several researchers. From experimental tests on perforated masonry units, it has been proved that this class of units, usually exhibit a very fragile behavior and the failure happens unexpectedly. Therefore, the use of perforated masonry units is not recommended for structures subjected to seismic actions. However, the Ecuadorian standard gives the following minimum values of compressive strength for the masonry units:

Table 3 Minimum compression resistance by NEC (NEC-SE-Vivienda, 2015)

Unit type	Compression resistance $f_{cu}$ (MPa)
Solid brick	2
Clay brick with horizontal perforation	3
Concrete or clay block with vertical perforation	3

#### Modulus of elasticity and Poisson's ratio

The deformation stress ratio of the masonry units depends significantly on the constituent material. Due to the absence of experimental information, it can be assumed that the baked clay bricks behave almost like a linear elastic material, while the concrete units exhibit a non-linear behavior similar to that observed in the concrete plates.

The modulus of elasticity of the masonry units,  $E_b$ , presents a wide range of variation and, basically, depends on the type of material and the compressive strength. There is no standard method to evaluate the modulus of elasticity. Usually, this is taken as the secant modulus of elasticity, from zero stress level to one third of the strength of the material (Gutiérrez, 2004).

#### *Moisture content and absorption*

The moisture content and absorption are the most important properties of the units and have a considerable effect about the characteristics of the masonry.

The moisture content is defined as the mass of water per unit volume, which can be expressed, in absolute terms or in relative terms to the density of the unit when it is dry. The typical range of this parameter is 50-60 kg/m<sup>3</sup> and 2-3% respectively (Gutiérrez, 2004).

#### **MORTAR**

It is known as a mortar to the grayish solid substance that is obtained from the mixture of a binder material, such as Portland cement. It is a filling material, made up of fine aggregates and water, which then hardens on contact with air.

Given their properties such as plasticity, consistency and being able to retain the minimum water for the hydration of the cement and, in addition, guarantee their adherence with the masonry units to develop their cementing action, they depend mainly on the quality of the materials used in their processing, and of course the dosage used, mortars have a wide variety of applications, such as the sticking of pieces of masonry or the coating of the same, better known in our environment as plaster.

The mortar paste should have good plasticity, consistency and be able to retain the minimum water for the hydration of the cement and also ensure its adherence with the masonry units to develop its action (NEC-SE-MP, 2015). The mortar used for the bonding of the masonry units has a volumetric dosage of 1: 3. The thickness of this mortar is 1.5cm, it is necessary to verify its uniform placement both horizontally and vertically on the masonry units.

The mortar used for the plaster of both masonries also has a volumetric dosage of 1: 3 and a thickness of 3cm measured from the masonry plane.

As well as the integrity of the masonry units, it depends on the manufacturer, in the case of the mortar, both the manufacturing process and placement (tasks performed by the mason) are finally those that define its quality.

## CONCRETE

It is known as concrete to the homogeneous mixture produced between portland cement, fine aggregate, coarse aggregate and water. In our environment, concrete is the main material within the structural elements and like the mortar paste it hardens on contact with air.

In the case of reinforced or confined masonry the concrete of the confinement elements shall have a compressive strength greater than or equal to  $210 \text{ kg / cm}^2$  or  $21 \text{ MPa}$  measured at 28 days and shall comply with the requirements established in the code (NEC-SE-MP, 2015), which are the existing standards and requirements for this type of structure. In my case I will use a concrete of  $210 \text{ kg / cm}^2$  for the beams, columns and the slab.

## 3.3 Origin of the problems in the masonry

### 3.3.1 Seismic

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Earthquake is called vibration or wave movement of the soil that occurs mainly by the release of energy, which accumulate in the earth due to strong stresses or pressures that occur in the interior. It is said that a building is a strong earthquake when it is designed with a suitable structural configuration, with components of appropriate dimensions and materials with a sufficient proportion and resistance to withstand the action of the forces caused by earthquakes (NEC-SE-DS, 2015).

### 3.3.2 Horizontal seismic forces

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The system of horizontal forces equivalent to the seismic action that is applied according to the direction of analysis considered, is established by determining, first the value of the resulting horizontal seismic force or shear stress, from which the component forces of the system which in turn are assumed concentrated at the level of the mezzanines and roof of the construction, in which the gravitational loads have been concentrated.

The resultant of the horizontal forces equivalent to the seismic action or shear stress according to the direction of analysis considered, will be determined by the following expression:

$$V_0 = C \cdot W \quad (1)$$

Where:

$V_0$  - it is the resultant of the equivalent horizontal forces or cutting effort;

$C$  - is the seismic coefficient of design, determined according to the seismic zone in analysis;

$W$  - the total gravitational load on the base level of the construction.

The seismic coefficient of design  $C$  will be determined as established in the Ecuadorian code and depending on the seismic zone.

### **3.3.3 Interaction between the seismic forces and the masonry**

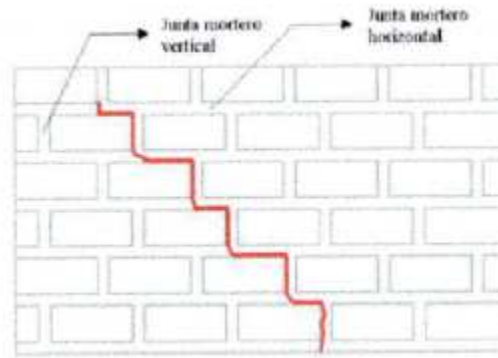
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It is important to take into account that during the occurrence of an earthquake, the masonry due to its great rigidity are the first elements that will suffer the consequence of said earthquake. The most important property of bearing masonry in seismic zones is its response to dynamic actions once the inelastic range is abandoned. The behavior of the masonry due to multiple stresses to which it is exposed, is variable.

It must be taken into account that the masonry is not only subject to lateral seismic loads, but also to axial or vertical loads due to the weight of the live load and the dead load, in addition there may be possible wind, water or earth loads. Because of the type of stress to which this subjected the masonry, it presents different types of behavior and therefore different types of failure, among the most important are (NEC-SE-MP, 2015):

✓ Shear failure

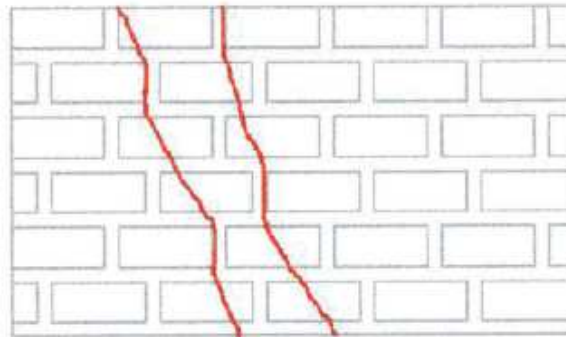
There are two types: failure by shear, when the crack is diagonal to 45° and runs only through the mortar joints, and fails by diagonal tension, when the crack is almost straight, breaking the pieces. The majority of these failures are due to the fact that the design in the structure is not taken care of by performing a proper test of the materials.



*Figure 8 Failure due to shear*  
(Gutiérrez, 2004)

✓ Diagonal tension failure

It is produced directly on the masonry units for values of moderate normal compression stresses. The shear strength of the mortar joints increases due to the effect of normal compression forces. Therefore, cracks occur in the units, as a result of tensile stresses induced by the state of compression and shear stresses. Figure 9 shows the distribution of the cracks that follow the direction of the vertical joints and pass through the units with an inclination that depends on the orientation of the main stresses in the unit.

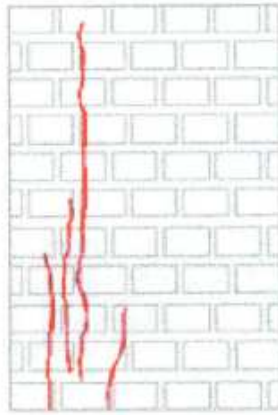


*Figure 9 Cracking of masonry units produced by diagonal tension failure*  
(Gutiérrez, 2004)

✓ Failure due to axial load

This type of failure depends on the interaction between the masonry units and the glue mortar, since the units restrict the transverse deformations of the mortar, inducing compressive forces in the transverse plane and, in turn, generating tensile stresses on the units. masonry decreasing its resistance.

It is unusual for this type of failure to occur, and may be caused by poor quality parts or because they have lost load capacity due to weathering; and they are identified by vertical cracks in the middle or at the ends of the masonry.



*Figure 10 Failure due to excessive vertical cracking caused by compression forces (Gutiérrez, 2004)*

✓ Failure modes due to direct tension

Different types of failure can occur according to: 1) the direction of the tensile load and 2) the relative magnitude of the bond strength and the tensile strength of the masonry units.

When the tensile stresses are parallel to the horizontal mortar joints, two types of cracking can occur: a) completely vertical cracks that pass through the units (see Figure 11 a), where the resistance is controlled by the tensile strength of the masonry units and, b) cracks that do not affect the units and only occur along the mortar joints (see Figure 11 b), where the most determining factors are the shear strength and the overlap length (part of one brick covered by another,  $L_0$ ).

On the other hand, when the tensile forces act perpendicular to the horizontal mortar joints, failure modes usually occur by separation of the mortar-unit interfaces (see Figure 11 c). However, it can also occur tensile failure of the units, as shown in Figure 11 d.

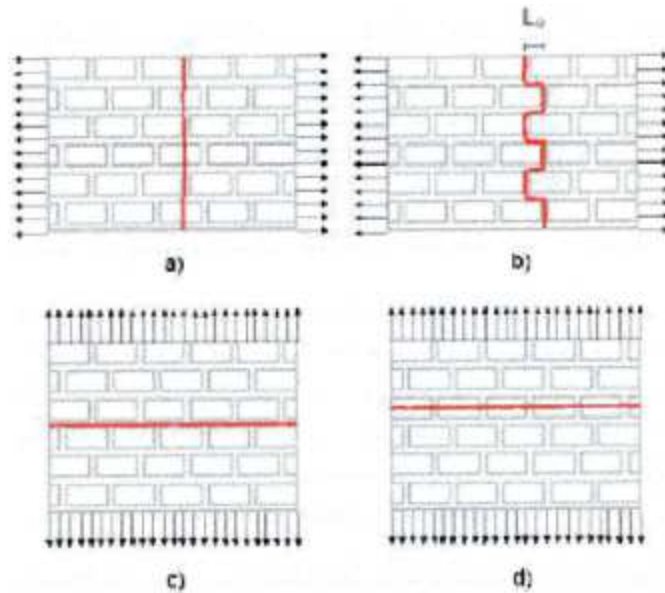


Figure 11 Failure modes of masonry subject to direct tension  
(Gutiérrez, 2004)

- ✓ Failure due to flexion.

This type of failure is mainly due to the fact that the masonry lacks reinforcing steel since it absorbs stresses. It is identified by horizontal cracks in the ends of the walls that are very easy to observe.

### 3.3.4 Faults in the calculation of the masonry structures

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Below are some cases in which calculations fail in the masonry.

- ✓ Soft Floor

This case occurs when the masonry is not continuous along the floors, generating greater rigidity in the upper floors and causing in the structure the effect of an inverted pendulum. This can be understood when in several floors, the ground floor lacks a sufficient number of walls, while in the upper floors the walls abound, in this case the problem of the flexible low floor and rigid upper floors has been generated, which in the presence of an earthquake efforts are concentrated in this soft floor.

- ✓ Insufficiency of reinforcement

This case is presented by pretending that the masonry alone absorbs the stresses of compression and tension. It is important that the reinforcement of masonry comply with the minimum reinforcement regulations. (NEC-SE-MP, 2015).

✓ Short walls

This case occurs when the masonry is incomplete in height, generating the problem of the short walls, which likewise presents planes of failure in elevation of the structure, because it stiffens these short elements remarkably and causes the collapse. When it is the case in which a panel does not contribute positively to the structure or building, it should be thought of eliminating it, replacing it with some other solution or simply isolating it from the rest of the structure by means of a construction board.

✓ Torsion

This phenomenon occurs when the floor of the building does not have uniformity or symmetry, in length and height in its configuration, which is known as irregularity in the floor, which generates an eccentricity between the center of mass and the center of rigidities, causing a moment due to the acting forces of the earthquake. Torque also occurs when there is a large concentration of masses, causing large rigidities in certain sectors of the structure. If the design and calculation of a structure does not comply with the requirements of geometry, this structure is considered vulnerable to the occurrence of an earthquake (NEC-SE-DS, 2015).

### **3.3.5 Differential settlements**

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This problem is mainly due to the poor quality of the soil on which the structure has been cemented, that is to say to have very soft soils, causing settlements, subsidence, cracks, and damage to non-structural elements and the weakening of the structure. The subsidence occurs due to the humidity of the land; on the other hand, the settlements are due to the weight of the structure.

### **3.3.6 Bad quality of materials**

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All the materials used in the construction of the masonry must be submitted to quality and resistance tests. A structure will be less vulnerable when the masonry units are of good quality and uniform, that is to say they do not present important cracks, they are not damaged or broken pieces. The masonry should be perfectly locked in all directions,

maintaining uniformity in all rows. The glue joints should be uniform and continuous, covering the masonry unit completely both horizontally and vertically, guaranteeing adherence in all units.

### **3.3.7 Behavior of load bearing masonry buildings**

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Because the structural response of masonry buildings varies according to the region of each country, these elements are susceptible to damage by earthquakes, so you can determine the typical efforts of: compression, flexo-compression, and shear. In the case of my project we are in zone V according to the seismic design map provided by the NEC-2011 ((NEC-SE-DS, 2015) that is of high seismic hazard and the value of its factor  $Z = 0.40$ , later we will use it for the calculations with the ETABS program.

### **3.3.8 Compression forces**

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In general, it can be said that the resistance to compression forces is acceptable for this type of structure, so, the resistance to axial loads is given mainly by the elements that make up the masonry, that is, the units of bricks, the mortar of glues and plaster. It must be taken into account that the two materials have different behaviors in terms of deformability, the brick is less deformable than the mortar, restricting the transverse deformations of the mortar despite being subjected to the same stresses.

### **3.3.9 Shear**

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The main fault element produced by these efforts is the appearance of cracks which cross the masonry, which present great weakness when subjected to earthquake or wind loads. This type of faults is manifested with a crack at 45 degrees, the same as can be seen in both directions, being able to observe an "X" known as the "X" of the earthquake.

## 4 Confined masonry

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Confined masonry construction consists of masonry walls and horizontal and vertical reinforced concrete (RC) confining elements built on all four sides of a masonry wall panel. Vertical elements, called tie-columns, resemble columns in RC frame construction except that they tend to be of far smaller cross-sectional dimensions. Most importantly, these RC members are built after the masonry wall has been completed. Horizontal elements, called tie-beams, resemble beams in RC frame construction but they are not intended to function as conventional beams since confined masonry walls are load-bearing. Alternative terms, horizontal ties and vertical ties, are sometimes used instead of tie-beams and tie-columns. The key features of structural components of a confined masonry building are discussed below:

- Masonry walls transmit the gravity load from the slab(s) above down to the foundation (along with the RC tie-columns). This document addresses confined masonry construction consisting of masonry walls made of solid clay bricks, hollow clay tiles, or concrete blocks. The walls act as bracing panels, which resist horizontal earthquake forces acting in-plane. The walls must be confined by RC tie-beams and tie-columns and should not be penetrated by significant openings to ensure satisfactory earthquake performance.

- Confining elements (RC tie-columns and RC tie-beams) are effective in improving stability and integrity of masonry walls for in-plane and out-of-plane earthquake effects. These elements prevent brittle seismic response of masonry walls and protect them from complete disintegration even in major earthquakes. Confining elements, particularly tie-columns, contribute to the overall building stability for gravity loads.

- Floor and roof slabs transmit both gravity and lateral loads to the walls. In an earthquake, floor and roof slabs behave like horizontal beams and are called diaphragms. The roof slabs are typically made of reinforced concrete, but light-weight roofs made of timber or light gage steel are also used.

- Plinth band transmits the load from the walls down to the foundation. It also protects the ground floor walls from excessive settlement in soft soil conditions and the moisture penetration into the building.

- Foundation transmits the loads from the structure to the ground (Meli, 2011).

## 4.1 Confined Masonry and Similar Building Technologies

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Confined masonry building technology is somewhat similar to both reinforced masonry and reinforced concrete frame construction with infill walls. It should be noted, however, that differences between these building technologies are significant in terms of construction sequence, complexity, and seismic performance.

For example, the main difference in the seismic behavior of the confined masonry (CM) and reinforced concrete frame buildings is that tie columns in confined masonry buildings cannot provide effective frame action due to its slenderness and not very high stiffness comparing to the masonry wall, because the size of the columns is relatively small. Also masonry wall in CM structure usually is integrated to the wall by the toothed surface of the wall and reinforcing bars. Moreover, in CM structures the masonry wall also supports gravity loads as well as tie columns.

When subjected to lateral seismic loads, walls in confined masonry buildings act as shear walls, similar to unreinforced or reinforced masonry walls or RC shear walls. On the other hand, infill wall panels in RC frame buildings do not act as shear walls - they act as diagonal struts.

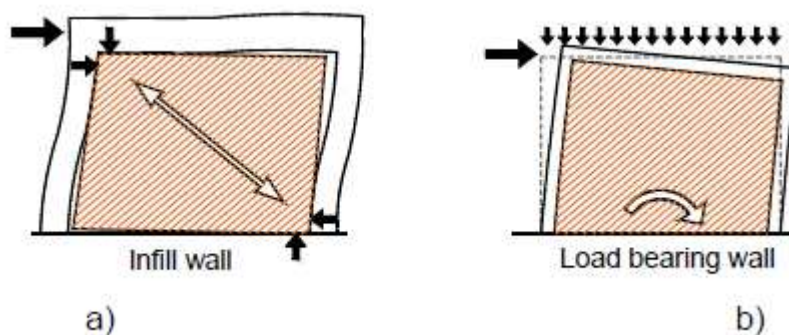


Figure 12 A comparison of RC frames with masonry infills (a), and confined masonry construction (b) (Meli, 2011)

## 4.2 Seismic behavior of confined masonry structure

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Seismic behavior of a confined masonry wall panel can be explained by composite (monolithic) action of a masonry wall and adjacent RC confining elements. This composite

action exists due to the tothing between the walls and the tie-columns - that is one of the key features of confined masonry construction. In the absence of tothing, composite action can be achieved by means of horizontal reinforcement (dowels).

A confined masonry wall panel can be modeled using the "strut and tie" model, where a vertical crack (separation) develops between the wall and the adjoining tie-columns. At the certain load level, the wall will start to act like a diagonal strut, while the adjacent columns act in tension and/or compression, depending on the direction of lateral earthquake forces.

Shear capacity of a confined masonry wall panel can be determined as the sum of contributions of the masonry wall and the adjacent RC tie-columns. The shear capacity of tie-columns can be reached only after the masonry has been severely cracked and its shear capacity has significantly decreased. As a result, it is recommended to consider only a partial contribution of tie-columns to the shear capacity of a confined masonry panel. A conservative estimate can be made by assuming that the tie-columns are integrated with the masonry wall, thus a cross-sectional area of the confined masonry wall can be calculated by taking into account the total panel length. This approach is the basis for deriving the minimum required wall density (Meli, 2011).

It can be seen from the diagram in Figure 13 that the stiffness and strength of a confined masonry panel drop following the onset of diagonal cracking in the wall (point 1). However, the load-resisting capacity of the panel is maintained until the critical regions of the confining elements experience significant cracking (point 2). This shows that a significant lateral deformation and ductility can be attained before the failure of a properly designed and constructed confined masonry panel (point 3).

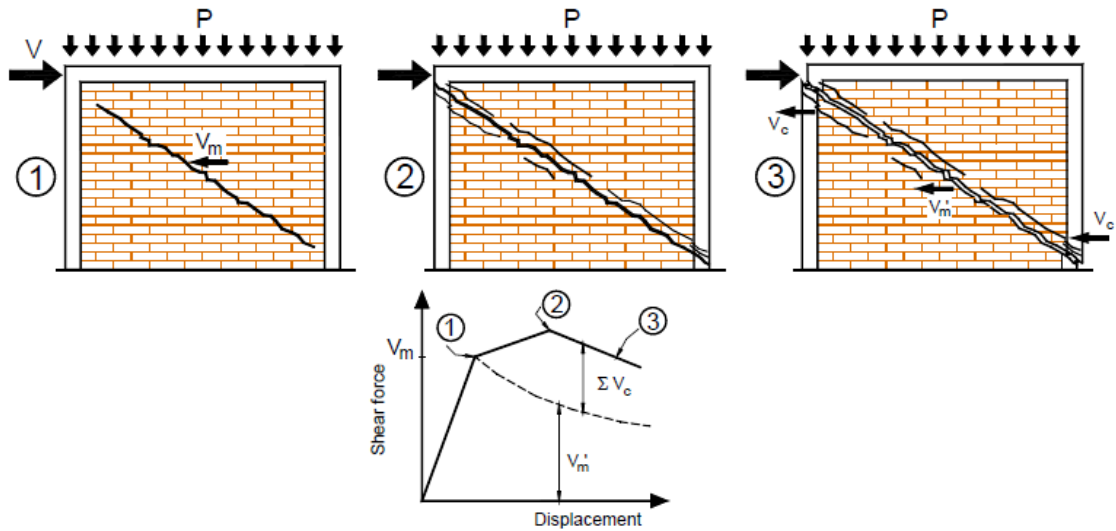


Figure 13 Mechanism of shear resistance for a confined masonry wall panel: 1) diagonal cracking in the masonry wall; 2) diagonal cracks have propagated from the wall into the tie-columns, and 3) shear failure of the RC tie-columns and the confined masonry wall panel (Meli, 2011)

Critical regions in a confined masonry structure are end zones of tie-columns (top and bottom region at each floor level), as shown in Figure 14.

In most cases, confined masonry panels demonstrate a shear-dominant seismic response. Longitudinal reinforcement in the RC tie-columns provides an adequate flexural resistance, thus the flexural failure mechanism does not govern.

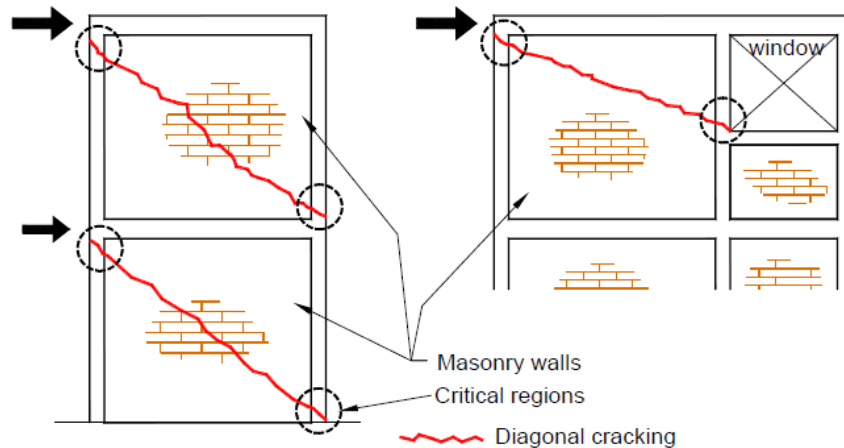


Figure 14 Critical regions in a confined masonry building: a general diagram showing critical regions in the RC tie-columns (Meli, 2011)

Confined masonry panels are subjected to the effects of axial gravity load (due to self-weight and tributary floor/roof loads). Figure 15a illustrates a confined masonry panel which resists the combined effect of axial load  $P$  and bending moment  $M$ . The capacity of the composite confined masonry panel section under the combined effect of axial load and

bending moment can be determined by treating the confined masonry panel similar to a RC shear wall acting in unison with the adjacent columns. The strain diagram shows that a portion of the panel is in tension, while the remaining portion is in compression (see Figure 15b). It is assumed that the masonry and concrete are not able to resist tension, hence tensile stresses are resisted by the longitudinal reinforcement in tie-columns. The compression stresses are resisted by concrete, masonry, and longitudinal reinforcement in tie-columns (see Figure 15c). The flexural capacity of the panel section is determined from the sum of moments created by various internal forces around point O (centroid of the section).

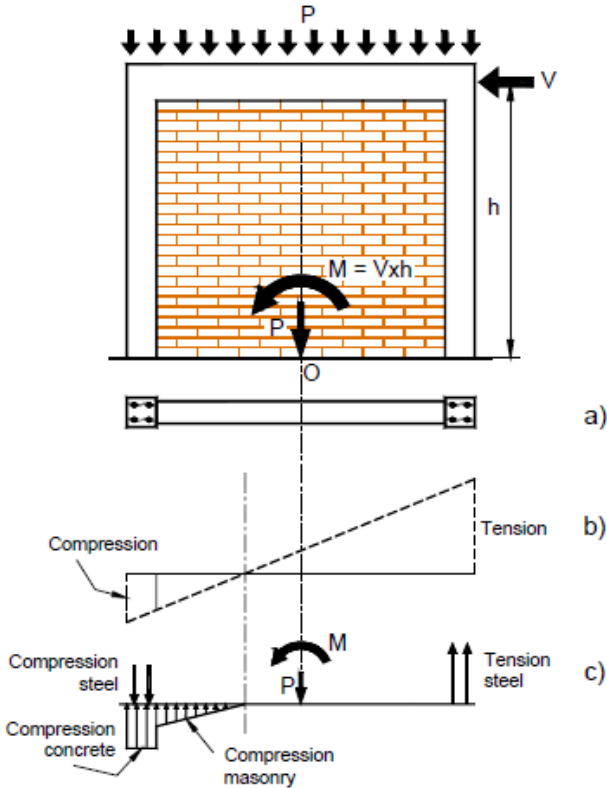


Figure 15 Confined masonry wall panel subjected to the combined axial load and bending: a) panel elevation and a typical cross-section; b) strain distribution, and c) internal force distribution. (Meli, 2011)

### 4.2.1 Typical Collapse Modes of Confined Masonry Buildings

a) Tearing Failure of Floor Diaphragms

Under earthquake load, cyclic tensile forces are generated in floor diaphragms. When masonry infill walls are added in a frame building, the structure resists the lateral

earthquake load mainly by the strut and tie action. Very high tensile forces can be induced in the diaphragms, in particular at the first floor of the building. Sufficient reinforcements have to be provided to resist the forces. Horizontal ties should not yield earlier than the occurrence of the ductile beam-sway mechanism for the building.

The field investigation revealed that tearing failures of floor diaphragms are not uncommon (Su *et al.*, 2011). Most of the structures are collapsed because of the lack of the tie-beams.

#### b) Tensile Failure of the Columns

Under the seismic actions, reversed cyclic axial forces are generated in tie columns. Such actions could cause the yielding of vertical ties and debonding between tie-beams and infill masonry walls. More importantly, the loss of confinement for the infill masonry walls could significantly weaken the lateral stability of the load-bearing walls. The extent and importance of such an effect could be investigated through the following simple analysis.

The lack of redundancy of simply supported precast slabs and the insufficient strength of the tie-beam and column systems are believed to be the two major causes of the widespread collapse of confined masonry residential buildings (Su *et al.*, 2011).

#### c) Sway Mode Failure Strength

Previous earthquakes and extensive studies have revealed that masonry buildings, even those designed with ductility considerations and with uniform storey stiffness, create a soft storey at the bottom of the building if the ground motion is strong enough. When the strength is inadequate, deformations of the building will concentrate in the weakest storey and will increase rapidly. Although it is difficult to accurately determine when a structure collapses, a masonry structure with a soft storey is dangerous enough in a normal design sense. Hence, the non-collapse design of low-rise masonry buildings should make sure that the structure has sufficient lateral strength.

To ensure the whole building resists the seismic induced loading, good structural integrity is vitally important. When premature failures such as those mentioned in Sections a) and b) have been effectively controlled, the entire building can undergo swaying deformations. The ultimate lateral strength associated with swaying is defined as the inherent lateral strength. The inherent strength considers the strengths from both non-structural and structural components, and can be estimated at peak loading (or yielding) status.

The inherent strength might be several times higher than the ultimate design lateral strength due to the following factors. First, the design of structural members for low-rise buildings located in low-seismicity areas is often controlled by gravity rather than lateral wind or earthquake loads. A very high safety margin is reserved for resisting additional lateral loads. Second, bare frame models are usually employed in the design. The stiffening and strengthening effects from non-structural components, such as infill walls and partitions, are conservatively ignored in earthquake design calculations. Thirdly, the actual strengths of construction materials are much higher than the design characteristic strengths (Su *et al.*, 2011).

There are a number of possible ways to prevent the premature failure of masonry walls: for example, one can reduce the height-to-depth ratio of the buildings, add more tie-columns and tie-reinforcement, and connect the infill walls to the tie-beams using tie-bars. The above provisions are very effective for improving the integrity of masonry buildings under seismic loads.

Finally, if the premature failures could be controlled for a multi-storey masonry building, a more deformable sway mode failure would probably happen. To further increase the lateral strength of the sway mode, one could increase the yield inter-storey drift ratio of the building by using higher strength construction materials, increase the initial stiffness (or shorten the initial period) of the buildings by increasing the size of structural members, and avoid having structural irregularities. Based on the above discussions, providing lateral strength should be a viable solution for the seismic resistant design of masonry buildings (Su *et al.*, 2011).

### **4.3 Wall-frame connection analysis**

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According to the experimental study on wall-frame connection of confined Masonry walls (Wijaya *et al.*, 2011), where 4 full scale (3x3m) confined masonry wall specimens were subjected to cyclic in-plane lateral load. A specimen with no anchorage between the wall and the reinforced concrete frame was chosen as a benchmark model. The other specimens were varied in the details of wall-frame connection, i.e. zigzag (toothing) connection, short anchor between column and wall, and continuous anchorage from column to column.

The parameters evaluated were crack patterns and failure mechanism of the wall panel, loading capacity, and energy dissipation. The study revealed that zigzag connection and short anchor did not improve the performance of the confined masonry wall; instead they were more likely to reduce the performance of the wall. Cracks and failures of the two specimens were initiated by vertical crack on the face of the wall-frame connection, which then reduced the confinement of the wall. Therefore, the final failure mode followed sliding shear patterns on the bed joint of brick-mortar, which produced more brittle failure.

Conversely, continuous anchorage strengthened the confinement, thus the diagonal crack patterns were observed on the wall and the strut and tie mechanism between the wall and the confining column was developed. Therefore, this specimen shows more ductile behavior as well as higher lateral load capacity. In conclusion, the study shows that installing proper wall-frame connection strategies is crucial in improving the structural performance.

*Table 4 Specimens details  
(Wijaya et al., 2011)*

No	Model	Wall-Frame Connection
1	A (Common Practice)	-
2	B (Short Anchorage)	Brick-column anchor $\phi 8$ @ 6 brick layer, anchor length 32cm
3	C (Zigzag Connection)	-
4	D (Continuous Anchorage)	2 continuous column to column anchor $\phi 8$ , and @6 brick layers $\phi 8$ - 32cm

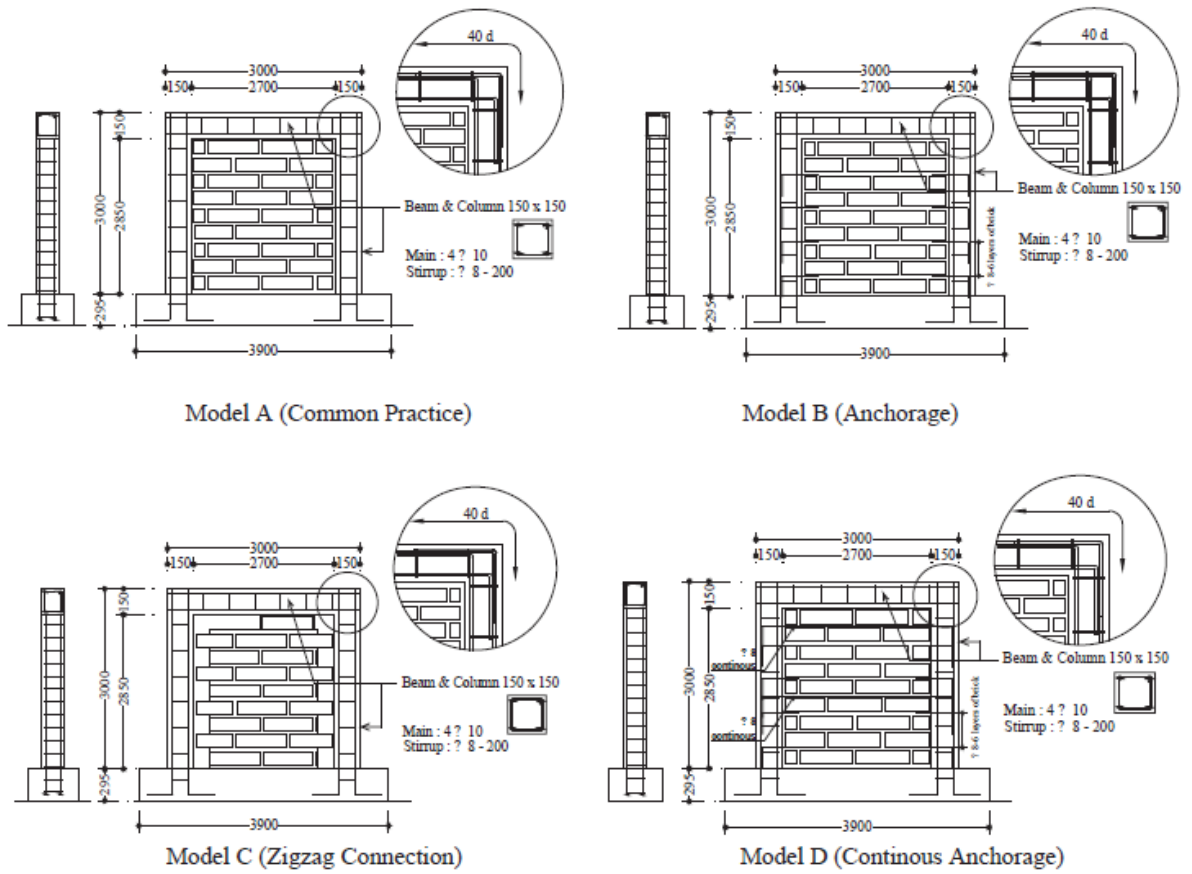


Figure 16 Test specimens  
(Wijaya et al., 2011)

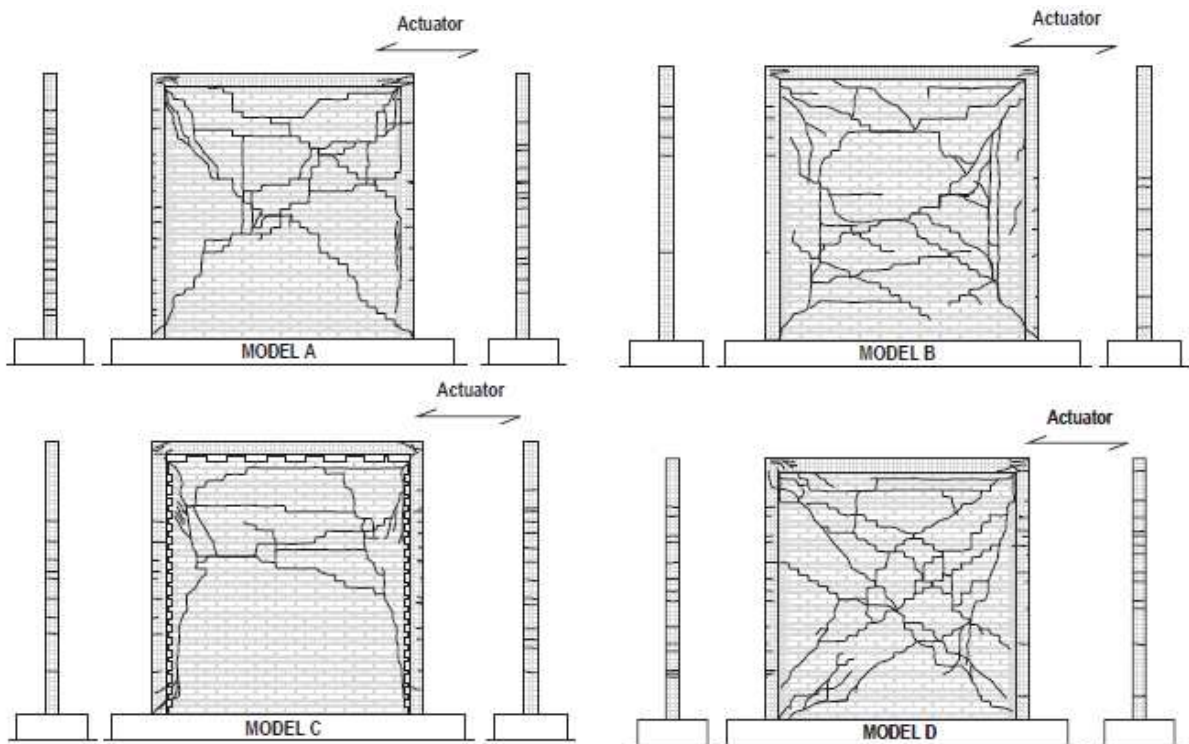


Figure 17 Crack patterns  
(Wijaya et al., 2011)

In general, initial cracks on the wall panel appeared at 2 mm lateral displacement (drift 0.067%). The crack pattern showed that Model C (zigzag connection) developed mostly sliding shear crack at the upper half of the wall. Other models showed similar confining effect on the wall and were able to develop diagonal crack mechanism and formed compression strut.

Model D (continuous anchorage) which has reinforcement on wall and columns succeeded in preventing large cracks on the wall and produced strong confinement during the experiment. The strong confinement has enable Model D to develop compression strut in both directions and therefore develop optimum lateral resistance.

## 4.4 Reinforcement of the confining elements

---

In designs for high seismic risk reinforced-concrete members shall satisfy ACI 318(American Concrete Institute (ACI), 2014). For seismic design, member sizes should be selected and reinforcing steel arranged to avoid congestion of the reinforcement. Careful selection of member size and reinforcing steel arrangement will help to avoid difficulties in the placement of the reinforcement and concrete(Felder *et al.*, 1999).

### 4.4.1 Columns of confinement

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Reinforced concrete elements that are placed on the two borders of the confining wall are considered confinement columns. The confinement columns must be continuous from the foundation to the top of the wall and may melt before or after the wall is lifted.

#### a) Minimum dimensions

The dimensions for the confinement elements must be the following:

- Minimum thickness: the minimum thickness of the confinement elements must be the same as the confined wall.

- Minimum area: the minimum area of the cross section of the confinement elements is 200 cm<sup>2</sup>.

#### b) Location

Confinement columns should be placed in the following places:

- At the ends of all load-bearing walls.

- At intersections with other bearing walls.
- In the intermediate places at distances not greater than 25 times the thickness of the wall, 1.5 times the vertical distance between horizontal elements of confinement or 4 m.

c) Reinforcement

The minimum reinforcement of the confinement column should be as follows:

In any case, it will consist of at least three bars, whose total area is at least equal to that obtained with the following equation:

$$A_s = 0.2 \cdot \frac{f'_c}{f_y} \cdot t^2 = 0.2 \cdot \frac{21}{420} \cdot 200^2 = 400 \text{ (mm}^2\text{)} \quad (2)$$

Where:

$A_s$  - Longitudinal reinforcement area not prestressed to tensile (mm<sup>2</sup>)

$f'_c$  - Specified resistance to concrete compression (MPa)

$f_y$  - Resistance to yield of reinforcing steel (MPa)

t - Thickness of the wall

The transversal reinforcement of will be formed by closed stirrups and with an area,  $A_{sc}$ , at least equal to calculated with the following equations:

$$A_{sc} = \frac{1000 \cdot s}{f_y \cdot h_c} = \frac{10000 \cdot 150}{420 \cdot 200} = 17.86 \text{ (mm}^2\text{)} \quad (3)$$

Where:

$A_{sc}$  - Longitudinal reinforcing area not tensioned to tensile (mm<sup>2</sup>);

$h_c$  - Dimension of the height of the beam in the wall plane;

$f_y$  - Resistance to yield of reinforcing steel (MPa);

s - Separation of the stirrups (mm). The separation shall not exceed 1.5 t or 200 mm.

*Longitudinal reinforcement*

With slabs: When the houses have slabs, including the roof, the reinforcement must not be less than 4 bars. The area of longitudinal reinforcement must be greater than or equal to 0.0075 times the area of the gross section of element (NEC-SE-Vivienda, 2015)

*Transverse reinforcement*

- Diameter:

○ Transverse reinforcement consisting of closed stirrups of at least 6 mm in diameter should be used.

○ When using electro-welded reinforcement, the minimum diameter of the abutments must be 4.2 mm spaced at a spacing no greater than 1.5 times the smallest dimension of the element or 200mm.

• Minimum cross-sectional area

It can be accepted if it complies with what is stated in section 6.3.2(NEC-SE-Vivienda, 2015), in any case, the transversal reinforcement may be less than that required to meet the design efforts.




Column's reinforcement				
TYPE		C-1	C-2	C-3
DIMENSION				
1°FLOOR 2°FLOOR	ACERO	•4 Φ10 Mc201	•4 Φ10 Mc202	•4 Φ10 Mc203
	STIRRUPS	☒Φ8 Mc204 1@ 0.05, 5@ 0.10 Rta@ 0.20	☒Φ8 Mc205 1@ 0.05, 5@ 0.10 Rta@ 0.20	☒Φ8 Mc206 1@ 0.05, 5@ 0.10 Rta@ 0.20
QUANTITY		30	4	48

Figure 18 Reinforcement of the columns in the project

However, in dwellings of up to two floors, the maximum separation between the abutments of the beams and confinement columns may be 20 cm when the cutting effort of the wall of which the confinement elements are part is equal to or less than 50 % of the admissible wall shear stress. Table below shows the maximum separation of stirrups in confinement elements:

Table 5 Maximum stirrups spacing in the confining elements (NEC-SE-Vivienda, 2015)

Confining element	Placement zone	Separation
Beam	Critic	10 cm
	Intermediate	20 cm
Column	Critic	10 cm or hc/2
	Intermediate	20 cm

d) Reinforcement anchor

The vertical reinforcement of the confinement columns must be anchored to the foundation system. Junction bars anchored to the foundation can be used by hooks at 90 °. These bars must protrude the splice length by overlap from the top face of the foundation.

The vertical reinforcement splices of the confinement columns must comply with the corresponding requirements established in the ACI Code 318 (American Concrete Institute (ACI), 2014). At the upper end of the confinement column the longitudinal reinforcements must be anchored in a confinement element transverse to its direction with a hook 90 °.

Column verticals can be spliced by lap splices, mechanical splices, or welded splices. Lap splices are permitted only within the center half of the column length and shall be designed as tension splices. ACI 318 (318M) requires that mechanical splices or welded splices shall be staggered at least 24 in. (600 mm) and applied to alternate verticals. Offsets of longitudinal reinforcement is not recommended within the joint.

Lap splices shall not be used for bars larger than No. 11 except as provided in 12.16.2 and 15.8.2.3 (Felder *et al.*, 1999). Lap splices shall be permitted only within the center half of the member length, shall be designed as tension lap splices, and shall be enclosed within transverse reinforcement conforming to 21.6.4.2 and 21.6.4 (American Concrete Institute (ACI), 2014):

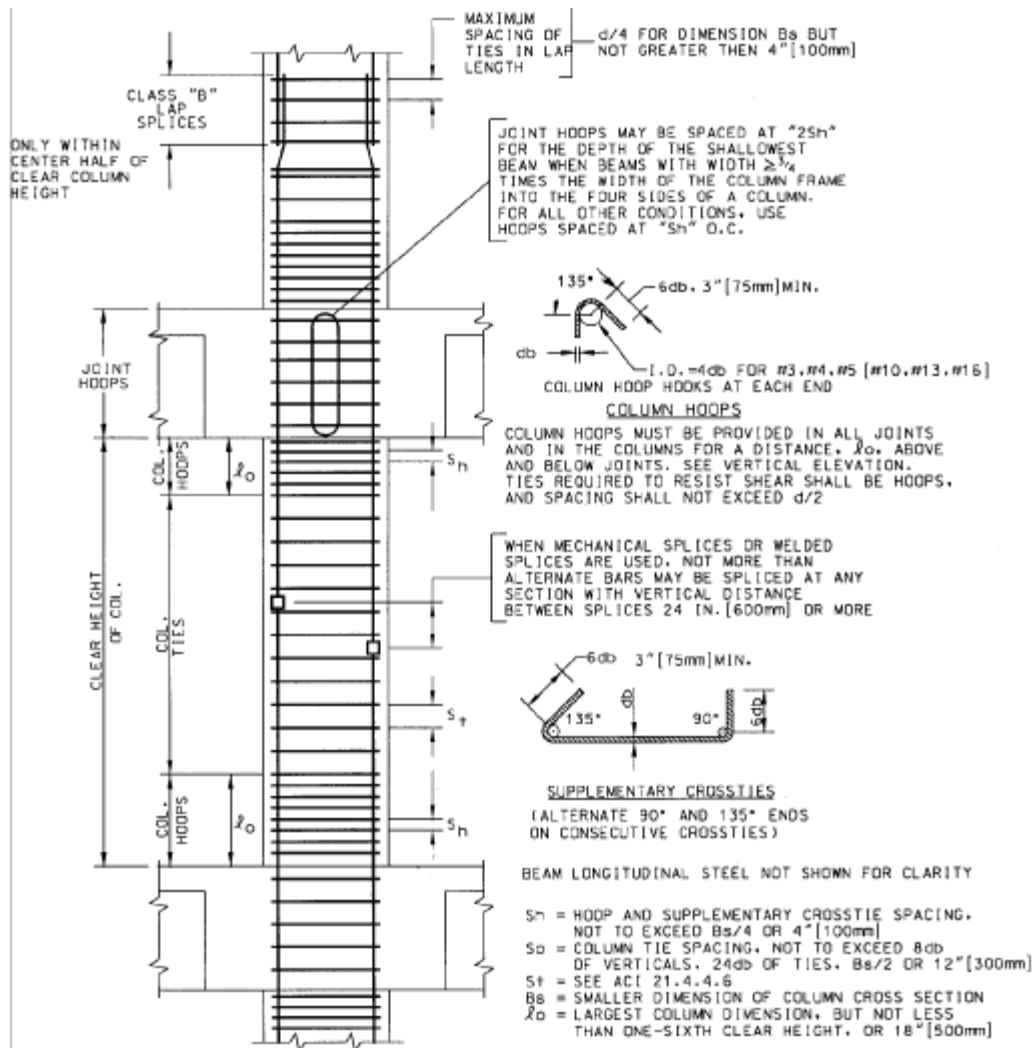


Figure 19 Typical seismic-resistant details: columns  
(American Concrete Institute (ACI), 2014)

## 4.4.2 Confining beams

The reinforced concrete elements that are placed in the lower and upper part of confined walls are considered confinement beams. Mooring beams can be emptied before or after the walls are lifted

### a) Minimum dimensions

The minimum dimensions for the confinement beams must be the following:

- Minimum thickness: the minimum thickness of the confinement beams must be the same as the confined wall.
- Minimum area: the minimum area of the cross section of the confinement elements is 200 cm<sup>2</sup>. If a slab of solid mezzanine with a thickness greater than or equal to 100 mm is

used. It is possible to do without the mooring beams projecting the slab on the confined masonry wall, placing the reinforcement required for the beam inside the slab. In beams that require veneering, the specified width can be reduced up to 75 mm, as long as its height is increased, in such a way that the transversal area is not less than the minimum

#### b) Location

Horizontal confinement beams should be placed at the start and at the top of the wall, in the mezzanines and at vertical free distances no greater than 25 times the thickness of the wall. The beams should be arranged forming closed figures in a horizontal plane interlacing the supporting walls in the two main directions, to form diaphragms with the help of the mezzanine or the roof. Mooring beams should be located at the following sites:

- At the foundation level. The foundation system constitutes the first level of horizontal mooring.

- At the level of the mezzanine system. Lashing beams must be part of the mezzanine system.

- At the deck level. Two options are presented for the location of the mooring beams and the configuration of the diaphragm.

- o Horizontal beams at the level of lintels plus lashing straps as the end of the cylinder heads.

- o Horizontal mooring beams in the walls without butts, combined with beams of inclined mooring, forming the ends of the cylinder heads.

#### c) Minimum reinforcement

The minimum reinforcement of the confinement beams should be as follows:

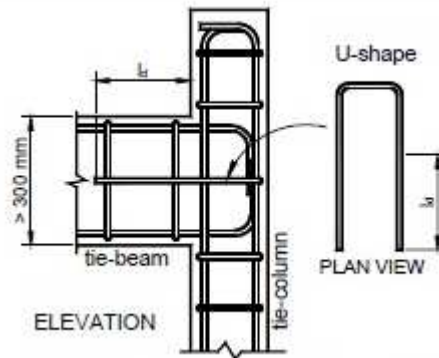
##### *Longitudinal reinforcement*

- With slabs: When the houses have slabs, including the roof, the reinforcement must not be less than 4 bars. The area of longitudinal reinforcement must be greater than or equal to 0.0075 times the area of the gross section of element. In any case, the longitudinal reinforcement cannot be less than required to meet the design efforts.

- For single-level dwellings that have light cover, without slab and lights less than 3.0 m, the reinforcement must not be less than 2 bars. The area of longitudinal reinforcement must be greater than or equal to 0.0075 times the area of the gross section of element.

### *Reinforcement anchor*

Where beams frame into only one side of a column, as at exterior columns, top and bottom beam reinforcing steel must have a 90-degree hook that extends to the far face of the confined region (core) and bends into the joint. The development length of the hook for tension shall not be less than  $8d_b$ , 6 in. (150 mm), or  $f_y d_b / (65 \sqrt{f'_c})$  [ $f_y d_b / (5.4 \sqrt{f'_c})$ ](Felder *et al.*, 1999).



*Figure 20 Additional confinement for vertical reinforcement in the tie-beam and tie-column joint region (Meli, 2011)*

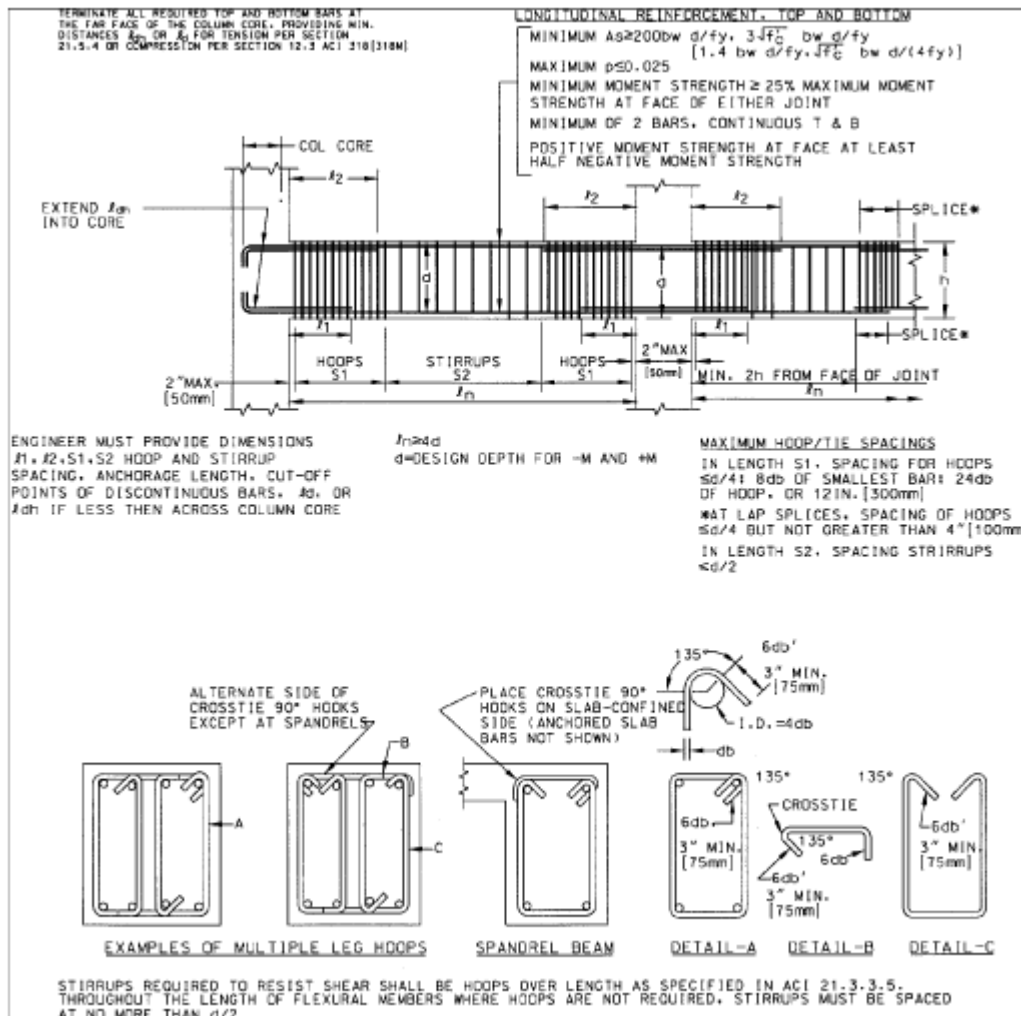


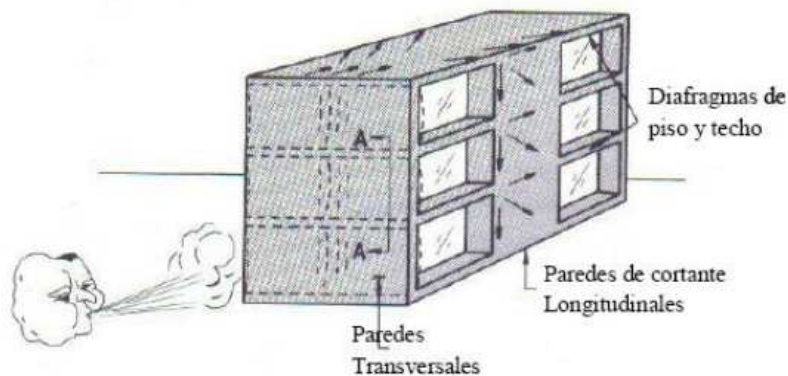
Figure 21 Typical seismic-resistant details: beams (American Concrete Institute (ACI), 2014)



## 5 Analysis of lateral and vertical forces of the structure

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The buildings not only resist vertical dead and live loads but also lateral forces caused by wind and earthquakes, as shown in figure below. Generally, these lateral forces are resisted by shear walls and frames resistant to moments. This section will outline how the shear walls act.



*Figure 22 Lateral force distribution in a building with shear walls (NEC-SE-MP, 2015)*

As shown in figure above, the lateral forces of severe winds or earthquakes flex the transverse walls between the floors. In box-type buildings, lateral loads are transmitted from these transverse walls to the lateral shear walls by the horizontal diaphragms.

### 5.1 Horizontal diaphragms

---

A horizontal diaphragm is analogous to a laminar beam in a horizontal plane, where the floor slab or roof deck functions as a membrane to withstand the shear force. Edge elements, such as link beams, serve as beam runners to develop the resistive moment.

By means of a suitable connection at the edges of the diaphragm, the horizontal shear is transferred directly to the cutting walls, as shown in Figure 23:

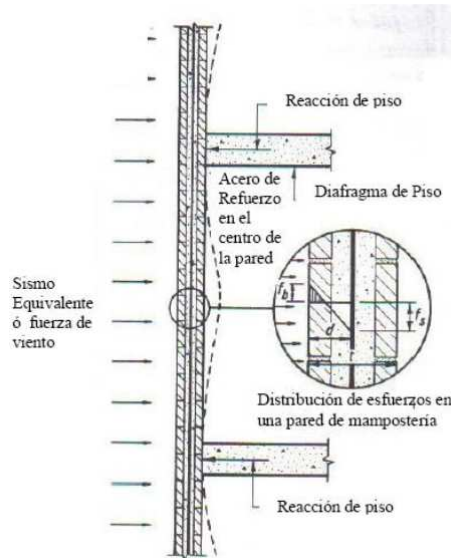


Figure 23 Distribution of loads and stresses in a wall  
(NEC-SE-MP, 2015)

As the diaphragm flexes and the shear develops, the loads are transferred to the diaphragm traction and compression elements. These shear walls must be able to resist shear and tumbling forces while the drag struts must carry the bending and axial forces. Likewise, masonry bonding beams act as wings for the diaphragms, they must be adequately reinforced to withstand the tension and compression forces applied.

Many types of diaphragm systems are used, most are made of reinforced concrete, metal or wood. The diaphragms can be flat, inclined or curved and can have openings although large openings should be avoided.

## 5.2 Deflexions in diaphragms and walls

Lateral loads on the walls due to wind or earthquakes cause the diaphragm to deflect and allow the walls to move relatively from their lower support. Since the masonry walls are relatively flexible perpendicular to the wall plane, they can tolerate a significant amount of bending and translation without damaging their shear strength parallel to the wall. The numerous horizontal mortar joints that can crack and open, providing an articulated wall that allows significant deflections of up to 0.007. The overstress in the masonry is not as critical as it is a significant embedment safety factor.

The deflection in the diaphragm can be calculated assuming that the walls are skids that resist flexion and deflection. These skid elements or wings can be considered as half the distance between the floors or parapet plus half the height of the wall from the floor to

the largest member. The height of the skate can also be assumed conservatively as 6 times the thickness of the wall (NEC-SE-MP, 2015).

## **5.3 Types of diaphragms**

---

As previously mentioned, the diaphragms can be constructed of concrete, metal, wood or other suitable materials. They can be flat, inclined, curved, folded or can be bent and also have openings. Generally, the diaphragms are classified as flexible or rigid depending on the deflections relative to the deflections of the resistant vertical walls(NEC-SE-MP, 2015). In my case we would have the rigid diaphragm in the slabs that connect with the bearing walls.

### **5.3.1 Flexible diaphragms**

---

Since wooden floors and ceilings are relatively flexible compared to masonry walls that are much more rigid, these are considered as flexible diaphragms. Because of their flexibility, it is assumed that they load the shear walls through the tributary area supported on each wall. They are also considered incapable of transmitting rotational or torsional forces. Flexible diaphragms are not common in confined masonry buildings. In my case we do not have this type of diaphragms.

### **5.3.2 Rigid diaphragms**

---

The floors or ceilings constructed of concrete and cast on a steel board are considered as rigid diagrams that can transmit rotational forces (torsional moments) towards the walls. Now for buildings with rigid diaphragms that do not have torsional irregularities in plant, it is allowed to evaluate the drift only in the center of mass of the diaphragm as in my case. When the structure has torsional irregularities, the drift must be evaluated on all the vertical axes of the column and on the vertical edges of the structural walls.

Due to their rigidity, rigid diaphragms are assumed to load the shear walls (resistant elements) in proportion to the relative stiffness of the same, therefore, if a rigid diaphragm is loaded along its axis, it is assumed that it will distribute the load to the shear walls in proportion to its rigidity. The more rigid the walls, the more force they will receive from the diaphragm (it is proportional) (NEC-SE-MP, 2015).

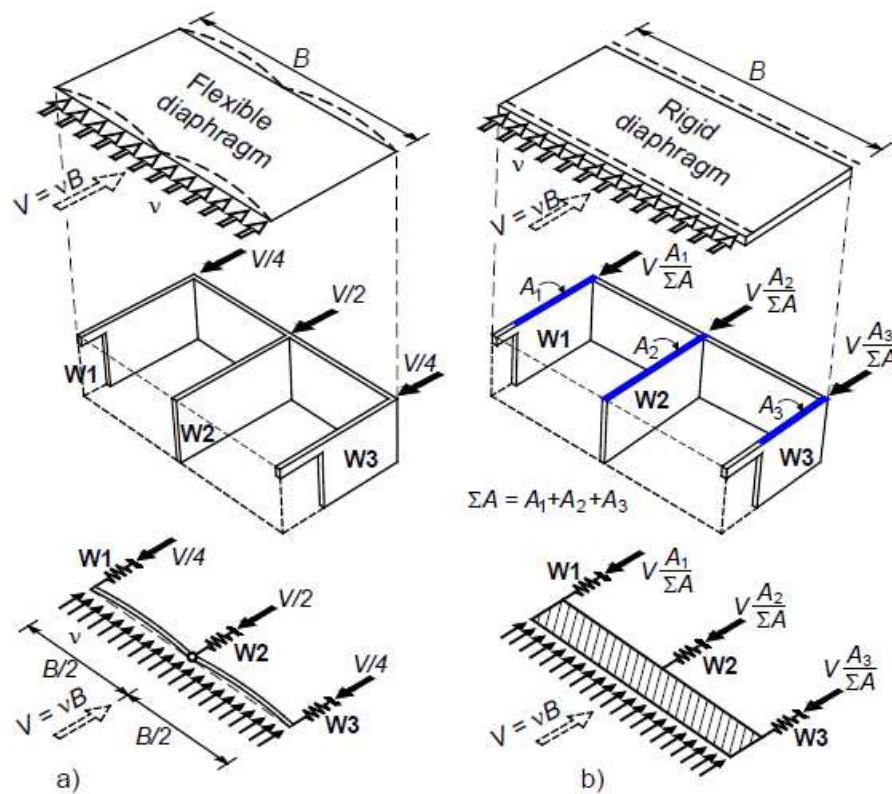


Figure 24 Distribution of lateral loads in buildings: a) flexible, and b) rigid diaphragm (Meli, 2011)

## 5.4 Stiffness of the walls

The stiffness of a wall element is dependent on its dimension, its modulus of elasticity, the modulus of rigidity or its shear modulus  $E_v$ ,  $\sigma$ ,  $G$ , and the support conditions of the wall above and below.

If the wall is fixed to the foundation (recessed), but the top part is free to move and rotate, it is considered a cantilevered or cantilever wall, this is similar to a cantilevered beam that deflects and rotates at its ends. If the wall is fixed both above and below it is considered a fixed or restricted wall, this is similar to a fixed or recessed beam at both ends.

The stiffness of the walls is defined as the reciprocal of the total deflection, which is formed by the sum of the deflection per moment and the deflection by shear, as shown in figure below (NEC-SE-MP, 2015).

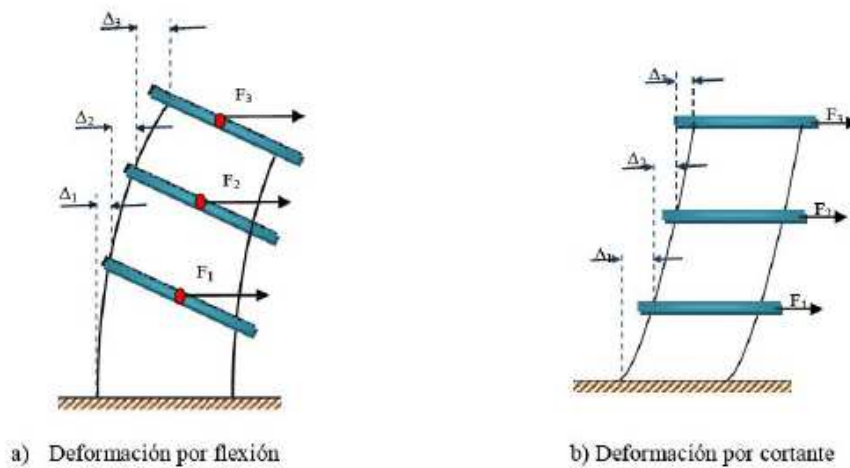


Figure 25 Deformation of the wall  
(NEC-SE-MP, 2015)

Walls with different configurations that have the same dimensions may have different stiffness, which changes the period of vibration of the structures, the response of the building and the amount of forces resisted by each wall or wall configuration. For example, walls with expansion joints will have a very low stiffness compared to walls of the same size without joints (NEC-SE-MP, 2015).

## 5.5 Overturning moment

The lateral and earthquake forces can create overturning moments in buildings, if the overturning moment is large enough, it can lift the dead weight of the structure and induce tension in the ends of the cutting walls, this also causes compression forces high that may require an increase in the  $f'_m$  of the masonry and also an increase in the compression steel in the wall, or an increase in the thickness or length of the shear wall.

In an evaluation of the stabilizing effect of the dead load on the moment of turning, 85% of the dead load could be used (NEC-SE-MP, 2015).

The overturning moment at the base of a structure is found by:

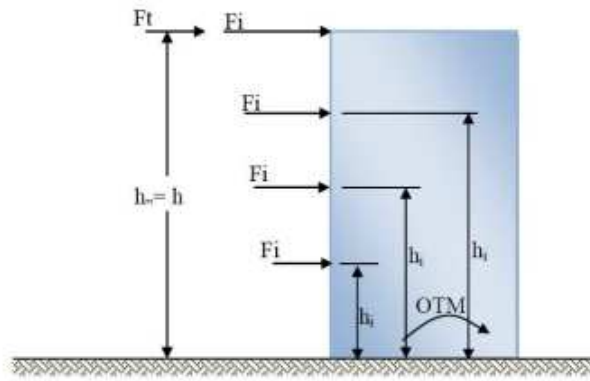


Figure 26 Overturning moment  
(NEC-SE-MP, 2015)

The turning moment for each wall should also be determined at the floor levels to establish the amount of reinforcement required, loads and efforts in the masonry, and is given by:

$$OTM_x = F_t \cdot (h_n - h_x) - \sum_1^n f_i \cdot (h_i - h_x) \quad (4)$$

The overturning moment at a level x, above the base is equal to the force,  $F_t$  (in case there is whip force) multiplied by the height up to the level x ( $h_n - h_x$ ), plus the sum of the forces of each  $F_i$  level multiplied by the height from level i to level x ( $h_i - h_x$ ) (NEC-SE-MP, 2015)

## 5.6 Torsion

In buildings with rigid diaphragms on the floor and roof levels, seismic forces are resisted by shear walls, in proportion to their stiffness. If all the elements resistant to the lateral forces are of the same size and are located symmetrically they will be loaded by an equal lateral force. It means that in these conditions the center of mass coincides with the center of rigidity.

However, if some walls are more rigid than others or if they are located asymmetrically, some elements resistant to lateral forces will resist more load than the others. This condition in which the center of rigidity does not coincide with the center of mass produces torsional moments. The center of mass tends to rotate around the center of rigidity.

If a building has a frontal opening, severe torsional stresses can occur because there is a large eccentricity between the center of mass and the center of rigidity of the building (see figure below).

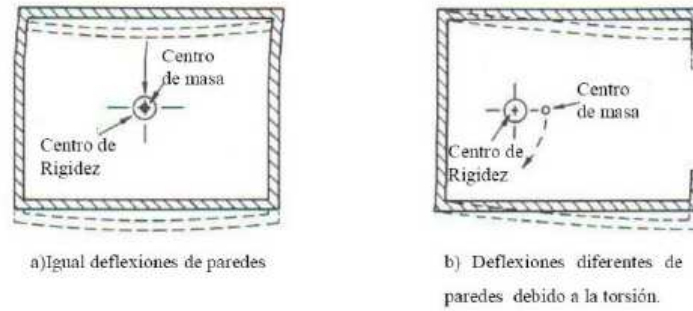


Figure 27 Lateral distortions of a building  
(NEC-SE-MP, 2015)

Due to the torsion the lateral force resisted by some cutting walls will be significantly increased. For safety, all buildings that have a rigid diaphragm must be designed considering at least 5% of torsional eccentricity to take into account the variations in materials and the locations of the walls (NEC-SE-MP, 2015). This eccentricity must be added to the calculated eccentricity.

Additionally, negative torsional effects must be ignored. The torsion for each direction will be:

$$M_{tx} = V_x \cdot (e_y) \quad (5)$$

$$M_{ty} = V_y \cdot (e_{yx}) \quad (6)$$

Where:

$V_x$  and  $V_y$ , are the shears calculated for each direction:

$$e_y = e_y(\text{calculated}) + 0.05 \cdot L \quad (7)$$

$$e_x = e_x(\text{calculated}) + 0.05 \cdot W \quad (8)$$



# 6 Project

## 6.1 Description of the project

### 6.1.1 Location

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The study project is located in Pichincha Province, Canton Quito, Poblacion Yaruqui. The Poblacion Yaruquí is located in the Metropolitan District of Quito, Province of Pichincha in the North-East, 32 Km. From the city of Quito. It borders to the North with the Czech Parish, to the South with the Parish of Pifo and Tababela, to the East with the Cordillera Central - Province of Napo, and to the West with the Parish of Tababela and Guayllabamba. Its altitude is 2,527 m.s.n.m (Medios, 2015).



Figure 28 Location of the land (Google maps)

### 6.1.2 Climate

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The urban parish of Yaruquí has an average temperature between 12°C and 28°C. The relative humidity is 86.1%; and the average annual rainfall is 0.4 to 29.4 mm<sup>3</sup>. The climate of Yaruquí is varied being most of the time cold and with great amount of winds, at an average temperature of 15° C, and a humidity relative of 86.1%; and the average annual rainfall is 0.4 to 29.4 mm. Because which is located in the Sierra region at a height of 2,527 meters above the level from sea (Medios, 2015).

### 6.1.3 Surface

Yaruquí has a territorial area of 3,116.28 km<sup>2</sup> (Medios, 2015).

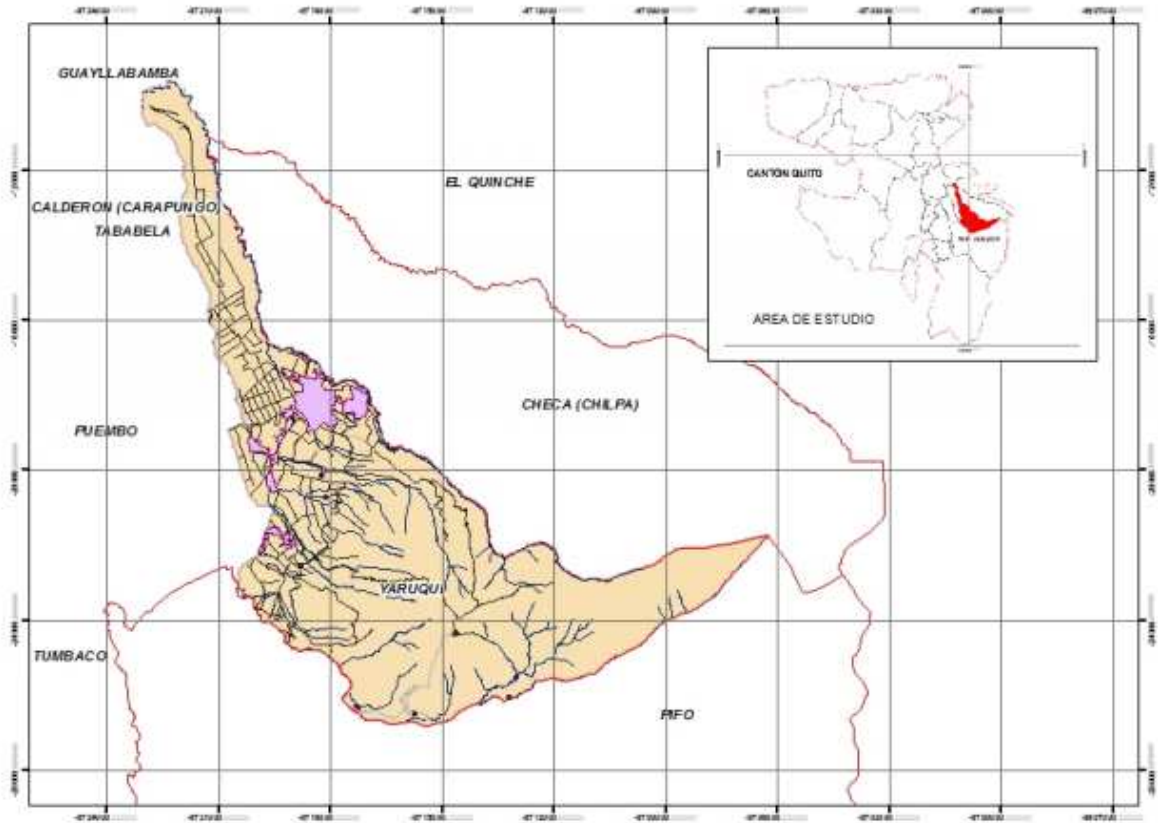


Figure 29 Yaruquí territory  
(Medios, 2015)

### 6.1.4 Topography

The terrain maintains a flatness throughout.

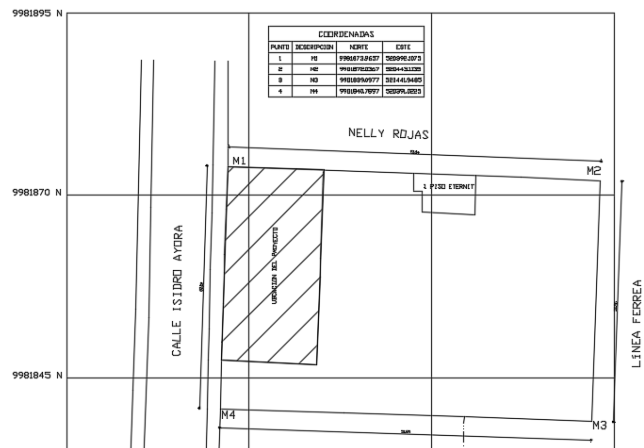


Figure 30 Topographic map

### 6.1.5 Relief

The relief is irregular, being part of several micro and sub basins, typical of the slopes and upper reliefs, as well as the lower slopes and reliefs of the inter-Andean basins, as well as being part of the basin reliefs, which favors to have several micro climates.

To the being irregular, in the general context it is part of a plain that in its development is taking height towards the páramos that conform the mountain range of The Andes.

*Table 6 Description of geomorphological units (Medios, 2015)*

Relieve	Descripción
Climas Fríos	Formas heredadas paleo-glaciares
Cimas frías de las cordilleras	Volcanes: Construcciones de tipo estrato-volcánico compuesto de proyecciones piroclásticas dominantes con intercalaciones de capas de lava
Relieves Interandinos	Vertientes y relieves superiores de las cuencas interandinas
	Vertientes inferiores y relieves de las cuencas interandinas
	Relieves de los fondos de cuencas

### 6.1.6 Geology

It is located in an area of direct influence of several volcanic structures, such as the Ilaló and the Ñuñurco, or others more distant such as the Antisana, the Sumaco and the Cotopaxi, whose direct and indirect influence by emanation of ash (mainly distal volcanic products) over the area, which gives this area the general characteristic of product filling Vulcano - sedimentary of Holocene age. To this powerful layer of volcanic products it is possible to observe the in the cuts of the rivers and in the slopes of the roads, clear and didactic cuts that are visible in the sector of the construction of the road to the new airport in Tababela; therefore, it is a soil of sedimentary vulcano lithologies from the Holocene to the present and colloquially referred to as Cangahua(Medios, 2015).

*Cangagua (Holocene)*

Most of the study area is covered by a compact material of creamy yellowish to dark brown color, the appearance of which is fine to coarse granulometry, so it has a variable

appearance, from sand to clay, predominating the sand - silt - clay mixture in varied proportions.

Its appearance is tobáceo and its thickness is variable, from centimetric to powerful banks of between 20 and 25 m; This is easily observable because it constitutes the vertical slope of most roads. Due to its form of sedimentation (precipitation from the air), small monoclinals are formed on the pre-existing relief, within the sequence it is possible to observe the presence of inter-stratifications of pumice, lapilli or cream-colored sand from light to light gray or dark.

#### *Volcanic Guayllabamba (Pleistocene)*

Its outcrops are located in the valley of the Guayllabamba River, correspond to agglomerates and tuffs without stratification whose size variations are important, from centimetric to several meters' radius, and are cemented by a silty sand matrix (ash); At specific sites, porphyritic lavas of intermediate chemistry are observed.

#### *Vulcano Sediments Chiche (Pleistocene)*

They are composed of layers of thick sands, ash and stratified tuffs of beige to beige cream, forming a powerful mass of sediments that are observed in the vertical walls of roads and ravines, the estimated power is around 70 meters.

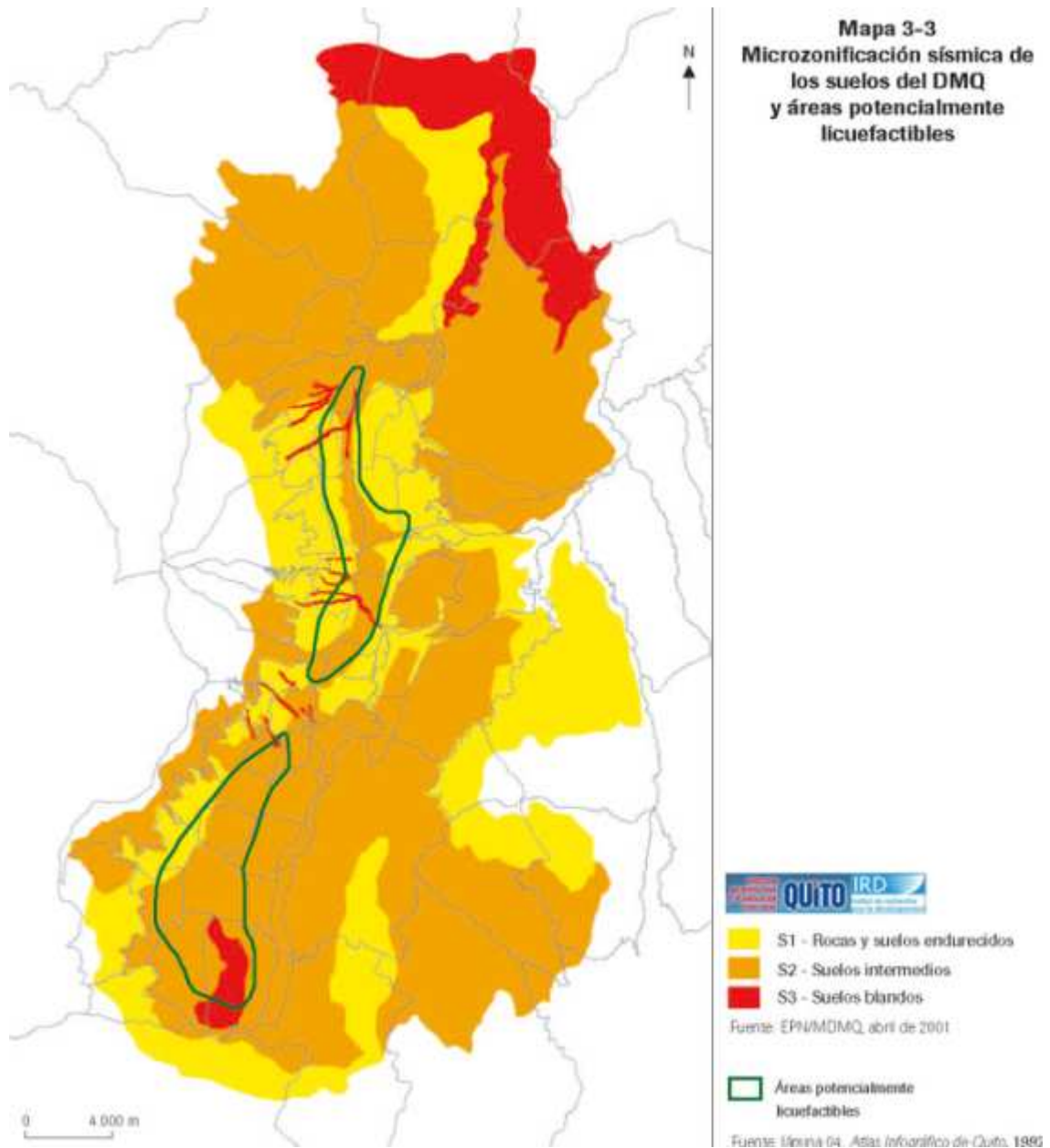
#### *Volcanic Ñuñurco*

These are attributed to the hill of the same name (3749 masl), located on the eastern side of the study area. The lava flow of andesite porphyritic - amphibolitic composition.

It should be mentioned, that these last two units do not appear in the study area, they are covered by powerful strata of the Cangahua unit, reason why it is underlying the latter (Medios, 2015).

#### *Classification of soils*

In the study conducted by Jorge Valverde (Valverde *et al.*, 2001) the soils of Quito are classified according to the soil profiles defined by the CEC 2000, which has 4 profiles that go from S1 to S4. According to this study, the city only has types S1, S2 and S3, as shown in figure below.



*Figure 31 Seismic microzonification of the soils of the Quito metropolitan district (Valverde et al., 2001)*

Profile S1 corresponds to rock or firm soil. To this group belong rocks or hardened soils with cutting wave velocities similar to that of a rock. The following types of soil are included: healthy or partially altered rock; sandy, silty or clayey, dense and dry gravels; hard cohesive soils with cut resistance greater than 1 kg / cm<sup>2</sup>; soils and volcanic deposits firmly cemented, tuffs and conglomerates.

Profile S2 corresponds to intermediate soils. Soils with intermediate characteristics or that do not conform to the conditions of soil profiles type S1 and S3.

Profile S3 corresponds to soft soils. This group includes soft, semi-soft, hard and very hard cohesive soils.

In general, it can be said that Quito's soils are good, since they settle on a Cangahua, which is a type of very resistant and suitable for construction. Only in the south of the city, in the Turubamba sector, the soils have little resistance to compression.

### **6.1.7 Volcanic risk**

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There are no volcanic centers that represent a serious danger for the Yaruquí area. However, volcanic ash showers may eventually occur through the Reventador, Guagua Pichincha volcanoes (1998-2002), which will depend on the direction of the wind. As it happened in the last eruptive process of great magnitude of the El Reventador volcano. The volcanic risk in the area can therefore be considered as very low (Medios, 2015).

### **6.1.8 Seismic risk**

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Ecuador registers important seismic activity in a large part of its territory, the area of Yaruquí (Yaruquí, Puembo, Pifo) is located in an area of high to medium seismic activity. Grade VI seismic events have been recorded in an extension range of 50 kilometers as a consequence of the subduction of the marine and continental plate, which evidences the influx of volcanoes mostly inactive and highly active in two specific cases.

In the same way, fault cliffs are observed evidencing damage, especially in the fault that runs along the Santa Rosa creek. The seismic risk for the area is therefore considered Medium (Medios, 2015).

## **6.2 Structural description**

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The project includes a two-story multifamily building with an area of 282.22 m<sup>2</sup> per floor in the city of Quito with a total construction area of 564.44 m<sup>2</sup>. The apartments of approximately 70 m<sup>2</sup> have two bedrooms, kitchen, living room, dining room and bathroom, distributed in the best possible way. The access is located in the central area of the first floor building, which lead to the staircase that joins the two levels. Being two (four for the second floor) apartments per floor, which gives us a total of 6 flats.

The building designed like a confined masonry structure. Therefore, it has to comply with the requirements mentioned earlier for the confined masonry structures. As it is

presented in the table below, the building has some irregularities like incorrect ratio between the length and the width of the building in plane, so we cannot use simplified method for the confined masonry design as it is mentioned in the Ecuadorian code NEC-2015(NEC-SE-Vivienda, 2015).

For the development of this design, the following documents were reviewed:

- a) Ecuadorian Construction Standard (NEC-2015);
- b) Colombian Resistive Earthquake Construction Regulations (NSR-10, 1997);
- c) Complementary Technical Standards (NTC, 2004).

The literal a) consists of several documents with different applications, specifically used the documents of structural masonry (NEC-SE-MP, 2015) and housing up to 2 floors with heights up to 5 m(NEC-SE-Vivienda, 2015). Information was also obtained from the Practical Guide to Housing Design of Up to 2 Floors with Heights of up to 5 meters(NEC-2015 Guía práctica, 2015). This guide is an easy-to-understand reference document implemented by the Ecuadorian Construction Standard (NEC-2015).

The Colombian Seismic Resistance Construction Regulation (NSR-10) listed in subparagraph b) consists of several titles. Title E - One and two story houses, and Title D - Structural masonry were taken into consideration.

The literal c) includes the Complementary Technical Standards, these are used for construction in the Government of the Federal District of Mexico. The document on Construction of Masonry Structures contains information regarding the design for structures in confined masonry.

According to the Colombian regulation codes (NSR-10, 2010) the structures of confined masonry have a limitation in the height according to the table below. Also Ecuadorian regulations allow to build no more than two stories.

Table 7 Structural system of load-bearing walls  
(NSR-10, 2010)

A. SISTEMA DE MUROS DE CARGA		Valor $R_0$ (Nota 2)	Valor $\Omega_0$ (Nota 4)	zonas de amenaza sísmica					
				alta		intermedia		Baja	
Sistema resistencia sísmica (fuerzas horizontales)	Sistema resistencia para cargas verticales			uso permit	altura máx.	uso permit	altura máx.	uso permit	Altura máx.
1. Paneles de cortante de madera	muros ligeros de madera laminada	3.0	2.5	si	6 m	si	9 m	si	12 m
<b>2. Muros estructurales</b>									
a. Muros de concreto con capacidad especial de disipación de energía (DES)	el mismo	5.0	2.5	si	50 m	si	sin límite	si	Sin límite
b. Muros de concreto con capacidad moderada de disipación de energía (DMO)	el mismo	4.0	2.5	no se permite		si	50 m	si	Sin límite
c. Muros de concreto con capacidad mínima de disipación de energía (DMI)	el mismo	2.5	2.5	no se permite		no se permite		si	50 m
d. Muros de mampostería reforzada de bloque de perforación vertical (DES) con todas las celdas rellenas	el mismo	3.5	2.5	si	50 m	si	sin límite	si	Sin límite
e. Muros de mampostería reforzada de bloque de perforación vertical (DMO)	el mismo	2.5	2.5	si	30 m	si	50 m	si	Sin límite
f. Muros de mampostería parcialmente reforzada de bloque de perforación vertical	el mismo	2.0	2.5	Grupo I	2 pisos	si	12 m	si	18 m
g. Muros de mampostería confinada	el mismo	2.0	2.5	Grupo I	2 pisos	Grupo I	12 m	Grupo I	18 m
h. Muros de mampostería de cavidad reforzada	el mismo	4.0	2.5	si	45 m	si	60 m	si	Sin límite
i. Muros de mampostería no reforzada (no tiene capacidad de disipación de energía)	el mismo	1.0	2.5	no se permite		no se permite		Grupo I (Nota 3)	2 pisos
<b>3. Pórticos con diagonales (las diagonales llevan fuerza vertical)</b>									
a. Pórticos de acero estructural con diagonales concéntricas (DES)	el mismo	5.0	2.5	si	24 m	si	30 m	si	Sin límite
b. Pórticos con diagonales de concreto con capacidad moderada de disipación de energía (DMO)	el mismo	3.5	2.5	no se permite		si	30 m	si	30 m
c. Pórticos de madera con diagonales	el mismo	2.0	2.5	si	12 m	si	15 m	si	18 m

To consider the walls as structural they should have continuity from the foundation to its upper level and should not have openings, so that an arrangement of elements is made in such a way that it adapts to the typical house and in turn complies with the existing regulations. Both the effectiveness of the joints in the diaphragms and the joint work of the structural systems depend on the vertical continuity and the regularity of the structure, both in plan and in height.

Table 8 Parameters of the structure

Plan regularity must comply:			
Condition	Code	Project	Conformity
Height	$\leq 6m$	5,8	ok!
Length/Width	$\leq 2$	2,2	not ok!
Height/Width	$\leq 1,5$	0,5	ok!
Floor openings	$\leq 25\%$	2,6	ok!
Thickness of bearing walls	$\geq 10cm$	14	ok!
Column area	$\geq 200$ $cm^2$	400	ok!

Distance between columns	25t	4	ok!
	≤ 4	4	ok!
	1,5h	4	ok!

## 6.2.1 Materials

### a) Concrete

All concrete elements will be constructed using a compressive strength of 21 MPa., And a steel of 420 MPa, that complies with NEC-2011 (NEC-SE-Vivienda, 2015).

The value of the modulus of elasticity for concrete of normal density is:

$$E_c = 4.7 \cdot \sqrt{f_c} = 21.54 \text{ (GPa)}(9)$$

The screenshot shows the 'Material Property Data' dialog box for a material named 'concrete210'. The dialog is organized into several sections:

- General Data:** Material Name is 'concrete210', Material Type is 'Concrete', Directional Symmetry Type is 'Isotropic', and Material Display Color is a pink square. Buttons for 'Change...' and 'Modify/Show Notes...' are present.
- Material Weight and Mass:** 'Specify Weight Density' is selected. Weight per Unit Volume is 2400 kgf/m³ and Mass per Unit Volume is 2400 kg/m³.
- Mechanical Property Data:** Modulus of Elasticity, E is 2154 kgf/mm²; Poisson's Ratio, U is 0.2; Coefficient of Thermal Expansion, A is 0.0000099 1/C; Shear Modulus, G is 897.5 kgf/mm².
- Design Property Data:** A button for 'Modify/Show Material Property Design Data...' is shown.
- Advanced Material Property Data:** Buttons for 'Nonlinear Material Data...', 'Material Damping Properties...', and 'Time Dependent Properties...' are shown.

At the bottom of the dialog are 'OK' and 'Cancel' buttons.

Figure 32 Materials properties assignment in ETABS (concrete)

The mortar used for the joint of the masonry units will be with a volumetric dosage of 1:3. The thickness of this mortar is approximately 1.5cm, which has a compressive strength of 15 Mpa and is used on both sides of the masonry.

b) Masonry

Masonry properties as a material: the confined masonry walls are modeled as a homogeneous material (common properties for the masonry wall and the confining elements).

The masonry of confined walls must be constructed using concrete or clay units. The masonry units can be perforated (vertically or horizontally) or solid.

Resistance to the Compression of the masonry ( $f_m$ ) assumed as the compressive strength of the artisanal brick of Susudel, province of Azuay; whose average value is 8MPa.

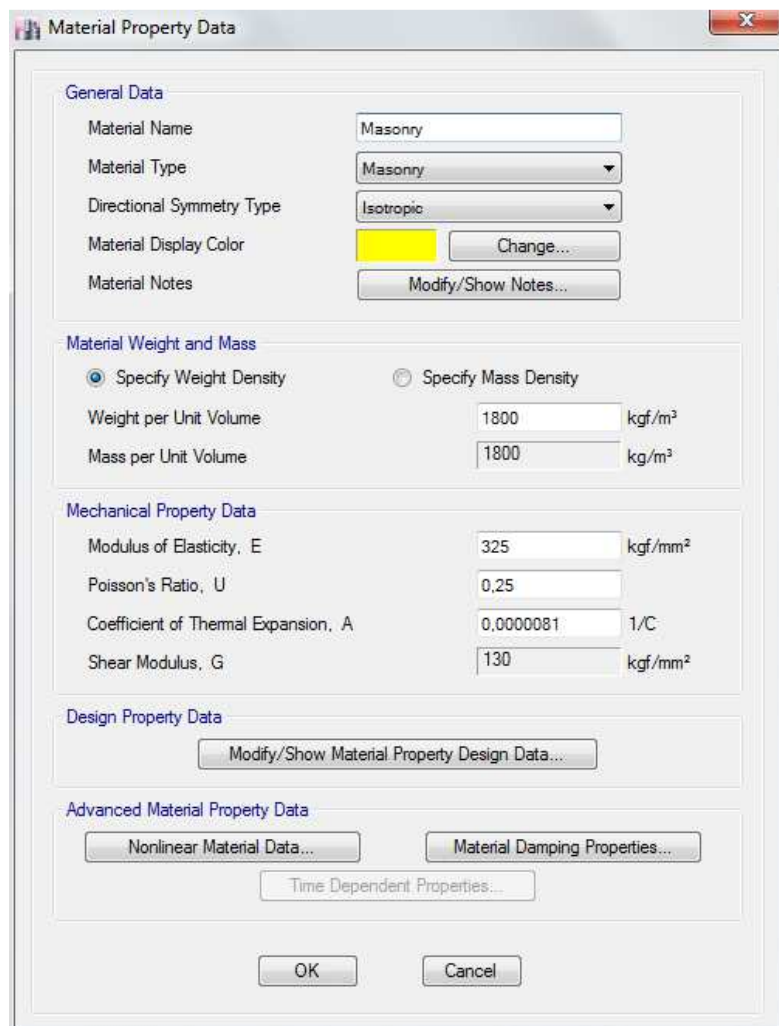


Figure 33 Materials properties assignment (masonry)

The sections used for the calculation are the average width of the solid brick unit in Ecuador: 140 mm and 200 mm according to following design.

In the interlocking method of blocks in structural masonry, the overlapping units with the upper and lower units must be placed at least one quarter of the length (NSR-10, 1997). The rigging must be done on a rope, that is, placing the longest side of the block horizontally to form a structure. Figure below shows graphically the rigging of interlocking blocks for structural masonry



Figure 34 Locked rigging units placed on a rope for structural masonry (<http://www.arkiplus.com/mamposteria>)

c) Mortar

Adhesive mortars for confined masonry constructions should have good plasticity, consistency and ensure their adherence with the masonry units (NEC-SE-MP, 2015). It is a requirement that the mortar glue comply with the NTE INEN 0247 standard (NTE INEN 0247, 2010). Laboratory tests of dosages should be performed or results of previous experiences used. The types of glue mortar will be classified according to the minimum dosage of their components as shown in Table below:

Table 9 Types of mortar, dosage and minimum resistance to compression at 28 days

Type of the mortar	Min compression resistance (28 days) Mpa	Volume composition		
		Cement	Lime	Sand
M20	20	1	--	2,5
M15	15	1	--	3,0
		1	0,5	4,0
M10	10	1	0,5	4,0

		1	0,5	4,0
M5	5	1	--	6,0
		1	1,0	7,0
M2,5	2,5	1	--	7,0
		1	2,0	9,0

The lime used in the dosage of the mortar paste will be hydrated lime. The aggregates for mortar paste must comply with the NTE INEN 2536 standard (NTE INEN 0247, 2010). It must be verified that the water is free of harmful elements such as oils, acids, alcohols, salts or organic materials, since these components can affect the quality of the mortar (NEC-SE-MP, 2015).

#### d) Reinforcing steel

The steel rods used in the load-transmitting element that is the beam, are corrugated rods, with an effort to the yield  $f_y = 4200 \left(\frac{kg}{cm^2}\right)$ , the diameter of this rod is Ø10-12 mm, which is a diameter that I chose for this particular analysis, a smaller diameter could be chosen later according to the design of the required wall.

## 6.3 Design loads

### 6.3.1 Dead loads

Gravitational actions that will act permanently on the structure and do not vary with time, and also indirect actions with permanence character, are considered dead charges; for the present case it has been considered:

- The own weight of the structure
- Additional dead load

These loads are applied simultaneously for the design of the elements of the structure.

Table 10 Dead loads

Dead load			
Element	Symbol	Value	Units
Masonry (140 mm and 200 mm)	D	154	kg/m <sup>2</sup>
Slab (waffle slab 200 mm)	D	264	kg/m <sup>2</sup>
Floor (levelling mortar + floor)	D	63	kg/m <sup>2</sup>
Roof (fiber cement sheet + clay tile without mortar)	D	15	kg/m <sup>2</sup>

### 6.3.2 Variable loads

The overloads that are used in the calculation depend on the occupation to which the building is intended and are made up of the weights of people, furniture, equipment and mobile or temporary accessories, merchandise in transition, and others. For our case it has been considered according (NEC-SE-CG, 2011):

Table 11 Live loads + Hile load

Variable loads			
Element	Value	Value	Units
Live Load - Floor	L	200	kg/m2
Live Load - Roof	Lr	70	kg/m2
Hail Load	S	100	kg/m2

Hail load has to be considered an accumulation of hail in a short time. It should be taken into account for regions of the country with more than 1500 m of elevation. For roofs with slopes less than 5% a minimum hail load of 1.0 kN/m2 must be considered (NEC-SE-CG, 2011)

For the wind loads the design speed up to 10 m in height of the building will be appropriate to the maximum speed for the location zone of the building, but it will not be less than 21m/s (75 km/h). The wind load pattern is design in ETABS software according ASCE 7-10 (ASCE, 2010) in two perpendicular directions X and Y:

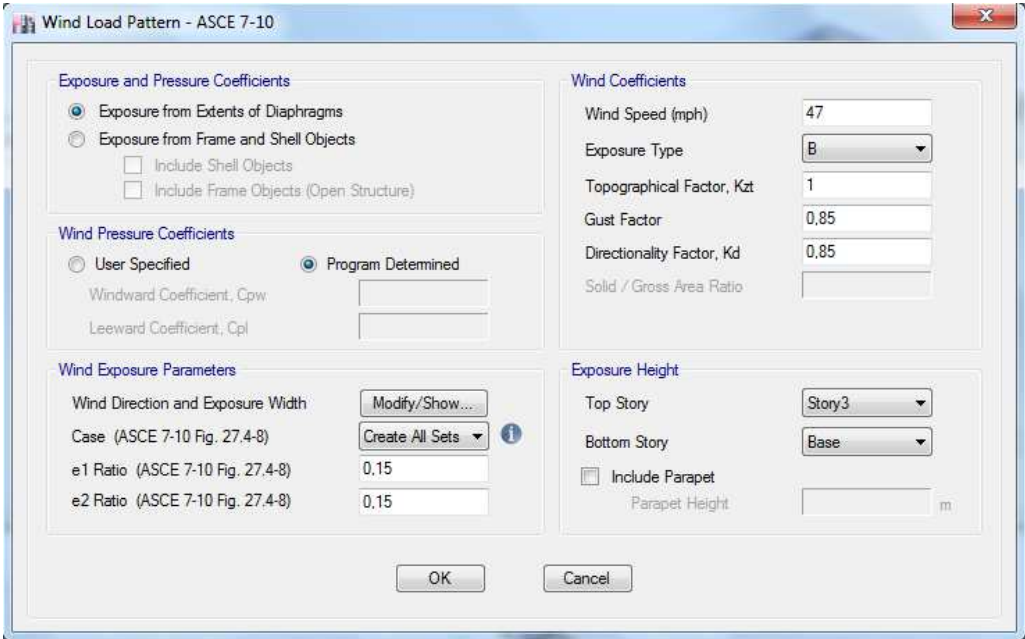


Figure 35 Wind load (ETABS)

### 6.3.3 Combinations

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When appropriate, each limit state of resistance should be investigated. The most unfavorable effects, both wind and earthquake, do not need to be considered simultaneously.

The structures, components and foundations must be designed in such a way that the design resistance equals or exceeds the effects of the increased loads, according to the following combinations:

*Table 12 Load combinations  
(NEC-SE-DS, 2015)*

Dead Load	<b>D</b>	Combination 1	$1,4D$
Seismic Load	<b>E</b>	Combination 2	$1,2D+1,6L+0,5\max(Lr; S; R)$
Live Load	<b>L</b>	Combination 3	$1,2D+1,6\max(Lr; S; R)+\max(L; 0,5W)$
Live Load (roof)	<b>Lr</b>	Combination 4	$1,2D+1,0W+L+0,5\max(Lr; S; R)$
Hile Load	<b>S</b>	Combination 5	$1,2D+1,0E+L+0,2S$
Wind Load	<b>W</b>	Combination 6	$0,9D+1,0W$
		Combination 7	$0,9D+1,0E$

## 7 Seismic zones in Ecuador

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The 13 million Ecuadorians live in a country that is crossed by geological faults, therefore it has a high seismic risk, the instability of the soils, the great probabilities of landslides, landslides and subsidence, in addition to the deforestation that increases the fragility of land: 180 thousand hectares per year, therefore all activity related to land use must take into account this reality. Seismicity or seismic activity in Ecuador is mainly due to the subduction of the Nazca plate under the South American plate known as the South American fault, in addition the contact with the Cocos plate in the North of the country have generated many disasters such as the one provoked in the city of Esmeraldas in the year 1906.

In addition, active faults have been located in the interior of Ecuador, mainly in the inter-Andean zone, which have caused the most destructive earthquakes in the country, such as in 1987 in Baeza, in 1990 in Pomasqui and in 1996 in Pujilí. more than 5000 houses, and had a magnitude of 5.7 on the Richter scale, locating its epicenter 95 km from Quito. All these disasters, support this research that prevention is necessary even more when dealing with unpredictable disasters. In several earthquakes in Ecuador as well as in the Andean region, it has been observed that in general the structures are traditional construction systems, including those of adobe, common brick or artisanal, which have a poor seismic behavior, even reaching collapse very quickly, even in the face of moderate earthquakes.

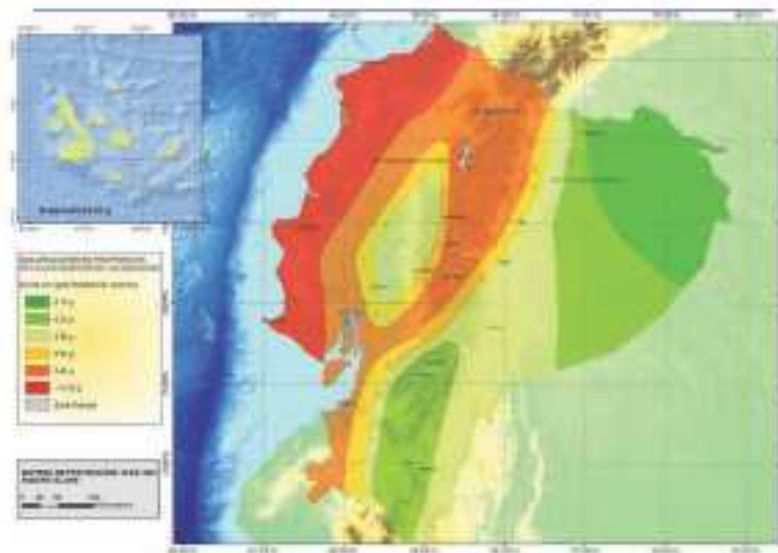
One of the earthquakes that have most affected Quito is that registered at 00h30' on March 22, 1859, leaving 30 dead in the capital, damages in some churches, among which the San Francisco and San Juan churches stand out. Destruction in several houses of 1 floor, built with adobe without major technique. This earthquake associated with subduction was felt in Guayaquil. In 1970, a 7.1-magnitude earthquake in Bahía de Caráquez destroyed much of this city, another strong earthquake occurred in Bahía de Caráquez (Manabi) in 1998. On August 4, two earthquakes occurred, which, according to the Richter scale, they were located in 5.1 and 7.1 degrees, they destroyed the city and several neighboring towns.

Other localities that experienced earthquakes of great magnitude were: Latacunga in 1976, the earthquake at that time was 6 degrees on the Mercalli scale - a measure used to

calculate the severity of earthquakes for damage before the Richter measurement -; Esmeraldas, in April of the same year another earthquake of 6.8 degrees lived on the Mercalli Scale; and other populations such as: Loja in 1970 and Pastaza in 1987. On Thursday, March 5, 1987, two earthquakes of magnitude 6.1 and 6.9 occurred along the eastern foothills of the Andes in the northwest of the country. Its epicenters were located in the province of Napo, approximately 100 km east-northeast of Quito and 25 km north of the Reventador volcano.

On the night of August 10, 1990, an earthquake of magnitude 5.3 located just 2 kilometers north of the tremor epicenter on August 12, 2014, destroyed adobe and mud houses within a radius of 12 kilometers in all Modern constructions, especially in the areas of Pomasqui and Pusuquí. The earthquake measuring 5.1 on the Richter scale, which occurred on Tuesday, August 12, 2014, is considered the largest earthquake that has been measured in the city. The more an historical earthquake is studied, the better it is for everyone, since there will be different points of view of an important event that for obvious reasons there are no instrumental records (Aguilar Falconí, 2013).

The map shown below is the Seismic Zoning Map of Ecuador, published in 2011, this is the map of current use for the seismic hazard in the country.



*Figure 36 Seismic design map  
(NEC-SE-DS, 2015)*

Table 13 Z-factor values depending on the seismic zone  
(NEC-SE-DS, 2015)

Seismic zone	I	II	III	IV	V	VI
Value of Z factor	0,15	0,25	0,30	0,35	0,40	$\geq 0,50$
Seismic threat	Intermediate	High	High	High	High	Very high

In addition, there are other dangers to consider as volcanic eruptions, tsunamis and natural disasters, which have left their traces throughout history, with different consequences. However, it is the earthquakes that have caused the most damage in Ecuador in recent times.

## 7.1 Seismic analysis

---

The seismic analysis of a structure is the study of its behaviour in front of possible telluric movements, obtaining the response in forces produced in the different elements of the building and its displacements.

The design must be able to meet the following objectives:

- During mild earthquakes, the structure must not present any damage.
- During moderate earthquakes, the structure must withstand the forces produced, experiencing possible damage within tolerable limits, with the possibility of being repaired.
- During severe earthquakes, the structure must avoid collapse and protect the lives of the occupants. For this work, the ETABS program was used, taking into account the provisions of the NEC 2015 (NEC-SE-DS, 2015).
- The building was modeled considering the two floors forming a three-dimensional model, restricting the movement of the base of the elements of the first floor so that it is an embedment to represent the foundation of the building.
- The slabs in two directions were modeled as membrane elements because they have the function of distributing the load of the roof to the bearing walls.
- The mass of the structure was defined according to the loads assigned to the model, located in the central mass of the rigid diaphragms of each level.
- Design loads and ultimate load combinations were defined to obtain the load envelope for the design of the structural elements of the building.

- Three vibration modes were defined for each level, 2 for translation in the XX and YY directions, and one for rotation around the ZZ axis. Referring to a possible earthquake.

## 7.2 Seismic parameters

Below is the procedure for choosing data that we will put in the ETABS 2016 Program based on the NEC-2015 standard (NEC-SE-DS, 2015).

*Table 14 Design parameters*

N	Name	Symbol	Value
1	Factor depending on the seismic zone	Z	0,4
2	Importance factor	I	1
3	Soil coefficient	Ct	0,0020
4	Maximum height	h	6
5	Vibration period	T	0,0113
6	Reduction coefficient	R	1,5
7	Coefficient of soil amplification	Sa	0,52
8	Displacement for the design in the rock	Fd	1,4
9	Non-linear behavior in soils	Fs	1,5

### ❖ Zone Factor (Z)

The NEC (NEC-SE-DS, 2015) divides the territory of Ecuador in six seismic zones, characterized by the zone Z factor according to the map shown in figure 36.

In the case at hand, the city of Quito is located in zone V, with a high seismic threat and the value of its Z factor is 0.40.

*Table 15 Ecuadorian provinces and value of Z-factor  
(NEC-SE-DS, 2015)*

Ecuadorian provinces and value of the Z factor				
POBLACIÓN	PARROQUIA	CANTÓN	PROVINCIA	Z
Yaruqui	Yaruqui	Quito	Pichincha	0,4

### ❖ Geotechnical Conditions Factors

According to (NEC-SE-DS, 2015), the soil is a type D, therefore:

- Fa - coefficient of soil amplification.

Table 3 (NEC-SE-DS, 2015) shows the values of the Fa coefficient that amplifies the ordinates of the elastic response spectrum of accelerations for rock design, taking into account the effects of site in our case we will use the value  $Fa = 1.2$ .

- $F_d$  - displacements coefficient for rock design.

Table 4 (NEC-SE-DS, 2015) shows the values of the  $F_d$  coefficient that amplifies the ordinates of the displacement response elastic spectrum for rock design, considering the effects of site in our case we will use the value  $F_d = 1.4$ .

- $F_s$  - coefficient considering non-linear behavior of soils

Table 5 (NEC-SE-DS, 2015) shows the values of the  $F_s$  coefficient, which consider the non-linear behavior of the soils, the degradation of the period of the site that depends on the intensity and frequency content of the seismic excitation and the relative displacements of the soil, for the spectra of accelerations and displacements, in our case we will use the value  $F_s = 1.5$ .



## 8 Analysis and design of the building using computational program

---

The analysis of structural masonry systems have to comply with the requirements imposed by the Ecuadorian Construction Standard NEC-2015 and must be done using rational methods based on principles accepted by good engineering practice such as awareness of national problems, worrying about knowing them to contribute with their professional and human activity to the solution of them, the commitment to the preservation and conservation of the environment and to eliminate pollution as much as possible and should reflect the characteristics and behaviour of this kind of structures (NEC-SE-DS, 2015).

There are computer programs that use complex methodologies such as finite element method that give an exact solution, but that due to the size and configuration restriction can be time-consuming compared to the ones presented below, which give generally acceptable results for design.

This chapter shows how the program ETABS (Extended Three Dimensional Analysis of Building System) Version 2016, makes the design of masonry walls with confinement, because in itself, this does not include this type of design, so it has been raised a methodology creating an equivalent model that allows analysing and structurally designing this type of material. The logical sequence that has been raised has been divided into three stages: modelling of the structure, analysis and design.

ETABS is a sophisticated, easy-to-use, purpose-built analysis and design program developed specifically for building systems. It has a powerful and intuitive graphical interface with unparalleled modelling, analysis and design procedures, all integrated using a common database. Although easy and simple for simple structures (Computers and Structures Inc., 2014).

It can also handle large and complex building models including a wide range of non-linear behaviours, making it the tool of choice for structural engineers in the construction industry, offering the most analysis and design tools available to the structural engineer working with building structures. The following list represents only a sample of the types of systems and analyses that ETABS can easily handle.

- Facilities in the modelling of commercial buildings of multiple floors
- Parking with linear and structural ramps
- Buildings with steel floor, concrete, composite or trusses
- It has a powerful design in structural steel and reinforced concrete, including shear walls, fully integrated, all available from the same interface used to model and analyse the model;
- The design of steel members allows initial pre-dimensioning and interactive optimization, and the design of concrete elements includes the calculation of the amount of reinforcing steel required, even considering earthquake-resistant condition
- Buildings based on reference line systems (Gris lines)
- Buildings of flat or reticular concrete slabs
- Buildings subject to any number of load cases and combinations, both lateral and vertical, including automatic wind and earthquake loads
- Multiple load cases by response spectra, with predetermined curves. Automatic transfer of vertical loads from floors to beams and walls.

## 8.1 Building design in ETABS

---

The structural analysis of the building was performed using computational software ETABS 2016.

The modeling is done in ETABS as follows (Kumar, 2015):

1. The structure is divided into distinct shell element. The shell element combines membrane and plate bending behavior, as shown in figure below. It has six degrees of freedoms in each corner point. It is a simple quadrilateral shell element which has size of 24 x 24 stiffness matrix.

2. Grid lines are made for the x, y and z coordinates and the wall is drawn from scratch.

3. Boundary conditions are assigned to the nodes wherever it is required. Boundary conditions are assigned at the bottom of the wall i.e., at ground level where restraints should be against all movements to imitate the behavior of shear wall.

4. The material properties are defined such as mass, weight, modulus of elasticity, Poisson's ratio, strength characteristics etc. The material properties used in the models are shown in table below.

5. The geometric properties of the elements are dimensions for the wall section.

6. Loads are assigned to the joints as they will be applied in the real structure.
7. The model should be ready to be analyzed forces, stresses and displacements.

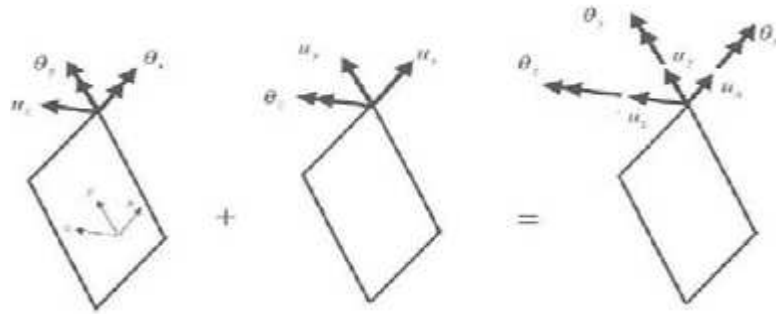


Figure 37 Shell element  
(Kumar, 2015)

In ETABS single walls are modeled as a pier/spandrel system, that is, the wall is divided into vertical piers and horizontal spandrels. This is a powerful mechanism to obtain design moments, shear forces and normal forces across a wall section. Appropriate meshing and labeling is the key to proper modeling and design. Loads are only transferred to the wall at the corner points of the area objects that make up the wall. Generally, the membrane or shell type element should be used to model walls. Here the shell type is used for modeling the wall element. There are three types of deformation that a single shell element can experience axial deformation, shear deformation and bending deformation as shown in figure below (Kumar, 2015):

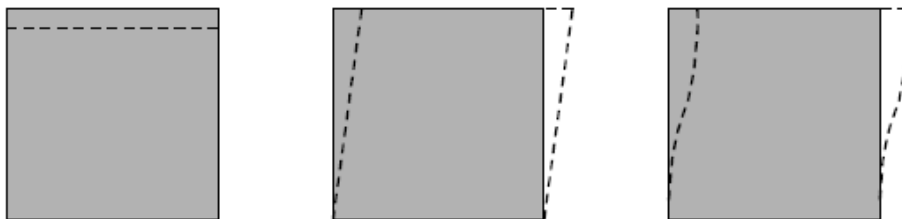
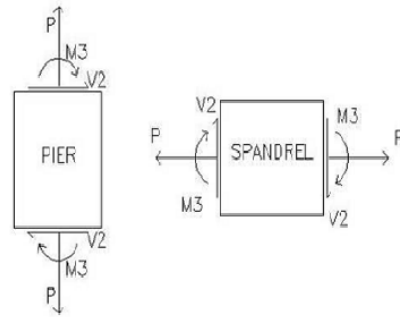


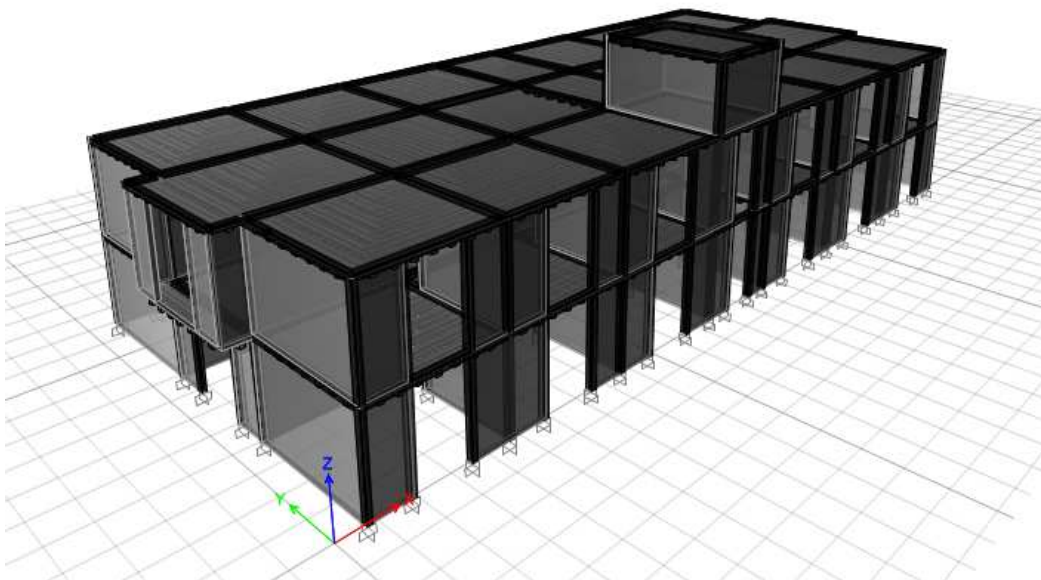
Figure 38 Deformation of shell element: a) axial deformation; b) shear deformation; c) bending (Etabs Shear walls design manual, 2010)

Wall pier forces are output at the top and bottom of wall pier elements and wall spandrel forces are output at the left and right ends of wall spandrel element, see Figure below:



*Figure 39 Pier and sprandel forces in ETABS  
(Etabs Shear walls design manual, 2010)*

On the figure below is presented the model of the building that I analyse in this report.



*Figure 40 Structure model in ETABS*

## 9 Design based on forces (DBF)

---

Static-linear and dynamic methods are both mandatory for all types of structures, with the exception of totally regular structures. The DBF is mandatory for all types of structures. The DBD is an alternative to DBF(NEC-SE-DS, 2015).

However, there are identified some of the problems associated with force-based design, such as:

- Force-based design relies on estimates of initial stiffness to determine the period and the distribution of design forces between different structural elements. Since the stiffness is dependent on the strength of the elements, this cannot be known until the design process is complete.

- Allocating seismic force between elements based on initial stiffness (even if accurately known) is illogical for many structures, because it incorrectly assumes that the different elements can be forced to yield simultaneously.

- Force-based design is based on the assumption that unique force-reduction factors (based on ductility capacity) are appropriate for a given structural type and material. This is demonstrably invalid.

Despite these criticisms it should be emphasized that current force-based seismic design, when combined with capacity design principles and careful detailing, generally produces safe and satisfactory designs. However, the degree of protection provided against damage under a given seismic intensity is very non-uniform from structure to structure. Thus, the concept of “uniform risk” which is implicit in the formulation of current seismic design intensity, has not been continued into the structural design(Powell, 2008).

### 9.1 General objectives and requirements

---

The structures must be designed to resist seismic forces coming from the combinations of the acting horizontal forces. The determination of the magnitude of the horizontal forces based on the conditions of the NEC 2015(NEC-SE-DS, 2015) to be applied to the structure, 100% of the dead load through a linear modal spectrum.

### **9.1.1 Design objectives**

---

For structures of normal occupation, the design objective is:

- Prevent damage to non-structural and structural elements, in the face of small and frequent earthquakes, which may occur during the life of the structure.
- Prevent serious structural damage to the bearing masonry walls and control non-structural damage, in the face of moderate and infrequent earthquakes, which may occur during the useful life of the structure.
- Avoid collapse in the face of severe earthquakes that may occur rarely during the life of the structure, trying to safeguard the life of its occupants.
- The structure must have the capacity to withstand the forces specified by this standard.
- The structure must be able to dissipate inelastic deformation energy, making use of the design techniques by capacity or by means of the use of seismic control devices (NEC-SE-MP, 2015).

### **9.1.2 Requirements RDBF: internal forces**

---

The RDBF resistance requirement involves the calculation of the internal forces acting on each structural element. These translate the mechanical requests To meet this requirement, the total results of the analysis should include(NEC-SE-DS, 2015):

- Deflections
- Drifts
- Forces at the floors, and in the elements
- Floor moments
- Shear at the base

### **9.1.3 DDBF requirements: floor drifts**

---

Likewise, the deformations generated by the seismic forces will be calculated according to the type of material and structure, in accordance with the NEC 2011-SE-HM, NEC-SE-AC, NEC-SE-MP and NEC-SE-MD standards. This drift according to the norm should not be greater than 0.01(NEC-SE-MP, 2015).

## 9.1.4 Weight of the building for seismic calculations (W)

The seismic load  $W$  represents the earthquake reactive load. Regardless of the method of analysis described in section 6.2(NEC-SE-DS, 2015), the following reactive seismic load  $W$  will be used.

The analysis is carried out with the program ETABS and the seismic loads are calculated:

Table 16 Mass source

MassFrom	Lat Only	Lump at stories	Load	Multiplier	SortID
Loads	Yes	Yes	Dead Load	1	1

Table 17 Centers of mass and rigidity

Story	Diphragm	Mass X kg	Mass Y kg	XCM m	YCM m	Cumulative X kg	Cumulative Y kg
Story3	D3	9527,04	9527,04	14,2	1,998	9527,04	9527,04
Story2	D2	191213,52	191213,52	14,1919	5,929	191213,52	191213,52
Story1	D1	257248,32	257248,32	14,2377	6,1287	257248,32	257248,32

Table 18 Weight of the structure

Story	h (m)	Mass (t)	Weight (kN)	W·h (kN·m)
Story3	7,8	9,53	93,36	522,84
Story2	5,6	191,21	1873,89	5246,90
Story1	2,8	257,25	2521,03	0,00
Summary		457,99	4488,29	5769,74

## 9.2 Methods of the analysis for DBF

The DBF will present the same general methodology for the 3 methods of analysis that (NEC-SE-DS, 2015) allows:

- Static (1)
- Dynamic analyzes
  - Spectral dynamic analysis (2)

- Dynamic analysis step by step in time (3)

Consequently, the accelerated design spectrum  $S_a(T)$  will be determined from the PGA (maximum seismic acceleration in the terrain). Static linear and pseudo-dynamic methods are both mandatory for all types of structures, with the exception of totally regular structures.

### 9.2.1 Static procedure

---

Application conditions: the static method for all types of structures will be applied as a minimum. The Z factor defined in section 3.1.1(NEC-SE-DS, 2015) will be used to define the PGA, and hence the accelerating spectrum  $S_a(T)$ .

### 9.2.2 Dynamic procedures for calculating seismic forces

---

#### a) Processes and application conditions

These methods will be applied to all types of structures (with the exception of totally regular structures).

The designer can use the following 2 methods:

- Spectral analysis: the seismic spectrum of elastic response in accelerations described in section 3.3.1 (NEC-SE-DS, 2015) shall be used, or the spectrum shall be constructed by the hazard curves defined in section 3.1.2. This method will be mandatory for all types of irregular structures.

- Step-by-step analysis over time: accelerograms will be used for the specific site that are representative of the real earthquakes expected in the area, complying with the requirements established in sections 3.2 and 10.5.4. This method will be mandatory for structures with energy dissipating systems (see section 8 (NEC-SE-DS, 2015))

#### b) Adjustment of the basal cut of the results obtained by the dynamic analysis

The value of the total dynamic shear in the base obtained by any dynamic analysis method should not be:

- <80% of the basal shear  $V$  obtained by the static method (regular structures)
- <85% of the basal shear  $V$  obtained by the static method (irregular structures).

### 9.2.3 Representation and determination of the horizontal seismic load

---

The basis of the analysis will constitute an appropriate representation of the seismic action, in accordance with the principles of structural dynamics.

The seismic action determined by a dynamic procedure must represent, at least, the design earthquake (return period of 475 years), without the application of the response reduction factor  $R$ , and may be one of the following:

- The elastic response spectrum of accelerations, expressed as a fraction of the acceleration of gravity;
- An elastic response spectrum of accelerations obtained for a specific site, based on geology, tectonics, seismology and local soil characteristics, fulfilling the requirements of sections 10.5.3 and 10.5.4(NEC-SE-DS, 2015). The spectrum must be developed for a fraction of the damping with respect to the critical 0.05, unless the use of other values is consistent with the expected structural behavior and the intensity of the earthquake established for the site;
- Accelerograms developed for the specific site that are representative of the actual earthquakes expected in the area, meeting the requirements of section 10.5.4(NEC-SE-DS, 2015).

### 9.2.4 Procedure 1: spectral analysis

---

The seismic spectrum of elastic response in accelerations described in section 3.3.1 shall be used or the spectrum shall be constructed by seismic hazard curves in section 3.1(NEC-SE-DS, 2015).

The following information regarding the number of modes should be considered in the analysis:

- all modes of vibration that contribute significantly to the total response of the structure, through the various periods of vibration;
- all modes that involve the participation of an accumulated modal mass of at least 90% of the total mass of the structure, in each of the main horizontal directions considered.

*Reduction of dynamic elastic response forces for design*

Condition of application: in no case the parameters of elastic response will be reduced to values such that the basal shear of reduced design is smaller than the basal shear of elastic response divided by R. The forces obtained by dynamic elastic analysis can be reduced for the purposes of Design under this condition.

In a similar way as in what is stated in section 6.3(NEC-SE-DS, 2015)., the dynamic forces obtained should also be modified by the following factors:

- I (importance factor);
- $\emptyset E_i$  and  $\emptyset P_i$  (structural configuration factors in plant and elevation).

## 9.3 DBF calculation procedure

### 9.3.1 Steps of the method

---

The steps to follow are those.

- Determination of the design spectrum  $S_a(T)$  in accordance with the geotectonic characteristics of the site of the structure (see section 3.3),
- Estimated approximation of the fundamental vibration period  $T_a$ ,
- Determination of the base shear  $V$  with the results of the previous steps,
- Determination of the vertical and horizontal distributions of  $V$ ,
- Direction of application of these seismic forces and verification that the drift rates do not exceed the allowed value.

### 9.3.2 Horizontal distribution of shear

---

The floor shear  $V$  must be distributed among the different elements of the system resistant to lateral loads in proportion to their stiffness, considering the stiffness of the floor. In a flexible floor system, the distribution of the floor shear to the elements of the resistant system will be made taking into account that condition, both for the static calculation and for the dynamic calculation.

The mass of each level should be considered as concentrated in the center of mass of the floor, but displaced from a distance equal to 5% of the maximum dimension of the building on that floor, perpendicular to the direction of application of the lateral forces

under consideration, in order to take into account, the possible effects of accidental torsion, both for regular structures and for irregular structures. The effect of this displacement must be included in the distribution of the floor shear and in the torsional moments (NEC-SE-MP, 2015).

The basal floor shear will be calculated with the following expression:

$$V_{base} = \frac{I \cdot S_a(T_a) \cdot W}{R \cdot \varphi_p \cdot \varphi_E} \quad (10)$$

Where:

$S_a(T_a)$  - acceleration design spectrum;

$\varphi_p$  and  $\varphi_E$  - coefficients of the irregularity in plan and elevation configuration;

I - coefficient of importance;

R - seismic resistance reduction factor;

V - total design basal shear;

W - reactive seismic load;

$T_a$  - vibration period; see in section (NEC-SE-DS, 2015).

### 9.3.3 Elastic spectrum horizontal accelerations design

---

The elastic response spectrum of accelerations  $S_a$ , expressed as a fraction of the acceleration of gravity, for the level of the design earthquake, is given in Figure 3, consistent with:

- the seismic zone factor Z,
- the type of soil at the site of the structure,
- the consideration of the values of the soil amplification coefficients  $F_a$ ,  $F_d$ ,  $F_s$ .

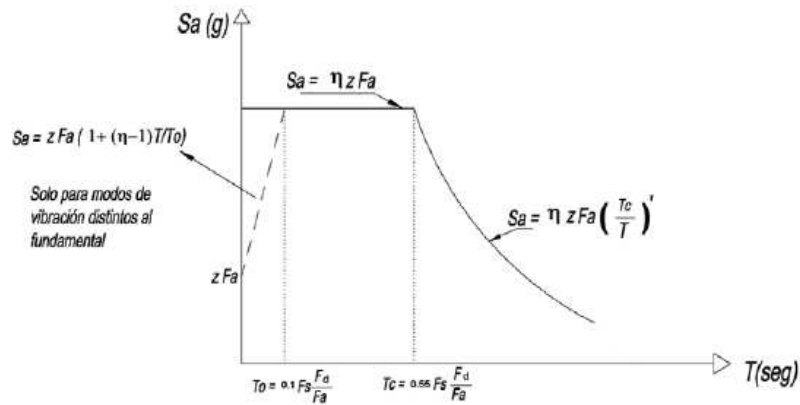


Figure 41 Elastic seismic spectrum of accelerations that represents the design earthquake (NEC-SE-DS, 2015)

Where:

$\eta$  - ratio between the spectral acceleration  $S_a$  ( $T = 0.1$  s) and the PGA for the selected return period. For the provinces of the Sierra, Esmeraldas and Galápagos:

$$\eta = 2.48$$

$F_a$  - coefficient of soil amplification in the zone of cut period. Amplify the ordinates of the spectrum Elastic acceleration response for rock design, considering the effects of site:

$$F_a = 1.2$$

$F_d$  - coefficient of soil amplification. Amplify the ordinates of the elastic response spectrum of displacements for rock design, considering the effects of the site:

$$F_d = 1.4$$

$F_s$  - coefficient of soil amplification. Consider the nonlinear behavior of soils, the degradation of the period of the site that depends on the intensity and frequency content of the excitation seismic and the relative displacements of the ground, for acceleration and displacement spectra:

$$F_s = 1.5$$

$S_a$  - acceleration elastic response spectrum (expressed as a fraction of the acceleration of gravity  $g$ ). Depends on the period or mode of vibration of the structure:

$$S_a = \eta \cdot Z \cdot F_a \text{ for } 0 \leq T \leq T_c \tag{11}$$

$$S_a = \eta \cdot Z \cdot F_a \cdot \left(\frac{T_c}{T}\right)^r \text{ for } T > T_c \tag{12}$$

r - factor used in the elastic design spectrum, whose values depend on the geographical location of the project. For all soils except soil type E:

$$r = 1$$

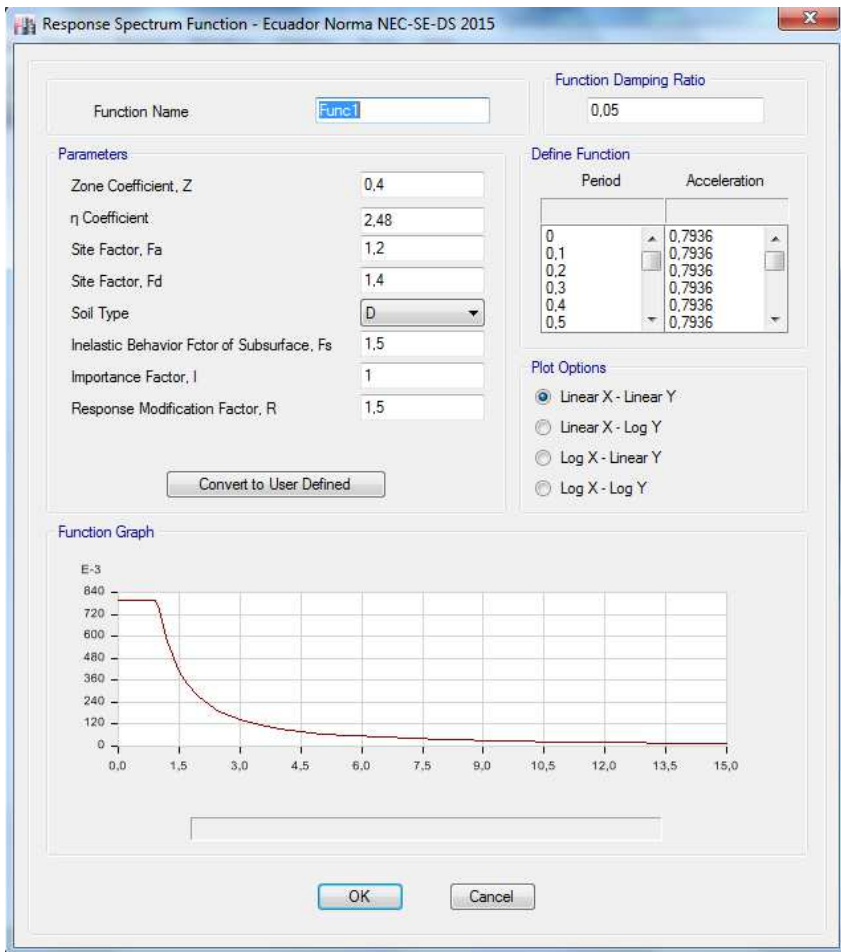
T - fundamental period of vibration of the structure;

$T_c$  - limit period of vibration in the elastic seismic spectrum of accelerations that represents the earthquake of design

$$T_c = 0.55 \cdot F_s \cdot \frac{F_d}{F_a} = 0.55 \cdot 1.5 \cdot \frac{1.4}{1.2} = 0.963 \quad (13)$$

Z - maximum acceleration in rock expected for the design earthquake, expressed as fraction of acceleration of gravity g:

$$Z = 0.4$$



Period (s)	Acceleration (m/s <sup>2</sup> )
0	0,7936
0,1	0,7936
0,2	0,7936
0,3	0,7936
0,4	0,7936
0,5	0,7936
0,6	0,7936
0,7	0,7936
0,8	0,7936
0,9	0,7936
1	0,7494
1,2	0,5701
1,5	0,4079
1,7	0,3381
2	0,2649
2,5	0,1896
3	0,1442
3,5	0,1144
4	0,0937
5	0,067
8	0,0331
11	0,0205
15	0,0129

Figure 42 Response spectrum function (Etabs)

The method that was used in ETABS to estimate a peak value of displacement or force within a structure is the Complete Quadratic Combination, CQC (see in Figure 43).

It is based on random vibration theories and has found wide acceptance by most engineers and has been incorporated as an option in most modern computer programs for seismic analysis.

The peak value of a typical force can now be estimated, from the maximum modal values, by the CQC method with the application of the following double summation equation:

$$F = \sqrt{\sum_n \sum_m f_n \cdot \rho_{nm} \cdot f_m} \quad (14)$$

Where  $f_n$  is the modal force associated with mode n. The double summation is conducted over all modes. Similar equations can be applied to node displacements, relative displacements and base shears and overturning moments.

The cross-modal coefficients,  $\rho_{nm}$ , for the CQC method with constant damping are:

$$\rho_{nm} = \frac{8\zeta^2(1+r)r^{3/2}}{(1-r^2)^2 + 4\zeta^2r(1+r)^2} \quad (15)$$

Where  $r = \omega_n/\omega_m$  and must be equal to or less than 1.0. It is important to note that the cross-modal coefficient array is symmetric and all terms are positive (Wilson, 1981).

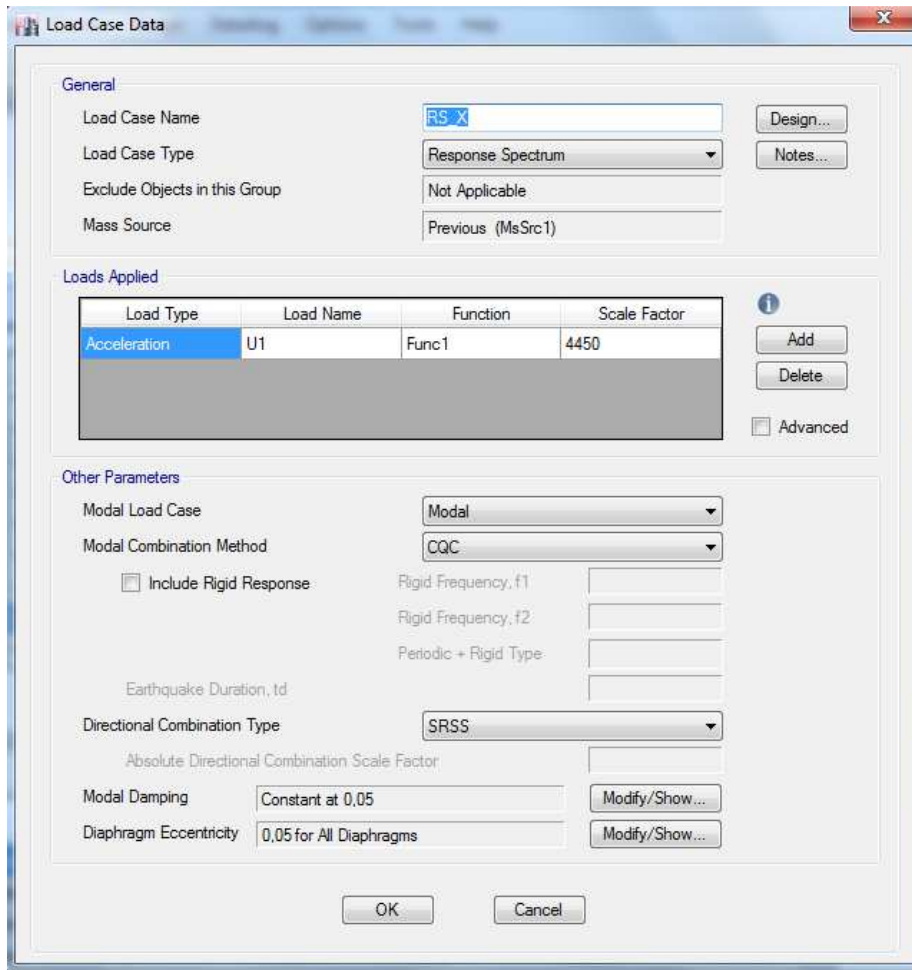


Figure 43 Response spectrum load case (Etabs)

On the Tables 19-20 are presented results for modal analysis and in the Figures 44-46 three first main mode shapes.

Table 19 Modal periods and frequencies (Etabs)

Case	Mode	Period	Frequency	Circular Frequency	Eigenvalue
		sec	cyc/sec	rad/sec	rad <sup>2</sup> /sec <sup>2</sup>
Modal	1	0,088	11,39	71,59	5125,50
Modal	2	0,074	13,54	85,08	7239,22
Modal	3	0,071	14,08	88,44	7822,04
Modal	4	0,033	29,93	188,05	35364,10
Modal	5	0,03	33,29	209,19	43761,28
Modal	6	0,029	34,04	213,86	45736,64
Modal	7	0,027	36,69	230,51	53135,40
Modal	8	0,026	38,68	243,03	59063,13

Table 20 Modal participating mass ratios (ETABS)

Case	Mode	Period	UX	UY	UZ	Sum UX	Sum UY	Sum UZ
		sec						
Modal	1	0,088	0,8978	0,0001	0	0,8978	0,0001	0
Modal	2	0,074	0,0001	0,903	0	0,8979	0,903	0
Modal	3	0,071	1,61E-05	0,0002	0	0,8979	0,9032	0
Modal	4	0,033	0,0446	5,63E-07	0	0,9425	0,9032	0
Modal	5	0,03	0,0566	0,0002	0	0,9991	0,9034	0
Modal	6	0,029	0,0001	0,0774	0	0,9992	0,9808	0
Modal	7	0,027	0,0008	2,83E-05	0	1	0,9809	0
Modal	8	0,026	0	0,0003	0	1	0,9812	0

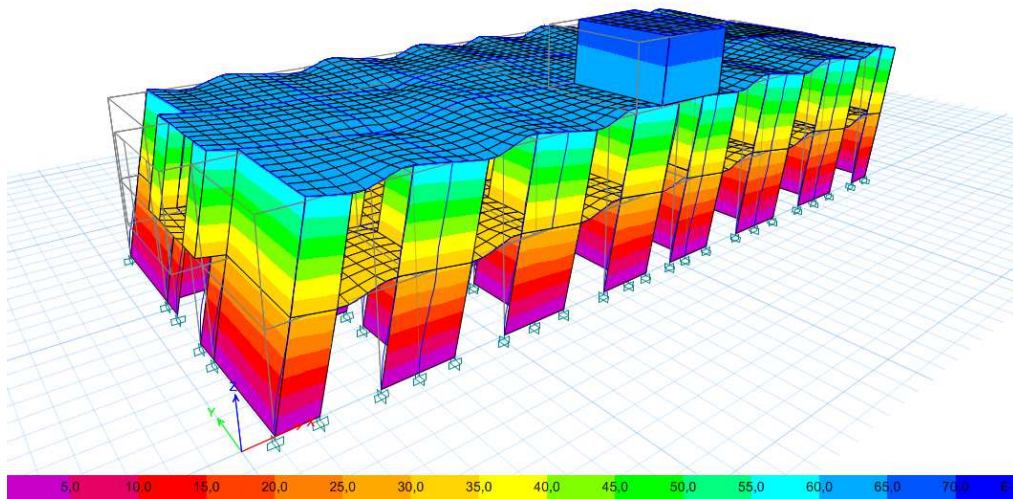


Figure 44 Mode 1.  $T=0.088$  (Etabs)

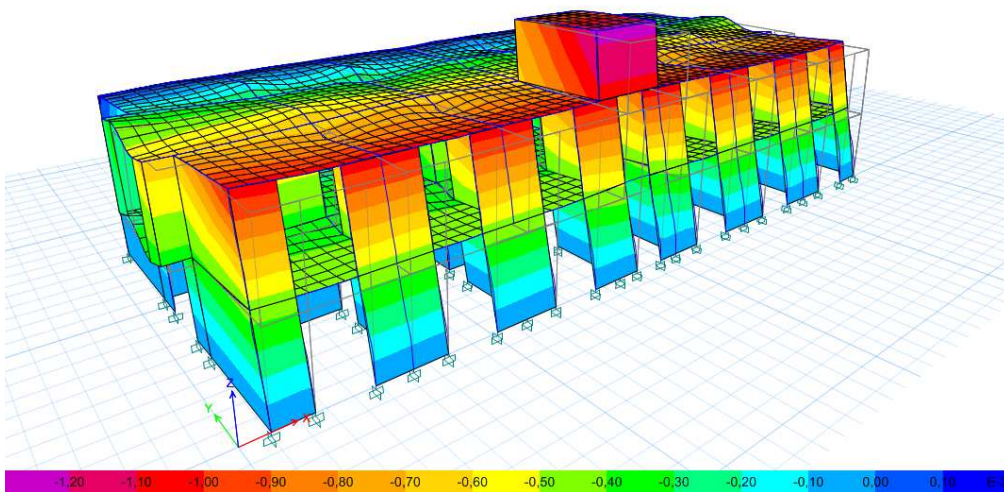


Figure 45 Mode 2.  $T=0.074$  (Etabs)

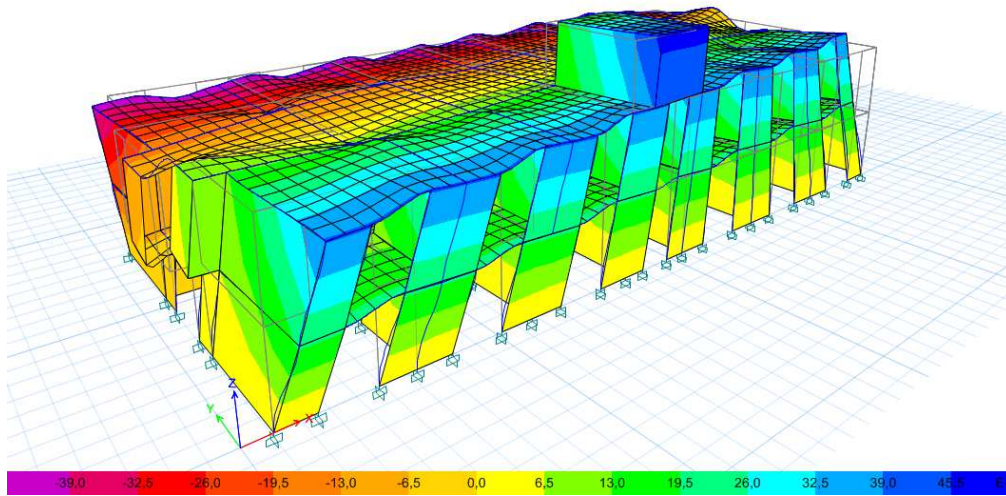


Figure 46 Mode 3.  $T=0.071$  (Etabs)

### 9.3.4 Determination of the vibration period T

The approximate vibration period of structure T, for each principal direction, will be estimated from one of the 2 methods described in the code (NEC-SE-DS, 2015).

The value of T obtained when using these methods is a reasonable initial estimate of the structural period that allows the calculation of the seismic forces to be applied to the structure and perform its sizing.

For building structures, the value of T can be determined roughly by the expression:

$$T = C_t \cdot h_n^\alpha \quad (16)$$

Where:

$C_t$  - coefficient that depends on the type of building;

$h_n^\alpha$  - maximum height of the building of n floors, measured from the base of the structure, in meters.

For structures with structural walls of reinforced concrete or structural masonry (with  $\alpha = 1$ ):

$$C_t = \frac{0.0062}{\sqrt{C_w}}; C_w = \frac{100}{A_B} \sum_{i=1}^{n_w} \left[ \left( \frac{h_n}{h_{wi}} \right)^2 \cdot \frac{A_{wi}}{1+0.83 \left( \frac{h_{wi}}{l_{wi}} \right)^2} \right] \quad (17)$$

Where:

$A_B$  - area of the building at its base, in square meters.

$n_w$  - number of building walls designed to withstand seismic forces in the study direction.

$h_{wi}$  - height of the wall i measured from the base, in meters.

$A_{wi}$  - minimum shear area of the section of a structural wall i, measured in a horizontal plane, at the first level of the structure and in the study direction, in square meters.

$l_{wi}$  - length measured horizontally, in meters, of a structural wall i at the first level of the structure and in the study direction.

Table 21 Calculation of the coefficients  $C_t$  and  $C_w$

Direction Y		
A <sub>wi</sub>	L <sub>wi</sub>	$\left(\frac{h_n}{h_{wi}}\right)^2 \frac{A_{wi}}{1+0.83\left(\frac{h_{wi}}{l_{wi}}\right)^2}$
0,84	4,2	2,45
0,84	4,2	2,45
0,42	3	0,98
0,42	3	0,98
0,42	3	0,98
0,42	3	0,98
0,42	3	0,98
0,56	4	1,59
0,42	3	0,98
0,6	3	1,39
0,6	3	1,39
0,8	4	2,27
0,6	3	1,39
0,6	3	1,39
0,34	1,7	0,42
0,3	1,5	0,31
0,42	3	0,98
0,56	4	1,59
0,42	3	0,98
0,42	3	0,98
0,42	3	0,98
0,42	3	0,98
0,8	4	2,27
0,8	4	2,27
Sum		30,97

Direction X		
A <sub>wi</sub>	L <sub>wi</sub>	$\left(\frac{h_n}{h_{wi}}\right)^2 \frac{A_{wi}}{1+0.83\left(\frac{h_{wi}}{l_{wi}}\right)^2}$
0,189	1,35	0,17
0,287	2,5	0,56
0,287	2,5	0,56
0,301	2,15	0,50
0,301	2,15	0,50
0,287	2,5	0,56
0,287	2,5	0,56
0,189	1,35	0,17
0,84	4,2	2,45
0,84	4,2	2,45
0,84	4,2	2,45
0,84	4,2	2,45
1,44	7,2	5,12
0,6	3	1,39
1,44	7,2	5,12
0,189	1,35	0,17
0,35	2,5	0,69
0,287	2,05	0,45
0,238	1,7	0,29
0,238	1,7	0,29
0,287	2,05	0,45
0,35	2,5	0,69
0,189	1,35	0,17
Sum		28,22

Direction Y	
Cw	9,4010
Ct	0,0020
$\alpha$	1,00
Ab	329,40m2
nw	23
hwi	2,8
hn	5,6
Ty	0,0113

Direction X	
Cw	8,5659
Ct	0,0021
$\alpha$	1,00
Ab	329,40m2
nw	23
hwi	2,8
hn	5,6
Tx	0,0119

### 9.3.5 Ductility and seismic resistance reduction factor R

---

#### a) Recommendations and applicability

For the definition of the resistance reduction factor R, refer to the recommendations of the codes ASCE-7-10(ASCE, 2010) and NSR-10 (see section 1.4.2(NSR-10, 1997)).

Conceptually, they are also considered:

- criteria related to grouping aspects of structuring, differences between constructive realities and quality between materials and construction,
- penalties directed towards certain types of structures that do not allow for adequate global ductility to withstand the inelastic deformations required by the design earthquake.

#### b) Determining of the R factor

The engineer must select the structural system that will be used for the construction, from 2 groups, the ductile structural systems and the structural systems of limited ductility, which are described in Table 1(NEC-SE-DS, 2015), together with the value of reduction of seismic resistance R correspondent. The lowest of the R values should be taken for the cases in which the structural resistant system results in a combination of several systems such as those described in Table 16(NEC-SE-DS, 2015).

The R factor allows a reduction of the seismic design forces, which is allowed provided that the structures and their connections are designed to develop a foreseeable failure mechanism with adequate ductility, where the damage is concentrated in specially detailed sections to function as plastic ball joints (NEC-SE-DS, 2015)

In the case of the DBF method, it will be considered as a constant parameter dependent only on the structural typology. (NEC-SE-DS, 2015)

Criteria for definition of R

The resistance reduction factors R really depend on some variables, such as:

- type of structure,
- type of soil,
- considered vibration period
- ductility factors, over resistance, redundancy and damping of a structure under boundary conditions.

For the structural load-bearing walls of the confined masonry buildings: R=3

### 9.3.6 CALCULATION OF DRIFT

---

A deformation control has to be made through the calculation of the maximum inelastic floor drifts. It is necessary to check that the structure will present controllable inelastic deformations, substantially improving the conceptual design. The maximum values have been established considering that the calculator will use cracked sections (section 6 .1.6-b (NEC-SE-DS, 2015)). However, for walls with height / length ratio less than 1.5, it is not necessary to use cracked inertia values

For the revision of the floor drifts, the value of the maximum inelastic response in displacements  $\Delta_M$  of the structure, caused by the design earthquake, will be used.

The drifts obtained as a consequence of the application of the design lateral forces reduced by the DBF method, whether static or dynamic, for each direction of application of the lateral forces, will be calculated, for each floor, by performing an elastic analysis of the structure subjected to the lateral forces calculated (NEC-SE-DS, 2015).

Limits of the drift: the maximum inelastic drift  $\Delta_M$  of each floor must be calculated by:

$$\Delta_M = 0.75 \cdot R \cdot \Delta_E \quad (18)$$

Where:

$\Delta_M$  - inelastic maximum drift;

$\Delta_E$  - displacement obtained in application of reduced design lateral forces;

R - resistance reduction factor (see section 6.3.4 (NEC-SE-DS, 2015)).

$\Delta_M$  cannot exceed the values established in Table 7(NEC-SE-DS, 2015), which must be satisfied in all columns of the building.

It will be verified that:

$$\Delta_M < \Delta_M \text{maxima} \quad (19)$$

Where

$\Delta_M \text{maxima}$  - is determined according to section 4.2.2 (NEC-SE-DS, 2015).

The maximum drift for any floor will not exceed the inelastic drift limits established in the following table, in which the maximum drift is expressed as a percentage of the floor height:

Table 22 Maximum  $\Delta_M$  values, expressed as a fraction of the floor height (NEC-SE-DS, 2015)

Type of structure	$\Delta_m$ maximum (without units)
Reinforced concrete, steel and wood structures	0,02
Masonry	0,01

Table 23 Maximum  $\Delta_M$  values for the project obtained in Etabs

Story	Load case/Combo	Label	Coordinates (m)			Drift X max	Drift Y max	$\Delta X$	$\Delta Y$
			x	y	z				
3	Seismic X-e	20	15,6	0	7,8	8,50E-05		0,000191	
	Comb11	18	12,8	4	7,8		0,000116		0,000261
2	Seismic X+e	87	9,95	12	5,6	0,000189		0,000425	
	Seismic Y-e	91	0	7	5,6		0,000157		0,000353
1	Seismic X-e	64	26,45	0	2,8	1,96E-04		0,000441	
	Seismic Y+e	92	27,6	7	2,8		0,000166		0,000374
							Max drift	0,000441	0,000374
								<0,01	
								Ok!	Ok!



# 10 Detailed design of seismic analysis of confined masonry

## 10.1 Scope

---

Masonry structures of confined walls of more than 2 stories, or those that do not meet the conditions for the application of the simplified method, must be analyzed and designed based on a rational analysis method according to the requirements of data in this section:

## 10.2 Values of the resistance reduction factor

---

The following values should be used:

*Table 24 Resistance reduction factor  
(NEC-SE-Vivienda, 2015)*

Type of load	Resistance reduction factors $\phi$
Axial compression load, with or without bending	0,7
Axial tensile load	0,9
Flexion without axial load	0,9
Shear	0,6

## 10.3 Design hypothesis

---

The following assumptions in the design of confined masonry walls should be considered:

- It should be considered in the case of confined wall masonry, that the wall is a homogeneous element that includes the masonry portion and the confinement elements.
- For the purposes of applying the effective dimensional characteristics indicated in section 7.5.3, it should be considered that the confinement elements are equivalent to cells injected with filling mortar. (NEC-SE-Vivienda, 2015).

## 10.4 Design for compression axial load

---

Brick masonry is a material that exhibits optimal performance when subjected to compression loads. Their behavior and failure modes at axial loads, depend on the interaction of parts and mortar; this can be interpreted in the following way: the pieces and the mortar have different stress-deformation characteristics. Therefore, when subjected to the same effort, there is an interaction between both, which is that the less deformable material, the pieces in general, restricts the transverse deformations of the most deformable material (the mortar), introduces compression efforts of transversal direction. On the other hand, in the least deformable, transverse tensile stresses are introduced, and their resistance with respect to that obtained in the compression test if isolated material (Gutiérrez, 2004).

The wall, overall, must be verified for the axial loads to understand, according to what is indicated in section 7.5.4. The reinforcing area to be used there corresponds to that of the longitudinal steel of the confinement columns (NSR-10, 1997).

When the design procedures require that the axial resistances of the confinement elements be verified, the following resistances can be used:

- ✓ Nominal resistance to axial compression,  $P_{nc}$

$$P_{nc} = 0.80 \cdot [0.85 \cdot f'_c \cdot (A_{ci} - A_{st}) + f_y \cdot A_{st}] \quad (20)$$

$$P_{nc} = 0.80 \cdot [0.85 \cdot 21 \cdot (200 \cdot 200 - 452) + 420 \cdot 452] = 716.62 \text{ (kN)}$$

Where

$A_{ci}$  - section area of the confinement column  $i$  ( $\text{mm}^2$ );

$A_{st}$  - total area of reinforcing steel in the wall section, or total area of the reinforcing steel longitudinal of the confinement element ( $\text{mm}^2$ );

$f_y$  - resistance to yield of reinforcing steel (MPa);

$f'_c$  - specified resistance to concrete compression (MPa).

- ✓ Nominal axial tensile strength,  $P_{nt}$

$$P_{nt} = -f_y \cdot A_{st} = -420 \cdot 452 = -189 \text{ (kN)} \quad (21)$$

Where:

$P_{nt}$  - nominal axial tensile strength (N);

$f_y$  - resistance to yield of reinforcing steel (MPa);

$A_{st}$  - total area of reinforcing steel in the wall section, or total area of the longitudinal reinforcing steel of the confinement element (mm<sup>2</sup>).

The detailed calculation is presented in the Appendix B Table 31

## 10.5 Design of the wall in the direction perpendicular to its plane

---

The requirements of this section are used for the design by the limit state method of resistance, of confined masonry walls for the effect of the horizontal loads perpendicular to the wall plane, in addition to the vertical forces acting on the wall.

### *a. Flexo-compression resistance*

The resistance to bending of the wall produced by horizontal forces perpendicular to its own plane, must be evaluated based on the following requirements:

- Flexo-compression resistance is only contributed by the confinement columns.
- As effective width,  $b$ , only that of the confinement columns, measured in the direction of the wall, should be taken.
- The design is made in its entirety in accordance with the reinforced concrete requirements of the ACI 318.
- The axial load,  $P_U$ , on the confinement element should be considered twice as much as that obtained proportionally to the areas of masonry and confinement columns, unless a more detailed analysis is made, taking into account the modular relationships and the position of the charges that induce it.

### *b. Shear strength*

The wall shear strength produced by horizontal forces perpendicular to its own plane shall be evaluated based on the requirements of D.5.7.4(NSR-10, 1997).

The detailed calculation is presented in the Appendix B Table 31, 36

## 10.6 Flexo-compression design of the wall in the direction parallel to its plane

---

The requirements of this section are used for the design of flexo-compression by the limit state method of resistance of confined masonry walls, for the effect of horizontal

loads parallel to the wall plan, in addition to the vertical forces acting on it. The design can be done by one of the two procedures given below.

## 10.6.1 Flexo-compression resistance neglecting the contribution of masonry

---

In this procedure, the contribution of masonry to the flexural strength of the wall is neglected.

The following conditions must be met in each of the confinement columns of the wall:

$$P_{uc} \leq \varphi P_{nc}; P_{ut} \leq \varphi P_{nt} \quad (22)$$

Where:

$P_u$ - axial load;

$P_{uc}$ - maximum compressive axial forces requested;

$P_{ut}$ - maximum requested axial tensile forces;

$P_{nc}$  - nominal resistance to axial compression;

$P_{nt}$  - nominal tensile strength;

$\varphi$ - resistance reduction factor.

When dealing with a confined wall that only has two confining columns equal in their edges, the equations are simplified to:

$$P_{uc} = \frac{P_u}{2} + \Delta P_u \quad (23)$$

$$P_{ut} = \frac{P_u}{2} - \Delta P_u \leq 0 \quad (24)$$

Where:

$P_u$  and  $\Delta P_u$  are always positive and  $\Delta P_u$  is obtained by the following equation:

$$\Delta P_u = \frac{M}{l_w} \quad (25)$$

M - acting moment that occurs simultaneously with V;

V - acting shear force occurring simultaneously with M;

$l_w$  - horizontal length of the wall (mm), or total horizontal length of the wall, measured center to center between edge confinement columns.

The maximum requested axial forces,  $P_{uc}$  compression and  $P_{ut}$  tensile force, must be calculated on each of the confinement columns, by means of the following equations

$$P_{uc} = \frac{A_{ci}}{A_{ct}} \cdot P_u + \Delta P_{ui}; P_{ut} = \frac{A_{ci}}{A_{ct}} \cdot P_u - \Delta P_{ui} \quad (26)$$

Where:

$A_{ci}$  - section area of the confinement column i (mm<sup>2</sup>);

$A_{ct}$  - total area of the wall confinement columns (mm<sup>2</sup>);

$P_u$  - axial load;

$P_{uc}$  - maximum compressive axial forces requested;

$P_{ut}$  - maximum requested axial tensile forces;

$\Delta P_{ui}$  - is obtained by means of the following equation:

$$\Delta P_{ui} = \left| \frac{M_u \cdot A_{ci} \cdot (x_i - \bar{x})}{I_{ct}} \right|; A_{ct} = \sum_i A_{ci} \quad (27)$$

$I_{ct}$  - moment of inertia of the columns of confinement of the wall, with respect to its centroid (mm<sup>4</sup>);

$x_i$  - distance from the confinement column i to the edge of the wall (mm);

$\bar{x}$  - distance to the edge of the centroid wall from the areas of all the wall confinement columns (mm):

$$\bar{x} = \frac{\sum_i A_{ci} \cdot x_i}{A_{ct}}; I_{ct} = \sum_i A_{ci} \cdot (x_i - \bar{x})^2 \quad (28)$$

$P_u$  and  $\Delta P_{ui}$  are always positive.

The detailed calculation is presented in the Appendix B Table 31

## 10.6.2 Resistance to flexo-compression taking into account the contribution of the masonry

---

The requested design moment  $M_u$  accompanying the axial load  $P_u$  must meet the condition given by the following equation, for the load level  $P_u$ .

$$M_u \leq \varphi M_n \quad (29)$$

Where:

$M_n$  - nominal resistance to bending;

$M_u$  - requested major moment of wall design.

$M_n$  - is obtained taking into account the interaction between moment and axial load, which allow to calculate a diagram of interaction of the wall, using the coefficient of reduction of resistance  $\varnothing$

The detailed calculation is presented in the Appendix B Table 36

## 10.7 Design shear wall in the direction parallel to the plane

---

For the design of masonry structures, subjected to induced lateral loads by wind or by seismic movements, it is necessary to properly evaluate the shear strength. Generally, shear efforts are combined with the compression efforts produced by gravitational loads or other actions. Consequently, the pure case of shear has no practical application and, therefore, the shear strength of the masonry is investigated considering the effect of the compression efforts in the normal direction to horizontal joints. In other cases, the state of stress in the masonry is more complex (biaxial stresses) and it is necessary to use a general failure criterion to evaluate the resistance of the masonry.

In the masonry of confined walls all the shear force on the wall must be taken by the masonry and it is assumed that there is no contribution to the shear strength on the part of the confinement elements. The requested design resistance,  $V_u$ , must meet the following condition:

$$V_u \leq \varphi V_n \quad (30)$$

Where:

$V_n$  - nominal resistant wall shear strength (N);

$V_u$  - major shear strength requested for wall design (N).

The normal resistance to shear by diagonal traction, is obtained from:

$$V_n = \left( \frac{1}{12} \cdot \sqrt{f'_m} + \frac{P_u}{3 \cdot A_e} \right) \cdot A_{mv} \leq \frac{1}{6} \sqrt{f'_m} \cdot A_{mv} \quad (31)$$

Where:

$A_e$  - effective area of the masonry section (mm<sup>2</sup>);

$A_{mv}$  - effective area to determine shear stress (mm<sup>2</sup>);

$f'_m$  - specified resistance to masonry compression (MPa);

$P_u$  - axial load (N);

$V_n$  - nominal resistant wall shear strength (N).

In this case, it is the maximum axial load acting simultaneously with the maximum shear force requested.

The detailed calculation is presented in the Appendix B Table 32

## 10.8 Verification by crushing the wall's soul

---

It must be verified that the wall cloth framed by the beams and columns of confinement, does not fail by crushing.

For this purpose, a compression rod is considered on the diagonal of the wall, which has an effective width equal to 1/5 of the length of the diagonal.

The condition must be met:

$$P_{ud} \leq \varphi P_{nd} \quad (32)$$

Where:

$P_{nd}$ - nominal compressive strength of masonry;

$P_{ud}$  - axial force acting on the diagonal connecting rod of the wall (N);

$\varphi$  - resistance reduction factor.

The axial force acting on the diagonal,  $P_{ud}$ , is obtained by:

$$P_{ud} = \frac{h'}{l_w} \cdot V_u \quad (33)$$

Where:

$h'$  - length of the diagonal of the wall cloth between confining elements;

$l_w$  - total length of the wall over which the horizontal shear of the requested design acts  $V_u$ ;

$P_{ud}$  - axial force acting on the diagonal connecting rod of the wall (N);

$V_u$  - Major shear strength requested for wall design (N).

The nominal resistance to crushing is obtained by means of the equation of section 7.5.3 (NEC-SE-Vivienda, 2015), where a length for evaluation of buckling  $h'$  must be used, equal to the dimension of the diagonal of the wall in the cloth under study and an effective thickness for buckling  $t$ , equal to the thickness of the wall.

The area of the compression rod  $A_{md}$  is equal to the effective width of the rod,  $h' / 5$ , multiplied by the effective wall thickness,  $b$ , for effects in the direction parallel to the wall plane.

$$P_{nd} = 0.80 \cdot (0.85 \cdot f'_m \cdot A_{md}) \cdot R_e \quad (34)$$

$$R_e = 1 - \left[ \frac{h'}{42 \cdot t} \right]^2 \quad \text{for } \frac{h'}{t} \leq 30 \quad (35)$$

$$R_e = \left[ \frac{21 \cdot t}{h'} \right]^2 \quad \text{for } \frac{h'}{t} > 30 \quad (36)$$

$t$  - effective thickness of the section to evaluate buckling effects;

$A_{md}$  - effective masonry area for crush verification.

The detailed calculation is presented in the Appendix B Table 33

## 10.9 Verification to shear in the elements of confinement of the wall

---

The confinement elements receive the force of the compression rod at the intersection corner between beams and confinement columns, therefore there is a need to verify that they are capable of resisting as applied shear force, transverse to the longitudinal axis of the confinement element, at least one shearing force equal to one half of the corresponding component of the compression force acting on the connecting rod.

It must be fulfilled:

$$V_{uc} \leq \varphi V_{nc} \quad (37)$$

Where:

$V_{nc}$  - rated heavy duty shear force for a reinforced concrete section;

$V_{uc}$  - required design shear force acting on the confinement columns near the intersection with the confinement beam;

$\varphi$  - Resistance reduction factor.

$V_{nc}$  for confinement element must be calculated in accordance with the requirements of ACI318 Code(American Concrete Institute (ACI), 2014).

The acting shear force,  $V_{uc}$ , will be calculated by the following equations:

On the confinement column and beam:

$$V_{uc} = \frac{l_c}{2 \cdot l_w} \cdot V_u; V_{uc} = \frac{h_p}{2 \cdot l_w} \cdot V_u \quad (38)$$

Where:

$h_p$  - height of the floor located above the element under study, measured center to center between confinement beams (mm);

$l_c$  - horizontal distance between confinement columns, measured center to center, for the confined wall cloth under study (mm);

$l_w$  - total length of the wall over which the horizontal shear of the requested design acts  $V_u$ ;

$P_{ud}$  - axial force acting on the diagonal connecting rod of the wall (N);

$V_u$  - major shear strength requested for wall design (N);

$V_{uc}$  - required design shear force acting on the confinement columns near the intersection with the confinement beam (N).

Nominal shear strength for two-way members with shear reinforcement is calculated by (American Concrete Institute (ACI), 2014):

$$V_n = V_c + V_s \quad (39)$$

Where:

$V_c$  - nominal shear strength provided by concrete;

$V_s$  - nominal shear strength provided by shear reinforcement;

For members subjected to shear and flexure only:

$$V_c = 2 \cdot \lambda \cdot \sqrt{f'_c \cdot b_w \cdot d} \quad (40)$$

Where shear reinforcement perpendicular to axis of member is used:

$$V_s = \frac{A_v \cdot f_{yt} \cdot d}{s} \quad (41)$$

$A_v$  - the area of shear reinforcement within spacing  $s$ ;

$b_w$  - web width, or diameter of circular section;

$d$  - distance from extreme compression fiber to centroid of longitudinal tension reinforcement.

The detailed calculation is presented in the Appendix B Table 34

## 10.10 Design of the longitudinal steel of the confinement beam

---

The horizontal component of the compression rod that acts on the diagonal of the wall must be resisted as a tensile force in the confinement beam that reaches the same corner of the wall cloth where the compression rod acts. This tensile force is equal to the shear force that the wall cloth carries, therefore:

$$P_{ut} = -\frac{l_c}{l_w} \cdot V_u \quad (42)$$

Where:

$l_c$  - horizontal distance between confinement columns, measured center to center, for the confined wall cloth under study (mm);

$l_w$  - total length of the wall over which the horizontal shear of the requested design acts  $V_u$ ;

$P_{ut}$  - maximum requested axial forces of traction (N);

$V_u$  - major shear strength requested for wall design (N).

The axial tensile force on the confinement beam must be resisted in its entirety by the longitudinal reinforcing steel of the beam:

$$-P_{ut} \leq -\phi P_{nt} \quad (43)$$

Where:

$P_{nt}$  - nominal axial tensile strength (N);

$P_{ut}$  - maximum requested axial forces of traction (N).

The detailed calculation is presented in the Appendix B table 35

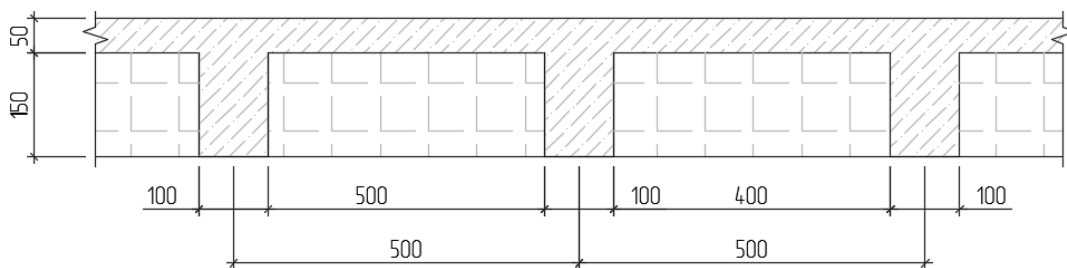
# 11 Design of the Waffle Slab

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According to engineer Florentino Regalado (Galeb and Atiyah, 2011), ‘the waffle slab belongs to the family of reinforced concrete slabs that are non-homogeneous, lightened, and reinforced in two orthogonal directions, forming a ribbed plate’.

Waffle slab construction consists of rows of concrete joists at right angles to each other with solid heads at the column (needed for shear requirements) or with solid wide beam sections on the column centerlines for uniform depth construction. Waffle slab construction allows a considerable reduction in dead load as compared to conventional flat slab construction since the slab thickness can be minimized due to the short span between the joists (Galeb and Atiyah, 2011).

Waffle slab characteristics are defined by the following parameters in ETABS are presented in the figure 48.



*Figure 47 Waffle Slab section*

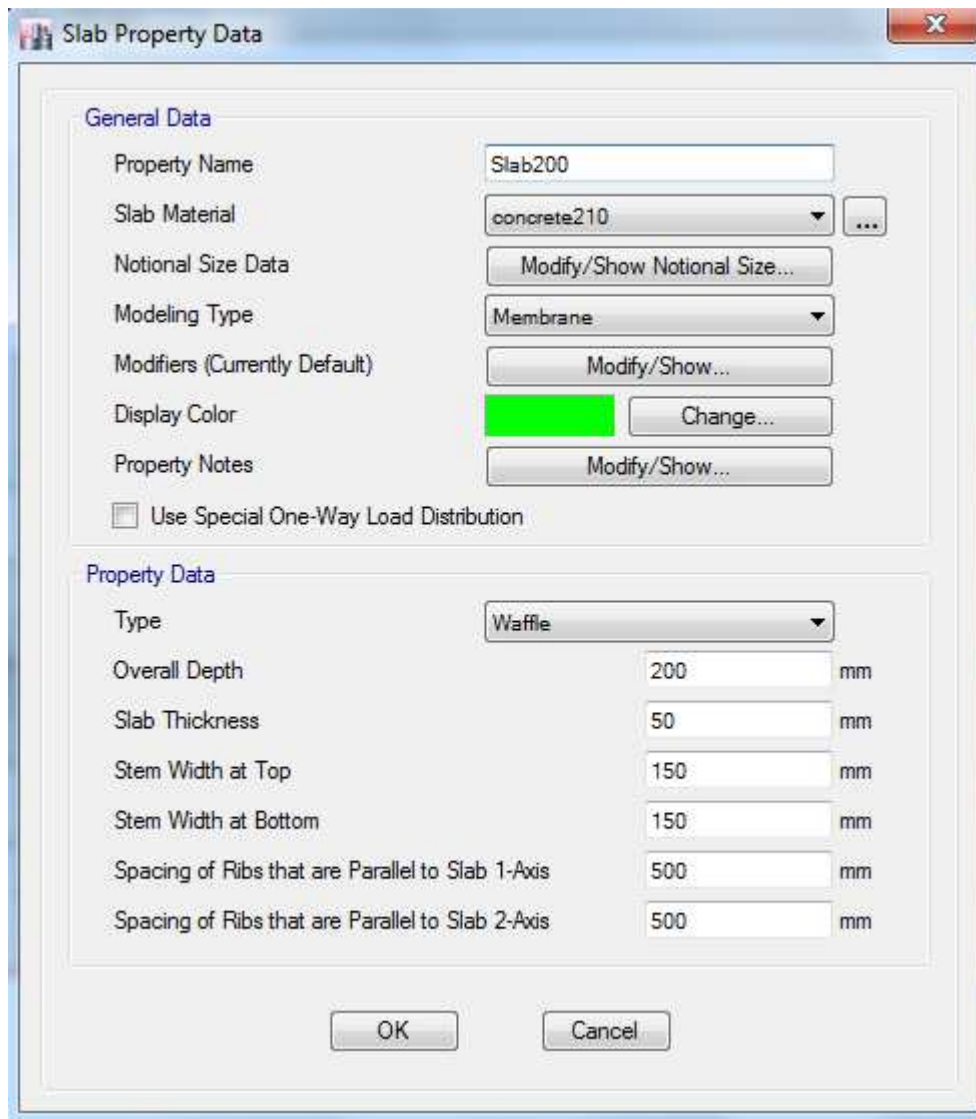


Figure 48 Waffle Slab characteristics from ETABS

Since the tensile strength of concrete is mainly satisfied by the steel bar reinforcement, only the “ribs” containing the reinforcement are kept where the remaining ‘unused’ concrete portion below the neutral axis are removed, to reduce the self-weight of the slab. This is achieved by placing clay pots or other shapes on the form work before casting of the concrete (Adeleke, 2017).

This type of slab is able, very adequately, to support distributed and point vertical actions, and to a lesser extent also horizontal actions. They are bi-directional or two-way slabs, because of the two orthogonal directions of their reinforcement, and slab bending can be broken down and analyzed according to those two reinforcement directions (Hamoda, 2016).

Benefits:

- Flexible
- Relatively light, therefore less foundation costs and longer spans are economic
- Speed of construction
- Fairly slim floor depths
- Robustness
- Excellent vibration control
- Thermal mass
- Good for services integration
- Durable finishes
- Fire resistance



*Figure 49 Example of the construction of the two-way waffle slab  
([https://fotos.habitissimo.com.mx/foto/losa-nervada\\_300902](https://fotos.habitissimo.com.mx/foto/losa-nervada_300902))*

Waffles are generally limited to the interior of a slab, leaving one or two of the forms out to create a solid fill around the supports. The solid fills provide the strength required for shear transfer to the supports. The fills also reduce the compression stresses at the soffit of the floor around the supports, thus avoiding the necessity of bottom reinforcement in this region.

## **11.1 Preliminary Member Sizing of the Two-Way Joist concrete waffle slab**

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- a) Slab minimum thickness – Deflection (8.3.1.1 (American Concrete Institute (ACI), 2014))

In lieu of detailed calculation for deflections, ACI 318 minimum slab thickness for two-way construction without interior beams is given in Table 8.3.1.1 (American Concrete Institute (ACI), 2014).

For flat plate slab system, the minimum slab thickness per ACI 318-14 are:

· Exterior Panels:

$$h_s = \frac{l_n}{31} = \frac{3800}{31} = 122.58 \text{ (mm)} \quad (44)$$

But not less than 125 mm.

· Interior Panels:

$$h_s = \frac{l_n}{31} = \frac{3800}{31} = 122.58 \text{ (mm)} \quad (45)$$

But not less than 125 mm.

Where  $l_n$  - length of clear span in the long direction

We will use 200 mm for all panels.

b) Slab shear strength – one way shear

Evaluate the average effective depth:

$$d_l = h_s - c_{clear} - d_b - \frac{d_b}{2} = 200 - 25 - 16 - \frac{16}{2} = 151 \text{ (mm)} \quad (46)$$

$$d_t = h_s - c_{clear} - \frac{d_b}{2} = 200 - 25 - \frac{16}{2} = 174 \text{ (mm)} \quad (47)$$

$$d_{avg} = \frac{d_l + d_t}{2} = \frac{151 + 174}{2} = 162 \text{ (mm)} \quad (48)$$

Check the adequacy of slab thickness for beam action (one-way shear) ACI 318-14 (22.5)

Consider a 1 m wide strip. The critical section for one-way shear is located at a distance  $d$ , from the face of support.

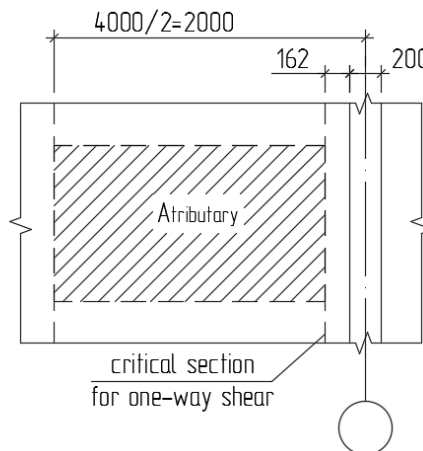


Figure 50 Critical section for one-way shear

Tributary are for one-way shear is:

$$A_{tributary} = \left( \frac{4}{2} - \frac{0.2}{2} - 0.162 \right) = 1.738 \text{ (m}^2\text{)} \quad (49)$$

$$V_u = q_u \cdot A_{tributary} = 395.6 \cdot 1.738 = 687.55 \text{ (kg)} \quad (50)$$

$$q_u = 1.2 \cdot D + 1.6 \cdot L = 1.2 \cdot 63 + 1.6 \cdot 200 = 395.6 \left( \frac{\text{kg}}{\text{m}^2} \right) \quad (51)$$

$$V_c = 2 \cdot \lambda \cdot \sqrt{f'_c} \cdot b_w \cdot d = 2 \cdot 1 \cdot \sqrt{21} \cdot 200 \cdot 162 = 25241 \text{ (N)} = 2576 \text{ (kg)} \quad (52)$$

$$\phi V_c = 0.75 \cdot 2576 = 1932 \text{ (kg)} \quad (53)$$

Where  $\lambda = 1$  for normal weight concrete.

In this case  $\phi V_c > V_u$ , therefore slab thickness of 20 cm is adequate for one-way shear

c) Check the applicable joist dimensional limitations as follows (American Concrete Institute (ACI), 2014):

1. Width of ribs shall be at least 100 mm at any location along the depth. ACI 318-14 (9.8.1.2)
2. Overall depth of ribs shall not exceed 3.5 times the minimum width. We will use ribs 120 mm depth.

$$3.5 \cdot 100 = 350 \text{ (mm)}$$

3. Clear spacing between ribs shall not exceed 760 mm. ACI 318-14 (9.8.1.4). We will use 500 mm of spacing
4. Slab thickness (with removable forms) shall be at least the greater of: ACI 318-14 (8.8.3.1). We will use a slab thickness 50 mm.
  - a) 1/12 clear distance between ribs:  $\frac{1}{12} \cdot 500 = 42 \text{ (mm)}$
  - b) 50 mm.

## 11.2 Design

---

The Direct Design Method is an approximate procedure for analyzing two-way slab systems subjected to gravity loads only (Galeb and Atiyah, 2011).

Since it is approximate, the method is limited to slab systems meeting some limitations, these are:

1. There must be three or more continuous spans in each direction;
2. Slab panels must be rectangular with a ratio of longer to shorter span (center line to center line) of supports) not greater than 2;
3. Successive span lengths (center line to center line of supports) in each direction must not differ by more than (1/3) of the longer span;

4. Columns must not be offset more than 10% of the span (in direction of offset) from either axis between centerlines of successive columns;
5. Loads must be uniformly distributed, with the unfactored or service live load not more than 2 times the unfactored or service dead load.

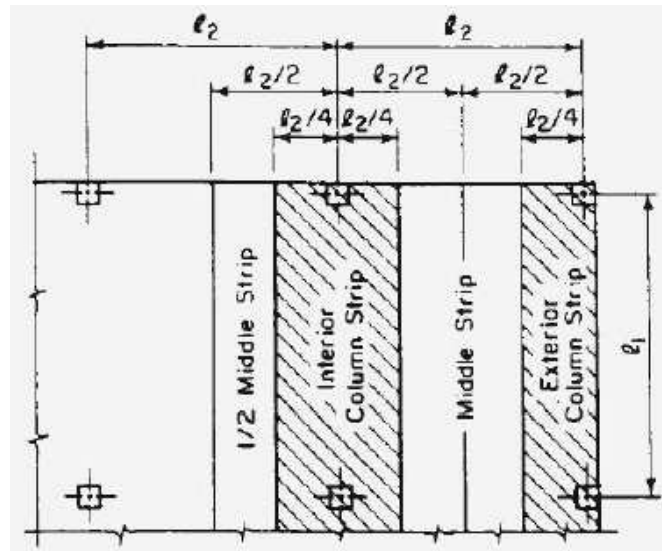


Figure 51 Definition of Design Strips  
(Galeb and Atiyah, 2011)

For the purpose of analysis, the slab system is divided into design strips consisting of column strip and middle strips using ETABS 2016 software. For the second floor the strips are analogous, but with the prefix "2".

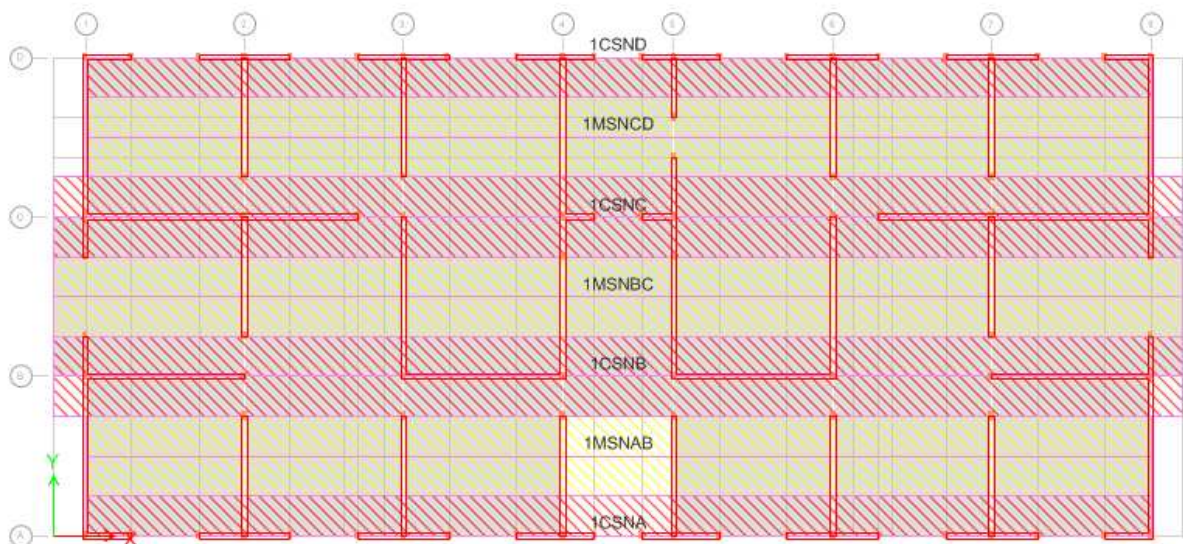


Figure 52 Plan of the slab with the strips. Layer A. 1st floor

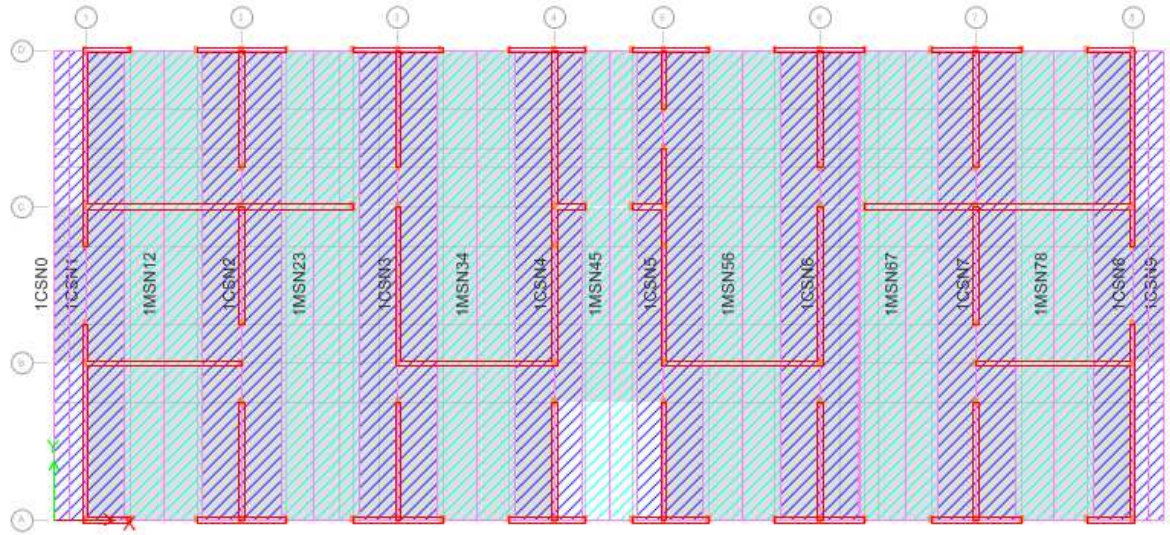


Figure 53 Plan of the slab with the strips. Layer B. 1st floor

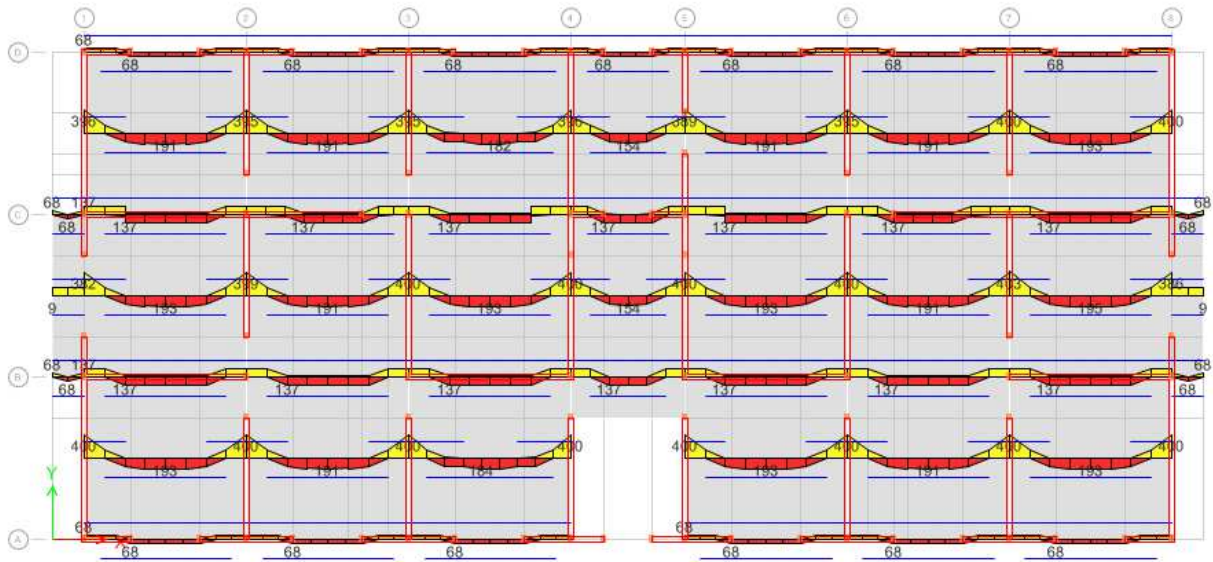


Figure 54 Plan of the slab. 1st floor. Layer A. Top and bottom reinforcement

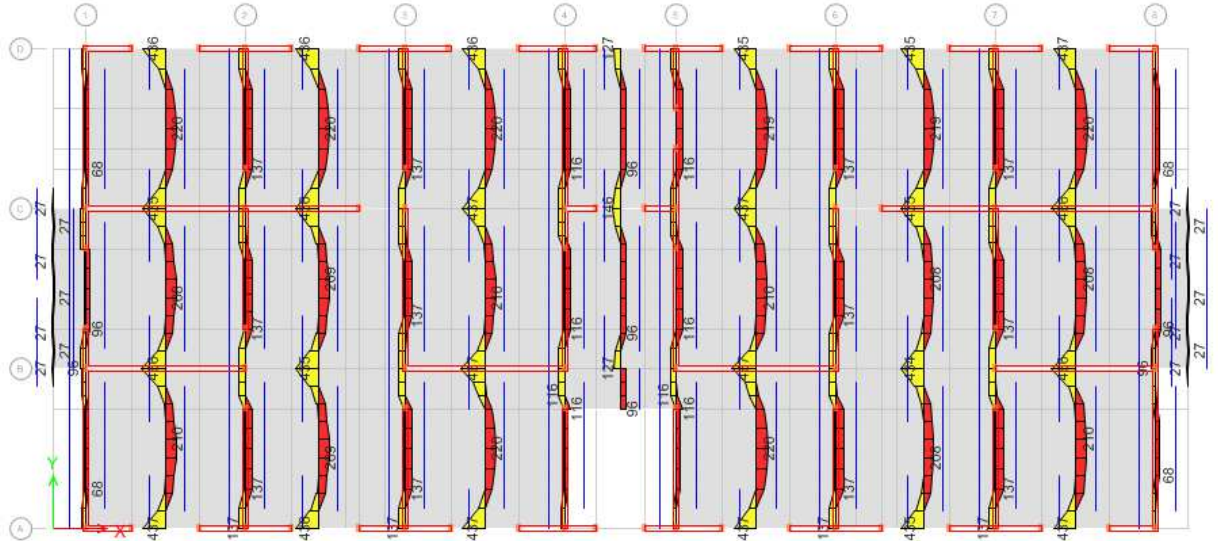


Figure 55 Plan of the slab. 1st floor. Layer B. Top and bottom reinforcement

The results for the strip design are provided in the table below. For the reinforcing display type was chosen total rebar area for a strip.

Table 25 Slab reinforcement results from ETABS. 1st floor

Strip name	Moment location	Mu max (Nmm)	As (mm <sup>2</sup> )	Strip width (mm)	n of ribs	n of bars	As (mm <sup>2</sup> ) per bar	Diameter of a bar
1_CSNA	Mtop (conf.col)	-2224600	122	1000	2	2	61	φ10
	Mbot	714420	18	1000	2	2	9	φ8
1_CSNB	Mtop (conf.col)	-4686360	122	2000	4	4	31	φ8
	Mbot	1518020	39	2000	4	4	10	φ8
1_CSNC	Mtop (conf.col)	-4686360	122	2000	4	4	31	φ8
	Mbot	1589560	41	2000	4	4	10	φ8
1_CSND	Mtop (conf.col)	2224600	58	2000	4	4	15	φ8
	Mbot	728140	19	2000	4	4	5	φ8
1_CSNE	Mtop (conf.col)	2358860	70	1400	3	3	25	φ8
	Mbot	766360	22	1400	3	3	8	φ8
1_CSNE	Mtop (conf.col)	-4314940	127	2000	4	4	32	φ8
	Mbot	1532720	44	2000	4	4	11	φ8
1_CSNE	Mtop (conf.col)	-4501140	133	2000	4	4	33	φ8
	Mbot	1532720	44	2000	4	4	11	φ8
1_CSNE	Mtop	-3039960	89	1700	3	3	26	φ8

	(conf.col)							
	Mbot	934920	27	1700	3	3	8	φ8
1_CSN5	Mtop (conf.col)	-3039960	89	1700	3	3	26	φ8
	Mbot	959420	28	1700	3	3	8	φ8
1_CSN6	Mtop (conf.col)	-4501140	133	2000	4	4	33	φ8
	Mbot	1531740	44	2000	4	4	11	φ8
1_CSN7	Mtop (conf.col)	-4501140	133	2000	4	4	33	φ8
	Mbot	1528800	44	2000	4	4	11	φ8
1_CSN8	Mtop (conf.col)	-2379440	70	1400	3	3	25	φ8
	Mbot	760480	22	1400	3	3	8	φ8
1_MSNA B	Mtop (support)	14536340	400	2000	4	4	100	φ12
	Mbot (span)	7487200	193	2000	4	4	48	φ8
1_MS NBC	Mtop (support)	14642180	403	2000	4	4	101	φ12
	Mbot (span)	7560700	195	2000	4	4	49	φ8
1_MS NCD	Mtop (support)	14536340	400	2000	4	4	100	φ12
	Mbot (span)	7487200	193	2000	4	4	48	φ8
1_MS N12	Mtop (support)	14629440	466	2000	4	4	117	φ14
	Mbot (span)	7542080	220	2000	4	4	55	φ10
1_MS N23	Mtop (support)	13775860	436	2000	4	4	109	φ12
	Mbot (span)	3538780	102	2000	4	4	26	φ8
1_MS N34	Mtop (support)	14650020	467	2000	4	4	117	φ14
	Mbot (span)	7557760	220	2000	4	4	55	φ10
1_MS N45	Mtop (support)	4841200	146	2000	4	4	37	φ8
	Mbot (span)	2021740	58	2000	4	4	15	φ8
1_MS N56	Mtop (support)	14650020	467	2000	4	4	117	φ14
	Mbot (span)	7557760	220	2000	4	4	55	φ10
1_MS N67	Mtop (support)	13726860	435	2000	4	4	109	φ12
	Mbot	7516600	219	2000	4	4	55	φ10

	(span)							
1_MS78	Mtop (support)	14629440	466	2000	4	4	117	φ14
	Mbot (span)	7557760	220	2000	4	4	55	φ8

Table 26 Slab reinforcement results from ETABS. 2nd floor

Strip name	Moment location	Mu max (Nmm)	As (mm <sup>2</sup> )	Strip width (mm)	n of ribs	n of bars	As (mm <sup>2</sup> ) per bar	Diameter of a bar
2_CSNA	Mtop (conf.col)	- 2224600	58	1000	2	2	29	φ8
	Mbot	714420	18	1000	2	2	9	φ8
2_CSNB	Mtop (conf.col)	- 3718120	96	2000	4	4	24	φ8
	Mbot	1208340	31	2000	4	4	8	φ8
2_CSNC	Mtop (conf.col)	- 3590720	93	2000	4	4	23	φ8
	Mbot	1151500	29	2000	4	4	7	φ8
2_CSND	Mtop (conf.col)	1754200	45	2000	4	4	11	φ8
	Mbot	574280	15	2000	4	4	4	φ8
2_CSN1	Mtop (conf.col)	6260240	191	1400	3	3	68	φ10
	Mbot	2983120	86	1400	3	3	31	φ8
2_CSN2	Mtop (conf.col)	- 3579940	105	2000	4	4	26	φ8
	Mbot	1246560	36	2000	4	4	9	φ8
2_CSN3	Mtop (conf.col)	- 3664220	108	2000	4	4	27	φ8
	Mbot	1246560	36	2000	4	4	9	φ8
2_CSN4	Mtop (conf.col)	- 2235380	65	1700	3	3	19	φ8
	Mbot	727160	21	1700	3	3	6	φ8
2_CSN5	Mtop (conf.col)	- 2235380	65	1700	3	3	19	φ8
	Mbot	727160	21	1700	3	3	6	φ8
2_CSN6	Mtop (conf.col)	- 3439800	101	2000	4	4	25	φ8
	Mbot	1205400	35	2000	4	4	9	φ8
2_CSN7	Mtop (conf.col)	- 3465280	102	2000	4	4	26	φ8
	Mbot	1205400	35	2000	4	4	9	φ8
2_CSN8	Mtop (conf.col)	- 6260240	191	1400	3	3	68	φ10
	Mbot	2983120	86	1400	3	3	31	φ8

2_MSNA B	Mtop (support)	1209712 0	328	2000	4	4	82	φ12
	Mbot (span)	6239660	160	2000	4	4	40	φ8
2_MSNC C	Mtop (support)	1211476 0	329	2000	4	4	82	φ12
	Mbot (span)	6239660	160	2000	4	4	40	φ8
2_MSNC D	Mtop (support)	1146306 0	310	2000	4	4	78	φ12
	Mbot (span)	5904500	152	2000	4	4	38	φ8
2_MSNA12	Mtop (support)	1421980 0	452	2000	4	4	113	φ14
	Mbot (span)	7484260	218	2000	4	4	55	φ10
2_MSNA23	Mtop (support)	1160124 0	361	2000	4	4	90	φ12
	Mbot (span)	2339260	68	2000	4	4	17	φ8
2_MSNA34	Mtop (support)	1155224 0	359	2000	4	4	90	φ12
	Mbot (span)	2182460	63	2000	4	4	16	φ10
2_MSNA45	Mtop (support)	3811220	113	2000	4	4	28	φ8
	Mbot (span)	1594460	46	2000	4	4	12	φ8
2_MSNA56	Mtop (support)	1153754 0	359	2000	4	4	90	φ12
	Mbot (span)	5693800	165	2000	4	4	41	φ10
2_MSNA67	Mtop (support)	1082508 0	335	2000	4	4	84	φ12
	Mbot (span)	5928020	172	2000	4	4	43	φ10
2_MSNA78	Mtop (support)	1421980 0	452	2000	4	4	113	φ14
	Mbot (span)	7484260	218	2000	4	4	55	φ8

Table 27 Concrete slab design. Check of the maximum and minimum reinforcement area. 1st floor

	X direction	Y direction
Asmax Top cm <sup>2</sup>	113,1	153,9
	φ12	φ14
Asmax Bot cm <sup>2</sup>	113,1	78,5
	φ12	φ10
ρ	0,0141	0,0145

Steel requirements check:

The minimum longitudinal steel value required for bending is found in the ecuadorian standard ( 4.2.5 (NEC-SE-HM, 2015))

$$A_s \geq \max \left[ \frac{1.4}{f_y} \cdot b_w \cdot d; A_{s,min} = \frac{\sqrt{f'_c}}{4f_y} \cdot b_w \cdot d \right] \quad (54)$$

The greatest of these two is chosen.

For this case it will be:

Table 28 Concrete slab design. Minimum reinforcement area

fc'	21	Mpa
fy	420	Mpa
bw	100	mm
d	160	mm
Asmin1	43,64	mm <sup>2</sup>
Asmin2	53,33	mm <sup>2</sup>

The maximum allowable amount for seismic zones is then calculated and compared with the determined amount of steel:

$$\rho_b = 0.85 \cdot \beta_1 \cdot \frac{f'_c}{f_y} \cdot \frac{0.003}{\frac{f_y}{E_s} + 0.003} \quad (55)$$

$$\beta_1 = 0.85 \quad (4.1 \text{ (NEC-SE-HM, 2015)})$$

$$\rho_{min} = \frac{\sqrt{f'_c}}{4 \cdot f_y} \geq \frac{1.4}{f_y} \quad (56)$$

$$\rho_{max} = 0.025$$

Table 29 Concrete slab design. Amount of reinforcement

ρmin	0,0027
	0,0033
ρmax	0,0250
ρb	0,0213

$$\rho_{min} \leq \rho = 0.0141 \leq \rho_{max}$$

$$\rho_{min} \leq \rho = 0.0145 \leq \rho_{max}$$

Minimum amount of steel reinforcing mesh:

$$\rho_{temp} = 0.0018$$

$$A_{stemp} = \rho_{temp} \cdot b_w \cdot d = 0.0018 \cdot 100 \cdot 140 = 0.252 \text{ (cm}^2\text{)} \quad (57)$$

We will use one layer of the mesh  $\Phi 6$  150x150 mm.

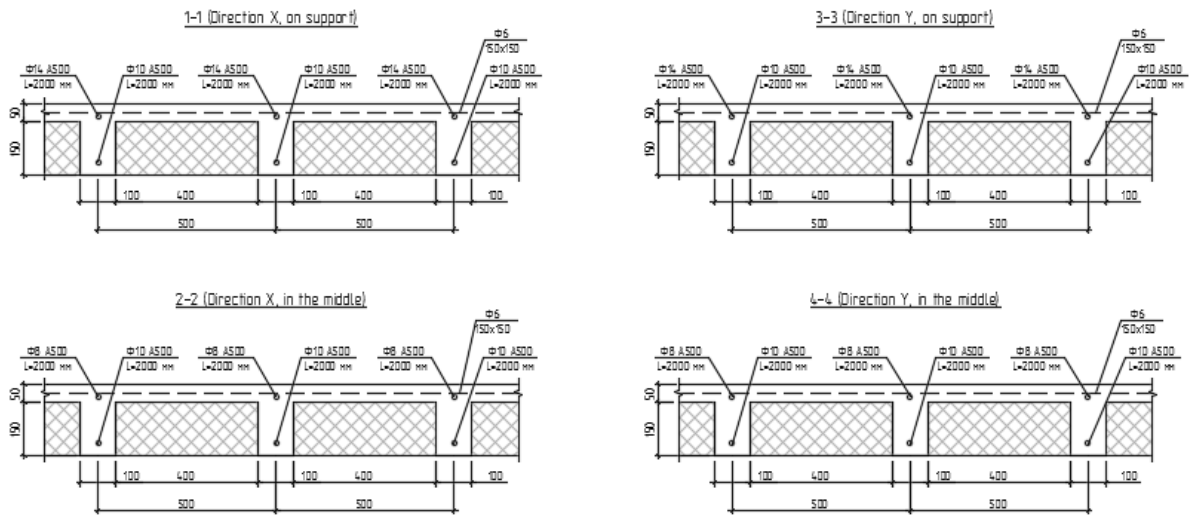


Figure 56 Sections of the slab in X and Y directions



## 12 Strip foundations

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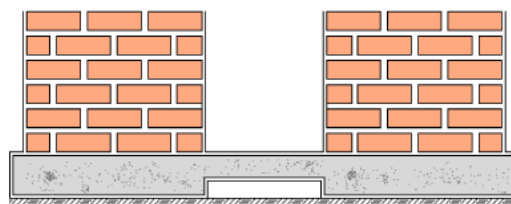
Strip foundations (or strip footings) are a type of shallow foundation that are used to provide a continuous, level (or sometimes stepped) strip of support to a linear structure such as a wall or closely-spaced rows of columns built centrally above them.

Strip footings may be designed to support single or two-storey structures and may be placed on level, gently sloping or steeply sloping sites. Where the site is sloping it is necessary to create 'steps' and 'benches' from the highest to the lowest level. This allows the top and bottom of the footing to remain level, thus avoiding slip under load. The footing is lapped in its length where steps occur so the design strength is maintained. The depth of these steps is usually determined by brick courses to avoid unnecessary cutting or splitting of bricks when the walls are laid (TAFE, 2012)

The most relevant conclusions regarding the study of structural wall systems supported by continuous footings, presented by Herrón Durán (Herrón Durán, 1988) are:

✓ The foundation of a structural masonry without spans (garage, passageway, etc.) must provide sufficient rigidity to prevent the wall from absorbing the demands induced by differential settlements. This is achieved by providing an important cant to the section and a minimum amount of reinforcing steel.

✓ It is recommended (and this is an innovative contribution of Herrón - Garza), for the continuous shoe that serves as a foundation to a masonry with spans (garage, passage, etc), not to support it in the area where the wall does not transmit load, as illustrated in Figure 53. This model controls much better the differential settlements and considerably reduces the diagram of moments in a percentage that can reach 500%, depending on the load transmitted by the wall and the length of the hole, it is then possible to use more flexible and efficient shoes.



*Figure 57 Detail where the interruption of the support in a continuous shoe is indicated when there are openings in the masonry.*

✓ The variation of the stiffness of the continuous shoe that serves as a foundation to a structural masonry, in most cases, does not vary substantially the moment regime. It is possible to work with the minimum stiffness required by the Standard, since it gives tolerable differential settlements.

✓ The influence of the compressibility of the stratum on the mechanical elements of the foundation structure is very little, almost negligible. This variable affects the value of the total settlement almost directly (Garza, 2004).

## **12.1 Minimum requirements for foundation of load-bearing walls (NEC Vivienda)**

---

It must exist under all the wall axes and must be continuous even in openings such as doors and windows, in addition it must have upper and lower longitudinal reinforcement and confining stirrups along its entire length. The dimensions and reinforcement of the foundations are presented in Table 30.

The lower level of the foundation struts must be at a minimum depth of 500 mm below the finishing level of the ground floor or according to what is specified by the soil study.

For load-bearing walls, with or without polystyrene core, reinforced concrete or reinforced mortar, anchoring must be provided to the foundation slab system, with steel reinforcement as dowel pins or inserts, anchoring chicotes, which comply with the length of development established in ACI 318 (American Concrete Institute (ACI), 2014).

The foundation for these two systems may be superficial and designed according to the carrying capacity of the soil and its structural verification.

Table 30 Dimensions and reinforcement of the strip foundations  
(NEC-SE-Vivienda, 2015)

Cimentación corrida	Un piso	Dos pisos	Resistencia Mínima	
			Acero Refuerzo	de Hormigón
			$f_y$ (MPa)	$f'_c$ (MPa)
Ancho	250 mm	300 mm	* 420 (barra corrugada)	18
Altura	200 mm	300 mm		
Acero longitudinal	4 $\phi$ 10* mm	4 $\phi$ 12* mm		
Estribos	$\phi$ 8* mm @ 200mm	$\phi$ 8* mm @ 200mm		
Acero para anclaje de muros	10* mm	10* mm		

When reinforcing steel with specified yield strength greater than 420 MPa (4200 kg / cm<sup>2</sup>) is used, the calculated steel amounts may be reduced by multiplying them by 420 /  $f_y$ , in MPa (4200 /  $f_y$ , in kg / cm<sup>2</sup>).

To ensure the durability of the foundation struts, since they will be in contact with the ground, a coating of at least 5 cm should be provided (NEC-SE-Vivienda, 2015)

## 12.2 Wall Footing design (Strip Footing)

Wall footings or strip footings display essentially one-dimensional action, cantilevering out on each side of the wall. A wall footing behaves similarly to a cantilever beam, where the cantilever extends out from the wall and is loaded in an upward direction by the soil pressure.

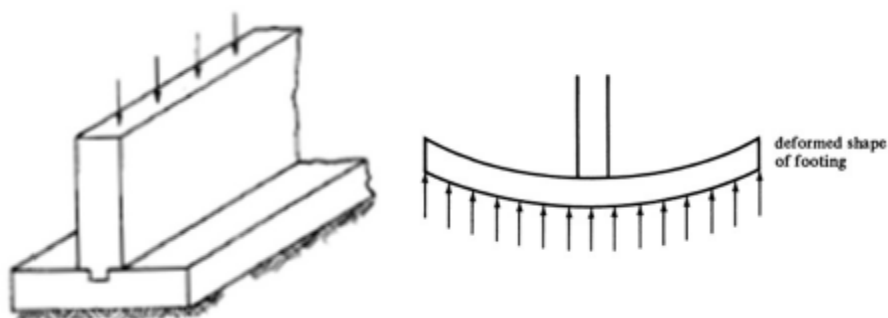


Figure 58 Strip foundation example and its deformed shape  
(Qaisar, 2016)

The wall footing has bending in only one direction, it is generally designated in much the same manner as a one-way slab, by considering a typical 12-in. wide strip along the wall length. The simple principles of beam action apply to wall footings with only minor modifications.

Reinforcement:

- a) Main reinforcement for flexure is placed at the bottom of the footing;
- b) Temperature reinforcement is placed at the bottom of the footing parallel to the wall along the long direction and perpendicular to the wall along the short direction, as shown.

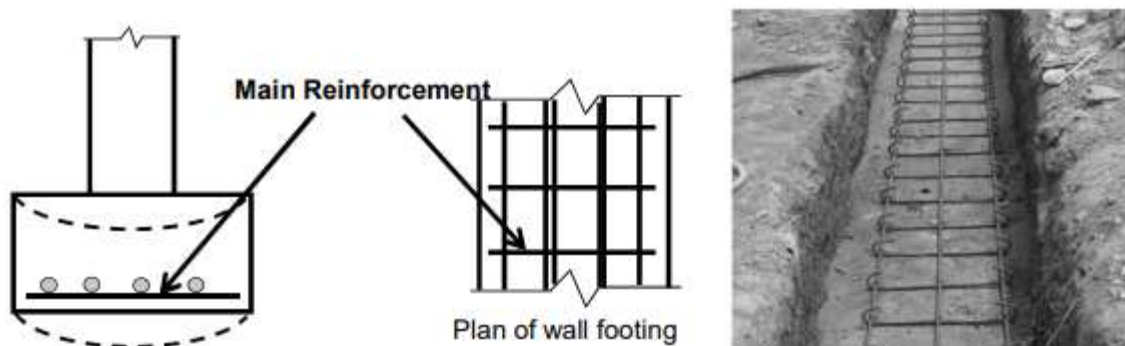


Figure 59 Typical reinforcement of the strip foundation  
(Qaisar, 2016)

ACI section 13.3 (American Concrete Institute (ACI), 2014) contains provisions for shallow foundations.

- a) Required Footing Area
  - Footing bearing area is calculated based on unfactored forces or service loads (ACI 13.3.1.1 (American Concrete Institute (ACI), 2014))

- Bearing area:

$$A_{req} = \frac{\text{Service Load}}{q_e} \quad (58)$$

- Where Effective bearing capacity,  $q_e = q_a - W$ , where ( $W$  = Weight of fill + weight of concrete)

- b) Bearing pressure for strength design of footing:

$$q_u = \frac{\text{factored load on column}}{A_{req}} \quad (59)$$

- c) Design Considerations in Flexure

- The maximum factored moment is calculated at critical section

For a footing supporting masonry wall, critical section is located between the edge and the middle of the wall (American Concrete Institute (ACI), 2014):

$$M_u = \frac{q_u \cdot (k + \frac{b}{4})^2}{2} \quad (60)$$

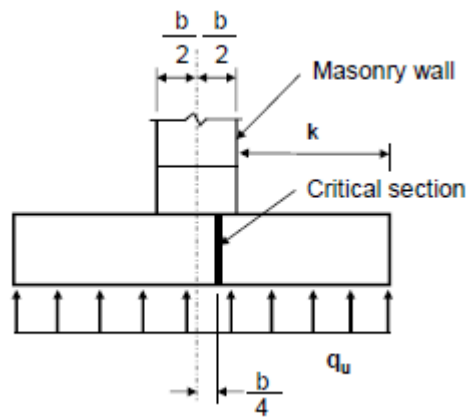


Figure 60 Location of the critical section  
(Qaisar, 2016)

- Minimum reinforcement Requirement,  $A_{smin}$  (ACI 7.6.1.1):

For less than Grade 60,  $A_{smin} = 0.0020 bh$

For Grade 60,  $A_{smin} = 0.0018 bh$

- Maximum spacing requirement:  $3h$  or  $18''$

· Clear cover: minimum  $3''$  clear cover must be provided to protect the bars from corrosion.

#### d) Distribution of Reinforcement

ACI 13.3.2.2 states that in one-way footings, reinforcement shall be distributed uniformly across entire width of footing

#### e) Design Considerations in Shear

- The behavior of footings in shear is similar to beams.
- Only one-way shear or beam shear is significant in wall footing. Hence determining critical shear at critical section which is at a distance “ $d$ ” from the face of support.

- Calculation of Critical shear at distance ‘ $d$ ’:

$$V_u = q_{ub} \cdot (k - d) \quad (61)$$

Where  $b$  is unit width equal to 1 meter.

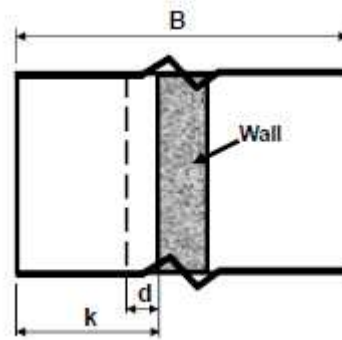


Figure 61 Geometrical parameters for calculation  
(Qaisar, 2016)

- Beam shear capacity ( $\phi V_c$ )

$$\phi V_c = \phi \cdot 0.17 \sqrt{f'_c} \cdot b \cdot d \quad (62)$$

If  $\phi V_c < V_u$ , the depth of footing should be increased instead of providing any shear reinforcement.

## 12.3 Design Procedure

---

Step # 01: Estimate the thickness of footing, h

Assume thickness h of the footing which must satisfy the shear requirements. (Min. thickness of footing on soil = 9 in (229 mm))

Step # 02: Calculate weight of fill + weight of concrete, W

$$\begin{aligned} W &= W_{conc} + W_{inf} = (0,8 * 0,6 + 0,8 * 0,2) * 2500 + 1 * 0,2 * 2 * 1800 \\ &= 2320 \text{ (kg/m)} \end{aligned}$$

Step # 03: Calculate effective bearing capacity,  $q_e$

$$q_e = q_a - W = 20394 - 2320 = 18074 \text{ (kg/m)}^2$$

$q_a = 200 \text{ (kN/m}^2) = 20394 \text{ (kg/m}^2)$  - allowable bearing capacity of soil

Step # 04: Calculate bearing area,  $A_{req}$

$$A_{req} = \text{Serviceload} / q_e = 4864 / 18074 = 0,27 \text{ (m}^2)$$

Assume that  $b=0,6$  m (because the minimum width of the footing is equal to 0.5m (NEC-SE-Vivienda, 2015))

Step # 05: Calculate design pressure on base of footing due to factored loads,  $q_u$

$$q_u = \text{Factoredload} / \text{Bearingarea} = 7548 / 0,6 = 12580 \text{ (kg)}^2$$

Step # 06: Calculate the critical shear,  $V_u$

$$V_u = q_u \cdot b \cdot (k - d) = 12580 \cdot 1 \cdot (0,25 - 0,124) = 1585,08 \text{ (kg/m)}$$

$$d = b_w - a - \frac{\Phi}{2} = 200 - 70 - \frac{12}{2} = 124 \text{ (mm)}$$

Step # 07: Check the shear capacity,  $\phi V_c$

$$\phi V_c = \phi \cdot 0,17 \cdot \sqrt{f'_c} \cdot b_w \cdot d = 0,75 \cdot 0,17 \cdot \sqrt{204} \cdot 100 \cdot 12,4 = 2258,12 \text{ (kg)}$$

$\phi V_c$  shall be equal to or greater than  $V_u = 1585,08$  kg

Since  $V_u < \phi V_c$ , the footing depth is OK.

If  $V_u$  is larger or considerably smaller than  $\phi V_c$ , then chose a new thickness and repeat the previous steps.

Assume 0,8 m thick and 0,6 m in wide footing.

Step # 08: Calculate maximum moment,  $M_u$

The maximum factored moment is calculated at critical section

$$M_u = \frac{q_u \cdot (k + b/4)^2}{2} = \frac{12580 \cdot (0,25 + 0,2/4)^2}{2} = 56610 \text{ (kgcm)}$$

13.2.7.1  $M_u$  at the supported member shall be permitted to be calculated at the critical section defined in accordance with Table 13.2.7.1.

Step # 09: Calculate steel area,  $A_s$

$$A_s = \frac{M_u}{\phi \cdot f_y \cdot (d - a/2)} = \frac{56610}{0,75 \cdot 4200 \cdot (12,4 - 16/2)} = 4,08 \text{ (cm}^2 \text{ per 1 m)}$$

$$a = 0,2 \cdot h = 0,2 \cdot 80 = 16 \text{ (cm)}$$

Step # 10: Minimum reinforcement and maximum spacing check

$$A_{s,min} = A_{dist} = 0,0018 \cdot b \cdot h = 0,0018 \cdot 60 \cdot 80 = 8,64 \text{ (cm}^2 \text{)}$$

$$A_s \geq A_{s,min}$$

In our case  $A_s < A_{s,min}$ , so assume that  $A_s = 8,64 \text{ (cm}^2 \text{)}$

Step # 11: Bars Spacing/Placement

$$\text{Main Bars: } s = A_b \cdot 12 / A_s = 0,785 \cdot 12 / 8,64 = 1,09$$

Assume bars  $\Phi 14$  mm,  $A_b = 1,539$  ( $cm^2$ )

$6\Phi 14 - A_s = 9,24$ ( $cm^2$ )

## 12.4 The seismic design of foundations

---

The seismic design of foundations has largely evolved from static design considerations.

However, the performance requirements for foundations under design seismic loadings differ from the static requirements in several important aspects. The loadings (or more accurately, the displacements) imposed by the earthquake are of very short duration and hence some permanent soil-structure deflections under the peak actions may be permissible and even desirable. The foundation must continue to support the structure during and after the seismic event and foundation deflections must remain within the range that the structure can withstand. In order to limit the transient deflections it is very desirable that the stiffness of the combined building and foundation system does not cause the structure to respond within the peak frequency range of the seismic motion. These considerations require different analytical approaches to those used for static design. The designer of shallow foundations subject to seismic loading ideally seeks robust solutions to the following questions (May, 2015):

- ✓ What transient and permanent foundation deflections will occur?

Knowledge of the seismic bearing capacity will assist in answering this question but will not provide the complete solution.

- ✓ What transient accelerations, forces and/or displacements will be transmitted through the foundation and into the structure?

- ✓ Will the foundation continue to support the structure following the seismic event?

- ✓ How can economic foundation solutions be provided for new and existing structures?

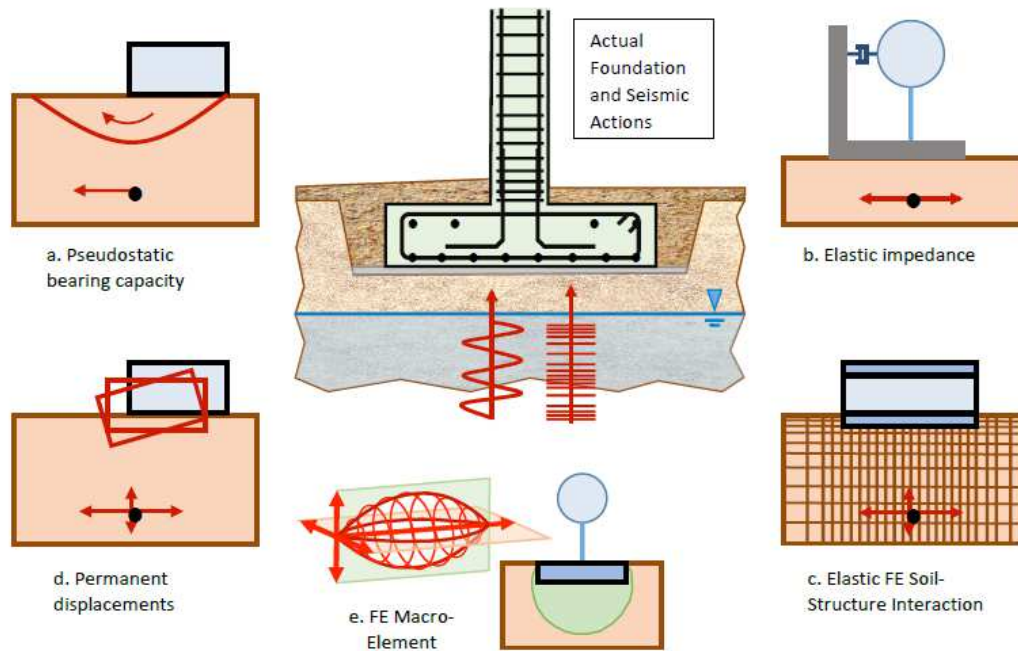


Figure 62 Analytical methods for seismic design of shallow foundations  
(May, 2015)

When considering the seismic performance of shallow foundations under seismic loading a range of issues should be borne in mind by the designer. These include:

- ✓ The properties of the soil beneath the foundation (including its strength and stiffness) will change over time due to consolidation, creep, the effects of low level cyclic loading and moisture content change in partially saturated conditions. Hence the foundation response at the start of a severe or design earthquake may be affected by soil conditions which differ appreciably from those determined from a pre-construction green-field site investigation.

- ✓ The stiffness of soil and (to a lesser extent) weak rock is highly non-linear as discussed below. The soil is also anisotropic in its stiffness and strength properties which means that considerable care is required when interpreting measured ground properties for use in simplified linear or non-linear isotropic analytical models.

- ✓ Under cyclic loading the stiffness of the soil is “reset” to its current small strain value on each abrupt change in direction of the stress path. Degradation of stiffness due to increasing strain occurs along the most recent stress path until the next abrupt change occurs. Degradation of stiffness also occurs under continued cyclic loading due to the build-up of pore pressure in the soil. This occurs more rapidly when the cyclic loading includes substantial rotations of principal stresses (as is common in seismic loading).

✓ Considerable energy is dissipated by foundations due to hysteretic damping in the soil and by radiation damping. The hysteretic component is strain-dependent. Radiation damping may be affected by the position of the ground water level.

✓ If the soil yields under cyclic loading the strains will become localised on shear surfaces. On these surfaces the strength of the soil will tend to be reduced from the peak values of shearing resistance. In granular soils the strength is likely to approach the critical state values while in cohesive soils the undrained strength may approach remoulded values. It should be noted that the shear surfaces generated due to seismic loading of a foundation are likely to be in substantially different planes to those relevant to static (i.e. pre- and post-seismic) loading conditions. (May, 2015)

### 12.4.1 “Traditional” methodology

---

Traditionally, the design of shallow foundations is executed using a methodology based in the global safety factor concept. According to this methodology, the designer must ensure that the acting stress on soil foundation is lower than the allowable bearing capacity of soil. Allowable bearing capacity is obtained dividing the ultimate bearing capacity of soil by a global safety factor as suggested in the equation:

$$q_{all} = \frac{q_{ult}}{GSF} \quad (63)$$

In this equation,  $q_{all}$ ,  $q_{ult}$  and  $GSF$  represents, respectively, the allowable bearing capacity, the ultimate bearing capacity and the global safety factor. The ultimate bearing capacity of soil was obtained before according the ACI regulations. In this paper, the global safety factor considered was obtained in table 6 (NEC-SE-GC, 2015) and it is equal to 1.5.

This methodology is also applied to a seismic situation. For this situation, the allowable stress of soil is obtained increasing the static value by percentages ranging from 20% to 50%. This increasing of the allowable soil stress is related with the lower probability of occurrence of a seismic event and can be seen like a reduction in the global safety factor (Serenó, 2016).

The differences between the verifications on the bearing capacity of shallow foundations for a seismic situation according the EC8-5 and the “traditional” methodology were analysed through a parametric study in the work performed by Pedro Miguel Sereno (Serenó, 2016).

In his parametric study, three comparisons were performed:

1. Comparison of collapse loads for a static and seismic situation obtained from analytical formulation proposed in Annex D of EC7-1 and Annex F of EC8-5 respectively.
2. Comparison of design loads for a static and seismic situation obtained according EC7-1 and EC8-5, respectively.
3. Comparison of design loads for a seismic situation obtained according EC8-5 and according the “traditional” approach.

The results of this study indicate that, for cohesionless soils, and until values of  $a_g/g = 0,2$ , the methodology based in the global safety factor concept is more conservative. For cohesive soils, this methodology, is always more conservative.



## 13 Conclusions

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Completing my internship in Ecuador I got a highly valuable experience of working on different types of projects in completely different environment. Not only I had to learn the new language due to the fact that all regulatory documents are written exceptionally in Spanish, but also to know how to work as a structural engineer in a country with a high level of seismic activity. It is kind of very challenging activity because as engineer you need to consider much more information when you are designing even a simple structure. I think, that after I return to Russia the designing will look like a very simple process comparing with the experience that I got in Ecuador.

During my internship I designed generally all the most common types of structures like steel frames, concrete frames, masonry. Therefore, I learned the majority of the Ecuadorian building codes NEC-2015 regarding these structures and also Columbian codes NSR-10, Mexican codes NTC and American codes ACI.

Moreover, I had to deal with the new software for me like SAP 200 and ETABS 2016, that most engineers use in Ecuador and whole Latin America.

As a part of my internship was also visiting the construction sites where I could observe the construction process, the problems that the builders have in the particular region and the construction process organization in general.

I also learned a lot of new information about the design and the construction processes in Ecuador, especially the requirements for confined masonry buildings, due to the detailed project which I elaborated in this report according these documents: Housing of Up to 2 Floors with Lights of up to 5 m (NEC-SE-Vivienda, 2015) and Structural Masonry(NEC-SE-MP, 2015). There are all the parameters and requirements necessary to build safe buildings of confined masonry in Ecuador.

I received invaluable knowledge working in Ecuador and performing seismic design of different structures. Unfortunately, Russian regulations according seismology are completely outdated and most of the engineers use translated versions of American Codes, so my experience will help me in my future projects.



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# Appendices

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## Appendix A Results of a structural analysis using ETABS 2016 Software

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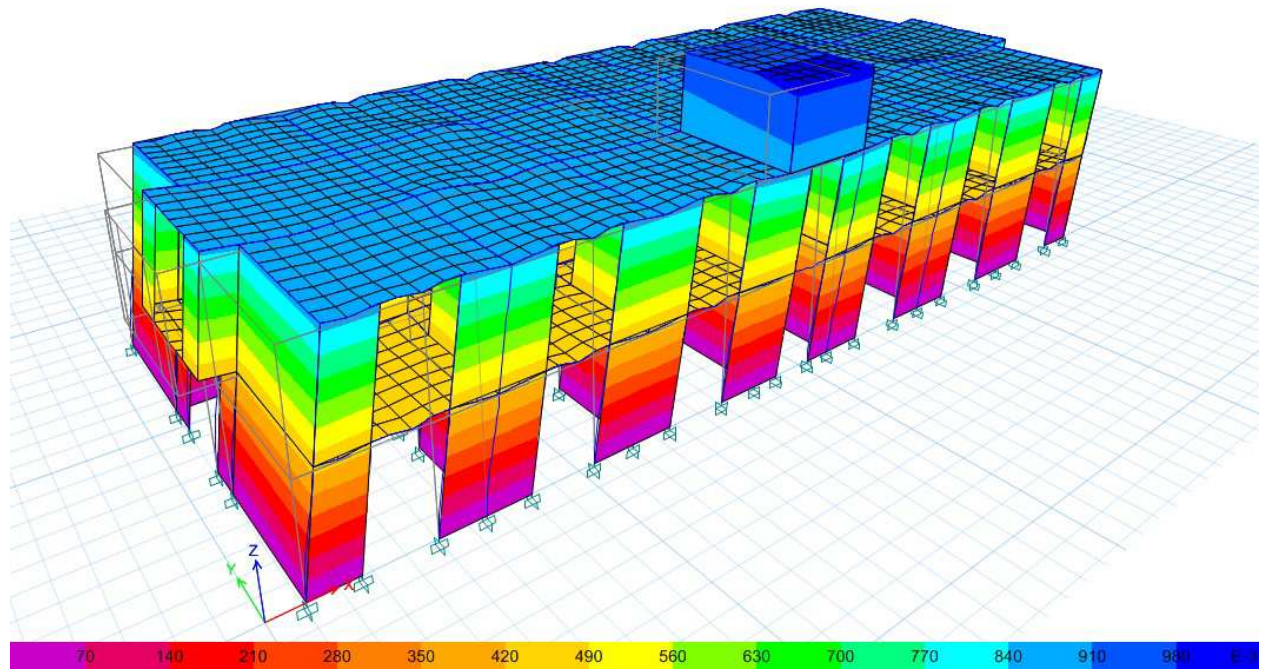


Figure 63 Displacements RS-X

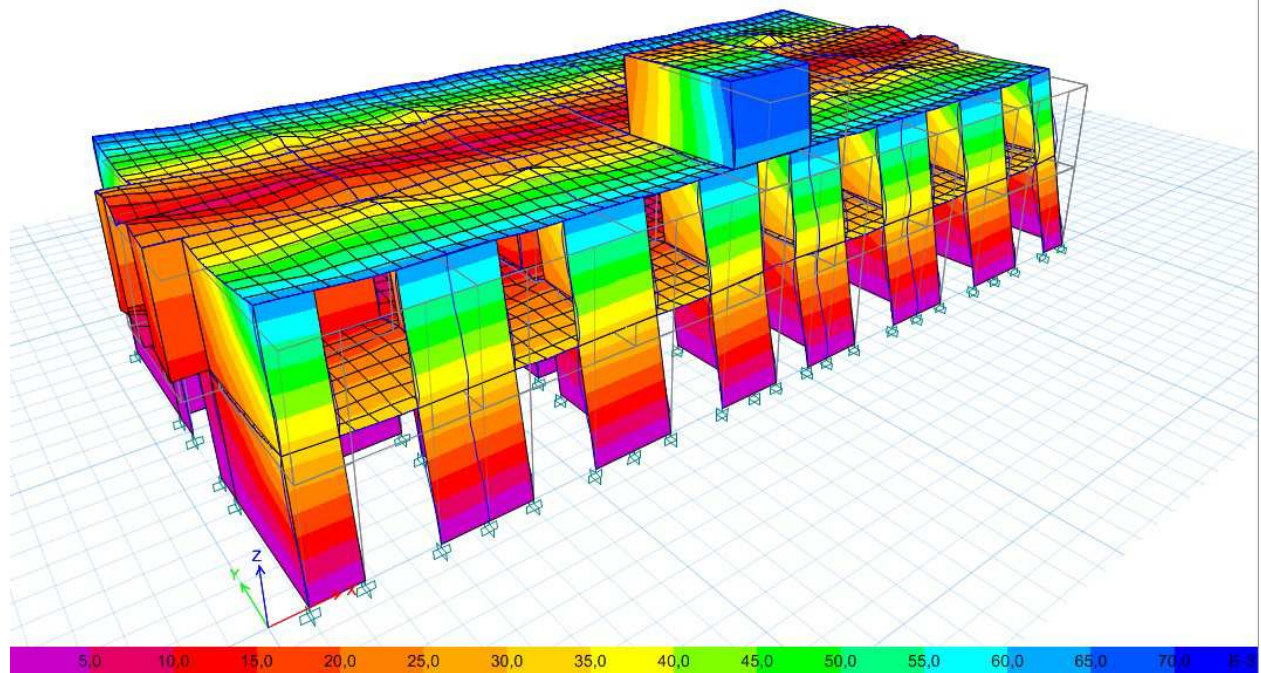


Figure 64 Displacements RS-Y

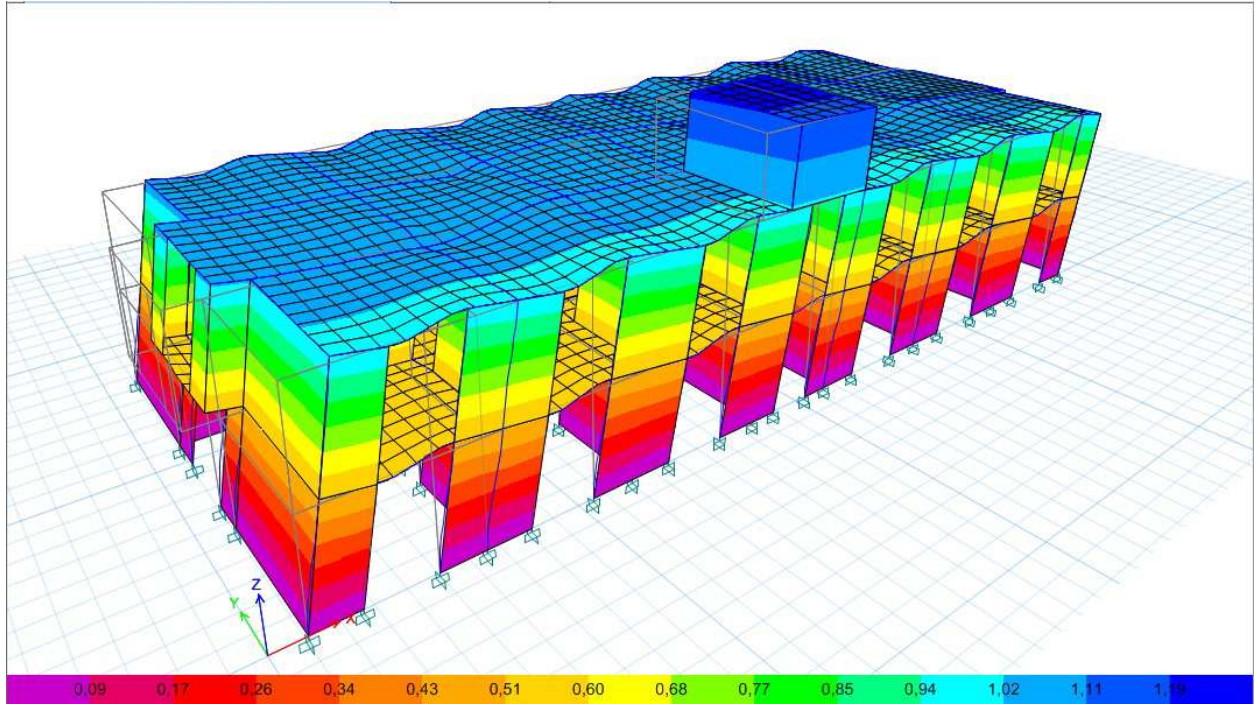


Figure 65 Displacements Seismic X+e

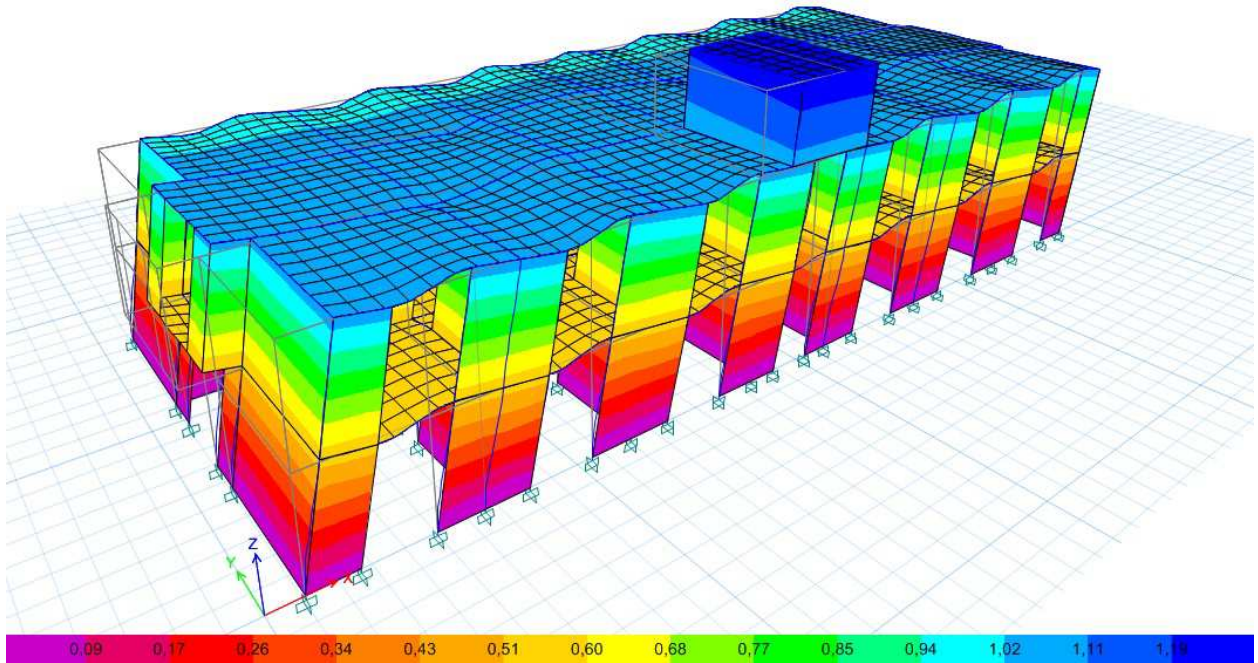


Figure 66 Displacements Seismic X-e

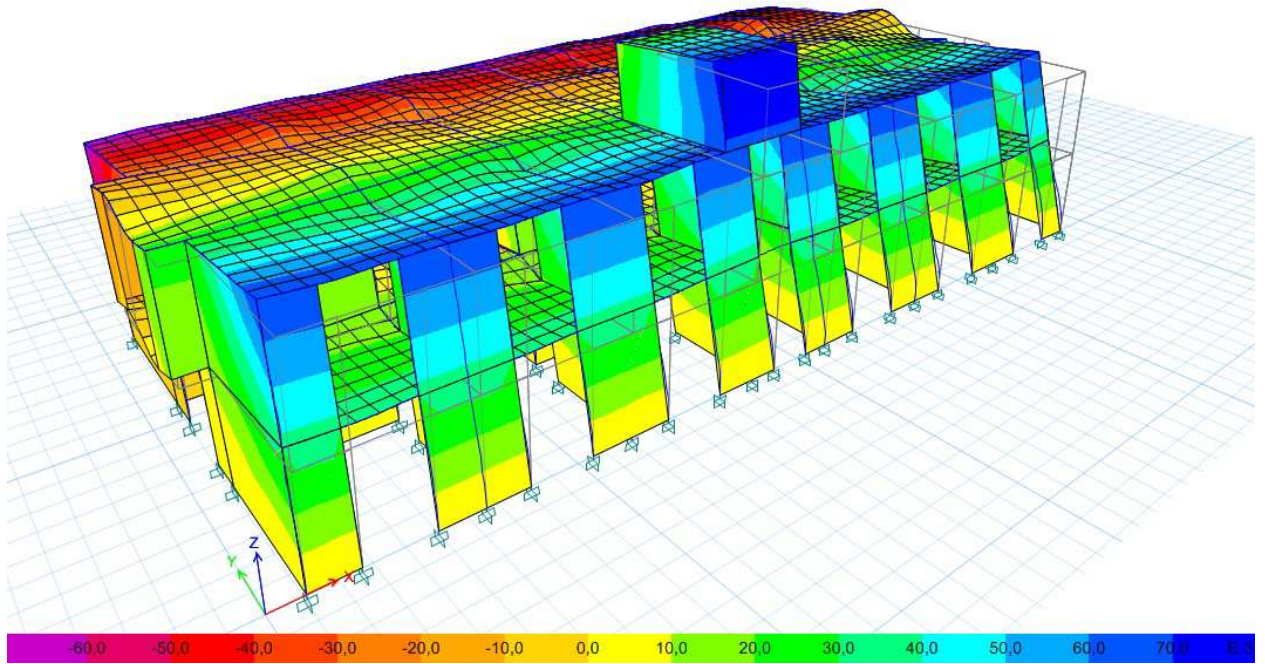


Figure 67 Displacements Seismic Y+e

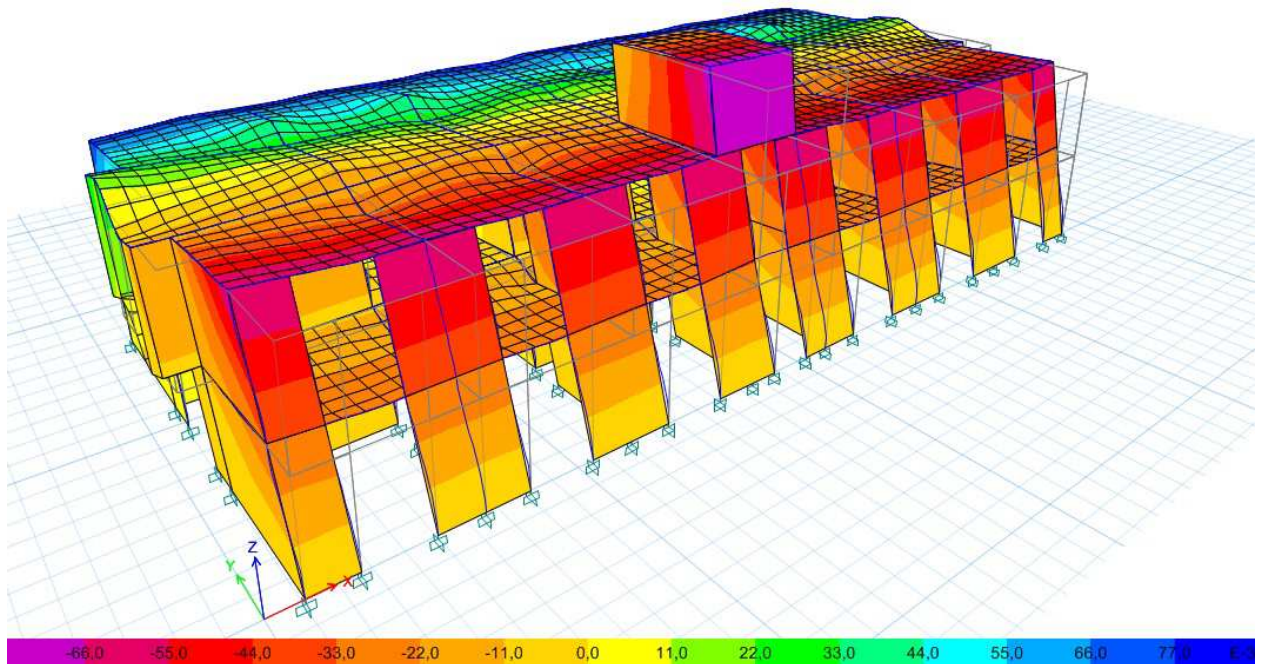
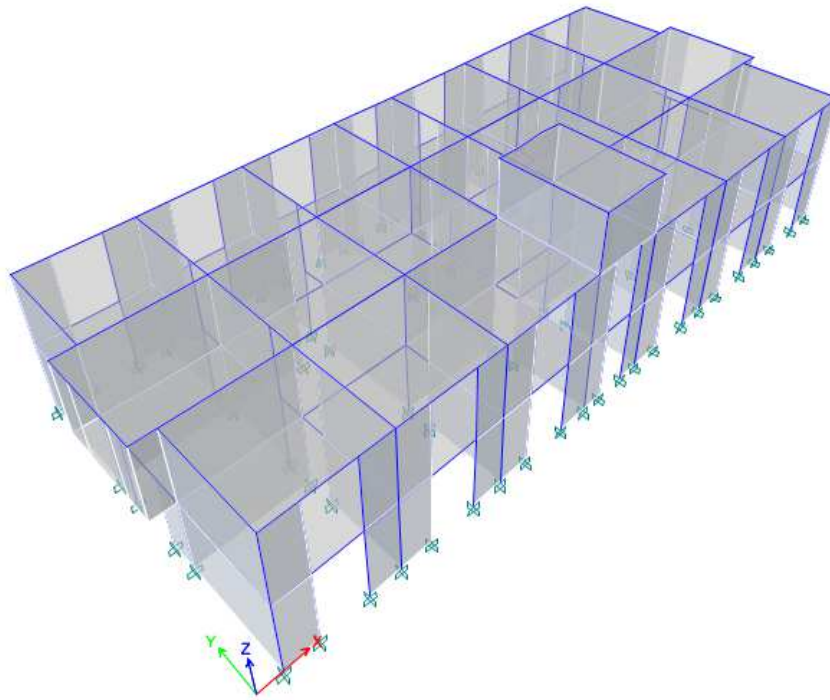


Figure 68 Displacements Seismic Y-e



## Project Report

Model File: Model etabs 5, Revision 0  
22.09.2018

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# 1 Structure Data

This chapter provides model geometry information, including items such as story levels, point coordinates, and element connectivity.

## 1.1 Story Data

Table 1.1 - Story Data

Name	Height mm	Elevation mm	Master Story	Similar To	Splice Story
Story3	2200	7800	Yes	None	No
Story2	2800	5600	Yes	None	No
Story1	2800	2800	No	None	No
Base	0	0	No	None	No

## 1.2 Mass

Table 1.2 - Mass Source

Name	Include Elements	Include Added Mass	Include Loads	Include Lateral	Include Vertical	Lump at Stories	IsDefault	Load Pattern	Multiplier
MsSrc1	Yes	No	Yes	Yes	No	Yes	Yes	Dead	1

Table 1.3 - Centers of Mass and Rigidity

Story	Diaphragm	Mass X kg	Mass Y kg	XCM m	YCM m	Cumulative X kg	Cumulative Y kg	XCCM m	YCCM m	XCR m	YCR m
Story3	D3	9527,04	9527,04	14,2	1,998	9527,04	9527,04	14,2	1,998		
Story2	D2	191213,52	191213,52	14,1919	5,929	191213,52	191213,52	14,1919	5,929		
Story1	D1	257248,32	257248,32	14,2377	6,1287	257248,32	257248,32	14,2377	6,1287		

Table 1.4 - Mass Summary by Diaphragm

Story	Diaphragm	Mass X kg	Mass Y kg	Mass Moment of Inertia tonf-m-s <sup>2</sup>	X Mass Center m	Y Mass Center m
Story3	D3	9527,04	9527,04	3,988	14,2	1,998
Story2	D2	191213,52	191213,52	1568,4847	14,1919	5,929
Story1	D1	257248,32	257248,32	2208,1082	14,2377	6,1287

Table 1.5 - Mass Summary by Story

Story	UX kg	UY kg	UZ kg
Story3	9527,04	9527,04	0
Story2	191213,52	191213,52	0
Story1	259161,84	259161,84	0
Base	61069,68	61069,68	0

## 1.3 Groups

Table 1.6 - Group Definitions

Name	Color
All	Yellow

## 2 Properties

This chapter provides property information for materials, frame sections, shell sections, and links.

### 2.1 Materials

**Table 2.1 - Material Properties - Summary**

Name	Type	E kgf/mm <sup>2</sup>	v	Unit Weight kgf/m <sup>3</sup>	Design Strengths
A416Gr270	Tendon	20037,48	0	7849,05	Fy=172,32 kgf/mm <sup>2</sup> , Fu=189,83 kgf/mm <sup>2</sup>
A615Gr60	Rebar	20389,02	0,3	7849,05	Fy=42,18 kgf/mm <sup>2</sup> , Fu=63,28 kgf/mm <sup>2</sup>
concrete210	Concrete	2154	0,2	2400	Fc=2,1 kgf/mm <sup>2</sup>
Masonry	Masonry	325	0,25	1800	

### 2.2 Frame Sections

**Table 2.2 - Frame Sections - Summary**

Name	Material	Shape
B200x200	concrete210	Concrete Rectangular
C200x200	concrete210	Concrete Rectangular

### 2.3 Shell Sections

**Table 2.3 - Shell Sections - Summary**

Name	Design Type	Element Type	Material	Total Thickness mm
Masonry140	Wall	Shell-Thin	Masonry	140
Slab200	Slab	Membrane	concrete210	50

### 3 Loads

This chapter provides loading information as applied to the model.

#### 3.1 Load Patterns

**Table 3.1 - Load Patterns**

Name	Type	Self Weight Multiplier	Auto Load
Dead weight	Dead	1	
Live Roof Lr	Roof Live	0	
Live Floor L	Live	0	
Hile Load S	Snow	0	
Wind	Wind	0	ASCE 7-10
Seismic X+e	Seismic	0	User Coefficient
Seismic X-e	Seismic	0	User Coefficient
Seismic Y+e	Seismic	0	User Coefficient
Seismic Y-e	Seismic	0	User Coefficient
Dead	Superimposed Dead	0	

#### 3.2 Auto Wind Loading

## ASCE 7-10 Auto Wind Load Calculation

This calculation presents the automatically generated lateral wind loads for load pattern Wind according to ASCE 7-10, as calculated by ETABS.

### Exposure Parameters

Exposure From = Diaphragms

Exposure Category = B

Wind Direction = 0 degrees

Basic Wind Speed, V [ASCE 26.5.1]

$V = 47 \text{ mph}$

Windward Coefficient,  $C_{p,wind}$  [ASCE 27.4.1]

$C_{p,wind} = \text{Varies}$

Leeward Coefficient,  $C_{p,lee}$  [ASCE 27.4.1]

$C_{p,lee} = \text{Varies}$

Wind Case = All Cases

Top Story = Story3

Bottom Story = Base

Include Parapet = No

### Factors and Coefficients

Gradient Height,  $z_g$  [ASCE Table 26.9-1]

$z_g = 1200$

Emperical Exponent,  $\alpha$  [ASCE Table 26.9-1]

$\alpha = 7$

Velocity Pressure Exposure Coefficient,  $K_z$  [ASCE Table 27.3-1]

$$K_z = 2.01 \left( \frac{z}{z_g} \right)^{\frac{2}{\alpha}} \text{ for } 15\text{ft} \leq z \leq z_g$$

$$K_z = 2.01 \left( \frac{15}{z_g} \right)^{\frac{2}{\alpha}} \text{ for } z < 15\text{ft}$$

Topographical Factor,  $K_{zt}$  [ASCE 26.8.2]

$K_{zt} = 1$

Directionality Factor,  $K_d$  [ASCE 26.6]

$K_d = 0,85$

Gust Effect Factor, G [ASCE 26.9]

$G = 0,85$

### Lateral Loading

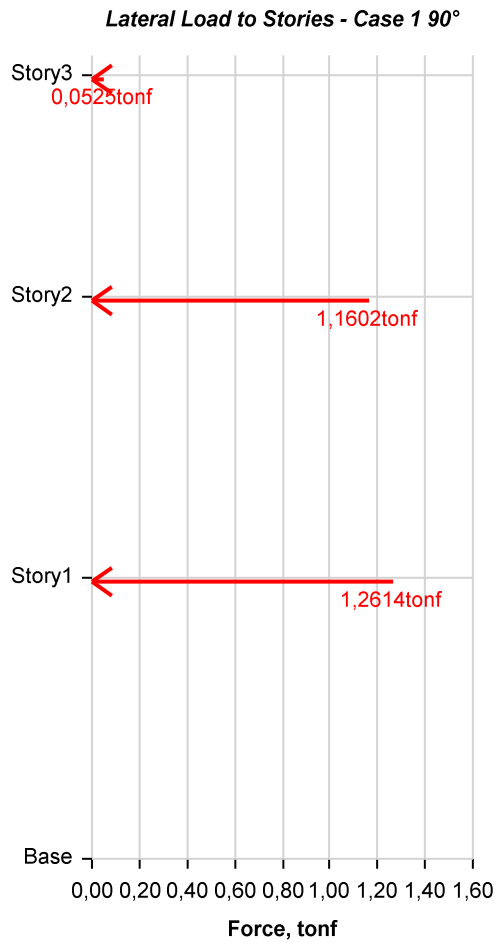
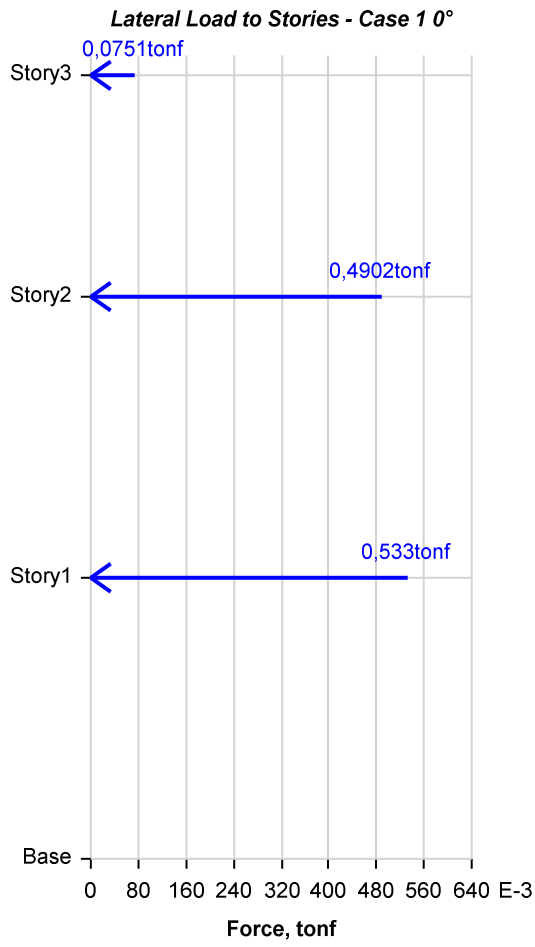
Velocity Pressure,  $q_z$  [ASCE 27.3.2 Eq. 27.3-1]

$$q_z = 0.00256 K_z K_{zt} K_d V^2$$

Design Wind Pressure, p [ASCE 27.4.2 Eq. 27.4-2]

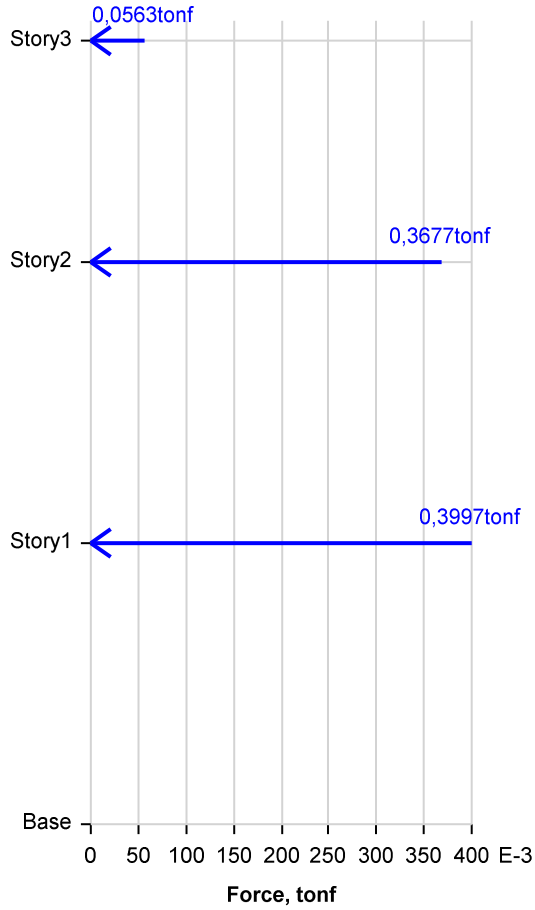
$$p = qG C_{p,wind} + q_h ( G C_{p,lee} )$$

### Applied Story Forces



Story	Elevation m	X-Dir tonf	Y-Dir tonf
Story3	7,8	0,0751	0
Story2	5,6	0,4902	0
Story1	2,8	0,533	0
Base	0	0	0

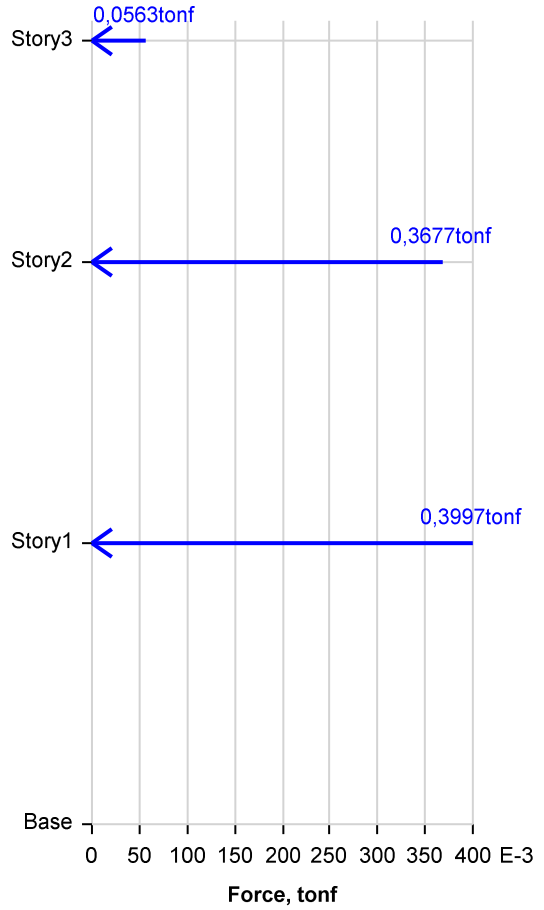
Lateral Load to Stories - Case 2 0°



Story	Elevation m	X-Dir tonf	Y-Dir tonf
Story3	7,8	0,0563	0
Story2	5,6	0,3677	0
Story1	2,8	0,3997	0
Base	0	0	0

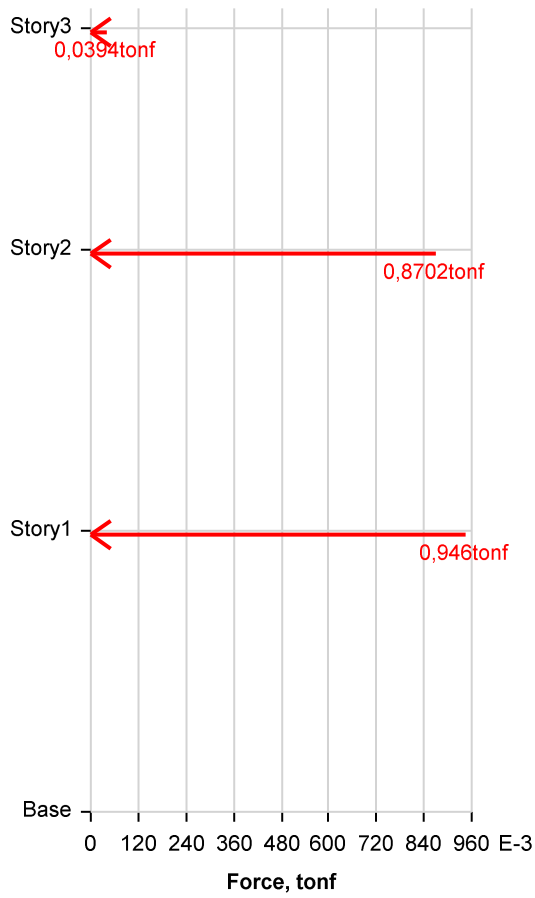
Story	Elevation m	X-Dir tonf	Y-Dir tonf
Story3	7,8	0	0,0525
Story2	5,6	0	1,1602
Story1	2,8	0	1,2614
Base	0	0	0

Lateral Load to Stories - Case 2 0°



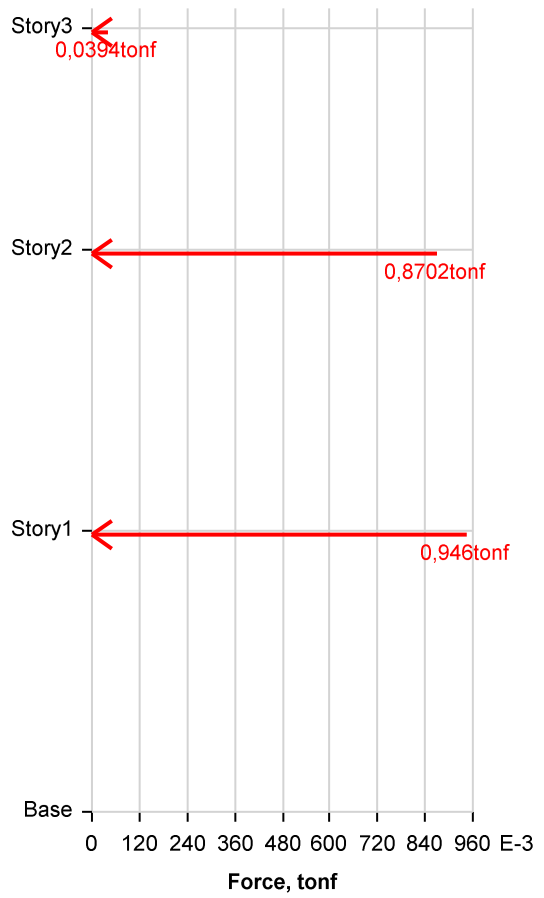
Story	Elevation m	X-Dir tonf	Y-Dir tonf
Story3	7,8	0,0563	0
Story2	5,6	0,3677	0
Story1	2,8	0,3997	0
Base	0	0	0

**Lateral Load to Stories - Case 2 90°**

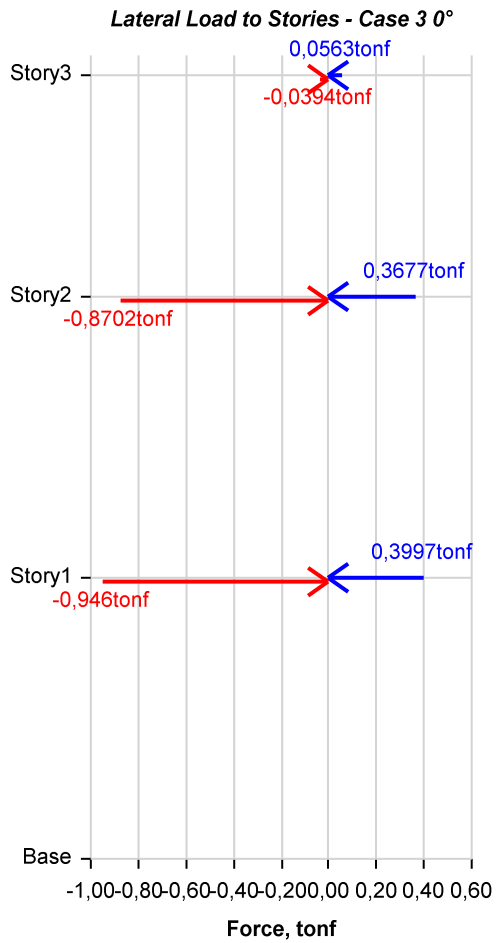


Story	Elevation m	X-Dir tonf	Y-Dir tonf
Story3	7,8	0	0,0394
Story2	5,6	0	0,8702
Story1	2,8	0	0,946
Base	0	0	0

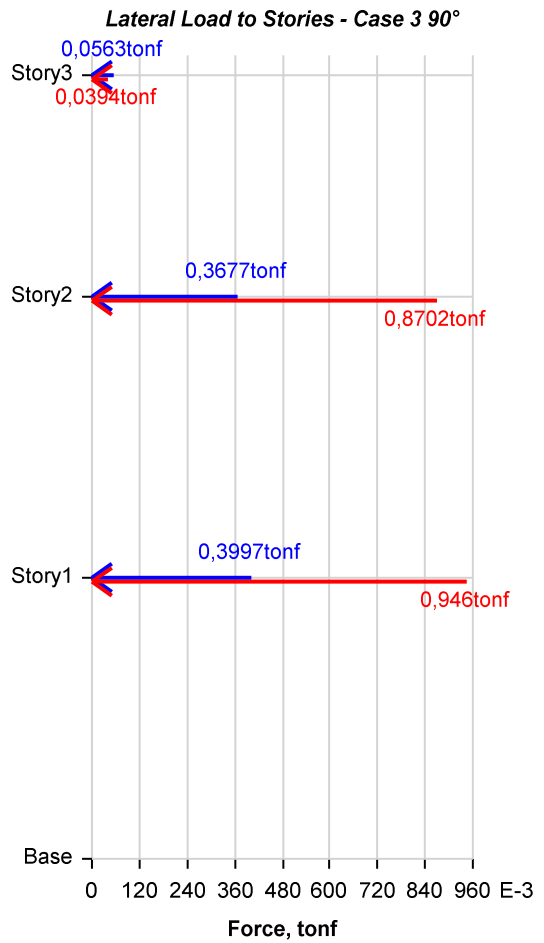
**Lateral Load to Stories - Case 2 90°**



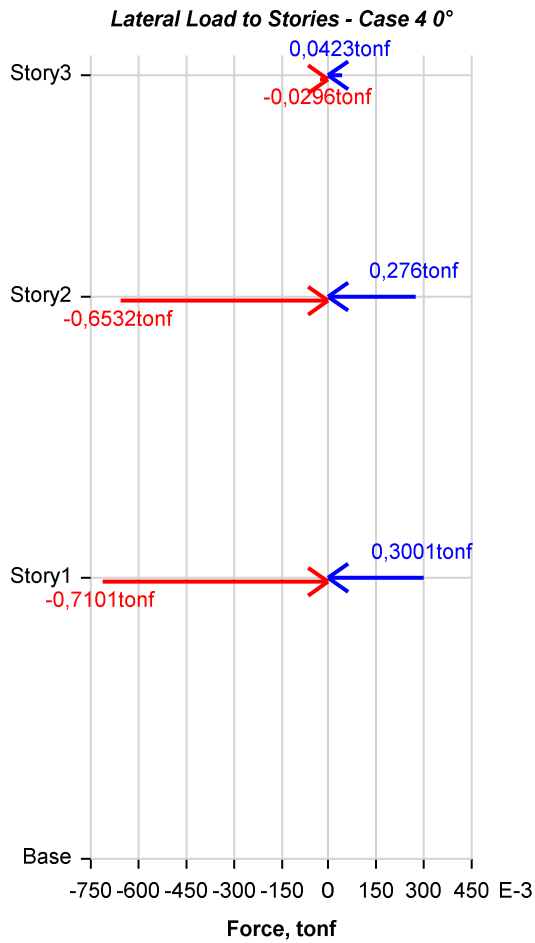
Story	Elevation m	X-Dir tonf	Y-Dir tonf
Story3	7,8	0	0,0394
Story2	5,6	0	0,8702
Story1	2,8	0	0,946
Base	0	0	0



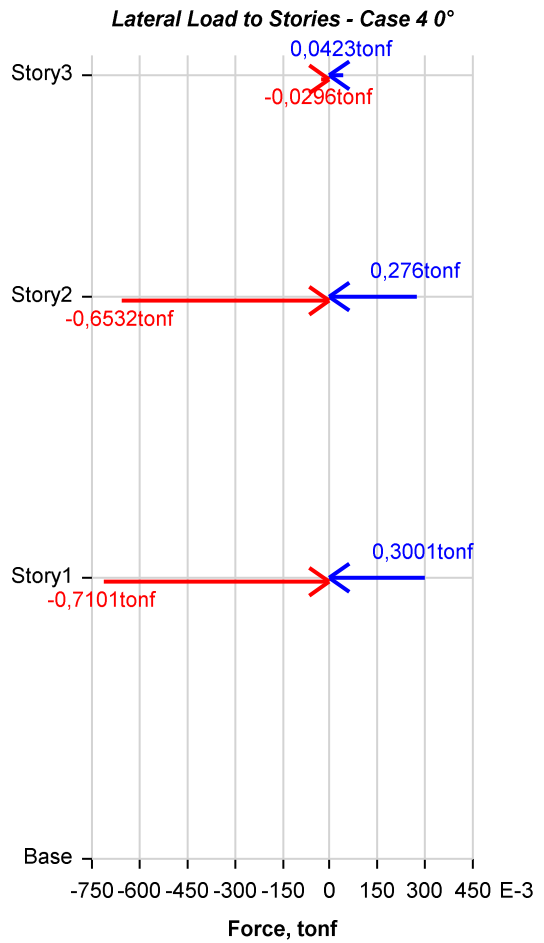
Story	Elevation m	X-Dir tonf	Y-Dir tonf
Story3	7,8	0,0563	-0,0394
Story2	5,6	0,3677	-0,8702
Story1	2,8	0,3997	-0,946
Base	0	0	0



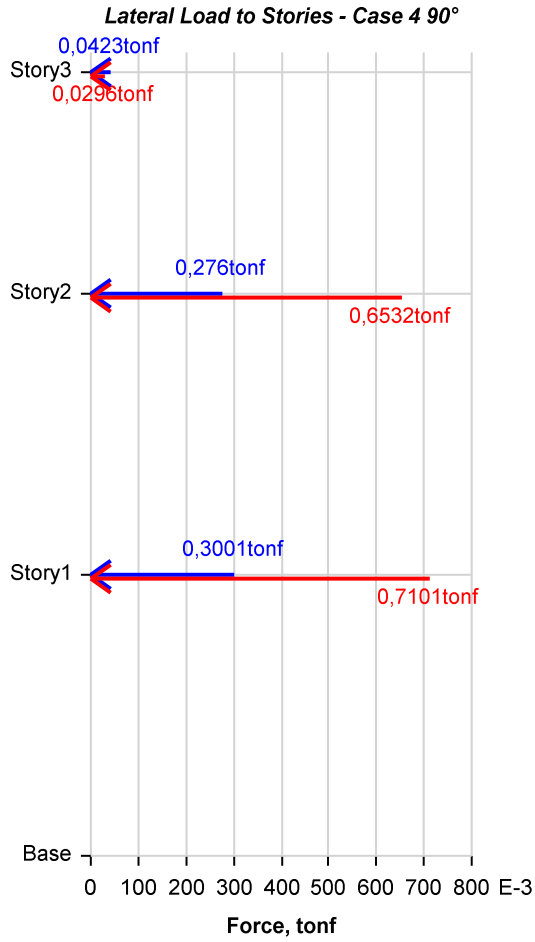
Story	Elevation m	X-Dir tonf	Y-Dir tonf
Story3	7,8	0,0563	0,0394
Story2	5,6	0,3677	0,8702
Story1	2,8	0,3997	0,946
Base	0	0	0



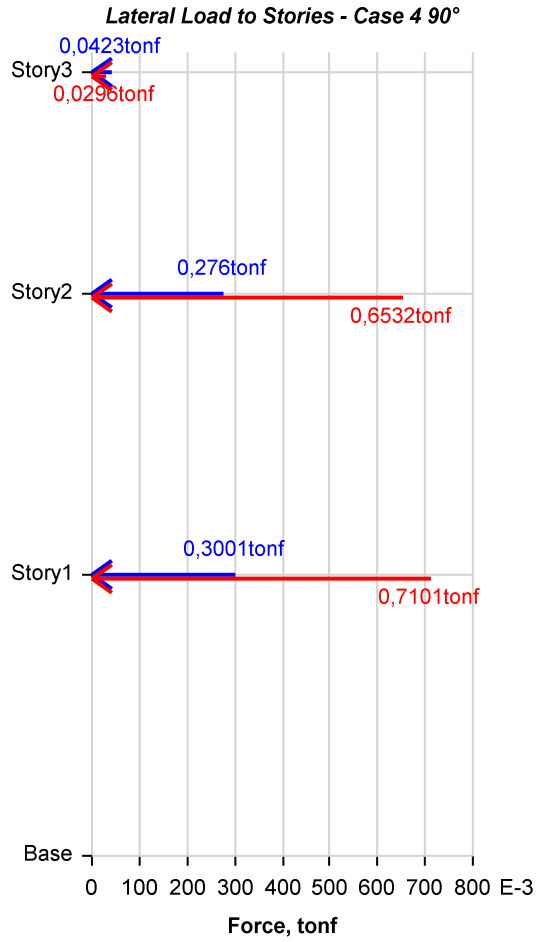
Story	Elevation m	X-Dir tonf	Y-Dir tonf
Story3	7,8	0,0423	-0,0296
Story2	5,6	0,276	-0,6532
Story1	2,8	0,3001	-0,7101
Base	0	0	0



Story	Elevation m	X-Dir tonf	Y-Dir tonf
Story3	7,8	0,0423	-0,0296
Story2	5,6	0,276	-0,6532
Story1	2,8	0,3001	-0,7101
Base	0	0	0



Story	Elevation m	X-Dir tonf	Y-Dir tonf
Story3	7,8	0,0423	0,0296
Story2	5,6	0,276	0,6532
Story1	2,8	0,3001	0,7101
Base	0	0	0



Story	Elevation m	X-Dir tonf	Y-Dir tonf
Story3	7,8	0,0423	0,0296
Story2	5,6	0,276	0,6532
Story1	2,8	0,3001	0,7101
Base	0	0	0

### **3.3 Auto Seismic Loading**

### User Coefficient Auto Seismic Load Calculation

This calculation presents the automatically generated lateral seismic loads for load pattern Seismic X+e using the user input coefficients, as calculated by ETABS.

#### Direction and Eccentricity

Direction = X + Eccentricity Y

Eccentricity Ratio = 5% for all diaphragms

#### Factors and Coefficients

##### Equivalent Lateral Forces

Base Shear Coefficient, C

$C = 0,394$

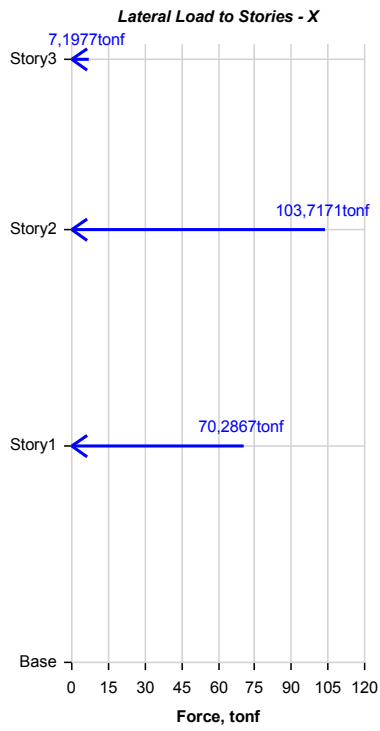
Base Shear, V

$$V = CW$$

##### Calculated Base Shear

Direction	Period Used (sec)	C	W (tonf)	V (tonf)
X + Ecc. Y	0	0	459,9024	181,2015

##### Applied Story Forces



Story	Elevation m	X-Dir tonf	Y-Dir tonf
Story3	7,8	7,1977	0
Story2	5,6	103,7171	0
Story1	2,8	70,2867	0
Base	0	0	0

### User Coefficient Auto Seismic Load Calculation

This calculation presents the automatically generated lateral seismic loads for load pattern Seismic X-e using the user input coefficients, as calculated by ETABS.

#### Direction and Eccentricity

Direction = X - Eccentricity Y

Eccentricity Ratio = 5% for all diaphragms

#### Factors and Coefficients

##### Equivalent Lateral Forces

Base Shear Coefficient, C

$$C = 0,394$$

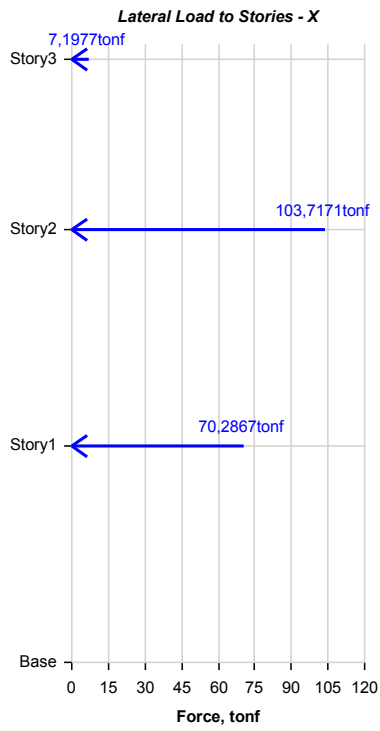
Base Shear, V

$$V = CW$$

##### Calculated Base Shear

Direction	Period Used (sec)	C	W (tonf)	V (tonf)
X - Ecc. Y	0	0	459,9024	181,2015

#### Applied Story Forces



Story	Elevation m	X-Dir tonf	Y-Dir tonf
Story3	7,8	7,1977	0
Story2	5,6	103,7171	0
Story1	2,8	70,2867	0
Base	0	0	0

### User Coefficient Auto Seismic Load Calculation

This calculation presents the automatically generated lateral seismic loads for load pattern Seismic Y+e using the user input coefficients, as calculated by ETABS.

#### Direction and Eccentricity

Direction = Y + Eccentricity X

Eccentricity Ratio = 5% for all diaphragms

#### Factors and Coefficients

##### Equivalent Lateral Forces

Base Shear Coefficient, C

$C = 0,394$

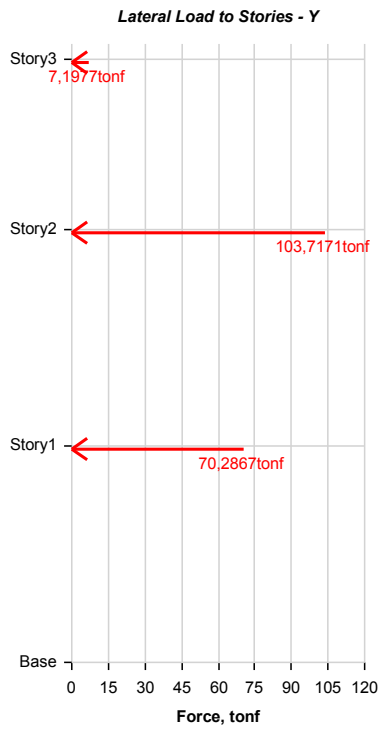
Base Shear, V

$$V = CW$$

##### Calculated Base Shear

Direction	Period Used (sec)	C	W (tonf)	V (tonf)
Y + Ecc. X	0	0	459,9024	181,2015

##### Applied Story Forces



Story	Elevation m	X-Dir tonf	Y-Dir tonf
Story3	7,8	0	7,1977
Story2	5,6	0	103,7171
Story1	2,8	0	70,2867
Base	0	0	0

### User Coefficient Auto Seismic Load Calculation

This calculation presents the automatically generated lateral seismic loads for load pattern Seismic Y-e using the user input coefficients, as calculated by ETABS.

#### Direction and Eccentricity

Direction = Y - Eccentricity X

Eccentricity Ratio = 5% for all diaphragms

#### Factors and Coefficients

##### Equivalent Lateral Forces

Base Shear Coefficient, C

$C = 0,394$

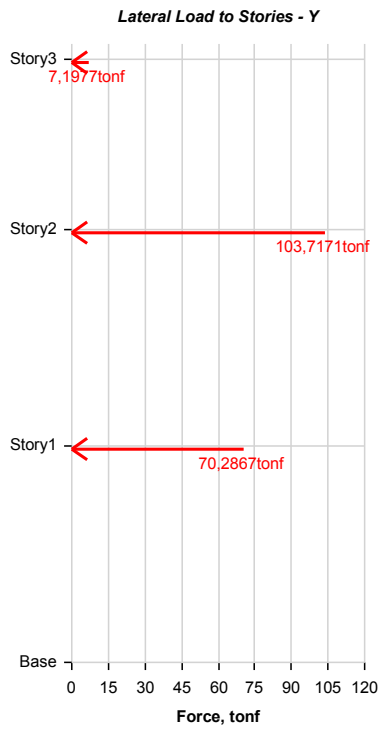
Base Shear, V

$$V = CW$$

##### Calculated Base Shear

Direction	Period Used (sec)	C	W (tonf)	V (tonf)
Y - Ecc. X	0	0	459,9024	181,2015

##### Applied Story Forces



Story	Elevation m	X-Dir tonf	Y-Dir tonf
Story3	7,8	0	7,1977
Story2	5,6	0	103,7171
Story1	2,8	0	70,2867
Base	0	0	0

3.4 Load Cases

Table 3.7 - Load Cases - Summary

Name	Type
Dead	Linear Static
Live Roof Lr	Linear Static
Live Floor L	Linear Static
Hile Load S	Linear Static
Wind X	Linear Static
Seismic X+e	Linear Static
Seismic X-e	Linear Static
Seismic Y+e	Linear Static
Seismic Y-e	Linear Static
Dead1	Linear Static
RS <sub>x</sub>	Response Spectrum
RS <sub>y</sub>	Response Spectrum
~TorsionRS <sub>x</sub>	Linear Static
~TorsionRS <sub>y</sub>	Linear Static

3.5 Load Combinations

Table 3.8 - Load Combinations

Name	Load Case/Combo	Scale Factor	Type	Auto
Comb1	Dead	1,4	Linear Add	No
Comb1	Dead1	1,4		No
Comb2	Dead	1,2	Linear Add	No
Comb2	Live Floor L	1,6		No
Comb2	Live Roof Lr	0,5		No
Comb2	Dead1	1,2		No
Comb3	Dead	1,2	Linear Add	No
Comb3	Live Floor L	1,6		No
Comb3	Hile Load S	0,5		No
Comb3	Dead1	1,2		No
Comb4	Dead	1,2	Linear Add	No
Comb4	Live Roof Lr	1,6		No
Comb4	Live Floor L	1		No
Comb4	Dead1	1,2		No
Comb5	Dead	1,2	Linear Add	No
Comb5	Hile Load S	1,6		No
Comb5	Live Floor L	1		No
Comb5	Dead1	1,2		No
Comb6	Dead	1,2	Linear Add	No
Comb6	Wind X	1		No
Comb6	Live Floor L	1		No
Comb6	Live Roof Lr	0,5		No
Comb6	Dead1	1,2		No
Comb7	Dead	1,2	Linear Add	No
Comb7	Wind X	1		No
Comb7	Live Floor L	1		No
Comb7	Hile Load S	0,5		No
Comb7	Dead1	1,2		No
Comb8	Dead	1,2	Linear Add	No

**Table 3.8 - Load Combinations (continued)**

<b>Name</b>	<b>Load Case/Combo</b>	<b>Scale Factor</b>	<b>Type</b>	<b>Auto</b>
Comb8	Seismic X+e	1		No
Comb8	Live Floor L	1		No
Comb8	Hile Load S	0,2		No
Comb8	Dead1	1,2		No
Comb9	Dead	1,2	Linear Add	No
Comb9	Seismic X-e	1		No
Comb9	Live Floor L	1		No
Comb9	Hile Load S	0,2		No
Comb9	Dead1	1,2		No
Comb10	Dead	1,2	Linear Add	No
Comb10	Seismic Y+e	1		No
Comb10	Live Floor L	1		No
Comb10	Hile Load S	0,2		No
Comb10	Dead1	1,2		No
Comb11	Dead	1,2	Linear Add	No
Comb11	Seismic Y-e	1		No
Comb11	Live Floor L	1		No
Comb11	Hile Load S	0,2		No
Comb11	Dead1	1,2		No
Comb12	Dead	0,9	Linear Add	No
Comb12	Wind X	1		No
Comb12	Dead1	0,9		No
Comb13	Dead	0,9	Linear Add	No
Comb13	Seismic X+e	1		No
Comb13	Dead1	0,9		No
Comb14	Dead	0,9	Linear Add	No
Comb14	Seismic X-e	1		No
Comb14	Dead1	0,9		No
Comb15	Dead	0,9	Linear Add	No
Comb15	Seismic Y+e	1		No
Comb15	Dead1	0,9		No
Comb16	Dead	0,9	Linear Add	No
Comb16	Seismic Y-e	1		No
Comb16	Dead1	0,9		No

## 4 Analysis Results

This chapter provides analysis results.

### 4.1 Structure Results

**Table 4.1 - Base Reactions**

Load Case/Combo	FX tonf	FY tonf	FZ tonf	MX tonf-m	MY tonf-m	MZ tonf-m	X m	Y m	Z m
Dead	0	0	791,0376	4721,1938	-11263,1516	0	0	0	0
Live Roof Lr	0	0	23,739	139,2967	-337,0937	0	0	0	0
Live Floor L	0	0	63,895	390,9154	-907,3078	0	0	0	0
Hile Load S	0	0	15,3564	86,6976	-212,6181	0	0	0	0
Wind X 1	-1,0983	0	0	0	-4,823	6,2893	0	0	0
Wind X 2	0	-2,4741	0	10,4388	0	-35,1324	0	0	0
Wind X 3	-0,8237	0	0	0	-3,6173	3,3019	0	0	0
Wind X 4	-0,8237	0	0	0	-3,6173	6,1321	0	0	0
Wind X 5	0	-1,8556	0	7,8291	0	-34,1028	0	0	0
Wind X 6	0	-1,8556	0	7,8291	0	-18,5958	0	0	0
Wind X 7	-0,8237	1,8556	0	-7,8291	-3,6173	31,0663	0	0	0
Wind X 8	-0,8237	-1,8556	0	7,8291	-3,6173	-21,6323	0	0	0
Wind X 9	-0,6183	1,3929	0	-5,8771	-2,7154	16,4379	0	0	0
Wind X 10	-0,6183	1,3929	0	-5,8771	-2,7154	30,203	0	0	0
Wind X 11	-0,6183	-1,3929	0	5,8771	-2,7154	-23,1212	0	0	0
Wind X 12	-0,6183	-1,3929	0	5,8771	-2,7154	-9,3561	0	0	0
Seismic X+e	-181,2015	0	0	0	-833,761	1162,438	0	0	0
Seismic X-e	-181,2015	0	0	0	-833,761	951,3771	0	0	0
Seismic Y+e	0	-181,2015	0	833,761	0	-2822,2126	0	0	0
Seismic Y-e	0	-181,2015	0	833,761	0	-2327,5003	0	0	0
Dead1	0	0	27,2875	160,2069	-376,1165	0	0	0	0
RS <sub>x</sub> Max	149,5253	1,5761	0	7,2244	677,9758	957,3726	0	0	0
RS <sub>y</sub> Max	1,5761	150,2543	0	680,1748	6,9923	2347,0485	0	0	0
Comb1	0	0	1145,655	6833,961	-16294,9754	0	0	0	0
Comb2	0	0	1096,0915	6552,7939	-15587,3612	0	0	0	0
Comb3	0	0	1091,9002	6526,4944	-15525,1234	0	0	0	0
Comb4	0	0	1083,8674	6471,471	-15413,7795	0	0	0	0
Comb5	0	0	1070,4553	6387,3125	-15214,6185	0	0	0	0
Comb6 Max	0	1,8556	1057,7545	6328,6834	-15042,9765	31,0663	0	0	0
Comb6 Min	-1,0983	-2,4741	1057,7545	6310,4155	-15047,7995	-35,1324	0	0	0
Comb7 Max	0	1,8556	1053,5632	6302,3839	-14980,7387	31,0663	0	0	0
Comb7 Min	-1,0983	-2,4741	1053,5632	6284,116	-14985,5617	-35,1324	0	0	0
Comb8	-181,2015	0	1048,9563	6265,9358	-15750,7143	1162,438	0	0	0
Comb9	-181,2015	0	1048,9563	6265,9358	-15750,7143	951,3771	0	0	0
Comb10	0	-181,2015	1048,9563	7099,6968	-14916,9533	-2822,2126	0	0	0
Comb11	0	-181,2015	1048,9563	7099,6968	-14916,9533	-2327,5003	0	0	0
Comb12 Max	0	1,8556	736,4925	4403,6994	-10475,3414	31,0663	0	0	0
Comb12 Min	-1,0983	-2,4741	736,4925	4385,4315	-10480,1644	-35,1324	0	0	0
Comb13	-181,2015	0	736,4925	4393,2606	-11309,1024	1162,438	0	0	0
Comb14	-181,2015	0	736,4925	4393,2606	-11309,1024	951,3771	0	0	0
Comb15	0	-181,2015	736,4925	5227,0216	-10475,3414	-2822,2126	0	0	0
Comb16	0	-181,2015	736,4925	5227,0216	-10475,3414	-2327,5003	0	0	0

**Table 4.2 - Centers of Mass and Rigidity**

Story	Diaphragm	Mass X kg	Mass Y kg	XCM m	YCM m	Cumulative X kg	Cumulative Y kg	XCCM m	YCCM m	XCR m	YCR m
Story3	D3	9527,04	9527,04	14,2	1,998	9527,04	9527,04	14,2	1,998		
Story2	D2	191213,52	191213,52	14,1919	5,929	191213,52	191213,52	14,1919	5,929		
Story1	D1	257248,32	257248,32	14,2377	6,1287	257248,32	257248,32	14,2377	6,1287		

**Table 4.3 - Diaphragm Center of Mass Displacements**

Story	Diaphragm	Load Case/Combo	UX mm	UY mm	RZ rad	Point	X m	Y m	Z m
Story3	D3	Dead	-0,001	0,061	-3,217E-08	21	14,2	1,998	7,8
Story3	D3	Live Roof Lr	-7,887E-05	0,005	-8,039E-09	21	14,2	1,998	7,8
Story3	D3	Live Floor L	-9,139E-06	0,007	-9,217E-09	21	14,2	1,998	7,8
Story3	D3	Hile Load S	-0,002	0,002	9,072E-08	21	14,2	1,998	7,8
Story3	D3	Wind X 1	0,008	1,725E-05	2,976E-08	21	14,2	1,998	7,8
Story3	D3	Wind X 2	2,979E-05	0,01	-2,342E-09	21	14,2	1,998	7,8
Story3	D3	Wind X 3	0,006	1,142E-05	1,869E-07	21	14,2	1,998	7,8
Story3	D3	Wind X 4	0,006	1,446E-05	-1,423E-07	21	14,2	1,998	7,8
Story3	D3	Wind X 5	0,001	0,008	3,881E-07	21	14,2	1,998	7,8
Story3	D3	Wind X 6	-0,001	0,008	-3,916E-07	21	14,2	1,998	7,8
Story3	D3	Wind X 7	0,006	-0,008	2,408E-08	21	14,2	1,998	7,8
Story3	D3	Wind X 8	0,006	0,008	2,056E-08	21	14,2	1,998	7,8
Story3	D3	Wind X 9	0,005	-0,006	4,343E-07	21	14,2	1,998	7,8
Story3	D3	Wind X 10	0,003	-0,006	-3,981E-07	21	14,2	1,998	7,8
Story3	D3	Wind X 11	0,006	0,006	4,316E-07	21	14,2	1,998	7,8
Story3	D3	Wind X 12	0,003	0,006	-4,008E-07	21	14,2	1,998	7,8
Story3	D3	Seismic X+e	1,194	0,003	-5E-06	21	14,2	1,998	7,8
Story3	D3	Seismic X-e	1,231	0,003	1,4E-05	21	14,2	1,998	7,8
Story3	D3	Seismic Y+e	0,045	0,894	1,5E-05	21	14,2	1,998	7,8
Story3	D3	Seismic Y-e	-0,041	0,895	-1,5E-05	21	14,2	1,998	7,8
Story3	D3	Dead1	0,001	0,003	-2,934E-08	21	14,2	1,998	7,8
Story3	D3	RS <sub>x</sub> Max	0,976	0,01	1,1E-05	21	14,2	1,998	7,8
Story3	D3	RS <sub>y</sub> Max	0,044	0,718	1,3E-05	21	14,2	1,998	7,8
Story3	D3	Comb1	-0,0003489	0,089	-8,611E-08	21	14,2	1,998	7,8
Story3	D3	Comb2	-0,0003532	0,089	-9,258E-08	21	14,2	1,998	7,8
Story3	D3	Comb3	-0,001	0,088	-4,319E-08	21	14,2	1,998	7,8
Story3	D3	Comb4	-0,0004344	0,091	-9,589E-08	21	14,2	1,998	7,8
Story3	D3	Comb5	-0,004	0,086	6,213E-08	21	14,2	1,998	7,8
Story3	D3	Comb6 Max	0,007	0,095	3,472E-07	21	14,2	1,998	7,8
Story3	D3	Comb6 Min	-0,002	0,077	-4,878E-07	21	14,2	1,998	7,8
Story3	D3	Comb7 Max	0,006	0,094	3,966E-07	21	14,2	1,998	7,8
Story3	D3	Comb7 Min	-0,003	0,076	-4,384E-07	21	14,2	1,998	7,8
Story3	D3	Comb8	1,193	0,086	-5E-06	21	14,2	1,998	7,8
Story3	D3	Comb9	1,23	0,086	1,4E-05	21	14,2	1,998	7,8
Story3	D3	Comb10	0,045	0,977	1,5E-05	21	14,2	1,998	7,8
Story3	D3	Comb11	-0,041	0,978	-1,5E-05	21	14,2	1,998	7,8
Story3	D3	Comb12 Max	0,008	0,067	3,789E-07	21	14,2	1,998	7,8
Story3	D3	Comb12 Min	-0,001	0,049	-4,561E-07	21	14,2	1,998	7,8
Story3	D3	Comb13	1,194	0,06	-5E-06	21	14,2	1,998	7,8
Story3	D3	Comb14	1,231	0,06	1,4E-05	21	14,2	1,998	7,8
Story3	D3	Comb15	0,045	0,951	1,5E-05	21	14,2	1,998	7,8

**Table 4.3 - Diaphragm Center of Mass Displacements (continued)**

Story	Diaphragm	Load Case/Combo	UX mm	UY mm	RZ rad	Point	X m	Y m	Z m
Story3	D3	Comb16	-0,041	0,952	-1,5E-05	21	14,2	1,998	7,8
Story2	D2	Dead	-0,001	-0,005	-2,559E-08	22	14,1919	5,929	5,6
Story2	D2	Live Roof Lr	-1,489E-05	-0,0002115	-8,91E-09	22	14,1919	5,929	5,6
Story2	D2	Live Floor L	-9,731E-05	0,001	-3,69E-09	22	14,1919	5,929	5,6
Story2	D2	Hile Load S	-0,001	-0,0003177	-6,204E-09	22	14,1919	5,929	5,6
Story2	D2	Wind X 1	0,006	1,6E-05	6,909E-09	22	14,1919	5,929	5,6
Story2	D2	Wind X 2	3,535E-05	0,009	-2,334E-09	22	14,1919	5,929	5,6
Story2	D2	Wind X 3	0,004	1,011E-05	6,678E-08	22	14,1919	5,929	5,6
Story2	D2	Wind X 4	0,004	1,389E-05	-5,641E-08	22	14,1919	5,929	5,6
Story2	D2	Wind X 5	-2,277E-05	0,007	3,324E-07	22	14,1919	5,929	5,6
Story2	D2	Wind X 6	7,579E-05	0,007	-3,359E-07	22	14,1919	5,929	5,6
Story2	D2	Wind X 7	0,004	-0,007	6,933E-09	22	14,1919	5,929	5,6
Story2	D2	Wind X 8	0,004	0,007	3,431E-09	22	14,1919	5,929	5,6
Story2	D2	Wind X 9	0,003	-0,005	3,022E-07	22	14,1919	5,929	5,6
Story2	D2	Wind X 10	0,003	-0,005	-2,918E-07	22	14,1919	5,929	5,6
Story2	D2	Wind X 11	0,003	0,005	2,996E-07	22	14,1919	5,929	5,6
Story2	D2	Wind X 12	0,003	0,005	-2,945E-07	22	14,1919	5,929	5,6
Story2	D2	Seismic X+e	1,043	0,003	-5E-06	22	14,1919	5,929	5,6
Story2	D2	Seismic X-e	1,041	0,003	5E-06	22	14,1919	5,929	5,6
Story2	D2	Seismic Y+e	0,001	0,732	1,1E-05	22	14,1919	5,929	5,6
Story2	D2	Seismic Y-e	0,005	0,733	-1,2E-05	22	14,1919	5,929	5,6
Story2	D2	Dead1	0,0002825	3,068E-06	-7,976E-09	22	14,1919	5,929	5,6
Story2	D2	RS <sub>x</sub> Max	0,851	0,008	5E-06	22	14,1919	5,929	5,6
Story2	D2	RS <sub>y</sub> Max	0,009	0,598	1E-05	22	14,1919	5,929	5,6
Story2	D2	Comb1	-0,0004102	-0,006	-4,7E-08	22	14,1919	5,929	5,6
Story2	D2	Comb2	-0,001	-0,004	-5,064E-08	22	14,1919	5,929	5,6
Story2	D2	Comb3	-0,001	-0,004	-4,929E-08	22	14,1919	5,929	5,6
Story2	D2	Comb4	-0,0004727	-0,005	-5,823E-08	22	14,1919	5,929	5,6
Story2	D2	Comb5	-0,001	-0,005	-5,39E-08	22	14,1919	5,929	5,6
Story2	D2	Comb6 Max	0,005	0,005	2,839E-07	22	14,1919	5,929	5,6
Story2	D2	Comb6 Min	-0,0004791	-0,011	-3,843E-07	22	14,1919	5,929	5,6
Story2	D2	Comb7 Max	0,005	0,005	2,853E-07	22	14,1919	5,929	5,6
Story2	D2	Comb7 Min	-0,001	-0,011	-3,829E-07	22	14,1919	5,929	5,6
Story2	D2	Comb8	1,042	-0,001	-5E-06	22	14,1919	5,929	5,6
Story2	D2	Comb9	1,041	-0,002	5E-06	22	14,1919	5,929	5,6
Story2	D2	Comb10	0,001	0,727	1,1E-05	22	14,1919	5,929	5,6
Story2	D2	Comb11	0,004	0,728	-1,2E-05	22	14,1919	5,929	5,6
Story2	D2	Comb12 Max	0,006	0,005	3,021E-07	22	14,1919	5,929	5,6
Story2	D2	Comb12 Min	-0,0002865	-0,011	-3,661E-07	22	14,1919	5,929	5,6
Story2	D2	Comb13	1,043	-0,001	-5E-06	22	14,1919	5,929	5,6
Story2	D2	Comb14	1,041	-0,001	5E-06	22	14,1919	5,929	5,6
Story2	D2	Comb15	0,001	0,728	1,1E-05	22	14,1919	5,929	5,6
Story2	D2	Comb16	0,005	0,729	-1,2E-05	22	14,1919	5,929	5,6
Story1	D1	Dead	0,0001058	-0,0003354	7,382E-08	26	14,2377	6,1287	2,8
Story1	D1	Live Roof Lr	-9,323E-07	-5,209E-05	3,155E-09	26	14,2377	6,1287	2,8
Story1	D1	Live Floor L	-2,381E-05	0,001	6,468E-09	26	14,2377	6,1287	2,8
Story1	D1	Hile Load S	-0,0001357	-8,331E-05	1,36E-09	26	14,2377	6,1287	2,8
Story1	D1	Wind X 1	0,003	4,412E-06	4,763E-09	26	14,2377	6,1287	2,8
Story1	D1	Wind X 2	9,601E-06	0,005	1,595E-09	26	14,2377	6,1287	2,8

**Table 4.3 - Diaphragm Center of Mass Displacements (continued)**

Story	Diaphragm	Load Case/Combo	UX mm	UY mm	RZ rad	Point	X m	Y m	Z m
Story1	D1	Wind X 3	0,002	5,547E-06	3,726E-08	26	14,2377	6,1287	2,8
Story1	D1	Wind X 4	0,002	1,071E-06	-3,011E-08	26	14,2377	6,1287	2,8
Story1	D1	Wind X 5	-3,719E-05	0,004	1,854E-07	26	14,2377	6,1287	2,8
Story1	D1	Wind X 6	5,159E-05	0,004	-1,83E-07	26	14,2377	6,1287	2,8
Story1	D1	Wind X 7	0,002	-0,004	2,377E-09	26	14,2377	6,1287	2,8
Story1	D1	Wind X 8	0,002	0,004	4,768E-09	26	14,2377	6,1287	2,8
Story1	D1	Wind X 9	0,002	-0,003	1,653E-07	26	14,2377	6,1287	2,8
Story1	D1	Wind X 10	0,002	-0,003	-1,618E-07	26	14,2377	6,1287	2,8
Story1	D1	Wind X 11	0,002	0,003	1,671E-07	26	14,2377	6,1287	2,8
Story1	D1	Wind X 12	0,002	0,003	-1,6E-07	26	14,2377	6,1287	2,8
Story1	D1	Seismic X+e	0,532	0,001	-2E-06	26	14,2377	6,1287	2,8
Story1	D1	Seismic X-e	0,531	0,001	3E-06	26	14,2377	6,1287	2,8
Story1	D1	Seismic Y+e	-0,001	0,382	6E-06	26	14,2377	6,1287	2,8
Story1	D1	Seismic Y-e	0,002	0,381	-6E-06	26	14,2377	6,1287	2,8
Story1	D1	Dead1	8,41E-05	6,919E-05	1,359E-09	26	14,2377	6,1287	2,8
Story1	D1	RS <sub>x</sub> Max	0,437	0,004	3E-06	26	14,2377	6,1287	2,8
Story1	D1	RS <sub>y</sub> Max	0,005	0,315	5E-06	26	14,2377	6,1287	2,8
Story1	D1	Comb1	0,0002658	-0,0003727	1,052E-07	26	14,2377	6,1287	2,8
Story1	D1	Comb2	0,0001893	0,0004896	1,021E-07	26	14,2377	6,1287	2,8
Story1	D1	Comb3	0,0001219	0,000474	1,012E-07	26	14,2377	6,1287	2,8
Story1	D1	Comb4	0,0002026	0,0001191	1,017E-07	26	14,2377	6,1287	2,8
Story1	D1	Comb5	-1,301E-05	6,917E-05	9,886E-08	26	14,2377	6,1287	2,8
Story1	D1	Comb6 Max	0,003	0,005	2,836E-07	26	14,2377	6,1287	2,8
Story1	D1	Comb6 Min	0,0001664	-0,004	-8,472E-08	26	14,2377	6,1287	2,8
Story1	D1	Comb7 Max	0,003	0,005	2,827E-07	26	14,2377	6,1287	2,8
Story1	D1	Comb7 Min	9,904E-05	-0,004	-8,562E-08	26	14,2377	6,1287	2,8
Story1	D1	Comb8	0,532	0,001	-2E-06	26	14,2377	6,1287	2,8
Story1	D1	Comb9	0,531	0,001	3E-06	26	14,2377	6,1287	2,8
Story1	D1	Comb10	-0,001	0,382	6E-06	26	14,2377	6,1287	2,8
Story1	D1	Comb11	0,003	0,381	-6E-06	26	14,2377	6,1287	2,8
Story1	D1	Comb12 Max	0,003	0,005	2,53E-07	26	14,2377	6,1287	2,8
Story1	D1	Comb12 Min	0,0001337	-0,004	-1,153E-07	26	14,2377	6,1287	2,8
Story1	D1	Comb13	0,532	0,0003465	-2E-06	26	14,2377	6,1287	2,8
Story1	D1	Comb14	0,531	0,001	3E-06	26	14,2377	6,1287	2,8
Story1	D1	Comb15	-0,001	0,382	6E-06	26	14,2377	6,1287	2,8
Story1	D1	Comb16	0,002	0,381	-6E-06	26	14,2377	6,1287	2,8

**4.2 Story Results**

**Table 4.4 - Story Max/Avg Displacements**

Story	Load Case/Combo	Direction	Maximum mm	Average mm	Ratio
Story3	Dead	Y	0,061	0,061	1,001
Story2	Dead	X	0,001	0,001	1,268
Story2	Dead	Y	0,005	0,005	1,08
Story1	Dead	X	0,001	0,0001153	4,841
Story1	Dead	Y	0,001	0,0003382	4,099
Story3	Live Roof Lr	Y	0,005	0,005	1,002
Story2	Live Roof Lr	Y	0,0003381	0,0002116	1,598
Story1	Live Roof Lr	Y	9,701E-05	5,221E-05	1,858

**Table 4.4 - Story Max/Avg Displacements (continued)**

Story	Load Case/Combo	Direction	Maximum mm	Average mm	Ratio
Story3	Live Floor L	Y	0,007	0,007	1,002
Story2	Live Floor L	Y	0,001	0,001	1,046
Story1	Live Floor L	Y	0,001	0,001	1,176
Story3	Hile Load S	X	0,002	0,002	1,079
Story3	Hile Load S	Y	0,002	0,002	1,054
Story2	Hile Load S	X	0,001	0,001	1,058
Story2	Hile Load S	Y	0,0004059	0,0003178	1,277
Story1	Hile Load S	X	0,0001437	0,0001355	1,06
Story1	Hile Load S	Y	0,0001027	8,336E-05	1,232
Story3	Wind X 1	X	0,008	0,008	1,008
Story2	Wind X 1	X	0,006	0,006	1,007
Story1	Wind X 1	X	0,003	0,003	1,009
Story3	Wind X 2	Y	0,01	0,01	1
Story2	Wind X 2	Y	0,009	0,009	1,004
Story1	Wind X 2	Y	0,005	0,005	1,004
Story3	Wind X 3	X	0,006	0,006	1,061
Story2	Wind X 3	X	0,005	0,004	1,092
Story1	Wind X 3	X	0,003	0,002	1,095
Story3	Wind X 4	X	0,006	0,006	1,05
Story2	Wind X 4	X	0,005	0,004	1,077
Story1	Wind X 4	X	0,003	0,002	1,076
Story3	Wind X 5	X	0,002	0,001	1,621
Story3	Wind X 5	Y	0,008	0,008	1,07
Story2	Wind X 5	Y	0,012	0,007	1,693
Story1	Wind X 5	Y	0,006	0,004	1,685
Story3	Wind X 6	X	0,002	0,001	1,65
Story3	Wind X 6	Y	0,008	0,008	1,07
Story2	Wind X 6	Y	0,012	0,007	1,698
Story1	Wind X 6	Y	0,006	0,004	1,678
Story3	Wind X 7	X	0,006	0,006	1,008
Story3	Wind X 7	Y	0,008	0,008	1,004
Story2	Wind X 7	X	0,004	0,004	1,01
Story2	Wind X 7	Y	0,007	0,007	1,014
Story1	Wind X 7	X	0,002	0,002	1,006
Story1	Wind X 7	Y	0,004	0,004	1,009
Story3	Wind X 8	X	0,006	0,006	1,007
Story3	Wind X 8	Y	0,008	0,008	1,004
Story2	Wind X 8	X	0,004	0,004	1,005
Story2	Wind X 8	Y	0,007	0,007	1,007
Story1	Wind X 8	X	0,002	0,002	1,012
Story1	Wind X 8	Y	0,004	0,004	1,018
Story3	Wind X 9	X	0,006	0,005	1,158
Story3	Wind X 9	Y	0,006	0,006	1,104
Story2	Wind X 9	X	0,005	0,003	1,565
Story2	Wind X 9	Y	0,009	0,005	1,839
Story1	Wind X 9	X	0,003	0,002	1,567
Story1	Wind X 9	Y	0,005	0,003	1,817
Story3	Wind X 10	X	0,004	0,003	1,241
Story3	Wind X 10	Y	0,006	0,006	1,096

**Table 4.4 - Story Max/Avg Displacements (continued)**

Story	Load Case/Combo	Direction	Maximum mm	Average mm	Ratio
Story2	Wind X 10	X	0,005	0,003	1,524
Story2	Wind X 10	Y	0,009	0,005	1,812
Story1	Wind X 10	X	0,003	0,002	1,543
Story1	Wind X 10	Y	0,005	0,003	1,797
Story3	Wind X 11	X	0,006	0,006	1,156
Story3	Wind X 11	Y	0,006	0,006	1,103
Story2	Wind X 11	X	0,005	0,003	1,553
Story2	Wind X 11	Y	0,009	0,005	1,83
Story1	Wind X 11	X	0,003	0,002	1,569
Story1	Wind X 11	Y	0,005	0,003	1,822
Story3	Wind X 12	X	0,004	0,003	1,24
Story3	Wind X 12	Y	0,006	0,006	1,096
Story2	Wind X 12	X	0,005	0,003	1,523
Story2	Wind X 12	Y	0,009	0,005	1,814
Story1	Wind X 12	X	0,003	0,002	1,534
Story1	Wind X 12	Y	0,005	0,003	1,789
Story3	Seismic X+e	X	1,204	1,194	1,009
Story2	Seismic X+e	X	1,073	1,043	1,029
Story1	Seismic X+e	X	0,545	0,532	1,026
Story3	Seismic X-e	X	1,258	1,231	1,022
Story2	Seismic X-e	X	1,071	1,041	1,029
Story1	Seismic X-e	X	0,548	0,531	1,032
Story3	Seismic Y+e	Y	0,915	0,894	1,023
Story2	Seismic Y+e	Y	0,895	0,732	1,223
Story1	Seismic Y+e	Y	0,469	0,382	1,229
Story3	Seismic Y-e	Y	0,916	0,895	1,024
Story2	Seismic Y-e	Y	0,902	0,733	1,232
Story1	Seismic Y-e	Y	0,464	0,381	1,217
Story3	Dead1	X	0,001	0,001	1,117
Story3	Dead1	Y	0,003	0,003	1,015
Story2	Dead1	X	0,0003309	0,000283	1,169
Story1	Dead1	X	9,243E-05	8,427E-05	1,097
Story1	Dead1	Y	8,844E-05	6,914E-05	1,279
Story3	RS <sub>x</sub> Max	X	0,998	0,976	1,022
Story2	RS <sub>x</sub> Max	X	0,879	0,866	1,015
Story1	RS <sub>x</sub> Max	X	0,45	0,444	1,013
Story3	RS <sub>y</sub> Max	Y	0,736	0,735	1,001
Story2	RS <sub>y</sub> Max	Y	0,738	0,67	1,1
Story1	RS <sub>y</sub> Max	Y	0,388	0,353	1,099
Story3	Comb1	Y	0,089	0,089	1,001
Story2	Comb1	Y	0,007	0,006	1,105
Story1	Comb1	X	0,001	0,0002794	3,26
Story1	Comb1	Y	0,002	0,0003767	4,968
Story3	Comb2	Y	0,089	0,089	1,001
Story2	Comb2	X	0,001	0,001	1,594
Story2	Comb2	Y	0,004	0,004	1,193
Story1	Comb2	X	0,001	0,0002025	4,027
Story1	Comb2	Y	0,002	0,0004857	3,986
Story3	Comb3	Y	0,088	0,088	1,001

**Table 4.4 - Story Max/Avg Displacements (continued)**

Story	Load Case/Combo	Direction	Maximum mm	Average mm	Ratio
Story2	Comb3	X	0,001	0,001	1,358
Story2	Comb3	Y	0,004	0,004	1,185
Story1	Comb3	X	0,001	0,000135	5,5
Story1	Comb3	Y	0,002	0,0004702	4,058
Story3	Comb4	Y	0,091	0,091	1,001
Story2	Comb4	X	0,001	0,0004686	1,746
Story2	Comb4	Y	0,005	0,005	1,178
Story1	Comb4	X	0,001	0,0002157	3,83
Story1	Comb4	Y	0,002	0,0001153	13,531
Story3	Comb5	Y	0,086	0,086	1,001
Story2	Comb5	X	0,002	0,001	1,22
Story2	Comb5	Y	0,006	0,005	1,159
Story1	Comb5	Y	0,001	6,544E-05	22,451
Story3	Comb6 Max	Y	0,095	0,095	1,001
Story2	Comb6 Max	X	0,006	0,005	1,046
Story2	Comb6 Max	Y	0,008	0,006	1,288
Story1	Comb6 Max	X	0,004	0,003	1,184
Story1	Comb6 Max	Y	0,008	0,006	1,269
Story3	Comb6 Min	Y	0,077	0,077	1,001
Story2	Comb6 Min	X	0,003	0,002	1,566
Story2	Comb6 Min	Y	0,014	0,013	1,132
Story1	Comb6 Min	X	0,001	0,001	2,238
Story1	Comb6 Min	Y	0,006	0,005	1,348
Story3	Comb7 Max	Y	0,094	0,094	1,001
Story2	Comb7 Max	X	0,005	0,005	1,047
Story2	Comb7 Max	Y	0,008	0,006	1,288
Story1	Comb7 Max	X	0,004	0,003	1,186
Story1	Comb7 Max	Y	0,008	0,006	1,268
Story3	Comb7 Min	Y	0,076	0,076	1
Story2	Comb7 Min	X	0,003	0,002	1,477
Story2	Comb7 Min	Y	0,014	0,013	1,131
Story1	Comb7 Min	X	0,002	0,001	2,124
Story1	Comb7 Min	Y	0,006	0,005	1,345
Story3	Comb8	X	1,204	1,193	1,009
Story2	Comb8	X	1,073	1,043	1,029
Story1	Comb8	X	0,545	0,532	1,025
Story3	Comb9	X	1,257	1,23	1,022
Story2	Comb9	X	1,07	1,04	1,029
Story1	Comb9	X	0,549	0,531	1,033
Story3	Comb10	Y	0,998	0,977	1,021
Story2	Comb10	Y	0,89	0,728	1,223
Story1	Comb10	Y	0,471	0,382	1,232
Story3	Comb11	Y	0,999	0,978	1,022
Story2	Comb11	Y	0,899	0,728	1,234
Story1	Comb11	Y	0,463	0,381	1,213
Story3	Comb12 Max	X	0,008	0,008	1,007
Story3	Comb12 Max	Y	0,067	0,067	1,001
Story2	Comb12 Max	X	0,006	0,006	1,025
Story2	Comb12 Max	Y	0,008	0,006	1,247

**Table 4.4 - Story Max/Avg Displacements (continued)**

Story	Load Case/Combo	Direction	Maximum mm	Average mm	Ratio
Story1	Comb12 Max	X	0,004	0,003	1,13
Story1	Comb12 Max	Y	0,007	0,006	1,239
Story3	Comb12 Min	Y	0,049	0,049	1,001
Story2	Comb12 Min	X	0,002	0,001	1,634
Story2	Comb12 Min	Y	0,014	0,012	1,122
Story1	Comb12 Min	X	0,001	0,001	2,147
Story1	Comb12 Min	Y	0,006	0,005	1,269
Story3	Comb13	X	1,204	1,194	1,009
Story2	Comb13	X	1,073	1,043	1,029
Story1	Comb13	X	0,545	0,532	1,025
Story3	Comb14	X	1,258	1,231	1,022
Story2	Comb14	X	1,071	1,041	1,029
Story1	Comb14	X	0,549	0,531	1,033
Story3	Comb15	Y	0,972	0,951	1,022
Story2	Comb15	Y	0,89	0,728	1,223
Story1	Comb15	Y	0,47	0,381	1,232
Story3	Comb16	Y	0,973	0,952	1,022
Story2	Comb16	Y	0,899	0,728	1,234
Story1	Comb16	Y	0,463	0,381	1,215

**Table 4.5 - Story Drifts**

Story	Load Case/Combo	Direction	Drift	Label	X m	Y m	Z m
Story3	Dead	Y	3E-05	18	12,8	4	7,8
Story3	Live Roof Lr	Y	2E-06	21	15,6	4	7,8
Story3	Live Floor L	Y	2E-06	18	12,8	4	7,8
Story3	Hile Load S	X	1E-06	21	15,6	4	7,8
Story3	Hile Load S	Y	1E-06	21	15,6	4	7,8
Story3	Wind X 1	X	1E-06	20	15,6	0	7,8
Story3	Wind X 2	Y	1E-06	18	12,8	4	7,8
Story3	Wind X 3	X	1E-06	20	15,6	0	7,8
Story3	Wind X 4	X	1E-06	21	15,6	4	7,8
Story3	Wind X 5	Y	4,722E-07	21	15,6	4	7,8
Story3	Wind X 6	Y	4,728E-07	18	12,8	4	7,8
Story3	Wind X 7	X	1E-06	20	15,6	0	7,8
Story3	Wind X 7	Y	4,475E-07	18	12,8	4	7,8
Story3	Wind X 8	X	1E-06	20	15,6	0	7,8
Story3	Wind X 8	Y	4,484E-07	21	15,6	4	7,8
Story3	Wind X 9	X	1E-06	20	15,6	0	7,8
Story3	Wind X 9	Y	4,121E-07	18	12,8	4	7,8
Story3	Wind X 10	X	1E-06	21	15,6	4	7,8
Story3	Wind X 10	Y	3,951E-07	21	15,6	4	7,8
Story3	Wind X 11	X	1E-06	20	15,6	0	7,8
Story3	Wind X 11	Y	4,121E-07	21	15,6	4	7,8
Story3	Wind X 12	X	1E-06	21	15,6	4	7,8
Story3	Wind X 12	Y	3,963E-07	18	12,8	4	7,8
Story3	Seismic X+e	X	7,8E-05	21	15,6	4	7,8
Story3	Seismic X-e	X	8,5E-05	20	15,6	0	7,8
Story3	Seismic Y+e	Y	7,6E-05	21	15,6	4	7,8

Table 4.5 - Story Drifts (continued)

Story	Load Case/Combo	Direction	Drift	Label	X m	Y m	Z m
Story3	Seismic Y-e	Y	7,6E-05	18	12,8	4	7,8
Story3	Dead1	Y	1E-06	18	12,8	4	7,8
Story3	RS <sub>x</sub> Max	X	7,5E-05	20	15,6	0	7,8
Story3	RS <sub>y</sub> Max	Y	6,4E-05	18	12,8	4	7,8
Story3	Comb1	Y	4,3E-05	18	12,8	4	7,8
Story3	Comb2	Y	4,2E-05	18	12,8	4	7,8
Story3	Comb3	Y	4,2E-05	21	15,6	4	7,8
Story3	Comb4	Y	4,3E-05	18	12,8	4	7,8
Story3	Comb5	Y	4,1E-05	21	15,6	4	7,8
Story3	Comb6 Max	Y	4,1E-05	18	12,8	4	7,8
Story3	Comb6 Min	Y	4E-05	18	12,8	4	7,8
Story3	Comb7 Max	Y	4,1E-05	21	15,6	4	7,8
Story3	Comb7 Min	Y	4E-05	21	15,6	4	7,8
Story3	Comb8	X	7,8E-05	21	15,6	4	7,8
Story3	Comb8	Y	4E-05	18	12,8	4	7,8
Story3	Comb9	X	8,5E-05	20	15,6	0	7,8
Story3	Comb9	Y	4,5E-05	21	15,6	4	7,8
Story3	Comb10	Y	0,000115	21	15,6	4	7,8
Story3	Comb11	Y	0,000116	18	12,8	4	7,8
Story3	Comb12 Max	Y	2,8E-05	18	12,8	4	7,8
Story3	Comb12 Min	Y	2,7E-05	18	12,8	4	7,8
Story3	Comb13	X	7,8E-05	21	15,6	4	7,8
Story3	Comb13	Y	2,8E-05	18	12,8	4	7,8
Story3	Comb14	X	8,5E-05	20	15,6	0	7,8
Story3	Comb14	Y	3,3E-05	21	15,6	4	7,8
Story3	Comb15	Y	0,000104	21	15,6	4	7,8
Story3	Comb16	Y	0,000104	18	12,8	4	7,8
Story2	Dead	X	1E-06	51	13,6	0	5,6
Story2	Dead	Y	2E-06	89	28,4	7	5,6
Story2	Live Roof Lr	X	6,397E-08	51	13,6	0	5,6
Story2	Live Roof Lr	Y	1,181E-07	89	28,4	7	5,6
Story2	Live Floor L	X	7,27E-08	51	13,6	0	5,6
Story2	Live Floor L	Y	2,732E-07	91	0	7	5,6
Story2	Hile Load S	X	2,215E-07	51	13,6	0	5,6
Story2	Hile Load S	Y	1,221E-07	89	28,4	7	5,6
Story2	Wind X 1	X	1E-06	51	13,6	0	5,6
Story2	Wind X 2	Y	1E-06	51	13,6	0	5,6
Story2	Wind X 3	X	1E-06	51	13,6	0	5,6
Story2	Wind X 4	X	1E-06	87	9,95	12	5,6
Story2	Wind X 5	Y	2E-06	89	28,4	7	5,6
Story2	Wind X 6	Y	2E-06	91	0	7	5,6
Story2	Wind X 7	X	1E-06	52	14,8	0	5,6
Story2	Wind X 7	Y	1E-06	91	0	7	5,6
Story2	Wind X 8	X	1E-06	51	13,6	0	5,6
Story2	Wind X 8	Y	1E-06	51	13,6	0	5,6
Story2	Wind X 9	X	1E-06	52	14,8	0	5,6
Story2	Wind X 9	Y	1E-06	91	0	7	5,6
Story2	Wind X 10	X	1E-06	87	9,95	12	5,6
Story2	Wind X 10	Y	1E-06	89	28,4	7	5,6

Table 4.5 - Story Drifts (continued)

Story	Load Case/Combo	Direction	Drift	Label	X m	Y m	Z m
Story2	Wind X 11	X	1E-06	51	13,6	0	5,6
Story2	Wind X 11	Y	1E-06	89	28,4	7	5,6
Story2	Wind X 12	X	1E-06	87	9,95	12	5,6
Story2	Wind X 12	Y	1E-06	91	0	7	5,6
Story2	Seismic X+e	X	0,000189	87	9,95	12	5,6
Story2	Seismic X-e	X	0,000188	51	13,6	0	5,6
Story2	Seismic Y+e	Y	0,000152	89	28,4	7	5,6
Story2	Seismic Y-e	Y	0,000157	91	0	7	5,6
Story2	Dead1	X	9,1E-08	87	9,95	12	5,6
Story2	Dead1	Y	7,096E-08	89	28,4	7	5,6
Story2	RS <sub>x</sub> Max	X	0,000155	87	9,95	12	5,6
Story2	RS <sub>y</sub> Max	Y	0,000128	91	0	7	5,6
Story2	Comb1	X	2E-06	51	13,6	0	5,6
Story2	Comb1	Y	3E-06	89	28,4	7	5,6
Story2	Comb2	X	2E-06	51	13,6	0	5,6
Story2	Comb2	Y	2E-06	89	28,4	7	5,6
Story2	Comb3	X	2E-06	51	13,6	0	5,6
Story2	Comb3	Y	2E-06	89	28,4	7	5,6
Story2	Comb4	X	2E-06	51	13,6	0	5,6
Story2	Comb4	Y	3E-06	89	28,4	7	5,6
Story2	Comb5	X	2E-06	51	13,6	0	5,6
Story2	Comb5	Y	3E-06	89	28,4	7	5,6
Story2	Comb6 Max	X	1E-06	52	14,8	0	5,6
Story2	Comb6 Max	Y	1E-06	91	0	7	5,6
Story2	Comb6 Min	X	2E-06	51	13,6	0	5,6
Story2	Comb6 Min	Y	4E-06	89	28,4	7	5,6
Story2	Comb7 Max	X	1E-06	52	14,8	0	5,6
Story2	Comb7 Max	Y	1E-06	91	0	7	5,6
Story2	Comb7 Min	X	2E-06	51	13,6	0	5,6
Story2	Comb7 Min	Y	4E-06	89	28,4	7	5,6
Story2	Comb8	X	0,000189	87	9,95	12	5,6
Story2	Comb9	X	0,000188	52	14,8	0	5,6
Story2	Comb10	Y	0,00015	89	28,4	7	5,6
Story2	Comb11	Y	0,000156	91	0	7	5,6
Story2	Comb12 Max	X	1E-06	52	14,8	0	5,6
Story2	Comb12 Max	Y	1E-06	91	0	7	5,6
Story2	Comb12 Min	X	1E-06	51	13,6	0	5,6
Story2	Comb12 Min	Y	3E-06	89	28,4	7	5,6
Story2	Comb13	X	0,000189	87	9,95	12	5,6
Story2	Comb14	X	0,000188	52	14,8	0	5,6
Story2	Comb15	Y	0,00015	89	28,4	7	5,6
Story2	Comb16	Y	0,000156	91	0	7	5,6
Story1	Dead	X	1E-06	51	13,6	0	2,8
Story1	Dead	Y	1E-06	51	13,6	0	2,8
Story1	Live Roof Lr	X	3,978E-08	51	13,6	0	2,8
Story1	Live Roof Lr	Y	1,16E-07	51	13,6	0	2,8
Story1	Live Floor L	Y	2,173E-07	92	27,6	7	2,8
Story1	Hile Load S	X	6,956E-08	52	14,8	0	2,8
Story1	Hile Load S	Y	9,871E-08	52	14,8	0	2,8

Table 4.5 - Story Drifts (continued)

Story	Load Case/Combo	Direction	Drift	Label	X m	Y m	Z m
Story1	Wind X 1	X	1E-06	64	26,45	0	2,8
Story1	Wind X 2	Y	2E-06	92	27,6	7	2,8
Story1	Wind X 3	X	1E-06	64	26,45	0	2,8
Story1	Wind X 4	X	1E-06	87	9,95	12	2,8
Story1	Wind X 5	Y	2E-06	92	27,6	7	2,8
Story1	Wind X 6	Y	2E-06	94	0,8	7	2,8
Story1	Wind X 7	X	1E-06	64	26,45	0	2,8
Story1	Wind X 7	Y	1E-06	94	0,8	7	2,8
Story1	Wind X 8	X	1E-06	64	26,45	0	2,8
Story1	Wind X 8	Y	1E-06	92	27,6	7	2,8
Story1	Wind X 9	X	1E-06	64	26,45	0	2,8
Story1	Wind X 9	Y	2E-06	94	0,8	7	2,8
Story1	Wind X 10	X	1E-06	87	9,95	12	2,8
Story1	Wind X 10	Y	2E-06	92	27,6	7	2,8
Story1	Wind X 11	X	1E-06	64	26,45	0	2,8
Story1	Wind X 11	Y	2E-06	92	27,6	7	2,8
Story1	Wind X 12	X	1E-06	87	9,95	12	2,8
Story1	Wind X 12	Y	2E-06	94	0,8	7	2,8
Story1	Seismic X+e	X	0,000195	87	9,95	12	2,8
Story1	Seismic X-e	X	0,000196	64	26,45	0	2,8
Story1	Seismic Y+e	Y	0,000166	92	27,6	7	2,8
Story1	Seismic Y-e	Y	0,000164	94	0,8	7	2,8
Story1	Dead1	X	5,402E-08	51	13,6	0	2,8
Story1	Dead1	Y	4,122E-08	52	14,8	0	2,8
Story1	RS <sub>x</sub> Max	X	0,000161	87	9,95	12	2,8
Story1	RS <sub>y</sub> Max	Y	0,000137	92	27,6	7	2,8
Story1	Comb1	X	1E-06	51	13,6	0	2,8
Story1	Comb1	Y	1E-06	51	13,6	0	2,8
Story1	Comb2	X	1E-06	51	13,6	0	2,8
Story1	Comb2	Y	1E-06	92	27,6	7	2,8
Story1	Comb3	X	1E-06	51	13,6	0	2,8
Story1	Comb3	Y	1E-06	92	27,6	7	2,8
Story1	Comb4	X	1E-06	51	13,6	0	2,8
Story1	Comb4	Y	1E-06	51	13,6	0	2,8
Story1	Comb5	X	1E-06	51	13,6	0	2,8
Story1	Comb5	Y	1E-06	51	13,6	0	2,8
Story1	Comb6 Max	X	2E-06	51	13,6	0	2,8
Story1	Comb6 Max	Y	3E-06	92	27,6	7	2,8
Story1	Comb6 Min	Y	2E-06	94	0,8	7	2,8
Story1	Comb7 Max	X	2E-06	51	13,6	0	2,8
Story1	Comb7 Max	Y	3E-06	92	27,6	7	2,8
Story1	Comb7 Min	Y	2E-06	94	0,8	7	2,8
Story1	Comb8	X	0,000195	87	9,95	12	2,8
Story1	Comb9	X	0,000196	64	26,45	0	2,8
Story1	Comb10	Y	0,000166	92	27,6	7	2,8
Story1	Comb11	Y	0,000164	94	0,8	7	2,8
Story1	Comb12 Max	X	2E-06	51	13,6	0	2,8
Story1	Comb12 Max	Y	2E-06	92	27,6	7	2,8
Story1	Comb12 Min	X	1E-06	52	14,8	0	2,8

**Table 4.5 - Story Drifts (continued)**

Story	Load Case/Combo	Direction	Drift	Label	X m	Y m	Z m
Story1	Comb12 Min	Y	2E-06	94	0,8	7	2,8
Story1	Comb13	X	0,000195	87	9,95	12	2,8
Story1	Comb14	X	0,000196	64	26,45	0	2,8
Story1	Comb15	Y	0,000166	92	27,6	7	2,8
Story1	Comb16	Y	0,000164	94	0,8	7	2,8

**Table 4.6 - Story Forces**

Story	Load Case/Combo	Location	P tonf	VX tonf	VY tonf	T tonf-m	MX tonf-m	MY tonf-m
Story3	Dead	Top	7,5859	0	0	0	15,1718	-107,7201
Story3	Dead	Bottom	20,6082	0	0	0	41,2163	-292,6359
Story3	Live Roof Lr	Top	0,784	0	0	0	1,568	-11,1328
Story3	Live Roof Lr	Bottom	0,784	0	0	0	1,568	-11,1328
Story3	Live Floor L	Top	0	0	0	0	0	0
Story3	Live Floor L	Bottom	0	0	0	0	0	0
Story3	Hile Load S	Top	0,56	0	0	0	1,12	-7,952
Story3	Hile Load S	Bottom	0,56	0	0	0	1,12	-7,952
Story3	Wind X 1	Top	0	-0,0751	0	0,1501	0	0
Story3	Wind X 1	Bottom	0	-0,0751	0	0,1501	0	-0,1651
Story3	Wind X 2	Top	0	0	-0,0525	-0,7461	0	0
Story3	Wind X 2	Bottom	0	0	-0,0525	-0,7461	0,1156	0
Story3	Wind X 3	Top	0	-0,0563	0	0,0788	0	0
Story3	Wind X 3	Bottom	0	-0,0563	0	0,0788	0	-0,1238
Story3	Wind X 4	Top	0	-0,0563	0	0,1464	0	0
Story3	Wind X 4	Bottom	0	-0,0563	0	0,1464	0	-0,1238
Story3	Wind X 5	Top	0	0	-0,0394	-0,5761	0	0
Story3	Wind X 5	Bottom	0	0	-0,0394	-0,5761	0,0867	0
Story3	Wind X 6	Top	0	0	-0,0394	-0,543	0	0
Story3	Wind X 6	Bottom	0	0	-0,0394	-0,543	0,0867	0
Story3	Wind X 7	Top	0	-0,0563	0,0394	0,6721	0	0
Story3	Wind X 7	Bottom	0	-0,0563	0,0394	0,6721	-0,0867	-0,1238
Story3	Wind X 8	Top	0	-0,0563	-0,0394	-0,447	0	0
Story3	Wind X 8	Bottom	0	-0,0563	-0,0394	-0,447	0,0867	-0,1238
Story3	Wind X 9	Top	0	-0,0423	0,0296	0,4668	0	0
Story3	Wind X 9	Bottom	0	-0,0423	0,0296	0,4668	-0,0651	-0,093
Story3	Wind X 10	Top	0	-0,0423	0,0296	0,5423	0	0
Story3	Wind X 10	Bottom	0	-0,0423	0,0296	0,5423	-0,0651	-0,093
Story3	Wind X 11	Top	0	-0,0423	-0,0296	-0,3733	0	0
Story3	Wind X 11	Bottom	0	-0,0423	-0,0296	-0,3733	0,0651	-0,093
Story3	Wind X 12	Top	0	-0,0423	-0,0296	-0,2977	0	0
Story3	Wind X 12	Bottom	0	-0,0423	-0,0296	-0,2977	0,0651	-0,093
Story3	Seismic X+e	Top	0	-7,1977	0	15,8205	0	0
Story3	Seismic X+e	Bottom	0	-7,1977	0	15,8205	0	-15,835
Story3	Seismic X-e	Top	0	-7,1977	0	12,9414	0	0
Story3	Seismic X-e	Bottom	0	-7,1977	0	12,9414	0	-15,835
Story3	Seismic Y+e	Top	0	0	-7,1977	-103,2157	0	0
Story3	Seismic Y+e	Bottom	0	0	-7,1977	-103,2157	15,835	0
Story3	Seismic Y-e	Top	0	0	-7,1977	-101,2003	0	0
Story3	Seismic Y-e	Bottom	0	0	-7,1977	-101,2003	15,835	0

Table 4.6 - Story Forces (continued)

Story	Load Case/Combo	Location	P tonf	VX tonf	VY tonf	T tonf-m	MX tonf-m	MY tonf-m
Story3	Dead1	Top	0,7056	0	0	0	1,4112	-10,0195
Story3	Dead1	Bottom	0,7056	0	0	0	1,4112	-10,0195
Story3	RS <sub>x</sub> Max	Top	0	6,0723	0,0672	12,8634	0	0
Story3	RS <sub>x</sub> Max	Bottom	0	6,0723	0,0672	12,8634	0,1478	13,3591
Story3	RS <sub>y</sub> Max	Top	0	0,0692	6,1837	88,693	0	0
Story3	RS <sub>y</sub> Max	Bottom	0	0,0692	6,1837	88,693	13,6041	0,1523
Story3	Comb1	Top	11,6081	0	0	0	23,2163	-164,8354
Story3	Comb1	Bottom	29,8393	0	0	0	59,6785	-423,7175
Story3	Comb2	Top	10,3418	0	0	0	20,6836	-146,8539
Story3	Comb2	Bottom	25,9685	0	0	0	51,937	-368,7529
Story3	Comb3	Top	10,2298	0	0	0	20,4596	-145,2635
Story3	Comb3	Bottom	25,8565	0	0	0	51,713	-367,1625
Story3	Comb4	Top	11,2042	0	0	0	22,4084	-159,1
Story3	Comb4	Bottom	26,8309	0	0	0	53,6618	-380,999
Story3	Comb5	Top	10,8458	0	0	0	21,6916	-154,0107
Story3	Comb5	Bottom	26,4725	0	0	0	52,945	-375,9097
Story3	Comb6 Max	Top	10,3418	0	0,0394	0,6721	20,6836	-146,8539
Story3	Comb6 Max	Bottom	25,9685	0	0,0394	0,6721	52,0526	-368,7529
Story3	Comb6 Min	Top	10,3418	-0,0751	-0,0525	-0,7461	20,6836	-146,8539
Story3	Comb6 Min	Bottom	25,9685	-0,0751	-0,0525	-0,7461	51,8503	-368,918
Story3	Comb7 Max	Top	10,2298	0	0,0394	0,6721	20,4596	-145,2635
Story3	Comb7 Max	Bottom	25,8565	0	0,0394	0,6721	51,8286	-367,1625
Story3	Comb7 Min	Top	10,2298	-0,0751	-0,0525	-0,7461	20,4596	-145,2635
Story3	Comb7 Min	Bottom	25,8565	-0,0751	-0,0525	-0,7461	51,6263	-367,3276
Story3	Comb8	Top	10,0618	-7,1977	0	15,8205	20,1236	-142,8779
Story3	Comb8	Bottom	25,6885	-7,1977	0	15,8205	51,377	-380,6119
Story3	Comb9	Top	10,0618	-7,1977	0	12,9414	20,1236	-142,8779
Story3	Comb9	Bottom	25,6885	-7,1977	0	12,9414	51,377	-380,6119
Story3	Comb10	Top	10,0618	0	-7,1977	-103,2157	20,1236	-142,8779
Story3	Comb10	Bottom	25,6885	0	-7,1977	-103,2157	67,2121	-364,7769
Story3	Comb11	Top	10,0618	0	-7,1977	-101,2003	20,1236	-142,8779
Story3	Comb11	Bottom	25,6885	0	-7,1977	-101,2003	67,2121	-364,7769
Story3	Comb12 Max	Top	7,4624	0	0,0394	0,6721	14,9247	-105,9656
Story3	Comb12 Max	Bottom	19,1824	0	0,0394	0,6721	38,4804	-272,3899
Story3	Comb12 Min	Top	7,4624	-0,0751	-0,0525	-0,7461	14,9247	-105,9656
Story3	Comb12 Min	Bottom	19,1824	-0,0751	-0,0525	-0,7461	38,2781	-272,555
Story3	Comb13	Top	7,4624	-7,1977	0	15,8205	14,9247	-105,9656
Story3	Comb13	Bottom	19,1824	-7,1977	0	15,8205	38,3648	-288,2249
Story3	Comb14	Top	7,4624	-7,1977	0	12,9414	14,9247	-105,9656
Story3	Comb14	Bottom	19,1824	-7,1977	0	12,9414	38,3648	-288,2249
Story3	Comb15	Top	7,4624	0	-7,1977	-103,2157	14,9247	-105,9656
Story3	Comb15	Bottom	19,1824	0	-7,1977	-103,2157	54,1998	-272,3899
Story3	Comb16	Top	7,4624	0	-7,1977	-101,2003	14,9247	-105,9656
Story3	Comb16	Bottom	19,1824	0	-7,1977	-101,2003	54,1998	-272,3899
Story2	Dead	Top	219,6506	0	0	0	1247,2793	-3119,0357
Story2	Dead	Bottom	407,5684	0	0	0	2394,0627	-5818,7489
Story2	Live Roof Lr	Top	23,739	0	0	0	139,2967	-337,0937
Story2	Live Roof Lr	Bottom	23,739	0	0	0	139,2967	-337,0937
Story2	Live Floor L	Top	0	0	0	0	0	0

Table 4.6 - Story Forces (continued)

Story	Load Case/Combo	Location	P tonf	VX tonf	VY tonf	T tonf-m	MX tonf-m	MY tonf-m
Story2	Live Floor L	Bottom	0	0	0	0	0	0
Story2	Hile Load S	Top	15,3564	0	0	0	86,6976	-212,6181
Story2	Hile Load S	Bottom	15,3564	0	0	0	86,6976	-212,6181
Story2	Wind X 1	Top	0	-0,5653	0	3,0915	0	-0,1651
Story2	Wind X 1	Bottom	0	-0,5653	0	3,0915	0	-1,7479
Story2	Wind X 2	Top	0	0	-1,2128	-17,2211	0,1156	0
Story2	Wind X 2	Bottom	0	0	-1,2128	-17,2211	3,5113	0
Story2	Wind X 3	Top	0	-0,424	0	1,623	0	-0,1238
Story2	Wind X 3	Bottom	0	-0,424	0	1,623	0	-1,3109
Story2	Wind X 4	Top	0	-0,424	0	3,0142	0	-0,1238
Story2	Wind X 4	Bottom	0	-0,424	0	3,0142	0	-1,3109
Story2	Wind X 5	Top	0	0	-0,9096	-16,6393	0,0867	0
Story2	Wind X 5	Bottom	0	0	-0,9096	-16,6393	2,6335	0
Story2	Wind X 6	Top	0	0	-0,9096	-9,1924	0,0867	0
Story2	Wind X 6	Bottom	0	0	-0,9096	-9,1924	2,6335	0
Story2	Wind X 7	Top	0	-0,424	0,9096	15,2345	-0,0867	-0,1238
Story2	Wind X 7	Bottom	0	-0,424	0,9096	15,2345	-2,6335	-1,3109
Story2	Wind X 8	Top	0	-0,424	-0,9096	-10,5972	0,0867	-0,1238
Story2	Wind X 8	Bottom	0	-0,424	-0,9096	-10,5972	2,6335	-1,3109
Story2	Wind X 9	Top	0	-0,3183	0,6828	8,1188	-0,0651	-0,093
Story2	Wind X 9	Bottom	0	-0,3183	0,6828	8,1188	-1,9769	-0,9841
Story2	Wind X 10	Top	0	-0,3183	0,6828	14,7532	-0,0651	-0,093
Story2	Wind X 10	Bottom	0	-0,3183	0,6828	14,7532	-1,9769	-0,9841
Story2	Wind X 11	Top	0	-0,3183	-0,6828	-11,2722	0,0651	-0,093
Story2	Wind X 11	Bottom	0	-0,3183	-0,6828	-11,2722	1,9769	-0,9841
Story2	Wind X 12	Top	0	-0,3183	-0,6828	-4,6378	0,0651	-0,093
Story2	Wind X 12	Bottom	0	-0,3183	-0,6828	-4,6378	1,9769	-0,9841
Story2	Seismic X+e	Top	0	-110,9149	0	692,9897	0	-15,835
Story2	Seismic X+e	Bottom	0	-110,9149	0	692,9897	0	-326,3967
Story2	Seismic X-e	Top	0	-110,9149	0	565,6501	0	-15,835
Story2	Seismic X-e	Bottom	0	-110,9149	0	565,6501	0	-326,3967
Story2	Seismic Y+e	Top	0	0	-110,9149	-1722,441	15,835	0
Story2	Seismic Y+e	Bottom	0	0	-110,9149	-1722,441	326,3967	0
Story2	Seismic Y-e	Top	0	0	-110,9149	-1425,869	15,835	0
Story2	Seismic Y-e	Bottom	0	0	-110,9149	-1425,869	326,3967	0
Story2	Dead1	Top	7,1605	0	0	0	37,0685	-90,3146
Story2	Dead1	Bottom	7,1605	0	0	0	37,0685	-90,3146
Story2	RS <sub>x</sub> Max	Top	0	90,8215	0,9833	570,2975	0,1478	13,3591
Story2	RS <sub>x</sub> Max	Bottom	0	90,8215	0,9833	570,2975	2,8828	264,9502
Story2	RS <sub>y</sub> Max	Top	0	0,9158	90,2082	1401,5011	13,6041	0,1523
Story2	RS <sub>y</sub> Max	Bottom	0	0,9158	90,2082	1401,5011	264,6606	2,6527
Story2	Comb1	Top	317,5356	0	0	0	1798,0869	-4493,0904
Story2	Comb1	Bottom	580,6205	0	0	0	3403,5837	-8272,6889
Story2	Comb2	Top	284,0429	0	0	0	1610,8657	-4019,7671
Story2	Comb2	Bottom	509,5442	0	0	0	2987,0058	-7259,423
Story2	Comb3	Top	279,8516	0	0	0	1584,5662	-3957,5293
Story2	Comb3	Bottom	505,353	0	0	0	2960,7063	-7197,1852
Story2	Comb4	Top	310,1558	0	0	0	1764,0921	-4390,5702
Story2	Comb4	Bottom	535,6572	0	0	0	3140,2322	-7630,2261

Table 4.6 - Story Forces (continued)

Story	Load Case/Combo	Location	P tonf	VX tonf	VY tonf	T tonf-m	MX tonf-m	MY tonf-m
Story2	Comb5	Top	296,7437	0	0	0	1679,9336	-4191,4092
Story2	Comb5	Bottom	522,245	0	0	0	3056,0737	-7431,0651
Story2	Comb6 Max	Top	284,0429	0	0,9096	15,2345	1610,9813	-4019,7671
Story2	Comb6 Max	Bottom	509,5442	0	0,9096	15,2345	2990,5171	-7259,423
Story2	Comb6 Min	Top	284,0429	-0,5653	-1,2128	-17,2211	1610,779	-4019,9323
Story2	Comb6 Min	Bottom	509,5442	-0,5653	-1,2128	-17,2211	2984,3723	-7261,171
Story2	Comb7 Max	Top	279,8516	0	0,9096	15,2345	1584,6818	-3957,5293
Story2	Comb7 Max	Bottom	505,353	0	0,9096	15,2345	2964,2176	-7197,1852
Story2	Comb7 Min	Top	279,8516	-0,5653	-1,2128	-17,2211	1584,4795	-3957,6945
Story2	Comb7 Min	Bottom	505,353	-0,5653	-1,2128	-17,2211	2958,0728	-7198,9332
Story2	Comb8	Top	275,2447	-110,9149	0	692,9897	1558,5569	-3909,579
Story2	Comb8	Bottom	500,746	-110,9149	0	692,9897	2934,697	-7459,7965
Story2	Comb9	Top	275,2447	-110,9149	0	565,6501	1558,5569	-3909,579
Story2	Comb9	Bottom	500,746	-110,9149	0	565,6501	2934,697	-7459,7965
Story2	Comb10	Top	275,2447	0	-110,9149	-1722,441	1574,3919	-3893,7439
Story2	Comb10	Bottom	500,746	0	-110,9149	-1722,441	3261,0937	-7133,3998
Story2	Comb11	Top	275,2447	0	-110,9149	-1425,869	1574,3919	-3893,7439
Story2	Comb11	Bottom	500,746	0	-110,9149	-1425,869	3261,0937	-7133,3998
Story2	Comb12 Max	Top	204,13	0	0,9096	15,2345	1156,0286	-2888,4152
Story2	Comb12 Max	Bottom	373,256	0	0,9096	15,2345	2191,5294	-5318,1572
Story2	Comb12 Min	Top	204,13	-0,5653	-1,2128	-17,2211	1155,8263	-2888,5804
Story2	Comb12 Min	Bottom	373,256	-0,5653	-1,2128	-17,2211	2185,3846	-5319,9051
Story2	Comb13	Top	204,13	-110,9149	0	692,9897	1155,913	-2904,2503
Story2	Comb13	Bottom	373,256	-110,9149	0	692,9897	2188,0181	-5644,5538
Story2	Comb14	Top	204,13	-110,9149	0	565,6501	1155,913	-2904,2503
Story2	Comb14	Bottom	373,256	-110,9149	0	565,6501	2188,0181	-5644,5538
Story2	Comb15	Top	204,13	0	-110,9149	-1722,441	1171,7481	-2888,4152
Story2	Comb15	Bottom	373,256	0	-110,9149	-1722,441	2514,4148	-5318,1572
Story2	Comb16	Top	204,13	0	-110,9149	-1425,869	1171,7481	-2888,4152
Story2	Comb16	Bottom	373,256	0	-110,9149	-1425,869	2514,4148	-5318,1572
Story1	Dead	Top	607,6608	0	0	0	3615,2823	-8660,0579
Story1	Dead	Bottom	791,0376	0	0	0	4721,1938	-11263,1516
Story1	Live Roof Lr	Top	23,739	0	0	0	139,2967	-337,0937
Story1	Live Roof Lr	Bottom	23,739	0	0	0	139,2967	-337,0937
Story1	Live Floor L	Top	63,895	0	0	0	390,9154	-907,3078
Story1	Live Floor L	Bottom	63,895	0	0	0	390,9154	-907,3078
Story1	Hile Load S	Top	15,3564	0	0	0	86,6976	-212,6181
Story1	Hile Load S	Bottom	15,3564	0	0	0	86,6976	-212,6181
Story1	Wind X 1	Top	0	-1,0983	0	6,2893	0	-1,7479
Story1	Wind X 1	Bottom	0	-1,0983	0	6,2893	0	-4,823
Story1	Wind X 2	Top	0	0	-2,4741	-35,1324	3,5113	0
Story1	Wind X 2	Bottom	0	0	-2,4741	-35,1324	10,4388	0
Story1	Wind X 3	Top	0	-0,8237	0	3,3019	0	-1,3109
Story1	Wind X 3	Bottom	0	-0,8237	0	3,3019	0	-3,6173
Story1	Wind X 4	Top	0	-0,8237	0	6,1321	0	-1,3109
Story1	Wind X 4	Bottom	0	-0,8237	0	6,1321	0	-3,6173
Story1	Wind X 5	Top	0	0	-1,8556	-34,1028	2,6335	0
Story1	Wind X 5	Bottom	0	0	-1,8556	-34,1028	7,8291	0
Story1	Wind X 6	Top	0	0	-1,8556	-18,5958	2,6335	0

**Table 4.6 - Story Forces (continued)**

Story	Load Case/Combo	Location	P tonf	VX tonf	VY tonf	T tonf-m	MX tonf-m	MY tonf-m
Story1	Wind X 6	Bottom	0	0	-1,8556	-18,5958	7,8291	0
Story1	Wind X 7	Top	0	-0,8237	1,8556	31,0663	-2,6335	-1,3109
Story1	Wind X 7	Bottom	0	-0,8237	1,8556	31,0663	-7,8291	-3,6173
Story1	Wind X 8	Top	0	-0,8237	-1,8556	-21,6323	2,6335	-1,3109
Story1	Wind X 8	Bottom	0	-0,8237	-1,8556	-21,6323	7,8291	-3,6173
Story1	Wind X 9	Top	0	-0,6183	1,3929	16,4379	-1,9769	-0,9841
Story1	Wind X 9	Bottom	0	-0,6183	1,3929	16,4379	-5,8771	-2,7154
Story1	Wind X 10	Top	0	-0,6183	1,3929	30,203	-1,9769	-0,9841
Story1	Wind X 10	Bottom	0	-0,6183	1,3929	30,203	-5,8771	-2,7154
Story1	Wind X 11	Top	0	-0,6183	-1,3929	-23,1212	1,9769	-0,9841
Story1	Wind X 11	Bottom	0	-0,6183	-1,3929	-23,1212	5,8771	-2,7154
Story1	Wind X 12	Top	0	-0,6183	-1,3929	-9,3561	1,9769	-0,9841
Story1	Wind X 12	Bottom	0	-0,6183	-1,3929	-9,3561	5,8771	-2,7154
Story1	Seismic X+e	Top	0	-181,2015	0	1162,438	0	-326,3967
Story1	Seismic X+e	Bottom	0	-181,2015	0	1162,438	0	-833,761
Story1	Seismic X-e	Top	0	-181,2015	0	951,3771	0	-326,3967
Story1	Seismic X-e	Bottom	0	-181,2015	0	951,3771	0	-833,761
Story1	Seismic Y+e	Top	0	0	-181,2015	-2822,2126	326,3967	0
Story1	Seismic Y+e	Bottom	0	0	-181,2015	-2822,2126	833,761	0
Story1	Seismic Y-e	Top	0	0	-181,2015	-2327,5003	326,3967	0
Story1	Seismic Y-e	Bottom	0	0	-181,2015	-2327,5003	833,761	0
Story1	Dead1	Top	27,2875	0	0	0	160,2069	-376,1165
Story1	Dead1	Bottom	27,2875	0	0	0	160,2069	-376,1165
Story1	RS <sub>x</sub> Max	Top	0	149,5253	1,5761	957,3726	2,8828	264,9502
Story1	RS <sub>x</sub> Max	Bottom	0	149,5253	1,5761	957,3726	7,2244	677,9758
Story1	RS <sub>y</sub> Max	Top	0	1,5761	150,2543	2347,0485	264,6606	2,6527
Story1	RS <sub>y</sub> Max	Bottom	0	1,5761	150,2543	2347,0485	680,1748	6,9923
Story1	Comb1	Top	888,9276	0	0	0	5285,6848	-12650,6442
Story1	Comb1	Bottom	1145,655	0	0	0	6833,961	-16294,9754
Story1	Comb2	Top	876,0394	0	0	0	5225,7001	-12463,6487
Story1	Comb2	Bottom	1096,0915	0	0	0	6552,7939	-15587,3612
Story1	Comb3	Top	871,8481	0	0	0	5199,4005	-12401,4109
Story1	Comb3	Bottom	1091,9002	0	0	0	6526,4944	-15525,1234
Story1	Comb4	Top	863,8153	0	0	0	5144,3772	-12290,067
Story1	Comb4	Bottom	1083,8674	0	0	0	6471,471	-15413,7795
Story1	Comb5	Top	850,4032	0	0	0	5060,2187	-12090,906
Story1	Comb5	Bottom	1070,4553	0	0	0	6387,3125	-15214,6185
Story1	Comb6 Max	Top	837,7024	0	1,8556	31,0663	4994,6621	-11919,264
Story1	Comb6 Max	Bottom	1057,7545	0	1,8556	31,0663	6328,6834	-15042,9765
Story1	Comb6 Min	Top	837,7024	-1,0983	-2,4741	-35,1324	4988,5173	-11921,0119
Story1	Comb6 Min	Bottom	1057,7545	-1,0983	-2,4741	-35,1324	6310,4155	-15047,7995
Story1	Comb7 Max	Top	833,5111	0	1,8556	31,0663	4968,3626	-11857,0262
Story1	Comb7 Max	Bottom	1053,5632	0	1,8556	31,0663	6302,3839	-14980,7387
Story1	Comb7 Min	Top	833,5111	-1,0983	-2,4741	-35,1324	4962,2178	-11858,7741
Story1	Comb7 Min	Bottom	1053,5632	-1,0983	-2,4741	-35,1324	6284,116	-14985,5617
Story1	Comb8	Top	828,9042	-181,2015	0	1162,438	4938,842	-12119,6375
Story1	Comb8	Bottom	1048,9563	-181,2015	0	1162,438	6265,9358	-15750,7143
Story1	Comb9	Top	828,9042	-181,2015	0	951,3771	4938,842	-12119,6375
Story1	Comb9	Bottom	1048,9563	-181,2015	0	951,3771	6265,9358	-15750,7143

**Table 4.6 - Story Forces (continued)**

Story	Load Case/Combo	Location	P tonf	VX tonf	VY tonf	T tonf-m	MX tonf-m	MY tonf-m
Story1	Comb10	Top	828,9042	0	-181,2015	-2822,2126	5265,2387	-11793,2408
Story1	Comb10	Bottom	1048,9563	0	-181,2015	-2822,2126	7099,6968	-14916,9533
Story1	Comb11	Top	828,9042	0	-181,2015	-2327,5003	5265,2387	-11793,2408
Story1	Comb11	Bottom	1048,9563	0	-181,2015	-2327,5003	7099,6968	-14916,9533
Story1	Comb12 Max	Top	571,4534	0	1,8556	31,0663	3401,4516	-8132,557
Story1	Comb12 Max	Bottom	736,4925	0	1,8556	31,0663	4403,6994	-10475,3414
Story1	Comb12 Min	Top	571,4534	-1,0983	-2,4741	-35,1324	3395,3068	-8134,3049
Story1	Comb12 Min	Bottom	736,4925	-1,0983	-2,4741	-35,1324	4385,4315	-10480,1644
Story1	Comb13	Top	571,4534	-181,2015	0	1162,438	3397,9403	-8458,9537
Story1	Comb13	Bottom	736,4925	-181,2015	0	1162,438	4393,2606	-11309,1024
Story1	Comb14	Top	571,4534	-181,2015	0	951,3771	3397,9403	-8458,9537
Story1	Comb14	Bottom	736,4925	-181,2015	0	951,3771	4393,2606	-11309,1024
Story1	Comb15	Top	571,4534	0	-181,2015	-2822,2126	3724,337	-8132,557
Story1	Comb15	Bottom	736,4925	0	-181,2015	-2822,2126	5227,0216	-10475,3414
Story1	Comb16	Top	571,4534	0	-181,2015	-2327,5003	3724,337	-8132,557
Story1	Comb16	Bottom	736,4925	0	-181,2015	-2327,5003	5227,0216	-10475,3414

**4.3 Modal Results**

**Table 4.7 - Modal Periods and Frequencies**

Case	Mode	Period sec	Frequency cyc/sec	Circular Frequency rad/sec	Eigenvalue rad <sup>2</sup> /sec <sup>2</sup>
Modal	1	0,088	11,394	71,5926	5125,5025
Modal	2	0,074	13,541	85,0836	7239,2243
Modal	3	0,071	14,076	88,4423	7822,0384
Modal	4	0,033	29,93	188,0534	35364,0976
Modal	5	0,03	33,294	209,192	43761,2805
Modal	6	0,029	34,037	213,8613	45736,6393
Modal	7	0,027	36,687	230,5112	53135,401
Modal	8	0,026	38,679	243,0291	59063,1297

**Table 4.8 - Modal Participating Mass Ratios (Part 1 of 2)**

Case	Mode	Period sec	UX	UY	UZ	Sum UX	Sum UY	Sum UZ
Modal	1	0,088	0,8978	0,0001	0	0,8978	0,0001	0
Modal	2	0,074	0,0001	0,903	0	0,8979	0,903	0
Modal	3	0,071	1,613E-05	0,0002	0	0,8979	0,9032	0
Modal	4	0,033	0,0446	5,626E-07	0	0,9425	0,9032	0
Modal	5	0,03	0,0566	0,0002	0	0,9991	0,9034	0
Modal	6	0,029	0,0001	0,0774	0	0,9992	0,9808	0
Modal	7	0,027	0,0008	2,834E-05	0	1	0,9809	0
Modal	8	0,026	0	0,0003	0	1	0,9812	0

**Table 4.8 - Modal Participating Mass Ratios (Part 2 of 2)**

Case	Mode	RX	RY	RZ	Sum RX	Sum RY	Sum RZ
Modal	1	3,114E-05	0,3514	3,375E-05	3,114E-05	0,3514	3,375E-05
Modal	2	0,3447	2,248E-05	0,0001	0,3448	0,3514	0,0002
Modal	3	7,103E-06	0,0011	0,8997	0,3448	0,3526	0,8998

**Table 4.8 - Modal Participating Mass Ratios (Part 2 of 2, continued)**

Case	Mode	RX	RY	RZ	Sum RX	Sum RY	Sum RZ
Modal	4	1,408E-06	0,3886	0,0021	0,3448	0,7412	0,9019
Modal	5	0,0012	0,2551	0,0061	0,346	0,9964	0,9081
Modal	6	0,5752	0,0007	0,0002	0,9212	0,9971	0,9082
Modal	7	0,0008	0,0029	0,0916	0,922	1	0,9998
Modal	8	2,029E-05	0	2,35E-05	0,9221	1	0,9998

**Table 4.9 - Modal Load Participation Ratios**

Case	Item Type	Item	Static %	Dynamic %
Modal	Acceleration	UX	100	100
Modal	Acceleration	UY	99,76	98,12
Modal	Acceleration	UZ	0	0

**Table 4.10 - Modal Direction Factors**

Case	Mode	Period sec	UX	UY	UZ	RZ
Modal	1	0,088	1	0	0	0
Modal	2	0,074	0	1	0	0
Modal	3	0,071	0,01	0	0	0,99
Modal	4	0,033	0,988	0	0	0,012
Modal	5	0,03	0,963	0,002	0	0,035
Modal	6	0,029	0,003	0,996	0	0,001
Modal	7	0,027	0,036	0,003	0	0,961
Modal	8	0,026	0	0,999	0	0,001

## Story Response - Story Shears

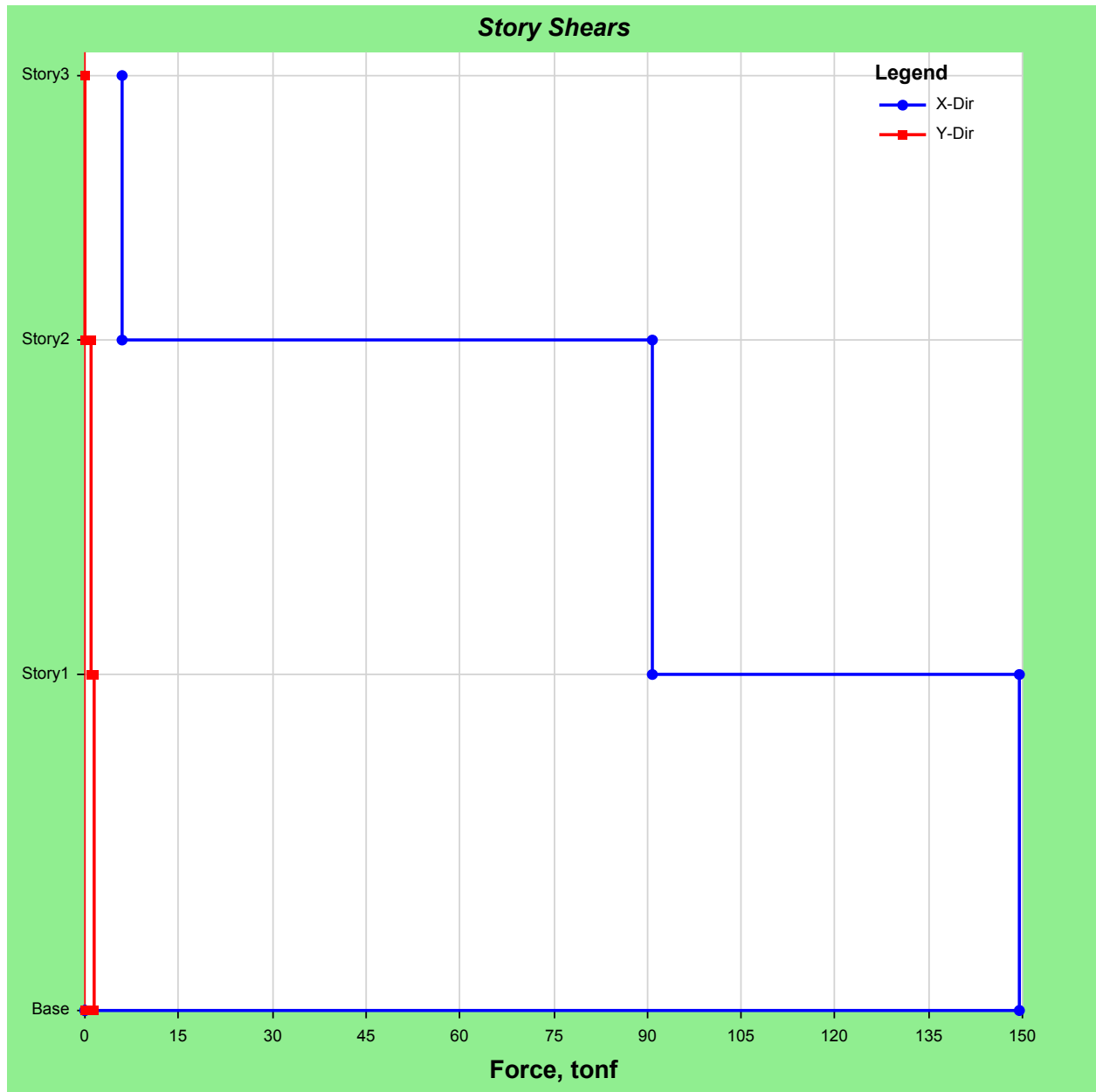
### Summary Description

This is story response output for a specified range of stories and a selected load case or load combination.

### Input Data

Name	StoryResp2	Story Range	All Stories
Display Type	Story shears	Top Story	Story3
Load Case	RS <sub>x</sub>	Bottom Story	Base
Output Type	Not Applicable		

### Plot



# Story Response - Story Shears

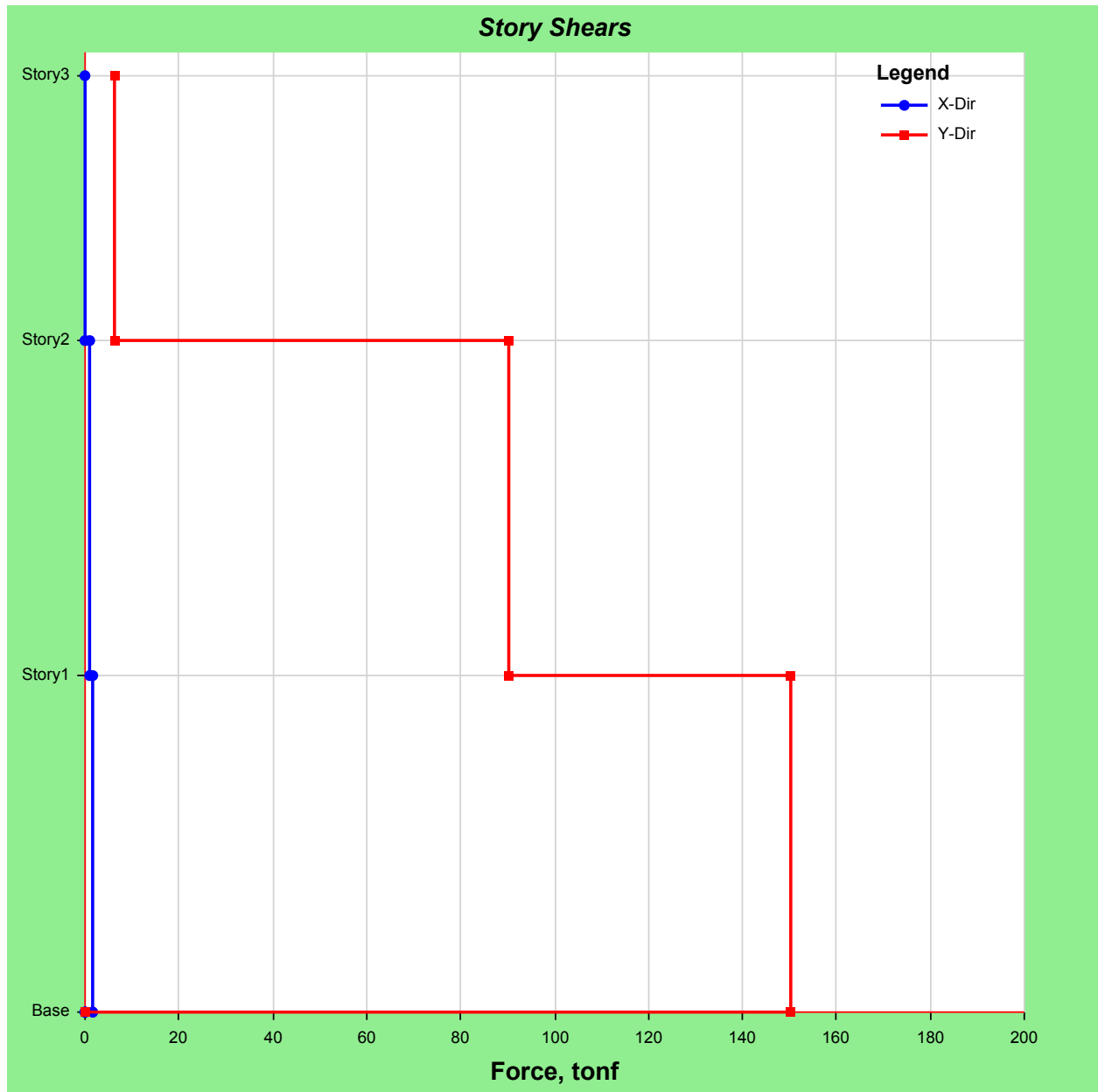
## Summary Description

This is story response output for a specified range of stories and a selected load case or load combination.

## Input Data

Name	StoryResp2	Story Range	All Stories
Display Type	Story shears	Top Story	Story3
Load Case	RS <sub>y</sub>	Bottom Story	Base
Output Type	Not Applicable		

## Plot



# Story Response - Story Shears

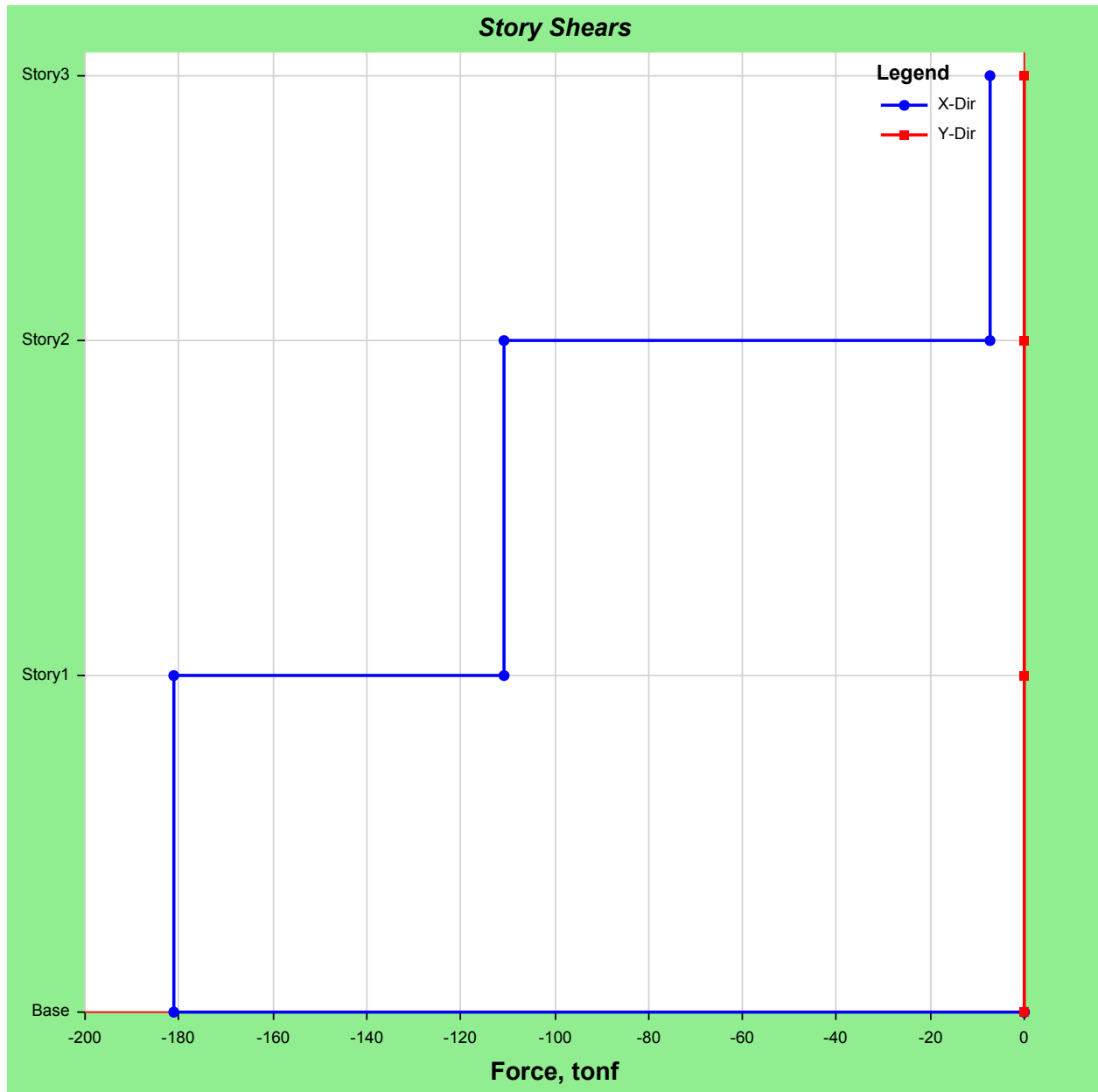
## Summary Description

This is story response output for a specified range of stories and a selected load case or load combination.

## Input Data

Name	StoryResp2	Story Range	All Stories
Display Type	Story shears	Top Story	Story3
Load Case	Seismic X-e	Bottom Story	Base
Output Type	Not Applicable		

## Plot



# Story Response - Story Shears

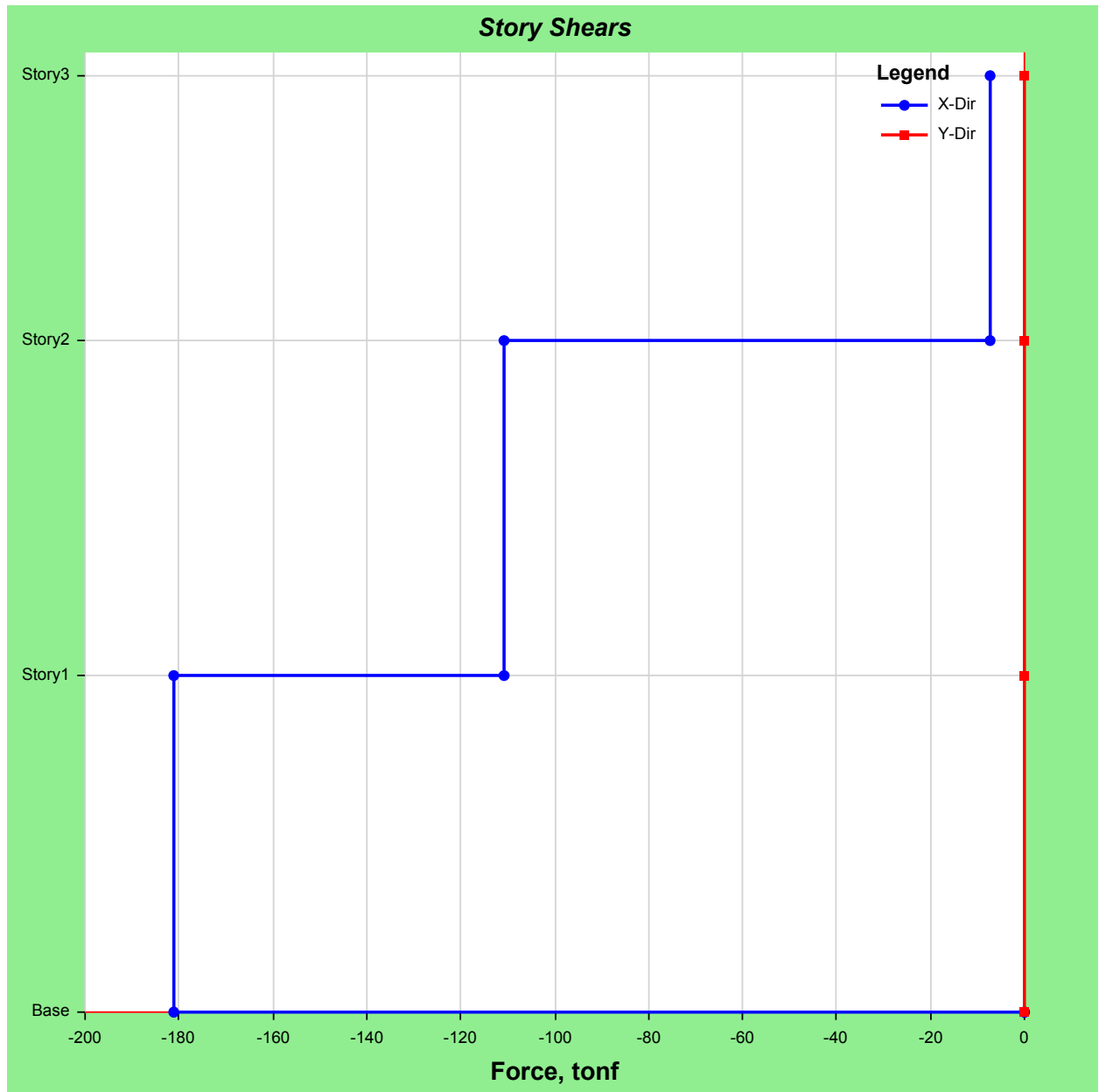
## Summary Description

This is story response output for a specified range of stories and a selected load case or load combination.

## Input Data

Name	StoryResp2	Story Range	All Stories
Display Type	Story shears	Top Story	Story3
Load Case	Seismic X+e	Bottom Story	Base
Output Type	Not Applicable		

## Plot



# Story Response - Story Shears

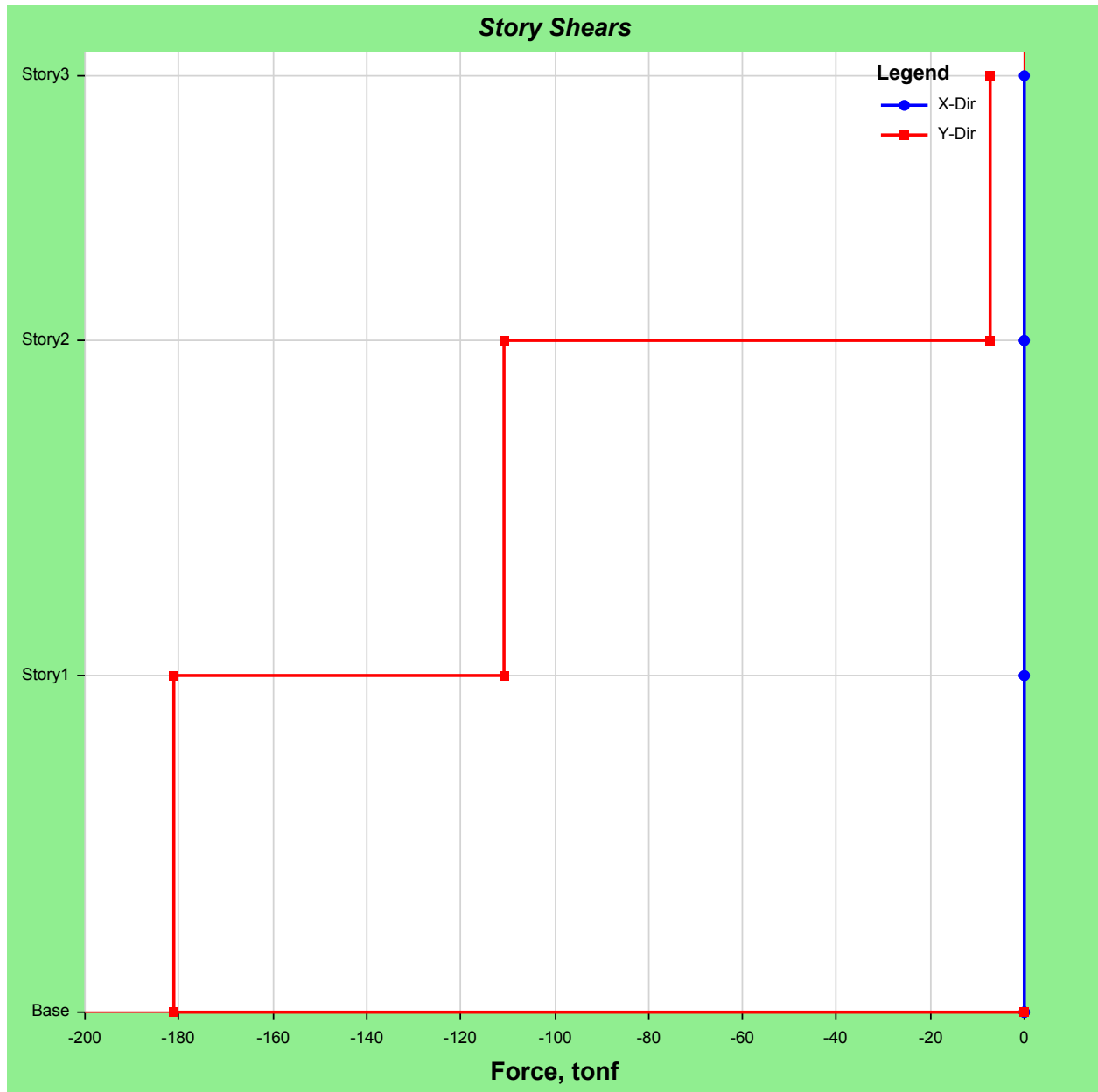
## Summary Description

This is story response output for a specified range of stories and a selected load case or load combination.

## Input Data

Name	StoryResp2	Story Range	All Stories
Display Type	Story shears	Top Story	Story3
Load Case	Seismic Y+e	Bottom Story	Base
Output Type	Not Applicable		

## Plot



# Story Response - Story Shears

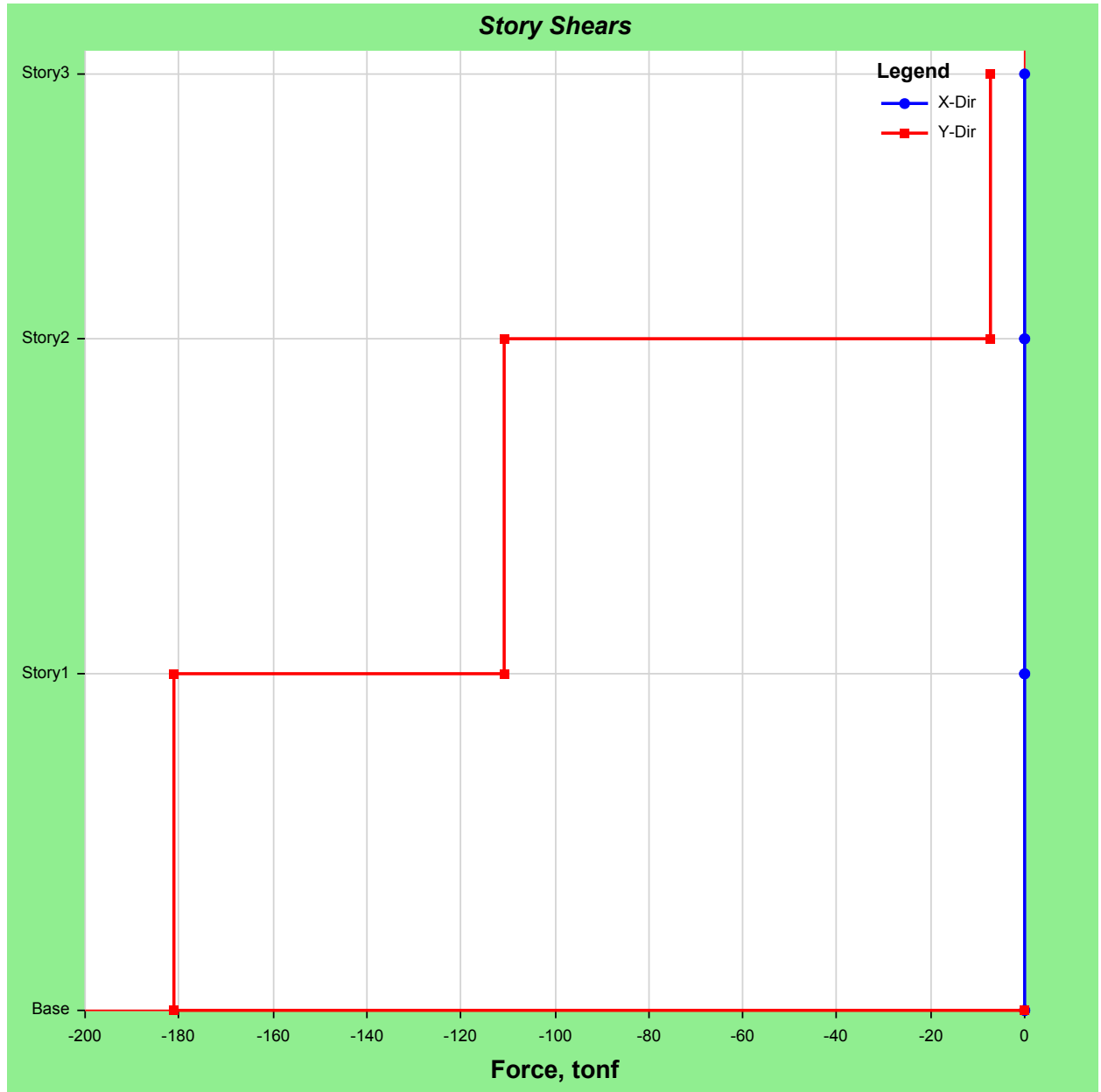
## Summary Description

This is story response output for a specified range of stories and a selected load case or load combination.

## Input Data

Name	StoryResp2	Story Range	All Stories
Display Type	Story shears	Top Story	Story3
Load Case	Seismic Y-e	Bottom Story	Base
Output Type	Not Applicable		

## Plot



## Story Response - Maximum Story Displacement

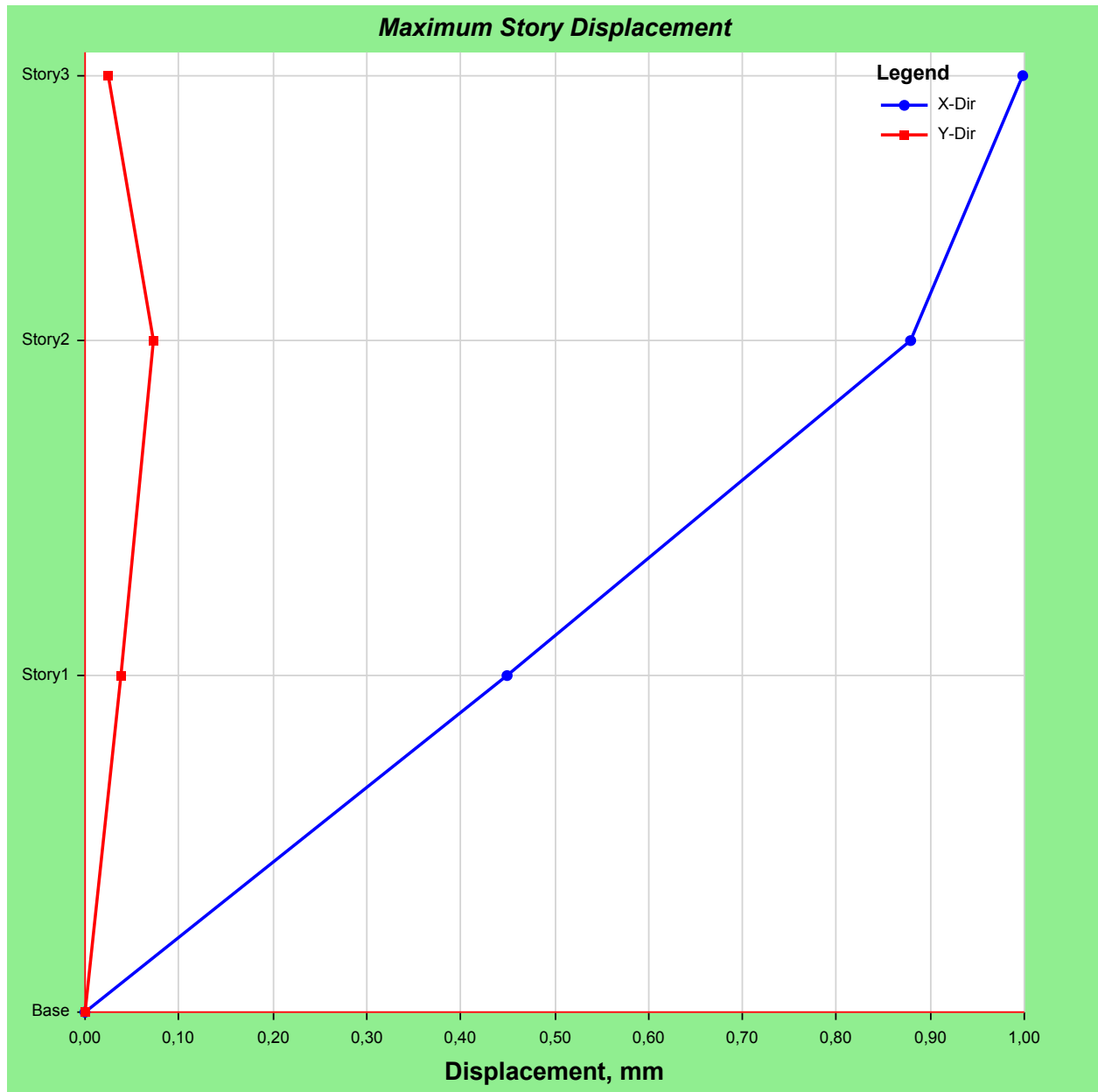
### Summary Description

This is story response output for a specified range of stories and a selected load case or load combination.

### Input Data

Name	StoryResp2	Story Range	All Stories
Display Type	Max story displ	Top Story	Story3
Load Case	RS <sub>x</sub>	Bottom Story	Base
Output Type	Not Applicable		

### Plot



## Story Response - Maximum Story Displacement

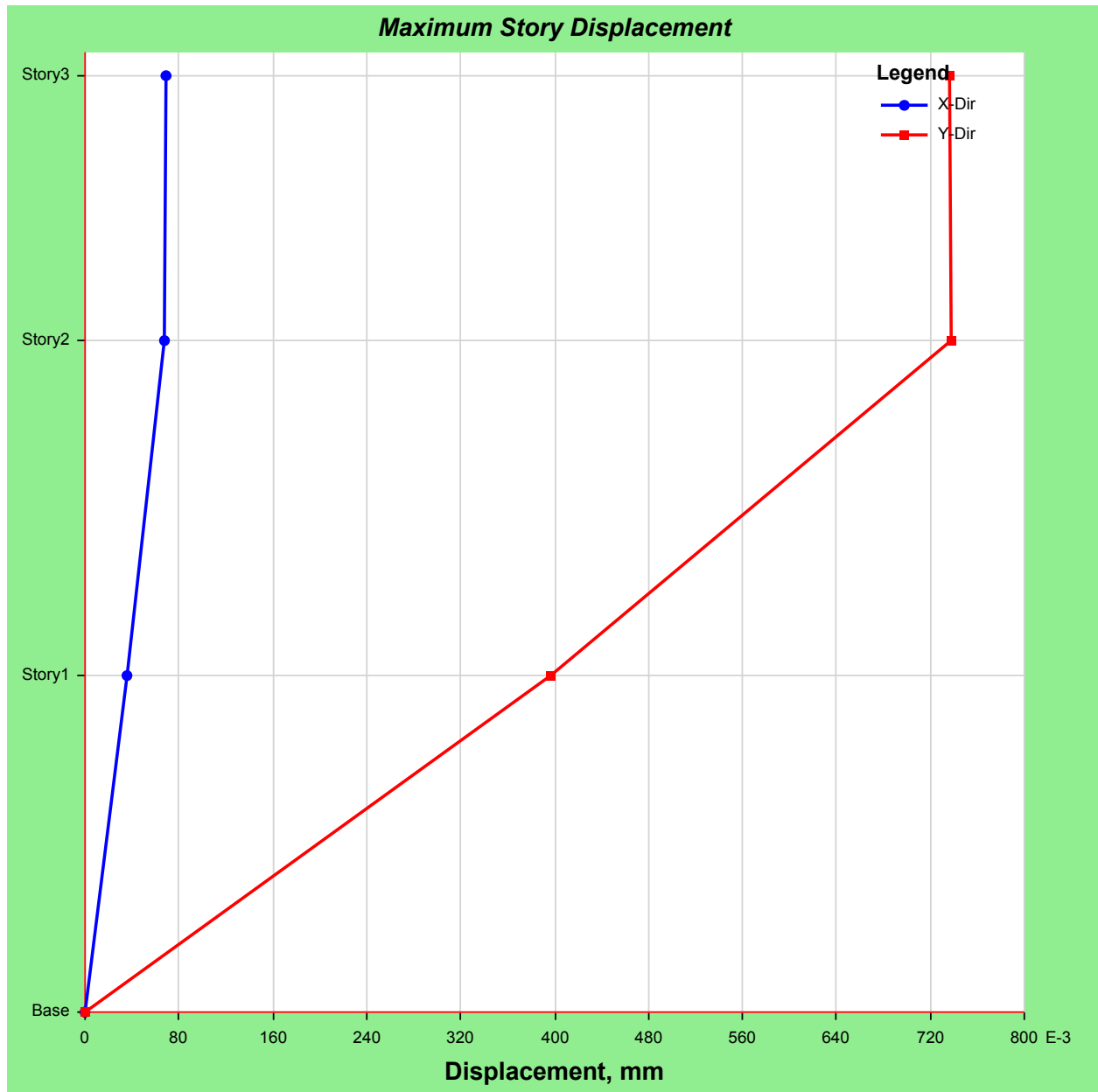
### Summary Description

This is story response output for a specified range of stories and a selected load case or load combination.

### Input Data

Name	StoryResp2		
Display Type	Max story displ	Story Range	All Stories
Load Case	RS <sub>y</sub>	Top Story	Story3
Output Type	Not Applicable	Bottom Story	Base

### Plot



## Story Response - Maximum Story Displacement

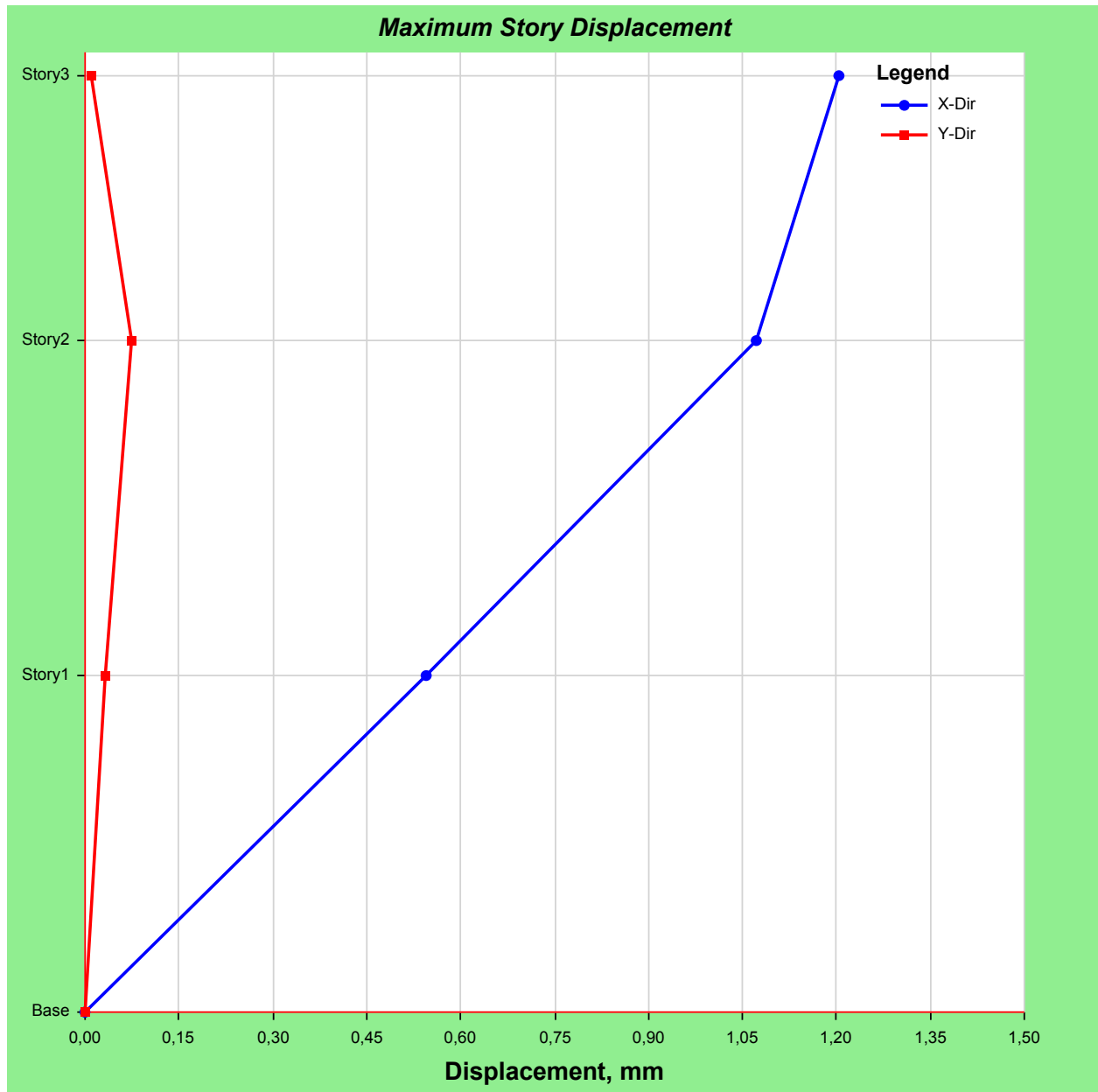
### Summary Description

This is story response output for a specified range of stories and a selected load case or load combination.

### Input Data

Name	StoryResp2		
Display Type	Max story displ	Story Range	All Stories
Load Case	Seismic X+e	Top Story	Story3
Output Type	Not Applicable	Bottom Story	Base

### Plot



## Story Response - Maximum Story Displacement

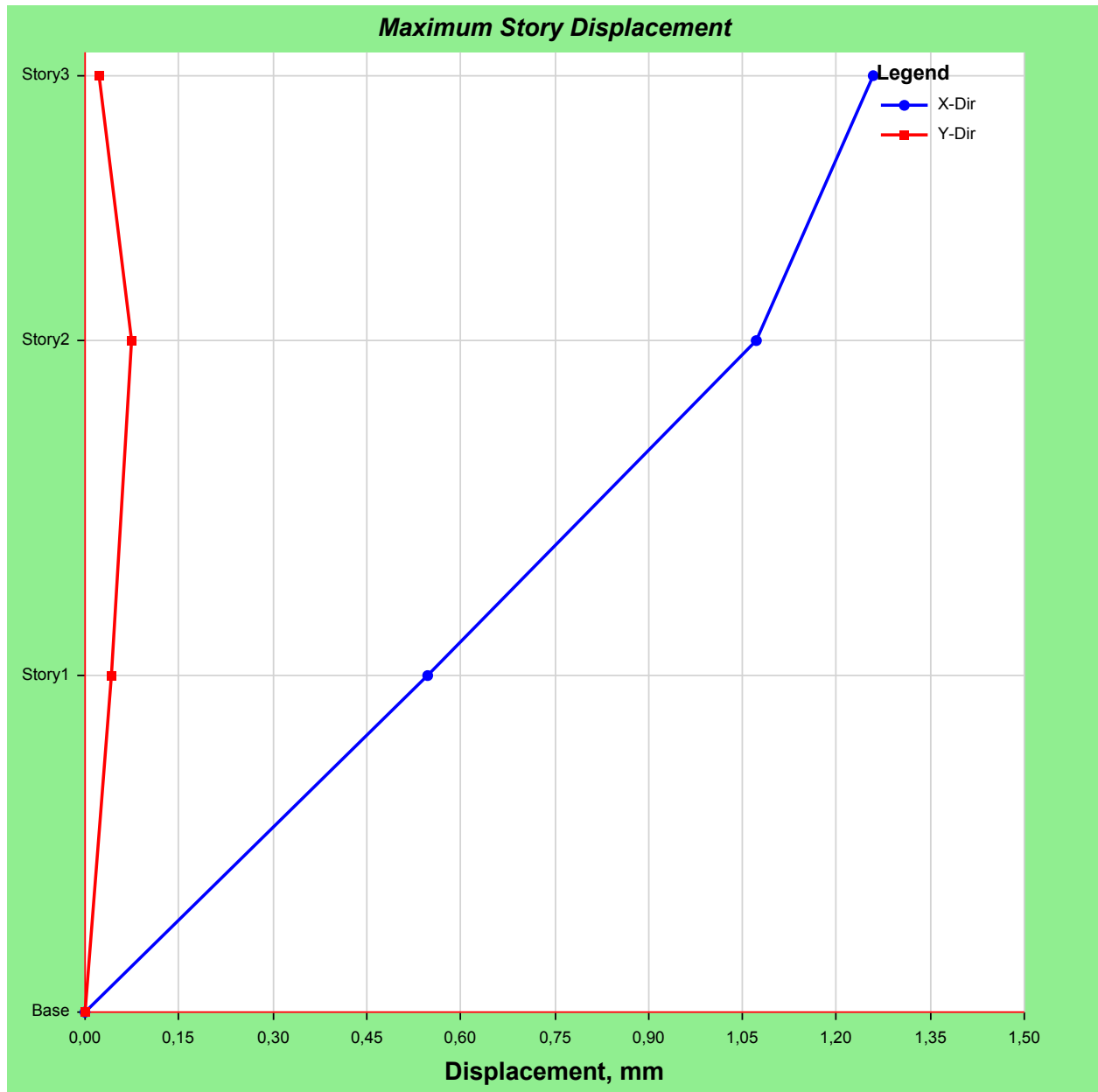
### Summary Description

This is story response output for a specified range of stories and a selected load case or load combination.

### Input Data

Name	StoryResp2	Story Range	All Stories
Display Type	Max story displ	Top Story	Story3
Load Case	Seismic X-e	Bottom Story	Base
Output Type	Not Applicable		

### Plot



## Story Response - Maximum Story Displacement

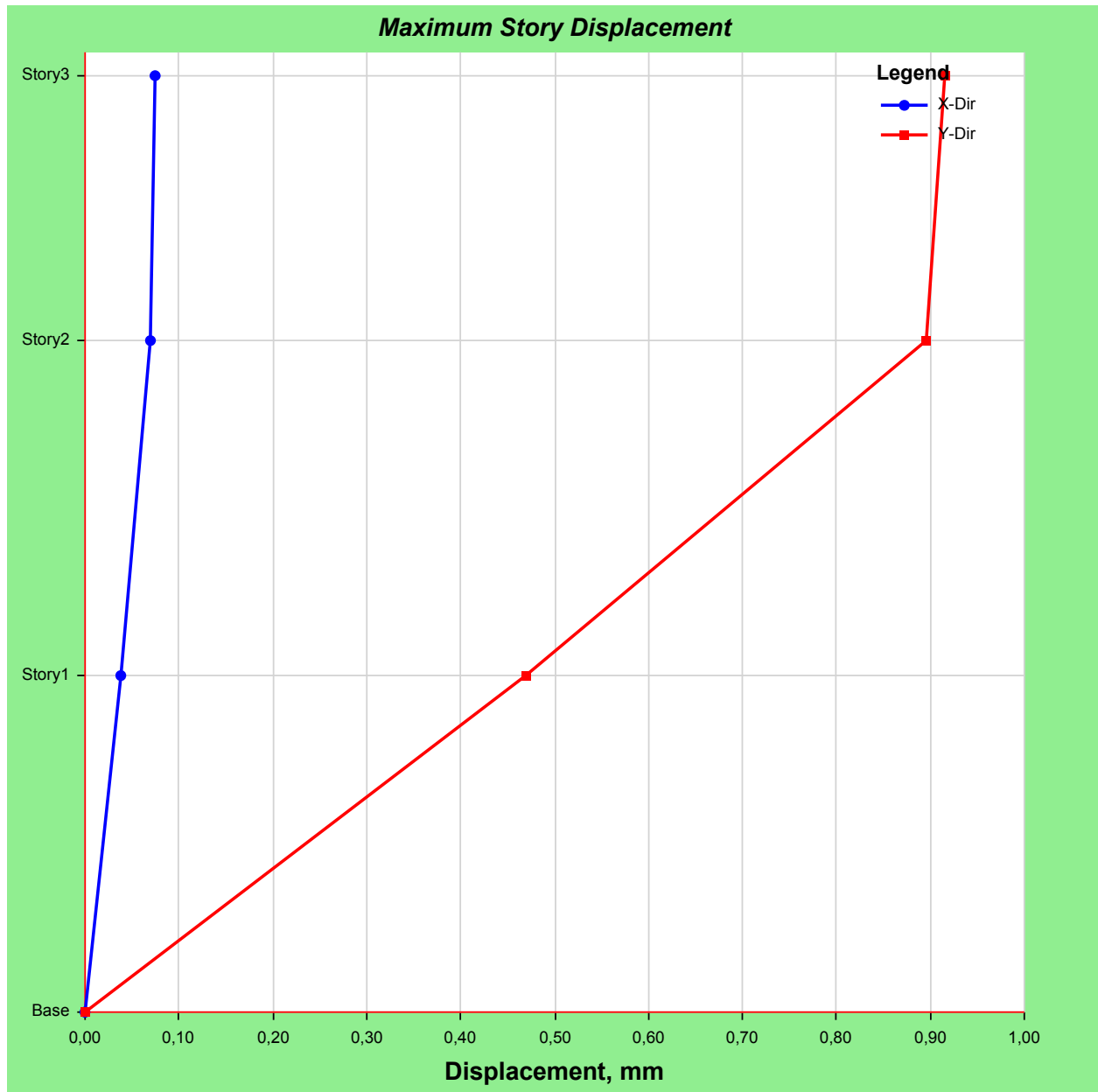
### Summary Description

This is story response output for a specified range of stories and a selected load case or load combination.

### Input Data

Name	StoryResp2		
Display Type	Max story displ	Story Range	All Stories
Load Case	Seismic Y+e	Top Story	Story3
Output Type	Not Applicable	Bottom Story	Base

### Plot



## Story Response - Maximum Story Displacement

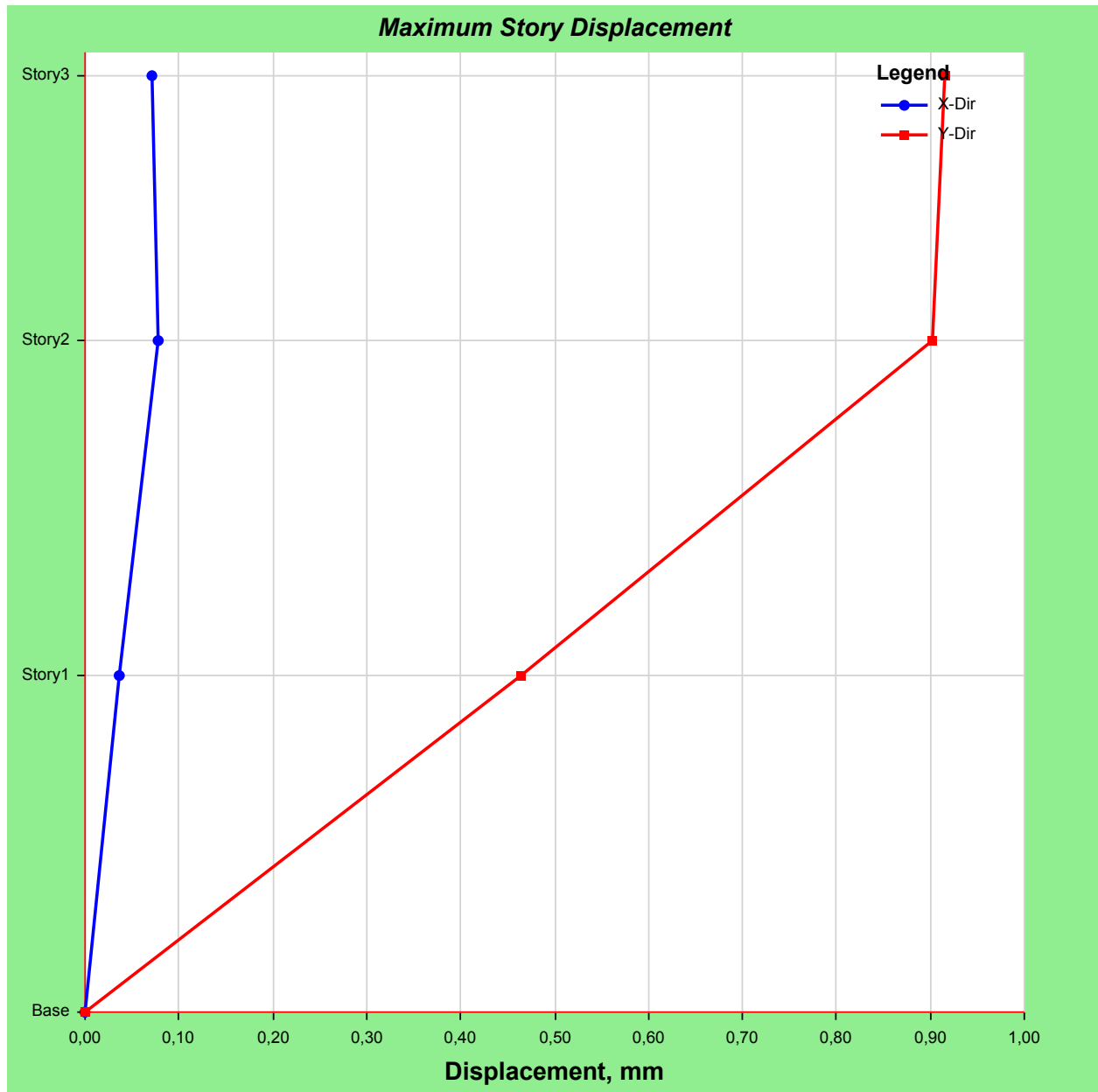
### Summary Description

This is story response output for a specified range of stories and a selected load case or load combination.

### Input Data

Name	StoryResp2		
Display Type	Max story displ	Story Range	All Stories
Load Case	Seismic Y-e	Top Story	Story3
Output Type	Not Applicable	Bottom Story	Base

### Plot



## Appendix B Detailed design of confined masonry structure

Table 31 Design for compression axial load. Flexo-compression resistance neglecting the contribution of masonry

Wall	$\Sigma A_{ci}$	$\bar{x}$ (mm)	$x_i$	$x_i - \bar{x}$ (mm)	$I_{ct}$	$\mu_u$ (tm)	$\mu_u$ (kNmm)	$\square_{pu}$ (kN)	$P_u$ (t)	$P_u$ (kN)	$P_{uc}$ (kN)	$P_{ut \leq 0}$ (kN)	$P_{uc \leq \emptyset P_n c}$ (kN)	$P_{ut \geq \emptyset P_n t}$ (kN)
1M-A.1	80000	625	1250	625	3,13E+10	0,026	258,72	0,21	4,35	42,65	21,53	21,12	ok!	ok!
1M-A.2	120000	833	2500	1667	3,33E+11	0,012	115,64	0,02	9,07	88,84	29,64	29,59	ok!	ok!
		417	1250	833	8,33E+10	0,012	115,64	0,05	9,07	88,84	29,66	29,57	ok!	ok!
		833	2500	1667	3,33E+11	0,012	115,64	0,02	9,07	88,84	29,64	29,59	ok!	ok!
1M-A.3	120000	833	2500	1667	3,33E+11	0,012	114,66	0,02	9,07	88,92	29,66	29,62	ok!	ok!
		417	1250	833	8,33E+10	0,012	114,66	0,05	9,07	88,92	29,69	29,59	ok!	ok!
		833	2500	1667	3,33E+11	0,012	114,66	0,02	9,07	88,92	29,66	29,62	ok!	ok!
1M-A.4	120000	717	2150	1433	2,47E+11	0,015	147,98	0,03	8,49	83,22	27,77	27,71	ok!	ok!
		417	1250	833	8,33E+10	0,015	147,98	0,06	8,49	83,22	27,80	27,68	ok!	ok!
		717	2150	1433	2,47E+11	0,015	147,98	0,03	8,49	83,22	27,77	27,71	ok!	ok!
1M-A.5	120000	717	2150	1433	2,47E+11	0,015	147,98	0,03	8,73	85,54	28,55	28,48	ok!	ok!
		417	1250	833	8,33E+10	0,015	147,98	0,06	8,73	85,54	28,57	28,45	ok!	ok!
		717	2150	1433	2,47E+11	0,015	147,98	0,03	8,73	85,54	28,55	28,48	ok!	ok!
1M-A.6	120000	800	2400	1600	3,07E+11	0,017	169,54	0,04	9,29	91,02	30,38	30,30	ok!	ok!
		417	1250	833	8,33E+10	0,017	169,54	0,07	9,29	91,02	30,41	30,27	ok!	ok!
		800	2400	1600	3,07E+11	0,017	169,54	0,04	9,29	91,02	30,38	30,30	ok!	ok!
1M-A.7	120000	800	2400	1600	3,07E+11	0,012	112,70	0,02	9,00	88,17	29,41	29,37	ok!	ok!
		417	1250	833	8,33E+10	0,012	112,70	0,05	9,00	88,17	29,44	29,35	ok!	ok!
		800	2400	1600	3,07E+11	0,012	112,70	0,02	9,00	88,17	29,41	29,37	ok!	ok!
1M-A.8	80000	625	1250	625	3,13E+10	0,021	204,82	0,16	4,36	42,76	21,54	-0,16	ok!	ok!
1M-B.1	80000	2100	4200	2100	3,53E+11	0,009	90,16	0,02	26,83	262,92	131,48	131,44	ok!	ok!
1M-B.2	80000	2100	4200	2100	3,53E+11	0,010	101,92	0,02	27,26	267,16	133,60	133,55	ok!	ok!

1M-B.3	80000	2100	4200	2100	3,53E+11	0,015	142,10	0,03	27,45	269,00	134,54	134,47	ok!	ok!
1M-B.4	80000	2100	4200	2100	3,53E+11	0,110	1073,10	0,26	25,19	246,84	123,67	123,16	ok!	ok!
1M-C.1	120000	2400	7200	4800	2,76E+12	0,003	27,44	0,00	35,11	344,13	114,71	114,71	ok!	ok!
		1400	4200	2800	9,41E+11	0,003	27,44	0,00	35,11	344,13	114,71	114,71	ok!	ok!
		2400	7200	4800	2,76E+12	0,003	27,44	0,00	35,11	344,13	114,71	114,71	ok!	ok!
1M-C.2	80000	450	900	450	1,62E+10	0,001	5,88	0,01	3,93	38,48	19,25	19,24	ok!	ok!
1M-C.3	80000	450	900	450	1,62E+10	0,001	5,88	0,01	4,11	40,28	20,15	20,13	ok!	ok!
1M-C.4	120000	2400	7200	4800	2,76E+12	0,001	7,84	0,00	31,53	308,99	103,00	102,99	ok!	ok!
		1400	4200	2800	9,41E+11	0,001	7,84	0,00	31,53	308,99	103,00	102,99	ok!	ok!
		2400	7200	4800	2,76E+12	0,001	7,84	0,00	31,53	308,99	103,00	102,99	ok!	ok!
1M-D.1	80000	625	1250	625	3,13E+10	3,806	37299,78	29,84	11,15	109,29	84,49	24,81	ok!	ok!
1M-D.2	120000	833	2500	1667	3,33E+11	0,061	599,76	0,12	10,68	104,71	35,02	34,78	ok!	ok!
		417	1250	833	8,33E+10	0,061	599,76	0,24	10,68	104,71	35,14	34,66	ok!	ok!
		833	2500	1667	3,33E+11	0,061	599,76	0,12	10,68	104,71	35,02	34,78	ok!	ok!
1M-D.3	120000	833	2500	1667	3,33E+11	0,059	577,22	0,12	10,56	103,52	34,62	34,39	ok!	ok!
		417	1250	833	8,33E+10	0,059	577,22	0,23	10,56	103,52	34,74	34,27	ok!	ok!
		833	2500	1667	3,33E+11	0,059	577,22	0,12	10,56	103,52	34,62	34,39	ok!	ok!
1M-D.4	120000	717	2150	1433	2,47E+11	0,055	542,92	0,13	9,18	89,96	30,11	29,86	ok!	ok!
		417	1250	833	8,33E+10	0,055	542,92	0,22	9,18	89,96	30,20	29,77	ok!	ok!
		717	2150	1433	2,47E+11	0,055	542,92	0,13	9,18	89,96	30,11	29,86	ok!	ok!
1M-D.5	120000	717	2150	1433	2,47E+11	0,045	441,98	0,10	7,15	70,09	23,47	23,26	ok!	ok!
		417	1250	833	8,33E+10	0,045	441,98	0,18	7,15	70,09	23,54	23,19	ok!	ok!
		717	2150	1433	2,47E+11	0,045	441,98	0,10	7,15	70,09	23,47	23,26	ok!	ok!
1M-D.6	120000	800	2400	1600	3,07E+11	0,060	584,08	0,12	10,52	103,10	34,49	34,24	ok!	ok!
		417	1250	833	8,33E+10	0,060	584,08	0,23	10,52	103,10	34,60	34,13	ok!	ok!
		800	2400	1600	3,07E+11	0,060	584,08	0,12	10,52	103,10	34,49	34,24	ok!	ok!
1M-D.7	120000	800	2400	1600	3,07E+11	0,061	600,74	0,13	12,00	117,61	39,33	39,08	ok!	ok!
		417	1250	833	8,33E+10	0,061	600,74	0,24	12,00	117,61	39,44	38,96	ok!	ok!

		800	2400	1600	3,07E+11	0,061	600,74	0,13	12,00	117,61	39,33	39,08	ok!	ok!
1M-D.8	80000	625	1250	625	3,13E+10	0,049	477,26	0,38	6,25	61,24	31,00	30,24	ok!	ok!
1M-1.1	80000	2100	4200	2100	3,53E+11	0,066	648,76	0,15	22,33	218,83	109,57	109,26	ok!	ok!
1M-1.2	80000	2100	4200	2100	3,53E+11	0,070	688,94	0,16	22,19	217,43	108,88	108,55	ok!	ok!
1M-2.1	80000	1600	3200	1600	2,05E+11	0,000	3,92	0,00	15,36	150,50	75,25	75,25	ok!	ok!
1M-2.2	80000	1600	3200	1600	2,05E+11	0,080	780,08	0,24	18,67	182,92	91,70	91,22	ok!	ok!
1M-2.3	80000	1600	3200	1600	2,05E+11	0,000	1,96	0,00	13,78	135,01	67,50	67,50	ok!	ok!
1M-3.1	80000	1550	3100	1550	1,92E+11	0,006	54,88	0,02	14,99	146,93	73,48	73,45	ok!	ok!
1M-3.2	80000	2100	4200	2100	3,53E+11	0,267	2619,54	0,62	29,92	293,25	147,25	146,00	ok!	ok!
1M-3.3	80000	1600	3200	1600	2,05E+11	0,021	208,74	0,07	13,82	135,42	67,77	67,64	ok!	ok!
1M-4.1	80000	1600	3200	1600	2,05E+11	0,028	275,38	0,09	14,97	146,71	73,44	73,27	ok!	ok!
1M-4.2	80000	1600	3200	1600	2,05E+11	0,011	108,78	0,03	23,78	233,04	116,55	116,49	ok!	ok!
1M-4.3	80000	2050	4100	2050	3,36E+11	0,023	227,36	0,06	20,31	199,08	99,60	99,49	ok!	ok!
1M-5.1	80000	1600	3200	1600	2,05E+11	0,028	276,36	0,09	15,50	151,85	76,01	75,84	ok!	ok!
1M-5.2	80000	1600	3200	1600	2,05E+11	0,010	101,92	0,03	23,72	232,50	116,28	116,22	ok!	ok!
1M-5.3	80000	800	1600	800	5,12E+10	0,020	196,98	0,12	9,21	90,29	45,27	45,02	ok!	ok!
1M-5.4	80000	800	1600	800	5,12E+10	0,012	120,54	0,08	5,46	53,55	26,85	26,70	ok!	ok!
1M-6.1	80000	1600	3200	1600	2,05E+11	0,004	40,18	0,01	14,57	142,82	71,42	71,40	ok!	ok!
1M-6.2	80000	2100	4200	2100	3,53E+11	0,373	3650,50	0,87	32,44	317,86	159,80	158,06	ok!	ok!
1M-6.3	80000	1600	3200	1600	2,05E+11	0,016	160,72	0,05	13,82	135,41	67,76	67,66	ok!	ok!
1M-7.1	80000	1600	3200	1600	2,05E+11	0,004	42,14	0,01	15,22	149,11	74,57	74,54	ok!	ok!
1M-7.2	80000	1600	3200	1600	2,05E+11	0,002	19,60	0,01	20,38	199,73	99,87	99,86	ok!	ok!
1M-7.3	80000	1600	3200	1600	2,05E+11	0,000	-0,98	0,00	15,33	150,21	75,10	75,10	ok!	ok!
1M-8.1	80000	2100	4200	2100	3,53E+11	0,197	1932,56	0,46	28,50	279,25	140,09	139,17	ok!	ok!
1M-8.2	80000	2100	4200	2100	3,53E+11	0,205	2012,92	0,48	29,25	286,62	143,79	142,83	ok!	ok!

Table 32 Design shear wall in the direction parallel to the plane

Wall	Pu (t)	Pu (N)	Vu (t)	Vu (N)	Lm (mm)	t (mm)	Ae (mm <sup>2</sup> )	f <sub>m</sub> (MPa)	V <sub>n</sub> (N)	$\frac{1}{6}\sqrt{f'_m}A_{mv}$	$V_n \leq \frac{1}{6}\sqrt{f'_m}A_{mv}$	ØV <sub>n</sub> (N)	$V_u \leq \phi V_n$
1M-A.1	0,52	5141,08	2,13	20833,82	1350	140	189000	8	46261	89095	ok!	27757	ok!
1M-A.2	0,19	1893,36	4,71	46161,92	2500	140	350000	8	83127	164992	ok!	49876	ok!
1M-A.3	0,16	1537,62	4,71	46184,46	2500	140	350000	8	83008	164992	ok!	49805	ok!
1M-A.4	1,31	12837,02	4,03	39474,40	2150	140	301000	8	75225	141893	ok!	45135	ok!
1M-A.5	0,96	9366,84	4,09	40083,96	2150	140	301000	8	74069	141893	ok!	44441	ok!
1M-A.6	0,38	3682,84	4,79	46911,62	2500	140	350000	8	83723	164992	ok!	50234	ok!
1M-A.7	0,16	1542,52	4,71	46162,90	2500	140	350000	8	83010	164992	ok!	49806	ok!
1M-A.8	0,60	5886,86	2,14	20977,88	1350	140	189000	8	46510	89095	ok!	27906	ok!
1M-B.1	3,37	33053,44	9,95	97501,18	4200	200	840000	8	209008	395980	ok!	125405	ok!
1M-B.2	2,16	21157,22	10,95	107352,14	4200	200	840000	8	205042	395980	ok!	123025	ok!
1M-B.3	1,78	17410,68	11,01	107852,92	4200	200	840000	8	203793	395980	ok!	122276	ok!
1M-B.4	3,27	32058,74	9,93	97299,30	4200	200	840000	8	208676	395980	ok!	125206	ok!
1M-C.1	3,64	35679,84	18,55	181821,36	7200	200	1440000	8	351305	678823	ok!	210783	ok!
1M-C.2	0,58	5659,50	2,15	21033,74	1000	200	200000	8	49027	94281	ok!	29416	ok!
1M-C.3	0,60	5899,60	2,15	21021,98	1000	200	200000	8	49107	94281	ok!	29464	ok!
1M-C.4	3,75	36738,24	18,61	182403,48	7200	200	1440000	8	351657	678823	ok!	210994	ok!
1M-D.1	0,20	2002,14	2,12	20781,88	1350	140	189000	8	45215	89095	ok!	27129	ok!
1M-D.2	0,21	2042,32	4,69	45932,60	2500	140	350000	8	83177	164992	ok!	49906	ok!
1M-D.3	0,20	1992,34	4,65	45571,96	2500	140	350000	8	83160	164992	ok!	49896	ok!
1M-D.4	0,50	4870,60	3,86	37829,96	2150	140	301000	8	72570	141893	ok!	43542	ok!
1M-D.5	0,41	4006,24	3,86	37830,94	2150	140	301000	8	72282	141893	ok!	43369	ok!
1M-D.6	0,11	1054,48	4,69	45951,22	2500	140	350000	8	82847	164992	ok!	49708	ok!
1M-D.7	0,19	1829,66	4,69	45924,76	2500	140	350000	8	83106	164992	ok!	49863	ok!
1M-D.8	0,60	5891,76	2,12	20781,88	1350	140	189000	8	46512	89095	ok!	27907	ok!

1M-1.1	0,45	4449,20	11,23	110061,84	5350	200	1070000	8	253684	504403	ok!	152211	ok!
1M-1.2	1,01	9855,86	11,27	110405,82	5350	200	1070000	8	255487	504403	ok!	153292	ok!
1M-2.1	2,05	20138,02	5,64	55304,34	3200	140	448000	8	112307	211189	ok!	67384	ok!
1M-2.2	1,94	19024,74	5,61	54945,66	3200	140	448000	8	111936	211189	ok!	67162	ok!
1M-2.3	1,91	18745,44	5,69	55723,78	3200	140	448000	8	111843	211189	ok!	67106	ok!
1M-3.1	1,79	17498,88	5,40	52957,24	3000	140	420000	8	104828	197990	ok!	62897	ok!
1M-3.2	1,89	18526,90	7,67	75211,08	4000	140	560000	8	138169	263987	ok!	82901	ok!
1M-3.3	1,73	16999,08	5,42	53163,04	3000	140	420000	8	104661	197990	ok!	62797	ok!
1M-4.1	0,46	4496,24	5,58	54686,94	3000	200	600000	8	142920	282843	ok!	85752	ok!
1M-4.2	0,64	6297,48	8,90	87244,50	4000	200	800000	8	190661	377124	ok!	114397	ok!
1M-4.3	2,08	20421,24	8,42	82493,46	4000	200	800000	8	195369	377124	ok!	117221	ok!
1M-5.1	0,45	4363,94	5,59	54825,12	3000	200	600000	8	142876	282843	ok!	85726	ok!
1M-5.2	0,50	4935,28	8,69	85208,06	3000	200	600000	8	143066	282843	ok!	85840	ok!
1M-5.3	1,62	15896,58	2,70	26457,06	1500	200	300000	8	76010	141421	ok!	45606	ok!
1M-5.4	0,66	6475,84	2,21	21636,44	1500	200	300000	8	72869	141421	ok!	43722	ok!
1M-6.1	1,09	10698,66	5,65	55355,30	3000	140	420000	8	102561	197990	ok!	61537	ok!
1M-6.2	2,41	23626,82	7,84	76867,28	4200	140	588000	8	146469	277186	ok!	87881	ok!
1M-6.3	1,72	16873,64	5,50	53927,44	3200	140	448000	8	111219	211189	ok!	66731	ok!
1M-7.1	2,05	20094,90	5,78	56596,96	3200	140	448000	8	112293	211189	ok!	67376	ok!
1M-7.2	2,06	20188,00	5,79	56742,00	3200	140	448000	8	112324	211189	ok!	67394	ok!
1M-7.3	2,34	22962,38	5,47	53618,74	3000	140	420000	8	106649	197990	ok!	63989	ok!
1M-8.1	0,55	5420,38	11,52	112894,04	4200	200	840000	8	199797	395980	ok!	119878	ok!
1M-8.2	0,94	9204,16	11,57	113367,38	4200	200	840000	8	201058	395980	ok!	120635	ok!

Table 33 Verification for crushing

Wall	t (mm)	h (mm)	lw (mm)	h' (mm)	h'/t	Re	Amd (mm <sup>2</sup> )	Pnd (N)	Vu (t)	Vu (N)	Pud (N)	Ø×Pnd	$P_{ud} \leq \phi P_{nd}$
1M-A.1	140	2800	1350	2957	21,12	0,75	82796	316707,5	2,13	20833,8	45633,8	190024,5	ok!
1M-A.2	140	2800	2500	2957	21,12	0,75	82796	316707,5	4,71	46161,9	54600,3	190024,5	ok!
1M-A.3	140	2800	2500	2957	21,12	0,75	82796	316707,5	4,71	46184,5	54627,0	190024,5	ok!
1M-A.4	140	2800	2150	2957	21,12	0,75	82796	316707,5	4,03	39474,4	54291,1	190024,5	ok!
1M-A.5	140	2800	2150	2957	21,12	0,75	82796	316707,5	4,09	40084,0	55129,4	190024,5	ok!
1M-A.6	140	2800	2500	2957	21,12	0,75	82796	316707,5	4,79	46911,6	55487,1	190024,5	ok!
1M-A.7	140	2800	2500	2957	21,12	0,75	82796	316707,5	4,71	46162,9	54601,5	190024,5	ok!
1M-A.8	140	2800	1350	2957	21,12	0,75	82796	316707,5	2,14	20977,9	45949,3	190024,5	ok!
1M-B.1	140	2800	4200	4720	33,71	0,36	132160	240646,3	9,95	97501,2	109572,8	144387,8	ok!
1M-B.2	140	2800	4200	4720	33,71	0,36	132160	240646,3	10,95	107352,1	120643,4	144387,8	ok!
1M-B.3	140	2800	4200	4720	33,71	0,36	132160	240646,3	11,01	107852,9	121206,1	144387,8	ok!
1M-B.4	140	2800	4200	4720	33,71	0,36	132160	240646,3	9,93	97299,3	109345,9	144387,8	ok!
1M-C.1	140	2800	7200	4720	33,71	0,36	132160	240646,3	18,55	181821,4	119194,0	144387,8	ok!
1M-C.2	140	2800	1000	2668	19,06	0,79	74704	303738,1	2,15	21033,7	56118,0	182242,8	ok!
1M-C.3	140	2800	1000	2668	19,06	0,79	74704	303738,1	2,15	21022,0	56086,6	182242,8	ok!
1M-C.4	140	2800	7200	4720	33,71	0,36	132160	240646,3	18,61	182403,5	119575,6	144387,8	ok!
1M-D.1	140	2800	1350	2957	21,12	0,75	82796	316707,5	6,20	60784,5	133140,6	190024,5	ok!
1M-D.2	140	2800	2500	2957	21,12	0,75	82796	316707,5	4,69	45932,6	54329,1	190024,5	ok!
1M-D.3	140	2800	2500	2957	21,12	0,75	82796	316707,5	4,65	45572,0	53902,5	190024,5	ok!
1M-D.4	140	2800	2150	2957	21,12	0,75	82796	316707,5	3,86	37830,0	52029,4	190024,5	ok!
1M-D.5	140	2800	2150	2957	21,12	0,75	82796	316707,5	3,86	37830,9	52030,7	190024,5	ok!
1M-D.6	140	2800	2500	2957	21,12	0,75	82796	316707,5	4,69	45951,2	54351,1	190024,5	ok!
1M-D.7	140	2800	2500	2957	21,12	0,75	82796	316707,5	4,69	45924,8	54319,8	190024,5	ok!
1M-D.8	140	2800	1350	2957	21,12	0,75	82796	316707,5	2,12	20781,9	45520,0	190024,5	ok!

1M-1.1	200	2800	4200	4720	23,60	0,68	188800	661447	11,23	110061,8	123688,5	396868,2	ok!
1M-1.2	140	2800	4200	4720	33,71	0,36	132160	240646,3	11,27	110405,8	124075,1	144387,8	ok!
1M-2.1	140	2800	3200	3960	28,29	0,55	110880	310216,5	5,64	55304,3	68439,1	186129,9	ok!
1M-2.2	140	2800	3200	3960	28,29	0,55	110880	310216,5	5,61	54945,7	67995,3	186129,9	ok!
1M-2.3	140	2800	3200	3960	28,29	0,55	110880	310216,5	5,69	55723,8	68958,2	186129,9	ok!
1M-3.1	140	2800	3200	3960	28,29	0,55	110880	310216,5	5,40	52957,2	65534,6	186129,9	ok!
1M-3.2	140	2800	4200	4720	33,71	0,36	132160	240646,3	7,67	75211,1	84522,9	144387,8	ok!
1M-3.3	140	2800	3200	3960	28,29	0,55	110880	310216,5	5,42	53163,0	65789,3	186129,9	ok!
1M-4.1	140	2800	3200	3960	28,29	0,55	110880	310216,5	5,58	54686,9	67675,1	186129,9	ok!
1M-4.2	140	2800	3200	3960	28,29	0,55	110880	310216,5	8,90	87244,5	107965,1	186129,9	ok!
1M-4.3	140	2800	4200	4720	33,71	0,36	132160	240646,3	8,42	82493,5	92706,9	144387,8	ok!
1M-5.1	140	2800	3200	3960	28,29	0,55	110880	310216,5	5,59	54825,1	67846,1	186129,9	ok!
1M-5.2	140	2800	3200	3960	28,29	0,55	110880	310216,5	8,69	85208,1	105445,0	186129,9	ok!
1M-5.3	140	2800	1700	3087	22,05	0,72	86436	320573,8	2,70	26457,1	48042,9	192344,3	ok!
1M-5.4	140	2800	1700	3087	22,05	0,72	86436	320573,8	2,21	21636,4	39289,2	192344,3	ok!
1M-6.1	140	2800	3200	3960	28,29	0,55	110880	310216,5	5,65	55355,3	68502,2	186129,9	ok!
1M-6.2	140	2800	4200	4720	33,71	0,36	132160	240646,3	7,84	76867,3	86384,2	144387,8	ok!
1M-6.3	140	2800	3200	3960	28,29	0,55	110880	310216,5	5,50	53927,4	66735,2	186129,9	ok!
1M-7.1	140	2800	3200	3960	28,29	0,55	110880	310216,5	5,78	56597,0	70038,7	186129,9	ok!
1M-7.2	140	2800	3200	3960	28,29	0,55	110880	310216,5	5,79	56742,0	70218,2	186129,9	ok!
1M-7.3	140	2800	3200	3960	28,29	0,55	110880	310216,5	5,47	53618,7	66353,2	186129,9	ok!
1M-8.1	140	2800	4200	4720	33,71	0,36	132160	240646,3	11,52	112894,0	126871,4	144387,8	ok!
1M-8.2	140	2800	4200	4720	33,71	0,36	132160	240646,3	11,57	113367,4	127403,3	144387,8	ok!

Table 34 Checking the confining elements in shear. Columns and Beams

Confining columns 1st floor																
Wall	Lc (mm)	Lw (mm)	V <sub>u</sub> (t)	V <sub>u</sub> (N)	V <sub>uc</sub> (N)	S (mm)	S/2 (mm)	bw (mm)	d (mm)	s max (mm)	Av (mm <sup>2</sup> )	Vc (N)	Vs (N)	Vn (N)	Ø×V <sub>nc</sub>	V <sub>uc</sub> ≤ φV <sub>nc</sub>
1M-A.1	1150	1350	2,13	20833,82	8873,664	100	50	140	100	150	101	10906,53	28280	39186,53	29389,9	ok!
1M-A.2	1150	2500	4,71	46161,92	10617,24	100	50	140	100	150	101	10906,53	28280	39186,53	29389,9	ok!
1M-A.3	1150	2500	4,71	46184,46	10622,43	100	50	140	100	150	101	10906,53	28280	39186,53	29389,9	ok!
1M-A.4	1150	2150	4,03	39474,4	10557,11	100	50	140	100	150	101	10906,53	28280	39186,53	29389,9	ok!
1M-A.5	1150	2150	4,09	40083,96	10720,13	100	50	140	100	150	101	10906,53	28280	39186,53	29389,9	ok!
1M-A.6	1150	2500	4,79	46911,62	10789,67	100	50	140	100	150	101	10906,53	28280	39186,53	29389,9	ok!
1M-A.7	1150	2500	4,71	46162,9	10617,47	100	50	140	100	150	101	10906,53	28280	39186,53	29389,9	ok!
1M-A.8	1150	1350	2,14	20977,88	8935,023	100	50	140	100	150	101	10906,53	28280	39186,53	29389,9	ok!
1M-B.1	4000	4200	9,95	97501,18	46429,13	100	50	200	160	150	101	24929,21	45248	70177,21	52632,91	ok!
1M-B.2	4000	4200	10,95	107352,1	51120,07	100	50	200	160	150	101	24929,21	45248	70177,21	52632,91	ok!
1M-B.3	4000	4200	11,01	107852,9	51358,53	100	50	200	160	150	101	24929,21	45248	70177,21	52632,91	ok!
1M-B.4	4000	4200	9,93	97299,3	46333	100	50	200	160	150	101	24929,21	45248	70177,21	52632,91	ok!
1M-C.1	4000	7200	18,55	181821,4	50505,93	100	50	200	160	150	101	24929,21	45248	70177,21	52632,91	ok!
1M-C.2	2800	3000	2,15	21033,74	9815,745	100	50	140	100	150	101	10906,53	28280	39186,53	29389,9	ok!
1M-C.3	4000	7200	2,15	21021,98	5839,439	100	50	140	100	150	101	10906,53	28280	39186,53	29389,9	ok!
1M-C.4	4000	7200	18,61	182403,5	50667,63	100	50	200	160	150	101	24929,21	45248	70177,21	52632,91	ok!
1M-D.1	1150	1350	6,20	60784,5	25889,69	100	50	140	100	150	101	10906,53	28280	39186,53	29389,9	ok!
1M-D.2	1150	2500	4,69	45932,6	10564,5	100	50	140	100	150	101	10906,53	28280	39186,53	29389,9	ok!
1M-D.3	1150	2500	4,65	45571,96	10481,55	100	50	140	100	150	101	10906,53	28280	39186,53	29389,9	ok!
1M-D.4	1150	2150	3,86	37829,96	10117,31	100	50	140	100	150	101	10906,53	28280	39186,53	29389,9	ok!
1M-D.5	1150	2150	3,86	37830,94	10117,58	100	50	140	100	150	101	10906,53	28280	39186,53	29389,9	ok!
1M-D.6	1150	2500	4,69	45951,22	10568,78	100	50	140	100	150	101	10906,53	28280	39186,53	29389,9	ok!
1M-D.7	1150	2500	4,69	45924,76	10562,69	100	50	140	100	150	101	10906,53	28280	39186,53	29389,9	ok!

1M-D.8	1150	1350	2,12	20781,88	8851,541	100	50	140	100	150	101	10906,53	28280	39186,53	29389,9	ok!
1M-1.1	5350	5550	11,23	110061,8	53047,82	150	75	200	160	130	101	24929,21	52209,23	77138,44	57853,83	ok!
1M-1.2	5350	5550	11,27	110405,8	53213,62	100	50	200	160	130	101	24929,21	52209,23	77138,44	57853,83	ok!
1M-2.1	3000	3200	5,64	55304,34	25923,91	100	50	140	100	150	101	10906,53	28280	39186,53	29389,9	ok!
1M-2.2	3000	3200	5,61	54945,66	25755,78	100	50	140	100	150	101	10906,53	28280	39186,53	29389,9	ok!
1M-2.3	3000	3200	5,69	55723,78	26120,52	100	50	140	100	150	101	10906,53	28280	39186,53	29389,9	ok!
1M-3.1	3000	3200	5,40	52957,24	24823,71	100	50	140	100	150	101	10906,53	28280	39186,53	29389,9	ok!
1M-3.2	4000	4200	7,67	75211,08	35814,8	100	50	200	160	150	101	24929,21	45248	70177,21	52632,91	ok!
1M-3.3	3000	3200	5,42	53163,04	24920,18	100	50	140	100	150	101	10906,53	28280	39186,53	29389,9	ok!
1M-4.1	3000	3200	5,58	54686,94	25634,5	100	50	200	160	150	101	24929,21	45248	70177,21	52632,91	ok!
1M-4.2	3000	3200	8,90	87244,5	40895,86	100	50	200	160	150	101	24929,21	45248	70177,21	52632,91	ok!
1M-4.3	3000	3200	8,42	82493,46	38668,81	100	50	200	160	150	101	24929,21	45248	70177,21	52632,91	ok!
1M-5.1	3000	3200	5,59	54825,12	25699,28	100	50	200	160	150	101	24929,21	45248	70177,21	52632,91	ok!
1M-5.2	3000	3200	8,69	85208,06	39941,28	100	50	200	160	150	101	24929,21	45248	70177,21	52632,91	ok!
1M-5.3	1500	1700	2,70	26457,06	11672,23	100	50	200	160	150	101	24929,21	45248	70177,21	52632,91	ok!
1M-5.4	1500	1700	2,21	21636,44	9545,488	100	50	200	160	150	101	24929,21	45248	70177,21	52632,91	ok!
1M-6.1	3000	3200	5,65	55355,3	25947,8	100	50	140	100	150	101	10906,53	28280	39186,53	29389,9	ok!
1M-6.2	4000	4200	7,84	76867,28	36603,47	100	50	200	160	150	101	24929,21	45248	70177,21	52632,91	ok!
1M-6.3	3000	3200	5,50	53927,44	25278,49	100	50	140	100	150	101	10906,53	28280	39186,53	29389,9	ok!
1M-7.1	3000	3200	5,78	56596,96	26529,83	100	50	140	100	150	101	10906,53	28280	39186,53	29389,9	ok!
1M-7.2	3000	3200	5,79	56742	26597,81	100	50	140	100	150	101	10906,53	28280	39186,53	29389,9	ok!
1M-7.3	3000	3200	5,47	53618,74	25133,78	100	50	140	100	150	101	10906,53	28280	39186,53	29389,9	ok!
1M-8.1	5350	5550	11,52	112894	54412,89	100	50	200	160	130	101	24929,21	52209,23	77138,44	57853,83	ok!
1M-8.2	5350	5550	11,57	113367,4	54641,03	100	50	200	160	130	101	24929,21	52209,23	77138,44	57853,83	ok!
Confining beams 1st floor																
Wall	hp (mm)	Lw (mm)	Vu (t)	Vu (N)	Vuc (N)	S (mm)	S/2 (mm)	bw (mm)	d (mm)	s max (mm)	Av (mm <sup>2</sup> )	Vc (N)	Vs (N)	Vn (N)	Ø×Vnc	$V_{uc} \leq \phi V_{nc}$
1M-A.1	2600	1350	2,13	20833,82	20062,2	150	75	150	110	150	101	12854,12	31108	43962,12	32971,59	ok!

1M-A.2	2600	2500	4,71	46161,92	24004,2	150	75	150	110	150	101	12854,12	31108	43962,12	32971,59	ok!
1M-A.3	2600	2500	4,71	46184,46	24015,92	150	75	150	110	150	101	12854,12	31108	43962,12	32971,59	ok!
1M-A.4	2600	2150	4,03	39474,4	23868,24	150	75	150	110	150	101	12854,12	31108	43962,12	32971,59	ok!
1M-A.5	2600	2150	4,09	40083,96	24236,81	150	75	150	110	150	101	12854,12	31108	43962,12	32971,59	ok!
1M-A.6	2600	2500	4,79	46911,62	24394,04	150	75	150	110	150	101	12854,12	31108	43962,12	32971,59	ok!
1M-A.7	2600	2500	4,71	46162,9	24004,71	150	75	150	110	150	101	12854,12	31108	43962,12	32971,59	ok!
1M-A.8	2600	1350	2,14	20977,88	20200,92	150	75	150	110	150	101	12854,12	31108	43962,12	32971,59	ok!
1M-B.1	2600	4200	9,95	97501,18	30178,94	150	75	150	110	150	101	12854,12	31108	43962,12	32971,59	ok!
1M-B.2	2600	4200	10,95	107352,1	33228,04	150	75	200	160	150	101	24929,21	45248	70177,21	52632,91	ok!
1M-B.3	2600	4200	11,01	107852,9	33383,05	150	75	200	160	150	101	24929,21	45248	70177,21	52632,91	ok!
1M-B.4	2600	4200	9,93	97299,3	30116,45	150	75	150	110	150	101	12854,12	31108	43962,12	32971,59	ok!
1M-C.1	2600	7200	18,55	181821,4	32828,86	150	75	150	110	150	101	12854,12	31108	43962,12	32971,59	ok!
1M-C.2	2600	3000	2,15	21033,74	9114,621	150	75	150	110	150	101	12854,12	31108	43962,12	32971,59	ok!
1M-C.3	2600	1000	2,15	21021,98	27328,57	150	75	150	110	150	101	12854,12	31108	43962,12	32971,59	ok!
1M-C.4	2600	7200	18,61	182403,5	32933,96	150	75	200	160	150	101	24929,21	45248	70177,21	52632,91	ok!
1M-D.1	2600	1350	4,70	46060	44354,07	150	75	200	160	150	101	24929,21	45248	70177,21	52632,91	ok!
1M-D.2	2600	2500	4,69	45932,6	23884,95	150	75	150	110	150	101	12854,12	31108	43962,12	32971,59	ok!
1M-D.3	2600	2500	4,65	45571,96	23697,42	150	75	150	110	150	101	12854,12	31108	43962,12	32971,59	ok!
1M-D.4	2600	2150	3,86	37829,96	22873,93	150	75	150	110	150	101	12854,12	31108	43962,12	32971,59	ok!
1M-D.5	2600	2150	3,86	37830,94	22874,52	150	75	150	110	150	101	12854,12	31108	43962,12	32971,59	ok!
1M-D.6	2600	2500	4,69	45951,22	23894,63	150	75	150	110	150	101	12854,12	31108	43962,12	32971,59	ok!
1M-D.7	2600	2500	4,69	45924,76	23880,88	150	75	150	110	150	101	12854,12	31108	43962,12	32971,59	ok!
1M-D.8	2600	1350	2,12	20781,88	20012,18	150	75	150	110	150	101	12854,12	31108	43962,12	32971,59	ok!
1M-1.1	2600	4200	11,23	110061,8	34066,76	150	75	200	160	150	101	24929,21	45248	70177,21	52632,91	ok!
1M-1.2	2600	4200	11,27	110405,8	34173,23	150	75	200	160	150	101	24929,21	45248	70177,21	52632,91	ok!
1M-2.1	2600	3200	5,64	55304,34	22467,39	150	75	150	110	150	101	12854,12	31108	43962,12	32971,59	ok!
1M-2.2	2600	3200	5,61	54945,66	22321,67	150	75	150	110	150	101	12854,12	31108	43962,12	32971,59	ok!
1M-2.3	2600	3200	5,69	55723,78	22637,79	150	75	150	110	150	101	12854,12	31108	43962,12	32971,59	ok!

1M-3.1	2600	3200	5,40	52957,24	21513,88	150	75	150	110	150	101	12854,12	31108	43962,12	32971,59	ok!
1M-3.2	2600	4200	7,67	75211,08	23279,62	150	75	150	110	150	101	12854,12	31108	43962,12	32971,59	ok!
1M-3.3	2600	3200	5,42	53163,04	21597,49	150	75	150	110	150	101	12854,12	31108	43962,12	32971,59	ok!
1M-4.1	2600	3200	5,58	54686,94	22216,57	150	75	150	110	150	101	12854,12	31108	43962,12	32971,59	ok!
1M-4.2	2600	3200	8,90	87244,5	35443,08	150	75	200	160	150	101	24929,21	45248	70177,21	52632,91	ok!
1M-4.3	2600	3200	8,42	82493,46	33512,97	150	75	200	160	150	101	24929,21	45248	70177,21	52632,91	ok!
1M-5.1	2600	3200	5,59	54825,12	22272,71	150	75	200	160	150	101	24929,21	45248	70177,21	52632,91	ok!
1M-5.2	2600	3200	8,69	85208,06	34615,77	150	75	200	160	150	101	24929,21	45248	70177,21	52632,91	ok!
1M-5.3	2600	1700	2,70	26457,06	20231,87	150	75	150	110	150	101	12854,12	31108	43962,12	32971,59	ok!
1M-5.4	2600	1700	2,21	21636,44	16545,51	150	75	150	110	150	101	12854,12	31108	43962,12	32971,59	ok!
1M-6.1	2600	3200	5,65	55355,3	22488,09	150	75	150	110	150	101	12854,12	31108	43962,12	32971,59	ok!
1M-6.2	2600	4200	7,84	76867,28	23792,25	150	75	150	110	150	101	12854,12	31108	43962,12	32971,59	ok!
1M-6.3	2600	3200	5,50	53927,44	21908,02	150	75	150	110	150	101	12854,12	31108	43962,12	32971,59	ok!
1M-7.1	2600	3200	5,78	56596,96	22992,52	150	75	150	110	150	101	12854,12	31108	43962,12	32971,59	ok!
1M-7.2	2600	3200	5,79	56742	23051,44	150	75	150	110	150	101	12854,12	31108	43962,12	32971,59	ok!
1M-7.3	2600	3200	5,47	53618,74	21782,61	150	75	150	110	150	101	12854,12	31108	43962,12	32971,59	ok!
1M-8.1	2600	4200	11,52	112894	34943,39	150	75	200	160	150	101	24929,21	45248	70177,21	52632,91	ok!
1M-8.2	2600	4200	11,57	113367,4	35089,9	150	75	200	160	150	101	24929,21	45248	70177,21	52632,91	ok!

Table 35 Design of the longitudinal steel of the confinement beam

Wall	Lc (mm)	Lw (mm)	V <sub>u</sub> (t)	V <sub>u</sub> (N)	P <sub>ut</sub> (N)	A <sub>st</sub> (mm <sup>2</sup> )	Ø×Pnt (N)	$-P_{ut} \leq -\phi P_{nt}$
1M-A.1	1150	1350	2,13	20833,82	17747,33	452	170856	ok!
1M-A.2	1150	2500	4,71	46161,92	21234,48	452	170856	ok!
1M-A.3	1150	2500	4,71	46184,46	21244,85	452	170856	ok!
1M-A.4	1150	2150	4,03	39474,4	21114,21	452	170856	ok!
1M-A.5	1150	2150	4,09	40083,96	21440,26	452	170856	ok!
1M-A.6	1150	2500	4,79	46911,62	21579,35	452	170856	ok!
1M-A.7	1150	2500	4,71	46162,9	21234,93	452	170856	ok!
1M-A.8	1150	1350	2,14	20977,88	17870,05	452	170856	ok!
1M-B.1	4000	4200	9,95	97501,18	92858,27	452	170856	ok!
1M-B.2	4000	4200	10,95	107352,1	102240,1	452	170856	ok!
1M-B.3	4000	4200	11,01	107852,9	102717,1	452	170856	ok!
1M-B.4	4000	4200	9,93	97299,3	92666	452	170856	ok!
1M-C.1	4000	7200	18,55	181821,4	101011,9	452	170856	ok!
1M-C.2	800	1000	2,15	21033,74	16826,99	452	170856	ok!
1M-C.3	800	1000	2,15	21021,98	16817,58	452	170856	ok!
1M-C.4	4000	7200	18,61	182403,5	101335,3	452	170856	ok!
1M-D.1	1150	1350	6,20	60784,5	51779,39	452	170856	ok!
1M-D.2	1150	2500	4,69	45932,6	21129	452	170856	ok!
1M-D.3	1150	2500	4,65	45571,96	20963,1	452	170856	ok!
1M-D.4	1150	2150	3,86	37829,96	20234,63	452	170856	ok!
1M-D.5	1150	2150	3,86	37830,94	20235,15	452	170856	ok!
1M-D.6	1150	2500	4,69	45951,22	21137,56	452	170856	ok!
1M-D.7	1150	2500	4,69	45924,76	21125,39	452	170856	ok!
1M-D.8	1150	1350	2,12	20781,88	17703,08	452	170856	ok!

1M-1.1	4000	4200	11,23	110061,8	104820,8	452	170856	ok!
1M-1.2	4000	4200	11,27	110405,8	105148,4	452	170856	ok!
1M-2.1	3000	3200	5,64	55304,34	51847,82	452	170856	ok!
1M-2.2	3000	3200	5,61	54945,66	51511,56	452	170856	ok!
1M-2.3	3000	3200	5,69	55723,78	52241,04	452	170856	ok!
1M-3.1	3000	3200	5,40	52957,24	49647,41	452	170856	ok!
1M-3.2	4000	4200	7,67	75211,08	71629,6	452	170856	ok!
1M-3.3	3000	3200	5,42	53163,04	49840,35	452	170856	ok!
1M-4.1	3000	3200	5,58	54686,94	51269,01	452	170856	ok!
1M-4.2	3000	3200	8,90	87244,5	81791,72	452	170856	ok!
1M-4.3	3000	3200	8,42	82493,46	77337,62	452	170856	ok!
1M-5.1	3000	3200	5,59	54825,12	51398,55	452	170856	ok!
1M-5.2	3000	3200	8,69	85208,06	79882,56	452	170856	ok!
1M-5.3	1500	1700	2,70	26457,06	23344,46	452	170856	ok!
1M-5.4	1500	1700	2,21	21636,44	19090,98	452	170856	ok!
1M-6.1	3000	3200	5,65	55355,3	51895,59	452	170856	ok!
1M-6.2	4000	4200	7,84	76867,28	73206,93	452	170856	ok!
1M-6.3	3000	3200	5,50	53927,44	50556,98	452	170856	ok!
1M-7.1	3000	3200	5,78	56596,96	53059,65	452	170856	ok!
1M-7.2	3000	3200	5,79	56742	53195,63	452	170856	ok!
1M-7.3	3000	3200	5,47	53618,74	50267,57	452	170856	ok!
1M-8.1	4000	4200	11,52	112894	107518,1	452	170856	ok!
1M-8.2	4000	4200	11,57	113367,4	107968,9	452	170856	ok!

Table 36 Design of the wall in a direction perpendicular to its plane

Wall	N columns	Mu (tm)	Mu (kNm)	Pu (t)	Pu/N (kN)	Ast (mm <sup>2</sup> )	b (mm)	h (mm)	a (mm)	Mn (kNm)	Ø×Mn (kNm)	$\phi M_n \geq M_u$	Ø×Pn (kN)	$\phi P_n \geq P_u$
1M-A.1	2	0,026	0,26	4,35	21,32	314,00	150,00	150,00	99,37	15,60	10,92	ok!	316,75	ok!
1M-A.2	3	0,012	0,12	9,07	29,61	314,00	150,00	150,00	99,37	15,60	10,92	ok!	316,75	ok!
1M-A.3	3	0,012	0,11	9,07	29,64	314,00	150,00	150,00	99,37	15,60	10,92	ok!	316,75	ok!
1M-A.4	3	0,015	0,15	8,49	27,74	314,00	150,00	150,00	99,37	15,60	10,92	ok!	316,75	ok!
1M-A.5	3	0,015	0,15	8,73	28,51	314,00	150,00	150,00	99,37	15,60	10,92	ok!	316,75	ok!
1M-A.6	3	0,017	0,17	9,29	30,34	314,00	150,00	150,00	99,37	15,60	10,92	ok!	316,75	ok!
1M-A.7	3	0,012	0,11	9,00	29,39	314,00	150,00	150,00	99,37	15,60	10,92	ok!	316,75	ok!
1M-A.8	2	0,021	0,20	4,36	21,38	314,00	150,00	150,00	99,37	15,60	10,92	ok!	316,75	ok!
1M-B.1	2	0,009	0,09	26,83	131,46	314,00	150,00	150,00	99,37	15,60	10,92	ok!	316,75	ok!
1M-B.2	2	0,010	0,10	27,26	133,58	314,00	150,00	150,00	99,37	15,60	10,92	ok!	316,75	ok!
1M-B.3	2	0,015	0,14	27,45	134,50	314,00	150,00	150,00	99,37	15,60	10,92	ok!	316,75	ok!
1M-B.4	2	0,110	1,07	25,19	123,42	314,00	150,00	150,00	99,37	15,60	10,92	ok!	316,75	ok!
1M-C.1	3	0,003	0,03	33,65	109,92	314,00	150,00	150,00	99,37	15,60	10,92	ok!	316,75	ok!
1M-C.2	2	0,001	0,01	3,93	19,24	314,00	150,00	150,00	99,37	15,60	10,92	ok!	316,75	ok!
1M-C.3	2	0,001	0,01	4,11	20,14	314,00	150,00	150,00	99,37	15,60	10,92	ok!	316,75	ok!
1M-C.4	3	0,001	0,01	31,53	103,00	314,00	150,00	150,00	99,37	15,60	10,92	ok!	316,75	ok!
1M-D.1	2	0,061	0,60	11,15	54,65	314,00	150,00	150,00	99,37	15,60	10,92	ok!	316,75	ok!
1M-D.2	3	0,061	0,60	10,68	34,90	314,00	150,00	150,00	99,37	15,60	10,92	ok!	316,75	ok!
1M-D.3	3	0,059	0,58	10,56	34,51	314,00	150,00	150,00	99,37	15,60	10,92	ok!	316,75	ok!
1M-D.4	3	0,055	0,54	9,18	29,99	314,00	150,00	150,00	99,37	15,60	10,92	ok!	316,75	ok!
1M-D.5	3	0,045	0,44	7,15	23,36	314,00	150,00	150,00	99,37	15,60	10,92	ok!	316,75	ok!
1M-D.6	3	0,060	0,58	10,52	34,37	314,00	150,00	150,00	99,37	15,60	10,92	ok!	316,75	ok!
1M-D.7	3	0,061	0,60	12,00	39,20	314,00	150,00	150,00	99,37	15,60	10,92	ok!	316,75	ok!
1M-D.8	2	0,049	0,48	6,25	30,62	314,00	150,00	150,00	99,37	15,60	10,92	ok!	316,75	ok!

1M-1.1	3	0,066	0,65	22,33	72,94	314,00	150,00	150,00	99,37	15,60	10,92	ok!	316,75	ok!
1M-1.2	3	0,070	0,69	22,19	72,48	314,00	150,00	150,00	99,37	15,60	10,92	ok!	316,75	ok!
1M-2.1	2	0,000	0,00	15,36	75,25	314,00	150,00	150,00	99,37	15,60	10,92	ok!	316,75	ok!
1M-2.2	2	0,080	0,78	18,67	91,46	314,00	150,00	150,00	99,37	15,60	10,92	ok!	316,75	ok!
1M-2.3	2	0,000	0,00	13,78	67,50	314,00	150,00	150,00	99,37	15,60	10,92	ok!	316,75	ok!
1M-3.1	2	0,006	0,05	14,99	73,47	314,00	150,00	150,00	99,37	15,60	10,92	ok!	316,75	ok!
1M-3.2	2	0,267	2,62	29,92	146,62	314,00	150,00	150,00	99,37	15,60	10,92	ok!	316,75	ok!
1M-3.3	2	0,021	0,21	13,82	67,71	314,00	150,00	150,00	99,37	15,60	10,92	ok!	316,75	ok!
1M-4.1	2	0,028	0,28	14,97	73,35	314,00	150,00	150,00	99,37	15,60	10,92	ok!	316,75	ok!
1M-4.2	2	0,011	0,11	23,78	116,52	314,00	150,00	150,00	99,37	15,60	10,92	ok!	316,75	ok!
1M-4.3	2	0,023	0,23	20,31	99,54	314,00	150,00	150,00	99,37	15,60	10,92	ok!	316,75	ok!
1M-5.1	2	0,028	0,28	15,50	75,93	314,00	150,00	150,00	99,37	15,60	10,92	ok!	316,75	ok!
1M-5.2	2	0,010	0,10	23,72	116,25	314,00	150,00	150,00	99,37	15,60	10,92	ok!	316,75	ok!
1M-5.3	2	0,020	0,20	9,21	45,15	314,00	150,00	150,00	99,37	15,60	10,92	ok!	316,75	ok!
1M-5.4	2	0,012	0,12	5,46	26,77	314,00	150,00	150,00	99,37	15,60	10,92	ok!	316,75	ok!
1M-6.1	2	0,004	0,04	14,57	71,41	314,00	150,00	150,00	99,37	15,60	10,92	ok!	316,75	ok!
1M-6.2	2	0,373	3,65	32,44	158,93	314,00	150,00	150,00	99,37	15,60	10,92	ok!	316,75	ok!
1M-6.3	2	0,016	0,16	13,82	67,71	314,00	150,00	150,00	99,37	15,60	10,92	ok!	316,75	ok!
1M-7.1	2	0,004	0,04	15,22	74,55	314,00	150,00	150,00	99,37	15,60	10,92	ok!	316,75	ok!
1M-7.2	2	0,002	0,02	20,38	99,87	314,00	150,00	150,00	99,37	15,60	10,92	ok!	316,75	ok!
1M-7.3	2	0,000	0,00	15,33	75,10	314,00	150,00	150,00	99,37	15,60	10,92	ok!	316,75	ok!
1M-8.1	3	0,197	1,93	28,50	93,08	314,00	150,00	150,00	99,37	15,60	10,92	ok!	316,75	ok!
1M-8.2	3	0,205	2,01	29,25	95,54	314,00	150,00	150,00	99,37	15,60	10,92	ok!	316,75	ok!

# Appendix C Detailed drawings of the designed building

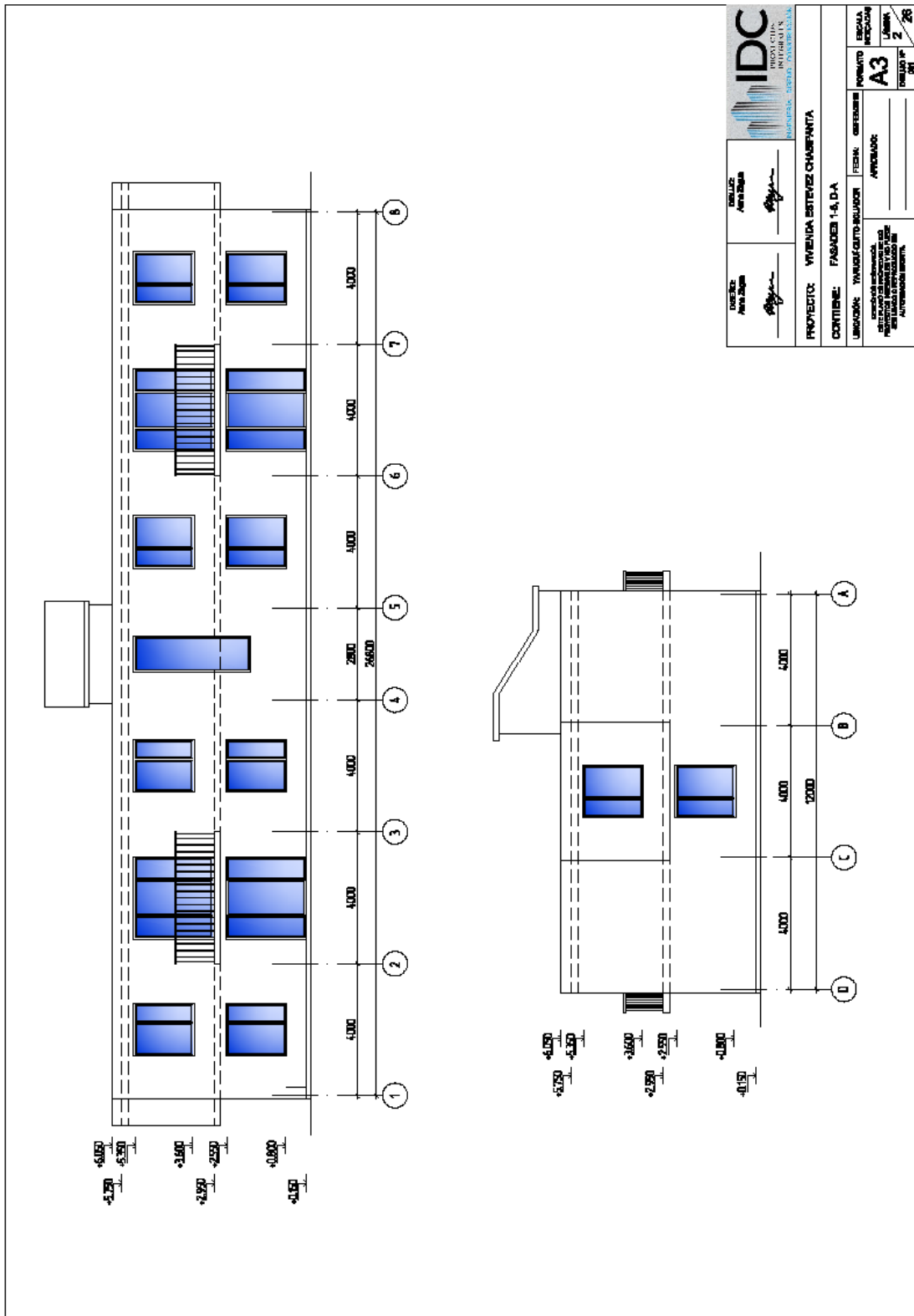
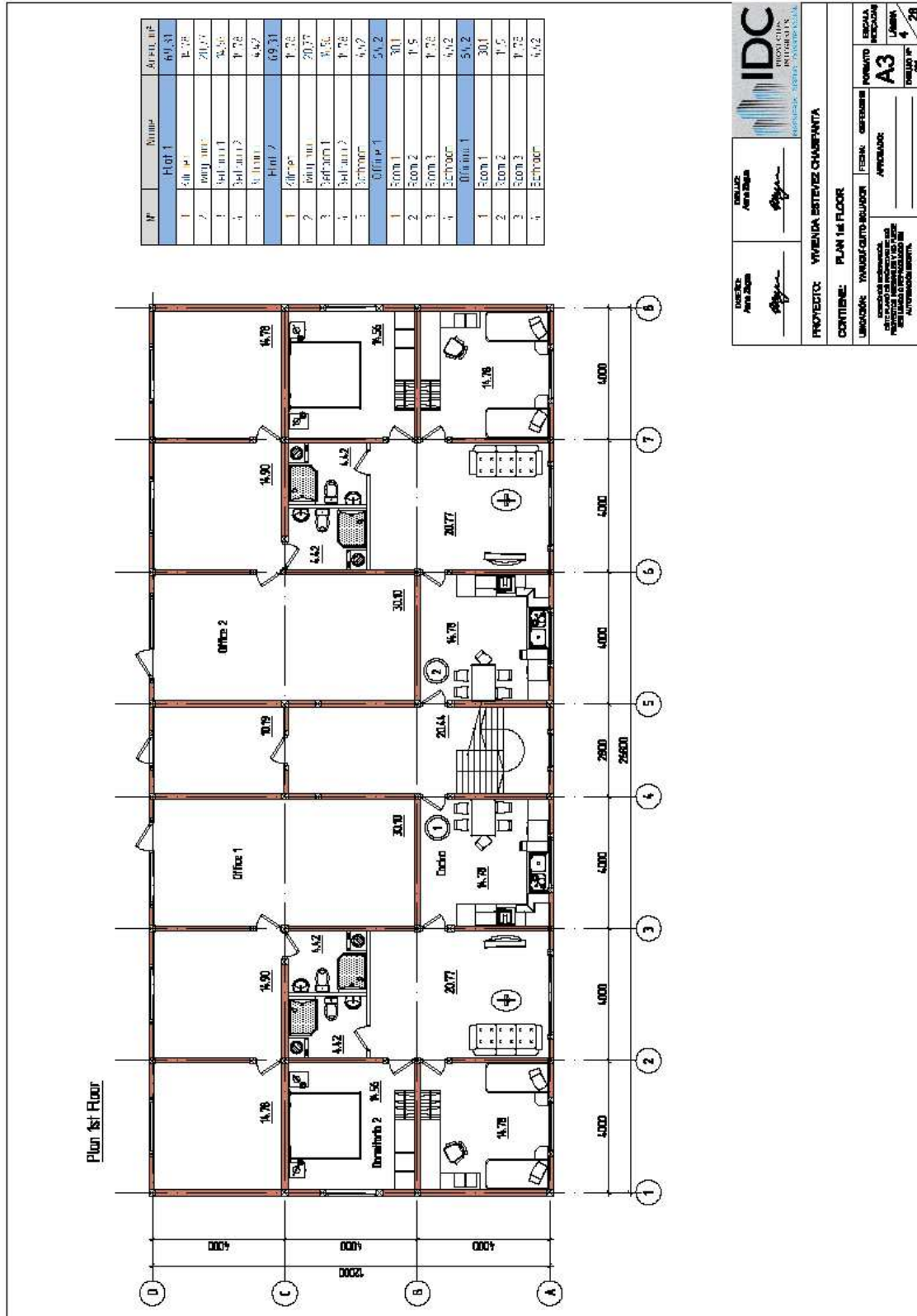


Figure 69 Facades 1-8, D-A





**IDC**  
INSTITUTO DE INVESTIGACIONES Y DESARROLLO TECNOLÓGICO

**PROYECTO: VIVIENDA ESTEREO CHABCANITA**

**CONTIENE: PLAN 1st FLOOR**

**UNIDAD DE INVESTIGACIÓN: INGENIERÍA CIVIL**

**FECHA: 08/08/2018**

**FORMATO: A3**

**ESCALA: 1/50**

**LÁMINA: 4/28**

**PROYECTISTA: [Signature]**

**APROBADO: [Signature]**

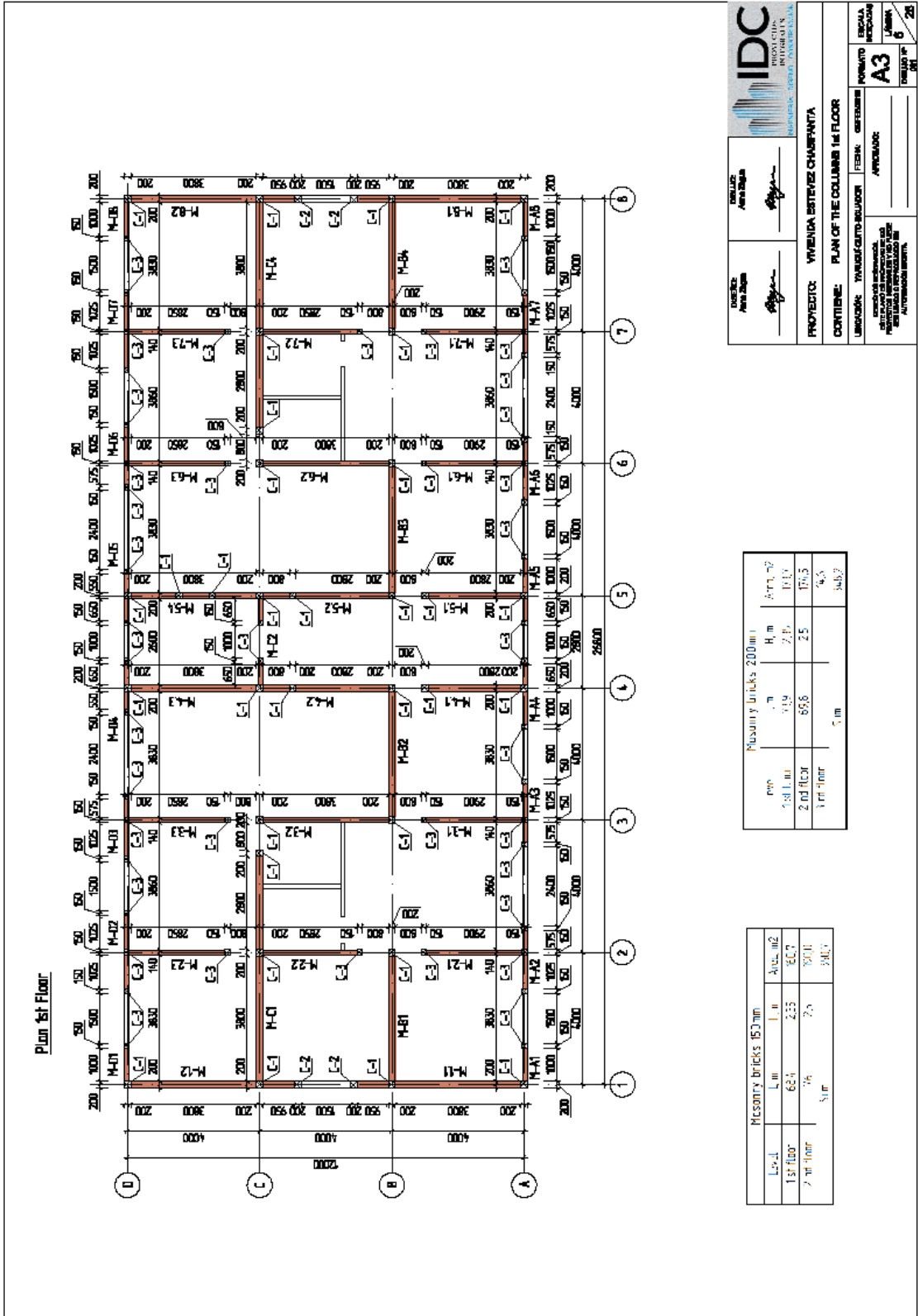
**REVISADO: [Signature]**

**ELABORADO: [Signature]**

Figure 71 Plan 1st floor



Figure 72 Plan 2nd floor



**IDC**  
INGENIERIA DISEÑO CONSTRUCCION  
INGENIERIA DISEÑO CONSTRUCCION

INGENIERO: *[Signature]*  
AUTORIZADO: *[Signature]*

PROYECTO: VIVIENDA ESTREVEZ CHARPANTA  
CONTIENE: PLAN OF THE COLUMNS 1st FLOOR

UNIVERSIDAD: UNIVERSIDAD BOLIVARIANA  
FACULTAD: FACULTAD DE INGENIERIA  
CARRERA: INGENIERIA CIVIL  
AUTORIZACION: AUTORIZACION N° 026

FORMATO: A3  
Escala: 1/50

Masonry Bricks 200mm			
esp	L m	H m	Area m <sup>2</sup>
1st floor	65.4	2.35	150.7
2nd floor	65.8	2.5	164.5
3rd floor	5 m	2.5	12.5

Masonry Bricks 150mm			
L m	L m	Area m <sup>2</sup>	
1st floor	65.4	2.35	150.7
2nd floor	65.8	2.5	164.5
3rd floor	5 m	2.5	12.5

Figure 73 Plan of the columns 1st floor

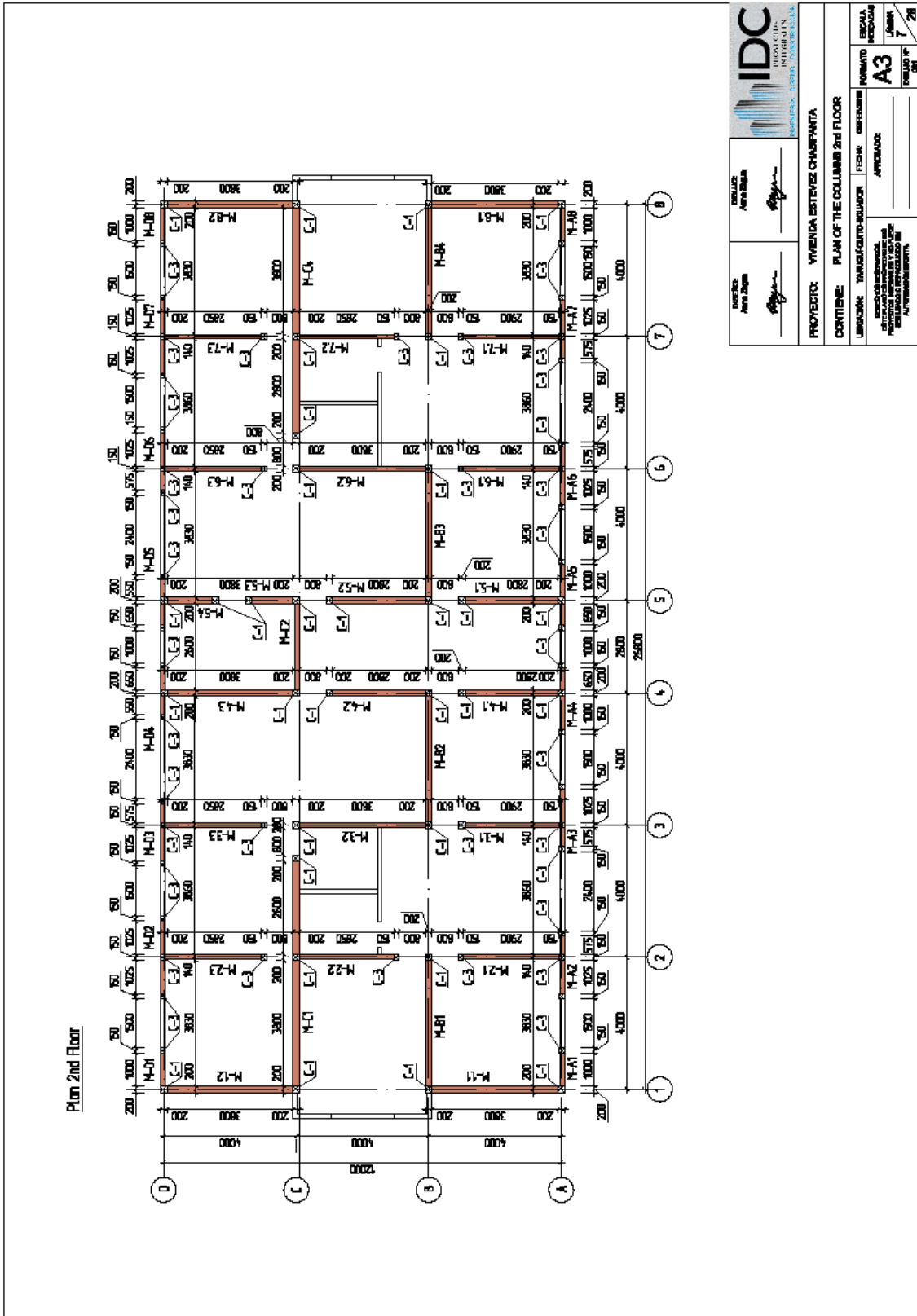


Figure 74 Plan of the columns 2nd floor

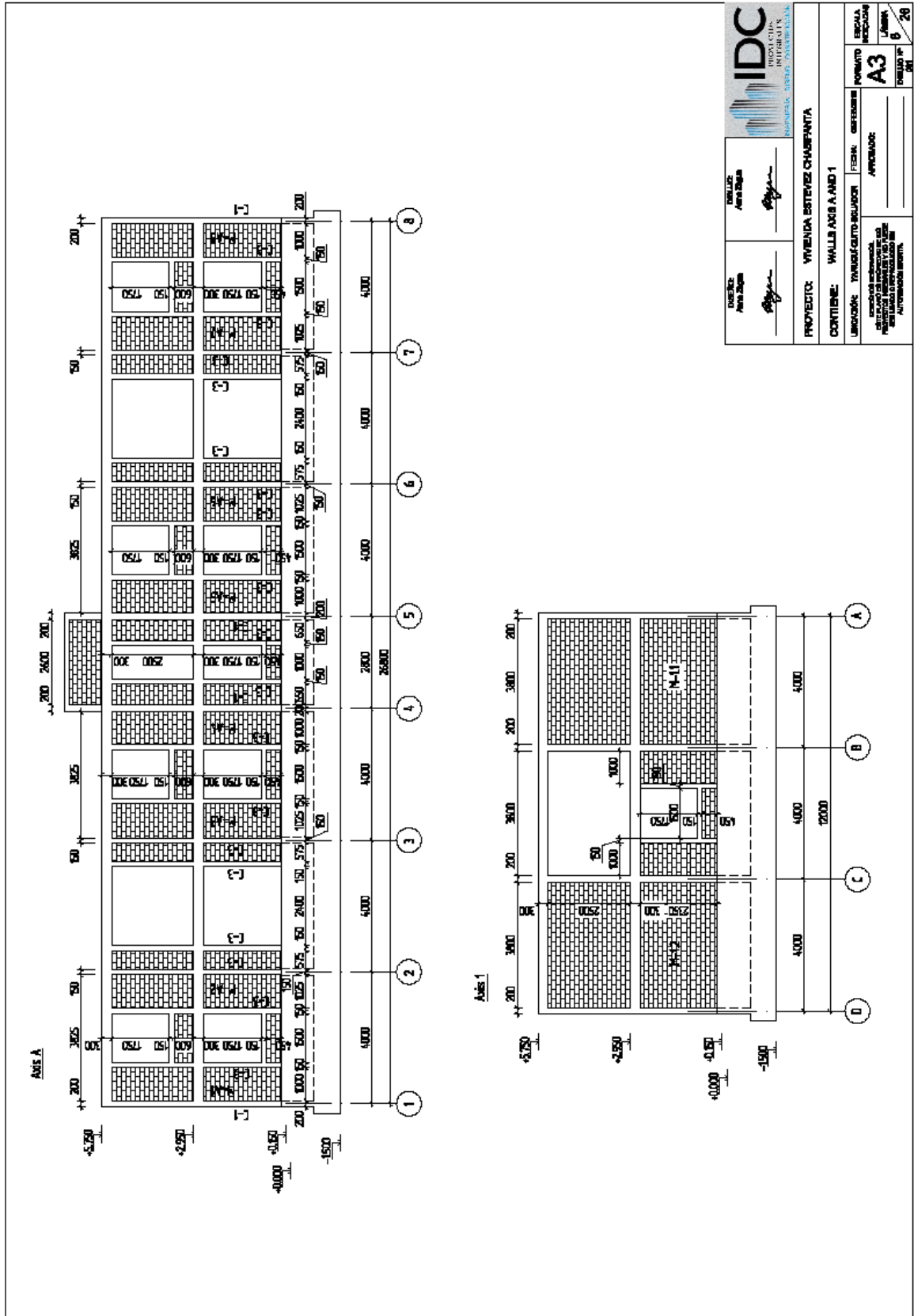


Figure 75 Walls axis A and 1

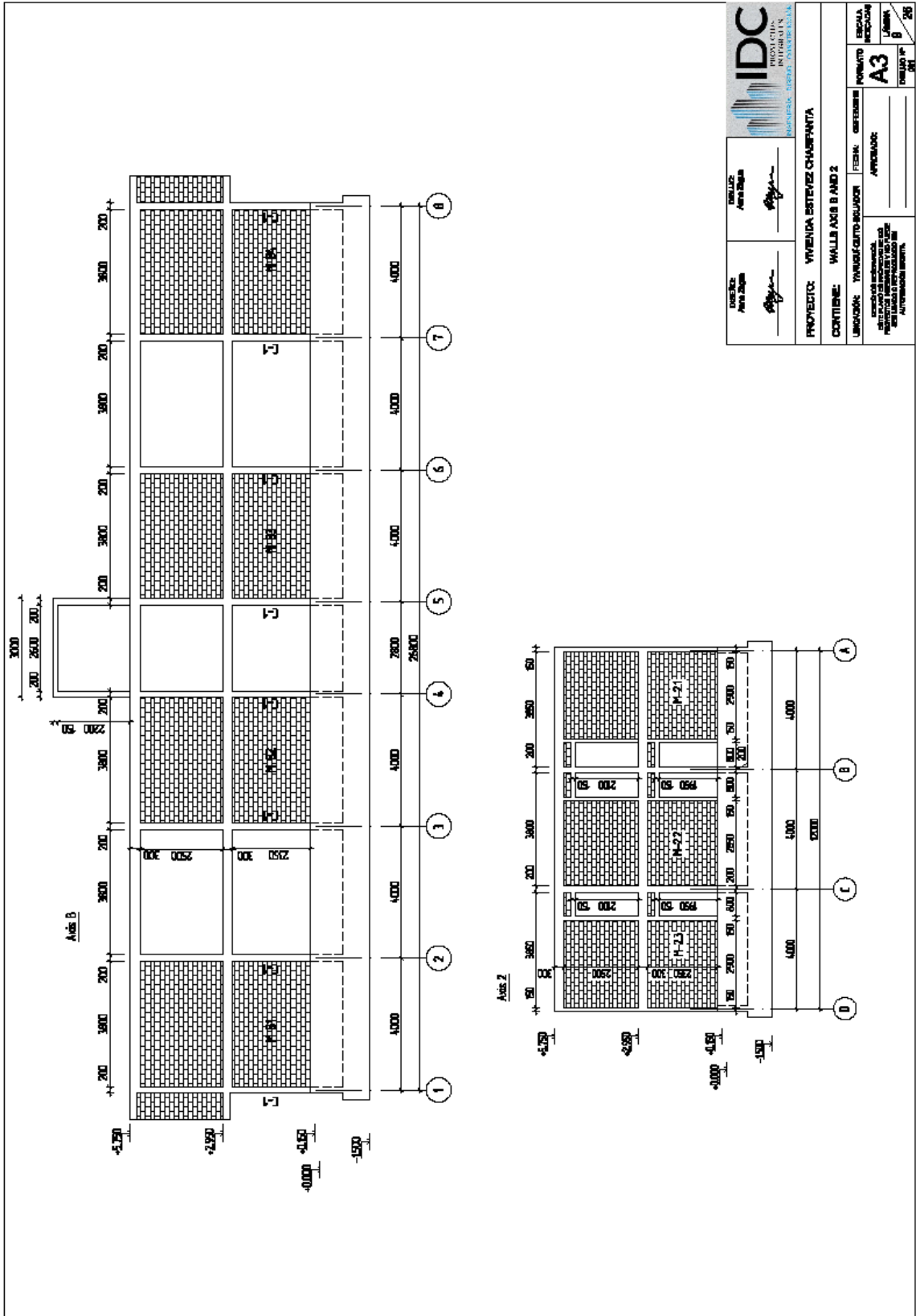


Figure 76 Walls axis B and 2

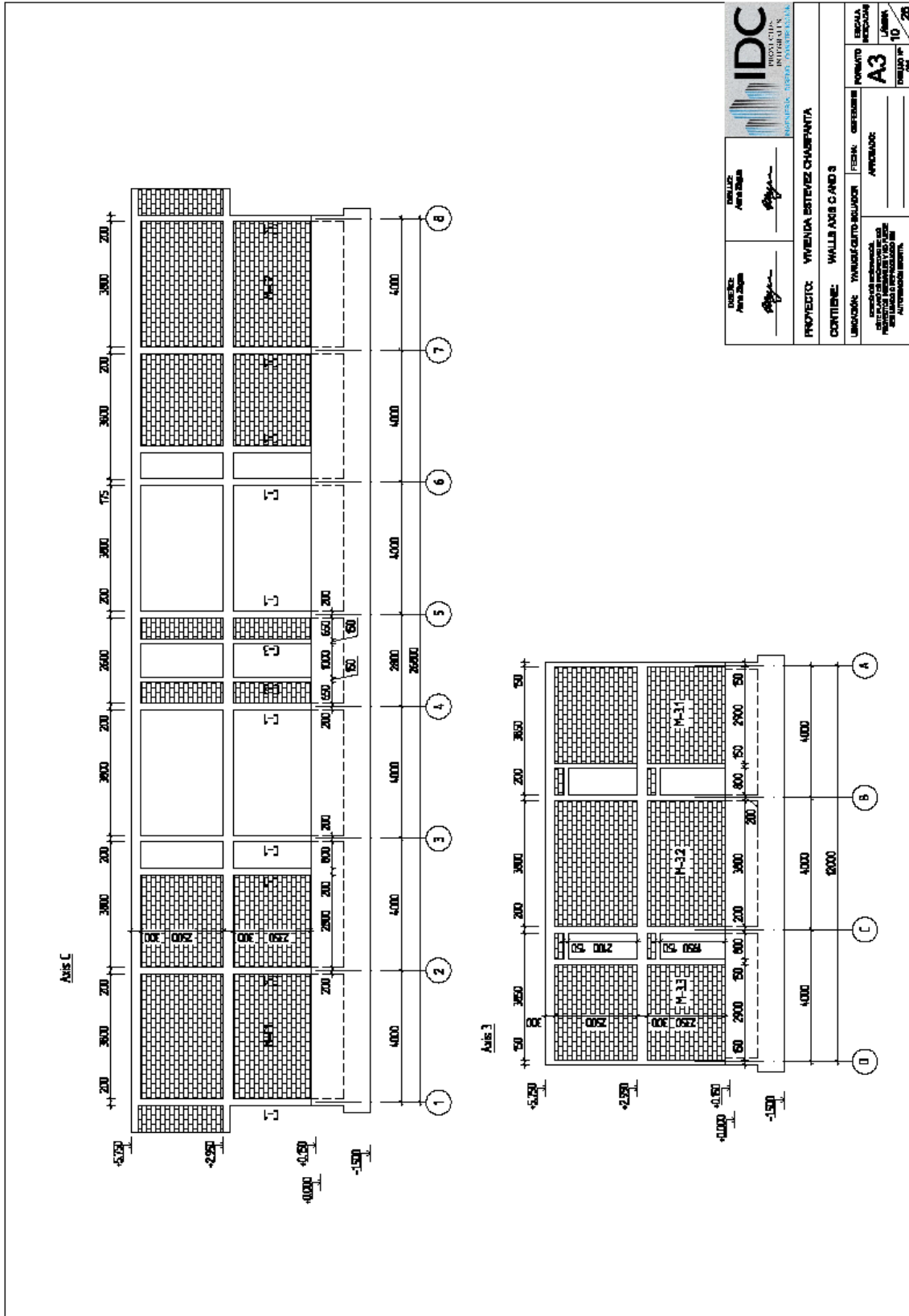


Figure 77 Walls axis C and 3

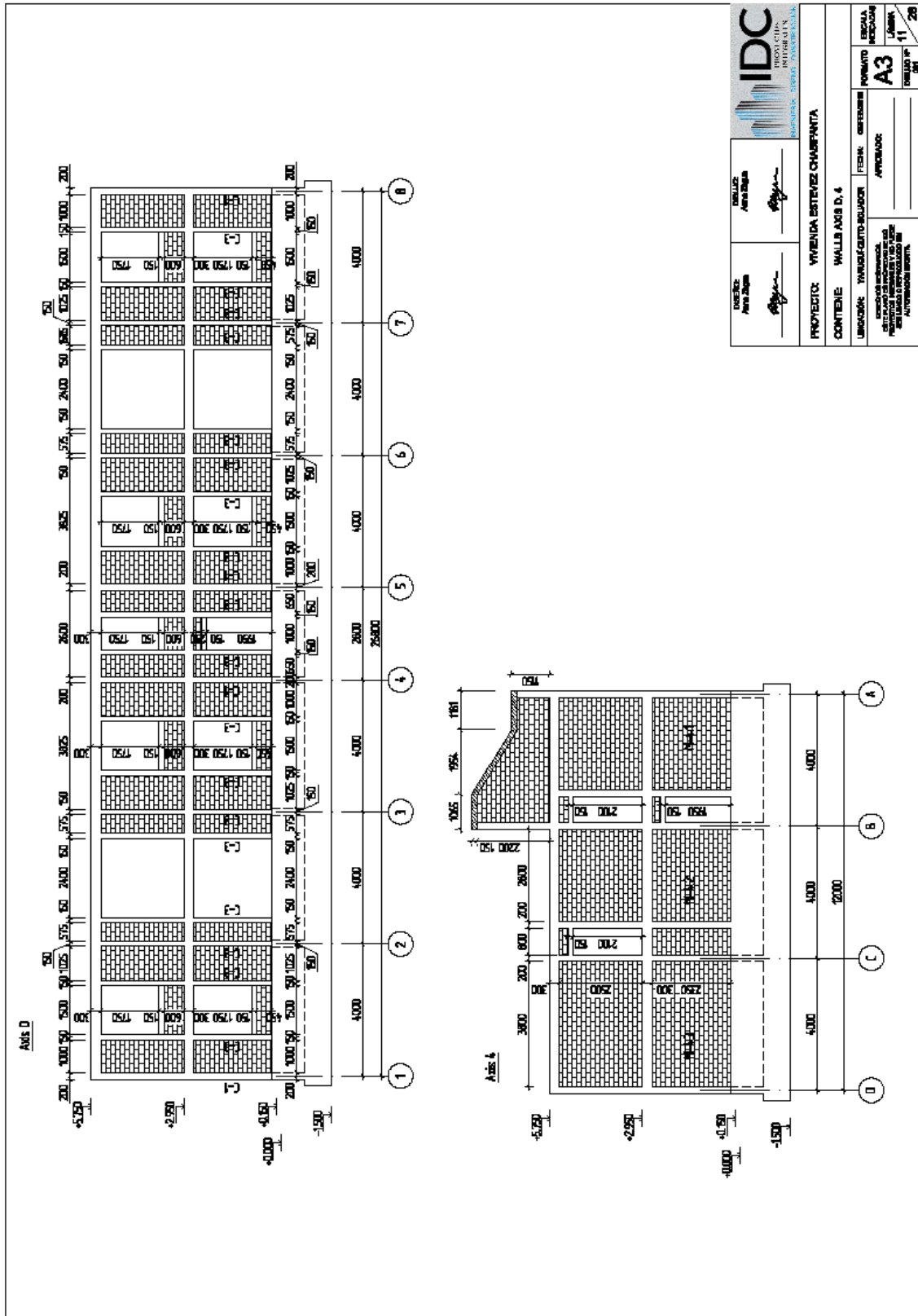


Figure 78 Walls axis D and 4

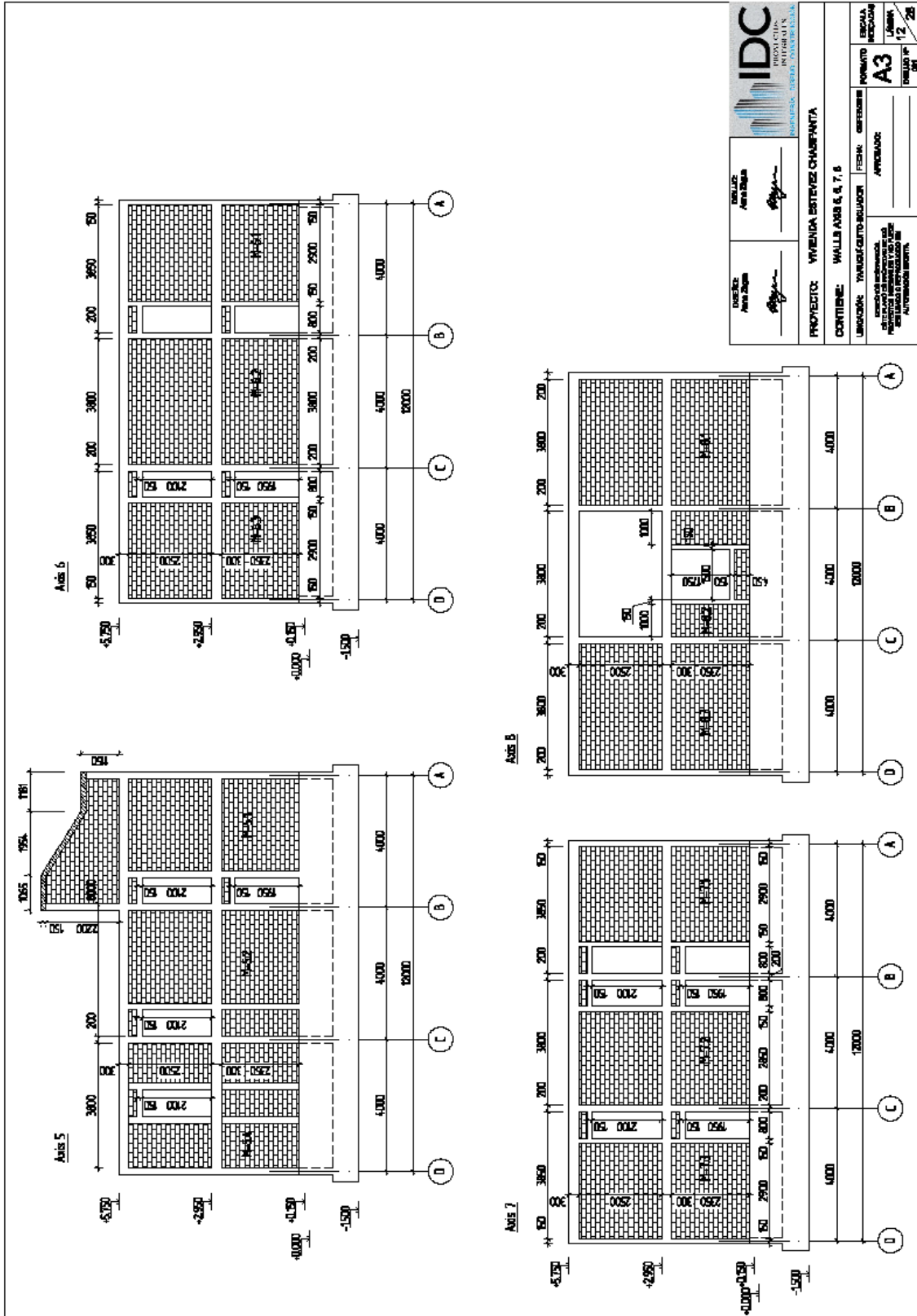


Figure 79 Walls axis 5, 6, 7, 8

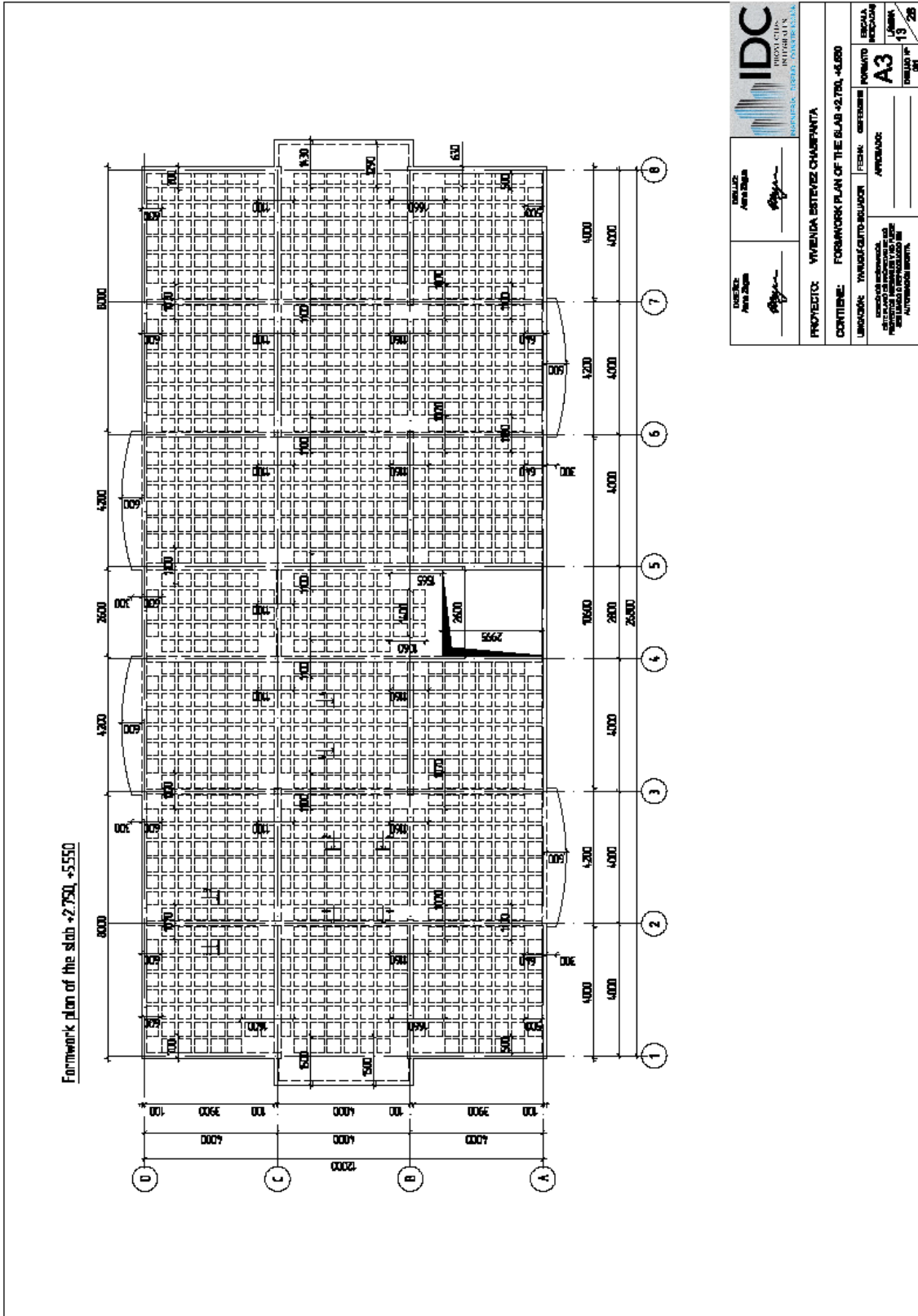


Figure 80 Formwork plan of the slab

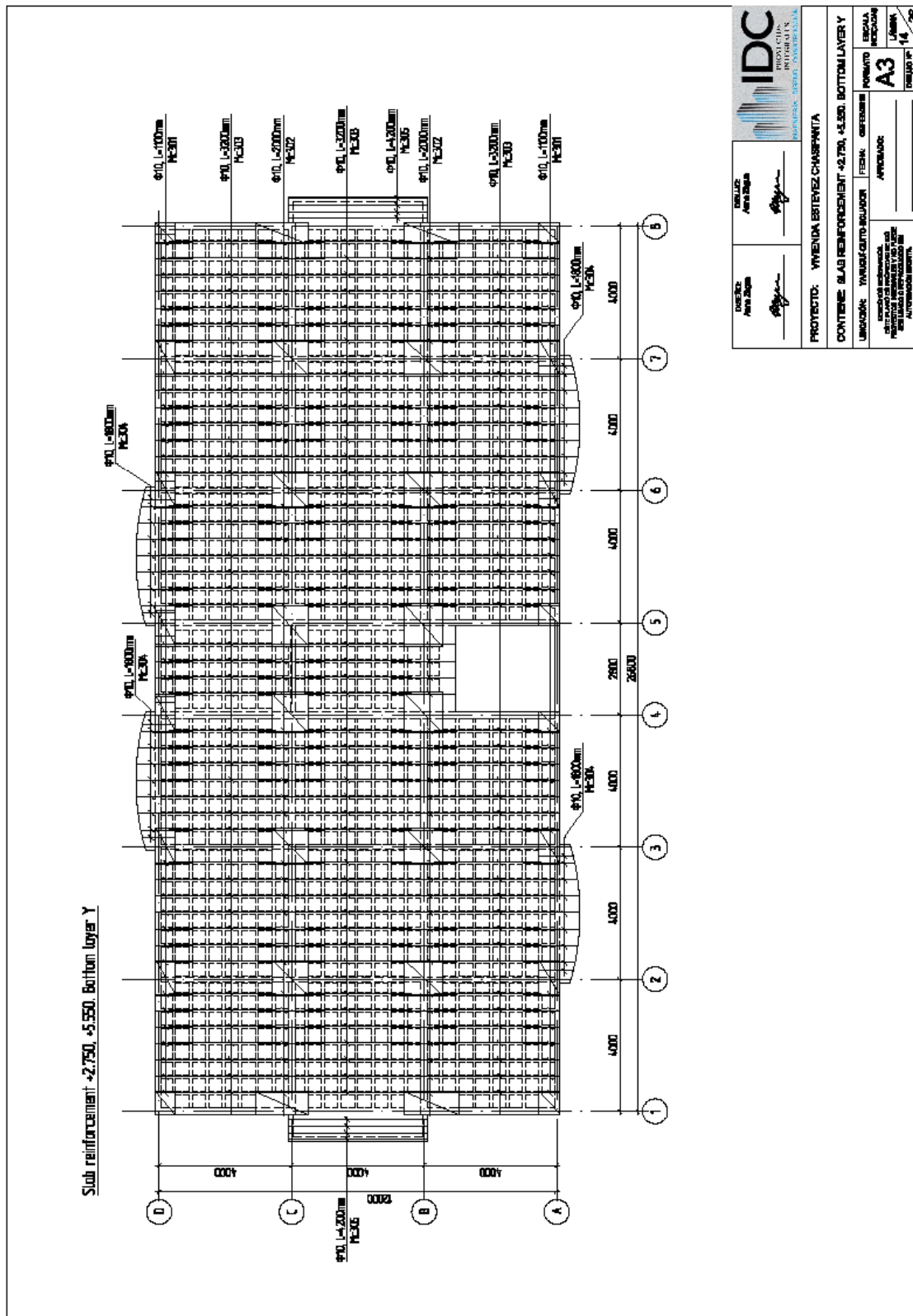


Figure 81 Slab reinforcement. Bottom layer Y

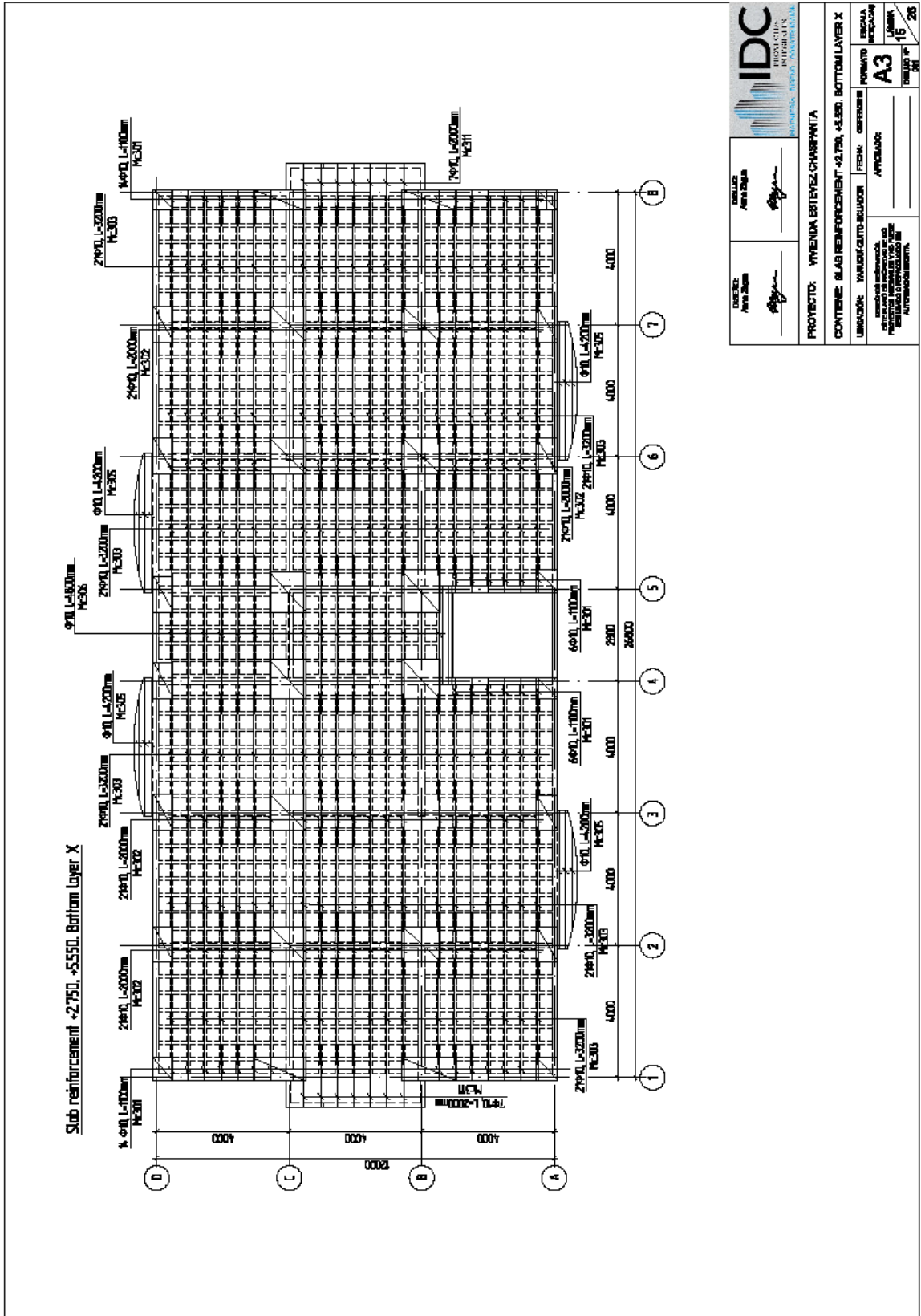
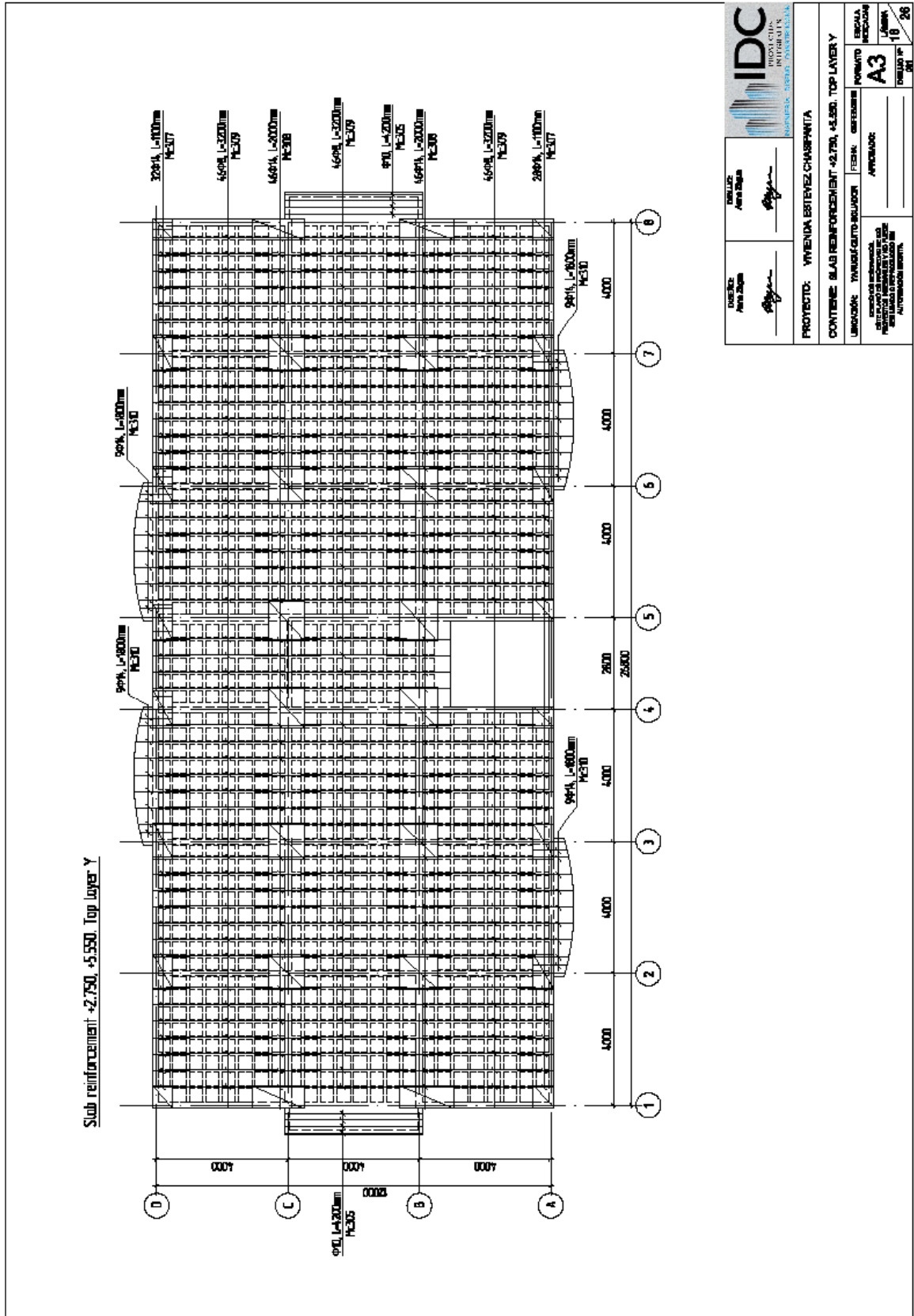


Figure 82 Slab reinforcement. Bottom layer X




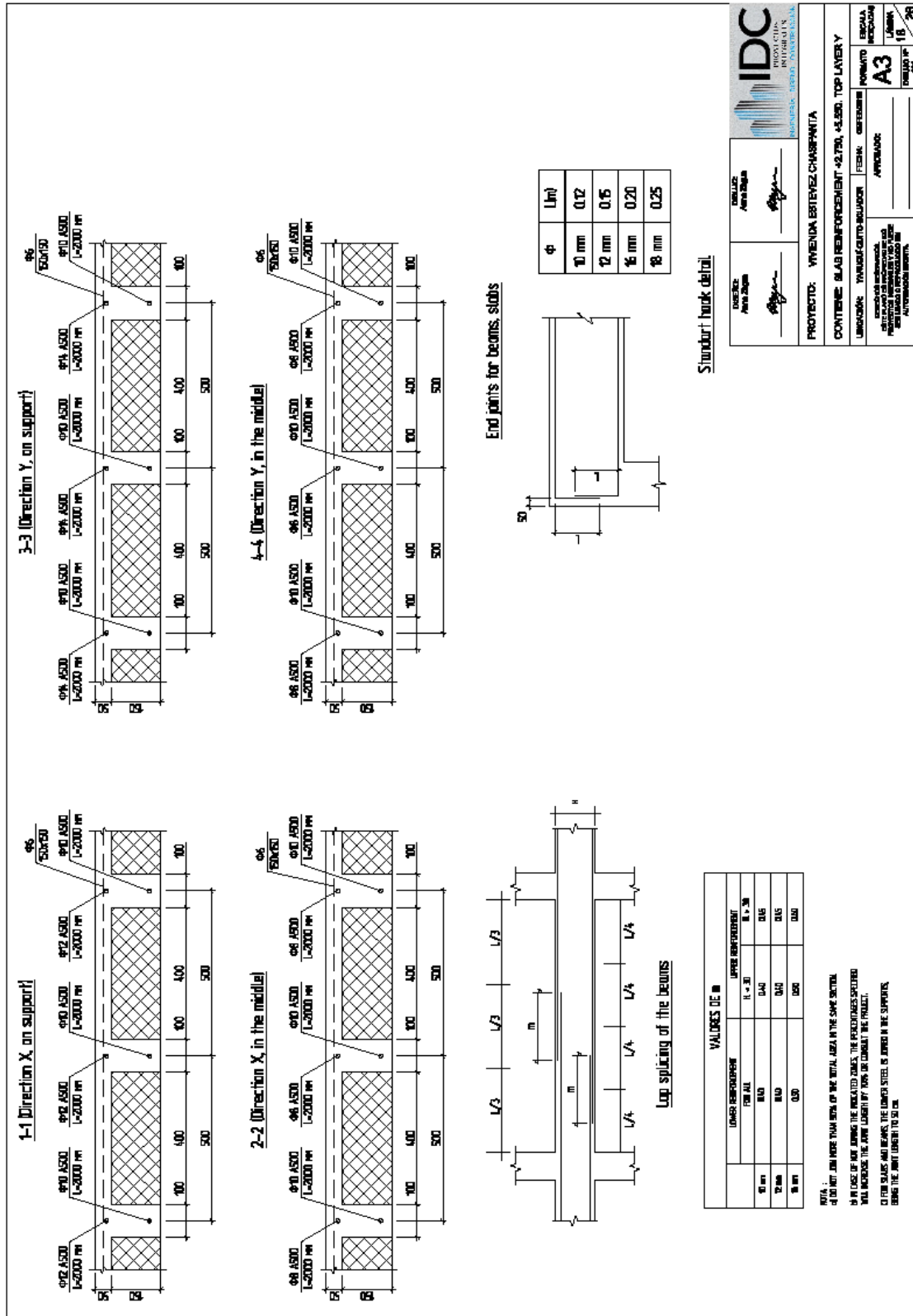
 <b>IDC</b> INSTITUTO DE INVESTIGACIONES CIENTÍFICAS Y TECNOLÓGICAS	
DISEÑO: ANA ZARZA	DIBUJO: ANA ZARZA
PROYECTO: VIVIENDA ESTRETEZ CHARBAMANTA	
CONTENIDO: SLAB REINFORCEMENT +2.750, +5.550, TOP LAYER Y	
LÍNEAS DE IDENTIFICACIÓN: DE ESTE PLANO DE REINFORZAMIENTO DE LOS ELEMENTOS DE CONCRETO ARMADO DE AUTOMÁTICA IDENTIFICACIÓN.	FECHA: 08/08/2018 APROBADO: ANA ZARZA
ESCALA: 1:100	FOLIO: 10 DE 28

Figure 83 Slab reinforcement. Top layer Y





Standard hook detail.

<b>PROYECTO:</b> VIVIENDA BRITANICA CHASAPANTA	<b>PROYECTO:</b> VIVIENDA BRITANICA CHASAPANTA
<b>CONTENIDO:</b> SLAB REINFORCEMENT +2.75M, +3.50, TOP LAYER Y	<b>CONTENIDO:</b> VIVIENDA BRITANICA CHASAPANTA
<b>UNIDAD:</b> INGENIERIA DE ESTRUCTURAS	<b>UNIDAD:</b> INGENIERIA DE ESTRUCTURAS
<b>PROYECTISTA:</b>	<b>PROYECTISTA:</b>
<b>APROBADO:</b>	<b>APROBADO:</b>
<b>ESCALA:</b>	<b>ESCALA:</b>
<b>FECHA:</b>	<b>FECHA:</b>
<b>FORMATO:</b>	<b>FORMATO:</b>
<b>LAJAS:</b>	<b>LAJAS:</b>
<b>HOJA:</b> 18	<b>HOJA:</b> 28

Figure 85 Slab reinforcement. Detailing

Mr	TYPE	φ mm	No	DIMENSIONES (mm)										L. JESOR I mdx I min I max	Vol. Com L. min L. max	Obsery
				a	b	c	d	e	f	g	h	i	j			
307	1	10	50	110	0.0E									160		17
308	1	10	54	200	0.0E								230		17	
305	1	10	36	180	0.0E								200		17	
306	1	10	36	180	0.0E								200		17	
305	1	10	36	180	0.0E								200		17	
REINFORCEMENT STEEL (HUSA x2 750 top layer X)																
Mr	TYPE	φ mm	No	DIMENSIONES (mm)										L. JESOR I mdx I min I max	Vol. Com L. min L. max	Obsery
				a	b	c	d	e	f	g	h	i	j			
307	1	10	50	110	0.0E								160		17	
308	1	10	54	200	0.0E								230		17	
305	1	10	36	180	0.0E								200		17	
306	1	10	36	180	0.0E								200		17	
305	1	10	36	180	0.0E								200		17	
REINFORCEMENT STEEL (HUSA x2 750 top layer Y)																
Mr	TYPE	φ mm	No	DIMENSIONES (mm)										L. JESOR I mdx I min I max	Vol. Com L. min L. max	Obsery
				a	b	c	d	e	f	g	h	i	j			
307	1	10	50	110	0.0E								160		17	
308	1	10	54	200	0.0E								230		17	
305	1	10	36	180	0.0E								200		17	
306	1	10	36	180	0.0E								200		17	
305	1	10	36	180	0.0E								200		17	
REINFORCEMENT STEEL (HUSA x2 750 top layer X)																
Mr	TYPE	φ mm	No	DIMENSIONES (mm)										L. JESOR I mdx I min I max	Vol. Com L. min L. max	Obsery
				a	b	c	d	e	f	g	h	i	j			
312	1	12	10	110	0.0E								160		3	
315	1	12	54	200	0.0E								230		17	
311	1	8	126	320	0.0E								330		37	
305	1	10	36	180	0.0E								200		17	
316	1	12	10	110	0.0E								160		3	
315	1	12	54	200	0.0E								230		17	









**PROYECTO: VIVIENDA ESTRECHAS CHARAPANTA**

**CONTENIDO: SPECIFICATION SLAB 42.730**

UNIDAD: UNIFORMADO	FECHA: 08/08/2018	PROYECTO: A3	ESCALA: 1/20
ELABORADO POR: INGENIERO Y ARQUITECTO	APROBADO POR: INGENIERO Y ARQUITECTO	FECHA DE APROBACION: 10/08/2018	FECHA DE EMISION: 10/08/2018

Figure 86 Slab reinforcement. Materials calculation

Mc	TYPET	φ mm	No	DIMENSIONES (m)						L. Desor			Var. Cum	Observ	
				u	h	l	d	p	r	l	m	l			u
REFORZAMIENTO S. FFI (OUSA -5,750) (m from Layer V)															
MARK 100															
3C1	∟	8	50	1.0	0.6	---	---	---	---	---	---	---	---	---	---
3C2	∟	8	92	2.00	0.6	---	---	---	---	---	---	---	---	---	---
3C3	∟	8	66	2.00	0.6	---	---	---	---	---	---	---	---	---	---
3C4	∟	8	6	1.0	0.6	---	---	---	---	---	---	---	---	---	---
3C5	∟	8	6	4.20	0.6	---	---	---	---	---	---	---	---	---	---
REFORZAMIENTO S. FFI (OUSA -5,750) (m from Layer X)															
OBTENCION S. JUN															
Mc	TYPET	φ	No	u	h	l	d	p	r	l	m	l	u	l	u
MARK 100															
3C1	∟	8	60	1.0	0.6	---	---	---	---	---	---	---	---	---	---
3C2	∟	8	66	2.00	0.6	---	---	---	---	---	---	---	---	---	---
3C3	∟	8	66	3.20	0.6	---	---	---	---	---	---	---	---	---	---
3C4	∟	8	12	4.20	0.6	---	---	---	---	---	---	---	---	---	---
3C5	∟	8	6	4.60	0.6	---	---	---	---	---	---	---	---	---	---
3C6	∟	8	6	2.00	0.6	---	---	---	---	---	---	---	---	---	---
REFORZAMIENTO S. FFI (OUSA -5,750) (m Layer VI)															
OBTENCION S. JUN															
Mc	TYPET	φ	No	u	h	l	d	p	r	l	m	l	u	l	u
MARK 100															
3C7	∟	8	50	1.0	0.6	---	---	---	---	---	---	---	---	---	---
3C8	∟	8	92	2.00	0.6	---	---	---	---	---	---	---	---	---	---
3C9	∟	8	66	3.20	0.6	---	---	---	---	---	---	---	---	---	---
3C0	∟	8	36	1.80	0.6	---	---	---	---	---	---	---	---	---	---
3C5	∟	8	6	4.20	0.6	---	---	---	---	---	---	---	---	---	---
REFORZAMIENTO S. FFI (OUSA -5,750) (m Layer XI)															
OBTENCION S. JUN															
Mc	TYPET	φ	No	u	h	l	d	p	r	l	m	l	u	l	u
MARK 100															
3C1	∟	8	60	1.0	0.6	---	---	---	---	---	---	---	---	---	---
3C2	∟	8	66	2.00	0.6	---	---	---	---	---	---	---	---	---	---
3C3	∟	8	66	3.20	0.6	---	---	---	---	---	---	---	---	---	---
3C4	∟	8	12	4.20	0.6	---	---	---	---	---	---	---	---	---	---
3C5	∟	8	6	4.60	0.6	---	---	---	---	---	---	---	---	---	---
3C6	∟	8	6	2.00	0.6	---	---	---	---	---	---	---	---	---	---

AMOUNT OF CONCRETE		
Quantity (m <sup>2</sup> )	h (m)	V (m <sup>3</sup> )
Floor slab u2	116.00	0.08
Slab -2,750	336.39	0.20
Slab -5,750	336.39	0.20
Slab -8,100	13.62	0.15
Concrete summas		85.3



IDESA  
AREA 2000



IDESA  
AREA 2000



IDC  
INTEGRATED DESIGN CONSULTANTS

PROYECTO: VIVIENDA ESTEREA CHARBANTA

CONTENIDO: ESPECIFICACION SLAB -2,750

LIMBOCAR: TITULO CANTO-BLAVO

FECHA: 08/08/2018

APROBADO:

REVISADO POR: [Signature]

VERIFICADO POR: [Signature]

APROBADO POR: [Signature]

ESCALA: 1/20

FORMATO: A3

FECHA DE EMISION: 20/08/18

RESUMEN DE MATERIALES (SLAB -2,750 -15,750)										
φ	h	l	u	h	l	u	h	l	u	h
3C1	8	60	1.0	0.6	---	---	---	---	---	---
3C2	8	92	2.00	0.6	---	---	---	---	---	---
3C3	8	66	3.20	0.6	---	---	---	---	---	---
3C4	8	12	4.20	0.6	---	---	---	---	---	---
3C5	8	6	4.60	0.6	---	---	---	---	---	---
3C6	8	6	2.00	0.6	---	---	---	---	---	---
3C7	8	50	1.0	0.6	---	---	---	---	---	---
3C8	8	92	2.00	0.6	---	---	---	---	---	---
3C9	8	66	3.20	0.6	---	---	---	---	---	---
3C0	8	36	1.80	0.6	---	---	---	---	---	---
3C5	8	6	4.20	0.6	---	---	---	---	---	---
3C6	8	6	2.00	0.6	---	---	---	---	---	---

Figure 87 Slab reinforcement. Materials calculation

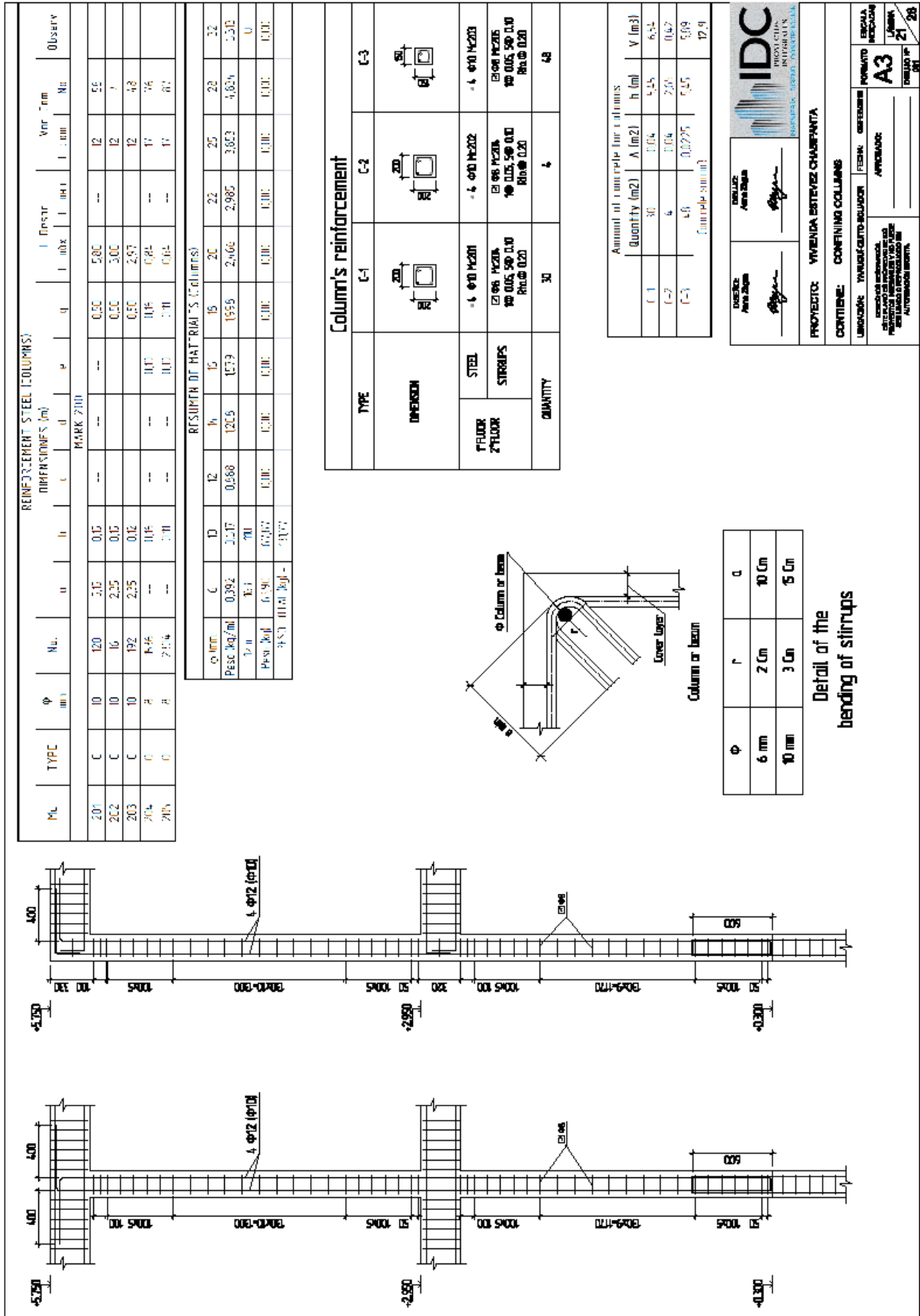


Figure 88 Confining columns

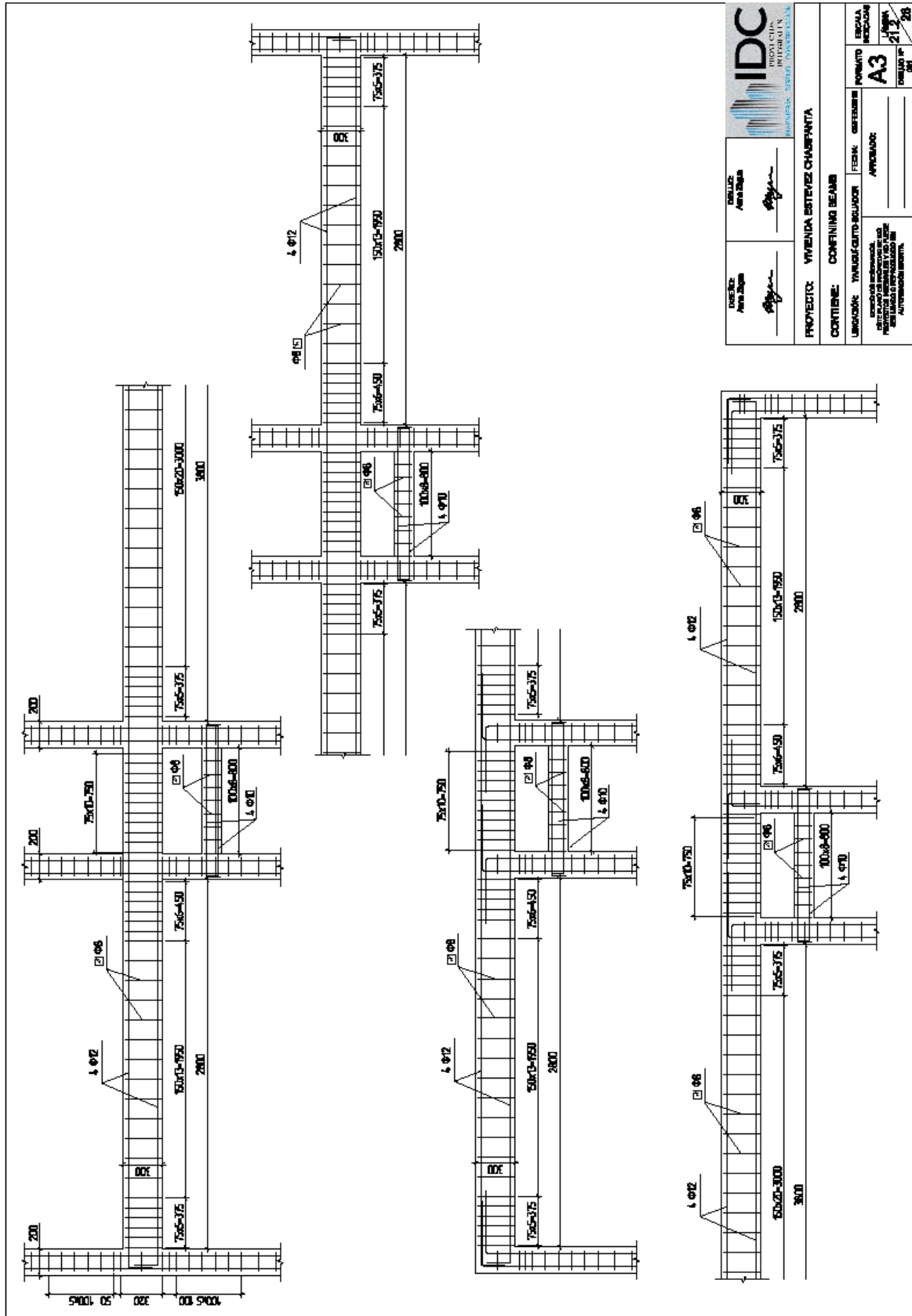


Figure 89 Confining beams

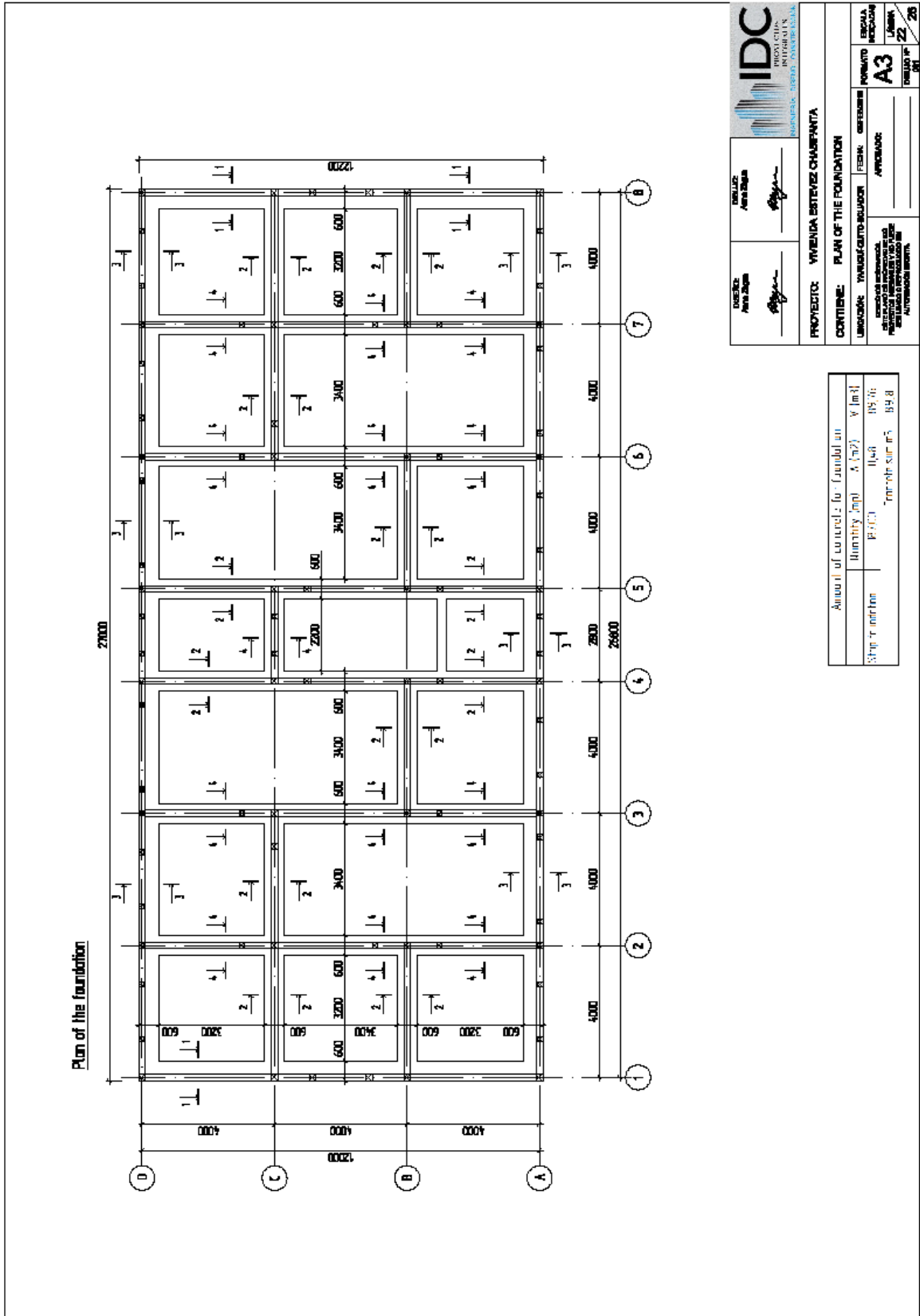
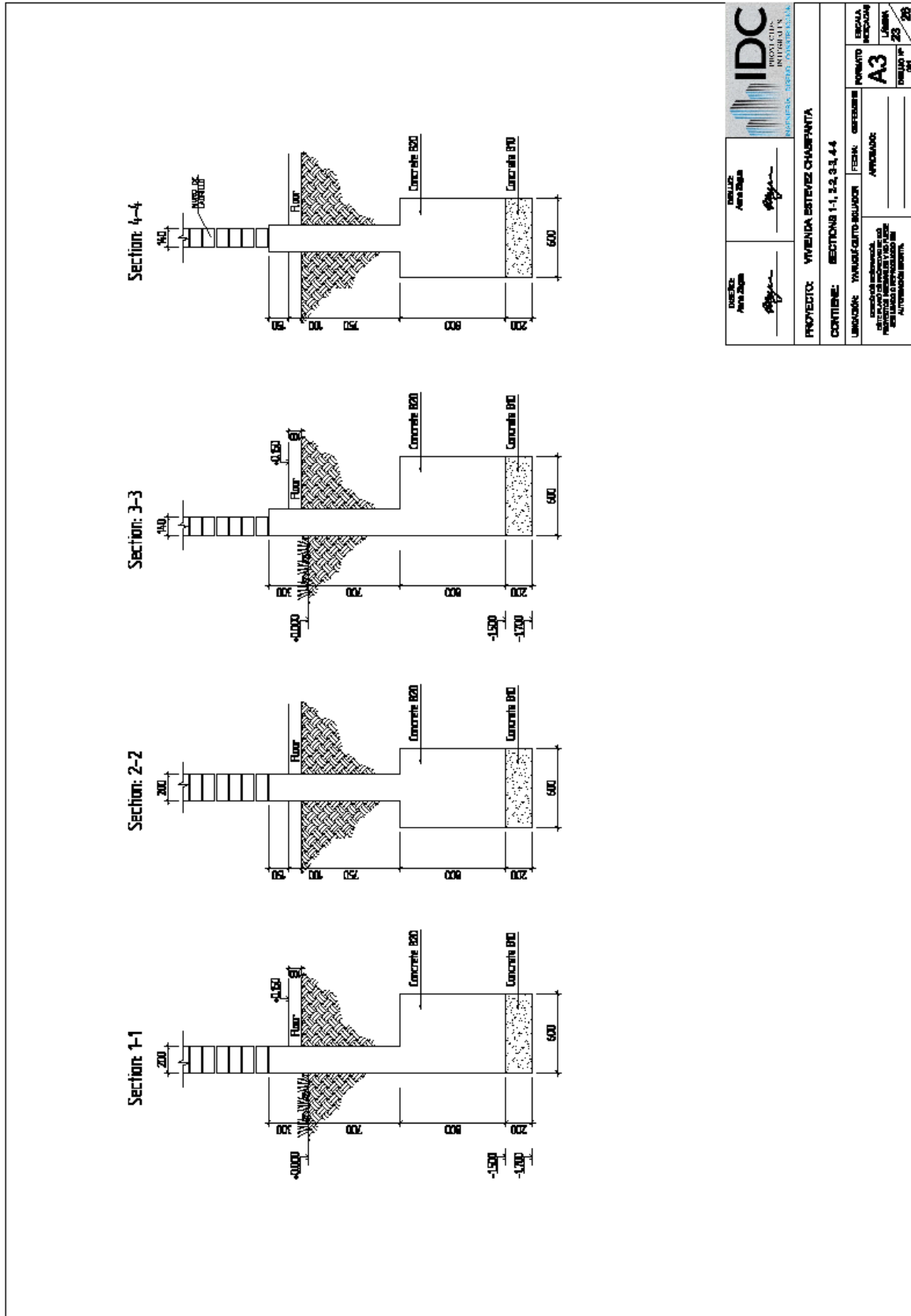


Figure 90 Plan of the foundation










		DISEÑO: ARCHITECTO 	DISEÑO: ARCHITECTO 
PROYECTO: VIVIENDA ESTIPEZE CHAMFANTA			
CONTENIDO: SECCIONES 1-1, 2-2, 3-3, 4-4			
LÍNEA DE TRAZADO DEL DISEÑO <small>INDICAR LA ESCALA DE LA PROYECCIÓN Y EL TIPO DE PROYECCIÓN (PRIMERA O SEGUNDA ANGULACIÓN)</small>	FECHA: 08/08/2011	PLANEO: A3	ESCALA: PROPORCIÓN: LÍNEA: TIPO DE LÍNEA: TIPO DE LÍNEA:
AUTORIZACIÓN DEL CLIENTE:		AUTORIZACIÓN DEL INGENIERO:	
			
FECHA: 08/08/2011		FECHA: 08/08/2011	
AUTORIZACIÓN DEL CLIENTE:		AUTORIZACIÓN DEL INGENIERO:	
			
FECHA: 08/08/2011		FECHA: 08/08/2011	


Figure 91 Foundation. Sections



PLANILLA DE ACERO (Beams for Two Floors)													
Pc	L (m)	φ (mm)	V <sub>d</sub>	DIMENSIONES (m)								Var. Con. No.	Obsery.
				a	b	c	d	e	f	g	h		
400	1	10	17	27.000	0.20	--	--	--	--	27.400	--	17	14
400	1	10	54	12.20	0.20	--	--	--	--	12.60	--	17	62
400	0	8	565	0.25	0.25	--	--	0.75	0.80	--	--	17	15
400	0	8	555	0.25	0.25	--	--	0.75	0.70	--	--	17	65

RESUMEN DE MATERIALES (Beams)												
φ (m)	d	J	V <sub>d</sub>	a	b	c	d	e	f	g	h	L (m)
400	0.252	0.17	0.688	205	1573	955	2,466	2,985	3,653	4,621	28	14
17m	14.1	17										0.310
Peso (kg)	45.27	84.53	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
PESO TOTAL = 145.00												

Beams			
	Quantity	Rate (Z)	Value (Z)
Beams 400	1.28	3.2	4.1
Beams 400	1.28	1.1	1.4
Beams 400	1.28	3.2	4.1
Beams 400	1.28	1.1	1.4
TOTAL BEAMS			11.0







---

**PROYECTO:** VIVIENDA ESTEVEZ CHAUFARITA

**CONTIENE:** DETAILS OF THE FOUNDATION

UNIVERSIDAD AUTÓNOMA DE BUENOS AIRES	FIRM. GERENTE	ESCALA
APROBADO	APROBADO	PROYECTO
A3	A3	A3
DISEÑO Nº 25	DISEÑO Nº 25	DISEÑO Nº 25

Figure 93 Foundation. Specification



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