Seismic Design of Steel Special Moment Frames

Senior Design Project

Kaan Bilgin

ISTANBUL, 2019

MEF UNIVERSITY
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Project Title	: Seismic Design of Steel Special Moment Frames
Student(s) Name	: Kaan Bilgin
Date	: 12/06/2019

I hereby certify that the design project titled ‘Seismic Design of Steel Special Moment Frames’ prepared by Kaan Bilgin has been completed under my supervision.

23/05/2019
Asst. Prof. Onur Şeker

ACADEMIC HONESTY PLEDGE

In keeping with MEF University Student Code of Conduct, I pledge that this work is my own and that I have not received inappropriate assistance in its preparation. I further declare that all resources are explicitly cited.

NAME	DATE	SIGNATURE
Kaan Bilgin	12/06/2019	

ABSTRACT

SEISMIC DESIGN OF STEEL SPECIAL MOMENT FRAMES

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In seismically active areas, earthquake resistant buildings need to dissipate notable energy during an earthquake to reduce the total building damage. This damage reducing providing can only achieve with high ductility, flexibility and low weight. With being lighter than concrete buildings, properly designed steel structures perform well during an earthquake with their strong, highly ductile material behavior and relatively low seismic design forces acting on them which is an after effect of their low structural mass. This Design Project contains the seismic design of a steel office building with special moment frames within four stages; Choosing the Design Parameters, Drift (stiffness) Design, Strong Column Weak Beam Design and last, Strength Design. First Stage is determining structural design parameters such as span lengths, story heights, building location, intended use of the building, soil classification, seismic design parameters and the general boundary conditions for the Analyses. Second stage is chosen as Drift (stiffness) design with the idea that in order to limit lateral deflections of flexible moment frames, larger sections are needed, and larger sections will provide enough strength. Third stage is the Strong Column Weak Beam Design after Drift Design to carry forward the second stage's idea and making an effort to reduce the iteration numbers of design such flexible frames. Fourth and the last stage is Strength Design and the clarification of the final sections with discussion and conclusion.

Keywords: Steel Special Moment Frame, Seismic Design, Structural Engineering, Reduced Beam Section Design

ÖZET

ÇELİK MOMENT ÇERÇEVELERİN SİSMİK DİZAYNI

Kaan Bilgin

MEF ÜNİVERSİTESİ
Mühendislik Fakültesi
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Sismik aktif bölgelerde depreme dayanıklı binaların toplam bina hasarını azaltmak için deprem sırasında kayda değer miktarda enerjiyi dağıtması gerekir. Bu hasarı azaltan tedarik ancak yüksek süneklik, esneklik ve düşük ağırlıkla başarılabilinir. Beton binalardan daha hafif olmasıyla beraber uygun şekilde tasarlanmış çelik yapılar, güçlü, yüksek sünekli malzeme davranışları ve düşük yapısal kütlelerinin bir etkisi olan üzerlerine etki eden nispeten düşük sismik tasarım kuvvetleri ile bir deprem sırasında iyi performans gösterir. Bu Tasarım Projesi, özel moment çerçeveli çelik ofis binasının sismik tasarımını içermektedir. Dört aşamadan oluşan bu tasarımın aşamaları sırasıyla şunlardır; Tasarım Parametrelerinin Seçimi, Drift (Rijitlik) Tasarımı, Güçlü Kolon Zayıf Kiriş Tasarımı ve sonuncu olarak, Mukavemet Tasarımı. Birinci Aşama, kiriş açıklıkları, kat yükseklikleri, bina lokasyonu, yapının kullanım amacı, zemin sınıflandırması, sismik tasarım parametreleri ve analizler için genel sınır şartları gibi yapısal tasarım parametrelerini belirlemektir. İkinci aşama, esnek moment çerçevelerinin, yanal ötelenmelerini sınırlamak için daha büyük profil seçimi gerektirdiği ve daha büyük profillerin yeterli mukavemeti sağlayacağı düşüncesiyle Drift (Rijitlik) tasarımı olarak seçilmiştir. Üçüncü aşama, ikinci aşamanın fikrini ileri götürmek ve bu tür esnek çerçevelerin tasarımın sürecindeki yineleme sayılarını azaltmak için Güçlü Kolon Zayıf Kiriş Tasarımı olarak uygulanılmıştır. Dördüncü ve son aşama ise Mukavemet Tasarımı ardından aşamalardaki sonuçların tartışma ve sonuç ile açıklanmasıdır.

Anahtar Kelimeler: Çelik Moment Çerçeve, Sismik Tasarım, Zayıflatılmış Kiriş Enkesitli Kiriş-Kolon Birleşimler

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LIST OF ABBREVIATIONS

Δ	Story Drift
$\Delta F_{NE}^{(x)}$	Additional Force at the Roof, *kN*
a	Horizontal Distance from Face of Column Flange to Start of the RBS Cut, *mm*
A_g	Gross Area, mm^2
b	Length of the RBS Cut, *mm*
b_f	Beam Flange Width, *mm*
BYS	Allowed Building Height Class
c	Centre Depth of the RBS Cut, *mm*
C_t	Approximate Period Coefficient
D	Overstrength Factor
DAM	Direct Analysis Method
d_b	Beam Depth, *mm*
d_c	Column Depth, *mm*
DD-1	Over 50 Years of Probability of Exceeding 2% (Recurrence Period 2475 years) Earthquake Ground Motion Level
DD-2	Over 50 Years of Probability of Exceeding 10% (Recurrence Period 475 years) Earthquake Ground Motion Level
DD-3	Over 50 Years of Probability of Exceeding 50% (Recurrence Period 72 years) Earthquake Ground Motion Level
DD-4	Over 50 Years of Probability of Exceeding 68% (Recurrence Period 43 years) Earthquake Ground Motion Level
EA	Axial Stiffness
$E_d^{(H)}$	Horizontal Seismic Load, *kN*
$E_d^{(Z)}$	Vertical Seismic Load, *kN*
EI	Flexural Stiffness
F_1	1.0 Second Period Local Ground Affection Coefficient

F_{cr}	Average Compressive Stress, *MPa*
$F_{iE}^{(x)}$	Seismic Force Acting on at Story i, *kN*
F_s	Short Period Region Local Ground Affection Coefficient
F_u	Yield Strength of Steel Section, *MPa*
F_y	Ultimate Strength of Steel Section, *MPa*
F_{yb}	Specified Minimum Yield Stress of Beam, *MPa*
F_{yc}	Specified Minimum Yield Stress of Column, *MPa*
g	Ground Acceleration, m/s^2
H_n	Height Over Seismic Base, *m*
I	Building Importance Factor
K	Effective Length Factor
L_r	Notional Roof Live Load, *kN*
LRFD	Load and Resistance Factor Design
M_c	Flexural Design Strength
M_f	Probable Maximum Moment at Centre of the RBS, *kNm*
m_i	Story Mass, *T*
M_{pr}	Maximum Moment at Centre of the RBS, *kNm*
M_r	Required Moment Capacity, *kNm*
m_T	Total Mass of a Structure, *T*
M_v	Additional Moment Due to Shear Amplification from the Location of the Plastic Hinge to the Column Centreline, *kNm*
M_y	Moment Capacity, *kNm*
n	Live Load Contribution Factor
N	Number of Stories
N_D	Notional Dead Load, *kN*
N_i	Notional Load Applied at level i, *kN*

N_L	Notional Live Load, *kN*
N_{Lr}	Notional Roof Live Load, *kN*
P_c	Compressive Strength, *MPa*
P_{cr}	Critical Load, *kN*
P_E	Euler Buckling Load, *kN*
PGA	Peak Ground Acceleration, g
PGV	Peak Ground Velocity, cm/s
P_{max}	Ultimate Load Capacity of an Imperfect Inelastic Column, *kN*
P_r	Required Compressive Strength, *MPa*
P_y	Yield Strength, *MPa*
r	Radius of Gyration, *mm*
R	Response Modification Coefficient
R_y	Tensile Strength Ratio, *unitless*
S_1	1.0 Second Period Map Spectral Acceleration Coefficient, *unitless*
$S_{ae}(T)$	Elastic Horizontal Design Spectral Acceleration, *g*
$S_{aR}(T)$	Reduced Design Spectral Acceleration, g
$S_{aeD}(T)$	Elastic Vertical Design Spectral Acceleration, *g*
SC-WB	Strong Column Weak Beam
S_{D1}	1.0 Second Period Design Spectral Acceleration Coefficient, *unitless*
$S_{de}(T)$	Elastic Horizontal Design Spectral Displacement, *m*
S_{DS}	Short Period Design Spectral Acceleration Coefficient, *unitless*
S_h	Distance from Face of Column to Plastic Hinge Region, *mm*
SMF	Special Moment Frame
S_S	Short Period Map Spectral Acceleration Coefficient, *unitless*
T	Natural Vibration Period, *s*
T_A	Elastic Horizontal Design Acceleration Spectrum Corner Period, *s*

T_B	Elastic Horizontal Design Acceleration Spectrum Corner Period, *s*
t_{bf}	Beam Flange Thickness, *mm*
t_{bw}	Beam Web Thickness, *mm*
t_{cf}	Column Flange Thickness, *mm*
t_{cw}	Column Web Thickness, *mm*
T_L	Elastic Horizontal Design Spectrum to Constant Displacement Region Transition Period, *s*
T_{LD}	Elastic Vertical Design Spectrum's Constant Displacement Region Transition Period, *s*
T_p	Structure's Natural Vibration Period, *s*
T_{pA}	Approximate Period, *s*
V_{rbs}	Shear Force at Centre of the RBS, *kN*
$V_t^{(x)}$	Equivalent Lateral Force, *kN*
W	Wind Load
W_D	Line Distributed Dead Load, *kN/m*
W_G	Self Weight (Dead Load Component)
W_i	Effective Seismic Weight, *T*
W_L	Line Distributed Live Load, *kN/m*
W_Q	Live Load Component
Y_i	Gravity Load Applied at Level i, *kN*
Z	Plastic Section Modulus, mm^3
Z_C	Plastic Section Modulus of Column, mm^3
Z_{rbs}	Plastic Section Modulus of RBS, mm^3
Δ_0	Initial Deflection, *mm*
λ_f	Width to Thickness Ratio
λ_{hd}	Limiting Width to Thickness Ratio
τ_b	Stiffness Reduction Parameter

1. INTRODUCTION

Aim of this project is to make a proper seismic design and from the results of this design, project will lean to highlight the design process steps within a hierarchical, in order to assist further seismic designs and seismic design topics.

1.2. Motivation

In 17 August 1999, Turkey has seen one of the most devastating, literature breaker earthquake in history, the Marmara Earthquake, also known as the Gölcük Earthquake. This name is given because Gölcük was the epicenter of the Earthquake. According to Kandilli Observatory [(1)], 17480 citizens lost their lives and 73342 building have been damaged, afterward economy is struggled for years. Being a Turkish Citizen and a future structural engineer, I felt exigency to doing a study about how to withstand against seismicity and it led me to design a seismic design of steel structure for my Senior design project.

1.3. Impact of Project

As explained in topic 1.2., seismic -active regions, like Turkey, needs to improve their built and will-build structures to withstand earthquake loads to prevent major collapses, catastrophes and economic crisis. I believe that proper designed, highly ductile and flexible steel buildings will replace the limited-flexible reinforced concrete structures with low ductility, at the seismic zones, which will lead to saving lives, cities and economy with ease.

1.4. Project Report Outline

The remaining chapters of this report is organized as follows:

2. Literature Review
3. Problem Definition
4. Analyze and Design Process
5. Results
6. Conclusion

2. LITERATURE REVIEW

In this section, literature and history about all the design steps will briefly explain with the previous nomenclature.

2.1. Seismic Design Philosophy

If there is not enough rigidity in the structure despite the prescribed safety conditions, very large deformations and displacements occur that are inconvenient to the eye and or if irritating vibrations occur, a suitable sizing is cannot be mentioned and despite the general belief that seismic codes are stands for preventing structures damaging, the truth is quite controversy, the codes and design are for the prevent life loses. Economic design solutions can only be achieved by recognizing damage to the structure. Acceptable damage limits should vary gradually according to the earthquake at each targeted level. While ensuring life safety, potential damage should be limited and should be repairable after the earthquake.

The design of the buildings by considering the seismic loads started with a devastating earthquake in the California area (San Francisco Earthquake of 1906) (2) and has come to the present day with various developments. The philosophy of earthquake resistant structure design has been developed based on the assumption that structures with a service life ranging from 50 to 100 years are economically impossible to design in such a way that they remain elastic against the huge horizontal loads that the rare destructive earthquakes have affected the structure. Therefore, during the design of the horizontal load-bearing systems, a certain proportion of the structure weight (depending on the type of structure and the natural vibration period, the earthquake zone and the ground conditions, generally a value between 5% and 15% of the weight of the structure can be expected) is effected as a horizontal load. It is ensured that the non-consumable part of the seismic energy entering the structure is consumed by the plastic deformation capability under the repeated cyclic loads. In other words, the philosophy of seismic design aims at the fact that structural elements can suffer serious damage when exposed to strong ground motions of the structure, but that the foreseen damage does not lead to migration or loss of life (FEMA355D, 2000). (3) For this reason, the determination of the mechanism states and the failure performance of the structures in case of earthquakes are very critical in terms of building safety. Today, the performance objectives underlined for the structures designed using the philosophy of seismic design can be summarized as follows:

(a) No damage is observed in structural (column, beam etc.) and non-structural elements (fill walls, etc.) during common weak ground movements;

(b) In the case of medium-sized earthquakes, limited damage observation on non-structural elements, and no damage observation on structural elements;

(c) Damage to the structural and non-structural elements may occur during strong ground motions, but overall collapse and loss of life should be prevented. [4]

The capacity-based design principle is an effective design method developed to achieve the objectives of this universal earthquake resistant design philosophy. This principle is intended to be controlled by the designer and designed for reduced earthquake loads with limited to "Seismic Fuse" elements (Akbaş, 2008), in structures with foresaw nonlinear behavior caused by strong ground movement, which will not lead to the collapse of the structure. As is known, the earthquake load reduction coefficient (coefficient R) and the desired energy consumption mechanism (fuse element) differ for each horizontal load-bearing system (moment frames, concentric or eccentric diagonal frames etc.).

2.3. Steel Moment Frame

This discussion of previous research shows that previous studies show that the welded flange / bolted tape connection provides acceptable seismic behavior. These compounds were initially used on almost all beam-column connections in the structural system. This results in a good distribution of lateral rigidity and strength, so that the size of the elements and the interconnections are relatively small. Many 20 and 30 story steel structures were built in the 1960s and early 1970s with W21, W24 and W27 beams. These member sizes were not inappropriate compared to the connection tests done at that time. For more efficiency, engineers later designed seismic resistance as the structures' perimeter. "Perimeter frames reduced the number of moment-frame connections and increased the member and connection size. Further reductions in the number of connections later occurred because of concentration of the seismic resistance into individual isolated frames or bays of frames, and this also resulted in a further increase in member size" (FEMA-335D 2000). This concentration of seismic safety on small parts of the structure meant that even 2 and 3-storey buildings required W36 and W40 beams. This practice should have been called into question because no tests were carried out about connections to members who approach these sizes, just before North Ridge Earthquake. A recent study (Roeder and Foutch, 1996) of previously aggregated data revealed that compounds prior to Northridge could be affected by factors that are not normally considered in the design. This comparison showed that deeper beams would lead to a significant reduction in the plastic rotational capacity of the joint. [3]

Steel moment frames comes in three forms: Ordinary (OMF), Intermediate (IMF) and Special moment frames (SMF). The name changing difference comes from their expectancy to withstand remarkable inelastic deformation in their members (caused by lateral forces). SMF are usually using at mid-to-high seismic active regions.

2.4. Drift (Stiffness) Design

Design for drift and lateral stability is a topic that should be addressed in the early stages of design development. In many cases, especially in high-rise buildings, drift criteria can be a crucial factor in choosing the right building system. The lateral displacement or drift of a structural system under the action of wind loads or earthquake loads, is important from three different points of view: overall structural stability; architectural aesthetics and possible damage on the non-structural components; and human comfort during and after the structure undergoes these forces.

2.5. Strong Column Weak Beam

The structures must have a uniform energy dissipation under a seismic load. This is done using a capacity design approach that requires good ductility and a desired collapse mechanism in the structure and it called as Strong Column Weak Beam. Overall design philosophy for SC-WB is to ensure that the energy dissipation under a seismic load is occurring not on the column-beam connection. Design will proceed with selecting column profiles which have higher plastic moment capacity than the beams connected to that specific column and ensuring if the design is on safe side in the terms of plastic hinge formation.

2.6. Reduced Beam Section Design

Moment frame's beam sections will cut on the edges, with a distance of S_h. This process is called Reduced Beam Section Connection Design, this will guarantee the control on plastic hinges of the structure under a seismic activity. RBS design is an essential design strategy for earthquake resistant steel structures with steel special moment resisting frame, in order to control plastic hinge formation of the structure against earthquake loads. In a connection with a reduced beam section (RBS), parts of the flanges of the beams are selectively radius cut in the area adjacent to the beam connection to columns. Yielding and formation of hinges aim to happen primarily on reduced part of the beam (FEMA-355D, 2000).

To find such a, b and c values that verifies the requirements for this application is a long, iterative process. To reduce the time and effort, I developed a MATLAB code (Appendix A) which working by only 15 user input such as selected frame's profiles' mechanical properties and loads acting on them and finding an optimal a, b and c value to make cut region plastic moment capacity lower than the connection are and error if RBS application is not available for that given profiles.

2.7. Strength Design

One of the two main approaches to the design of building structural systems under the influence of earthquake is the Design for Strength approach:

(a) Reduced earthquake loads corresponding to the ductility capacity of the carrier system defined for a specified performance target are determined.

(b) Linear earthquake calculation is done for the carrier system under reduced earthquake loads.

(c) The element strength requirements are compared with the element internal force capacities (strength capacities) defined for the foreseen performance target.

(d) The relative displacement obtained from the earthquake account are compared with the permissible limits.

(e) The design is completed by demonstrating that the strength requirements are below the strength capacities and while the relative displacements are below the permissible limits. Otherwise, the element sections are changed, and the result is repeated.

3. PROBLEM DEFINITION

Seismic design of the steel building mainly consists of two design steps, drift design and strength design. Choosing the adequate profiles for the beams and columns is an iterative process since both design steps needs to be done properly. One design step will always govern before another step and that step will be the critical design step. And ensuring that one design step is designed properly does not mean that the overall design is properly done. This right here becomes the main problem, iterating between design steps.

Since the seismic design requires to withstand huge amount of lateral forces, building will tend to withstand these forces with remarkable inelastic deformations and to have a remarkable inelastic deformation capability is the specialty of SMF systems. In order to stay within drift limitations of the respective seismic code, adequate selected profiles will be thought to be over-designed for the strength design step, so starting with drift design step for seismic design of the SMF systems with huge amount of lateral force acting on it might be shorten the iterative process.

3.1. Project Statement, Objectives and Scope

Project will start with drift design before the strength design to see if the iterative process really shortens and by the light of the results of the seismic design, the project will try to propose overall design strategy for the seismic design of SMF systems.

3.2. Data Description

Reported design project's steel building will construct at 40.7115°N and 29.7996°E, Gölcük/Kocaeli within the boundaries of Table 1. Design will be done for S355 steel grade. Building will have five story and a basement over a 2500 m^2 area as can be seen in Figure 2-3-4. Related Seismic Design Spectral Acceleration coefficients S_{DS} and S_{D1}.

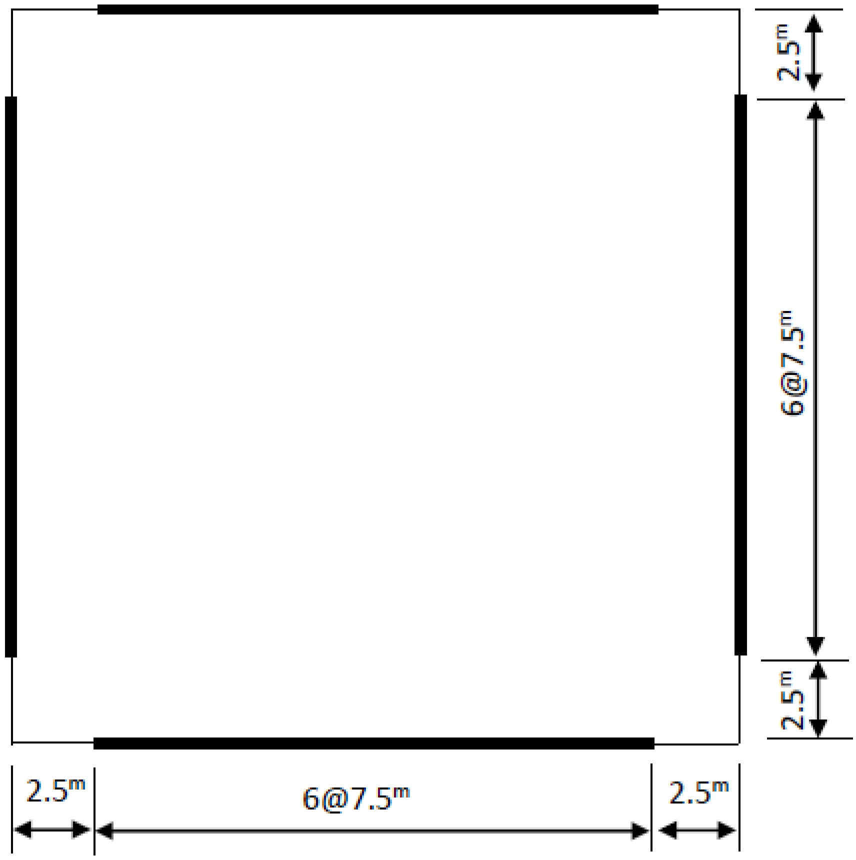

Figure 1: Prescribed Plan View of the Building and SMF Locations

Table 1: Prescribed Design Parameters of the Building

Prescribed Parameters	
Seismic Force-Resisting System	Steel Special Moment Frame
EQ Ground Motion Level	DD-2
Soil Type	ZC
Location	Gölcük, Kocaeli
Occupancy	Office Building

4. ANALYSES AND DESIGN PROCESS

In this section, previous stated design procedures will be done in detail with the design steps, hand calculations, Analyze Criteria and Analyze results.

4.1. Codes and Specifications

Codes and specifications used in this design project are listed below:

- TSC 2018, Turkish Seismic Code
- TS498, Minimum Design Loads for Structural Elements

4.2. Analysis, Design Criteria and Uncertainties

In this section, after every analysis result, design is done according to the data obtained. Design can be changed if any condition is not met while the analysis process is in progress. Plan-view (Fig 2.) and side-view (Fig 3.) drawings of the building. Span length of 2.5-meter decided between secondary beams. Interior frames' beam-to-column connections in both E-W and N-S directions assumed as shear connections. Bi-axial bending at the corner columns, the span edges on lines 1-9 are designed as part of gravity frames. SMF columns are assumed to be pinned (Fig 3.) at the base and roller at the ground level. No field test prepared or done for the seismic parameters. All values are taken from AFAD seismic maps. Steel grade and strength capacities are taken from respective codes and specifications, therefore no element capacity experiment prepared or done. Snow Load acting on the building is neglected due to its low-to-none effect on it. Design will be done for the 9-axis SMF.

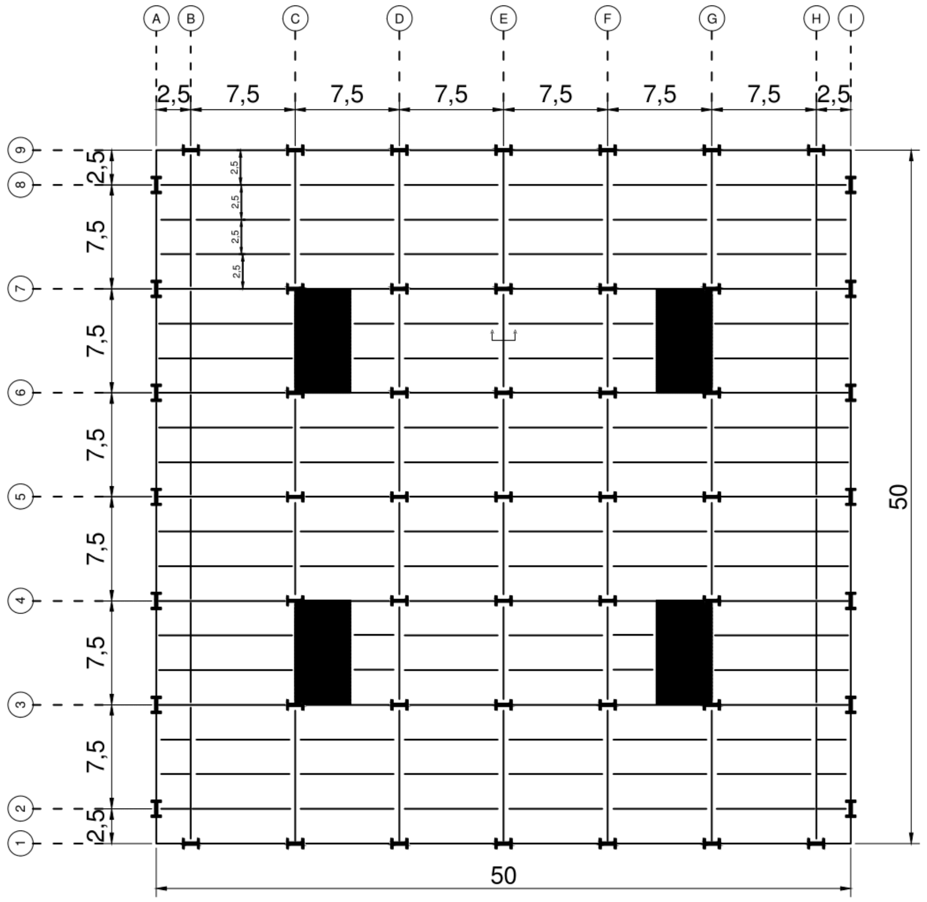

Figure 2: Detailed Plan Drawing of the Building

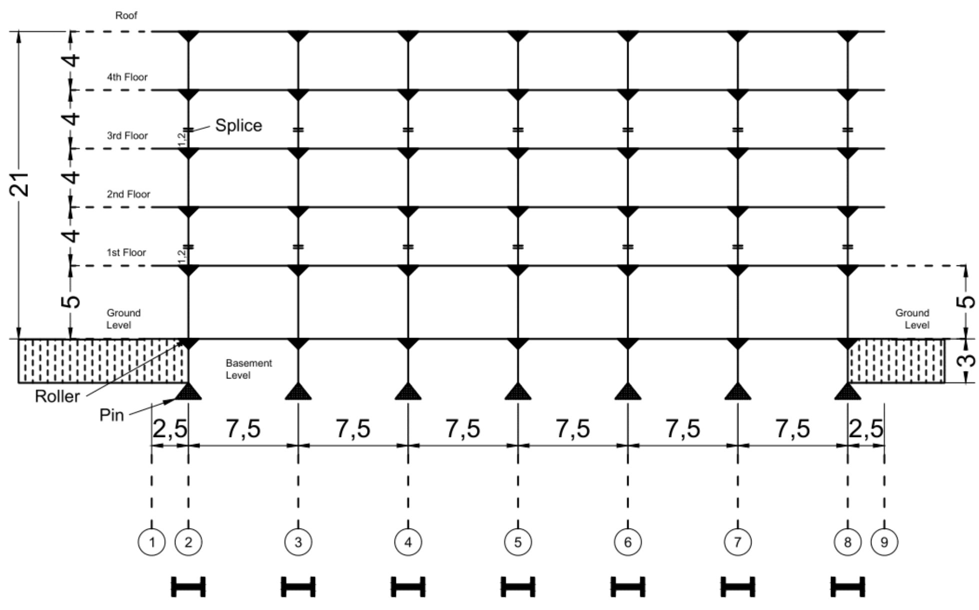

Figure 3: Frame Drawing with Story Heights, Connection Types, Splice Levels and Boundary Conditions

4.3. Analyses and Design

Design of the SMF will proceed on the way of analyses results while achieving the prequalified criteria of the TSC 2018. Procedure chapters are reported in design order.

4.3.1 Structural Load Distribution

Structural loads or actions are forces, deformations or acceleration applied to a structure or components. Loads cause stresses, deformations and displacements in structures. The evaluation of the effects is made by structural analysis methods.

4.3.1.1 Gravity Load Distribution

Dead load and Live load parameters (Table 2.) are respectively taken from TS 498, Chapter 12.1. Load path are shown in Fig 5. To calculate gravity loads acting on the SMF, tributary area (Fig 5.) and beam reaction (Fig 6.) calculations done. Purpose of the tributary area calculations are to find the vertical forces, which will be carried by the perimeter frame, SMF. Grey areas on the Fig 5. Refers that the area load on this area will transport to the above beam, and white areas refers that the area load on this area will transport to the below beam. Each square element on the Fig 6. refers to the connection of secondary beams. 3 horizontal secondary beams will transfer their load to the vertical secondary beam, which is on B-axis, and the beams on axis 8-9 calculated with this. Left-hand side secondary beams' span lengths are equal to 2.5 meters and right-hand side secondary beams' span lengths are equal to 7.5 meters. Circles in Fig 5. indicates that, wherever reach the circles perimeters, those areas will be the tribute areas.

Table 2: Vertical Loads Acting on the Building

Vertical Loads		
	Floor	Roof
Dead Load (kN/m^2)	4	2
Live Load (kN/m^2)	2	2

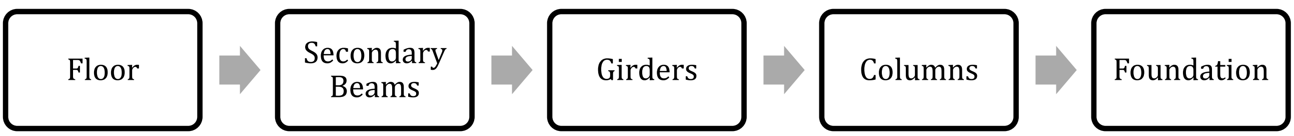

Figure 4: Load Path

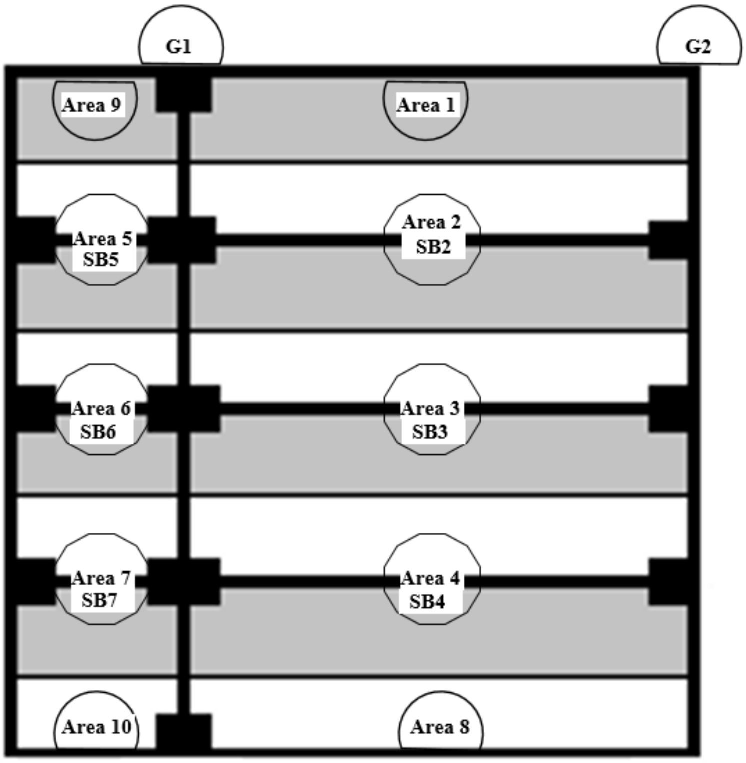

Figure 5: Tributary Area Between Axis 7 and C

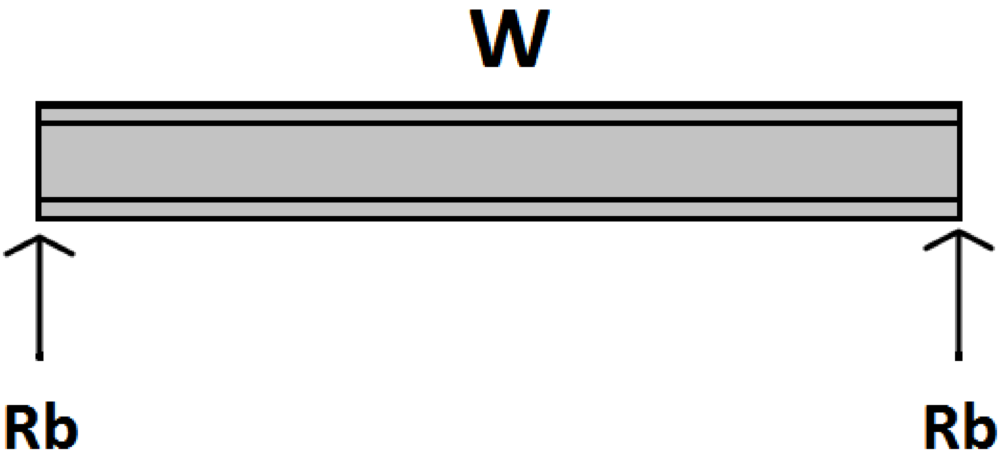

Figure 6: Free Body Diagram of Beams Reacting to Distributed Load W

Area Calculation (Fig 6.):

$$A_2 = A_3 = A_4 = (2.5^m \times 7.5^m) = \mathbf{18.75\ m^2}$$

$$A_5 = A_6 = A_7 = (2.5^m \times 2.5^m) = \mathbf{6.25\ m^2}$$

Load distribution on floor:

$$q_{Dfloor} = 2.5^m \times\ 4\ \frac{kN}{m^2} = \mathbf{10}\,\frac{\boldsymbol{kN}}{\boldsymbol{m}}$$

$$q_{Droof} = 2.5^m \times\ 1.6\ \frac{kN}{m^2} = \mathbf{4}\,\frac{\boldsymbol{kN}}{\boldsymbol{m}}$$

$$q_{Lfloor} = 2.5^m \times\ 2\ \frac{kN}{m^2} = \mathbf{5}\,\frac{\boldsymbol{kN}}{\boldsymbol{m}}$$

$$q_{Lfroof} = 2.5^m \times\ 2\ \frac{kN}{m^2} = \mathbf{5}\,\frac{\boldsymbol{kN}}{\boldsymbol{m}}$$

Reaction Forces of Secondary Beam:

$$R_{SB} = Secondary\ Beam\ Reaction\ Force$$

$$R_G = Girder\ Reaction\ Force$$

For Floor:

$$W_{floor} = q_{Dfloor} + q_{Lfloor} =\ \mathbf{15}\,\frac{\boldsymbol{kN}}{\boldsymbol{m}}$$

Reaction forces of the secondary beams:

$$R_{SB2} = R_{SB3} = R_{SB4} = \frac{15}{2}\frac{kN}{m} \times 7.5^m = \mathbf{56.25}\ \boldsymbol{kN}$$

$$R_{SB5} = R_{SB6} = R_{SB7} = \frac{15}{2}\frac{kN}{m} \times 2.5^m = \mathbf{18.75}\ \boldsymbol{kN}$$

Reaction forces of the girders:

$$R_{G1} = \frac{3 \times (R_{SB2} + R_{SB5})}{2} = \frac{3 \times (56.25\ kN + 18.25\ kN)}{2} = \mathbf{112.5}\ \boldsymbol{kN}$$

$$R_{G2} = \frac{6 \times R_{SB2}}{2} = \frac{3 \times 56.25\ kN}{2} = \mathbf{168.75}\ \boldsymbol{kN}$$

For Roof:

$$W_{roof} = q_{Droof} + q_{Lroof} =\ \mathbf{9}\,\frac{\boldsymbol{kN}}{\boldsymbol{m}}$$

Reaction forces of the secondary beams:

$$R_{SB2} = R_{SB3} = R_{SB4} = \frac{9}{2}\frac{kN}{m} \times 7.5^m = \mathbf{33.75}\ \boldsymbol{kN}$$

$$R_{SB5} = R_{SB6} = R_{SB7} = \frac{9}{2}\frac{kN}{m} \times 2.5^m = \mathbf{11.25}\ \boldsymbol{kN}$$

Reaction forces of the girders:

$$R_{G1} = \frac{3 \times (R_{SB2} + R_{SB5})}{2} = \frac{3 \times (33.75\ kN + 11.25\ kN)}{2} = \mathbf{67.5}\ \boldsymbol{kN}$$

$$R_{G2} = \frac{6 \times R_{SB2}}{2} = \frac{3 \times 33.75\ kN}{2} = \mathbf{101.25}\ \boldsymbol{kN}$$

4.3.1.2 Wind Load Calculation

Wind pressure acting on the exterior walls are assumed uniformly distributed over the area indicated at Fig 3. for 50-meters. Wind load acting on the building will distribute evenly between parallel SMFs. Building height over ground is 21-meters, q (kN/m^2) velocity pressure is taken as 1.1 kN/m^2 from TS498, Chapter 11.3. Equivalent forces acting on diaphragms (Fig 7.) again, calculated with tributary areas. Every diaphragm will take the equivalent wind load acting on the both above and below stories' half height.

$$W_q = q \times \frac{50^m}{2} = 1.1\frac{kN}{m^2} \times \frac{50^m}{2} = \mathbf{27.5\frac{kN}{m}}$$

Wind load acting on a diaphragm:

$$F = W_q \times (\frac{Above\ Story\ Height}{2} + \frac{Below\ Story\ Height}{2})$$

Wind load calculation:

$$F_1 = 27.5\frac{kN}{m} \times \left(\frac{4^m}{2} + \frac{5^m}{2}\right) = \mathbf{124\ kN}$$

$$F_2 = F_3 = F_4 = 27.5\frac{kN}{m} \times \left(\frac{4^m}{2} + \frac{4^m}{2}\right) = \mathbf{110\ kN}$$

$$F_1 = 27.5\frac{kN}{m} \times \left(\frac{4^m}{2}\right) = \mathbf{55\ kN}$$

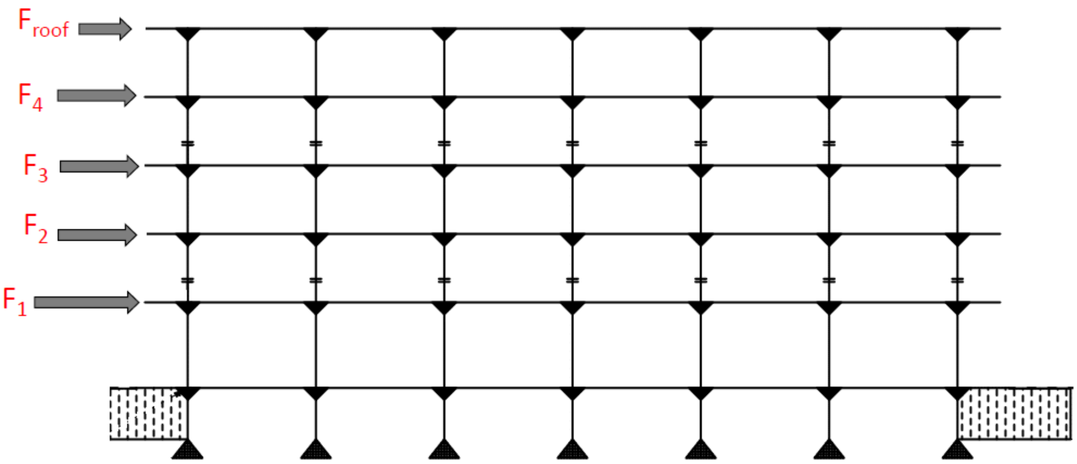

Figure 7: Wind Load Acting on Diaphragms

4.4.2 Seismic Load Calculation

Seismic load acting on the building calculated with equivalent earthquake load method. In this method, the equivalent total seismic load (base shear) is distributed in static lateral load on the diaphragm levels. This distribution is triangular (zero in the lower range and maximum in the upper range, that is, simulation of the first mode of the dynamic response). The distributed equivalent load is applied to the center of the mass on each floor.

4.4.2.1 Seismic Parameters

Table 3: Seismic Design Values Taken from AFAD

Seismic Parameters	
Ss	1,72
S1	0,466
Sds	2,064
Sd1	0,699

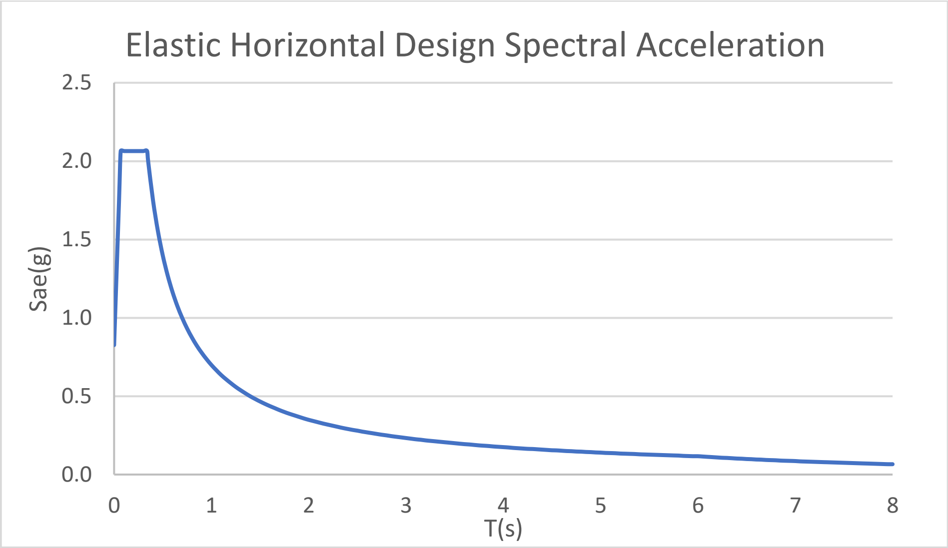

Figure 8: Elastic Horizontal Design Spectral Acceleration of Gölcük for DD-2 Ground Movement and ZC Soil

4.4.2.2 Natural Vibration Period Calculation of the Building

In order to choose a preliminary profile sections of beams and columns for drift design, natural period of the building extract from the approximate period formula to calculate equivalent earthquake load acting on the building:

$$T_{pA} = C_t \times {H_n}^{3/4}$$

Ct value is taken as 0.08(4.7.3.4 TSC 2018) for steel buildings and since the respective H_n value of the building is 21-meters from the seismic base, natural period of the building calculated as:

$$T_{pA} = 0.08 \times (21)^{3/4} = \mathbf{0.7848}\boldsymbol{s}$$

And from this natural period elastic horizontal design spectral acceleration is calculated as:

$$S_{Ae}\left(T_{pA}\right) = \frac{S_{D1}}{T_{pA}} = \frac{0.699}{0.7850} = \mathbf{0.89}\boldsymbol{g}$$

4.4.2.3 Mass Participation

Mass participation is a necessity in order to calculate equivalent earthquake load. Mass participation ratio value 0.3 is taken from (4.5.9.2 TSC 2018). For a building with N story, 5 for this report, participating total mass calculation started with determining the total force acting on the floor:

$$W_i = W_g + nW_q$$

One floor’s participating mass:

$$m_i = \frac{W_i}{g}$$

All floors’ participating mass:

$$m_T = \sum_{i=1}^{N} m_i$$

One floor’s total force acting on it:

$$W_i = 4\,\frac{kN}{m^2} \times (50^m \times 50^m) + 0.3 \times 2\,\frac{kN}{m^2} \times (50^m \times 50^m) = \mathbf{11500}\ \boldsymbol{kN}$$

One floor’s participating mass:

$$m_i = \frac{11500\ kN}{9.81\,\frac{m}{s^2}} = \mathbf{1172.3}\ \boldsymbol{tonne}$$

Roof’s total force acting on it:

$$W_r = 1{,}6\frac{kN}{m^2} \times (50^m \times 50^m) + 0.3 \times 2\frac{kN}{m^2} \times (50^m \times 50^m) = \boldsymbol{5500\ kN}$$

Roof’s participating mass:

$$m_i = \frac{5500\ kN}{9.81\frac{m}{s^2}} = \boldsymbol{560.6\ tonne}$$

Total Participated Mass for this building with four stories and a roof:

$$m_T = \sum_{i=1}^{N} m_i = \sum_{i=1}^{5} m_i = m_1 + m_2 + m_3 + m_4 + m_{roof}$$

$$m_T = 4 \times (1172.3\ tonne) + 560.6\ tonne = \boldsymbol{5250\ tonne}$$

4.4.2.4 Equivalent Earthquake Load

In order to start seismic design, the earthquake loads acting on the building need to find for buildings natural period and as no section for the building selected at this step, estimated natural period used in to calculate the total base shear force.

Equivalent earthquake method is a method such that the total base shear of a building will selectively be operated on the floors, force will be distributed to diaphragms while increasing from bottom to top, like as imposing the 1st mode. (Fig 9.)

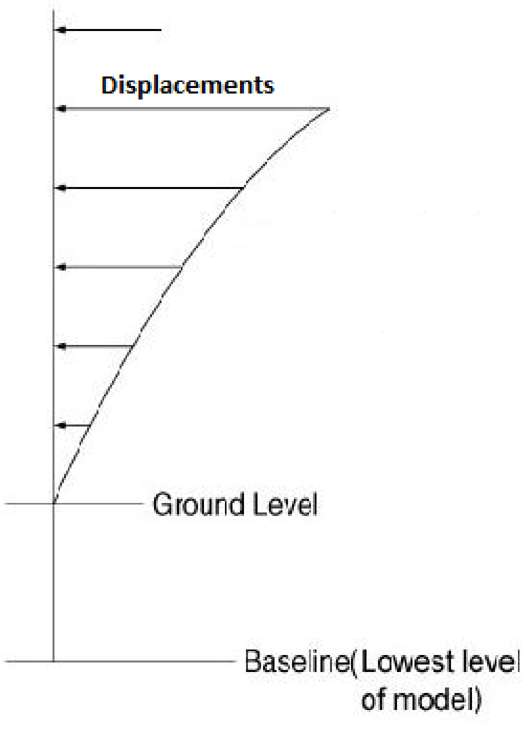

Figure 9: Equivalent Earthquake Method

Method simply starts with the calculation of the total base shear:

$$V_{te(x)} = m_T \times \mathrm{S}_{aR}(\mathrm{T}_p(\mathrm{x})) \times \mathrm{g} \geq 0.04 \times m_T \times I \times S_{DS} \times \mathrm{g}$$

And to find reduced design spectral acceleration, horizontal $\mathrm{S}_{ae}(\mathrm{T}_1)$ will reduced with the response modification factor:

$$\mathrm{S}_{aR}(\mathrm{T}_p) = \frac{\mathrm{S}_{ae}(\mathrm{T}_1)}{R/I} = \frac{0.890}{8/1} = \mathbf{0.1112}\ \boldsymbol{g}$$

Then total base shear simply calculated as:

$$V_{te(x)} = 5250\ tonne \times 0.1112 \times 9.81 \frac{m}{s^2} \geq 0.04 \times 5250\ tonne \times 1 \times 2.064 \times 9.81 \frac{m}{s^2}$$

$$V_{te}(x) = \mathbf{5727}\ \boldsymbol{kN} \geq 4251\ \mathrm{kN}$$

$$V_{te}(x) = \mathbf{5727}\ \boldsymbol{kN}$$

As can be seen that the mass participation is lower at the roof (Fig 9., Table 4.), additional earthquake load of $\Delta F_{NE}(x)$ will add to the roof diaphragm.

$$\Delta F_{NE}(x) = 0.0075 \times N \times V_{te}(x)$$

For a building with 5 story height and total base shear of 5727 kN, additional load is equal to:

$$\Delta F_{NE}(x) = 0.0075 \times 5 \times 5727kN = \mathbf{215}\ \boldsymbol{kN}$$

As there are SMF in all perimeters, earthquake loads acting on building will distribute evenly with the perimeter in the same direction.

Table 4: Equivalent Earthquake Load Distribution

Story	Height from seismic basement (m)	mi	mihi	mihi/total mjhj	Vte (kN)	ΔFNE (kN)	F (kN)	Per Frame (kN)
5	21	560,6	11772,6	0,185823108	5727	215	1239,25697	619,6284848
4	17	1172,3	19929,1	0,314568345			1733,900716	866,9503581
3	13	1172,3	15239,9	0,240552264			1325,924077	662,9620386
2	9	1172,3	10550,7	0,166536183			917,947438	458,973719
1	5	1172,3	5861,5	0,092520101			509,9707989	254,9853995
		total mjhj	63353,8	1			5512	2756

4.5.3 Drift (Stiffness) Requirement

Moment Frames comes in three forms: Ordinary, Intermediate and Special. The building has Special Moment Frames which indicates the capability to withstand remarkable inelastic deformation at the members and the connections of the frames as the result of lateral loads. This deformation and displacement limitations restricted in (4.9.1.3 TSC 2018) to stay on the safe side.

If flexible joints are made between the infill walls and the frame elements made of brittle material, if the façade elements are connected to the external frames with flexible connections or if the infill wall element is independent of the frame:

$$\lambda \frac{\delta_{i,max}^{(x)}}{h_i} \leq 0.016K$$

Where $\delta_{i,max}^{(x)}$ is the maximum effective relative drift, h_i is the story height and the λ is the ratio of DD-3 elastic design spectral acceleration to DD-2 elastic design spectral acceleration, (defined in 4.34b TSC 2018) for the dominant vibration period in earthquake direction:

$$\lambda = \frac{S_{ae}^{DD_3}}{S_{ae}^{DD_2}}$$

K is a coefficient and equals to the 0.5 for the steel structures (4.9.1.4 TSC 2018) Drift limitation then becomes:

$$\delta_{i,max}^{(x)} \leq \frac{0.016 \times 0.5 \times h_i}{\frac{0.2918}{0.89}} = \begin{cases} \mathbf{0.097}, & if\ 4 - meters\ story\ heigth \\ \mathbf{0.122}, & if\ 5 - meters\ story\ heigth \end{cases}$$

4.5.3.1 Creating Building Model

Building modeled in X-Z direction to analyze the perimeter frame, SMF. Basement supports' movement restrained in x, y, z directions and buildings lateral displacement at the seismic base level is restrained. All beams in each floor are constraint as diaphragm for its respective floor level. Equivalent earthquake loads are applied for their respective diaphragms and model analyzed with the limited first order.

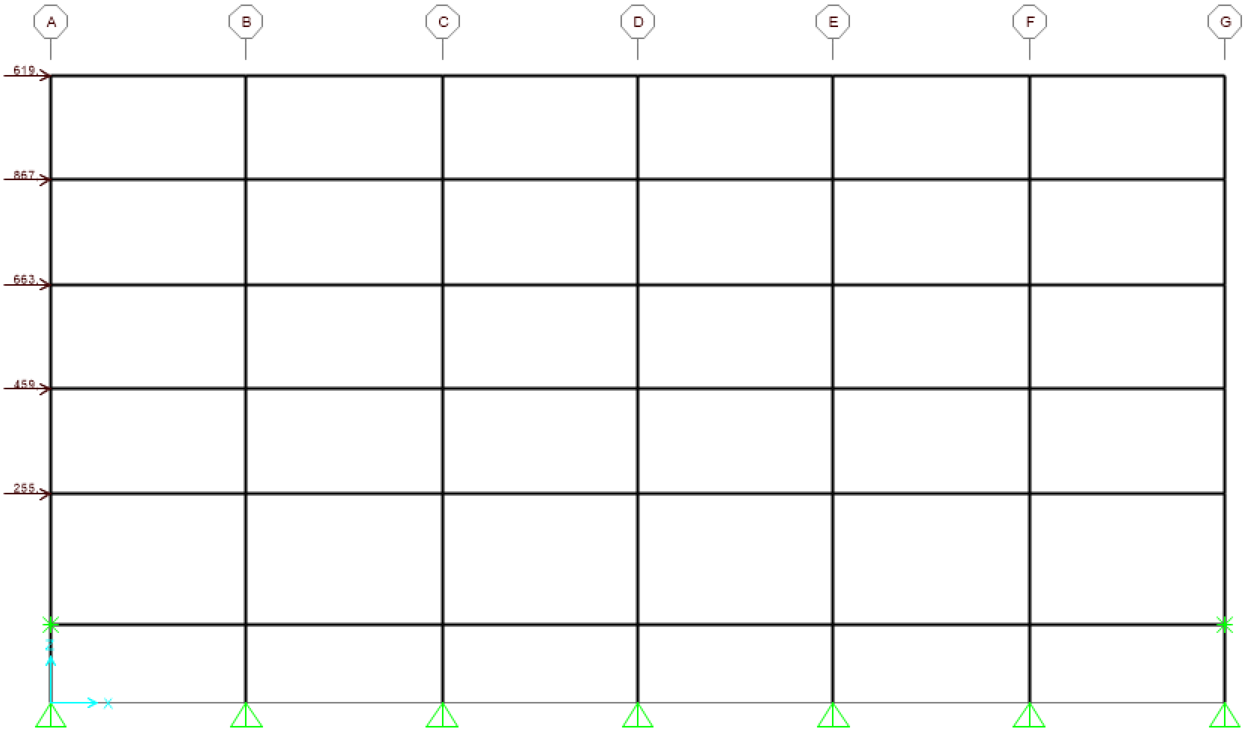

Figure 10: Building Model and Earthquake Loads Acting on Model

4.5.3.2 Relative Displacement Check

With selecting the limitation, profile selection for the sections comes and iteration starts to satisfy the drift limitation (Table 5.):

Table 5: First Section Selections and Limits

Story	Height from seismic basement (m)	mi	mihi	mihi/total mjhj	Vte (kN)	ΔFNE (kN)	F (kN)	Per Frame (kN)	Exterior	Interior	Beam	Translation to Limit Ratio
5	21	560,6	11772,6	0,19	5727	215	1239,3	619,6	**HE400M**	**HE550B**	**HE240A**	**0,857**
4	17	1172,3	19929,1	0,31			1733,9	867,0			**HE600A**	**0,893**
3	13	1172,3	15239,9	0,24			1325,9	663,0	**HE550M**	**HE650B**	**HE600A**	**0,949**
2	9	1172,3	10550,7	0,17			917,9	459,0			**HE700A**	**0,994**
1	5	1172,3	5861,5	0,09			510,0	255,0	**HE650M**	**HE700B**	**HE700A**	**0,864**
		total mjhj	63353,8	1,00			5512,0	2756,0			**HE800A**	**0**
		T	f									
		1,09763	0,91106									

After satisfying limitation for the equivalent earthquake loads for the assumed natural vibration period of the building, another profile selection for the sections comes, this time for the natural vibration period due to previous chosen profile sections:

Table 6: Final Section Selections and Limits

Story	Heigth from seismic basement (m)	mi	mihi	mihi/total mjhj	Vte (kN)	ΔFNE (kN)	F (kN)	Per Frame (kN)	Exterior	Interior	Beam	Translation to Limit Ratio
5	21	560,6	11772,6	0,19	4251	159,4	919,7	459,9	**HE340M**	**HE400M**	**HE240A**	**0,958**
4	17	1172,3	19929,1	0,31			1287,1	643,5			**HE500A**	**0,960**
3	13	1172,3	15239,9	0,24			984,2	492,1	**HE400M**	**HE500M**	**HE550A**	**0,910**
2	9	1172,3	10550,7	0,17			681,4	340,7			**HE550A**	**0,987**
1	5	1172,3	5861,5	0,09			378,6	189,3	**HE500M**	**HE600M**	**HE600A**	**0,813**
		total mjhj	63353,8	1,00			4091,6	2045,8			**HE600A**	**0**
		T	f									
		1,40175	0,71339									

4.6.4 Reduced Beam Section Design

In seismic active areas, buildings need to dissipate notable energy during earthquake to prevent major damages and life loss. This can be achieved with highly ductile structural elements. Reduced Beam Section Connection design is an essential design strategy for earthquake resistant steel structures with steel special moment frame. Reduce the beam section to improve the ductility of the beam, so that the stress concentration is transferred to an area away from the connection.

In a connection with reduced beam section (RBS), parts of the flanges of the beams are selectively radius cut in the area adjacent to the beam connection to columns. Yielding and formation of hinges aim to happen primarily on reduced part of the beam.

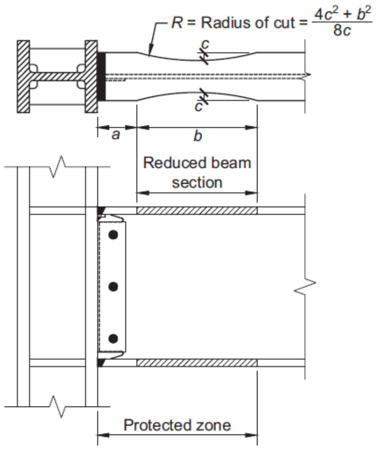

Figure 11: RBS Connection Detail

Limitations on Beam Dimensions [5] starts with; beam must not exceed 1040 mm, Beam must not exceed weight 447 kg/m, beam flange thickness must not exceed 44 mm, clear span to ratio over depth must be equal or greater than 7. Limitations on column dimensions starts with columns' depth must not exceed 1040 mm and there are no restraints on column weight, flange thickness and web thickness. After satisfying the limitations, procedure starts with selecting trials values of:

$$0.50b_{bf} \leq a \leq 0.75b_{bf}$$

$$0.65b_{bf} \leq b \leq 0.85b_{bf}$$

$$0.10b_{bf} \leq c \leq 0.25b_{bf}$$

Resuming with plastic section modulus calculation at the center:

$$Z_{RBS} = Z_x - 2ct_{bf}(d - t_{bf})$$

Maximum probable moment calculation at the center:

$$M_{pr} = C_{pr} R_y F_y Z_{RBS}$$

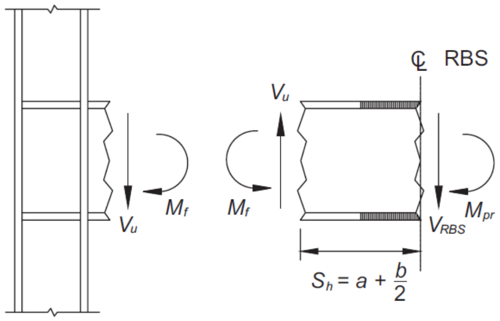

Figure 12: Free Body Diagram of RBS

After calculating M_{pr}, Shear force acting on the reduced beam section at each end calculation and Maximum probable moment at the face of the column calculation comes.

$$M_f = M_{pr} + V_{RBS} S_h$$

Plastic Moment of the beam based on the expected yield stress:

$$M_{pe} = R_y F_y Z_x$$

Final Flexural Strength of the beam checking:

$$M_f = \emptyset_d M_{pe}$$

Table 7: RBS Designs

	Beam Section	a (mm)	b (mm)	c (mm)	Z_{RBS} (mm^3)	C_{pr}	M_{pr} (kNm)	S_h (m)	L_h (m)	V_{RBS} (kN)	M_f (kNm)	φM_{pe} (kNm)
Exterior Bays	HEA240	150	173	42	524856	1.19	243.93	0.24	6.66	96.54	266.76	290.77
	HEA500	188	368	53	2810454	1.19	1306	0.37	6.39	431	1466.52	1542
	HEA550	188	405	53	3309296	1.19	1538	0.39	6.28	511.7	1737.82	1804.9
	HEA550	188	405	53	3309296	1.19	1538	0.39	6.28	511.7	1737.82	1804.9
	HEA600	188	443	53	3852750	1.19	1790	0.41	6.15	603.9	2037.87	2089.2
	HEA600	188	443	53	3852750	1.19	1790	0.41	6.15	603.9	2037.87	2089.2
Interior Bays	HEA240	150	173	42	524856	1.19	243.93	0.24	6.63	96.7	266.8	290.77
	HEA500	188	368	53	2810454	1.19	1306	0.37	6.36	432.8	1467	1542
	HEA550	188	405	53	3309296	1.19	1538	0.39	6.23	515.2	1739	1804.9
	HEA550	188	405	53	3309296	1.19	1538	0.39	6.23	515.2	1739	1804.9
	HEA600	188	443	53	3852750	1.19	1790	0.41	6	617.4	2043	2089.2
	HEA600	188	443	53	3852750	1.19	1790	0.41	6	617.4	2043	2089.2

4.6.5 Design for Stability

In order to ensure the stability of the whole structure and each of its elements, the effects of all the following factors on the stability of the structure and its elements must be considered; Deflections due to bending and axial parts, as well as all other deformations of components and connections, which contribute to the displacements of the structure, second order effects, imperfections due to geometry, stiffness reduction caused by inelasticity, uncertainty in members of the system, overall system and connections and stiffness.

4.6.5.1 Direct Analysis Method

Direct analysis of the design method requires static software and becomes the standard method for analyzing structural stability. The essence of this method is summarized below:

- There is no limit to the use of this method
- Axial forces acting solely on the gravity columns must be included in the structural model.
- All stiffness (bending, shearing) of the structural elements must be reduced 20%
- The theoretical loads Ni, given the imperfections, must be added to the load combinations involving only gravity, except in the case where the ratio of the maximum drift of the second order to the maximum drift of the first order exceeds 1.7, with fictitious charges must be added to combinations, the others contain lateral loads (earthquake and wind)
- The stiffness reduction parameter τb is used to adjust the stiffness of all elements

4.6.5.2 Leaning Column Model

Step-by-step analysis contains:

- Create a realistic digital model of lateral force resistance with "sloped columns" to introduce the influence of gravity frames on second order effects.
- Reduce the rigidity of the lateral force system.
- Applying fictitious loads as factorial or fictitious gravitational loads
- Offset to consider the initial imperfections.
- Perform a second-order analysis that considers the effects P - Δ and P δ.

- Determine the members ability with K = 1.0.
- Check the ratio of second-order galleries to first-order galleries ($\Delta 2^{nd}$ / $\Delta 1^{st}$)

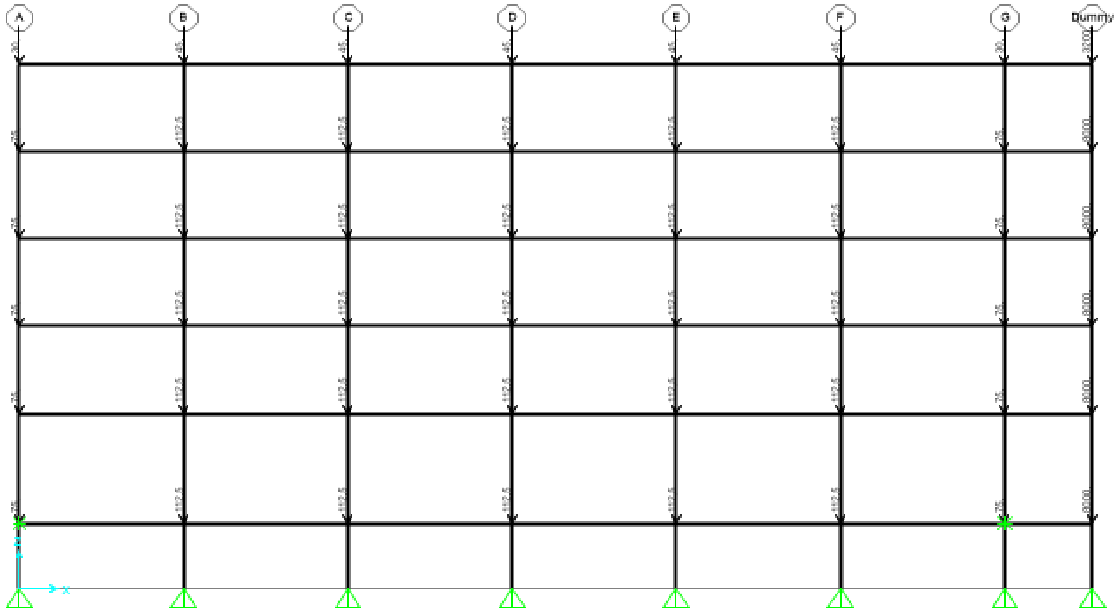

Figure 13: Dummy Model and Dead Load Acting on it

Tributary are for the dummy columns are consist of the total tributary area without the SMF's tributary areas.

$$Tributary\ area = 40m \times 50m = 2000\ m^2$$

And since the notion loads are equal to:

$$N_D, N_L, N_{LR} = 0.002 \times \alpha \times Y_i$$

Notional Dead loads (Fig 14.):

$$N_{D1} = N_{D2} = N_{D3} = N_{D4} = N_{D5} = 0.002 \times 1 \times (8937\ kN) = 17.9\ kN$$

$$N_{Droof} = 0.002 \times 1 \times (3575\ kN) = 7.15\ kN$$

Notional Live Loads:

$$N_{L1} = N_{L2} = N_{L3} = N_{L4} = N_{L5} = 0.002 \times 1 \times (4469\ kN) = 8.94\ kN$$

$$N_{LR} = 0.002 \times 1 \times (4469\ kN) = 8.94\ kN$$

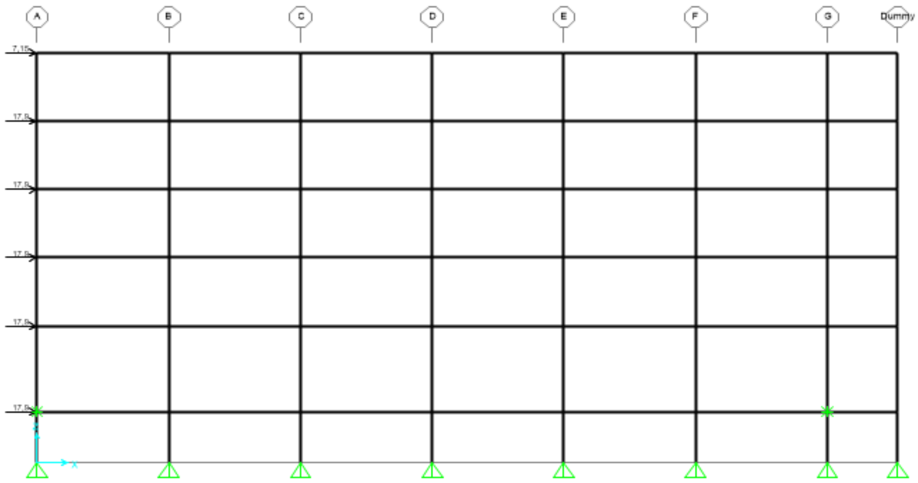

Figure 14 : Notional Dead Loads

4.6.5.3 Second Order Effects

Analyses done for these design load combinations:

Table 8: Design Combinations

Combination	Description
Comb1	1.4D ± 1.4ND
Comb2	1.2D + 1.6L + 0.5Lr ± (1.2ND + 1.6NL + 0.5NLr)
Comb3	1.2D + 1.0L + 1.6Lr ± (1.2ND + 1.0NL + 1.6NLr)
Comb4	1.2D + 1.6Lr ± 0.5W
Comb5	1.2D ± 1.0W + L + 0.5Lr
Comb6	0.9D ± 1.0W
Comb7	D + L + Ed(h) + 0.3Ed(z)
Comb8	0.9D + Ed(h) - 0.3Ed(z)

In future calculations, for example Comb2- will refer to the minus sign version of the Comb2. Results are reported in appendix B. Since the maximum $\Delta 2^{nd}$ / $\Delta 1^{st}$ ratio is 1.15 which is lesser than 1.7, there is no need to add notional loads to the lateral load combinations. Acting loads on the structure are granted thru the analysis and these loads will used in further calculations.

4.6.6 Strong Column Weak Beam

In the building, general principle for designing strong column weak beam is to ensure that the total plastic moment capacity of the columns at a joint is bigger that the total plastic moment capacity of the beams connecting to it. In this project, reduced beams sections used.

$$\frac{\sum M_{pc}}{\sum M_{pb}} > 1$$

$$\sum M_{pc} = \sum Z_c \times (F_{yc} - \frac{P_r}{A_g})$$

$$\sum M_{pb} = \sum \left(M_{pr} + M_v\right) = \sum \left[1.1 \times R_y \times F_y \times Z_{RBS} + V_{RBS} \times (S_h + \frac{d_c}{2})\right]$$

For further calculations, 3rd joint at the interior column axis chosen as the example calculation for the table 9. Total plastic moment capacity of columns:

$$\sum M_{pc} = \sum 7094\ cm^3 \times \left(2 \times 355\ MPa - \frac{704\ kN}{344.3\ cm^2} - \frac{1001.8\ kN}{344.3\ cm^2}\right) = \mathbf{4685\ kNm}$$

Total plastic moment capacity of beams:

$$\sum M_{pb} = \sum [1.1 \times 1.19 \times 355 MPa \times 3309\ cm^3 + 511.7\ kN \times (0.39 + 0.195) m]$$

$$\sum M_{pb} = \mathbf{3674\ kNm}$$

Total plastic moment capacity of columns to beams ratio:

$$\frac{\sum M_{pc}}{\sum M_{pb}} > \frac{4685\ kNm}{3674\ kNm} = \mathbf{1.27 > 1}$$

Table 9 : SC-WB Adequateness

	Joint	Column Above	Pr,above (kN)	Mpc,above (kNm)	Column Below	Pr,below (kN)	Mpc,below (kNm)	Total Mpc	Total Mpb	Mpc/Mpb
Exterior	6	-	0	0	HE340M	121,35	1656,76	1656,76	278,8	5,94
	5	HE340M	121,35	1656,76	HE340M	330	1625,6	3282,36	1517,8	2,16
	4	HE400M	330	1921,2	HE400M	625,5	1870,8	3792	1813,6	2,09
	3	HE400M	625,5	1921,2	HE400M	935,7	1766,8	3688	1813,6	2,03
	2	HE500M	935,7	2325,6	HE500M	1282,1	2254,2	4579,8	2155	2,13
	1	HE500M	1282,1	2254,2	HE500M	1521	2204,8	4459	2155	2,07
Interior	6	-	0	0	HE400M	192,2	1945	1945	562,9	3,46
	5	HE400M	192,2	1944,8	HE400M	445	1951,6	3896,4	1529,7	2,55
	4	HE500M	445	2426,7	HE500M	704	2373,3	4800	3674	1,31
	3	**HE500M**	**704**	**2373,3**	**HE500M**	**1001,8**	**2311,7**	**4685**	**3674**	**1,28**
	2	HE600M	1001,8	2872,4	HE600M	1300	2800,6	5673	4386,4	1,29
	1	HE600M	1300	2800,5	HE600M	1600	2728,2	5528,7	4386,4	1,26
									Minimum ratio	1,26

4.6.7 Member Requirements

SMF' systems require highly ductile, non-slender members so in order to ensure the adequateness of chosen profiles, slenderness checked for both columns and beams.

Table 10 : Column Member Requirement Check

	Column	b(mm)	tf(mm)	b/2tf	λhd,f	h(mm)	tw(mm)	h/tw	λhd,w	Slenderness	Ductility
Exterior	HE340M	309	40	3,86	13,3	377	21	17,95	35,3	Non-Slender	Highly Ductile
	HE400M	307	40	3,838	13,3	432	21	20,57	35,3	Non-Slender	Highly Ductile
	HE500M	306	40	3,825	13,3	524	21	24,95	35,3	Non-Slender	Highly Ductile
Interior	HE400M	307	40	3,838	13,3	432	21	20,57	35,3	Non-Slender	Highly Ductile
	HE500M	306	40	3,825	13,3	524	21	24,95	35,3	Non-Slender	Highly Ductile
	HE600M	305	40	3,813	13,3	620	21	29,52	35,3	Non-Slender	Highly Ductile

Table 11 : Beam Member Requirement Check

Story	Girder	b/2tf	h/tw	Slenderness	Compactness	Ductility
Roof	HE240A	-	-	Non-appliable for S355 Steal		-
5	HE500A	6,5	32,5	Non-Slender	Compact	Highly Ductile
4	HE550A	11	35	Non-Slender	Compact	Highly Ductile
3						
2	HE600A	6	37,4	Slender	Compact	Highly Ductile
1						

As can be seen in Table 10 and Table 11, column profiles are all highly ductile and non-slender but roof and first two story beams of the building are non-applicable and slender, so these profiles need to change.

Table 12 : Final Beam Member Requirement Check

Story	Girder	b/2tf	h/tw	Slenderness	Compactness	Ductility
Roof	HE240B	7,1	16,4	Non-slender	Compact	Highly Ductile
5	HE500A	6,5	32,5	Non-Slender	Compact	Highly Ductile
4	HE550A	11	35	Non-Slender	Compact	Highly Ductile
3						
2	HE600B	5	31,4	Non-slender	Compact	Highly Ductile
1						

New beam profiles chosen thicker than the previous ones to ensure the non-slenderness. Since at the SC-WB calculations, $\frac{\sum M_{pc}}{\sum M_{pb}}$ ratios are safe enough to increase the capacities of the beam section, so they're adequate.

4.6.8 Design for Strength

Design for strength comes in flexure and nominal strength capacities cases for columns, and flexural case for beam. Nominal capacities are calculated and checked
For the first two story columns (one interior-HE600M and one exterior-HE500M), located right above the seismic base. These two columns are checked for all eight combinations' required loads and required moments. Overall design philosophy is that capacity must be bigger than the demand.

Column Strength

Axial Strength

Axial strength calculations started with determining the maximum demanded axial force acting on it. In example calculation exterior column's (HE500M) design is shown below. Maximum demanded axial force acting on it $P_{rmax} = \mathbf{1282\ kN}$ $(comb\ 7-)$, and the yielding strength of the column is equal to:

$$P_y = A_g \times F_y$$

For a column with 344.3 cm^2 gross area and S355 steel grade, yielding strength is equal to:

$$P_y = 344.3 cm^2 \times 355\ MPa = \mathbf{12222.6\ kN}$$

Flexural stiffness adjustment based on the level of axial force to axial capacity ratio:

$$\frac{\alpha P_{rmax}}{P_y} = \frac{1.0 \times 1282\ kN}{12222.6\ kN} = \mathbf{0.101} \leq 0.5$$

thus the $\tau_b = \mathbf{1.0.}$

After the stiffness adjustment, determining capacities of members come, using K=1:

$$K_x = K_y = \mathbf{1.0}$$

Slenderness in both directions checked to find the buckling axis:

$$\left(\frac{L_c}{r}\right)_{max} = \left(\frac{L_c}{r}\right)_y = \frac{5000\ mm}{72.2\ mm} = \mathbf{69.25} < 4.71\sqrt{\frac{200\ GPa}{355\ MPa}} = \mathbf{111.8}$$

thus, inelastic buckling would occur.

Euler buckling strength need to calculate in order to find the critical strength:

$$F_e = \frac{\pi^2 E}{\left(\frac{KL_c}{r}\right)^2_y}$$

Euler buckling strength of the steel column with 200 GPa modulus of elasticity:

$$F_e = \frac{\pi^2 \times 200\ GPa}{(69.25)^2} = \mathbf{411.6\ MPa}$$

Average compressive strength of the column:

$$F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right) \times F_y$$

and for S355 steel grade, calculated as:

$$F_{cr} = \left(0.658^{\frac{355}{411.6}}\right) \times 355\ MPa = \mathbf{247.4\ MPa}$$

Nominal axial strength of the column is equal to the:

$$P_n = A_g \times F_{cr}$$

For the column with 344.3 cm^2 gross area and 247.4 MPa average compressive strength:

$$P_n = 344.3\ mm^2 \times 247.4\ MPa = \mathbf{8519\ kN}$$

Design axial strength is equal to the factored nominal axial strength.

$$P_c = \emptyset P_n = \emptyset_c \times F_{cr} \times A_g$$

$$P_c = 0.9 \times 8519\ kN = \mathbf{7667\ kN}$$

Flexural Strength

Flexural strength design is starts with checking lateral torsional buckling (LTB) and to check the LTB, L_p, L_r and L_b values needed to find

Limiting laterally unbraced length to prevent and LTB L_p, and laterally unbraced length L_b:

$$L_p = 1.76 i_y \times \sqrt{\frac{E}{F_y}}$$

$$L_p = 1.76 \times 74.6 \times \sqrt{\frac{2 \times 10^5\ MPa}{355\ MPa}} = \mathbf{3116.4\ mm} < L_b = \mathbf{5000\ mm}$$

Limiting laterally unbraced length to prevent elastic LTB:

$$L_r = 1.95 i_{ts} \times \frac{E}{0.7F_y} \times \sqrt{\frac{J_c}{w_{ex}h_o} + \sqrt{\left(\frac{J_c}{w_{ex}h_o}\right)^2 + 6.76 \times \left(\frac{0.7F_y}{E}\right)^2}}$$

$$L_r = \mathbf{14410}\ \boldsymbol{mm}$$

And since $L_p < L_b < L_r$

Nominal moment capacity becomes:

$$M_n = C_b \times \left[M_p - \left(M_p - 0.7F_y \times w_{ex}\right) \times \left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \leq M_p$$

LTB buckling modification factor C_b is:

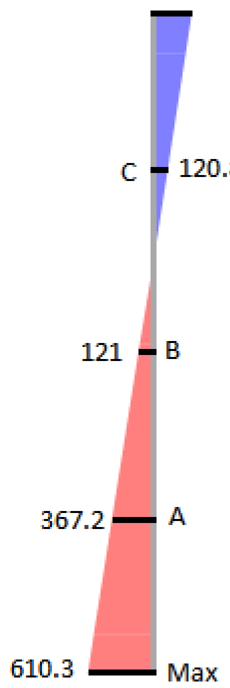

Figure 15 : Cb Factor Determination

$$C_b = \frac{12.5Max}{2.5Max + 3A + 4B + 3C}$$

$$C_b = \frac{12.5 \times 610.3\ kNm}{(2.5 \times 610.3 + 3 \times 367.2 + 4 \times 121 + 3 \times 120.8)\ kNm} = \mathbf{2.19}\ for\ all\ sections$$

Plastic moment capacity of the section:

$$M_p = Z_x F_y$$

$$M_p = 7094\ cm^3 \times 355\ MPa = \mathbf{2518.37}\ kNm$$

Thus, the nominal moment capacity becomes:

$$M_n = 2.19 \times \left[2518.4\ kNm - (2518.4\ kNm - 0.7 \times 355\ MPa \times 6180) \times \left(\frac{5000 - 3116.4}{14410 - 5000}\right)\right] \leq M_p$$

$$M_n = \mathbf{5084\ kNm} <\neq M_p = \mathbf{2518.4\ kNm}$$

Since nominal moment capacity cannot be greater than the plastic moment capacity, nominal capacity is become equal to the plastic moment capacity. And the design capacity is equal to the LRFD factored moment capacity.

$$M_C = \emptyset b \times M_p = 0.9 \times 2518.4\ kNm = \mathbf{2266.5\ kNm}$$

After finding the axial and flexural capacities, P-M interaction checking comes:

$$\frac{P_r}{2P_c} + \frac{M_r}{M_c} \leq 1.0, \qquad if\ \frac{P_r}{P_c} \leq 0.2$$

Or

$$\frac{P_r}{P_c} + \frac{8M_r}{9M_c} \leq 1.0, \qquad if\ \frac{P_r}{P_c} > 0.2$$

For the HE500M section:

$$\frac{P_r}{P_c} = \frac{1282\ kN}{7667\ kN} = 0.167 \leq 0.2$$

P-M:

$$\frac{1282\ kN}{2 \times 7667\ kN} + \frac{610\ kNm}{2518\ kNm} = 0.326 \leq 1.0$$

For all other load combinations, results are listed at Table 13-14-15.

Table 13 : Axial Strength Adequateness of the Exterior Column

Combination	Section	Pr (kN)	Py(kN)	Pr/Py	τb
COMB1	HE500M	537,528	12223	0,044	1.0
COMB1-	HE500M	632,128	12223	0,052	1.0
COMB2	HE500M	752,218	12223	0,062	1.0
COMB2-	HE500M	905,297	12223	0,074	1.0
COMB3	HE500M	676,346	12223	0,055	1.0
COMB3-	HE500M	859,126	12223	0,070	1.0
COMB4	HE500M	523,394	12223	0,043	1.0
COMB4-	HE500M	632,721	12223	0,052	1.0
COMB5	HE500M	603,201	12223	0,049	1.0
COMB5-	HE500M	826,699	12223	0,068	1.0
COMB6	HE500M	270,501	12223	0,022	1.0
COMB6-	HE500M	481,417	12223	0,039	1.0
COMB7	HE500M	277,728	12223	0,023	1.0
COMB7-	HE500M	1282,119	12223	0,105	1.0
COMB8	HE500M	258,516	12223	0,021	1.0
COMB8-	HE500M	665,362	12223	0,054	1.0
			Max Pr/Py	0,105	

Table 14 : Axial Strength Adequateness of the Interior Column

Combination	Section	Pr (kN)	Py(kN)	Pr/Py	τb
COMB1	HE600M	916,149	12911	0,071	1.0
COMB1-	HE600M	916,534	12911	0,071	1.0
COMB2	HE600M	1298,418	12911	0,101	1.0
COMB2-	HE600M	1299,029	12911	0,101	1.0
COMB3	HE600M	1200,933	12911	0,093	1.0
COMB3-	HE600M	1201,683	12911	0,093	1.0
COMB4	HE600M	903,351	12911	0,070	1.0
COMB4-	HE600M	903,804	12911	0,070	1.0
COMB5	HE600M	1119,626	12911	0,087	1.0
COMB5-	HE600M	1120,544	12911	0,087	1.0
COMB6	HE600M	588,64	12911	0,046	1.0
COMB6-	HE600M	589,513	12911	0,046	1.0
COMB7	HE600M	1220,459	12911	0,095	1.0
COMB7-	HE600M	1224,727	12911	0,095	1.0
COMB8	HE600M	316,785	12911	0,025	1.0
COMB8-	HE600M	320,747	12911	0,025	1.0
			Max Pr/Py	0,101	

Table 15 : Columns Axial and Flexural Strength Adequateness and P-M Interactions

Combination	Section	Pr (kN)	Pc (kN)	Mr (kNm)	Mc (kNm)	P-M Interaction
COMB1	HEM500	537,528	7667	47,915	2518,4	0,05408042
	HEM600	916,149	8099	100,8883	2518,4	0,09661986
COMB1-	HEM500	632,128	7667	81,362	2518,4	0,07353077
	HEM600	916,534	8099	103,5513	2518,4	0,09770105
COMB2	HEM500	752,218	7667	83,162	2518,4	0,08207716
	HEM600	1298,418	8099	167,3689	2518,4	0,14661758
COMB2-	HEM500	905,297	7667	130,900	2518,4	0,11101599
	HEM600	1299,029	8099	171,1472	2518,4	0,14815558
COMB3	HEM500	676,346	7667	101,940	2518,4	0,08458569
	HEM600	1200,933	8099	191,8816	2518,4	0,15033269
COMB3-	HEM500	859,126	7667	143,000	2518,4	0,1128096
	HEM600	1201,683	8099	195,5351	2518,4	0,15182971
COMB4	HEM500	523,394	7667	58,000	2518,4	0,0571634
	HEM600	903,351	8099	113,4567	2518,4	0,1008204
COMB4-	HEM500	632,721	7667	87,200	2518,4	0,07588778
	HEM600	903,804	8099	116,2799	2518,4	0,10196939
COMB5	HEM500	603,201	7667	129,620	2518,4	0,09080667
	HEM600	1119,626	8099	235,5041	2518,4	0,16263463
COMB5-	HEM500	826,699	7667	170,2346	2518,4	0,12150914
	HEM600	1120,544	8099	238,7808	2518,4	0,16399241
COMB6	HEM500	270,501	7667	130,171	2518,4	0,06932858
	HEM600	588,64	8099	221,9714	2518,4	0,12448014
COMB6-	HEM500	481,417	7667	151,6155	2518,4	0,0915985
	HEM600	589,513	8099	223,6758	2518,4	0,12521081
COMB7	HEM500	277,728	7667	508,927	2518,4	0,22019537
	HEM600	1220,459	8099	840,487	2518,4	0,40908476
COMB7-	**HEM500**	**1282,12**	**7667**	**610,300**	**2518,4**	**0,325949**
	HEM600	1224,727	8099	843,963	2518,4	0,41072849
COMB8	HEM500	258,516	7667	474,4342	2518,4	0,20524616
	HEM600	316,785	8099	760,2453	2518,4	0,32143335
COMB8-	HEM500	665,362	7667	485,21	2518,4	0,23605727
	HEM600	320,747	8099	761,1284	2518,4	0,32202861
					P-M max	**0,41072849**

Beam Strength

Beam strength covers only the flexural strength since beams do not carry axial loads. Design starts with calculating plastic moment capacity of the section (HE500A).

$$M_p = Z_x F_y$$

$$M_p = 3949\ cm^3 \times 355\ MPa = \mathbf{1401.9}\ kNm$$

And continues with calculating the design capacity which is the LRFD factored plastic moment capacity:

$$M_C = \emptyset b \times M_p = 0.9 \times 1401.9\ kNm = \mathbf{1261.7\ kNm}$$

And finally checking if capacity is bigger than the demand:

$$M_C = \mathbf{1261.7\ kNm} > M_r = \mathbf{362.2\ kNm}$$

thus, the sections are adequate Table 16.

Table 16 : Beams Flexural Strength Adequateness

Story	Section	Zx (cm3)	Mp (kNm)	Mc (kNm)	Mr (kNm)	Safety
Roof	HE240B	1053	373,815	336,4335	54,9	Safe
5	**HE500A**	**3949**	**1401,9**	**1261,7055**	**362,2**	**Safe**
4	HE550A	4622	1640,81	1476,729	468,13	Safe
3			1640,81	1476,729	515,74	Safe
2	HE600B	6425	2280,875	2052,7875	648,7	Safe
1			2280,875	2052,7875	248,3	Safe

5. RESULTS

Story	Exterior Column	Interior Column	Girders
Roof	HE340M	HE400M	HE240B
5			HE500A
4	HE400M	HE500M	HE550A
3			
2	HE500M	HE600M	HE600B
1			

6. CONCLUSIONS

Starting with the drift design enable the project to move forward without taking previous iteration steps simply by, while satisfying requirements of the drift design. This closely satisfied drift design ends up as overstrength profiles at strength design, making no iterative step.

6.1. Life-Long Learning

Additional knowledge, skills, and attitudes that would be appropriate for professional practice I think is that, as technology is getting better day by day, so does the methods for solving problems. With this incrementation and participation from the technology, structure analyses software are becoming an essential tool of a structural engineer and in order to keeping up with the changing-developing world, the one who wants to be a structural engineer must learn the currently developed goods and services, their attributes, advantages and disadvantages to find a way to implement this goods and services to civil engineering topics to find more sustainable, more reliable solutions for the past, present and future problems. As the internet becoming an open source for all relevant knowledge about any specific topic, civil engineers have no excuse for their self-development about their professions.

6.2. Professional and Ethical Responsibilities of Engineers

As a civil engineer candidate and a future member of the Turkish Civil Engineering Chamber, this project proceeded under the standards and regulations of the Turkish Republic. Being under the laws of the government, project executed with academic honesty with under the obligatory of the chamber. But not only laws and obligations are the ones that draw the path of our responsibilities. Our duty on public interest, to work for the safety of human life and making every-day life better also both restrict and encourage us to do our profession. Doing one of the oldest profession, we should not forget our motto:

“*Utilitas, Firmitas, Venustas.”*

-Vitruvius, De Architectura

6.3. Contemporary Issues/Future of Industry

One of the challenges I faced on this project was the numerous repeated calculations and the time lost to do these calculations. So, in order to keeping up with the challenges and take the glory lap, I took the aid of the computer science and coded algorithms to solve repeated calculations and this allow me to design faster and efficiently. I believe we, all civil engineers, need to learn the art of efficiency with computer programing and coding.

6.4. Teamwork

Throughout this project, we, I and my friends who shared the same design topic with me, supported and guided each other whenever one of us stumbled upon the project. This support had led us to learn that completing each other weaknesses makes the total effort more efficient.

APPENDIX A

```
function RBS_SI
i=1;
bbf =input('Please enter the beam b value in mm: ');
tbf =input('Please enter the beam flange thickness value in mm: ');
db  =input('Please enter the beam depth value in mm:  ');
Zx  =input('Please enter the plastic moment capacity value in cm^3: ');
Zx  =Zx*1000;
Zb  =Zx;
beam_weight=input('Please enter the weight value in kg/m: ');
tcf_left=input('Please enter the left column flange thickness value in mm: ');
dc_left =input('Please enter the left column depth value in mm: ');
hcl     =tcf_left+dc_left/2;
tcf_right=input('Please enter the right column flange thickness value in mm: ');
dc_right =input('Please enter the right column depth value in mm: ');
hcr      =tcf_right+dc_right/2;
Fy = input('Please enter the Yield Strength , Fy , value in MPa: ');
Fu = input('Please enter the Ultimate Strength , Fu , value in MPa: ');
Ry = input('Please enter the Ry coefficient: ');
WF_dead= input('Please enter the Dead load value in kN/m: ' );
WF_live= input('Please enter the Live load value in kN/m: ' );
L = input('Please enter the span length value in mm: ');
Clean_Span =(L-hcl-hcr);
span_ratio = Clean_Span/db;
while i==1
     if db < 1040
    i=0;
    else
    i=1;
    disp('beam depth exceeded 1040mm')
```

```
        db=input('Please enter a beam depth lower than 1040mm:  ');
        end
end
while i==0
        if beam_weight < 447
        i=1;
        else
        i=0;
        disp('beam depth exceeded 447kg/m')
        beam_weight=input('Please enter a beam weight lower than 447kg/m:  ');
        end
end
while i==1
        if tbf < 44
        i=0;
        else
        i=1;
        disp('beam flange thickness exceeded 44mm')
        tbf=input('Please enter a beam flange thickness lower than 44m:  ');
        end
end
while i==0
        if span_ratio >= 7
        i=1;
        else
        i=0;
        disp('span length over beam depth ratio is lower than 7')
        end
end
while i==1
```

```
        if dc_left < 1040
        i=0;
        else
        i=1;
         disp('beam depth exceeded 1040mm')
         end
    end
    while i==0
        if dc_right < 1040
        i=1;
        else
        i=0;
        disp('beam depth exceeded 1040mm')
        end
    end
    fprintf('No limitation on column weight and column flange thickness. Thus, the RBS can be used.')
    a = (0.5+0.75)*bbf/2;
    b = (0.65+0.85)*db/2;
    c = (0.1+0.25)*bbf/2;
    a = round(a);
    b = round(b);
    c = round(c);
    Z_rbs = Zx - (2*c*tbf)*(db-tbf);
    Cpr = (Fy+Fu)/(2*Fy)
      if Cpr > 1.2
        Cpr=1.2;
      end
    Mpr = Cpr*Ry*Fy*Z_rbs/1000000;
    w  = 1.2*WF_dead+ 0.5*WF_live;
```

```
Sh = a + b/2;
Lh = L -(2*Sh+dc_left/2+dc_right/2);
Sh=Sh/1000;
Lh=Lh/1000;
Vrbs_plus  = (2*Mpr/Lh)+(w*Lh/2);
Vrbs_minus = (2*Mpr/Lh)-(w*Lh/2);
Vrbs = max (Vrbs_plus, Vrbs_minus);
Mf  = Mpr + Vrbs*Sh;
Mpe = Ry*Fy*Zb;
Mpe = Mpe/1000000;
factor = 1;
Mfinal = factor*Mpe;
if Mf < Mfinal
  fprintf('\nMf is smaller than MFinal thus the process completed,     \n \n     \n a: %.0fmm    b: %.0fmm   c: %.0fmm Zrbs: %.0fmm^3 Cpr: %.2f \n \n Mpr: %.3fkNm Sh: %.3fm   Lh:   %.3fm   Vrbs:   %.3fkN   \n   \n   Mf:   %.3fkNm   Mfinal:   %.3fkNm', a,b,c,Z_rbs,Cpr,Mpr,Sh,Lh,Vrbs,Mf,Mfinal);

end
rbs_infunction
   function rbs_infunction
 j=1;
while j==1
Z_rbs = Zx - (2*c*tbf)*(db-tbf);
Cpr = (Fy+Fu)/(2*Fy);
   if Cpr > 1.2
      Cpr=1.2;
   end
Mpr = Cpr*Ry*Fy*Z_rbs/1000000
w  = 1.2*WF_dead+ 0.5*WF_live;
Sh = a + b/2;
Lh = L -(2*Sh+dc_left/2+dc_right/2);
```

```
Sh=Sh/1000;
Lh=Lh/1000;
Vrbs_plus  = (2*Mpr/Lh)+(w*Lh/2);
Vrbs_minus = (2*Mpr/Lh)-(w*Lh/2);
Vrbs = max (Vrbs_plus, Vrbs_minus);
Mf  = Mpr + Vrbs*Sh;
Mpe = Ry*Fy*Zb;
Mpe = Mpe/1000000;
factor = 1;
Mfinal = factor*Mpe;
if Mf < Mfinal

  fprintf('\nMf is smaller than MFinal thus the process completed,     \n \n     \n a: %.0fmm    b: %.0fmm   c: %.0fmm Zrbs: %.0fmm^3 Cpr: %.2f \n \n Mpr: %.3fkNm Sh: %.3fm Lh: %.3fm Vrbs: %.3fkN \n \n Mf: %.3fkNm Mfinal: %.3fkNm', a,b,c,Z_rbs,Cpr,Mpr,Sh,Lh,Vrbs,Mf,Mfinal);

  j=0
else
  c=c+1;
  j=1;
end
c_control = bbf/4;
if c > c_control
   disp(' c value exceeded, RBS cannot apply')
   j=0;
end
end
end
end
```

APPENDIX B

Combination	Story	First-order drift (cm)	Second-order drift (cm)	Δ2nd/Δ1st ratio
1	Roof	0,062616	0,066265	1,0583
	5	0,081272	0,086929	1,0696
	4	0,107856	0,117126	1,0859
	3	0,134955	0,148111	1,0975
	2	0,122391	0,13408	1,0955
1-	Roof	-0,063691	-0,06739	1,0581
	5	-0,082361	-0,088078	1,0694
	4	-0,108593	-0,117922	1,0859
	3	-0,135544	-0,148756	1,0975
	2	-0,122689	-0,134411	1,0955
2	Roof	0,086956	0,094936	1,0918
	5	0,122112	0,135397	1,1088
	4	0,166947	0,189719	1,1364
	3	0,211164	0,244169	1,1563
	2	0,1925	0,222021	1,1534
2-	Roof	-0,088425	-0,096507	1,0914
	5	-0,12368	-0,137093	1,1084
	4	-0,167994	-0,1909	1,1364
	3	-0,212003	-0,245138	1,1563
	2	-0,192925	-0,222524	1,1534
3	Roof	0,125979	0,140235	1,1132
	5	0,154248	0,172076	1,1156
	4	0,199325	0,225945	1,1336
	3	0,246965	0,283432	1,1477
	2	0,22286	0,254537	1,1421
3-	Roof	-0,127862	-0,142276	1,1127
	5	-0,155476	-0,173431	1,1155
	4	-0,200265	-0,227002	1,1335
	3	-0,247687	-0,28426	1,1477
	2	-0,223228	-0,254968	1,1422
4	Roof	0,079422	0,086834	1,0933
	5	0,096223	0,104508	1,0861
	4	0,12358	0,135123	1,0934
	3	0,152798	0,168118	1,1003
	2	0,13774	0,150874	1,0954
4-	Roof	-0,081114	-0,088642	1,0928
	5	-0,09703	-0,105387	1,0861
	4	-0,124272	-0,135875	1,0934

	3	-0,153312	-0,168684	1,1003
	2	-0,138004	-0,151168	1,0954
5	Roof	0,159859	0,172014	1,0760
	5	0,192595	0,210575	1,0934
	4	0,247403	0,276316	1,1169
	3	0,305753	0,34655	1,1334
	2	0,275562	0,311578	1,1307
5-	Roof	-0,161213	-0,173452	1,0759
	5	-0,19391	-0,211987	1,0932
	4	-0,248302	-0,277313	1,1168
	3	-0,306468	-0,347358	1,1334
	2	-0,275925	-0,311996	1,1307
6	Roof	0,160191	0,165905	1,0357
	5	0,192902	0,201442	1,0443
	4	0,247615	0,261274	1,0552
	3	0,305921	0,325103	1,0627
	2	0,275648	0,292599	1,0615
6-	Roof	-0,160882	-0,166617	1,0356
	5	-0,193603	-0,202167	1,0442
	4	-0,248089	-0,261772	1,0552
	3	-0,3063	-0,325504	1,0627
	2	-0,275839	-0,292804	1,0615
7	Roof	1,174695	1,243265	1,0584
	5	1,162528	1,260718	1,0845
	4	1,102692	1,23787	1,1226
	3	1,193129	1,36674	1,1455
	2	0,977752	1,121847	1,1474
7-	Roof	-1,175972	-1,244624	1,0584
	5	-1,164049	-1,262351	1,0844
	4	-1,103684	-1,238982	1,1226
	3	-1,19393	-1,367644	1,1455
	2	-0,978156	-1,122299	1,1474
8	Roof	1,175146	1,195056	1,0169
	5	1,163099	1,188648	1,0220
	4	1,10306	1,13642	1,0302
	3	1,193427	1,235333	1,0351
	2	0,977902	1,012382	1,0353
8-	Roof	-1,17552	-1,195438	1,0169
	5	-1,163478	-1,189035	1,0220
	4	-1,103316	-1,136687	1,0302
	3	-1,193632	-1,235535	1,0351
	2	-0,978006	-1,012472	1,0352
			Max Ratio	**1,1563**

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