ISTANBUL TECHNICAL UNIVERSITY ★GRADUATE SCHOOL OF EARTHQUAKE ENGINEERING AND DISASTER MANAGEMENT INSTITUTE

PERFORMANCE EVALUATION OF EAVE CONNECTION TYPES FOR COLD FORMED PORTAL FRAMES

M.Sc. THESIS

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SOĞUKTA ŞEKİL VERİLMİŞ PORTAL ÇERÇEVE DEEAVE BAĞLANTI TÜRLERİNİN PERFORMANS DEĞERLENDİRMESİ

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FORWARD

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ABBREVIATIONS

CFS = Cold Formed Steel Section

LGS = Light Gauge Steel

RC = Concrete Structure

HRS = Hot Rolled Steel Structure

LRFD = Load and Resistance Factor Design Method

FR = Fully Restrained Connection

PR = Partially Restrained Connection

SYMBOLS

C = Chemical symbol of Carbon element

Mn = Chemical symbol of Magnesium element

P = Chemical symbol of Phosphorus element

S = Chemical symbol of Sulfur element

F_{ye} = Expected yield stress of steel material

F_{ue} = Expected tensile stress of steel material

R_y = The ratio of the possible yield stress of steel material to the characteristic yield stress of steel material

R_t = The ratio of the possible tensile strength of steel material to the characteristic tensile strength of steel material

F_u = Minimum tensile stress (Minimum ultimate stress) of steel material

F_y = Minimum yield stress of steel material

Y = Weight of one unit volume of material

P_k = Snow load

P_{k0} = Self-snow load

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PERFORMANCE EVALUATION OF EAVE CONNECTION TYPES FOR COLD FORMED PORTAL FRAMES

SUMMARY

One of the most important connection in the structure is the beam to column connection. The behavior of beam to column connections intensively affect the overall structural performance of structure.

There are so many parameters that influence the connection characteristics and any changing of the connection characteristic will change the behavior of the structure under loadings too.

This thesis includes 6 sections:

In the first section; Introduction that describes the main idea of the thesis, the purpose, the scope and the methodology of this study.

In the second section, all required information for both steel and cold formed steel as historical development, manufacturing methods, fabrications methods and the material properties are mentioned. Also the steel material proporties changing by cold forming and the utilization of cold formed steel sections are explained.

In the third section, a industrial structure which constructed with cold formed steel sections is modeled with SAP2000 V20 program. The strength of cold formed steel sectionsare controlled in CFS V8.0 too and the analysis results of model is presented.

In the fourth section, the internal forces which are created in the beam -column joints are determined.3 different types of beam-column connection are designed manually and also these connections are modeled with IDEA StatiCa V10 program.

In the fifth section, The stiffness and strength analysis results of the 3 types of modeled connection are presented and the characteristics of considered connections are compared with each other.

And in the last part, the conclusion of this study is provided.

SOĞUKTA ŞEKİL VERİLMİŞ PORTAL ÇERÇEVE DEEAVE BAĞLANTI TÜRLERİNİN PERFORMANS DEĞERLENDİRMESİ

ÖZET

Yapıdaki en önemli bağlantılardan biri kiriş-kolon bağlantısı dır. Kiriş-kolon bağlantılarının davranışı, yapının genel yapısal performansını yoğun olarak etkiler.

Bağlantı özelliklerini etkileyen pek çok parametreler vardır ve bağlantı karakteristiğindeki herhangi bir değişiklik yapının yükler altındaki davranışını da değiştirecektir.

Bu tez 6 bölümden oluşmaktadır:

İlk bölümde; Tezin ana fikrini, çalışmanın amacını, kapsamını ve yapım yöntemini açıklamaktadır.

İkinci bölümde, çelik ve soğuk şekillendirilmiş çelik için gerekli tüm bilgiler, üretim yöntemleri, tarihi gelişmeleri ve malzeme özellikleri belirtilmiştir. Ayrıca soğuk şekillendirme ile değişen çelik malzeme özellikleri ve soğuk şekillendirilmiş çelik bölümlerin kullanımı açıklanmiştir.

Üçüncü bölümde, soğuk şekillendirilmiş çelik bölümler ile inşa edilmiş bir endüstriyel yapı SAP2000 V20 programı ile modellenmiştir. Soğuk şekillendirilmiş çelik bölümlerin dayanımı CFS V8.0 programı ile de kontrol edilmiş ve modelin analiz sonuçları sunulmuştur.

Dördüncü bölümde, kiriş-kolon bağlantılarında oluşan iç kuvvetler belirlenmiştir.3 farklı kiriş-kolon bağlantı tipleri manuel olarak tasarlanmıştır ve bu bağlantılar IDEA StatiCa V10 programı ile modellenmiştir.

Beşinci bölümde, 3 tip modelenmiş bağlantıların rijitlik ve dayanım analiz sonuçları gösterilmiş ve bağlantıların özellikleri birbirleriyle karşılaştırılmıştır.

Ve son bölümde ise bu çalışmanın sonucu verilmiştir.

1. INTRODUCTION

Todays, there are two types of steel load bearing elements which are used in steel constructions. The first and the most common group are hot rolled steel sections and the second group is the cold formed steel sections.

Cold formed steel sections are structural materials which are produced by forming steel flat sheets at a room temperature into different shapes and they can be used to provide functional and structural requirements as the secondary or the aprimary members of frame.Over the past two decades, the usage of cold formed steel sections as the main load bearing membershave been increased due to their inherent features [48].

Some inherent features of cold formed steel sections which makes them superior over other materials could be categorized into 3 groups [14]:

- In terms of architecture

Some important architectural benefits of cold formed steel sections are:

  - Lightness
  - Applicable in free forms
  - Applicable in large openings
  - Space savings
  - Flexibility

- In terms of structural system

Some important structural benefits of cold formed steel sections are:

  - Better performance in earthquake
  - High quality and controllability of production due to manufacturing in a factory environment.

- In terms of application

Some important application benefits of cold formed steel sections are:

- Reduction of construction time
- Reduction of foundation cost
- Demountable
- Easily adapt to change

So as it is expected the cold formed steel structures are the large part of structure types in the world especially in the industry type of structures because large free space which are required in this type of structure can be supplied by light cold formed steel sections economically,therefore, it is necessary to understand the characteristics and the performance of these structures and the factors which affect the performance of these structures.

One of the most important factor which influence the performance of the structure is the beam to column connection type. The columns and beams are connected together and form the load bearing steel frames of structure [51]. The behavior of thisconnection affect on the distribution of internal moments and forces within a frame and structure, and on the overall structure deformations so an appropriate design of this connection leads to have economic and high quality structure.

It is obvious that recognising the parameters which determine the connection behavior is the first step in the proper designing of column - beam connection. There are a lot of parameters that influence the connection characteristics. One of the most important of them are the localised deformation of column [85].The flat plates which are called column stiffener are used to stiffen the column against the localised deformation of column [59]. The stiffener type are defined by the stiffener application location. Depending on the stiffener type the specific type of localised deformation of column are restricted.The type and the amount of localised deformation of the column will vary with the application of different types of stiffeners and by the following of this the connection characteristics will be varied too.

It is noteworthy to mention that connection designing of cold formed steel members are quite different from those of hot rolled steel membersdue to their higher strength and the using of thin sheets of cold formed steel members [46].

1.1 Purpose of Study

The purpose of this study is to evaluate the effect of the column stiffener types on the characteristics of beam-column connection in industrial cold formed steel structure which plays an important role on the overall performance of structure and also to determine the optimum column stiffener to have more appropriate connection which leads to have a industrial structure with including all the advantages of the cold formed steel sections.

1.2 Scope of Study

The process of cold forming, the characteristic of cold formed steel member and also the advantages and disadvantages of cold formed steel members are explained.

The industrial cold formed steel structure is designed and the internal forces which are created in the beam-column joint due to the loading are determined.

The 3 different types of beam-column connection are designed and they are analyzed under the created internal forces.3 types of connections are different from each other according to the type of used column stiffeners.

The analysis results of connections are compared with each other according to the characteristics of connection and the optimum design of beam-column connection is determined.

1.3 Methodology

All required theoretical information was gathered by literature review. The information that gained via researching on the standards and the guidelines, national and international paper, reference books and conferences and the experts are used in this study.

The methodology of case studies evaluating is as below:

Selecting case studies:3 different types of beam-column connection are designed manually. These connections are different with each other according to the types of column stiffeners. These 3 cases of study are lised as below:

- Beam-column connection with full depth horizontal column stiffener
- Beam-column connection with supplementary web plate column stiffener
- Beam-column connection with K diagonal column stiffener

Evaluation method: The 3 types of connection are modeled withIDEA StatiCa V10 program by finite element method.

Application IDEA Statica V10 connection can design all types of bolted or welded connections, footing, base plates and anchoring. It provides accurate checks, results of stiffness, strength and buckling analysis of steel joints.

The models are analysed according to their strength and their stiffness and the results of analysis are compared with each other.

The required design forces are taken from analysed considered industrial model by SAP2000 V20 program.

SAP 2000 V20 is a finite element program which performs the dynamic or static, nonlinear or linear analysis of structural systems.

In addition, the strength of cold-formed steel sections which are used in considered industrial model are evaluated in CFS V8 program.

CFS V8 program is a comprehensive cold-formed component design tool that performs calculations according to the AISI S100 code.

2. GENERAL INFORMATION OF COLD FORMED STEEL SECTIONS

The purpose of this section is to prepare a suitable apprehension of the cold-formed steel's behavior and the cold formed technology development and also provide the brief information about the performance and the properties of the cold formed steel sections.

2.1 Steel

Steel is one of the most significant material in the world. It is used in every sight of our lives, in industry and construction. The production of steel are growing rapidly so that the world raw steel production reached 1,808.6 million tons in the year 2018[1], and the Turkey is in the eighth step of the world ranking in this field [2].

2.1.1 History of steel development

The use of iron has been strated at about 4000 BC. It has been proven that iron rods are used to assemble large stones in the pyramids of Egypt and in China, Shen Nung reaches iron for the first time in 2700 BC. For these reasons, the period of the mine, which was one of the prehistoric times, is also called the Iron Age.

Iron and its alloys which were hardened by methods such as forging and quenching had been widely used just as a complementary, auxiliary or ornamental element, such as clamps, straps or railings [3].

Over the time with the development of technology which was started with the industrial revolution, iron-carbon steel material was obtained by adding iron alloying elements. In the 1850s, Bessemer supplied the hot air to the cast iron for about 20 minutes to burn off the impurities and in this way he could converted the iron to steel. [5].Later open-pit method which is the another economic steel production method was developed in Europe in 1868 and a little later it is started to be used in America [4]. After that the steel was replaced by cast iron due to its strength, flexibility and elongation properties.

After the Second World War, in the second half of the 20th century,there were large amount of remained steel that steel manufacturers lead these capacities to construction sector which caused to that steel structures gained importance.

The first initiatives for the establishment of the iron and steel industry in Turkey were started in 1925[2] and after the 1999 Marmara Earthquake, because of the more better performance of steel structure in the earthquake, the importance and necessity of steel structures were to be emphasized, and the construction of steel structure have been increased in Turkey.

The first metal bearing structure is the 31 meters length cast iron bridge which was built on the Severn River in the UK in 1779[3].

Figure 2.1: Crossing the river Severn, Coalbrookdale, Ironbridge town, Shropshire County, England [9].

In the following of the progress in steel producing, the first steel bearing structure in the world was constructed. It was Crystal Palace which was built in London in 1851 by Joseph Paxton. This structure had 70.000 m² area that was made of glass and steel. It was built in 4 months that it is very short time according to the conditions of that time but it was destroyed in 1936[6].

Figure 2.2: Crystal Palace, Sydenham Hill, London [10].

One of the oldest steel structure in Turkey is Antalya Cam Piramit which was built in Antalya in 1997. This structure has approximately 4500 m^2 area and the roof and the walls of structure are covered by glass completely [7].

Figure 2.3: Antalya Cam Piramit, Antalya, Turkey [11].

2.1.2 Manufacture of steel

Raw iron consist of carbon, phosphorus, silicon, sulphur and some other ingredients. Due to high amount of these substances the raw iron can not be shaped with neither by rolling nor a hammer. In order to provide workability of raw iron, the amounts of these components especially carbon must be reduced.

For steel manufacturing, these substances are connected with basic slag by adding the lime in a method called heat treating. As a result of the chemical reaction that occurs during this method, the carbon elements are connected to oxygen elements and the carbon monoxide are formed so the carbon amount decreases and a large part of the formed carbon monoxide (CO) flies as a gas. The required oxygen for this method is supplied in two ways, from the air, by the blowing of the pure oxygen or from the oxygen which is connected to iron [5].

The amount of increased oxygen and oxygen compounds is reduced each time by adding substances such as aluminum and silicon and by this way, the oxygen present in the composition is dissolved and released [5].

In steel alloy, the carbon content is about %0.16~0.20, as the amount of carbon increases, both the strength and the hardness of the steel increase. The carbon content in the best combination is less than %1.2[13]. If the carbon content in steel alloy is greater than 1.7%, it is difficult to process the steel. This steel alloy is called ferrous metal.

The sulphur are removed from raw iron by adding soda, magnesium or calcium carbide. For the formation of manganese sulphides, which have a significant effect on the hardness of the steel, low levels of sulphur (0.02% - 0.05%) are required.

Phosphorus make the steel to become brittle. The steel alloy containing 0.2% phosphorous is brittle as glass.

Chromium, manganese and silicon increase the strength of the steel [5].

Vanadium and chromium make steel resistant to fire at high temperatures and increase the corrosion resistance of steel [14].

In steel manufacturing, two kinds of casting method is used [5]:

- Die Casting
- Strand Casting

In Die Casting method, molten steel is poured to certain molds from above and fill the entire molds and then they are let to cool and solidify in the molds.

Figure 2.4: Die casting method of steel manufacturing [5].

In Strand Casting method, the melt steel is transferred to a cooled copper mold and then leaved to solidify up to it becomes a semi-finished slab. In this process molten steel is pouring continuously[8].

Figure 2.5: Strand casting method of steel manufacturing [12].

2.1.3 Properties of steel

Steel properties change by changing its ingredients and the combination percentages of these ingredients.

Material properties play an important role in the behavior of structures so that understanding of steel properties could be helpful in having optimum design of structure. In this way the properties of steel are explained into 2 groups:

- Physical Properties

Some important physical properties of steel are listed as below:

  - Specific weight (γ): The specific weight of the steel varies according to the carbon content. The specific weight of hard steel is 7.89 gr/cm^3, and the normal steel is 7.85 gr/cm^3[16].
  - Melting temperature: Melting temperature is the amount of heat that have to be supplied to fluidize the solid. The melting temperature of the steel is 1400C. This is an important features for fire resistant steel [16].
  - Thermal conductivity: Thermal conductivity of steel is 35 W / mC. The smaller this coefficient, the smaller the heat loss and the better heat resistance [16].
  - Thermal expansion: It is the increase or decrease in the volume of an object with the effect of temperature difference. Thermal expansion of steel is 11 x $10^{-6}C^{-1}$ [16].

The physical properties comparison of steel, hard steel and iron are given in Table 2.1.

Table 2.1: Comparison of physical properties for steel, hard steel and iron [3].

	Carbon Component (%)	γ (gr/cm^3)	Melting Temperature (0^c)	Thermal Conductivity W / m ^{0C}	Thermal Expansion (x 10^{-6} 0^{c-1})
Iron	0.2	7.8	1530	61 – 50	12
Steel	0.3 –0.6	7.85	1400	35	15.1
Hard Steel	0.6 – 1.7	7.89	1300	40	15

- Mechanical Properties

The mechanical properties of the steel are determined by means of the stress-strain diagram which is obtained by the tensile test. The tensile force which is applied to the steel, increased from zero to the point where the sample will be broken, the resulting deformations are measured during the test and then by drawing the applied load versus resulting deformations in a chart, stress-strain diagram is obtained. The stress-strain diagram of steel is shown in Figure 2.6 [16].

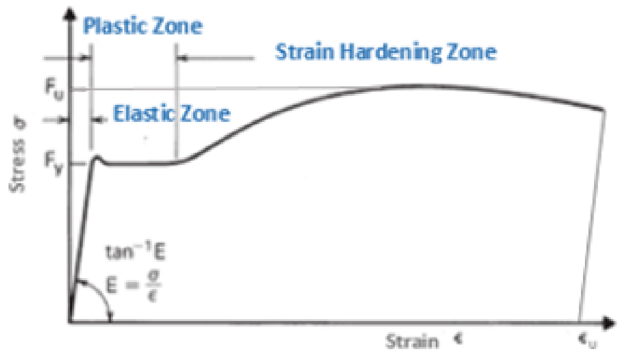


Figure 2.6: Stress-Strain diagram of mild steel [17].

According to the stress-strain diagram of steel some important mechanical properties of steel are as following:

- Elasticity module (E): The elasticity moduleis equal to the slope of the curve in the elastic region, and it shows the stiffness of material. Stiffness relates to how a material deflect under load while returning to its original shape once the load is removed. The stiffer the material is, the larger elasticity module [18]. The elasticity modulus of wood is 10200-16200 N/mm^2, the normal concrete BS25 is 30250 N/mm^2, the high strength concrete BS50 is 36950 N/mm^2, and the steel is 200000 N/mm^2. This shows clearly that the steel can deflect more elastically and return to its original shape under higher loads[19].
- Ultimate strength (F_u): Ultimate strength is the maximum stress that a steel can resist while being stretched before breaking.
- Breaking elongation (ε_u): Elongation at break, also called as fracture strain, is the initial length to the changed length ratio after breakage of steel [20].
- Ductility: The plastic deformation ability of the material after the yielding and before fracturing is called ductility, and it is about 100 times of elongation at break [21].

The mechanical properties comparison of steel, hard steel and iron is given in Table 2.2.

Table 2.2: Comparison of mechanical properties for steel, hard steel and iron [3].

	E (N/mm^2)[1]	γ (gr/cm^3)	F_u (N/mm^2)	Breaking Elongation (ε_u)[2]
Iron	200000	7.8	300 – 350	0.12 – 0.15
Steel	200000	7.85	400 - 480	0.25 – 0.28
Hard Steel	200000	7.89	650 - 750	0.14 – 0.18

1) Elasticity module is about constant. It is independent of processing and the carbon ingredient amount

2) Breaking elongation is unit-less.

2.1.4 Fabrication of steel

Once the steel with required properties has been selected, the next step is to form it into desired shape. There are two main methods in the fabrication process of steel [13]:

- Hot Forming

Hot forming process usually are performed on too thick or too hard steel. Hot forming process is done in 3 waysas followings [13]:

  - Forging
  - High temperature forming (Electroforming)
  - High energy rate forming

- Cold Forming

Cold forming process is not very common techniques but it is in growing rapidly. Cold forming process is explained in details in Sec 2.2.2

2.2 Cold Formed Steel

Cold formed steel sections (CFS) are called as Light steel or as Light gauge steel (or LGS) too.

Owing to the lightness and high structural performance of cold formed steel sections (CFS), the consumption of these sections has become more common. However formerly, these sections were used as secondary members in steel and concrete structures, but they are used as the main structural members nowadays [15].

Some comparison results are given briefly in Table 2.3which could be helpful in understanding of cold formed steel section effects on a construction project in general [15].

Table 2.3: Construction comparison results between cold formed steel structures and hot rolled steel structures as well as concrete structures [15].

Structure Type	Material Cost / Material Cost of CFS	Construction Cost / Construction Cost of CFS	Total Building Cost / Total Building Cost of CFS[1]	Construction Duration
Concrete (RC)	%134	%185	%161	%264
Hot Rolled Steel (HRS)	%189	%85	%135	%138

1) Total building costs = material cost + construction cost

As it can be seen, the construction cost of cold formed steel structures are %15 more expensive than the hot rolled steel structures. The main reason of this expensiveness is the foundation cost. Mat foundation is used in cold formed steel structure construction which is more expensive than pan footing that is used for both concrete and hot rolled steel structure constructions.

Cold formed steel structures are only 0.5% of the steel structures in Turkey whereas this rate reaches 25% in the United States.

2.2.1 History of cold formed steel development

Although many think cold rolling process is to be a new fabrication technology but in fact it has been existedin the North America for about 100 years [24]. In 1920s and 1930s cold formed steel was accepted as a construction material but no control or design criteria were determined for the cold rolling section until 1946 [23] whereas the design criteria of hot-rolling steel were standardized in the United States in the 1930s.

In 1946,as a result of the study ofProfessor George Winter in Cornell UniversityAISI (American Iron and Steel Institute) regulation which explained the strength design features for the first time was published. In 1991, Wei-Wen Yu and Theodore V Galambos published load and strength factor design principles. In 1996, two regulations were combined and the current regulation was formed and it has been revised continuously in the following years.

The development of the light steel in construction industry began in European countries at the end of World War Ⅱ (1945).

One of the first uses of cold formed steel sections as a building member is the Virginia Baptist Hospital which was constructed in 1925 in Lynchburg, Virginia. The floor system of this building was constructed with double back-to-back cold-formed lipped channels and the walls were constructed with load bearing masonry.

Figure 2.7: Cold formed steel floor system, Virginia Baptist Hospital, Lynchburg, Virginia [25].

It is seen that the production of light steel structures in Turkey does not exceed 40 years. In the past, there is no specific regulation which explained the cold formed steel structure design method so this restrict the use of cold formed steel sections in structures but this is going to be improved because the specific section for designing of the cold formed steel section is assigned in new published regulation, 'Türkiye Bina Deprem Yönetmeliği (TBDY 2018)' [34].

2.2.2 Fabrication of cold formed steel

The forming which is done under the recrystallization temperature of the material is called cold rolling [5].

A recrystallization temperature of material is the temperature at which low dislocation density new grains begin to replace with the high dislocation density grains [37].

Cold forming process is done at room temperature. It is performed on flat bar, tubular, sheets, strips or plates products and it includes shaping and bending the steel beyond its yield point as a result permanent shape is obtained. Generally, the thickness of steel sheets which are used in cold forming process is in the 0.5 – 4 mm range and the thickness of cold formed profiles is in 1 – 8 mm range [22].

There are 3 ways for fabrication of cold formed steel section [22]:

- Continuous process
  - Roll forming
- Discontinuous process
  - Press braking
  - Wipe bending

Two important criteria which determine that which of the mentioned method is suitable are the cost and the desired cross sectional shape. If large quantity and fast production is required or if the cross sectional shape is complex, the roll forming method is recommended [27].

2.2.2.1 Roll forming

In this method which is continuous processing method, the materials (Flat bars, sheets, strips, etc.) are passed through sets of rolls and each of sets works on the material until the intended shape is obtained. Each of these roll sets are called station.

The positioning of the cylinders which is used in the shaping process and the number of required stations are depend on the desired section shape. The more complex cross-sectional shape needs more stations to obtain the desired shape [28].A simple section may be formed in six pairs of roll but complex shape may require up to 30 pairs of roll.

This method has three distinct features [5]:

- The production speed is high. The speed of this process is in 6 – 92 m/min range. The length of the profile is usually 6 meters or 12 meters [27].
- pre-galvanized or painted sheet metal elements can be rolled without any damage to paint or galvanization [30].
- Any panels and trapezoidal sheeting and any cross section can be produced by this method.

Material with 0.1 – 0.79 mm thickness can be formed easily by this method [26]. It is worth mentioning that the process is done at room temperature.

The roll forming process is shown in Figure 2.8 schematically.

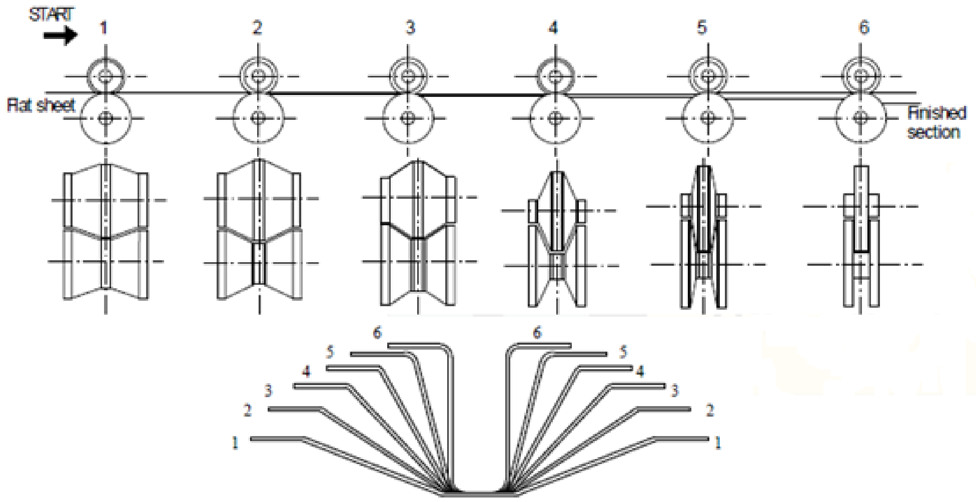


Figure 2.8: Roll forming process of cold forming method [29].

2.2.2.2 Press braking

In this method, which is discontinuous processing method, steel product is formed on a press brake by compressing it between a die block and a punch in one or more consecutive operations, and each corner in a cross section is created with a one stroke of press.

It is a manual controlled with limited production capacity process. This method is mainly used in the production of short-length elements, about 3 meters [27].

This method can be used in the following cases [28]:

- The section to be produced has a simple shape.
- The section to be produced is relatively wide section.
- The required production speed is constant and lower than 91.5 m/min.

The press barking method is not used for the production of trapezoidal sheeting or composite flooring sheets because the required press area should to be very large therefore very large power is required [5].

As shown in Figure 2.9, the bending process develops from the center to the outside. The curvature is not constant along the corner. Curvature at the center of the corner is greatest [31].

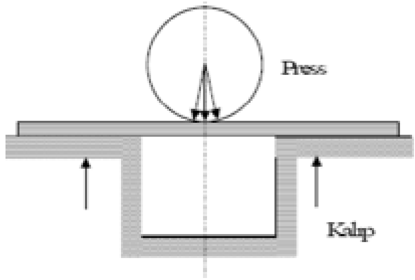


Figure 2.9: The shape forming by press [31].

The press braking process is shown in Figure 2.10 schematically.

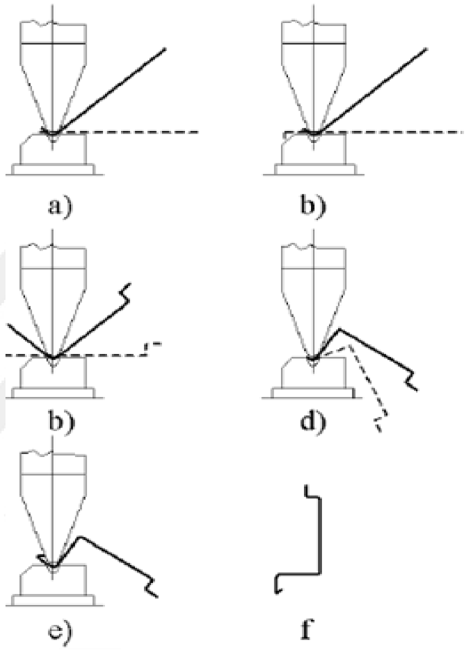


Figure 2.10: Press braking process of cold forming method [29].

2.2.2.3 Wipe bending

Wipe bending is also called edge bending. In this method, the sheet is held against the wipe die with a pressure pad. Then the punch presses against the edge of the steel sheet that spread beyond the die and pad. The sheet will curve against the radius of the wipe die edge [32].

This method is not very common and have some restrictions for example the width of cross section flange has to be equal to at least 4 times of the thickness of sheet plus the bend radius or any features like holes or slots which are located so close to a bend may be distorted [32] .

The wipe bending process is shown in Figure 2.11 schematically.

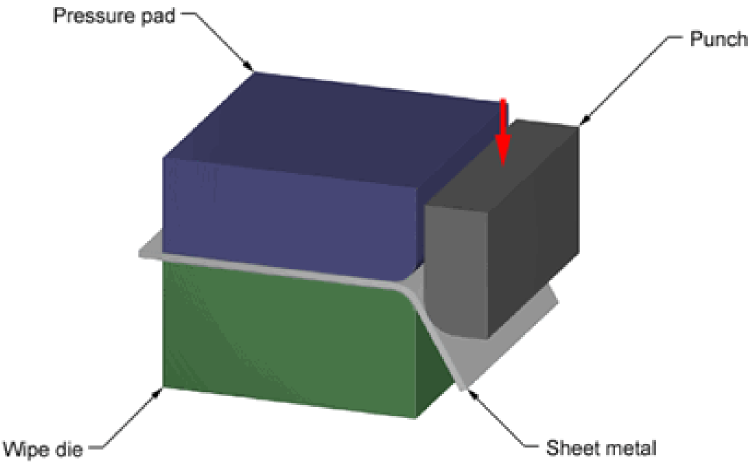


Figure 2.11: Wipe bending process of cold forming method [32].

2.2.3 Effects of cold forming on the properties of steel material

In the cold forming process, the rolls, pressing machine or bending punch compress the fed steel plate, thereby reduce the plate thickness by up to 50% and the texture of the steel change without the recrystallization of the grains [38], as a result these changes affect on the properties of steel.

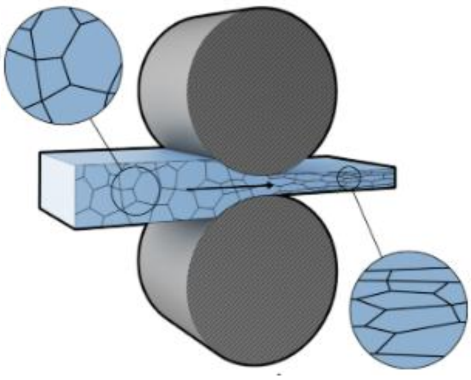


Figure 2.12: Material thickness reduction and the changing of the material texture during cold rolling [38].

In cold forming process, large deformations occur in the section. These deformations in plane part of sections are elastic but in the corners they are generally deformed plastically butafter removing the cold forming force and release the section, the elastic deformations of plane part unable to recycle completely due to the plastic deformation of corner part [43].The plastic deformation of corners spread from the corner to the edges in the amount of the maximum cross-sectional thickness [42].

The effect of cold forming is higher at cornersbecause the corners of the section are subjected to a very high degree of cold forming than the plane parts [42].

The magnitude of the effects of the cold forming depends on some items as listed below [36]:

- Type of steel material
- Method of cold forming process (applied compression or tensile stress)
- Direction of the applied stress (transverse or longitudinal)
- Ratio of ultimate tensile strength to yield strength (F_u/F_y)
- Ratio of radius to thickness (R/t), low rate requires more cold forming process [28].
- Degree of cold forming process, the higher the cold forming process, the higher the cold forming effect [28].

The effect of cold forming on steel properties are evaluated in some items as listed below:

- Stress – strain curve

The stress – strain curve of cold formed steel is shown in Figure 2.13.

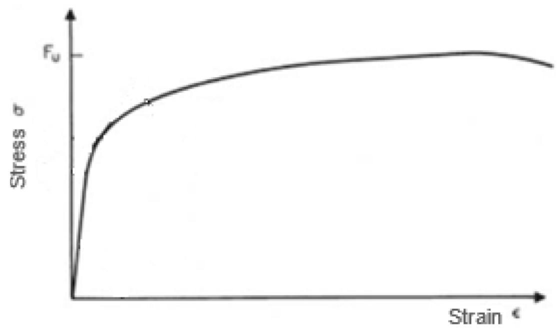


Figure 2.13: Stress – strain curve of cold formed steel section [17].

As it can be seen the cold formed steel section have abrupt yielding [50].

There are 2 ways in determining the yield point of cold formed steel material.

The first way is used for stainless and high alloy steel generally. In this way one line is drawn from the 0.2% of the permanent strain parallel to the linear part of the curve, the yield point is the intersection point of this line and the curve.

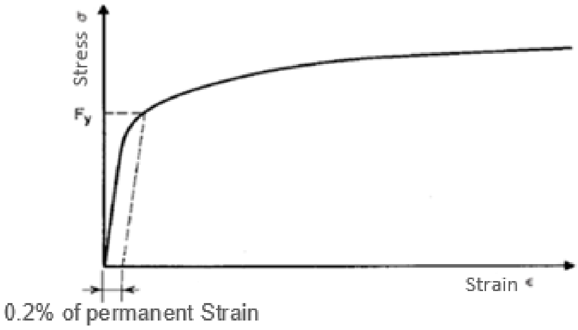


Figure 2.14: Determining yield point in stainless and high alloy cold formed steel section [50].

The second way is used for plates and low alloy steels. In this way the yield point is calculated as the point corresponding to the stated elongation under load.

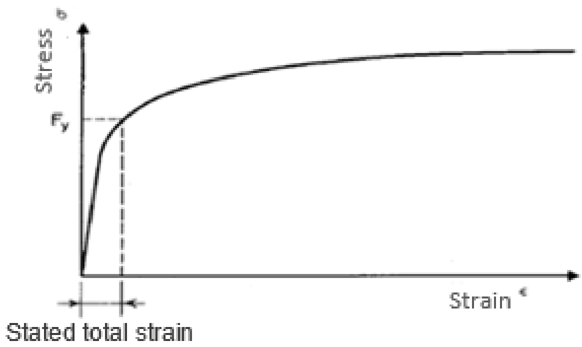


Figure 2.15: Determining yield point in plates and low alloy cold formed steel section [50].

- Ultimate tensile strength and yield strength

The "locking effect" of those twisted and distorted grains (metallic crystals) which cause strain hardening increase the ultimate tensile strength as well as yield strength [40], and this increases in the corners of section are more than in the plane part of section [36].

The change of steel ultimate tensile and yield strength parameters with increasing in the amount of cold forming, is given in Figure 2.16

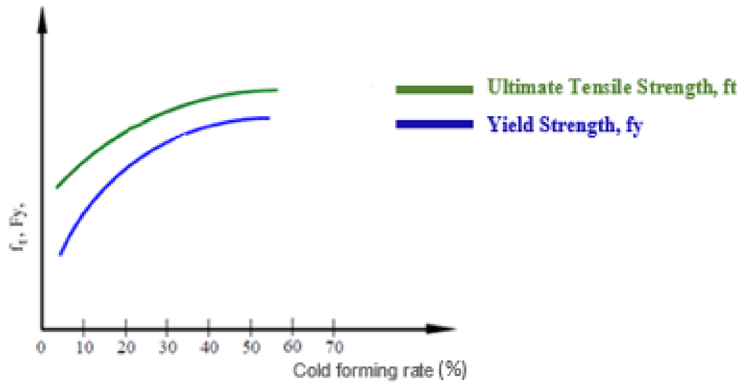


Figure 2.16: Cold forming rate and ultimate tensile strength and yield strength change [41].

The percentage increase in yield strength is much bigger than the increase in ultimate tensile strength [42].

- Ductility:

Cold forming reduces ductility so that the steel becomes brittle. Plastic design is not allowed for cold formed section.

Ductility is one of the important material properties in cold formed steel member design. As mentioned before, ductility is the ability of material to endure permanent strains or sizeable plastic deformation before fracturing, and it is important both for cold forming and for structural safety [13].

The following determinations are recommended to have sufficient ductility in cold formed steel members:

- The tensile to yield strength ratio should not be less than 1.08 [34].
- The entire elongation in a 51 mm gage length should not be less than 10%, or in a 203 mm gage length should not be less than 7 % [35].

Ductility is taken as an idex which shows the material ability to redistribute stresses.

The change of steel ductility parameters with increasing in the amount of cold forming, is given in Figure 2.17.

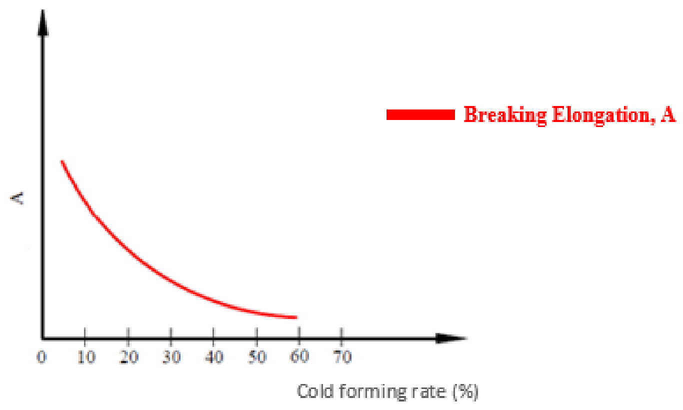


Figure 2.17: Cold forming rate and ductility change [41].

2.2.4 Utilization of cold formed steel sections

Cold formed steel structural elements are utilized as:

- Individual structural framing members
- Panels and decks

2.2.4.1 Individual structural framing members

Cold-formed members are produced in a large variety of sectional profiles. The commonly used cold formed sections as shown in Figures 2.18, 2.19 and 2.20 are single open section, open built-up sections and closed built-up sections [33]. These sections can be used as secondary or non-load bearing members such as purlins, studs, eave struts, etc. or as primary framing members such as columns, beams etc.

The plain sections are usually used as secondary members while the sections which are enhanced with web stiffeners or flange end stiffeners are used in primary structural applications. Effective area of members increase by making stiffeners in sections therefore local and overall buckling are expected to be resistedmore better [33].

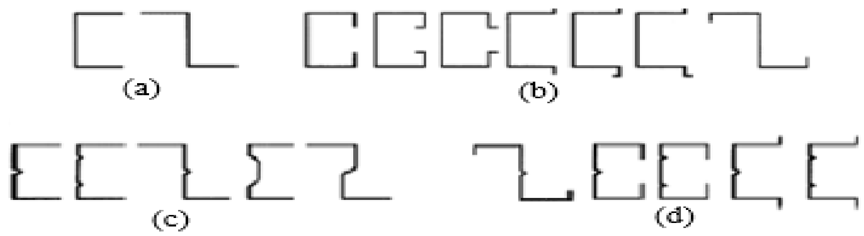


Figure 2.18: Single open sections; (a) Plain sections; (b) Sections with flange stiffeners; (c) Sections with web stiffeners; (d) Sections with flange and web stiffeners [33].

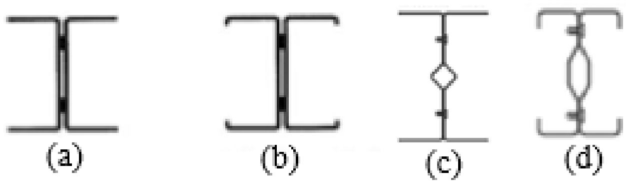


Figure 2.19: Open built-up section; (a) Plain section; (b) Section with flange stiffeners; (c) Section with web stiffeners; (d) Section with flange and web stiffeners [22].

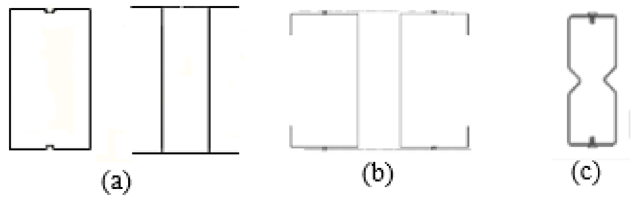


Figure 2.20: Closed built-up section; (a) Plain section; (b) Section with flange stiffeners; (c) Section with web stiffeners [22].

2.2.4.2 Panels and decks

The panel sections are usually used in roof slabs, floor slabs, panel walls, facade coverings [17].

The panels are classified into 2 groups [23]:

- Carcass system
- Sandwich panel system

The main difference between 2 groups is the production method of panels.

Since the sandwich panels are composed with insulation material, the panels do not have load bearing capacity and they are used just for detach the internal environment from the external environments and for the coating process but the other groups can carry the load and transfer the loads to the ground by the connections between columns and carcass frames [23].

Some panel profiles are shown in Figure 2.21.

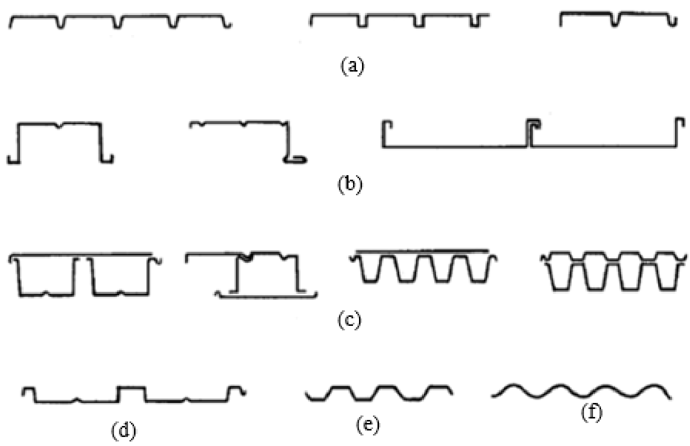


Figure 2.21: Profiles section; (a) Roof slabs; (b) Roof slabs in long spans; (c) Floor and roof panels; (d) Shear wall panels; (e) Flanged panels; (f) Wavy plates [17].

2.2.5 Advantages and disadvantages of cold formed steel sections

Cold formed steel sections have many advantages and some disadvantages both in application and manufacturing and also in designingwhen compared with other structural materials. Some of the advantages and disadvantages of cold formed steel structures are mentioned in the following sections.

2.2.5.1 Advantages

- In terms of application and manufacturing
  - Due to lightness, the use of cold formed steel section in large spans is more economical [27].
  - Due to the convenience in connecting the elements, construction period is more short [27].
  - It is possible to produce and stock the elements for installation [27].
  - They can be transported from the storeroom to the site in an economical and fast manner [27].
  - Fast and inexpensive installationcan be provided [27].
  - Construction of the structure is not affected by weather conditions [27].
  - Advanced and controllable quality of manufacture can be provided [28].
  - Smooth and good surface finish can be easily produced because oxide does not form on the surface during cold forming [45].
  - Modifying can be easily skilled, Replacement or disassembly of non-load-bearing walls can be done easily [28].
  - All steel products are recyclable [22].

- There is non creeping and non-shrinking at an ambient temperature [22].
- It is termite-proof and rot-proof [49].

- In terms of design
  - The load capacity to weight ratio is quite high so that this feature makes these section more economic and easy for on-site handling [27].
  - Cross-sectional shapes that are suitable for the architectural detailsand optimum bearing capacity can be produced [27].
  - Natural strength and non-combustible qualities of steel make the cold formed framed structure safe and endurable against fires, earthquakes, and hurricanes [28].
  - Due to lightness, it is suitable choice for structures which are built in seismic zone.
  - More accurate detailing can be provided.

2.2.5.2 Disadvantages

- In terms of application and manufacturing
  - The number of floors that can be produced is limited [27].
  - If the shaping process on the material has to be done in several stages, the force required for shaping must always be increased in each stage [27].
  - The number of forming operations is limited and after several operations, large stresses can occur on the material, which can cause breakage and cracking [27].
  - The improperly and non-uniformly cooling after cold forming causes residual stress in sections. These stresses cause to that the material yield earlier than expected [44].
  - Brittle materials cannot be cold formed easily [45].
  - The struture of grain are distorted [45].
  - Required forces for deformation are more higher than the required forces in hot working [45].
- In terms of design

Since cold formed sections are class 4 sections according to clause 5.5.2 in EC3 [39]. It means that these sections are slender sections which makes the designing of these sections more difficult.

EC 3 defines the class 4 cross sections as those in which local buckling will happen before the one or more parts of the cross-section reach the yield stress.

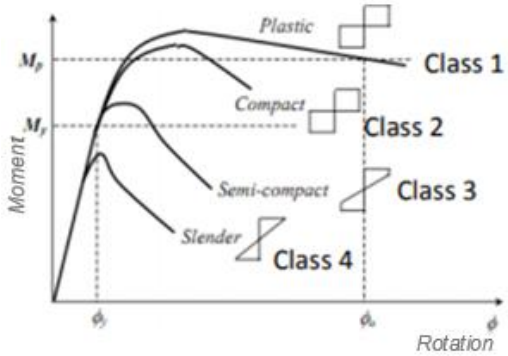


Figure 2.22: Cross section classification according to EC3 [39].

Some of that difficulties are listed as below:

- Cold formed sections are prone to buckling, the primary buckling modes of cold formed steel members are local buckling, distortional buckling and global buckling [46].

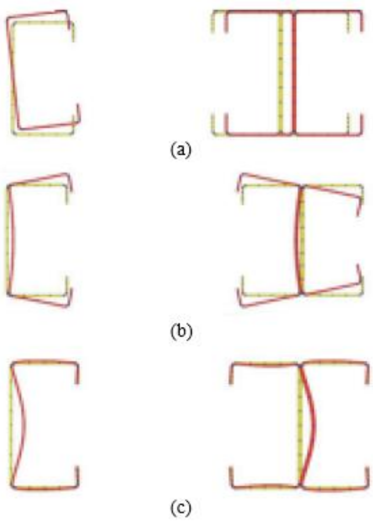


Figure 2.23: Buckling modes of cold formed cross section; (a) Global buckling; (b) Distortional buckling; (c) Local buckling [47].

- Due to the low torsional rigidity and loading eccentrically from the shear centers of the cross section torsional deformation is the dominant phenomenon in cold formed sections and it has to be inhibited [48].

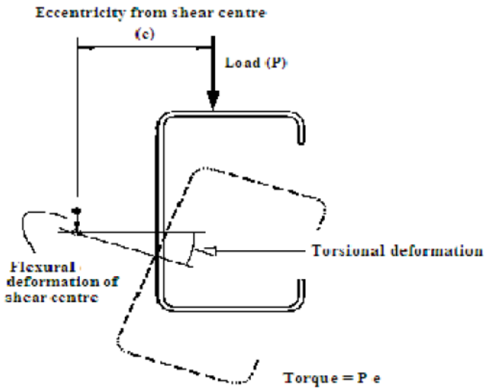


Figure 2.24: Torsional deformation of channel section [46].

- Edge stiffeners and intermediate stiffeners are required to improve the buckling resistance and load bearing capacity of cold formed sections [48].
- The section width to section thickness have limitation, when width to thickness of cold formed section which is subjected to compression is small the whole width of the cross section is fully effective however as the width to thickness ratio increases, the stress distributions become non-uniform in the compression element so the sectional properties are based on a reduced effective area [48].

2.5 Standards

There are several standard for cold-formed steel structures designing in the world. The major ones are:

- AISI S100 in USA
- EN 1993-1-3 in Europe
- AS / NZS in Australia
- S 136 in Canada

The considered industrial cold formed structure and all of the connections are designed according to AISI S100 and AISC 360-10 specifications by using of the load and resistance factor design (LRFD) method.

AISI S100 specification is used in designing of both cold formed steel structure and connections whereas AISC 360-10 specification is used in designing of hot rolled steel connections.

All the used standards in this thesis are shown in Table2.4.

Table 2.4: The used standards in this thesis.

Standard	Definition	Application
EN10346:2015 [76]	Continuously Hot-dip Coated Steel Flat Products - Technical Delivery Conditions	In defining of steel material properties
AISI S100 [35]	Design of steel structures - Part 1-5: Plated structural elements	In designing of cold formed steel members and connections
TS 648 [78]	Çelik Yapıların Tasarım, Hesap Ve Yapım Kuralları	In defining of load combinations
TS 498-1997 [53]	Yapı Elemanlarının Boyutlandırılmasında Alınacak Yüklerin Hesap Değerleri	In defining of load values
ASTM A325-14 [86]	Standard Specification for Structural Bolts, Steel, Heat Treated	In defining of bolt properties
TSENV-1998 [54]	Depreme Dayanıklı Yapıların Projelendirilmesi Tedbirleri	Earthquake load calculation
TBDY 2018 [34]	Türkiye Bina Deprem Yönetmeliği	

2.6 Cold Formed Steel Structures

Cold formed steel frames are suitable for any type of construction as following [58]:

- Housing

Using cold formed steel sections as primary members in construction is a good solution for single-story buildings or multi-story building (up to 6 floor).

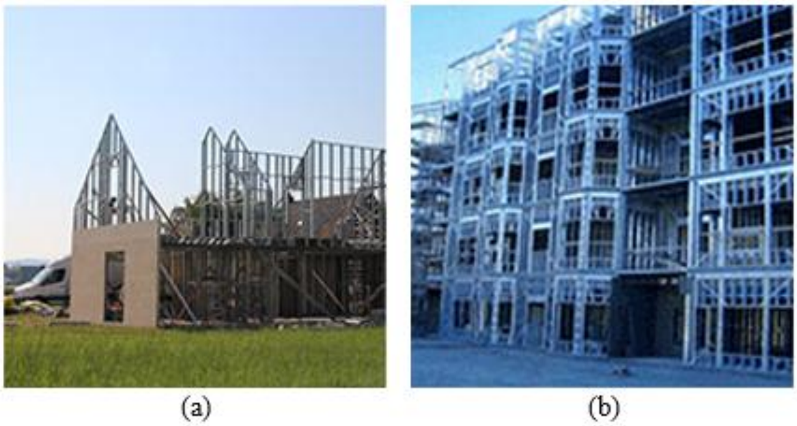


Figure 2.25: Cold formed steel used in main frame of house; (a) single story house; (b) multi story house [58].

- Storage racking system

Storage racking systems can be made from cold-formed steel components. The appropriate cross sections which haveintroduced holes and slots simplify rapid assembly and demounting of system [58].

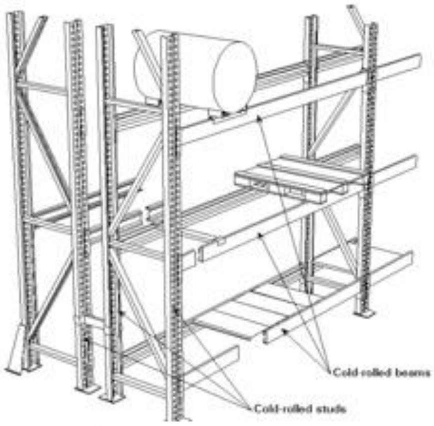


Figure 2.26: Cold formed storage racking system [58].

- Industrial construction

When cold formed steel sections are used as main frame members in industrial structure, the clear span up to 25 meter is possible, and if they are used as secondary members, 50 meter clear spans are possible [58].

The frame which is composed of columns and pitched or horizontal rafters that are connected by moment resisting connection is called portal frame [59]. Portal frame provides a clear span which is unobstructed by bracings.

Figure 2.27: Multi-bay portal frame [59].

2.6.1 Portal frame types

Portal frames have many types. Some of the most useful ones are listed as below [60]:

- Pitched roof symmetric portal frame

This type of portal frame is the most practical one.

Figure 2.28: Pitched roof symmetric portal frame [59].

- Portal frame with internal mezzanine floor

This type of portal frame is usually used for office accommodation [59].

Figure 2.29: Portal frame with internal mezzanine floor [61].

- Curved rafter portal frame

Sometimes for architectural reasons this type of portal frames may be constructed by using curved rafters [59].

Figure 2.30: Curved rafter portal frame [62].

2.6.2 Components of portal frame

Some components of portal frame are as following:

- Primary frame member

Primary frame members of portal frames are mentioned as below:

  - Column

Columns are the vertical members which are used to support rafters and side rails. Columns transfer the created forces due to the loadings in elements to the footing. There is no intermediate column in the portal frame [63].

  - Rafter

Rafter can be constructed horizontally or inclined. These members support the loadings which are imposed on the roofs. The angle of inclined rafter is usually inthe 5^0 - 10^0 range [59].

- Secondary frame member

Some secondary frame members of portal frames are mentioned as below:

  - Purlin

Purlins are horizontal structural members which are used in a roof. Purlins are supported by the rafters. They support the sheeting or roof decks which are located on them [64]. They transfer the loads from sheeting and roof decks to the primary steel frame [59].

  - Side rail

Side rails are horizontal structural members in a wall and have a same duty as purlins in a roof [59].

  - Eave strut

Eave struts are the transition pieces between the roof purlins and side rails along the eave [65].

  - Roof and wall bracing

Wall and roof bracing usually consist of panels of double diagonals. These members transfer the longitudinal wind and earthquake forces to the footing [66], so these members are too important to provide adequate lateral resistance in structure. Transverse wind and earthquake loads are resisted by inherent in-plane flexure of portal frame.

- Wall and roof cladding

The wall and roof cladding separate the external environment from enclosed space and also provide acoustic and thermal insulation. Most of the wall and roof claddings are composite sheets. They are composed of two thin aluminium or metal sheets which isolation material are sandwiched between them [59].

- Joints

Some important joints of portal frame are mentioned as below:

- Apex joint

The joint which two pitched rafters are connected together at the top are called Apex joint.

- Eave joint

The joint which a pitched rafter are connected to column at the edge are called Eave joint.

These portal frame components are shown in Figures 2.31 and 2.32.

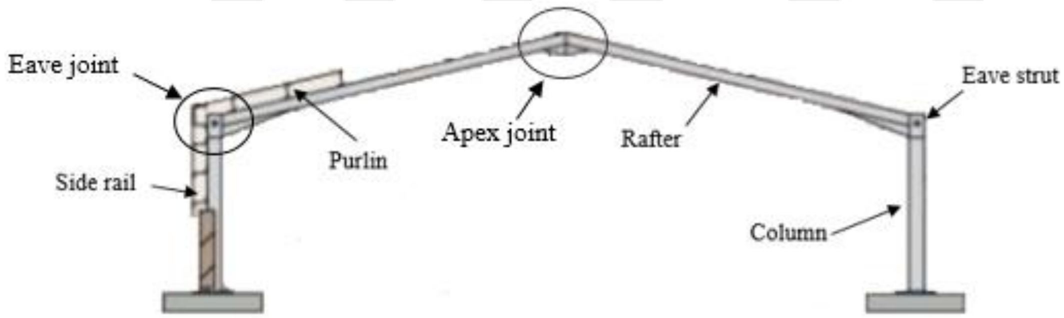


Figure 2.31: Components of portal frame [59].

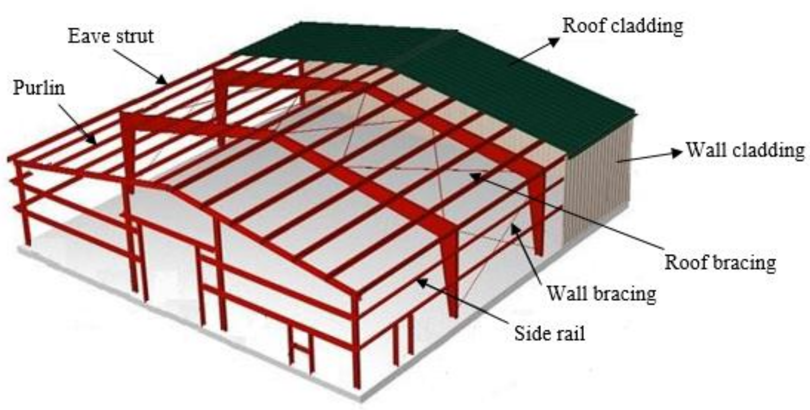


Figure 2.32: Components of portal frame [67].

2.7 Connections

Designing of connections are the important part of the structure designing because the connection behavior affect the structure performance directly.

It is obvious that the connection behavior primarily depends on [69]:

- The properties of used fasteners
- The properties of the connected elements (yield stress, thickness)

The properties of fasteners are depends on the used fasteners type.

The fasteners which are used in cold formed steel constructions can be classified into 3 groups [70]:

- Fastenings with mechanical fasteners
- Fastenings based on welding
- Fastenings based on adhesive bonding

2.7.1 Mechanical fasteners

Some of the practical mechanical fasteners are listed as below [70]:

- Bolts with nuts

Bolts have threads on one end and a head on the other head, and they are paired with nuts [71]. The bolts with nuts are assembled in holes which are performed through the elements to be joined.

Figure 2.33: Example of fully threaded bolt with nut [70].

- Screws

Screws are similar to bolts, have threads on one end and head on the other end but the threads part are longer than bolts. Screws are usually combined with washers in order to improve the load-bearing resistance of connection [70]. Screws can be made with slotted heads.

Two major types of screws are:

- Self-tapping screws
- Self-drilling screws

The screws which are used in cold formed steel section connections are self-drilling screws.

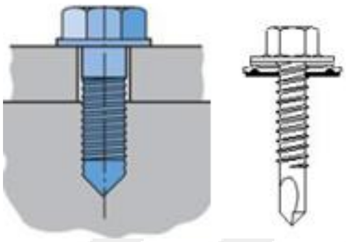


Figure 2.34: Example of self-drilling screws [71].

- Rivets

There are 2 main types of rivets as following:

- Hot rivets
- Cold rivets

The rivets which are used in cold formed steel section connections are cold rivets [70].

Cold rivets have many types, some of them are mentioned at below:

- Blind rivet: For application from one side
- Tubular rivet: To increase bearing area

Cold rivets are installed in pre-drilled holes in the thin elements. Cold rivets are used where access to the assembly is limited [71].

Figure 2.35: Example of cold rivets [72].

2.7.2 Welding

There are 2 types of welding methods as following [17]:

- Fusion welds (Arch welds)

Arc welding method is a series of processes where metals are welded together using weld metal without using mechanical pressure or compression on the surfaces which are to be joined [17].

- Resistance welds

In this method, the desired parts are melted by heating, and then the molten parts are joined together by applying force [17].

Arc welding methods are usually used in cold formed steel constructions.

Some types of arc welding methods are listed as below:

- Groove welds
- Arc spot welds (puddle welds)
- Arc seam welds
- Fillet welds
- Flare groove welds

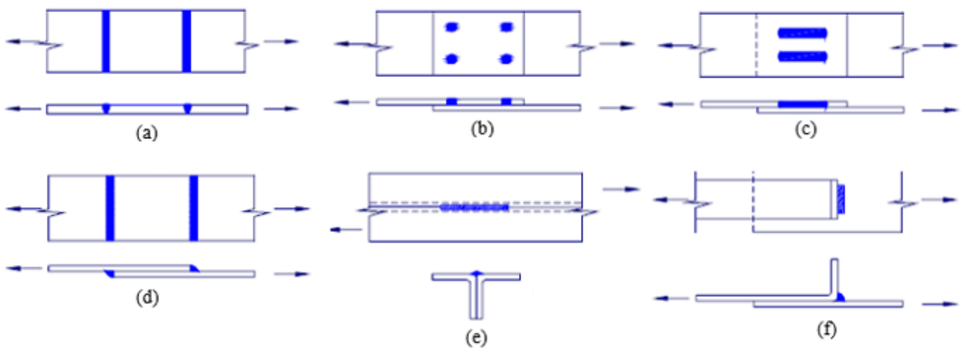


Figure 2.36: Arc welding types; (a) Groove welds; (b) Arc spot welds; (c) Arc seam welds; (d) Fillet welds; (e) Flare bevel groove welds; (f) Flare V- groove welds [72].

2.7.3 Adhesive bonding

In this method which is a new technology in connecting the cold formed steel members, two elements are connected together by glues and epoxies [73]. One of the advantages of this method is an uniform distribution of forces in the connection, and the disadvantages of this method is that the surface have to be clear and flat, and a hardening time is needed [17].

3 STRUCTURE MODEL

In this section the considered structure is modeled with SAP2000 program and the created forces in the joints due to the loadings are determined.

The considered model is a cold formed steel frame structure which will be used as a factory building. It has 20 meter length, 18 meter width and 8 meter height. The slope of pitched roof is 11.31^0. The columns and rafters are connected rigidly in X directions and the connection of elements in Y direction are simple pinned joints so that the created moment due to the lateral loadings are resisted by the rigid portal frame in X direction and by hinged bracings in Y direction.

The structure is assumed to be located at Genc Osman Street, Kartal, Istanbul. The altitude of structure location is 101 meter and the coordinate of the location is [80]:

Latitude: 40.915629

Longitude: 29.199879

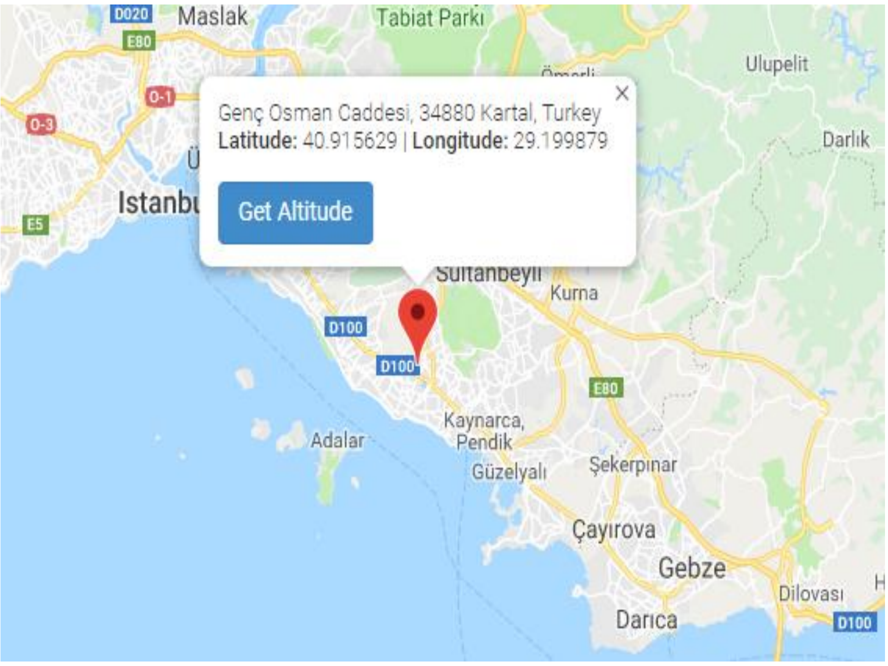


Figure 3.1: Coordinates of structure location [80].

Local ground consists of solid and hard rocks, so the local soil class is ZA [34].

The model is designed according to AISI S100 specification by using of the load and resistance factor design (LRFD) method. The model analyzed elastically.

The earthquake load is calculated by Equivalent Seismic method and the ground movement level is DD2. It is the ground movement level which the probability of exceeding in 50 years is 10% (recurrence period 475 years) [34]. It is assumed that a large part of earthquake energy is consumed with deformations without loss of strength; it means that the structure is considered to be high ductile system.

3.1 Material

The properties of used steel material in the structure are explained in this part.

3.1.1 Material properties

Used steel properties are categorized into 3 groups:

- Mechanical properties
- Physical properties
- Chemical compositions

3.1.1.1 Mechanical properties

The cold formed member are made of S350GD+Z steel.

The symbol of +Z shows that the coating type of steel is zinc [75].

The mechanical properties of S350GD+Z steel are given in Table 3.1 [76].

Table 3.1: Mechanical properties of S350GD steel [76].

Properties	Steel grade: S350GD+Z[1]
Modulus of elasticity (E)	200000
Poisson ratio (Ʋ)[2]	0.3
Shear modulus (G = E / [2 x (1 + Ʋ)])	76923.08
Tension safety stress (σ)	210
Shear safety stress (τ)	115
Minimum yield stress (F_y)	350
Minimum ultimate stress (F_u)	420
Elongation (min %)[3]	16

1) All of the units are N/mm^2.
2) Poisson ratio is unit-less.
3) Decreased minimum elongation, A80 values apply to thicknesses 0.50mm < t ≤ 0.70mm (minus 2 units) and t ≤ 0.50mm (minus 4 units).
3) It is measured in a 51 mm gage length.

3.1.1.2 Physical properties

Some physical properties of used steel are given inTable 3.2 [76].

Table 3.2: Physical properties of S350GD steel [76].

Properties	Values
Weight per unit volume (Y)	7850 kg/m^3 = 7.7 x l0 -5 N/mm^2
Coefficient of linear thermal expansion (A)	11 x 10^{-6} 1/℃

3.1.1.3 Chemical compositions

In the following table you can see the combination percentage of chemical components of used steel.

Table 3.3: Chemical composition of S350GD steel [76].

Steel grade		Coating type symbol	C	Si	Mn	P	S
Name	Number						
S350GD	1.0529	+Z	0.2	0.6	1.7	0.1	0.045

3.1.2 Defining material in SAP2000 program

The material which are used in Turkey are the same as the used material in Europe thus the same code is used for defining material in SAP2000 program.

The expected yield stress and tensile stress are defined by equations (3.1) and (3.2) respectively.

$F_{ye} = R_y \times F_y$ (Eq.3.1)

$F_{ue} = R_t \times F_u$ (Eq.3.2)

The values of R_y and R_t are given in Table 3.4.

The R_y and R_t values of S355 steel are approximately same as R_y and R_t values for S350 steel. These values are specified in Table 3.4.

$F_{ye} = 1.25 \times 350 = 437.5$ N/mm^2

$F_{ue} = 1.1 \times 420 = 462$ N/mm^2

Table 3.4: R_y and R_t Coefficients in TBDY 2018 code [34].

Structure Steel Class and Element Type	R_y	R_t
Rolling profiles and sheets made of steel S 235	1.4	1.1
Rolling profiles and sheets made of steel S 275	1.3	1.1
Rolling profiles and sheets made of steel S 355	1.25	1.1
Rolling profiles made of steel S 460	1.1	1.1
Pipe and Box profiles	1.4	1.3
Reinforcing steel	1.2	1.2

The S350GD+Z steel material was defined in SAP2000 program as it is shown in Figure 3.2.

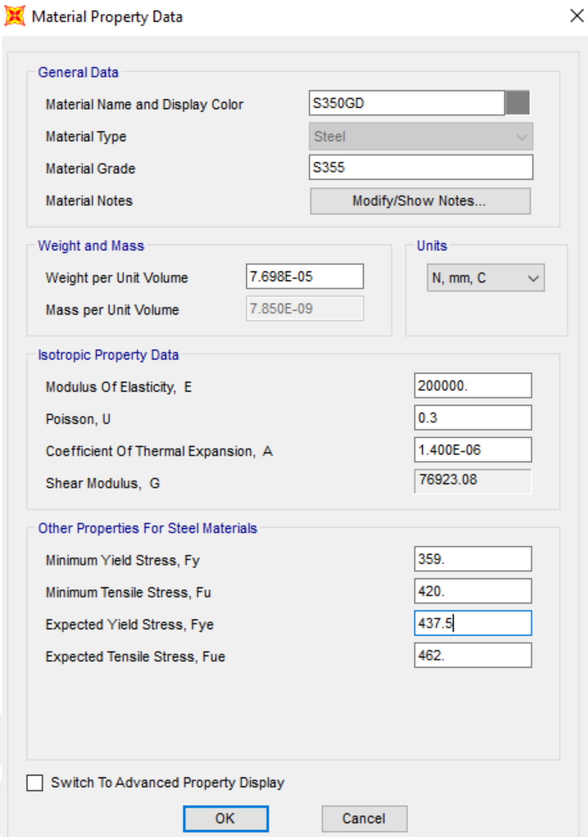


Figure 3.2: Defined S350GD+Z steel material in SAP2000 program.

3.1.3 Codes conditions of material

The conditions according to the TBDY 2018 code which are determined for the material used for cold formed steel members in this study are verified.

The material properties of the cold formed structural steel members should meet the following conditions [34]:

- The minimum yield stress of steel material should be 235 MPa.

Minimum yield stress (F_y) of S350GD steel is 350 N/mm^2 which is larger than 235 N/mm^2.

- The ratio of the tensile strength of the material to the yield stress of the material should be at least 1.08.

Minimum ultimate stress (F_u) of S350GD steel is 420 N/mm^2 so the ratio of the F_u to F_y for S350GD steel is 1.18 (F_u / Fy = 420 / 335 = 1.18) which is larger than 1.08.

- Materials with a minimum elongation rate of 10% are called normal ductile material, and materials with a minimum elongation rate of 16% are referred to as high ductile material. Normal ductile materials can only be used as purlin and non-bearing struts.

The cold formed sections which are used as load bearing members (as columns and rafters) have to be made of high ductile material, and minimum elongation of them should be more than %16.The thickness of used sections are more than 0.7 mm. As it is shown in Table 3.1, the minimum elongation of S350GD steel material is 16%and it provides this condition.

3.2 Dimensions

Dimensions of model are as following:

- Span = 20 m
- Total height = 10 m
- An eave height = 8 m
- A roof angle = 11.31^0
- A frame spacing = 6 m

These dimensions are shown in Figure 3.3.

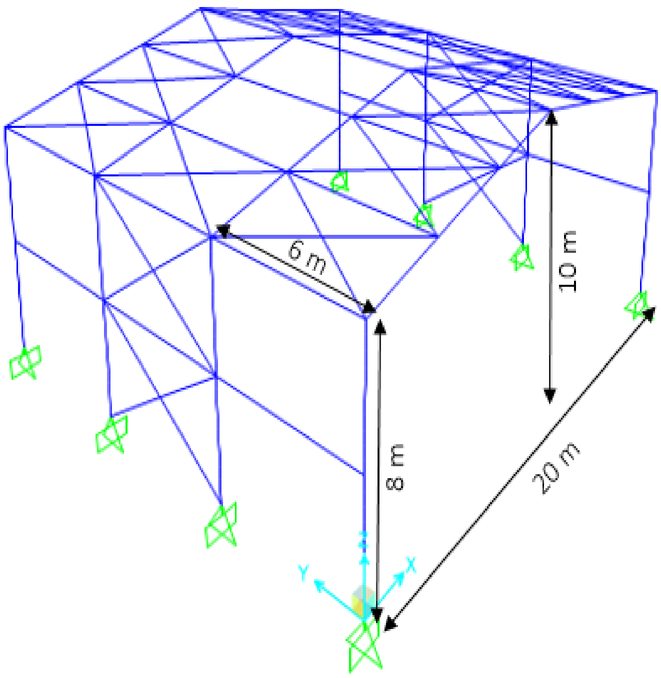


Figure 3.3: Defined model dimensions in SAP2000 program.

3.3 Cross Sections

The terms ofcold formed channel section are shown in Figure 3.4 [77].

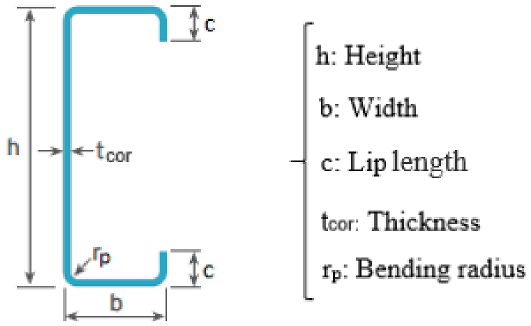


Figure 3.4: Terms of channel cold formed section [77].

The cross sections dimensions which are used in model are given in Table 3.5.

Table 3.5: Dimensions of used cross sections in model.

Cross section	h	b	c	t_{cor}	r_p
Column	600	160	60	4	3
Rafter	650	150	60	4	3
Stability beam	180	100	40	4	3
Diagonal	160	100	40	4	3

All dimensions are millimeter.

It should be noted that the sections which are used for columns and rafters are two channel sections which are attached together back to back. These channel sections dimensions are given in Table 3.5.

3.3.1 Defining cross sections in SAP2000 program

The defined cross section in SAP2000 program is shown in Figure 3.5.

The used cross sections are specified with different colors.

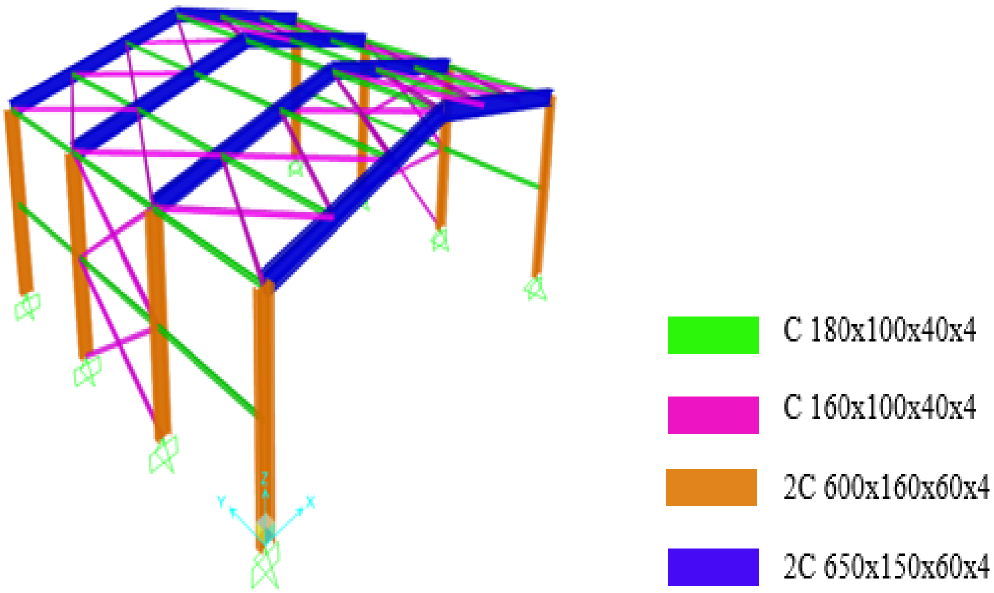


Figure 3.5: Defined model cross sections in SAP2000 program.

The dimensions of defined cross sections are shown in Figures 3.6, 3.7, 3.8 and 3.9.

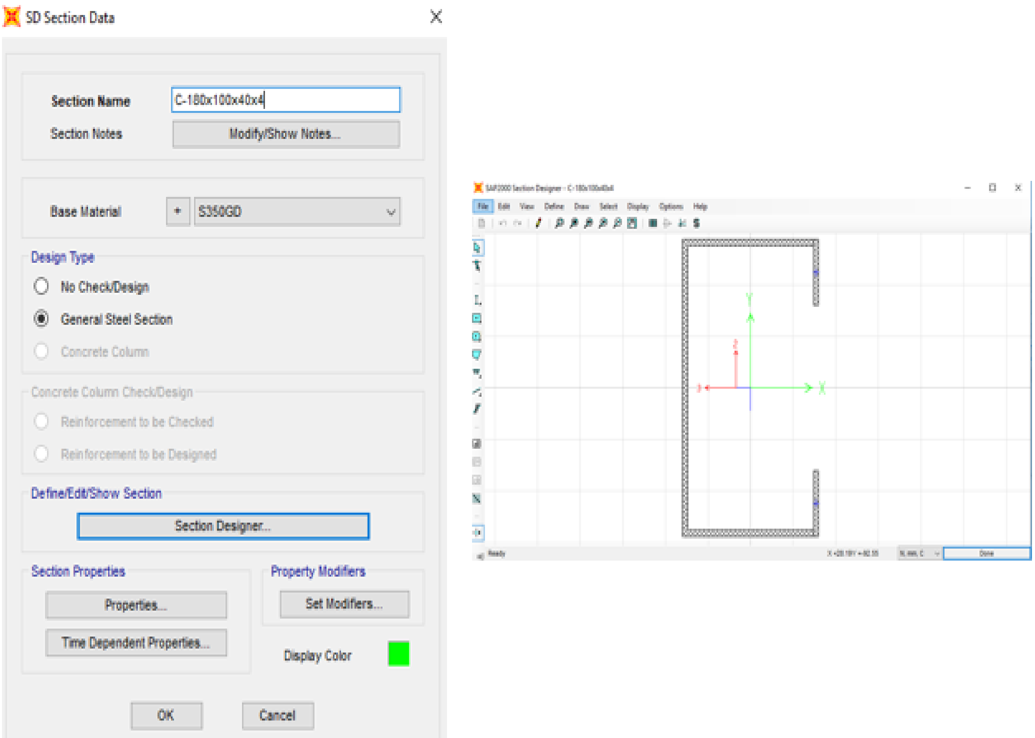


Figure 3.6:C 180x100x40x4 cross section defined in SAP2000 program.

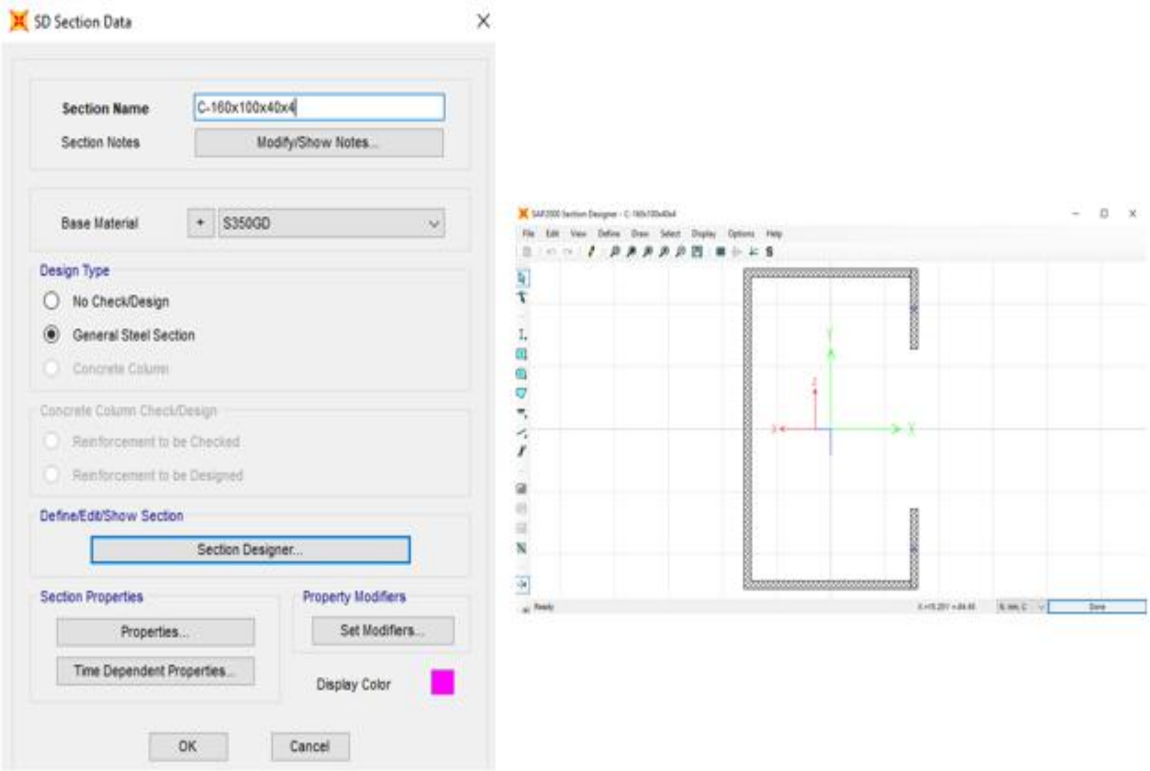


Figure 3.7:C 160x100x40x4 cross section defined in SAP2000 program.

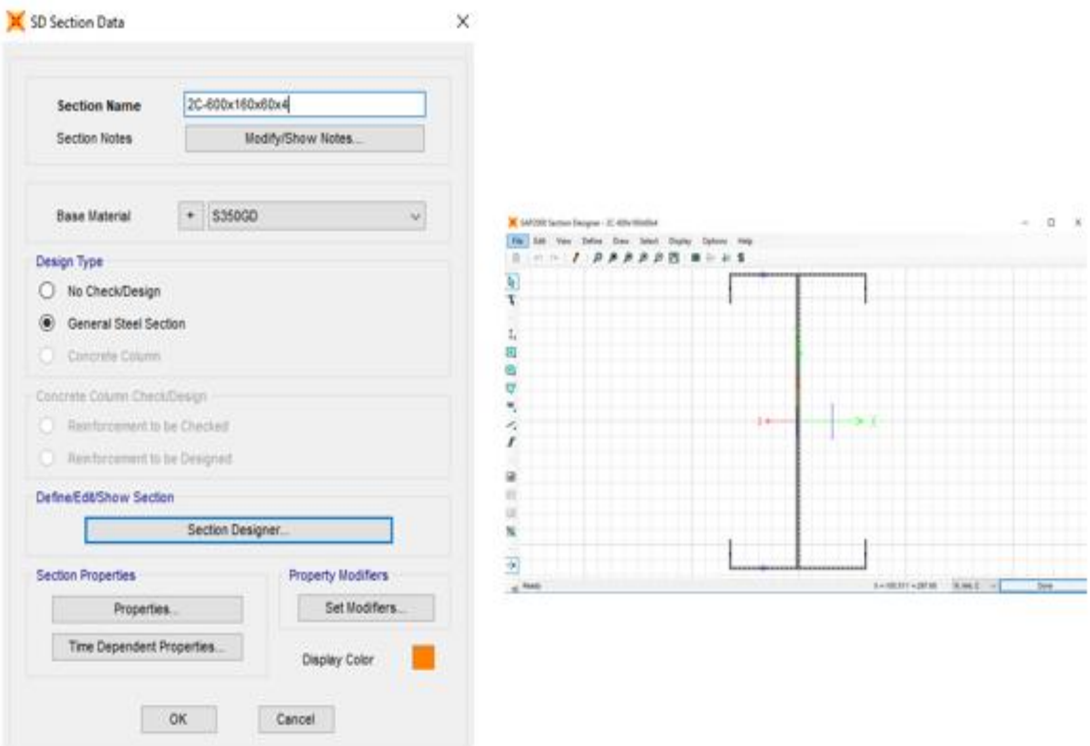


Figure 3.8:2C 600x160x60x4 cross section defined in SAP2000 program.

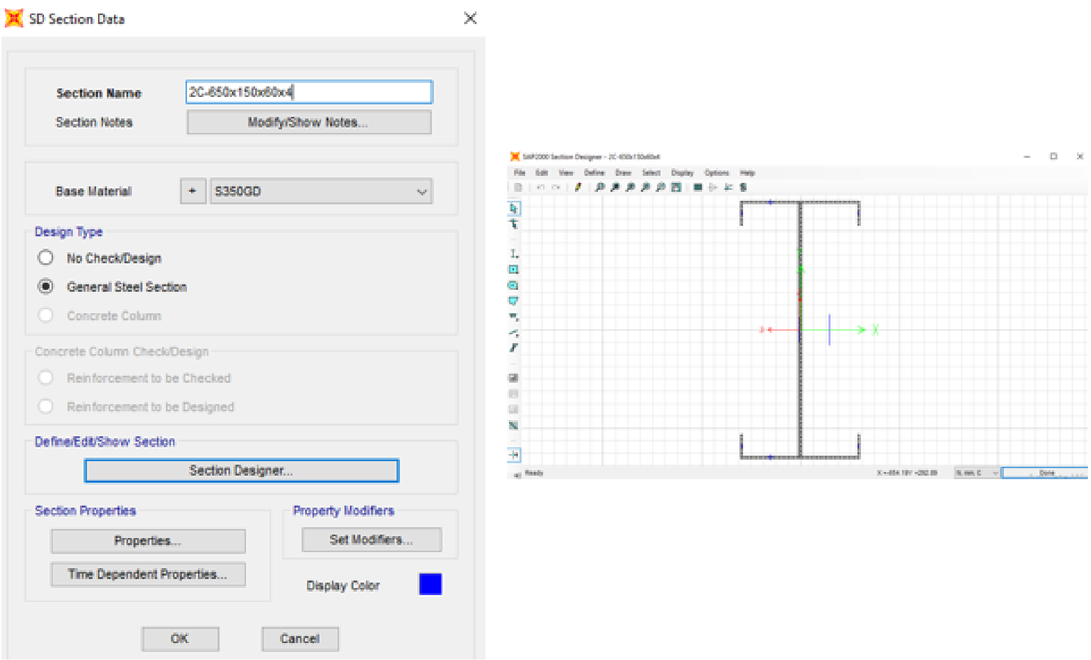


Figure 3.9:2C 650x150x60x4 cross section defined in SAP2000 program.

3.3.2 Code conditions of cross section

The conditions of TBDY 2018 code which are determined for dimensions of cold formed steel section are verified.

The dimensions of cold formed steel sections should meet the following conditions [34]:

- The allowed range of width-to-thickness ratios for cold formed steel sections are as following:

$b / t_{cor} < 60$

$c / t_{cor} < 50$

All the used sections are verified as can be seen below:

Column sections:

$160 / 4 = 40 < 60$

$60 / 4 = 15 < 50$

Rafter sections:

$150 / 4 = 37.5 < 60$

$60 / 4 = 15 < 50$

Stability beam sections:

$100 / 4 = 25 < 60$

$40 / 4 = 10 < 50$

Diagonal sections:

$100 / 4 = 25 < 60$

$40 / 4 = 10 < 50$

- The allowed size of used stiffeners in section for providing sufficient stiffness are given in following:

$0.2 \leq c / b \leq 0.6$

$0.1 \leq d / b \leq 0.3$

All the sections used in the design are verified as follows:

Column sections:

$0.2 \leq 60 / 160 = 0.375 \leq 0.6$

Rafter sections:

$0.2 \leq 60 / 150 = 0.4 \leq 0.6$

Stability beam sections:

$0.2 \leq 40 / 100 = 0.4 \leq 0.6$

Diagonal sections:

$0.2 \leq 40 / 100 = 0.4 \leq 0.6$

- If the bending radius of sections provides the $r_p<5t_{cor}$ and $r_p<0.1b$ conditions, the effect of sharp corner on their cross-sectional properties will be ignoredby taking $r_p = 0$.

All the used sections are checked.

Column sections:

$3 < 5 \times 4 = 20$

$3 < 0.1 \times 160 = 16$

Rafter sections:

$3 < 5 \times 4 = 20$

$3 < 0.1 \times 150 = 15$

Stability beam sections:

$3 < 5 \times 4 = 20$

$3 < 0.1 \times 100 = 10$

Diagonal sections:

$3 < 5 \times 4 = 20$

$3 < 0.1 \times 100 = 10$

So the bending radius is equal to zero in all sections as a result,there is no need to reduce the gross area and the inertia moment of cross sections due to the sharp corners.

3.4 Loads

The symbols of applied load on model are listed as below [78]:

- G : Dead load
- Q_r: Roof live load
- S : Snow load
- W_x : Wind load in X direction
- W_y : Wind load in Y direction
- E_x : Earthquake load in X direction
- E_y : Earthquake load in Y direction

3.4.1 Load values

The values of applied dead load, roof live load, snow load and wind load are determined according to TS 498-1997 code [53] but the applied earthquake load is determined according to TBDY 2018 code.

3.4.1.1 Dead load

Dead loads of structure are categorized into 2 groups:

- Self-weight of members (DEAD)

Dead loads due to self-weight of members are calculated automatically by program.

- Covering load

Both facade and roof are covered by sandwich panels.

The total covering load is the sum of the weight of sandwich panels, the weight of purlins and the weight of installations.

The covering load is calculated for a unit area.

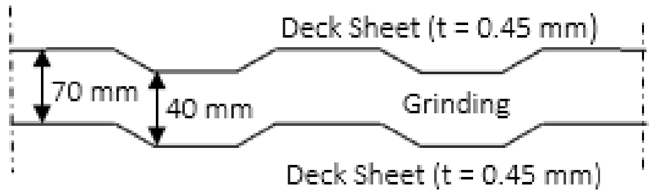


Figure 3.10: The used sandwich panel in roof and facade covering [79].

As it is shown in Figure 3.10, sandwich panel is composed of foamed concrete (grinding) and two deck sheets. The weight of sadiwich panel is calculated by summing up the weight of its components.

The weight of these components are calculated as following:

- Deck sheet:

Y_{foled} =76.98 KN/m^2 (7850 kg/m^2)

Thickness of deck sheet = 0.45 x 10^{-3} m

2 deck sheet are used, one in top and one in bottom.

Weight per unit area = 0.45 x 10^{-3} x 1 x 1 x 76.98 = 0.035 KN/m^2 (3.5 kg/m^2)

For 2 decks: 0.035 x 2 = 0.07 KN/m^2 (≈ 7 kg/m^2)

- Grinding:

$Y_{\text{foamed concrete}}$ = 0.98 KN/m^2 (100 kg/m^2)

Average thickness = (70 + 40) / 2 = 55 mm

Weight per unit area = 0.055 x 1 x 1 x 0.98 = 0.0539 KN/m^2 (≈5.5 kg/m^2)

So the total weight of sandwich panel for a unit area is equal to 0.124 KN/m^2(0.07 + 0.0539 = 0.124 KN/m^2)

The weight of purlins for a unit area is 0.147 KN/m^2 (≈15 kg/m^2).

The weight of installations for unit area is 0.196 KN/m^2(≈20 kg/m^2).

As a result, the total covering load is calculated as below:

0.124 +0.147+0.196 = 0.47 KN/m^2 (≈ 50 kg/m^2)

3.4.1.2 Roof live load

The roof live load of factory building is 0.78 KN/m^2 (≈80 kg/m^2)[53].

3.4.1.3 Snow load

The snow load is calculated by equation 3.3 [53].

P_k = m x P_{k0}(Eq.3.3)

Where,

m is a reduction value which is dependent on the roof inclination is obtained from Table 3.6.

Table 3.6: Reduction value dependent on roof inclination [53**].**

α	0°	1°	2°	3°	4°	5°	6°	7°	8°	9°
0-30°					1,0					
30°	1,00	0,97	0,95	0,92	0,90	0,87	0,85	0,82	0,80	0,77
40°	0,75	0,72	0,70	0,67	0,65	0,62	0,60	0,57	0,55	0,52
50°	0,50	0,47	0,45	0,42	0,40	0,37	0,35	0,32	0,30	0,27
60°	0,25	0,22	0,20	0,17	0,15	0,12	0,10	0,07	0,05	0,02
70° - 90°										

The inclination of pitched roof is 11.31 ^{0} som is qual to 1 according to Table 3.6.

Structure is located in Istanbul which is defined as 'snow zone Ⅰ', and the ground altitude at which the building is located is less than 200 meters from the sea level.

The P_{k0} is self snow load which is equal to 0.75 KN/m^2 (75 kg/m^2) according to Table 3.7.

As a result, the snow load is calculated as below:

P_k = 1 x 0.75 = 0.75 KN/m^2(75 kg/m^2)

Table 3.7: Self-snow load, the units are KN/m^2 [53].

	1	2	3	4	5
1	Height of the building location from the sea	Zones			
	m	I	II	III	IV
	≤ 200	0.75	0.75	o.75	0.75
2	300	0.75	0.75	0.75	0.8
	400	0.75	0.75	0.75	0.8
	500	0.75	0.75	0.75	0.85
3	600	0.75	0.75	0.8	0.9
	700	0.75	0.75	0.85	0.95
	800	0.8	0.85	1.25	1.40
4	900	0.8	0.95	1.30	1.50
	1000	0.8	1.05	1.35	1.60
5	>1000	For heights up to 1500 m %10 and for the heights more than 1500 m %15 of the values corresponding to 1000 m are taken			

3.4.1.4 Wind load

Wind load affects both facade and roofs.

The directions and the formulary for calculating of the wind load valuesare shown at Figure 3.11 [53].

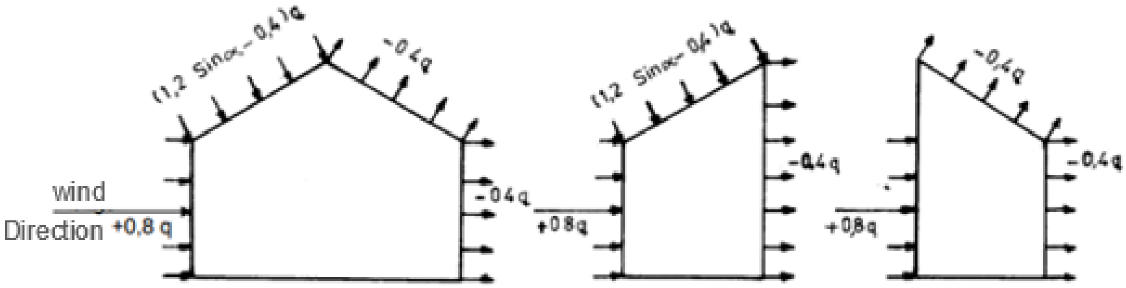


Figure 3.11: The Directions and the formulary for calculating of wind loads on roofs and columns[53].

The wind on roof and facade, is acting as pressure load on the members that impact them first in one direction and it is acting as suction load on the other members in the same direction.

The wind load on roof and facade are calculated as below:

q is a suction coefficient which is obtained from Table 3.8.

Table 3.8: Wind velocity and suction coefficient of wind [53].

Height (m)	Wind velocity (m/s)	Suction q (kN/m^2)
0 - 8	28	0,5
9 - 20	36	0,8
21 - 100	42	1,1
> 100	46	1,3

Wind load on columns is calculated as below:

The column height is 8 m so q is equal to 0.5 KN/m^2(51 Kg/m^2).

As it is shown in Figure 3.11, the pressure load and suction load on columns are calculated by equations 3.4 and 3.5 respectively.

Pressure load = 0.8 x q (Eq.3.4)

= 0.8 x 0.5 = 0.4 KN/m^2(≈ 41 Kg/m^2)

Suction load = 0.4 x q (Eq.3.5)

=0.4 x 0.5 = 0.2 KN/m^2(≈ 20.4 Kg/m^2)

Wind load on roof is calculated as below:

The Roof height is 2 m so the q is equal to 0.5 KN/m^2 (51 Kg/m^2) too.

The inclination of roof (α) is 11.31°.

As it is shown in Figure 3.11, the pressure load and suction load on roof are calculated by equations 3.6 and 3.7 respectively.

Pressure load = (1.2 x Sin_α - 0.4) x q (Eq.3.6)

= (1.2 x 0.196 – 0.4) x 0.5 = -0.082 KN/m^2(≈ -8.4 Kg/m^2)

Suction load = -0.4 x q (Eq.3.7)

= -0.4 x 0.5 = -0.2 KN/m^2(-20.4 Kg/m^2)

3.4.1.5 Earthquake load

The earthquake load is calculated by the Equivalent Seismic Load method.

The earthquale load in X and Y directions are calculated by equations 3.8 and 3.9 respectively.

$E_x = S_{aRx}(T) \times W_i$(Eq.3.8)

$E_y = S_{aRy}(T) \times W_i$(Eq.3.9)

Where,

W_i is a seismic weight of model.

$S_{aRx}(T)$ is reduced design spectral acceleration in X direction.

$S_{aRy}(T)$ is reduced design spectral acceleration in Y direction.

The seismic weight of model is calculated by equation 3.10.

$W_i = G + nQr + nS$ (Eq.3.10)

Where,

n is a live load participation factor and it is determined according to Table 3.9. It is equal to 0.3 for factory structure.

Table 3.9: Live load participation factor [34].

Building Type	n
Storage, ware house, etc.	0.8
School, student residence, club, cinema, theater, concert hall, temple, restaurant, shop, etc.	0.6
House, workplace, hotel, hospital, auto park, etc.	0.3

Seismic weight is calculated by defining the required parametersby the program automatically.

$S_{aRx}(T)$ and $S_{aRy}(T)$ is calculated by equations 3.11 and 3.12 respectively [34].

$S_{aRx}(T) = S_{aex}(T) / R_{ax}(T)$ (Eq.3.11)

$S_{aRy}(T) = S_{aey}(T) / R_{ay}(T)$ (Eq.3.12)

Where,

$S_{aex}(T)$ is horizontal elastic design acceleration in X direction.

$S_{aey}(T)$ is horizontalelastic design acceleration in Y direction.

$R_{ax}(T)$ is earthquake load reduction factor in X direction.

$R_{ay}(T)$ is earthquake load reduction factor in Y direction.

S_{ae} (T) is calculated by equation 3.13.

$S_{ae}(T) = [\ 0.4 + 0.6\ (\ T / T_A\ )\ ]\ S_{DS}\ 0 \leq T \leq T_A$

$S_{ae}(T) = S_{DS} T_A \leq T \leq T_B$ (Eq.3.13)

$S_{ae}(T) = S_{D1} / T \qquad T_B \leq T \leq T_L$

$S_{ae}(T) = (\ S_{D1} \times T_L\ ) / T^2\ T_L \leq T$

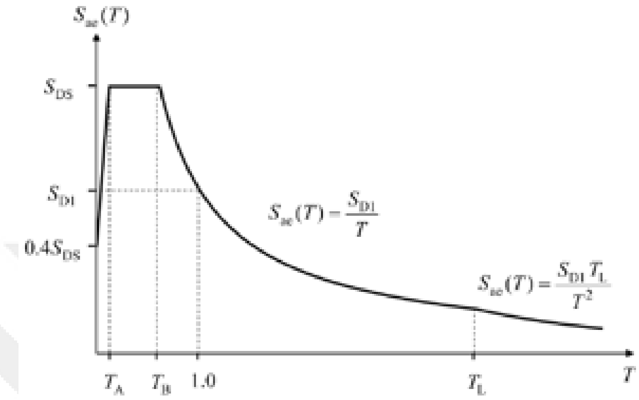


Figure 3.12: Horizontal elastic design acceleration spectrum [g] [34].

Where,

$T_A = 0.2 \times (\ S_{D1} / S_{DS}\ )$

$T_B = S_{D1} / S_{DS}$

$T_L = 6$ s

S_{DS} and S_{D1} are design spectral acceleration coefficients and they are obtained by equations 3.14 and 3.15 respevtively.

$S_{DS} = S_S \times Fs$ (Eq.3.14)

$S_{D1} = S_1 \times F_1$(Eq.3.15)

Where,

$S_1$is the earthquake spectral acceleration for 1 s period.

S_S is earthquake spectral acceleration for short period.

F_1 is coefficients of local ground effect for 1 s period.

Fs is coefficients of local ground effect for short period.

S_1 and Sscoefficients are determined by the Turkey Earthquake Hazard map. These coefficient according to DD2 ground movement level, the ZA local soil class and the given structure location coordinates are 0.281g and 1.022g respectively [81].

F_s and F_1 coefficients are defined according to Table 3.10 and 3.11 respectively.

Table 3.10: Local ground effect coefficient for short period zone (Fs) [34].

Local Soil Class	Local ground effect coefficient for short period zone					
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S = 1.25$	$S_S \ge 1.50$
ZA	0.8	0.8	0.8	0.8	0.8	0.8
ZB	0.9	0.9	0.9	0.9	0.9	0.9
ZC	1.3	1.3	1.2	1.2	1.2	1.2
ZD	1.6	1.4	1.2	1.1	1.0	1.0
ZE	2.4	1.7	1.3	1.1	0.9	0.8
ZF	Specific analysis will be done					

Fs with Ss = 1.22 is obtained by linear interpolation between $S_S = 1$ and $S_S = 1.25$. It isequal to 0.8.

Table 3.11: Local ground effect coefficient for 1 s period zone (F_1) [34].

Local Soil Class	Local ground effect coefficient for 1 s period zone					
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 = 0.50$	$S_1 \ge 0.60$
ZA	0.8	0.8	0.8	0.8	0.8	0.8
ZB	0.8	0.8	0.8	0.8	0.8	0.8
ZC	1.5	1.5	1.5	1.5	1.5	1.4
ZD	2.4	2.2	2.0	1.9	1.8	1.7
ZE	4.2	3.3	2.8	2.4	2.2	2.0
ZF	Specific analysis will be done					

$F_1$with $S_1 = 0.281$is obtained by linear interpolation between $S_1 = 0.2$ and $S_1 = 0.3$. It is equal to 0.8.

As a result, S_{DS} and S_{D1} coefficients are calculated as below:

S_{DS}= 1.022 x 0.8 = 0.8176

S_{D1}= 0.281 x 0.8 = 0.2248

And also T_A and T_B are calculated as below:

T_A: 0.2 x (0.225 / 0.818) = 0.055 s

T_B: 0.225 / 0.818 = 0.275 s

The first approximate period of structure is assumed to be 0.2s (T=0.2s) so the periods of structure in X and Y direction are equal and they are between the $T_A$and T_B boundaries.

As a result, $S_{aex}(T)$ and $S_{aey}(T)$ are calculated as below:

$S_{aex}(T) = S_{aey}(T) = S_{Ds} = 0.818g$

$R_a(T)$ is calculated by equation 3.16.

$$R_a(T) = R / I \qquad T > T_B$$
$$R_a(T) = D + [[(R / I) - D] \times (T / T_B)] \qquad T \leq T_B \qquad \text{(Eq.3.16)}$$

Where,

I is a importance factor of structure and it is depend on the building class. The building class of a factory structure is 3 (BKS = 3) soaccording to Table 3.12, the importance factor of considered structure is 1 [34].

Table 3.12: Structure class and importance factor of structure [34].

Bina Kullanım Sınıfı	Binanın Kullanım Amacı	Bina Önem Katsayısı (*I*)
BKS = 1	**Deprem sonrası kullanımı gereken binalar, insanların uzun süreli ve yoğun olarak bulunduğu binalar, değerli eşyanın saklandığı binalar ve tehlikeli madde içeren binalar** **a)** Deprem sonrasında hemen kullanılması gerekli binalar (Hastaneler, dispanserler, sağlık ocakları, itfaiye bina ve tesisleri, PTT ve diğer haberleşme tesisleri, ulaşım istasyonları ve terminalleri, enerji üretim ve dağıtım tesisleri, vilayet, kaymakamlık ve belediye yönetim binaları, ilk yardım ve afet planlama istasyonları) **b)** Okullar, diğer eğitim bina ve tesisleri, yurt ve yatakhaneler, askeri kışlalar, cezaevleri, vb. **c)** Müzeler **d)** Toksik, patlayıcı, parlayıcı, vb. özellikleri olan maddelerin bulunduğu veya depolandığı binalar	1.5
BKS = 2	**İnsanların kısa süreli ve yoğun olarak bulunduğu binalar** Alışveriş merkezleri, spor tesisleri, sinema, tiyatro, konser salonları, ibadethaneler, vb.	1.2
BKS = 3	**Diğer binalar** BKS=1 ve BKS=2 için verilen tanımlara girmeyen diğer binalar (Konutlar, işyerleri, oteller, bina türü endüstri yapıları, vb.)	1.0

R is the behavior factor of load bearing system. It is determined according to Table 3.13.

The structure is high ductile steel frame system in both directions.

The structure is steel moment resisting frame in X direction so that R_X is 8 and D_X is 3 while it is braced with pinned connection in Y direction so that R_y is 5 and D_Y is 2.

Table 3.13: Load bearing behavior factor and strength equivalence ratio [34].

Bina Taşıyıcı Sistemi	Taşıyıcı Sistem Davranış Katsayısı *R*	Dayanım Fazlalığı Katsayısı *D*	İzin Verilen Bina Yükseklik Sınıfları BYS
C. ÇELIK BINA TAŞIYICI SISTEMLERI			
C1. Süneklik Düzeyi Yüksek Taşıyıcı Sistemler			
C11. Deprem etkilerinin tamamının moment aktaran *süneklik düzeyi yüksek* çelik çerçevelerle karşılandığı binalar	8	3	BYS ≥ 3
C12. Deprem etkilerinin tamamının *süneklik düzeyi yüksek* dışmerkez veya burkulması önlenmiş merkezi çaprazlı çelik çerçeveler tarafından karşılandığı binalar	8	2.5	BYS ≥ 2
C13. Deprem etkilerinin tamamının *süneklik düzeyi yüksek* merkezi çaprazlı çelik çerçeveler tarafından karşılandığı binalar	5	2	BYS ≥ 4
C14. Deprem etkilerinin moment aktaran *süneklik düzeyi yüksek* çelik çerçeveler ile *süneklik düzeyi yüksek* dışmerkez veya burkulması önlenmiş merkezi çaprazlı çelik çerçeveler veya *süneklik düzeyi yüksek* bağ kirişli (boşluklu) betonarme perdeler tarafından birlikte karşılandığı binalar (Bkz.4.3.4.5)	8	3	BYS ≥ 2
C15. Deprem etkilerinin moment aktaran *süneklik düzeyi yüksek* çelik çerçeveler ile *süneklik düzeyi yüksek* merkezi çaprazlı çelik çerçeveler veya *süneklik düzeyi yüksek* boşluksuz betonarme perdeler tarafından birlikte karşılandığı binalar (Bkz.4.3.4.5)	6	2.5	BYS ≥ 2
C16. Deprem etkilerinin tamamının çatı düzeyindeki bağlantıları mafsallı olan ve yüksekliği 12 m'yi geçmeyen *süneklik düzeyi yüksek* çelik kolonlar tarafından karşılandığı tek katlı binalar	4	2	–
C2. Süneklik Düzeyi Karma Taşıyıcı Sistemler (Bkz. 4.3.4.1, 4.3.4.6)			
C21. Deprem etkilerinin moment aktaran *süneklik düzeyi sınırlı* çelik çerçeveler ile *süneklik düzeyi yüksek* dışmerkez veya burkulması önlenmiş merkezi çaprazlı çelik çerçeveler veya *süneklik düzeyi yüksek* bağ kirişli (boşluklu) betonarme perdeler tarafından birlikte karşılandığı binalar (Bkz.4.3.1.2)	6	2.5	BYS ≥ 4
C22. Deprem etkilerinin moment aktaran *süneklik düzeyi sınırlı* çelik çerçeveler ile *süneklik düzeyi yüksek* merkezi çaprazlı çelik çerçeveler veya *süneklik düzeyi yüksek* boşluksuz betonarme perdeler tarafından birlikte karşılandığı binalar (Bkz.4.3.1.2)	5	2	BYS ≥ 4
C3. Süneklik Düzeyi Sınırlı Taşıyıcı Sistemler (Bkz. 4.3.4.1, 4.3.4.7)			
C31. Deprem etkilerinin tamamının moment aktaran *süneklik düzeyi sınırlı* çelik çerçevelerle karşılandığı binalar	4	2.5	BYS ≥ 7
C32. Deprem etkilerinin tamamının *süneklik düzeyi sınırlı* merkezi çaprazlı çelik çerçevelerle karşılandığı binalar	3	2	BYS = 8
C33. Deprem etkilerinin moment aktaran *süneklik düzeyi sınırlı* çelik çerçeveler ile *süneklik düzeyi sınırlı* merkezi çaprazlı çelik çerçeveler tarafından birlikte karşılandığı binalar	4	2	BYS ≥ 7

$T_x = T_y = 0.2$ swhich is lower than $T_B = 0.275$ s, as a result $R_{ax}(T)$ and $R_{ay}(T)$ is calculated as below:

$R_{ax}(T) = 3 + [\ (8 - 3) \text{ x } (0.2 / 0.275)\ ] = 6.64$

$R_{ay}(T) = 2 + [\ (5 - 2) \text{ x } (0.2 / 0.275)\ ] = 4.18$

And also the $S_{aRx}(T)$ and $S_{aRy}(T)$ is calculated as below:

$S_{aRx}(T) = 0.818 / 6.64 \approx 0.123$

$S_{aRy}(T) = 0.818 / 4.18 \approx 0.2$

3.4.2 Defining loads in SAP2000 program

The loads which are determined in the previous section are applied on members in SAP2000 program.

3.4.2.1 Dead load

Frame spacing is 6 m so covering load which have to be applied in SAP2000 program is equal to:

0.47 KN/m^2 x 6 m ≈ 2.94 KN/m (≈300 kg/m)

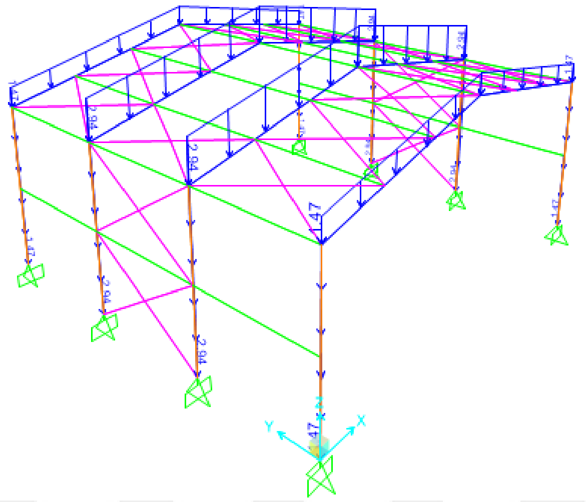


Figure 3.13: Applied covering loads in SAP2000 program (KN/m).

3.4.2.2 Roof live load

Frame spacing is 6 m so the roof live load which have to be applied in SAP2000 program is equal to:

0.78 KN/m^2 x 6 m = 4.7 KN/m (480 kg/m)

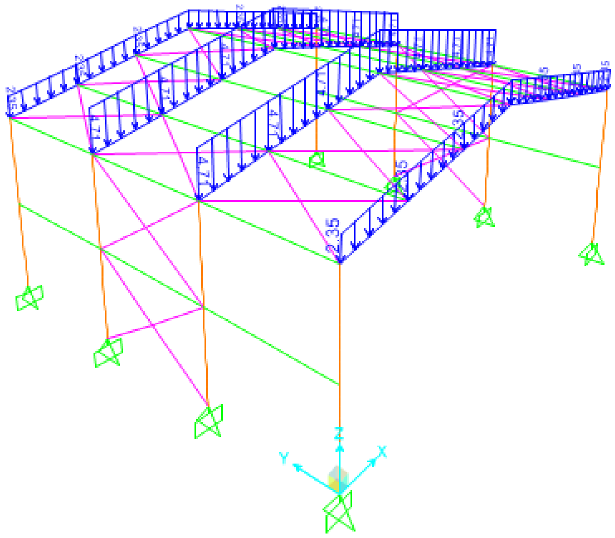


Figure 3.14: Applied roof live load in SAP2000 program (KN/m).

3.4.2.3 Snow load

Frame spacing is 6 m so the snow load which have to be applied in SAP2000 program is equal to:

0.75 KN/m^2 x 6 m = 4.5 KN/m (450 kg/m)

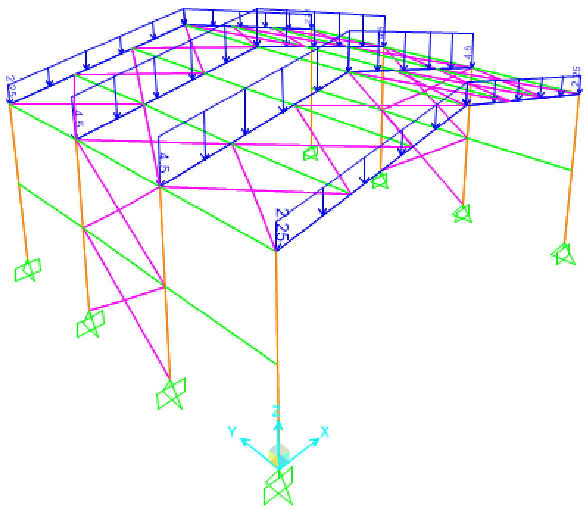


Figure 3.15: Applied snow load in SAP2000 program (KN/m).

3.4.2.4 Wind load

Wind load on roof and facade columns in X direction is applied as below:

Frame spacing of columns and rafters is 6 m so the pressure wind load and suction wind load on roof and columns which have to be applied in SAP2000 program are equal to:

Pressure wind load on columns = 0.4 KN/m^2 x 6 m ≈ 2.4 KN/m (246 kg/m)

Suction wind load on columns = 0.2 KN/m^2 x 6 m = 1.2 KN/m (123 kg/m)

Pressure wind load on rafters = -0.082 KN/m^2 x 6 m = -0.49 KN/m (- 50.4 Kg/m)

Suction wind load on rafters = -0.2 Kg/m^2 x 6 m = -1.2 KN/m (-122.4 Kg/m)

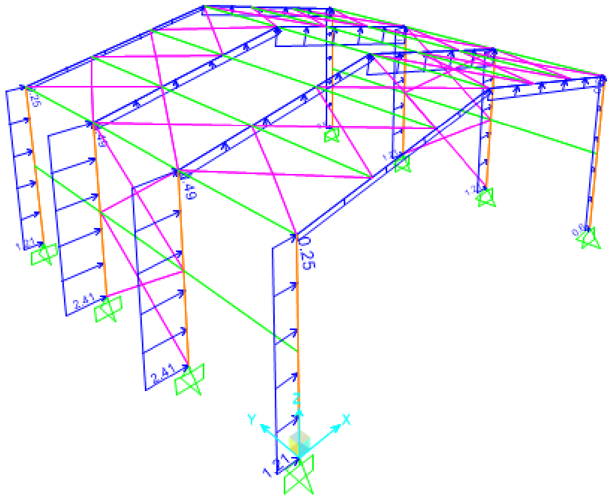


Figure 3.16: Applied wind load in X direction in SAP2000 program (KN/m).

Wind load on roof and facade column in Y direction is applied as below:

Frame spacing of columns in Y direction is 20 m so the pressure and suction wind load on columns which have to be applied in SAP2000 program are equal to:

Pressure wind load on columns = 0.4 x 10 = 4 KN/m (410 kg/m)

Suction wind load on columns = 0.2 x 10 = 2 KN/m (204 kg/m)

As it is shown in Figure 3.17, frame area of roof is 20 m^2(20 x 2 / 2 = 20 m^2) so the pressure and suction wind load on roof which have to be applied in SAP2000 program are equal to:

Pressure wind load on rafters = [(20 / 2) x 0.5 x 0.8] / 10.2 = 0.39 KN/m (40 kg/m)

Suction wind load on rafters = [(20 / 2) x 0.5 x 0.4] / 10.2 = 0.196 KN/m (20 kg/m)

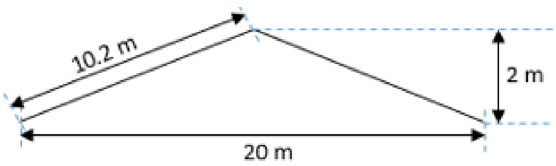


Figure 3.17: The considered roof area in calculating the wind load applied on the roof in Y direction.

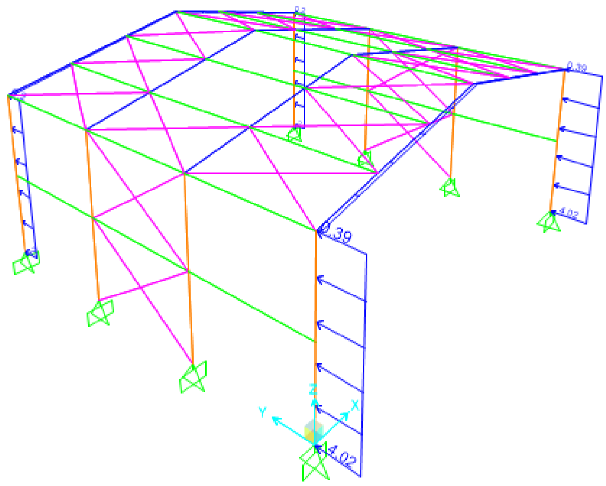


Figure 3.18: Applied wind load in Y direction in SAP2000 program (KN/m).

3.4.2.5 Earthquake load

Horizontal seismic load is automatically computed by SAP2000 program according to given parameters.

As it is determined in the 3.4.1.5 section of this thesis, live load participation factor (n) is 0.3.

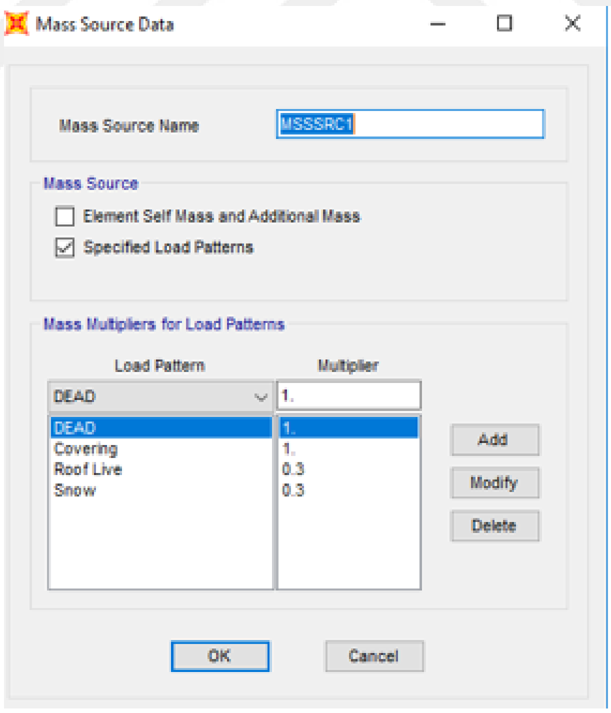


Figure 3.19: Defined seismic weight of structure in SAP2000 program.

The earthquake load should be shifted up to 5% of the length which is perpendicular to earthquake load direction form the center of the mass thuss the ECC (eccentricities) ratio should be %5 [34].

The required parameters to define earthquake load in X directionin SAP2000 programis determined as below:

Base shear coefficient (C) which is equal to $S_{aRx}(T)$ is 0.123.

Building height exponent (k) depends on the value of the structure period [82] which is defined by equation 3.17.

$K = 1$ $\qquad\qquad\qquad\qquad T \leq 0.5$

$K=2$ $\qquad\qquad\qquad\qquad T \geq 2.5$

(Eq.3.17)

K = Linearly interpolation between $k = 1$ and $k = 2$ $\qquad 0.5 < T < 2.5$

As a result, T_X is 0.2s so k_x is equal to 1.

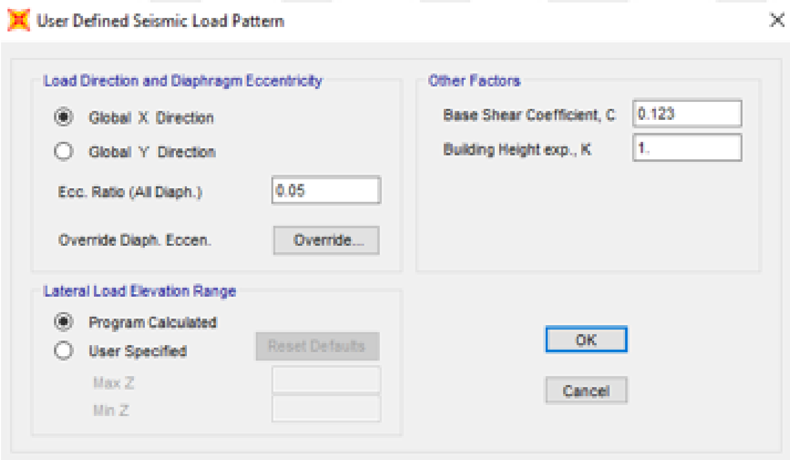


Figure 3.20: Defined parameters of X direction earthquake load in SAP2000 program.

The required parameters to define earthquake load in Y direction in SAP2000 program are determined as below:

Base shear coefficient (C) which is equal to $S_{aRy}(T)$ is 0.2.

T_y is equal to 0.2s so according to equation 3.17, K_y is equal to 1.

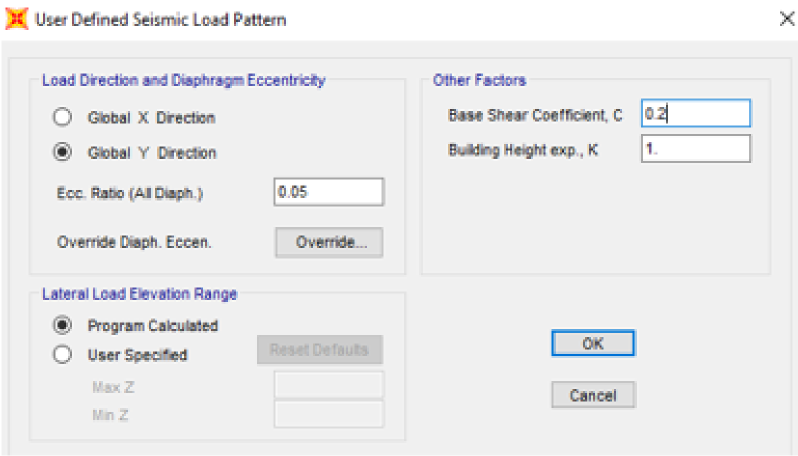


Figure 3.21: Defined parameters of Y direction earthquake load defined in SAP2000 program.

3.5 Load Combinations

There is load combinations needed for strength and displacement control of the computer model of the structure under acting loads. These combinations are given in the following.

3.5.1 Load combinations for strength control

The load combinations for strength control are [78]:

- 1.4G(X)
- 1.4G(Y)
- 1.2G + 0.5Qr(X)
- 1.2G + 0.5Qr(Y)
- 1.2G + 0.5S (X)
- 1.2G + 0.5S (Y)
- 1.2G + 1.6Qr ± 0.8Wx (X)
- 1.2G + 1.6Qr ± 0.8Wx (Y)
- 1.2G + 1.6Qr ± 0.8Wy (X)
- 1.2G + 1.6 Qr ± 0.8Wy (Y)
- 1.2G + 1.6S ± 0.8Wx (X)
- 1.2G + 1.6S ± 0.8Wx (Y)
- 1.2G + 1.6S ± 0.8Wy (X)
- 1.2G + 1.6S ± 0.8Wy (Y)

- 1.2G + 0.5Qr ± 1.6Wx (X)
- 1.2G + 0.5Qr ± 1.6Wx (Y)
- 1.2G + 0.5Qr ± 1.6Wy (X)
- 1.2G + 0.5Qr ± 1.6Wy (Y)
- 1.2G + 0.5S ± 1.6Wx (X)
- 1.2G + 0.5S ± 1.6Wx (Y)
- 1.2G + 0.5S ± 1.6Wy (X)
- 1.2G + 0.5S ± 1.6Wy (Y)
- 1.2G + 0.2S ± Ex ± 0.3 Ey
- 1.2G + 0.2S ± Ey ± 0.3 Ex
- 0.9G ± 1.6Wx (X)
- 0.9G ± 1.6Wx (Y)
- 0.9G ± 1.6Wy (X)
- 0.9G ± 1.6Wy (Y)
- 0.9G ± Ex ± 0.3Ey
- 0.9G ± Ey ± 0.3Ex

3.5.2 Load combinations for displacement control

The load combinations for displacement control by regarding the serviceability conditions are [78]:

- Vertical displacement of frame
  - DV- G + Qr
  - DV- G + 0.5S
- Horizontal displacement of frame
  - DH – G + 0.5Qr ± Wx
  - DH – G + 0.5Qr ± Wy
- Vertical displacement of apex connection [83]
  - DVA – G
  - DVA – G + Qr
  - DVA – G + 0.5S
  - DVA – G + Wx
  - DVA – G + Wy

3.6 Model Revising

Before analysing the model, the applied earthquake load have to be refined because the first earthquake calculation was done according to approximate period, and then approximate period should be refined after the analyzed model.

Refining of the applied earthquake load, the earthquake load should be checked considering 2 steps which are listed as below:

- Modes
- Base shear force

3.6.1 Modes

In Equivalent Seismic Load Method, the seismic mass of structure which contributes in earthquake in X direction and also in earthquake in Y direction has to be more than %75.

The modal participating mass ratios of model are shown in Table 3.14.

Table 3.14: The obtained modal participating mass ratios of model in SAP2000 program.

TABLE: Modal Participating Mass Ratios															
OutputCase	StepType	StepNum	Period	UX	UY	UZ	SumUX	SumUY	SumUZ	RX	RY	RZ	SumRX	SumRY	SumRZ
Text	Text	Unitless	Sec	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless
MODAL	Mode	1	0.552997	0.857	4.318E-08	8.411E-14	0.857	4.318E-08	8.411E-14	0.000000166	0.002643	9.524E-10	0.000000166	0.002643	9.524E-10
MODAL	Mode	2	0.356307	7.589E-12	3.965E-08	0.486	0.857	8.283E-08	0.486	2.345E-07	5.225E-13	7.665E-08	4.006E-07	0.002643	7.76E-08
MODAL	Mode	3	0.331672	0.000002251	0.127	6.428E-07	0.857	0.127	0.486	0.436	0.00001093	6.122E-12	0.436	0.002654	7.761E-08
MODAL	Mode	4	0.278771	1.946E-07	0.0003369	0.000000557	0.857	0.127	0.486	0.00007235	4.293E-08	3.711E-09	0.436	0.002654	8.132E-08
MODAL	Mode	5	0.27876	0.001704	3.975E-08	2.001E-09	0.858	0.127	0.486	2.164E-08	0.00124	0.00001773	0.436	0.003894	0.00001781
MODAL	Mode	6	0.278612	6.284E-09	0.00005669	0.001464	0.858	0.127	0.487	0.00002261	2.109E-08	6.765E-11	0.436	0.003894	0.00001781
MODAL	Mode	7	0.278449	0.0002542	3.931E-08	4.74E-11	0.858	0.127	0.487	1.586E-07	0.0003641	0.0001479	0.436	0.004258	0.0001657
MODAL	Mode	8	0.248979	0.000001241	0.291	9.23E-08	0.858	0.418	0.487	0.001018	0.000006862	6.273E-11	0.437	0.004265	0.0001657
MODAL	Mode	9	0.21109	3.409E-07	6.316E-09	0.000006193	0.858	0.418	0.487	4.606E-12	8.783E-08	0.641	0.437	0.004265	0.641
MODAL	Mode	10	0.204968	0.0000107	0.469	1.079E-11	0.858	0.888	0.487	0.018	0.0000858	8.271E-09	0.456	0.004351	0.641
MODAL	Mode	11	0.1803	1.376E-09	5.578E-10	0.002252	0.858	0.888	0.49	2.596E-10	1.659E-10	0.002601	0.456	0.004351	0.644
MODAL	Mode	12	0.16728	0.014	0.0002787	9.711E-11	0.872	0.888	0.49	0.001584	0.079	1.888E-07	0.457	0.083	0.644
MODAL	Mode	13	0.166983	4.557E-08	1.266E-12	0.0002169	0.872	0.888	0.49	8.859E-11	1.461E-08	0.083	0.457	0.083	0.727
MODAL	Mode	14	0.160552	0.0003643	0.000009085	6.291E-10	0.872	0.888	0.49	0.00203	0.002616	2.748E-10	0.459	0.086	0.727
MODAL	Mode	15	0.160296	1.009E-12	1.057E-13	0.004159	0.872	0.888	0.494	6.569E-11	3.946E-09	0.007218	0.459	0.086	0.734
MODAL	Mode	16	0.159289	0.000006948	0.0000251	8.108E-10	0.872	0.888	0.494	0.000004735	0.002865	2.615E-14	0.459	0.089	0.734
MODAL	Mode	17	0.159284	3.556E-11	6.009E-12	0.0005739	0.872	0.888	0.495	6.626E-12	3.175E-09	0.0001362	0.459	0.089	0.735
MODAL	Mode	18	0.159191	0.00003351	0.0001026	1.653E-14	0.873	0.888	0.495	0.0000357	0.002183	1.33E-11	0.459	0.091	0.735
MODAL	Mode	19	0.15915	1.651E-12	3.362E-12	0.0008574	0.873	0.888	0.495	1.11E-11	1.395E-10	0.00006731	0.459	0.091	0.735
MODAL	Mode	20	0.159103	0.0000124	0.000006152	1.898E-12	0.873	0.888	0.495	0.000003022	0.0008861	3.854E-11	0.459	0.092	0.735

U_x is the seismic mass of structure in X direction.

U_y is the seismic mass of structure in Y direction.

For mode 1 the U_X is 0.857 which is larger than 0.75. The period of structure in X direction (T_X) in this mode is 0.55 s.

For mode 10 the U_Y is 0.49 which is the maximum value of seismic mass of structure in Y direction is smaller than 0.75. The period of structure in Y direction (T_y) in this mode is 0.2 s.

So the earthquake load in X direction is recalculated according to the revised structure period in X direction (T_x) but the earthquake load in Y direction does not revised because the structure period in Y direction dose not changed.

The earthquake load in X direction is recalculated as below:

T_x is 0.55 s which is between the T_B = 0.275s and T_L = 6s boundaries for period. As a result,the reduced design spectral acceleration in X direction, $S_{aRx}(T)$, is recalculated as below:

$S_{aex}(T) = S_{D1} / T_x = 0.225 / 0.55 = 0.41$

$R_{ax}(T) = R_x / I = 8$

$S_{aRx}\ (T) = S_{aex}(T) / R_{ax}(T) = 0.41 / 8 \approx 0.1$

And also, building height exponent in X direction, K_x, is recalculated as below:

$K = 1 + (0.05 \text{ x } 0.5) = 1.025$

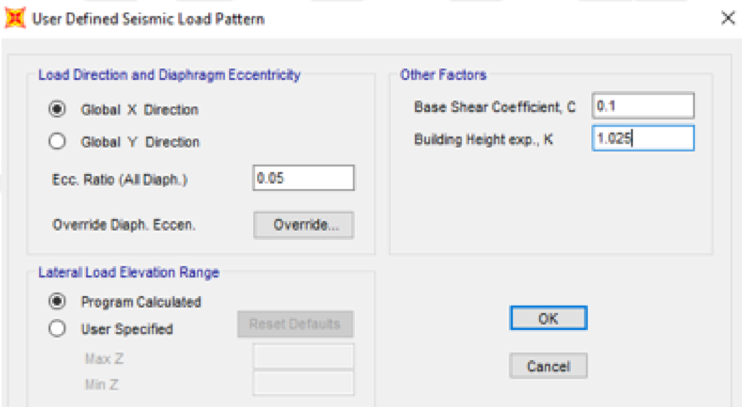


Figure 3.22: Revised parameters of X direction earthquake load in SAP2000 program.

3.6.2 Base shear force

The base shear force which is obtained by the SAP2000 program was compared with calculated base shear force in order to revise it if needed.

The base shear reactions of structure in different loadings are shown in Table 3.15.

Table 3.15: The created base shear reactions of structure in SAP2000 program.

TABLE: Base Reactions							
OutputCase	CaseType	GlobalFX	GlobalFY	GlobalFZ	GlobalMX	GlobalMY	GlobalMZ
Text	Text	KN	KN	KN	KN-cm	KN-cm	KN-cm
DEAD	LinStatic	-7.533E-13	-1.366E-14	148.209	133388.22	-148209.14	-1.953E-10
Covering	LinStatic	-1.193E-12	-5.742E-14	321.231	289108.12	-321231.25	-5.222E-10
Roof Live	LinStatic	-1.887E-12	-9.058E-14	288.025	259222.29	-288024.77	-9.494E-10
Snow	LinStatic	-1.648E-12	-6.076E-14	270.023	243020.9	-270023.22	-8.79E-10
Eathquake X	LinStatic	-63.145	3.611E-13	7.722E-13	-2.747E-10	-53288.73	56828.37
Eathquake Y	LinStatic	6.801E-13	-126.291	-4.46E-15	106423.25	4.138E-10	-126291

The values of global F_Z show the created base shear force of structure in different related loadings.

Base shear reaction of structure under dead load (G) = F_z(DEAD) + F_z(Covering)

F_z(DEAD) = 148.2 KN

Fz(Covering) = 321.23 KN

So the G is calculated az below:

G = 148.2 + 321.23 = 469.43 KN

F_z(Roof Live) = F_z(Qr) = 288 KN

F_z(Snow) = F_z(S) = 270 KN

Seismic weight of structure is equal to:

Wi = G + nQr + nS = 469.43 + 0.3 x (288 + 270) = 636.83 KN

So the earthquake load in X and Y directions are calculated as below respectively:

$F_{zx} = C_x \times W_i = 0.1 \times 636.83 = 63.68$ KN

$F_{zy} = C_y \times W_i = 0.2 \times 636.83 = 127.4$ KN

And as it is shown in Table 3.15, the base shear reactions of structure in X and Y directions which are obtained from SAP2000 program are equal to values seen below, respectively:

F_x(Earthquake X) = 63.14 KN

F_y(Earthquake Y) = 126.3 KN

So the obtained base shear reactions of structure from SAP2000 program are approximately equal to the calculated base shear reactions of structure and there is no need to revise the applied earthquake loads.

3.7 Analysis Results of Model

The modeled structure was checked according to 3 categories as listed below:

- Strength
- Displacement
- Drift

3.7.1 Strength

Cold forming effects the strength of sections. The strength of elements are controlled in CFS V8.0. The createdinternal forces due to loading in structural members are obtained from SAP2000 program.

- Columns

The label of columns and the created maximum stress ratio of columns are shown in Figure 3.23.

The maximum stress ratio is created on column 48 and it is equal to 0.659.

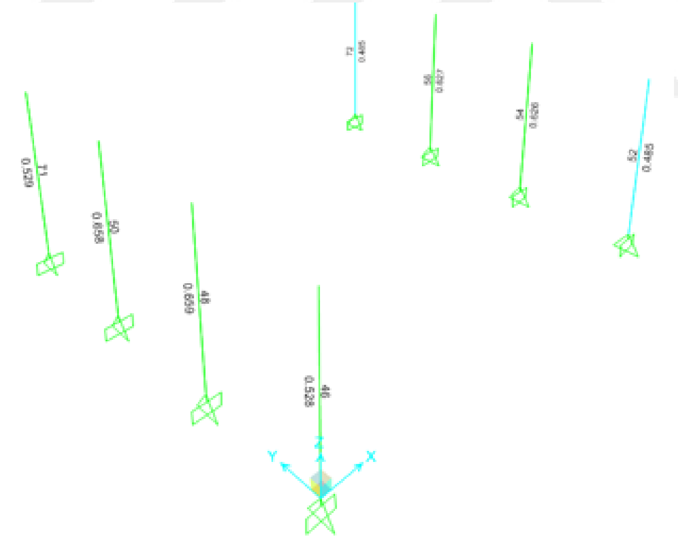


Figure 3.23: The label and the created maximum stress ratio of columns in SAP2000 program.

The analysis result summary of column 48 in SAP2000 program is given in Figure 3.24.

```
AISC 360-16 STEEL SECTION CHECK  (Summary for Combo and Station)
Units  : Kgf, m, C

Frame : 48        X Mid: 0.000      Combo: 1.2G+1.6Qr-0.8Wx Design Type: Column
Length: 8.000     Y Mid: 6.000      Shape: 2C-600x110x40x4  Frame Type: SMF
Loc   : 8.000     Z Mid: 4.000      Class: Non-Compact      Princpl Rot: 0.000 degrees

Provision: LRFD   Analysis: Direct Analysis
D/C Limit=0.950   2nd Order: General 2nd Order       Reduction: Tau-b Fixed
AlphaPr/Py=0.048  AlphaPr/Pe=0.071  Tau_b=1.000      EA factor=0.800   EI factor=0.800

PhiB=0.900        PhiC=0.900        PhiTY=0.900       PhiTF=0.750
PhiS=0.900        PhiS-RI=1.000     PhiST=0.900

A=0.007           I33=3.392E-04     r33=0.219         S33=0.001         Av3=0.002
J=0.000           I22=1.384E-05     r22=0.044         S22=1.258E-04     Av2=0.005
E=2.039E+10       fy=36199924.8     Ry=1.250          z33=0.001
RLLF=1.000        Fu=52005525.8                       z22=1.685E-04


STRESS CHECK FORCES & MOMENTS (Combo 1.2G+1.6Qr-0.8Wx (Y))
    Location             Pu        Mu33        Mu22         Vu2         Vu3          Tu
    8.000        -12343.290   29399.218       0.025   -5662.311      -1.381      -0.009
PMM DEMAND/CAPACITY RATIO   (H1-1b)
    D/C Ratio:    0.848 = 0.050 + 0.798 + 0.000
                        = (1/2)(Pr/Pc) + (Mr33/Mc33) + (Mr22/Mc22)

AXIAL FORCE & BIAXIAL MOMENT DESIGN   (H1-1b)
    Factor                L          K1          K2          B1          B2          Cm
    Major Bending     1.000       1.000       1.000       1.000       1.000       1.000
    Minor Bending     0.500       1.000       1.000       1.000       1.000       1.000

                       Lltb        Kltb          Cb
    LTB               0.500       1.000       2.133

                         Pu     phi*Pnc     phi*Pnt
                      Force    Capacity    Capacity
    Axial        -12343.290  124531.538  230405.282

                         Mu      phi*Mn      phi*Mn      phi*Mn
                     Moment    Capacity      No LTB        Cb=1
    Major Moment  29399.218   36836.135   36836.135   36836.135
    Minor Moment      0.025    4100.083

SHEAR CHECK
                         Vu      phi*Vn      Stress      Status
                      Force    Capacity       Ratio       Check
    Major Shear    5662.311   91987.381       0.062          OK
    Minor Shear       1.381   34572.275   3.993E-05          OK
```

Figure 3.24: The analysis result summary of column 48 in SAP2000 program.

The maximum internal forces are created in ‘ 1.2G + 1.6Qr – 0.8Wx ‘ loading combination.

The created internal forces are specified.

These data are inputted to CFS program in order to control the strength of column sections.

The analysis result summary of column section in CFS V8.0 program is given in Figure 3.25.

Section Inputs

```
Material: [S350GD+Z]
Apply cold work of forming strength increase.
Apply inelastic reserve strength increase.
Modulus of Elasticity, E          200000 MPa
Yield Strength, Fy                   350 MPa
Tensile Strength, Fu                 420 MPa
Torsion Constant Override, J           0 mm^4
Warping Constant Override, Cw  1,1857e12 mm^6
Connector Spacing                      0 mm

Right Channel, Thickness 4 mm
Placement of Part from Origin:
X to left edge                   0 mm
Y to center of gravity           0 mm
Outside dimensions, Open shape
        Length      Angle     Radius  Web        k    Hole Size   Distance
         (mm)       (deg)      (mm)            Coef.      (mm)       (mm)
  1      60,00    270,000    4,0000 None    0,000       0,00      30,00
  2     160,00    180,000    4,0000 Single  0,000       0,00      80,00
  3     600,00     90,000    4,0000 Double  0,000       0,00     300,00
  4     160,00      0,000    4,0000 Single  0,000       0,00      80,00
  5      60,00    -90,000    4,0000 None    0,000       0,00      30,00
Left Channel, Thickness 4 mm
Placement of Part from Origin:
X to right edge                  0 mm
Y to center of gravity           0 mm
Outside dimensions, Open shape
        Length      Angle     Radius  Web        k    Hole Size   Distance
         (mm)       (deg)      (mm)            Coef.      (mm)       (mm)
  1      60,00    -90,000    4,0000 None    0,000       0,00      30,00
  2     160,00      0,000    4,0000 Single  0,000       0,00      80,00
  3     600,00     90,000    4,0000 Double  0,000       0,00     300,00
  4     160,00    180,000    4,0000 Single  0,000       0,00      80,00
  5      60,00    270,000    4,0000 None    0,000       0,00      30,00
```

Member Check - AISI S100-16/S1-18, US, LRFD

```
Material Type: [S350GD+Z], Fy=350 MPa
Design Parameters:
Lx        8,0000 m      Ly        4,0000 m     Lt        4,0000 m
Kx        1,0000        Ky        1,0000       Kt        1,0000
Cbx       1,0000        Cby       1,0000       ex        0,0000 mm
Cmx       1,0000        Cmy       1,0000       ey        0,0000 mm
Braced Flange: None     kφ             0 kgf
Red. Factor, R: 0       Lm        8,0000 m

Loads:            P         Mx         Vy        My         Vx
                (kgf)     (kN-m)     (kgf)     (kN-m)     (kgf)
Entered         12344     288,31      5663      0,00          0
Applied         12344     291,01      5663      0,00          0
Strength        96720     336,18     20495     70,40      46871

Effective section properties at applied loads:
Ae         8081,0 mm^2   Ixe     420323400 mm^4   Iye      43011970 mm^4
                         Sxe(t)    1396179 mm^3   Sye(l)     268825 mm^3
                         Sxe(b)    1406011 mm^3   Sye(r)     268825 mm^3

Interaction Equations
Eq. H1.2-1    (P, Mx, My)  0,128 + 0,866 + 0,000 = 0,993 <= 1.0
Eq. H2-1         (Mx, Vy)     Sqrt(0,546 + 0,076)= 0,789 <= 1.0
Eq. H2-1         (My, Vx)     Sqrt(0,000 + 0,000)= 0,000 <= 1.0
```

Figure 3.25:The analysis result summary of column 48 in CFS V8.0 program.

- Rafters

The label of rafters and the created maximum stress ratio of rafters are shown in Figure 3.26.

The maximum stress ratio is created on rafter 29. It is qual to 0.604.

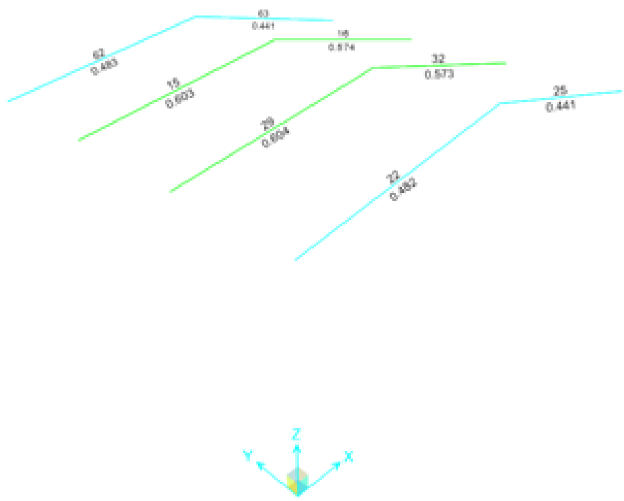
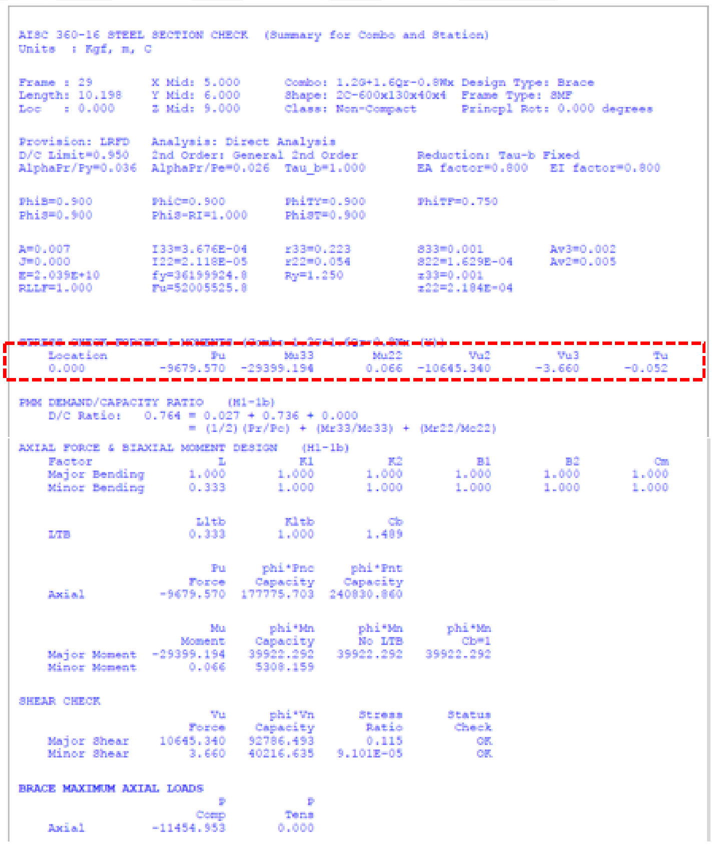


Figure 3.26: The label and thecreated maximum stress ratio of rafters in SAP2000 program.

The analysis result summary of rafter 29 in SAP2000 program is given in Figure 3.27.



```
AISC 360-16 STEEL SECTION CHECK  (Summary for Combo and Station)
Units  : Kgf, m, C

Frame : 29          X Mid: 5.000       Combo: 1.2G+1.6Qr-0.8Wx Design Type: Brace
Length: 10.198      Y Mid: 6.000       Shape: 2C-600x130x40x4  Frame Type: SMF
Loc   : 0.000       Z Mid: 9.000       Class: Non-Compact      Princpl Rot: 0.000 degrees

Provision: LRFD     Analysis: Direct Analysis
D/C Limit=0.950     2nd Order: General 2nd Order        Reduction: Tau-b Fixed
AlphaPr/Py=0.036    AlphaPr/Pe=0.026   Tau_b=1.000      EA factor=0.800   EI factor=0.800

PhiB=0.900          PhiC=0.900         PhiTY=0.900      PhiTF=0.750
PhiS=0.900          PhiS-RI=1.000      PhiST=0.900

A=0.007             I33=3.676E-04      r33=0.223        S33=0.001         Av3=0.002
J=0.000             I22=2.118E-05      r22=0.054        S22=1.629E-04     Av2=0.005
E=2.039E+10         fy=36199924.8      Ry=1.250         z33=0.001
RLLF=1.000          Fu=52005525.8                       z22=2.184E-04


STRESS CHECK FORCES & MOMENTS (Combo 1.2G+1.6Qr-0.8Wx (X))
    Location            Pu        Mu33        Mu22         Vu2        Vu3         Tu
    0.000          -9679.570  -29399.194     0.066  -10645.340     -3.660     -0.052

PMM DEMAND/CAPACITY RATIO   (H1-1b)
    D/C Ratio:   0.764 = 0.027 + 0.736 + 0.000
                       = (1/2)(Pr/Pc) + (Mr33/Mc33) + (Mr22/Mc22)
AXIAL FORCE & BIAXIAL MOMENT DESIGN   (H1-1b)
    Factor                 L          K1          K2          B1          B2          Cm
    Major Bending      1.000       1.000       1.000       1.000       1.000       1.000
    Minor Bending      0.333       1.000       1.000       1.000       1.000       1.000

                        Lltb        Kltb          Cb
    LTB                0.333       1.000       1.489

                          Pu     phi*Pnc     phi*Pnt
                       Force    Capacity    Capacity
    Axial          -9679.570  177775.703  240830.860

                          Mu      phi*Mn      phi*Mn      phi*Mn
                      Moment    Capacity      No LTB        Cb=1
    Major Moment  -29399.194   39922.292   39922.292   39922.292
    Minor Moment       0.066    5308.159

SHEAR CHECK
                          Vu      phi*Vn      Stress      Status
                       Force    Capacity       Ratio       Check
    Major Shear    10645.340   92786.493       0.115          OK
    Minor Shear        3.660   40216.635   9.101E-05          OK

BRACE MAXIMUM AXIAL LOADS
                           P           P
                        Comp        Tens
    Axial         -11454.953       0.000
```

Figure 3.27: The analysis result summary of rafter 29 in SAP2000 program.

The maximum internal forces are created in ‘ 1.2G + 1.6Qr – 0.8Wx ‘ loading combination.

The created internal forces are specified.

These data are inputted to CFS program in order to control the strength of rafter sections.

The analysis result summary of rafter section in CFS V8.0 program is given in Figure 3.28.

Section Inputs

```
Material: [S350GD+Z]
Apply cold work of forming strength increase.
Apply inelastic reserve strength increase.
Modulus of Elasticity, E           200000 MPa
Yield Strength, Fy                    350 MPa
Tensile Strength, Fu                  420 MPa
Torsion Constant Override, J            0 mm^4
Warping Constant Override, Cw  1,8198e12 mm^6
Connector Spacing                       0 mm

Right Channel, Thickness 4 mm
Placement of Part from Origin:
X to left edge                  0 mm
Y to center of gravity          0 mm
Outside dimensions, Open shape
       Length       Angle      Radius  Web        k     Hole Size    Distance
        (mm)        (deg)       (mm)             Coef.      (mm)        (mm)
  1    60,00      270,000     4,0000 None     0,000       0,00       30,00
  2   150,00      180,000     4,0000 Single   0,000       0,00       75,00
  3   650,00       90,000     4,0000 Double   0,000       0,00      325,00
  4   150,00        0,000     4,0000 Single   0,000       0,00       75,00
  5    60,00      -90,000     4,0000 None     0,000       0,00       30,00

Left Channel, Thickness 4 mm
Placement of Part from Origin:
X to right edge                 0 mm
Y to center of gravity          0 mm
Outside dimensions, Open shape
       Length       Angle      Radius  Web        k     Hole Size    Distance
        (mm)        (deg)       (mm)             Coef.      (mm)        (mm)
  1    60,00      -90,000     4,0000 None     0,000       0,00       30,00
  2   150,00        0,000     4,0000 Single   0,000       0,00       75,00
  3   650,00       90,000     4,0000 Double   0,000       0,00      325,00
  4   150,00      180,000     4,0000 Single   0,000       0,00       75,00
  5    60,00      270,000     4,0000 None     0,000       0,00       30,00
```

Member Check - AISI S100-16/S1-18, US, LRFD

```
Material Type: [S350GD+Z], Fy=350 MPa
Design Parameters:
Lx        20,000 m      Ly         3,300 m     Lt         3,320 m
Kx        1,0000        Ky        1,0000       Kt        1,0000
Cbx       1,0000        Cby       1,0000       ex        0,0000 mm
Cmx       1,0000        Cmy       1,0000       ey        0,0000 mm
Braced Flange: None     kϕ             0 kgf
Red. Factor, R: 0       Lm        20,000 m

Loads:          P        Mx        Vy        My        Vx
              (kgf)    (kN-m)    (kgf)     (kN-m)    (kgf)
Entered        9680    288,31    10645      0,00         0
Applied        9680    299,96    10645      0,00         0
Strength      91783    359,03    18879     65,04     43616

Effective section properties at applied loads:
Ae        7548,0 mm^2   Ixe     473996700 mm^4   Iye     36732560 mm^4
                        Sxe(t)    1389221 mm^3   Sye(l)    244884 mm^3
                        Sxe(b)    1534943 mm^3   Sye(r)    244884 mm^3

Interaction Equations
Eq. H1.2-1    (P, Mx, My)  0,105 + 0,835 + 0,000 = 0,941 <= 1.0
Eq. H2-1         (Mx, Vy)     Sqrt(0,607 + 0,318)= 0,962 <= 1.0
Eq. H2-1         (My, Vx)     Sqrt(0,000 + 0,000)= 0,000 <= 1.0
```

Figure 3.28: The analysis result summary of rafter 29 in CFS V8.0 program.

- Stability beams

The label of stability beams and the created maximum stress ratio of them are shown in Figure 3.29.

The maximum stress ratio is created on stability beam 45. It is qual to 0.562.

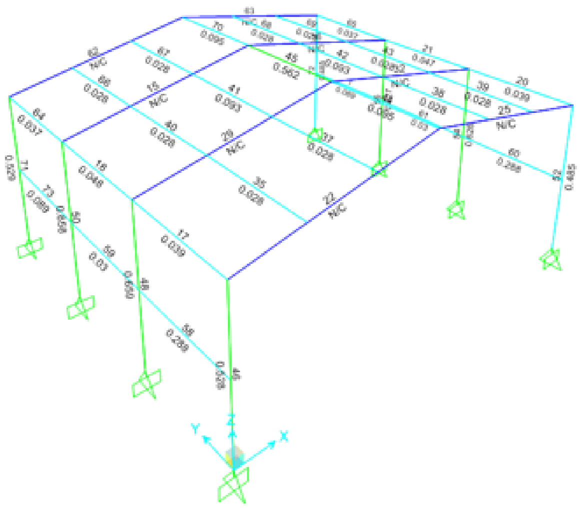


Figure 3.29: The label and thecreated maximum stress ratio of stability beams in SAP2000 program.

The analysis result summary of stability beam 45 in SAP2000 program is given in Figure 3.30.

```
AISC 360-16 STEEL SECTION CHECK  (Summary for Combo and Station)
Units  : Kgf, m, C

Frame : 45         X Mid: 10.000      Combo: 1.2G+1.6Qr-0.8Wx Design Type: Beam
Length: 6.000      Y Mid: 9.000       Shape: C-180x100x40x4   Frame Type: SMF
Loc   : 3.000      Z Mid: 10.000      Class: Non-Compact      Princpl Rot: 0.000 degrees

Provision: LRFD    Analysis: Direct Analysis
D/C Limit=0.950    2nd Order: General 2nd Order       Reduction: Tau-b Fixed
AlphaPr/Py=0.100   AlphaPr/Pe=0.428  Tau_b=1.000      EA factor=0.800   EI factor=0.800

PhiB=0.900         PhiC=0.900         PhiTY=0.900      PhiTF=0.750
PhiS=0.900         PhiS-RI=1.000      PhiST=0.900

A=0.002            I33=9.255E-06      r33=0.072        S33=1.028E-04     Av3=7.807E-04
J=0.000            I22=2.709E-06      r22=0.039        S22=4.455E-05     Av2=8.298E-04
E=2.039E+10        fy=36607811.3      Ry=1.072         z33=1.196E-04
RLLF=1.000         Fu=42828080.1                       z22=6.185E-05

STRESS CHECK FORCES & MOMENTS (Combo 1.2G+1.6Qr-0.8Wx (X))
    Location            Pu        Mu33        Mu22        Vu2         Vu3          Tu
    3.000         -6481.821      75.285      -0.151  -9.150E-05       0.005   3.340E-04

PMM DEMAND/CAPACITY RATIO   (H1-1a)
    D/C Ratio:   0.562 = 0.542 + 0.020 + 0.000
                       = (Pr/Pc) + (8/9)(Mr33/Mc33) + (8/9)(Mr22/Mc22)
AXIAL FORCE & BIAXIAL MOMENT DESIGN   (H1-1a)
    Factor               L          K1          K2          B1          B2          Cm
    Major Bending    1.000       1.000       1.000       1.000       1.000       1.000
    Minor Bending    1.000       1.000       1.000       1.000       1.000       1.000

                      Lltb        Kltb          Cb
    LTB              1.000       1.000       1.136

                        Pu     phi*Pnc     phi*Pnt
                     Force    Capacity    Capacity
    Axial        -6481.821   11955.683   57047.003

                        Mu      phi*Mn      phi*Mn      phi*Mn
                    Moment    Capacity      No LTB        Cb=1
    Major Moment    75.285    3388.138    3388.138    3388.138
    Minor Moment    -0.151    1467.781

SHEAR CHECK
                        Vu      phi*Vn      Stress      Status
                     Force    Capacity       Ratio       Check
    Major Shear  9.150E-05   16404.107       0.000          OK
    Minor Shear      0.005   15432.261       0.000          OK

CONNECTION SHEAR FORCES FOR BEAMS
                    VMajor      VMajor
                      Left       Right
    Major (V2)      58.555      58.557
```

Figure 3.30: The analysis result summary of stability beam 45 in SAP2000 program.

The maximum internal forces are created in ‘1.2G + 1.6Qr – 0.8Wx ‘loading combination.

The created internal forces are specified.

These data are inputted to CFS program in order to control the strength of stability beam sections.

The analysis result summary of stability beam section in CFS V8.0 program is given in Figure 3.31.

```
Section Inputs

Material: [S350GD+Z]
Apply cold work of forming strength increase.
Apply inelastic reserve strength increase.
Modulus of Elasticity, E            200000 MPa
Yield Strength, Fy                     350 MPa
Tensile Strength, Fu                   420 MPa
Torsion Constant Override, J             0 mm^4
Warping Constant Override, Cw            0 mm^6

Stiffened Channel, Thickness 4 mm
Placement of Part from Origin:
X to center of gravity             0 mm
Y to center of gravity             0 mm
Outside dimensions, Open shape
        Length      Angle     Radius  Web        k   Hole Size   Distance
          (mm)      (deg)       (mm)          Coef.       (mm)       (mm)
  1      40,00    270,000     4,0000 None    0,000       0,00      20,00
  2     100,00    180,000     4,0000 Single  0,000       0,00      50,00
  3     180,00     90,000     4,0000 Cee     0,000       0,00      90,00
  4     100,00      0,000     4,0000 Single  0,000       0,00      50,00
  5      40,00    -90,000     4,0000 None    0,000       0,00      20,00

Member Check - AISI S100-16/S1-18, US, LRFD

Material Type: [S350GD+Z], Fy=350 MPa
Design Parameters:
Lx         6,0000 m      Ly         6,0000 m      Lt         6,0000 m
Kx         1,0000        Ky         1,0000        Kt         1,0000
Cbx        1,0000        Cby        1,0000        ex         0,0000 mm
Cmx        1,0000        Cmy        1,0000        ey         0,0000 mm
Braced Flange: None      kϕ             0 kgf
Red. Factor, R: 0        Lm         6,0000 m

Loads:              P         Mx         Vy         My         Vx
                (kgf)     (kN-m)      (kgf)     (kN-m)      (kgf)
Entered          7300      0,700          0      0,000          0
Applied          7300      0,820          0      0,000          0
Strength         8631     15,107      13345      7,196      13671

Effective section properties at applied loads:
Ae         1734,8 mm^2   Ixe        8927094 mm^4  Iye        2604299 mm^4
                         Sxe(t)       99190 mm^3  Sye(l)       66893 mm^3
                         Sxe(b)       99190 mm^3  Sye(r)       42646 mm^3

Interaction Equations
Eq. H1.2-1    (P, Mx, My)  0,846 + 0,054 + 0,000 = 0,900 <= 1.0
Eq. H2-1         (Mx, Vy)     Sqrt(0,001 + 0,000)= 0,025 <= 1.0
Eq. H2-1         (My, Vx)     Sqrt(0,000 + 0,000)= 0,000 <= 1.0
```

Figure 3.31: The analysis result summary of stability beam 45 in CFS V8.0 program.

- Diagonals

The label of diagonals and the created maximum stress ratio of them are shown in Figure 3.32.

The maximum stress ratio is created on diagonal 77. It is qual to 0.607.

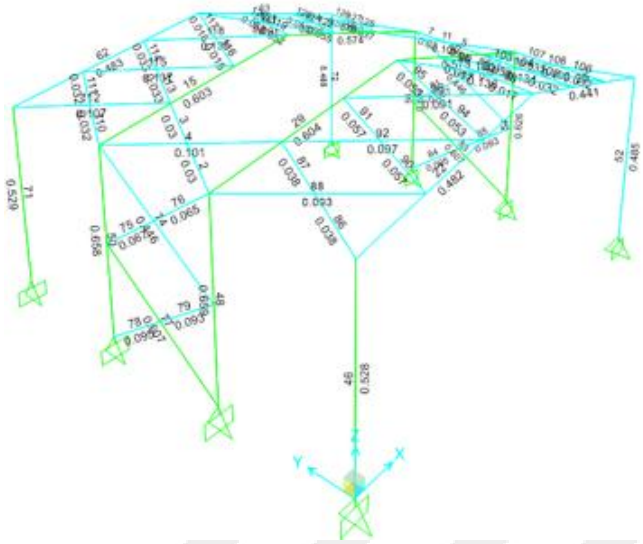


Figure 3.32: The label and thecreated maximum stress ratio of diagonals in SAP2000 program.

The analysis result summary of diagonal 77 in SAP2000 program is given in Figure 3.33.

```
AISC 360-16 STEEL SECTION CHECK  (Summary for Combo and Station)
Units  : Kgf, m, C

Frame : 77          X Mid: 0.000       Combo: 1.2G+0.5Qr-1.6Wy Design Type: Brace
Length: 7.211       Y Mid: 9.000       Shape: C-160x100x40x4   Frame Type: SMF
Loc   : 3.606       Z Mid: 2.000       Class: Non-Compact      Princpl Rot: 0.000 degrees

Provision: LRFD     Analysis: Direct Analysis
D/C Limit=0.950     2nd Order: General 2nd Order        Reduction: Tau-b Fixed
AlphaPr/Py=0.077    AlphaPr/Pe=0.474   Tau_b=1.000      EA factor=0.800    EI factor=0.800

PhiB=0.900          PhiC=0.900         PhiTY=0.900      PhiTF=0.750
PhiS=0.900          PhiS-RI=1.000      PhiST=0.900

A=0.002             I33=7.039E-06      r33=0.064        S33=8.799E-05      Av3=7.807E-04
J=0.000             I22=2.593E-06      r22=0.039        S22=4.391E-05      Av2=7.657E-04
E=2.039E+10         fy=36607811.3      Ry=1.072         z33=1.022E-04
RLLF=1.000          Fu=42828080.1                       z22=5.981E-05


STRESS CHECK FORCES & MOMENTS (Combo 1.2G+0.5Qr-1.6Wy (X))
    Location              Pu          Mu33          Mu22           Vu2           Vu3            Tu
    3.606          -4758.065       -21.395         0.084        29.495         4.688        -0.016

PMM DEMAND/CAPACITY RATIO    (H1-1a)
    D/C Ratio:    0.607 = 0.601 + 0.007 + 0.000
                        = (Pr/Pc) + (8/9)(Mr33/Mc33) + (8/9)(Mr22/Mc22)

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H1-1a)
    Factor                   L            K1            K2            B1            B2            Cm
    Major Bending        0.500         1.000         1.000         1.000         1.000         1.000
    Minor Bending        1.000         1.000         1.000         1.000         1.000         1.000


                          Lltb          Kltb            Cb
    LTB                  1.000         1.000         1.316


                            Pu      phi*Pnc       phi*Pnt
                         Force     Capacity      Capacity
    Axial            -4758.065     7922.698     54477.318


                            Mu       phi*Mn        phi*Mn        phi*Mn
                        Moment     Capacity        No LTB          Cb=1
    Major Moment       -21.395     2898.847      2898.847      2898.847
    Minor Moment         0.084     1446.682

SHEAR CHECK
                            Vu       phi*Vn        Stress        Status
                         Force     Capacity         Ratio         Check
    Major Shear         29.495    15136.179         0.002            OK
    Minor Shear          4.688    15432.142         0.000            OK
```

Figure 3.33: The analysis result summary of diagonal 77 in SAP2000 program.

The maximum internal forces are created in ‘ 1.2G + 0.5Qr – 1.6Wy ‘ loading combination.

The created internal forces are specified.

These data are inputted to CFS program in order to control the strength of diagonal sections.

The analysis result summary of diagonal section in CFS V8.0 program is given in Figure 3.34.

Section Inputs

```
Material: [S350GD+Z]
Apply cold work of forming strength increase.
Apply inelastic reserve strength increase.
Modulus of Elasticity, E           200000 MPa
Yield Strength, Fy                    350 MPa
Tensile Strength, Fu                  420 MPa
Torsion Constant Override, J            0 mm^4
Warping Constant Override, Cw           0 mm^6

Stiffened Channel, Thickness 4 mm
Placement of Part from Origin:
X to center of gravity            0 mm
Y to center of gravity            0 mm
Outside dimensions, Open shape
        Length      Angle      Radius  Web       k    Hole Size   Distance
         (mm)       (deg)       (mm)            Coef.    (mm)        (mm)
  1     40,00     270,000     4,0000 None     0,000     0,00       20,00
  2    100,00     180,000     4,0000 Single   0,000     0,00       50,00
  3    160,00      90,000     4,0000 Cee      0,000     0,00       80,00
  4    100,00       0,000     4,0000 Single   0,000     0,00       50,00
  5     40,00     -90,000     4,0000 None     0,000     0,00       20,00
```

Member Check - AISI S100-16/S1-18, US, LRFD

```
Material Type: [S350GD+Z], Fy=350 MPa
Design Parameters:
Lx        7,2110 m      Ly        7,2110 m      Lt        7,2110 m
Kx        1,0000        Ky        1,0000        Kt        1,0000
Cbx       1,0000        Cby       1,0000        ex        0,0000 mm
Cmx       1,0000        Cmy       1,0000        ey        0,0000 mm
Braced Flange: None     kφ            0 kgf
Red. Factor, R: 0       Lm        7,2110 m

Loads:            P         Mx         Vy         My         Vx
                (kgf)     (kN-m)     (kgf)     (kN-m)      (kgf)
Entered          4953      0,203          0     0,000          0
Applied          4953      0,250          0     0,000          0
Strength         5836     10,338      11718     5,106      13671

Effective section properties at applied loads:
Ae        1654,8 mm^2   Ixe      6780093 mm^4   Iye       2489797 mm^4
                        Sxe(t)     84751 mm^3   Sye(l)      61147 mm^3
                        Sxe(b)     84751 mm^3   Sye(r)      41999 mm^3

Interaction Equations
Eq. H1.2-1    (P, Mx, My)  0,849 + 0,024 + 0,000 = 0,873 <= 1.0
Eq. H2-1          (Mx, Vy)     Sqrt(0,000 + 0,000)= 0,009 <= 1.0
Eq. H2-1          (My, Vx)     Sqrt(0,000 + 0,000)= 0,000 <= 1.0
```

Figure 3.34: The analysis result summary of diagonal 77 in CFS V8.0 program.

3.7.2 Displacement

The deflection of members depend on loads, length of members, moment of inertia and the modulus of elasticity. These items do not change when the sections are hot rolled or cold formed so the displacement control of structure can be done by the analysis results of SAP200 program.

The displacement limits of cold formed portal frame are given in Table 3.16 [83].

Table 3.16: Displacement limits of cold formed portal frame [83].

Displacement Category	Load Combination	Displacement Limit
Frame Vertical Displacement	G + Qr	L [1]/ 200
	G + 0.5S	L[1] / 200
Frame Horizontal Displacement	DH - G + 0.5Qr ± Wx	h[2] / 300
	DH - G + 0.5Qr ± Wy	H[2] / 300
Vertical Apex Displacement	DVA – G	L[1] / 360
	DVA – G + Qr	L[1] / 240
	DVA – G + 0.5S	L[1] / 240
	DVA – G + Wx	L[1]/ 240
	DVA – G + Wy	L[1] / 240

1) L is span of portal frame
2) h is eave height of portal frame

- Vertical displacement of frame and apex

The inclination of pitched roof is (1/ 5) and it is more (1 / 20) inclination as a result the possibility of ponding can be ignored [84].

The vertical dislacement of portal frame in required combinationsare shown in Figures 3.35, 3.36, 3.37, 3.38 and 3.39 and the vertical displacement control according to displacement limitations are done after each figure.

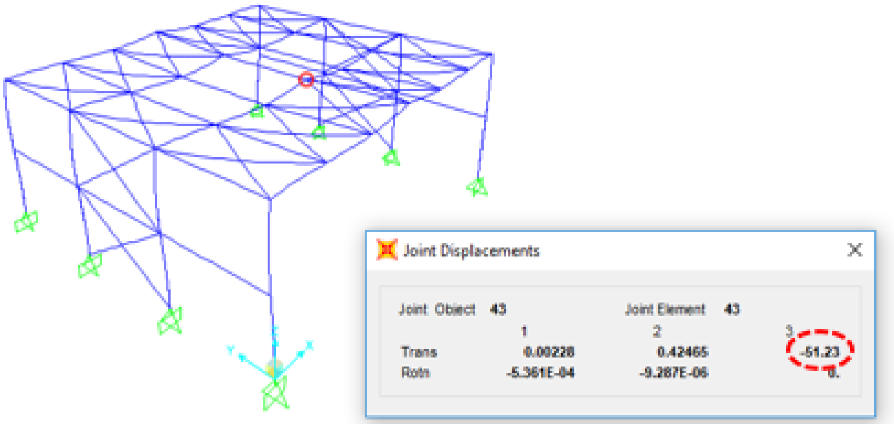


Figure 3.35: The vertical displacement of frame in “G + Qr” combination in SAP2000 program.

The vertical displacement of frame = 51.23 mm < 20000 / 200 = 100 mm

The vertical displacement of apexjoint = 51.23 mm < 20000 / 240 = 83.3 mm

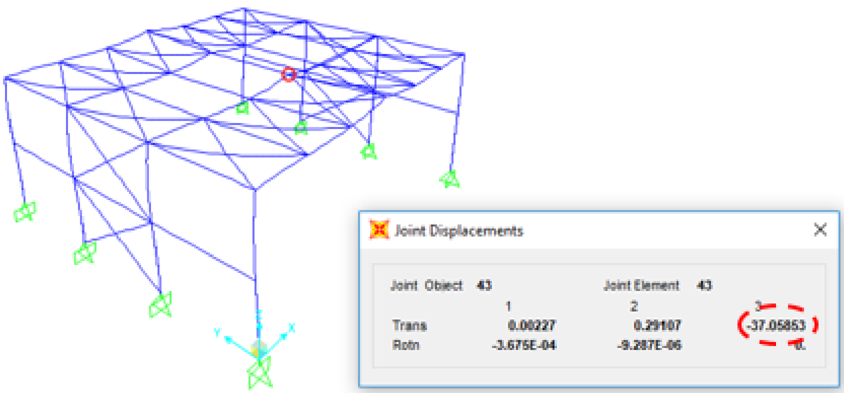


Figure 3.36: The vertical displacement of frame in “G + 0.5S” combination in SAP2000 program.

The vertical displacement of frame = 37 mm < 20000 / 200 = 100 mm

The vertical displacement of apex joint = 37 mm < 20000 / 240 = 83.3 mm

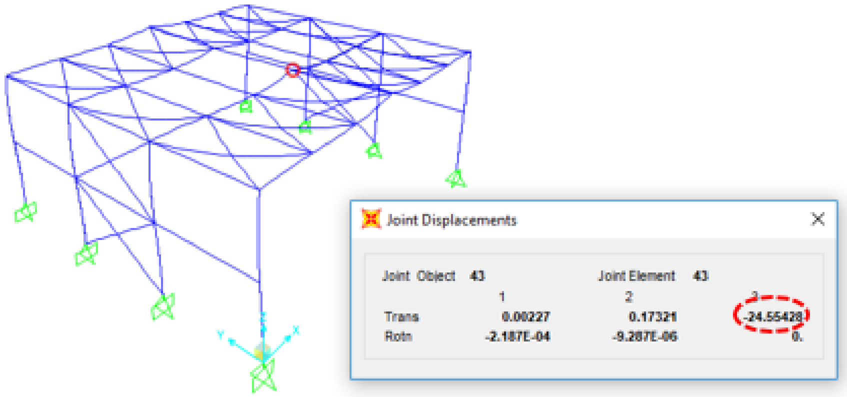


Figure 3.37: The vertical displacement of frame in “G” combination in SAP2000 program.

The vertical displacement of apex joint = 24 mm < 20000 / 360 = 55.56 mm

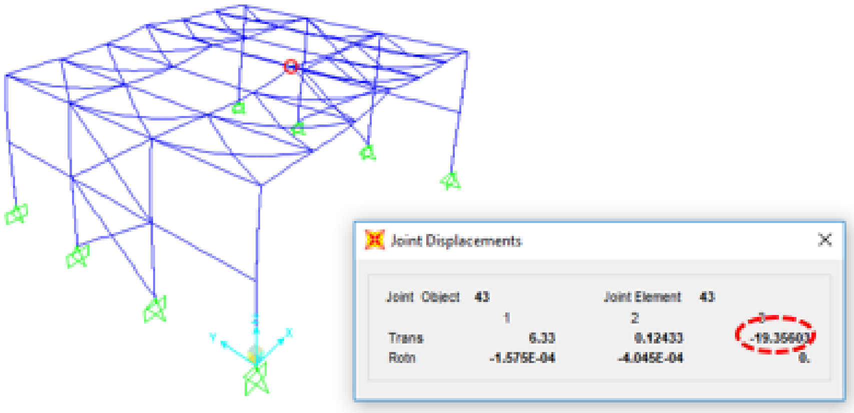


Figure 3.38: The vertical displacement of frame in “G + Wx” combination in SAP2000 program.

The vertical displacement of apex joint = 19.36 mm < 20000 / 240 = 83.3 mm

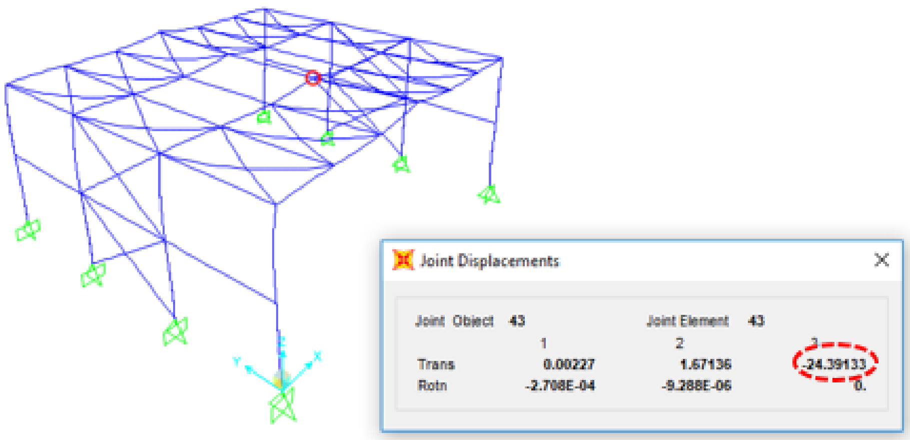


Figure 3.39: The vertical displacement of frame in “G + Wy” combination in SAP2000 program.

The vertical displacement of apex joint = 24.4 mm < 20000 / 240 = 83.3 mm

- Horizontal displacement of frame

The horizontal dislacement of portal frame in required combinations are shown in Figures 3.40 and 3.41 and the horizontal displacement control according to displacement limitations are done after each figure.

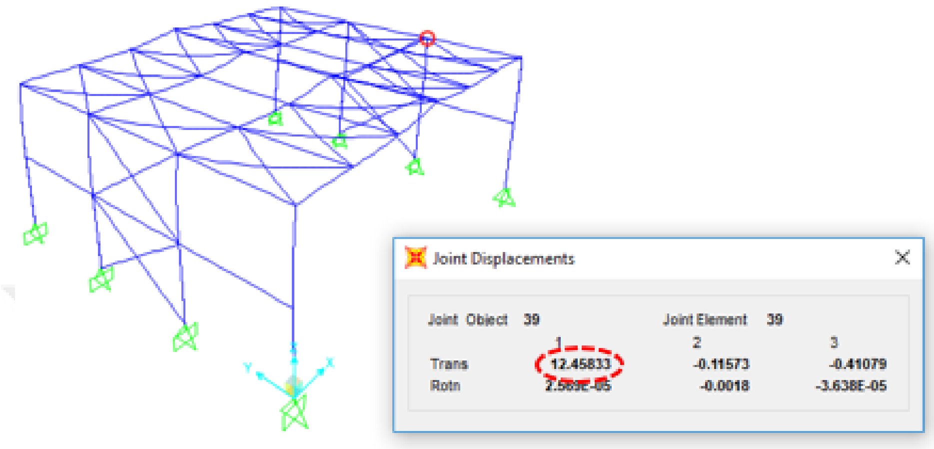


Figure 3.40: The horizontal displacement of frame in “G+ 0.5Qr ± Wx” combination in SAP2000 program.

The horizontaldisplacement of frame = 12.5 mm < 8000 / 300 = 26.67 mm

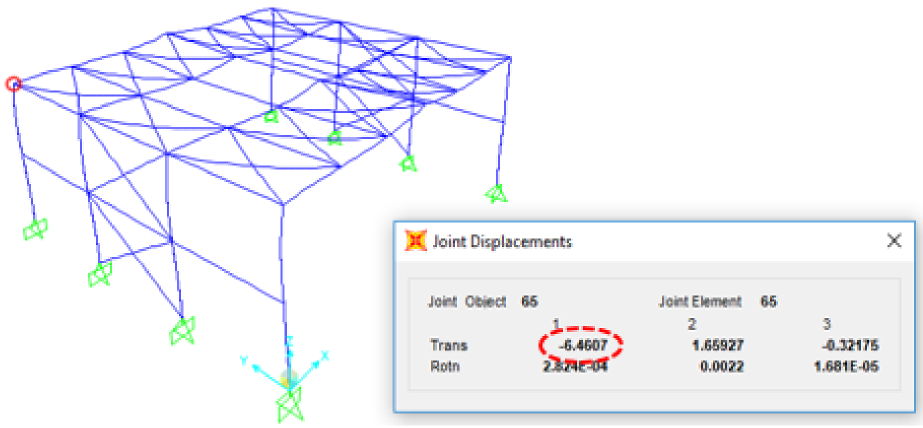


Figure 3.41: The horizontal displacement of frame in “G+ 0.5Qr ± Wy” combination in SAP2000 program.

The horizontaldisplacement of frame = 6.6 mm < 8000 / 300 = 26.67 mm

3.7.3 Drift

The drift limits of cold formed portal frame are given in Table 3.17 [83].

Table 3.17: Drift limits of cold formed portal frame [83].

Displacement Category	Load Combination	Displacement Limit
Drift	Ex	0.02 x h[1] / Rx[2]
	Ey	0.02 x h[1] / Ry[3]

1) h is eave height of portal frame
2) Rx is the behavior factor of portal frame in X direction
3) Ry is the behavior factor of portal frame in X direction

The drift of portal frame in required combinations are shown in Figures 3.42 and 3.43 and the drift control according to drift limitations are done after each figure.

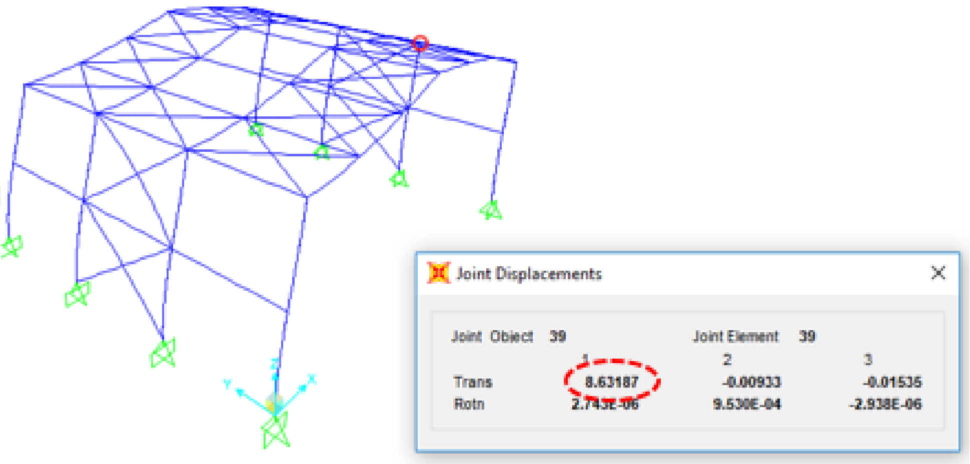


Figure 3.42: The drift of frame in “Ex” combination in SAP2000 program.

The drift of frame = 8.63 mm < 0.02 x 8000 / 8 = 20 mm

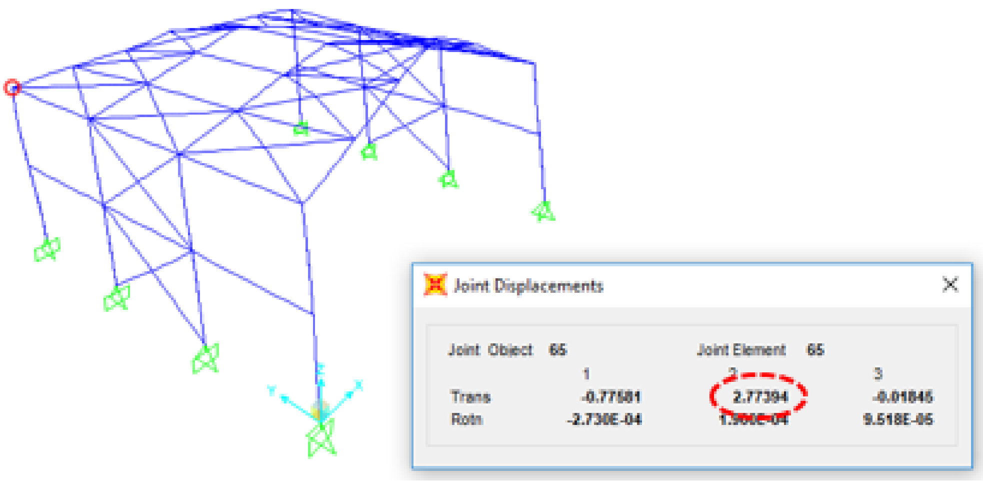


Figure 3.43: The drift of frame in “Ey” combination in SAP2000 program.

The drift of frame = 2.8 mm < 0.02 x 8000 / 8 = 20 mm

4 EAVE JOINTS MODEL

As it is mentioned before portal frame consists of two types of joint, Eave joint and Apex joint. In this thesis the types of Eave joint based on the used stiffeners in column are evaluated.

At considered Eave joint, the rafter are connected to column as bolted using an end plate connection. The rafter profile are welded to the end plate, and the end plate is bolted to the column flange.

The resistance of this type of connection (Bolted end plate connection) is provided by a combination of compression force in bearing at the lower flange of the column and tension force in the bolts which are attached to the upper flange of the column connected to the end plate. The total compression and tension forces are equal and opposite, unless there is an axial force existing in the rafter. Vertical shear force is assumed to be resisted primarily by bolts attached to the compression flange according to the code considered herein.[59].

These forces are shown diagrammatically in Figure 4.1.

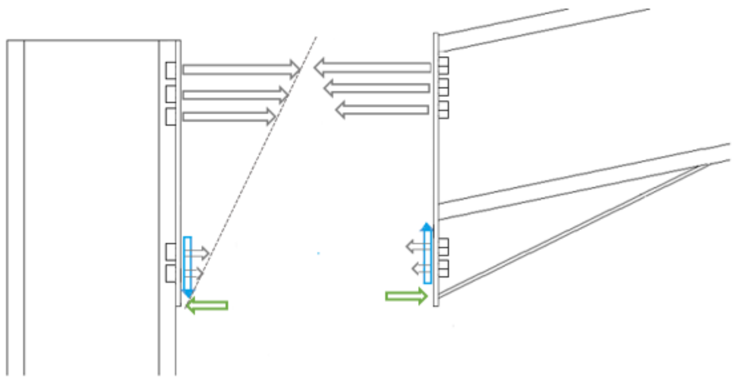


Figure 4.1: Forces in the huanched end plate connection [59].

According to the ultimate limit state, in the haunched end plate connection, the center of rotation is at the compression flange. The more distance between bolt rows and the compression flange, the more tension force they attract.Design practice assumes a 'triangular' distribution of forces, it means that the values of forces are pro rata to the distance from the compression flange [59].

4.1 Created Forces in Eave Joint

In designing of the connections, the Amplified Seismic Load Method is used [87]. In this method the earthquake loads in X and Y directions which are amplified by multiplying those to their over-strength (D) factors are considered in designing of the connections configuration.

In this way the scale factor of earthquake load in X direction is changed from 1 to 3 ($D_x = 3$), and the scale factor of earthquake load in Y direction is changed from 1 to 2 ($D_y = 2$).

It should be noted that the over-strength factors are obtained from Table 3.13.

The interchanging of the scale factor of Earthquake load in X and Y directions in SAP2000 program are shown in Figures 4.2 and 4.3.

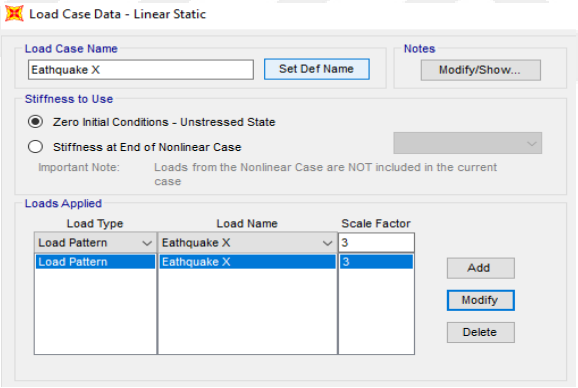


Figure 4.2: Interchanging of the scale factor of earthquake load in X direction from 1 to 3 in the model in SAP2000 program.

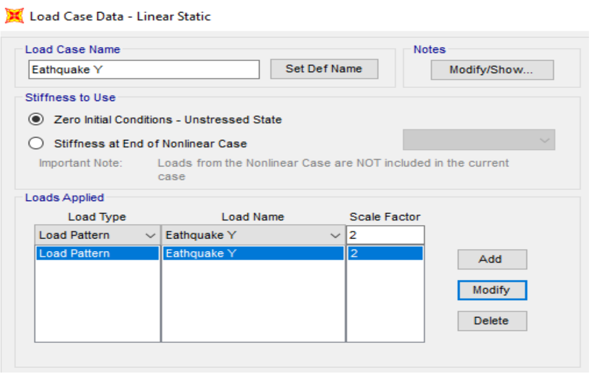


Figure 4.3: Interchanging of the scale factor of earthquake load in Y direction from 1 to 2 in the model in SAP2000 program.

The created forces due to loadings in different combinations in the column-rafter joints are obtained from SAP2000 program, and the greatest created forces are determined. Eave connections are designed according to these maximum values of forces.

The label of Eave joints of considered model in SAP2000 program are shown in Figure 4.4.

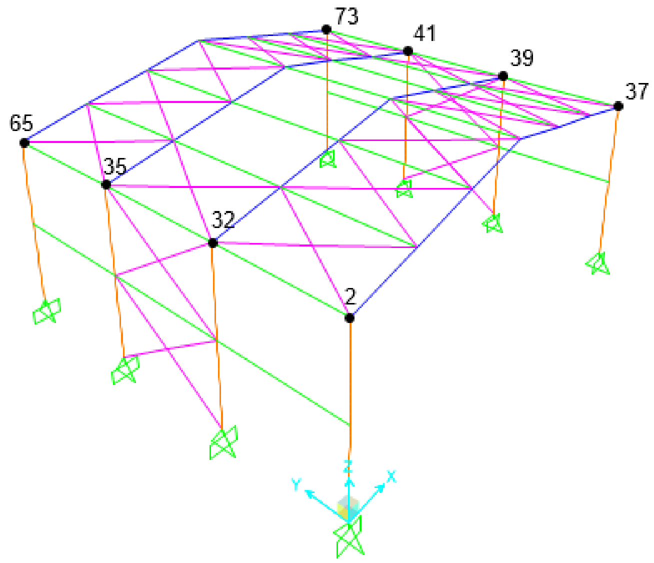


Figure 4.4: Label of Eave joints of considered model in SAP2000 program.

Maximum created forces in specified joints are shown in Table 4.1.

Table 4.1: Maximum created forces in specified Eave joints of model in SAP2000 program.

TABLE: Element Joint Forces - Frames							
Joint	Condition	F1	F2	F3	M1	M2	M3
Text		KN	KN	KN	KN-m	KN-m	KN-m
2	Max	39.11	0.81	83.11	0	-23.77	0
	Min	5.69	-0.87	15.65	0	-235.46	0
35	Max	69.87	0.05	122.68	0	-37.13	2E-05
	Min	13.71	-0.04	23.92	0	-291.37	0
32	Max	70.01	0.04	123.14	0	-37.54	0
	Min	13.71	-0.06	24.22	0	-291.96	3E-05
37	Max	-5.69	0.81	82.04	0	215.35	0
	Min	-37.40	-0.87	18.13	0	42.57	0
39	Max	-13.69	0.04	124.33	0	276.17	-2.7E-05
	Min	-73.21	-0.05	20.74	0	60.35	0
41	Max	-13.73	0.06	124.79	0	276.80	0
	Min	-73.27	-0.04	21.06	0	60.78	-1.5E-05
65	Max	39.17	0.45	83.25	2.4E-05	-23.79	0
	Min	5.70	-0.40	15.66	0	-235.88	0
73	Max	-5.69	0.45	81.98	4.39E-05	215.30	0
	Min	-37.02	-0.40	18.14	0	42.59	0

As it is specified the maximum values of forces are listed as below:

Maximum tension force (F_{1t}): 70 KN

Maximum compression force (F_{1C}): 73.3 KN

Maximum shear force in Y direction (F_2): 0.9 KN

Maximum shear force in Z direction (F_3): 125 KN

Maximum moment around Y axis (M_2): 292 KN-m

Maximum moment around X axis (M_1):0 KN-m

Maximum moment around Z axis (M_3):0 KN-m

4.2 Configuration of Eave Joint

In designing of Eave joint, if the thickness of the thinnest element of the connected parts is less than or equal to 4.76 mm , the AISI S100 (North American Specification for the Design of Cold-Formed Steel Structure Members) design rules are used but for the thickness more than 4.76 mm, the AISC 360 (Specification for Structural Steel Buildings) are used [90].

In both cases the LRFD (Load and Resistance Factor Design) method is used.

The considered haunched bolted end plate Eave joint is shown in Figure 4.5.

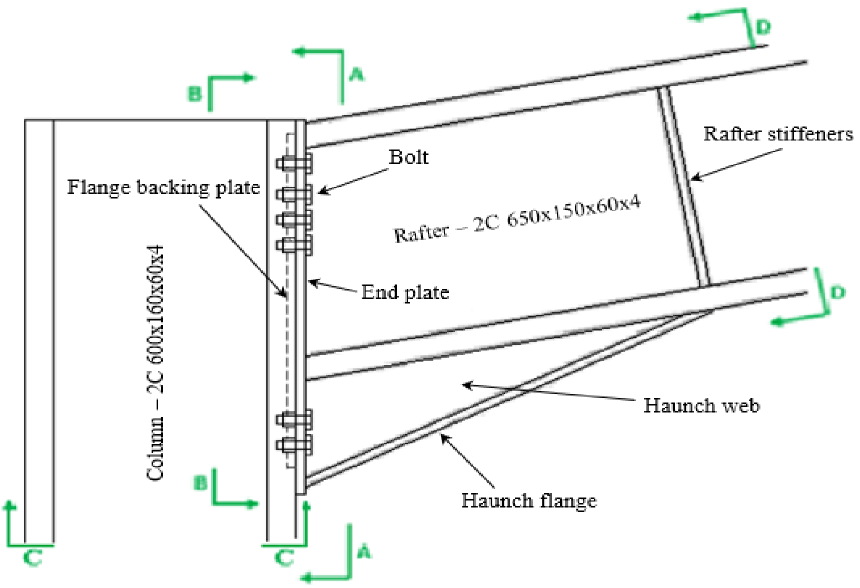


Figure 4.5: Designed Eave joint components.

4.2.1 Components of Eave joint configuration

The components of considered Eave joint are listed as below:

- Bolt

The bolt grade is A325 with diameter of 22 mm. The threads of bolts are out of shear planes, and washers are used under both nut and bolt head. The nut grade is same as bolt grade.

Preloading is not necessary in bolts because the ultimate strength of a bolted joint is independent of the preload level of the bolts,and simple squeezing bolts do not release or loosen the bolts [90].

The nominal tensile stress and the nominal shear stress of A325 bolt are 621 N/mm^2 and 372 N/mm^{2}respectively [88].

The total number of used bolts are 12 bolts which 8 bolts are installed at the top of the end plate, tension zone, and 4 bolts are used at the bottom of the end plate, compression zone.

The arrangement and distance between bolts are shown in Figure 4.6.

- End plate

The end plate is hot rolled S450JR steel plate, and its thickness is 12 mm. The end plate is attached to rafter by two-sided fillet welds and connected to column flange by bolts.

The yield stress and ultimate stress of S450JR steel material are 440 N/mm^2 and 550 N/mm^{2}respectively [78].

The dimensions of end plate is shown in Figure 4.6.

- Haunch

To have a rigid connection, the presence of haunch in the connection is necessary [59]. Haunch is a hot rolled profile which is attached to the bottom flange of rafter and to the end plate by two-sided filet welds.

Haunch decreases the tension force affecting on the top bolts and compression force affectingon the bearing flange which are arising from the created moment in the rafter by increasing the lever arm [89].

Haunch length is the distance between the surface of column flange and the plastic hinge point in the rafter which should be 1.5 times of rafter section height [34].The 1.5 times of rafter section height which is 650 mm is 975 mm.

Haunch material is S450JR steel too. The flange and web thickness of haunch is 12 mm.

The dimensions of haunch are shown in Figures 4.6 and 4.7.

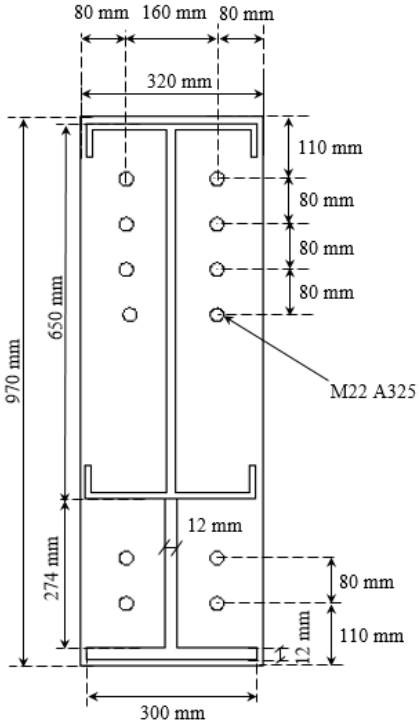


Figure 4.6: Section A-A of shownEave joint in Figure 4.5.

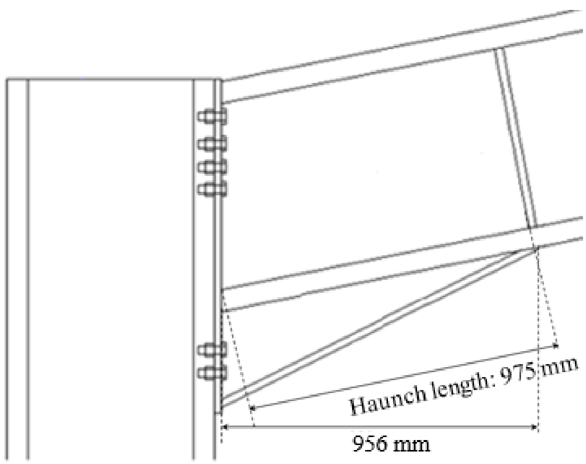


Figure 4.7: Designed haunch dimensions in considered Eave joint.

- Flange backing plate

Flange backing plates are hot rolled plates that are attached to the back of the column flange by bolts. These plates are used to thicken the connected tension parts to increase the column flange bending resistance against lamellar tearing.

The flange backing plates are made of S450JR steel, and its thickness is 12 mm.

The dimensions of these plates is shown in Figures 4.8 and 4.9.

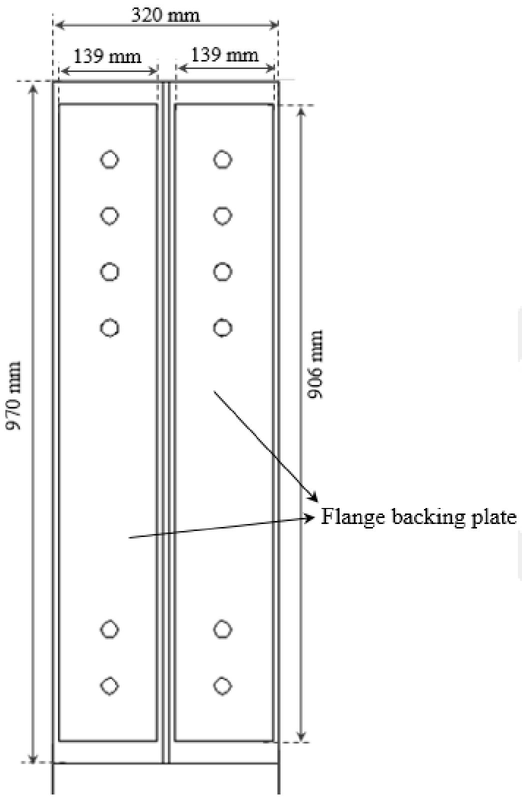


Figure 4.8: Section B-B of shown Eave joint in Figure 4.5.

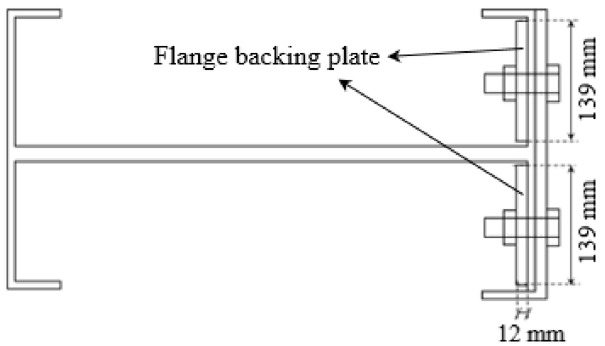


Figure 4.9: Section C-C of Eave joint of shown Eave joint in Figure 4.5.

4.2.2 Method of strengthening of Eave joint

One of the most important part in the designing of the bolted end plate connection is to choose the appropriate stiffeners. Stiffeners are secondary sections or plates which are connected to column or beam flanges or webs to stiffen them against out of plane deformations. Stiffeners are usually required in the connections to enhance the load capacity of joints and to have more rigid connections [88].

Stiffeners depended on application location are divided into 2 groups:

- Stiffeners applied on rafter
- Stiffeners applied on column

4.2.2.1 Stiffeners applied on rafter

Rafter stiffeners are secondary hot rolled flat plates that are attached to rafter web and flanges transversely. They are called transverse stiffeners (see figure 4.5)

These stiffeners are applied on the point of rafterweb where the haunch is welded to the bottomn flange of rafter because the loads are concentrated mainly on this point, means the prependicular distance of these stiffeners to column surface is 956 mm.

Rafter stiffeners are applied on either side of rafter web and their material are S450JR steel. Their thickness are 5 mm and they are attached to rafter flanges as well as web by two-sided fillet welds.

The dimensions of these stiffeners are shown in Figure 4.10.

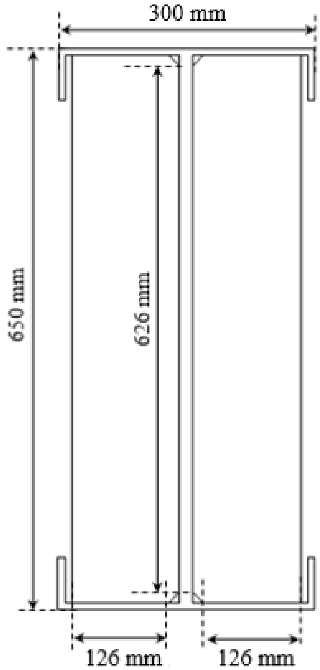


Figure 4.10: Section D-D of shown Eave joint in Figure 4.5.

4.2.2.2 Stiffeners applied on column

A lot of parameters influence the connection characteristics. The major factor that affect the joint characteristics is the flexibility sources of a connection [6]. These flexibility sources of a connection are listed as following:

- Deformation of connection components as plates, cleats, bolts… and the slip between its components
- Localized deformation of the column
- Localized deformation of the beam

Some of the localized deformation of column are column web in tension,column web in compression, column flange in bending and column web in shear [35]. These localised column deformation of typical bolted end plate connection is shown in Figure 4.11.

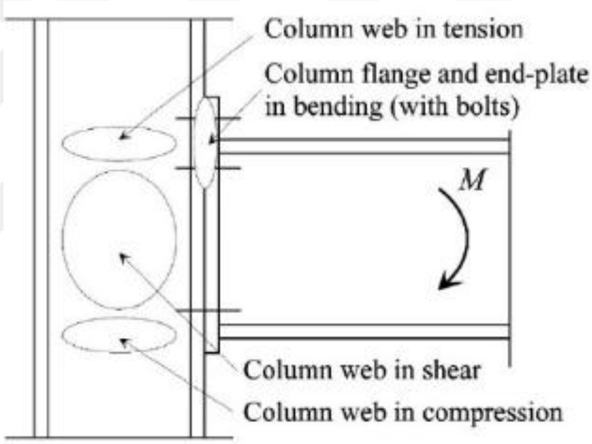


Figure 4.11: Localized deformation of column in typical bolted end-plate connection [35].

Stiffeners are used for strengthening of the column against these localiseddeformations.

There are several types of column stiffeners that each of them are used to strengthen the one zone or more. Some types of them are given inTable 4.2 [59].

Table 4.2: Column strengthening methods [59].

Type of column stiffener	Deficiency			
	Web in tension	Flange in bending	Web in compression	Web in shear
Full depth horizontal stiffener	•	•	•	
Partial depth horizontal stiffener	•	•	•	
Supplementary web plates	•		•	•
Diagonal stiffeners (N & K)	•	•		•

In this thesis the effect of column localized deformation types which are restricted by different types of stiffeners on the behavior of connection is evaluated.

The 3 different types of Eave joint are designed. Every connection details between 3 model types are the same and their difference is the column stiffener type only.

- Type 1: Full depth horizontal stiffener

Full depth horizontal stiffeners are hot rolled flat plates with 5 mm thickness which are attached to column flanges and web by two-sided fillet welds.

Totally 4 plates are applied, two of them are applied on the both side of the column web opposed to the tension region (see Plates a) and the other two plates are applied on the both side of the column web opposed to the compression region (see Plates b). It should to be noted that the stiffeners material are S450JR steel.

The dimensions of type 1 of Eave joint are shown in Figures 4.12 and 4.13. Every other details of connection components are mentioned in the previous parts.

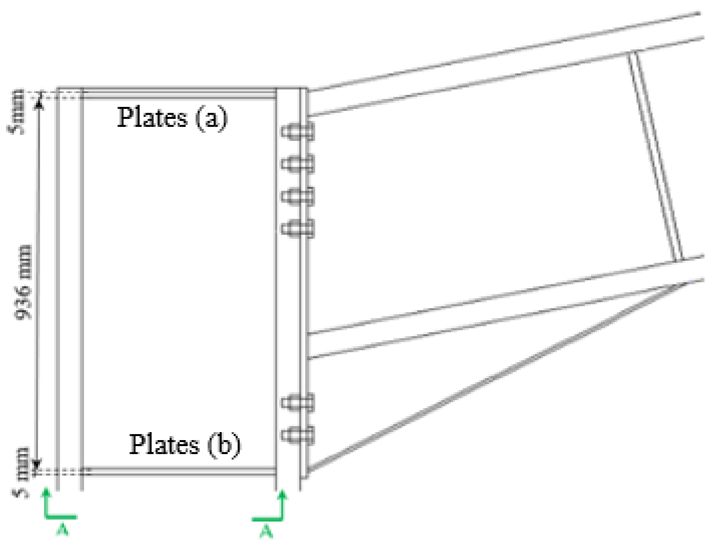


Figure 4.12: Type 1 of Eave joint, Eave joint with full depth horizontal column web stiffener.

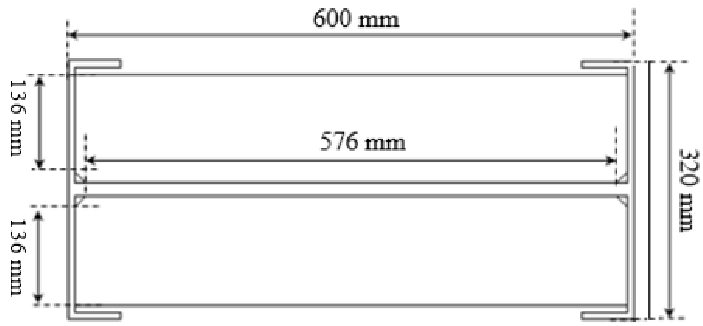


Figure 4.13: Section A-A of shown type 1 of Eave joint in Figure 4.12.

- Type 2: Supplementary web plate stiffener

Suplementary web plate stiffeners are hot rolled flat plates with 5 mm thickness which are attached to column web by two-sided fillet welds.

Totally 2 plates are applied on both sides of the column web having the same length of the end plate, and their material are S450JR steel.

The dimensions of type 2 of Eave joint are shown in Figure 4.14 and 4.15.

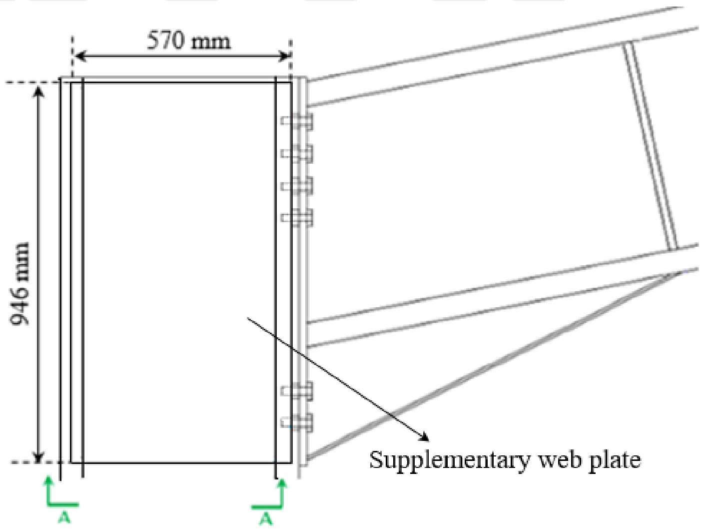


Figure 4.14: Type 2 of Eave joint, Eave joint with supplementary web plate stiffener.

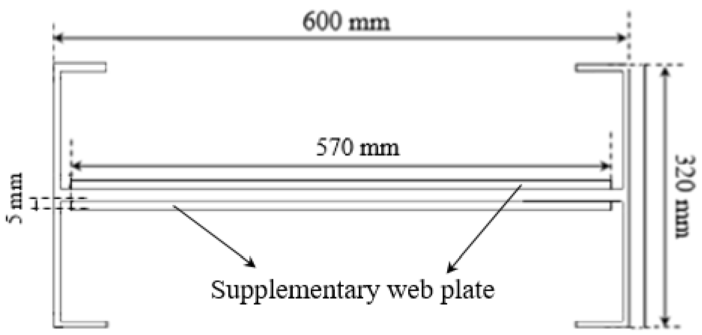


Figure 4.15: Section A-A of shown type 2 of Eave joint in Figure 4.14.

- Type 3: K diagonal stiffener

K diagonal stiffeners are hot rolled flat plates with 5 mm thickness which are attached to column web and flanges by two-sided fillet welds.

Totally 6 plates are applied on the column web, two of them are applied on the both side of the column web opposed to the tension region (see Plates a) and the other two plates are applied on the both side of the column web opposed to the compression region (see Plates b) and the other two plates are applied on the both side of the column web obliquely with angle of 31 degrees according to the perpendicular line (see Plates c). The material of stiffeners are S450JR steel.

The dimensions of type 3 of Eave joint is shown in Figure 4.16.

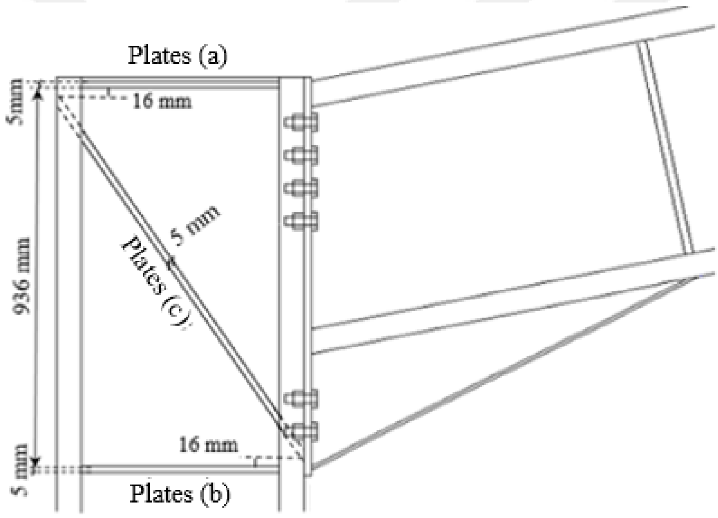


Figure 4.16: Type 3 of Eave joint, Eave joint with K diagonal stiffener.

4.2.3 Design calculation of Eave joint

The designed Eave joint are controlled according to 3 groups as listed below:

- Welds
- Bolts
- Plates

The plates control affected by created forces are done in section 4.3 by IDEA StatiCa 10 program.

The data which are used in the considered Eave design calculation are summarized as below:

Column steel grade: S350GD

Ultimate strength of column steel (F_{uc}): 420 N/mm^2

Yield strength of column steel (F_{yc}): 350 N/mm^2

Column section thickness (t_c): 4 mm

Column section height (d_c): 600 mm

Column section width (b_c): 320 mm

Column section radius (r_{pc}): 3 mm

Column section area (A_c): 8200 mm^2

Rafter steel grade: S350GD

Ultimate strength of rafter steel (F_{ur}): 420 N/mm^2

Yield strength of rafter steel (F_{yr}): 350 N/mm^2

Rafter section thickness (t_r): 4 mm

Rafter section height (d_r): 650 mm

Raftersection width (b_r): 300 mm

Rafter section radius (r_{pr}): 3 mm

Rafter section area (A_r): 8432 mm^2

Haunch steel grade: S450JR

Ultimate strength of haunch steel (F_{uh}): 550 N/mm^2

Haunch cutting depth (d_h): 274 mm

Haunch length (L_h): 975 mm

Haunch perpendicular length (L_{ph}): 956 mm

Haunch width (b_h): 300 mm

Haunch web thickness (t_{wh}): 12 mm

Haunch flange thickness (t_{fh}): 12 mm

End plate steel grade: S450JR

Ultimate strength of end plate steel (F_{uep}): 550 N/mm^2

Yield strength of rafter steel (F_{yep}): 440 N/mm^2

End plate length (L_e): 970 mm

End plate width (b_e): 320 mm

End plate thickness (t_e): 12 mm

Stiffener (1) steel grade: S450JR

Ultimate strength of stiffener (1) steel (F_{us1}): 550 N/mm^2

Stiffeners (1) length (L_{s1}): 626 mm

Stiffeners (1) width (b_{s1}): 126 mm

Stiffeners (1) thickness (t_{s1}): 5 mm

Full depth horizontal stiffener steel grade: S450JR

Ultimate strength of full depth horizontal stiffener steel (F_{us2}): 550 N/mm^2

Full depth horizontal stiffener length (L_{s2}): 576 mm

Full depth horizontal stiffener width (b_{s2}): 136 mm

Full depth horizontal stiffener thickness (t_{s2}): 5 mm

Supplementary web plate stiffener steel grade: S450JR

Ultimate strength of supplementary web plate stiffener steel (F_{us3}): 550 N/mm^2

Supplementary web plate stiffener length (L_{s3}): 946 mm

Supplementary web plate stiffener width (b_{s3}): 570 mm

Supplementary web plate stiffener thickness (t_{s3}): 5 mm

Plates (c) of K diagonal stiffener steel grade: S450JR

Ultimate strength of plates (c) of K diagonal stiffener steel (F_{us4}): 550 N/mm^2

Plates (c) of K diagonal stiffener length (L_{s4}): 1053 mm

Plates (c) of K diagonal stiffener width (b_{s4}): 136 mm

Plates (c) of K diagonal stiffener thickness (t_{s4}): 5 mm

Welding electrode class: E120XX

Minimum tensile strength of electrode (F_{xx}): 827.37 N/mm^2

Characteristic bolt diameter (d_b): 22

Gross cross-sectional area of bolt (A_b): 380.13 mm^2

Bolt grade: A325

Nominal tensile strength of bolt (F_{nt}): 621 N/mm^2

Nominal shear strength of bolt if the threads of bolts are out of shear plane (F_{nv}): 496 N/mm^2

Required shear stress of bolts (f_v): 124 N/mm^2

Number of bolts in tension zone (n_t): 8

Number of bolts in compression zone (n_c): 4

Design coefficient of longitudinal welding for cold-formed section if the ratio of welding length to minimum thickness of connected element is less than 25 (ϕ_{L1}): 0.6

Design coefficient of longitudinal welding for cold-formed section if the ratio of welding length to minimum thickness of connected element is more than 25 (ϕ_{L2}): 0.5

Design coefficient of transverse welding for cold-formed section (ϕ_t): 0.65

Design coefficient of nominal control welding cold-formed section (ϕ_a): 0.6

Design coefficient of welding for hot rolled section (ϕ): 0.75

Maximum tension force (F_{1t}): 70 KN

Maximum compression force (F_{1C}): 73.3 KN

Maximum shear force in Z direction (F_3): 125 KN

Maximum moment around Y axis (M_2): 292 KN-m

4.2.3.1 Welds

All welds, unless otherwise mentioned are two sided fillet welds and they were performed with E120XX welding electrode. Minimum tensile strength and minimum yield strength of class E120XX electrode are 827.37 N/mm^2(120000 psi) and 737.74 N/mm^2(107000 psi) respectively [89].

Welding procedure are done consistently, and landing gusset are used during weldings.

The welding are done as longitudinal weldings and transverse weldings. If the length of the weld is parallel to applied forcedirection then it is called longitudinal welding but if the length of the weld is perpendicular to the applied forcedirection then it is called transverse welding [10]. These welding types are shown in Figure 4.17.

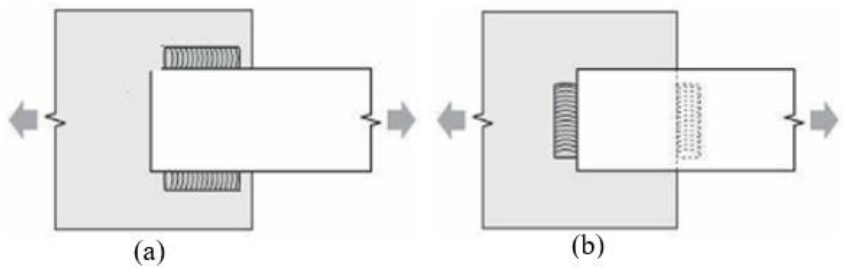


Figure 4.17: Fillet welding; (a) Longitudinal welding; (b) Transverse welding [10].

The welding design calculation forcold formed sections are different from hot rolled sections. For welding design of hot rolled sectionsthe AISC 360 specification are used and for welding design of cold-formed sections the AISI S100 specification are used [82,35].

For fillet welds of cold formed section, the leg lengths (W_1 and W_2) and the throat thickness (t_w) are as shown in Figure 4.18. The throat thickness is usually larger than the thickness of the steel sheets (t_1 and t_2) so ultimate failure is commonly occur by lamellar tearing of the plates adjacent to the fillet weld or along the fillet weld contour [35].

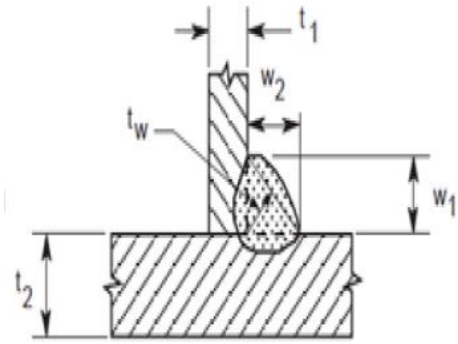


Figure 4.18: Fillet welding [90].

The welding calculation of designed Eave joints are done into 7 items as listed below:

- Welding 1: Welding of rafter profile to end plate
- Welding 2: Welding of haunch profile to end plate
- Welding 3: Welding of haunch profile to rafter

- Welding 4: Welding of stiffener (1) plates to rafter
- Welding 5: Welding of full depth horizontal stiffeners to column
- Welding 6: Welding of supplementary web plate stiffener to column
- Welding 7: Welding of K diagonal stiffener to column

These items are shown in Figure 4.19.

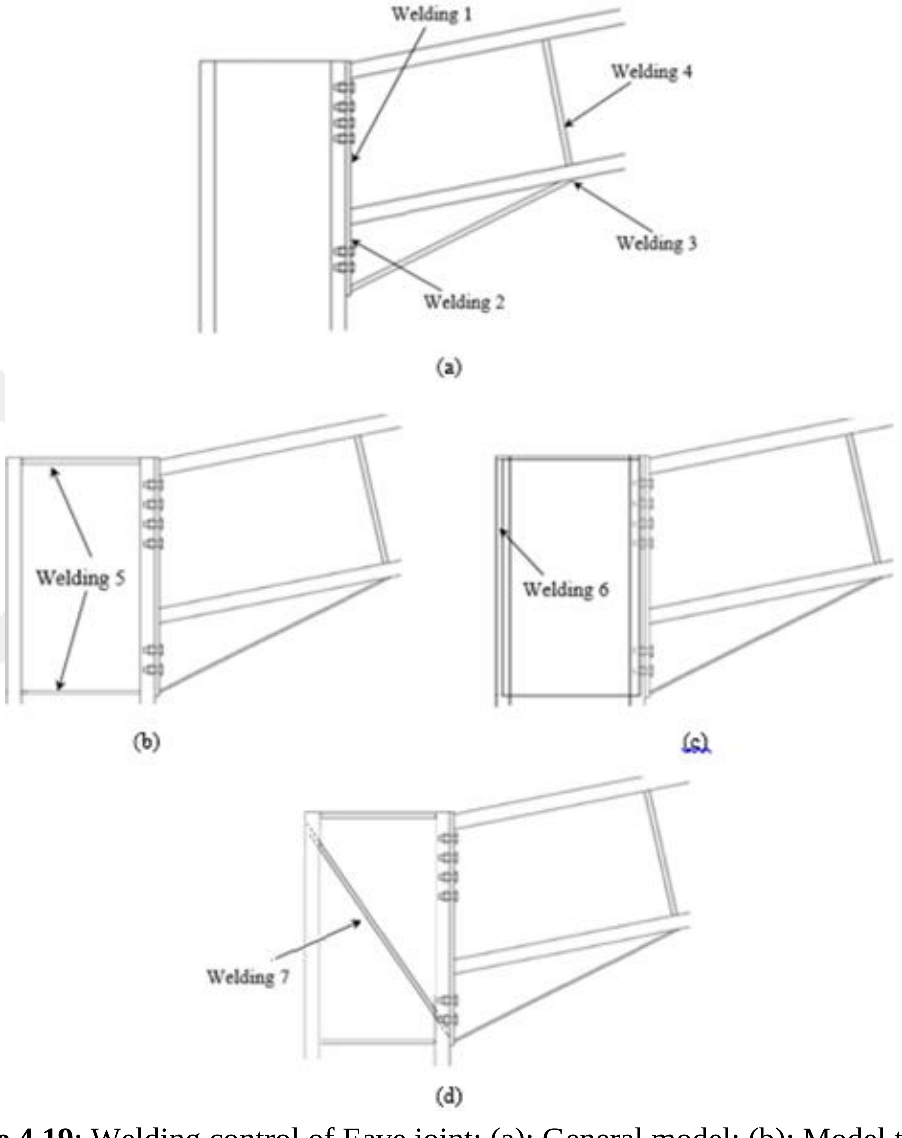


Figure 4.19: Welding control of Eave joint; (a): General model; (b): Model type 1; (c): Model type 2; (d): Model type 3.

The welding calculations of each item are given in the following:

- Welding 1: Welding of rafter profile to end plate

The welds aredesigned according to the tension and the shear force.

The welds are designed according to cold-formed steel sections welding design rules.

The rafter top flange are affected by tension and shear force and the rafter web are affected by shear force so the welding of rafter flange to end plate is transverse welding for both design shear force and design tension force and the welding of rafter web to end plate is longitudinal welding for design shear force.

Design shear force (P_{f1}) = F_3 = 125 KN

Moment lever arm (h) = $d_r + d_h + t_{fh}$= 650 + 274 + 12 = 936 mm

Design tension force (P_{f2}) = F_{1t} + (M_2 / h) = 70 + (292 / 0.936) = 381.97 KN

- Welding of rafter flange to end plate

Design thickness (t) = Min (t_e , t_r) = Min (12 , 4) = 4 mm

Leg of weld (W_1) = 7 mm (see figure 4.18)

Leg of weld (W_2) = 7 mm (see figure 4.18)

W = Min (W_1 , W_2) = 7 mm

Effective throat thickness of weld (t_w) = 0.707 x W = 5 mm

Length of weld (L_w) = $2b_r$ – ($4t_r + 4r_{pr} + 2t_w$) = 600 – (16 + 12 + 10) = 562 mm

Nominal strength of weld (P_n) = t x L_w x F_{ur} = (4 x 562 x 420) / 1000 = 944.16 KN

Design strength of weld (P_{nd}) = ϕ_t x P_n= 0.65 x 944.16 = 613.7 KN

So:

Design strength of weld is larger than design shear force as a result this weld can endure the applied design shear force.

And also, design strength of weld is larger than design tension force as a result this weld can endure the applied design tension force.

Nominal strength of control weld (P_a) =0.75 x t_w x L_w x F_{xx} = (0.75 x 5 x 562 x 827.37) / 1000 = 1743.68 KN

Design strength of control weld (P_{ad}) = ϕ_a x P_a = 0.6 x 1743.68 = 1046.2 KN

Condition (a): $P_{nd} \leq P_{ad}$, have to be provided.

So:

This weld provided the condition (a).

- Welding of rafter web to end plate

Design thickness (t) = Min (t_e , $2t_r$) = Min (12 , 8) = 8 mm

Leg of weld (W_1) = 7 mm (see figure 4.18)

Leg of weld (W_2) = 7 mm (see figure 4.18)

W = Min (W_1 , W_2) = 7 mm

Effective throat thickness of weld (t_w) = 0.707 x W = 5 mm

Length of weld (L_w) = d_r – ($2t_r$ + $2r_{pr}$ + $2t_w$) = 650 – (8 + 6 + 10) = 626 mm

The 'L_w / t'ratio is larger than 25 so nominal strength of weld (P_n) is calculated as below:

Nominal strength of weld (P_n) = 0.75 x t x L_w x F_{ur} = (0.75 x 8 x 626 x 420) / 1000 = 1577.5 KN

Design strength of weld (P_{nd}) = ϕ_{L2} x P_n= 0.5 x 1577.5 = 788.76 KN

So:

Design strength of weld is larger than design shear force as a result this weld can endure the applied design shear force.

Nominal strength of control weld (P_a) = 0.75 x t_w x L_w x F_{xx} = (0.75 x 5 x 626 x 827.37) / 1000 = 1942.25 KN

Design strength of control weld (P_{ad}) = ϕ_a x P_a = 0.6 x 1942.25 = 1165.35 KN

Condition (a): $P_{nd} \leq P_{ad}$, have to be provided.

So:

This weld provided the condition (a).

- Welding 2: Welding of haunch profile to end plate

The welds are designed according to shear force.

The haunch and end plate are hot rolled steel sections so the welds are designed according to hot rolled sections welding design rules.

The welding of haunch web to end plate is longitudinal welding for design shear force and the welding of haunch flange to end plate is transverse welding for design shear force.

Design shear force (P_{f1}) = F_3 = 125 KN

- Welding of haunch web to end plate

Design thickness (t) = Min (t_{wh} , t_e) = Min (12 , 12) = 12 mm

Maximum throat thickness of weld (t_{wmax}) = 0.7 x (t – 2) = 7 mm

Minimum throat thickness of weld (t_{wmin}) = 3.5 mm

Effective throat thickness of weld (t_w) = 5 mm

Length of weld (L_w) = d_h – $2t_w$ = 274 – 10 = 264 mm

Effective area of weld (A_w) = L_w x t_w = 264 x 5 = 1320 mm^2

Nominal strength of longitudinal weld (P_{nL}) = 0.6 x F_{xx} x A_w= (0.6 x 827.37 x 1320) / 1000 = 665.3 KN

Design strength of longitudinal weld (P_{nLd}) = ϕx P_{nL} = 0.75 x 665.3 = 491.5 KN

- Welding of haunch flange to end plate

Design thickness (t) = Min (t_{fh} , t_e) = Min (12 , 12) = 12 mm

Maximum throat thickness of weld (t_{wmax}) = 0.7 x (t – 2) = 7 mm

Minimum throat thickness of weld (t_{wmin}) = 3.5 mm

Effective throat thickness of weld (t_w) = 5 mm

Length of weld (L_w) = b_h – 2tw = 300 – 10 = 290 mm

Effective area of weld (A_w) = L_w x t_w = 290 x 5 = 1450 mm^2

Nominal strength of transverse weld (P_{nt}) = 0.6 x F_{xx} x A_w= (0.6 x 827.37 x 1450) / 1000 = 719.8 KN

Design strength of transverse weld (P_{ntd}) = ϕx P_{nt} = 0.75 x 719.8 = 539.86 KN

In total:

P_{nd1} = P_{nLd} + P_{ntd} = 491.5 + 539.86 = 1031.36 KN

P_{nd2} = (0.85 x P_{nLd}) + (1.5 x P_{ntd}) = (0.85 x 491.5) + (1.5 x 539.86) = 1227.57 KN

Design strength of weld (P_{nd}) = Max (P_{nd1} , P_{nd2}) = 1227.57 KN

So:

Design strength of weld is larger than design shear force as a result this weld can endure the applied design shear force.

- Welding 3: Welding of haunch profile to rafter

The weld is designed according to shear force which act as tension force and compression force which act as shear force for this welding.

The weld is designed according to cold-formed steel sections welding design rules.

The welding of haunch flange to rafter is transverse welding for both design tension force and design shear force.

Design tension force (P_{f1}) = F_3 = 125 KN

Design shear force (P_{f2}) = F_{1C} + [(M_2 – (F_3 x L_{ph})) / d_r] = 73.3 + [(292 – (125 x 0.956)) / 0.65] = 338.68 KN

Design thickness (t) = Min (t_{fh} , t_r) = Min (12 , 4) = 4 mm

Leg of weld (W_1) = 7 mm (see figure 4.18)

Leg of weld (W_2) = 7 mm (see figure 4.18)

W = Min (W_1 , W_2) = 7 mm

Effective throat thickness of weld (t_w) = 0.707 x W = 5 mm

Length of weld (L_w) = $2b_h$ – (t_{wh} + $4t_w$) = 600 – (12 + 20) = 568 mm

Nominal strength of weld (P_n) = t x L_w x F_{ur} = (4 x 568 x 420) / 1000 = 954.2 KN

Design strength of weld (P_{nd}) = ϕ_t x P_n= 0.65 x 954.2 = 620.2 KN

So:

Design strength of weld is larger than design shear force as a result this weld can endure the applied design shear force.

And also, design strength of weld is larger than design tension force as a result this weld can endure the applied design tension force.

Nominal strength of control weld (P_a) =0.75 x t_w x L_w x F_{xx} = (0.75 x 5 x 568 x 827.37) / 1000 = 1762.3 KN

Design strength of control weld (P_{ad}) = ϕ_a x P_a = 0.6 x 1762.3 = 1057.4 KN

Condition (a): $P_{nd} \leq P_{ad}$, have to be provided.

So:

This weld provided the condition (a).

- Welding 4: Welding of rafter stiffeners to rafter

The welds are designed according to shear force and axial(compression and tension) force which are act as shear force for these welds.

The welds are designed according to cold-formed steel sections welding design rules.

The weldings of stiffeners length to rafter web are longitudinal welding for design shear force and the weldings of stiffeners width to rafter flange are transverse welding for both design axial force and design shear force.

Number of rafter stiffeners (n_s) = 2

Design shear force (P_{f1}) = F_3 / n_s = 125 / 2 = 62.5 KN

The created compression force (F_{1C}) is larger than the created tension force (F_{1t})so the weldings are designed according to created compression force.

Design axial force (P_{f2}) = [F_{1C} + [(M_2 – (F_3 x L_{ph})) / d_r]] / n_s = [73.3 + [(292 – (125 x 0.956)) / 0.65]] / 2 = 169.34 KN

  - Welding of stiffeners length to rafter web

Design thickness (t) = Min (t_{s1} , 2t_r) = Min (5 , 8) = 5 mm

Leg of weld (W_1) = 7 mm (see figure 4.18)

Leg of weld (W_2) = 7 mm (see figure 4.18)

W = Min (W_1 , W_2) = 7 mm

Effective throat thickness of weld (t_w) = 0.707 x W = 5 mm

Length of weld (L_w) = L_{s1} – 2tw = 626 – 10 = 616 mm

The 'L_w / t' ratio is larger than 25 so nominal strength of weld (P_n) is calculated as below:

Nominal strength of weld (P_n) = 0.75 x t x L_w x F_{ur} = (0.75 x 5 x 616 x 420) / 1000 = 970.2 KN

Design strength of weld (P_{nd}) = ϕ_{L2} x P_n= 0.5 x 970.2 = 485.1 KN

So:

Design strength of weld is larger than design shear force as a result this weld can endure the applied design shear force.

Nominal strength of control weld (P_a) = 0.75 x t_w x L_w x F_{xx} = (0.75 x 5 x 616 x 827.37) / 1000 = 1911.22 KN

Design strength of control weld (P_{ad}) = ϕ_a x P_a = 0.6 x 1911.22 = 1146.73 KN

Condition (a): $P_{nd} \leq P_{ad}$, have to be provided.

So:

This weld provided the condition (a).

- Welding of stiffeners widths to rafter flanges

Design thickness (t) = Min (t_{s1} , t_r) = Min (5 , 4) = 4 mm

Leg of weld (W_1) = 7 mm (see figure 4.18)

Leg of weld (W_2) = 7 mm (see figure 4.18)

W = Min (W_1 , W_2) = 7 mm

Effective throat thickness of weld (t_w) = 0.707 x W = 5 mm

Length of weld (L_w) = b_{s1} – 2t_w = 126 - 10 = 116 mm

Nominal strength of weld (P_n) = t x L_w x F_{ur} = (4 x 116 x 420) / 1000 = 194.88 KN

Design strength of weld (P_{nd}) = 2 x ϕ_t x P_n= 2 x 0.65 x 194.88 = 253.34 KN

So:

Design strength of weld is larger than design shear force as a result this weld can endure the applied design shear force.

And also, design strength of weld is larger than design axial force as a result this weld can endure the applied design axial force.

Nominal strength of control weld (P_a) = 0.75 x t_w x L_w x F_{xx} = (0.75 x 5 x 116 x 827.37) / 1000 = 360 KN

Design strength of control weld (P_{ad}) = 2 x ϕ_a x P_a = 2 x 0.6 x 360 = 431.9 KN

Condition (a): $P_{nd} \leq P_{ad}$, have to be provided.

So:

This weld provided the condition (a).

- Welding 5: Welding of full depth horizontal stiffeners to column

Plates (a) are affected by tension force and plates (b) are affected by compression force. The created compression force are larger than created tension force so the weld are designed according to compression force.

The welds are designed according to cold-formed steel sections welding design rules.

The welding of full depth horizontal stiffener length to column web is longitudinal welding and the welding of full depth horizontal stiffener width to column flanges is transverse welding for design compression force.

The welding of plates b are written in this thesis. The welding of plates a are done as same as the plates b welding.

Number of plates b of full depth horizontal stiffeners (n_s) = 2

Moment lever arm (h) = d_r + d_h + t_{fh}= 650 + 274 + 12 = 936 mm

Design tension force (P_{f1}) = [F_{1C} + (M_2 / h)] / n_s = [73.3 + (292 / 0.936)] / 2 = 192.6 KN

  - Welding of stiffeners length to column web

Design thickness (t) = Min (t_{s2} , $2t_c$) = Min (5 , 8) = 5 mm

Leg of weld (W_1) = 7 mm (see figure 4.18)

Leg of weld (W_2) = 7 mm (see figure 4.18)

W = Min (W_1 , W_2) = 7 mm

Effective throat thickness of weld (t_w) = 0.707 x W = 5 mm

Length of weld (L_w) = L_{s2} – 2tw = 576 – 10 = 566 mm

The 'L_w / t ' ration is larger than 25 so nominal strength of weld (P_n) is calculated as below:

Nominal strength of weld (P_n) = 0.75 x t x L_w x F_{ur} = (0.75 x 5 x 566 x 420) / 1000 = 891.45 KN

Design strength of weld (P_{nd}) = ϕ_{L2} x P_n= 0.5 x 891.45 = 445.7 KN

So:

Design strength of weld is larger than design tension force as a result this weld can endure the applied design tension force.

Nominal strength of control weld (P_a) = 0.75 x t_w x L_w x F_{xx} = (0.75 x 5 x 566 x 827.37) / 1000 = 1756 KN

Design strength of control weld (P_{ad}) = ϕ_a x P_a = 0.6 x 1756 = 1053.66 KN

Condition (a): $P_{nd} \leq P_{ad}$, have to be provided.

So:

This weld provided the condition (a).

- Welding of stiffeners widths to rafter flanges

Design thickness (t) = Min (t_{s2} , t_c) = Min (5 , 4) = 4 mm

Leg of weld (W_1) = 7 mm (see figure 4.18)

Leg of weld (W_2) = 7 mm (see figure 4.18)

W = Min (W_1 , W_2) = 7 mm

Effective throat thickness of weld (t_w) = 0.707 x W = 5 mm

Length of weld (L_w) = b_{s2} – $2t_w$ = 136 - 10 = 126 mm

Nominal strength of weld (P_n) = t x L_w x F_{ur} = (4 x 126 x 420) / 1000 = 211.68 KN

Design strength of weld (P_{nd}) = 2 x ϕ_t x P_n= 2 x 0.65 x 211.68 = 275.2 KN

So:

Design strength of weld is larger than design tension force as a result this weld can endure the applied design compression force.

Nominal strength of control weld (P_a) = 0.75 x t_w x L_w x F_{xx} = (0.75 x 5 x 126 x 827.37) / 1000 = 391 KN

Design strength of control weld (P_{ad}) = 2 x ϕ_a x P_a = 2 x 0.6 x 391 = 469 KN

Condition (a): $P_{nd} \leq P_{ad}$, have to be provided.

So:

This weld provided the condition (a).

- Welding 6: Welding of supplementary web plate stiffener to column

The created compression force are larger than created tension force so the weld are designed according to compression force and also shear force.

The welds are designed according to cold-formed steel sections welding design rules.

The welding of supplementary web plate stiffener length to column web is longitudinal welding for design shear force and the welding of full depth horizontal stiffener width to column web is longitudinal welding for design compression force and is transverse welding for design shear force.

Number of supplementary web plate stiffeners (n_s) = 2

Moment lever arm (h) = d_r + d_h + t_{fh}= 650 + 274 + 12 = 936 mm

Design tension force (P_{f1}) = [F_{1C} + (M_2 / h)] / n_s = [73.3 + (292 / 0.936)] / 2 = 192.6 KN

Design shear force (P_{f1}) = F_3 / n_s = 125 / 2 = 62.5 KN

  - Welding of stiffener length to column web

Design thickness (t) = Min (t_{s3} , $2t_c$) = Min (5 , 8) = 5 mm

Leg of weld (W_1) = 7 mm (see figure 4.18)

Leg of weld (W_2) = 7 mm (see figure 4.18)

W = Min (W_1 , W_2) = 7 mm

Effective throat thickness of weld (t_w) = 0.707 x W = 5 mm

Length of weld (L_w) = L_{s3} – 2tw = 946 – 10 = 936 mm

The 'L_w / t ' ratio is larger than 25 so nominal strength of weld (P_n) is calculated as below:

Nominal strength of weld (P_n) = 0.75 x t x L_w x F_{ur} = (0.75 x 5 x 936 x 420) / 1000 = 1474.2 KN

Design strength of weld (P_{nd}) = 2 x ϕ_{L2} x P_n= 2 x 0.5 x 1474.2 = 1474.2 KN

So:

Design strength of weld is larger than design shear force as a result this weld can endure the applied design shear force.

Nominal strength of control weld (P_a) = 0.75 x t_w x L_w x F_{xx} = (0.75 x 5 x 936 x 827.37) / 1000 = 2904 KN

Design strength of control weld (P_{ad}) = 2 x ϕ_a x P_a = 2 x 0.6 x 2904 = 3485 KN

Condition (a): $P_{nd} \leq P_{ad}$, have to be provided.

So:

This weld provided the condition (a).

- Welding of stiffener widths to column web

Design thickness (t) = Min (t_{s3} , $2t_c$) = Min (5 , 8) = 5 mm

Leg of weld (W_1) = 7 mm (see figure 4.18)

Leg of weld (W_2) = 7 mm (see figure 4.18)

W = Min (W_1 , W_2) = 7 mm

Effective throat thickness of weld (t_w) = 0.707 x W = 5 mm

Length of weld (L_w) = b_{s3} – 2tw = 570 – 10 = 560 mm

As longitudinal weld:

The 'L_w / t' ratio is larger than 25 so nominal strength of weld (P_n) is calculated as below:

Nominal strength of weld (P_n) = 0.75 x t x L_w x F_{ur} = (0.75 x 5 x 560 x 420) / 1000 = 882 KN

Design strength of weld (P_{nd}) = ϕ_{L2} x P_n= 0.5 x 882 = 441 KN

So:

Design strength of weld is larger than design tension force as a result this weld can endure the applied design tension force.

Nominal strength of control weld (P_a) = 0.75 x t_w x L_w x F_{xx} = (0.75 x 5 x 560 x 827.37) / 1000 = 1737.5 KN

Design strength of control weld (P_{ad}) = ϕ_a x P_a = 0.6 x 1737.5= 1042.5 KN

Condition (a): $P_{nd} \leq P_{ad}$, have to be provided.

So:

This weld provided the condition (a).

As transverse weld:

Nominal strength of weld (P_n) = t x L_w x F_{ur} = (5 x 560 x 420) / 1000 = 1176 KN

Design strength of weld (P_{nd}) = 2 x ϕ_t x P_n= 2 x 0.65 x 1176 = 1529 KN

So:

Design strength of weld is larger than design shear force as a result this weld can endure the applied design shear force.

Nominal strength of control weld (P_a) = 0.75 x t_w x L_w x F_{xx} = (0.75 x 5 x 560 x 827.37) / 1000 = 1737.5 KN

Design strength of control weld (P_{ad}) = 2 x ϕ_a x P_a = 2 x 0.6 x 1737.5= 2085 KN

Condition (a): $P_{nd} \leq P_{ad}$, have to be provided.

So:

This weld provided the condition (a).

- Welding 7: Welding of K diagonal stiffener to column

The welding of plates (a) and plates (b) of K diagonal stiffeners to column are the same as the welding of the plates (a) and plates (b) of full depth horizontal stiffeners which are described in welding 5.

The welding of plates (c) widths to column flanges are the same as the welding of plates (a) and plates (b) widths welding to column flanges.

The welding of plates (c) length to column web are designed according to shear force.

The welds are designed according to cold-formed steel sections welding design rules.

The welding of plates (c) length to column web is longitudinal welding for design shear force.

Number of plates (c) of K diagonal stiffener (n_s) = 2

Design shear force (P_{f1}) = (F_3 / cos31) / n_s = 145.86 / 2 = 73 KN

- Welding of plates (c) length to column web

Design thickness (t) = Min (t_{s4} , $2t_c$) = Min (5 , 8) = 5 mm

Leg of weld (W_1) = 7 mm (see figure 4.18)

Leg of weld (W_2) = 7 mm (see figure 4.18)

W = Min (W_1 , W_2) = 7 mm

Effective throat thickness of weld (t_w) = 0.707 x W = 5 mm

Length of weld (L_w) = L_{s4} – 2tw = 1053 – 10 = 1043 mm

The 'L_w / t' ratio is larger than 25 so nominal strength of weld (P_n) is calculated as below:

Nominal strength of weld (P_n) = 0.75 x t x L_w x F_{ur} = (0.75 x 5 x 1043 x 420) / 1000 = 1642.7 KN

Design strength of weld (P_{nd}) = ϕ_{L2} x P_n= 0.5 x 1642.7 = 821.35 KN

So:

Design strength of weld is larger than design shear force as a result this weld can endure the applied design shear force.

Nominal strength of control weld (P_a) = 0.75 x t_w x L_w x F_{xx} = (0.75 x 5 x 1043 x 827.37) / 1000 = 3236 KN

Design strength of control weld (P_{ad}) = ϕ_a x P_a = 0.6 x 3236 = 1941.6 KN

Condition (a): $P_{nd} \leq P_{ad}$, have to be provided.

So:

This weld provided the condition (a).

4.2.3.2 Bolts

The bolt hole diameter of M22 (0.87 in) bolts is 23.6 mm (0.93 in) according to Table 4.3 [35].

Table 4.3: Maximum size of bolt holes, in AISI S100 specification [35].

Nominal Bolt Diameter, d in.	Standard Hole Diameter, d_h in.	Oversized Hole Diameter, d_h in.	Short-Slotted Hole Dimensions in.	Long-Slotted Hole Dimensions in.
$< 1/2$	$d + 1/32$	$d + 1/16$	$(d + 1/32)$ by $(d + 1/4)$	$(d + 1/32)$ by $(2 1/2\ d)$
$\geq 1/2$	$d + 1/16$	$d + 1/8$	$(d + 1/16)$ by $(d + 1/4)$	$(d + 1/16)$ by $(2 1/2\ d)$

The bolt diameter (d_b) is 0.87 in (22 mm)which is larger than 0.5 in (1 / 2 = 0.5 in) so the standard hole diameter (d_0) is 0.87 + (1 / 16) = 0.93 in that it is equal to 23.6 mm.

The bolt designing calculations of designed Eave joints are done into 6 items as listed below [35]:

- Spacing and edge distance control:
- Nominal shear strength of bolt control as affected by spacing and edge distance
- Bearing strength control of plates
- Shear strength control of bolts
- Tensile strength control of bolts
- Rupture controls of plates

The calculations of each item are given in the following:

- Spacing and edge distance control

As it is shown in Figure 4.19, e_2 is a distance between the center of bolt hole to the nearest edge of the end plate in the prependicular direction of shear force direction, and e_1 is a distance between the center of bolt hole to the nearest edge of the end plate in the parallel direction of shear force direction.

p_2 is a distance between the centers of two adjacent bolt holes in the prependicular direction of shear force direction, and p_1 is a distance between the centers of two adjacent bolt holes in the parallel direction of shear force direction.

These distances in the considered Eave joints are given in Table 4.4.

Table 4.4: Defined edge distance and middle distance between the bolts centers in End plate and flange backing plate.

	e_1	e_2	p_1	p_2
End plate	110	80	80	160
Flange backing plate	110	69.5	80	160

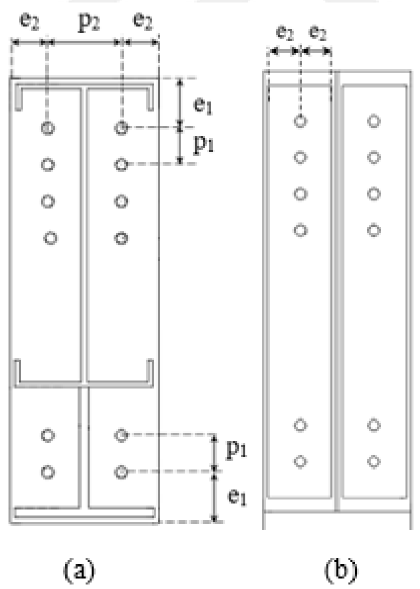


Figure 4.20: edge distance and middle distance between the bolts centers; (a) End plate; (b) Flange backing plate.

The conditions (b) and (c) have to be provided.

Condition (b): p_1 and p_2 have to be more than 3 times of bolt hole diameter.

So:

p_1 and p_2 are larger than 3 times of bolt hole diameter (3 x d_0 = 3 x 23.6 = 70.8 mm) as a result the condition (b) is provided.

Condition (c): e_1 and e_2 have to be more than 1.5 times of bolt hole diameter.

So:

e_1 and e_2 in both end plate and flange backing plate are larger than 1.5 times of bolt hole diameter (1.5 x d0 = 1.5 x 23.6 = 35.4 mm) as a result the condition (c) is provided in both end plate and flange backing plate.

- Nominal shear strength of plates control as affected by spacing and edge distance

The nominal shear strength of plates in the connection of end plate to column flange are controlled as below:

The material and the thickness of flange backing plate are the same as the end plate so the nominal shear strength of plates in the connection of column flange to flange backing plates are controlled in the same way.

Total number of bolts (n) = $n_t + n_c$ = 8 + 4 = 12

Design shear force = F_3 / n = 125 / 12 = 10.42 KN

Design tensile sress (F_{ud}) = Min (F_{uep} , F_{uc}) = 420 N/mm^2

Design yield stress (F_{yd})= Min (F_{yep} , F_{yc}) = 350 N/mm^2

Design thickness (t) = Min (t_{ep}, t_c) = 4 mm

e is the nearest distance to the edge in line of force so e is obtaine as below:

e = Min (e_1 , [p_1 - (d_0 / 2)]) = Min (110 , [80 – (d_0 / 2)]) = Min (110 , 68.2) = 68.2 mm

Nominal shear strength per bolt (P_n) : t x e x Fu = (4 x 68.2 x 420) / 1000 = 114.58 KN

The 'F_{ud} / F_{yd}' ratio is larger than 1.08 so the design coefficient (ϕ_{esd}) is 0.7 [35].

Design shear strength per bolt (P_{nd}) = ϕ_{esd} x P_n = 0.7 x 114.58 = 80.2 KN

So:

The bolt which could be affected by the edge distance and spacing can endure the design shear force.

- Bearing strength control of plates

The bearing strength of plates in the connection of bolted end plate to column flange are controlled as below:

The material and the thickness of flange backing plate are the same as the end plate so the bearing strength of plates in the connection of column flange to flange backing plates are controlled in the same way.

Deformation around the bolt hole is not considered.

Design shear force= F_3 =125 KN

Design thickness (t) = Min (t_{ep}, t_c) = 4 mm

Bearing factor is determined according to Table 4.5 [35].

Table 4.5: Bearing factor in AISI S100 specification [35].

Thickness of Connected Part, t, in. (mm)	Ratio of Fastener Diameter to Member Thickness, d/t	C
$0.024 \le t < 0.1875$ ($0.61 \le t < 4.76$)	$d/t < 10$	3.0
	$10 \le d/t \le 22$	4 - 0.1(d/t)
	$d/t > 22$	1.8

The 'd_b / t' ratio is equal to 5.5 (22 / 4 = 5.5), so the bearing factor (C) is 3.

Modification factor is determined according to Table 4.6 [35].

Table 4.6: Modification factor for type of bearing connection.

Type of Bearing Connection	m_f
Single Shear and Outside Sheets of Double Shear Connection with Washers under Both Bolt Head and Nut	1.00
Single Shear and Outside Sheets of Double Shear Connection without Washers under Both Bolt Head and Nut, or with only One Washer	0.75
Inside Sheet of Double Shear Connection with or without Washers	1.33

As it is mentioned before, the washers are used under both bolt head and nut. So the modification factor (m_f) is 0.75.

Design tensile stress (F_{ud}) = Min (F_{uep} , F_{uc}) = 420 N/mm^2

Nominal bearing strength of plate (P_n) = C x m_f x d_b x t x F_u = (3 x 0.75 x 22 x 4 x 420) / 1000 = 83.16 KN

Design coefficient (ϕ_{bd}) = 0.6

Number of bolts which are exposed to shear force only (n_{shear}) = n_c

Design bearing strength of plate (P_{nd}) = n_{shear} x ϕ_{bd}x P_n = 4 x 0.6 x 83.16 = 199.58 KN

So:

The column flange can endure the design shear force.

- Shear strength control of bolts

To being more conservative only the shear strength of the bolts which are in the compression zone are considered.

Design shear force= F_3 =125 KN

Design thickness (t) = Min (t_{ep}, t_c) = 4 mm

Nominal shear strength of bolt (P_n) = A_b x F_{nv} = (380.13 x 496) / 1000 = 188.55 KN

Design coefficient (ϕ_{sd}) = 0.65

Number of bolts which are assumed to be exposed to shear force (n_{shear}) = n_c

The connection is double shear connection so the shear strength of bolts are multiply with 2.

Design shear strength of bolts (P_{nd}) = 2 x [n_{shear} x ϕ_{sd}x P_n] = 2 x 4 x 0.65 x 188.55 = 980.4 KN

So:

The bolts can endure the design shear force.

- Tensile strength control of bolts

The tensile strength of bolts in tension zone are decreased because they are exposed to shear force simultaneously.

Moment lever arm (h) = d_r + d_h + t_{fh} = 650 + 274 + 12 = 936 mm

Design tension force = F_{1t} + (M_2 / h) = 70 + (292 / 0.936) = 381.97 KN

Allowable shear stress (F_{anv}) = ϕ_{sd}x F_{nv} = 0.65 x 496 = 322.4 N/mm^2

Decreased tensile strength (F'_{nt}) = (1.3 x F_{nt}) - [(F_{nt} / F_{anv}) x f_v] = 568.45 N/mm^2

Nominal tensile strength of bolt (P_n) = A_b x F'_{nt} = (380.13 x 568.45) / 1000 = 216 KN

Design coefficient (ϕ_{td}) = 0.75

Number of bolts which are assumed to be exposed to shear force and tension force simultaneously ($n_{shear\text{-}tension}$) = n_t

The connection is double shear connection so the shear strength of bolts are multiply with 2.

Design tensile strength of bolts (P_{nd}) = 2 x [$n_{shear\text{-}tension}$ x ϕ_{sd}x P_n] = 2 x 8 x 0.75 x 216 = 2592 KN

So:

The bolts can endure the design tension force.

- Rupture control of plates

There is 3 type of rupture control in bolted connection. These rupture controls are listed as below:

- Shear rupture
- Tension rupture
- Block shear rupture

The connection is double shear connection, the distribution of shear force is shown at figure 4.21 [90].

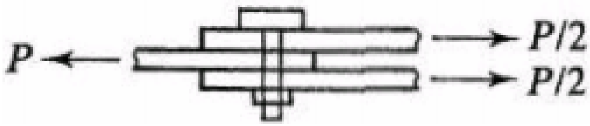


Figure 4.21: Distribution of shear force in double shear connection [90].

So the column flange which is between the end plate and flange backing plate is the more critical section to shear rupture so the rupture strength of column flange has been controlled only.

These rupture contols fo column flange are given in the following.

- Shear rupture

The possible shear rupture of column flange is shown in Figure 4.22.

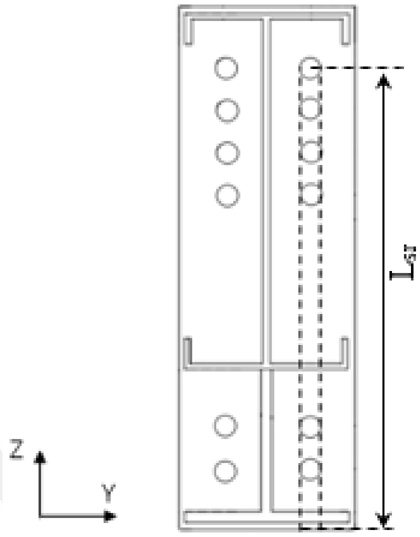


Figure 4.22: Possible shear rupture of column flange in considered Eave joints.

Design shear force= F_3 =125 KN

Number of bolts in Z direction (n) = 6

Design thickness (t) = t_c = 4mm

Shear rupture length (L_{sr}) = 860 mm

Net area which is exposed to shear force (A_n) = 2 x [L_{sr} – (((n – 1) + 0.5) x d_h)] x t = 2 x [860 – (5.5 x 23.6)] x 4 = 5841.6 mm^2

Nominal shear rupture strength of plate (P_n) = 0.6 x A_n x F_{uc} = (0.6 x 5841.6 x 420) / 1000 = 1472 KN

Design coefficient (ϕ_{srd}) = 0.75

Design shear rupture strength of plate (P_{nd}) = ϕ_{srd}x P_n = 0.75 x 1472 = 1104 KN

So:

There will not be any shear rupturing in the considered bolted connections.

- Tension rupture

There is no staggered hole patterns.

The load is not transmitted directly to all of the cross-sectional elements.

The possible tension rupture of column flange is shown in Figure 4.23.

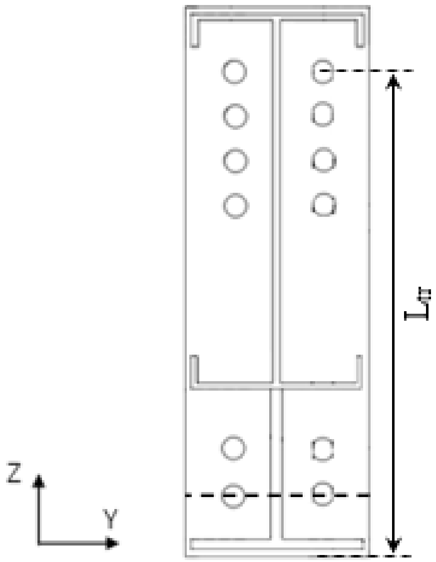


Figure 4.23: Possible tension rupture of column flange in considered Eave joints.

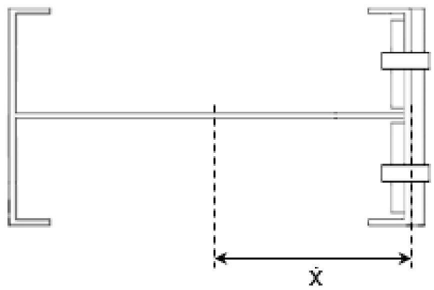


Figure 4.24: Distance from shear plane to centroid of the cross section.

Design shear force= F_3 =125 KN

Number of bolts in Y direction (n) = 2

Net area which is exposed to tension force (A_n) = [b_c - (n x d_0)] x t_c = [320 – (2 x 23.6)] x 4 = 1091.2 mm^2

Tension rupture length (L_{st}) = 860 mm

As it is shown in Figure 4.24, the distance from shear plane to centroid of the cross section ($\dot{x}$) = 300 mm

U = 1.0 –[0.36 ($\dot{x}$ / L_{tr})] = 1 – (0.36 x (300 / 860)) = 0.87

Effective net area which is exposed to tension force (A_{eff}) = U x A_n = 0.87 x 1091.2 = 949.3 mm^2

Nominal tension rupture strength of plate (P_n) = A_{eff} x F_{uc} = (949.3 x 420) / 1000 = 398.7 KN

Design coefficient (ϕ_{trd}) = 0.65

Design shear rupture strength of plate (P_{nd}) = ϕ_{trd}x P_n = 0.65 x 398.7 = 259.2 KN

So:

There will not be any tension rupturing in the considered bolted connections.

- Block shear rupture

The possible modes of block shear rupture is shown in Figure 4.25.

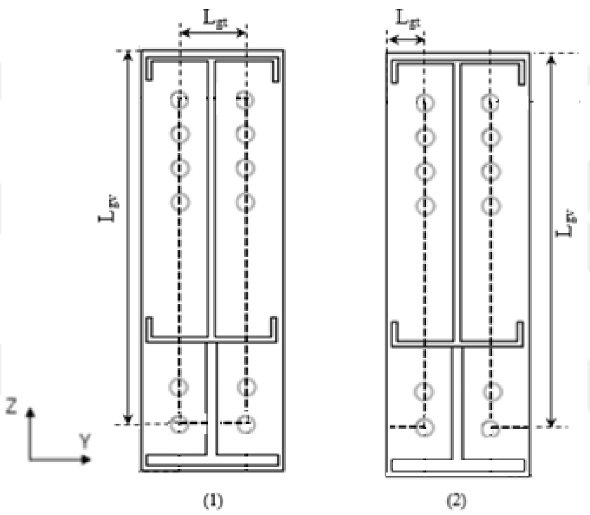


Figure 4.25: Possible block shear rupture of column flange in considered Eave joints.

- Mode (1)

L_{gv} = 860 mm

Number of bolts in Z direction (n_z) = 5.5

$L_{nv} = L_{gv} – [((n_z-1) + 0.5) x d_h]$ = 860 – (5.5 x 23.6) = 730.2 mm

Gross area which is subject to shear force (A_{gv}) = 2 x L_{gv} x t_c = 2 x 860 x 4 = 6880 mm^2

Net area which is subject to shear force (A_{nv}) = 2 x L_{nv} x t_c = 2 x 730.2 x 4 = 5841.6 mm^2

L_{gt} = 160 mm

Number of bolts in Y direction (n_y) = 2

$L_{nt} = L_{gt}$ – [(n_y – 1) x d_h) = 160 – (1 x 23.6) = 136.4 mm

Net area which is subject to tension force (A_{nt}) = L_{nt} x t_c = 136.4 x 4 = 545.6 mm^2

Nominal block shear rupture strength (P_{n1}) = (0.6 x F_{yc} x A_{gv}) + (F_{uc} x A_{nt}) = [(0.6 x 350 x 6880) + (420 x 545.6)] / 1000 = 230.6 KN

Nominal block shear rupture strength (P_{n2}) = (0.6 x F_{uc} x A_{nv}) + (F_{uc} x A_{nt}) = [(0.6 x 420 x 5841.6) + (420 x 545.6)] / 1000 = 230.6 KN

Nominal block shear rupture strength (P_n) = Min (P_{n1} , P_{n2}) = 230.6 KN

So:

There will not be any block shear rupturing as mode (1) in the considered bolted connections.

- Mode (2)

L_{gv} = 860 mm

Number of bolts in Z direction (n_z) = 5.5

$L_{nv} = L_{gv}$ – [((n_z-1) + 0.5) x d_h] = 860 – (5.5 x 23.6) = 730.2 mm

Gross area which is subject to shear force (A_{gv}) = 2 x L_{gv} x t_c = 2 x 860 x 4 = 6880 mm^2

Net area which is subject to shear force (A_{nv}) = 2 x L_{nv} x t_c = 2 x 730.2 x 4 = 5841.6 mm^2

L_{gt} = 80 mm

Number of bolts in Y direction (n_y) = 2

$L_{nt} = L_{gt}$ – [((n_y – 1) / 2) x d_h) = 80 – (0.5 x 23.6) = 68.2 mm

Net area which is subject to tension tension (A_{nt}) =2 x L_{nt} x t_c = 2 x 68.2 x 4 = 545.6 mm^2

Nominal block shear rupture strength (P_{n1}) = (0.6 x F_{yc} x A_{gv}) + (F_{uc} x A_{nt}) = [(0.6 x 350 x 6880) + (420 x 545.6)] / 1000 = 230.6 KN

Nominal block shear rupture strength (P_{n2}) = (0.6 x F_{uc} x A_{nv}) + (F_{uc} x A_{nt}) = [(0.6 x 420 x 5841.6) + (420 x 545.6)] / 1000 = 230.6 KN

Nominal block shear rupture strength (P_n) = Min (P_{n1} , P_{n2}) = 230.6 KN

So:

There will not be any block shear rupturing as mode (1) in the considered bolted connections.

4.3 Design Models of Eave Joint

As it is determined in the 4.2 section, there are 3 type of Eave joints which are different according to column stiffeners.

The 3 types of Eave joints are:

- Eave joint with full depth horizontal column stiffener
- Eave joint with supplementary web plate column stiffener
- Eave joint with K diagonal column stiffener

The 3 types of Eave joints are modeled in IDEA StatiCa V10 program and analyzed by finite element method.

Because the lip of two back to back cold formed channel sections are not applicable in IDEA StatiCa 10 program so the lip of column and rafter sections are not modeled but this does not change the 3 types of Eave joints comparison results.

In all 3 models, the column profile set as bearing and the plastic strain limit is 5%.

The resistance factors which are used in analysing the models by LRFD method in the IDEA Statica V10 program are defined as Figure 4.26.

LRFD - Resistance factors ϕ	
Tensile and shear strength - bolts	0.75
Combined tensile and shear strength - bolts	0.75
Bearing at bolt holes	0.75
Fillet welds	0.75
Material resistance factor	0.9
Slip resistant joint	1
Strength reduction factor for anchors in tension	0.7
Strength reduction factor for anchors in shear	0.65

Figure 4.26: Defined resistance factors in IDEA StatiCa V10 program.

All 3 types of Eave joints are modeled as rigid joint, it means that they can transfer axial force, shear forces and moments.

It should be noted that the created internal forces are applied in the node. The point which the rafter is connected to the column is called node.

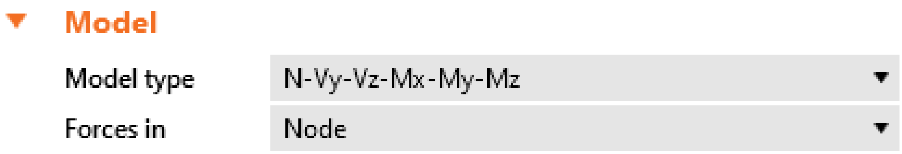
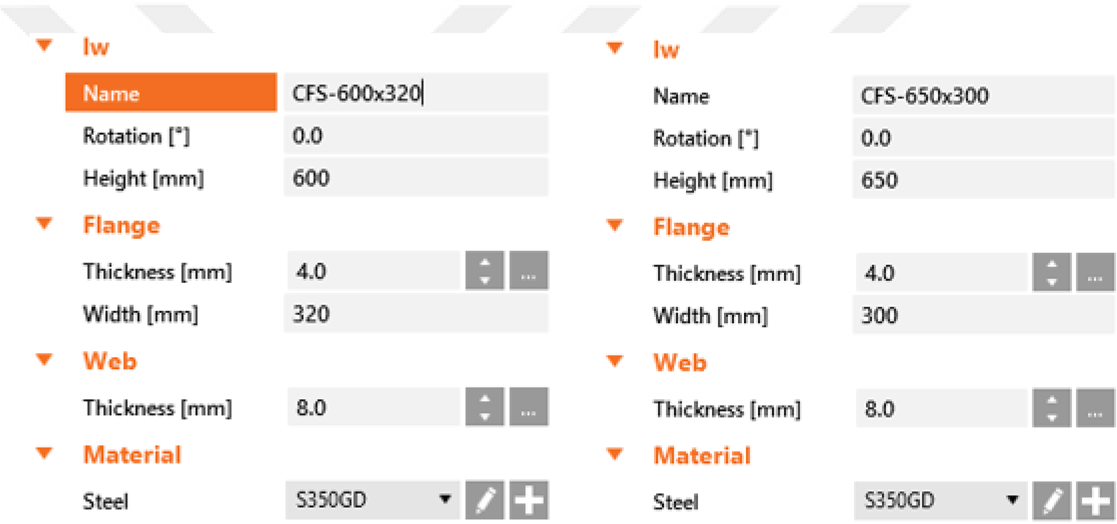


Figure 4.27: Defined conditions for models in IDEA StatiCa V10 program.

The column and the rafter sections in IDEA Statica V10 program are defined as Figure 4.28.



(a) (b)

Figure 4.28: Defined column and rafter sections in IDEA StatiCa V10 program; (a) column section; (b) rafter section.

The A325 bolts are defined in IDEA Statica V10 program as Figure 4.29.

Properties	
Name	7/8 A325
Bolt grade	A325
Bolt	
Diameter [mm]	22
Hole for bolt [mm]	24
Head diameter [mm]	37
Head diagonal diameter [mm]	42
Head height [mm]	15
Gross Cross-section area [mm2]	388
Tensile stress area [mm2]	298
Nut	
Thickness [mm]	22
Washer	
Thickness [mm]	5
At the head	✓
At the nut	✓

Figure 4.29: Defined A325 bolts in IDEA StatiCa V10 program.

4.3.1 Type (1): Eave joint with full depth horizontal column stiffener

The considered type (1) of Eave joint is modeled in IDEA StatiCa 10 program as it is shown in Figure 4.30.

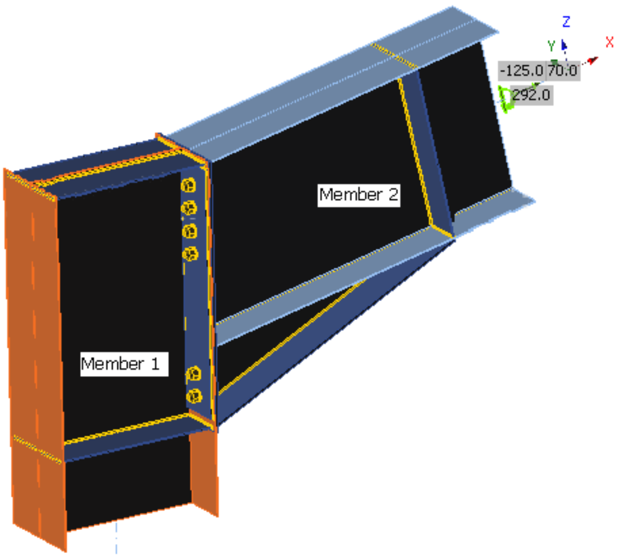


Figure 4.30: The modeled type (1) Eave joint in IDEA StatiCa 10 program.

The labels of considered connection components is shown in Figure 4.31.

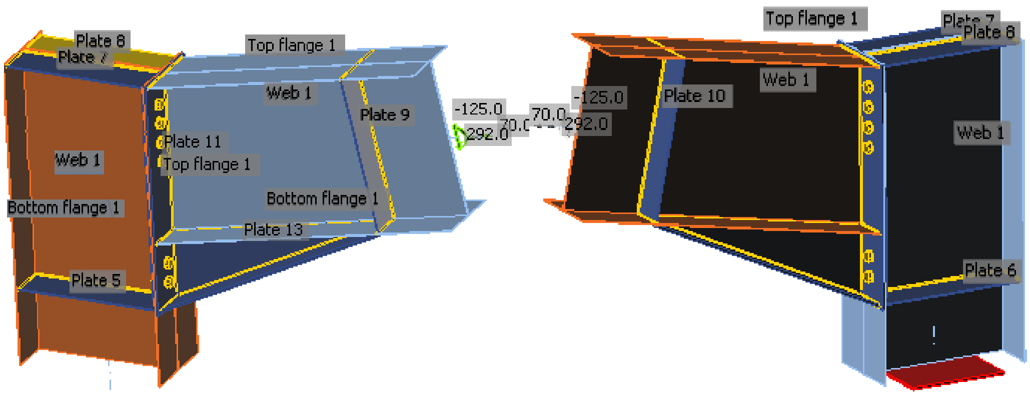


Figure 4.31: The labels of connection components in modeled type (1) Eave joint in IDEA StatiCa 10 program.

The applied loads in considered model are shown in Table4.7.

Table 4.7: The applied loads in type (1) model of Eave joint in IDEA StatiCa 10 program.

	Member	N [kN]	Vy [kN]	Vz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
>	Member 1 / End	0.0	0.0	0.0	0.0	0.0	0.0
	Member 2 / End	70.0	0.0	-125.0	0.0	292.0	0.0

The analyzed results summary of type (1) of Eave joint are shown in Table 4.8 and Figure 4.32.

Table 4.8: The analyzed results summary of type (1) model of Eave joint.

Summary

Name	Value	Check status
Analysis	100.0%	OK
Plates	2.4 < 5%	OK
Bolts	87.0 < 100%	OK
Welds	91.5 < 100%	OK
Buckling	Not calculated	

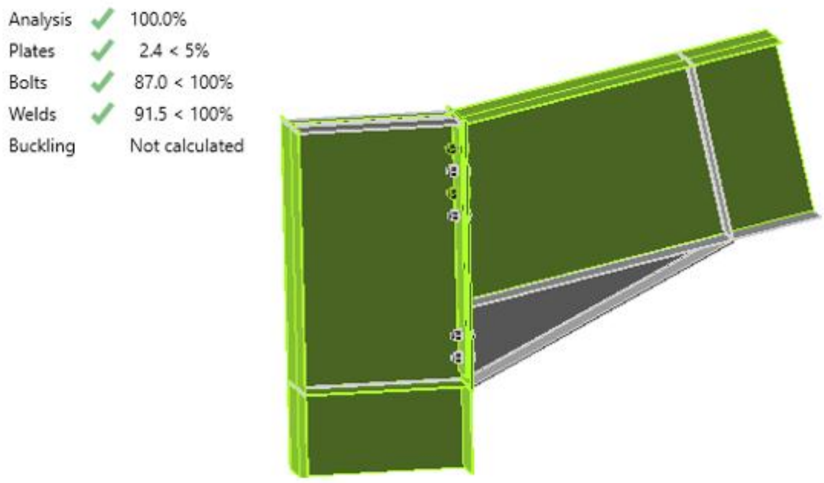


Figure 4.32: The analyzed results summary of type (1) model of Eave joint.

4.3.2 Type (2): Eave Joint with supplementary web plate column stiffener

The considered type (2) of Eave joint is modeled in IDEA StatiCa 10 program as it isshown in Figure 4.33.

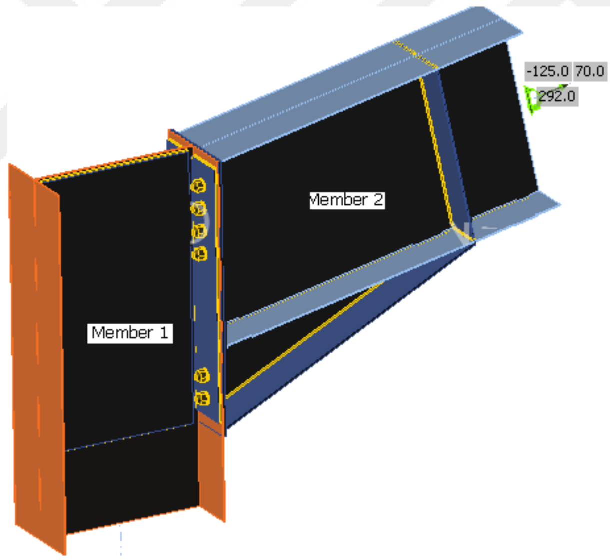


Figure 4.33: The modeled type (2) Eave joint in IDEA StatiCa 10 program.

The labels of considered connection components is shown in Figure 4.34.

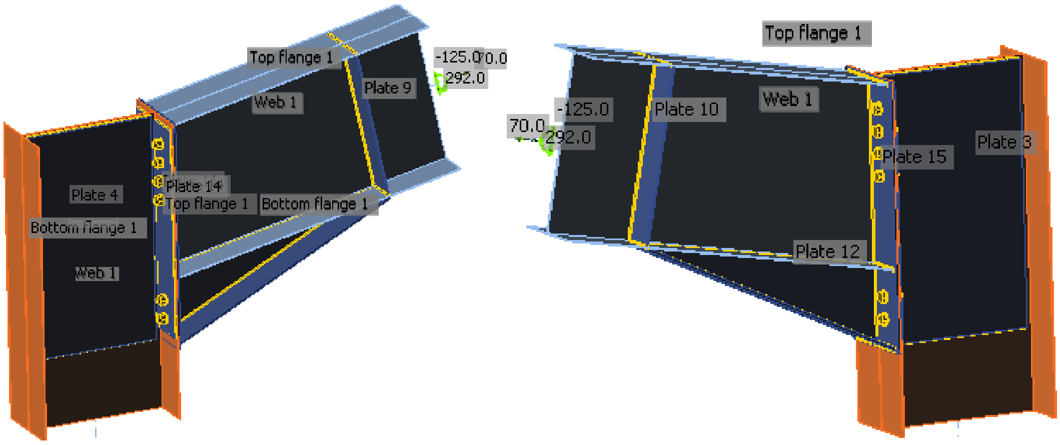


Figure 4.34: The labels of connection components in modeled type (2) Eave joint in IDEA StatiCa 10 program.

The applied loads in considered model are shown in Table 4.9.

Table 4.9: The applied loads in type (2) model of Eave joint in IDEA StatiCa 10 program.

	Member	N [kN]	Vy [kN]	Vz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
>	Member 1 / End	0.0	0.0	0.0	0.0	0.0	0.0
	Member 2 / End	70.0	0.0	-125.0	0.0	292.0	0.0

The analyzed results summary of type (2) of Eave joint are shown in Table 4.10 and Figure 4.35.

Table 4.10: The analyzed results summary of type (2) model of Eave joint.

Summary

Name	Value	Check status
Analysis	100.0%	OK
Plates	2.8 < 5%	OK
Bolts	82.0 < 100%	OK
Welds	96.4 < 100%	OK
Buckling	Not calculated	

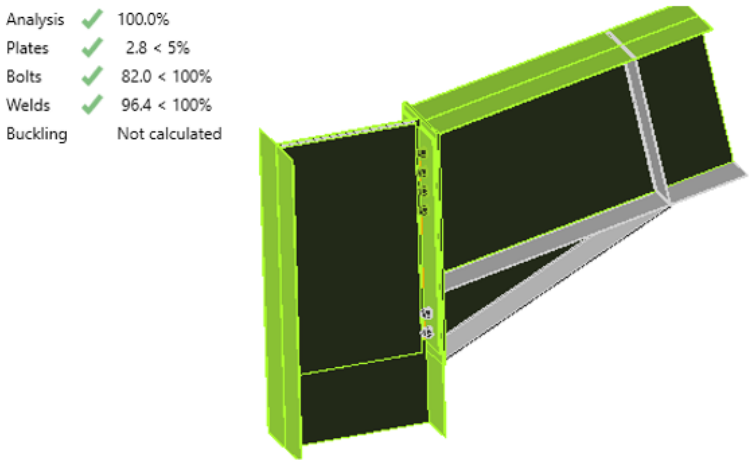


Figure 4.35: The analyzed results summary of type (2) model of Eave joint.

4.3.3 Type (3): Eave Joint with K diagonal column stiffener

The considered type (3) of Eave joint is modeled in IDEA StatiCa 10 program as it is shown in Figure 4.36.

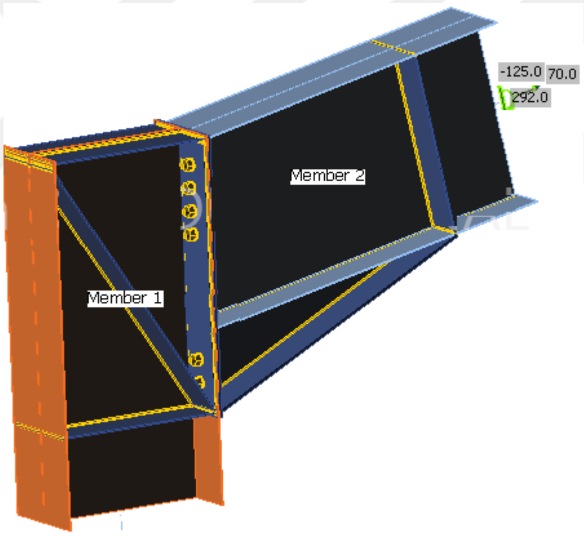


Figure 4.36: The modeled type (3) Eave joint in IDEA StatiCa 10 program.

The labels of considered connection components is shown in Figure 4.37.

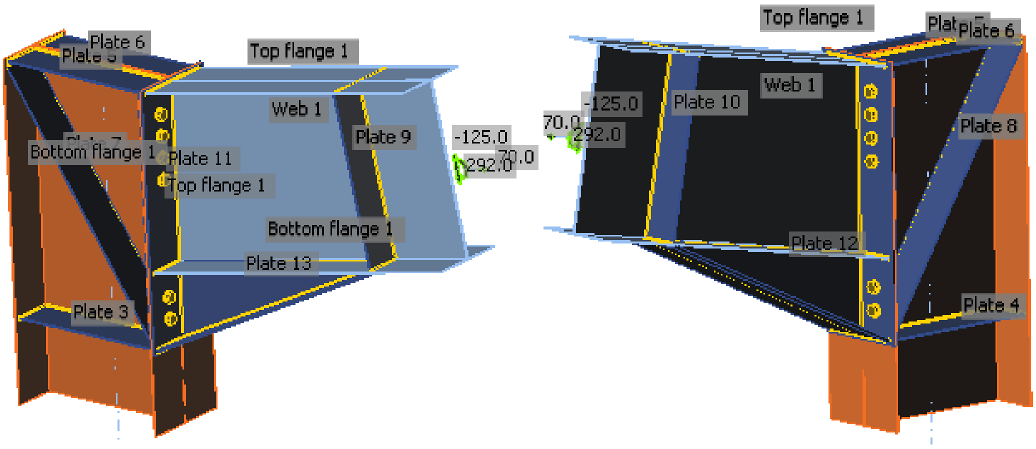


Figure 4.37: The labels of connection components in modeled type (3) Eave joint in IDEA StatiCa 10 program.

The applied loads in considered model are shown in Table 4.11.

Table 4.11: The applied loads in type (3) model of Eave joint in IDEA StatiCa 10 program.

	Member	N [kN]	Vy [kN]	Vz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
>	Member 1 / End	0.0	0.0	0.0	0.0	0.0	0.0
	Member 2 / End	70.0	0.0	-125.0	0.0	292.0	0.0

The analyzed results summary of type (3) of Eave joint are shown in Table 4.12 and Figure 4.38.

Table 4.12: The analyzed results summary of type (3) model of Eave joint.

Summary

Name	Value	Check status
Analsis	100.0%	OK
Plates	2.4 < 5%	OK
Bolts	80.6 < 100%	OK
Welds	84.0 < 100%	OK
Buckling	Not calculated	

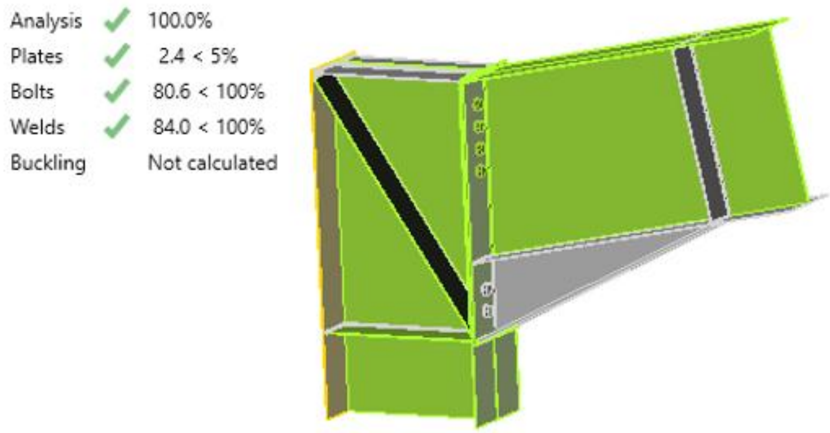


Figure 4.38: The analyzed results summary of type (3) model of Eave joint.

5 ANALYSIS RESULTS OF 3 TYPES OF DESIGNED EAVE JOINTS

The designed 3 models of Eave joint are evaluated into 2 categories:

- The connection components characteristics
- The connection characteristics

5.1 Connection Components Characteristics

The connection components which are evaluated in this section are column, beam and plates. The behavior of these components under applied loads are evaluated into 2 groups:

- The created strain on components
- The created stress on components

As it is known if a structural member is loaded it produces a stress that then causes a member to deform. Strain is defined as the amount of this deformation in the direction of the applied force divided by the initial length of the structural member [94].

In IDEA StatiCa program, the plates are modelled by using shell elements with elastic-plastic diagram of material.

5.1.1 Type (1) model of Eave joint

As it is expected, the largest strain and the largest stress are created in the tension region of the end plate and also the column flange and the column web which are exposed to compression force.

- The created strain

The created strain in the connection components of Eave joint with full depth horizontal column stiffener is shown in Figures 5.1 and 5.2.

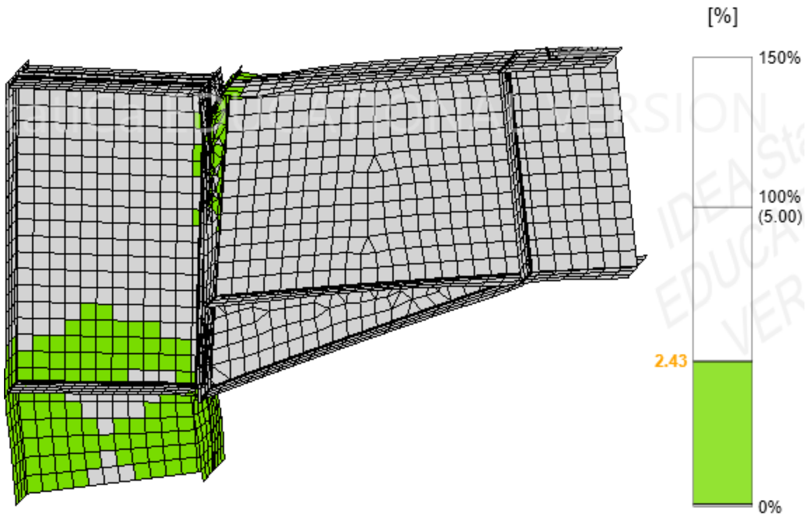
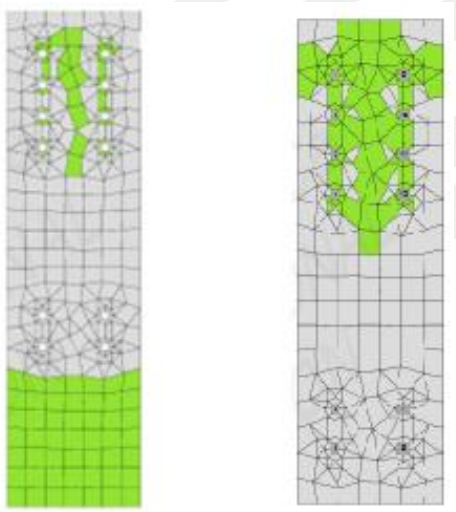


Figure 5.1: The created strain in the connection components of type (1) model of Eave joint.



(a) (b)

Figure 5.2: The distribution of created strain in the connection components of type (1) model of Eave joint; (a): Column flange which are connected to rafter; (b): End plate.

The largest created strain are 2.43 and 2.2 which are created in the column flange which are connected to rafter and the end plate respectively.

	Status	Item	Grade	Th [mm]	Loads	σ,Ed [MPa]	ε,Pl [%]
>		Member 1-tfl 1	S350GD	4.0	LE1	319.9	2.4
		Member 1-bfl 1	S350GD	4.0	LE1	318.7	1.9
		Member 1-w 1	S350GD	8.0	LE1	317.9	1.5
		Member 2-tfl 1	S350GD	4.0	LE1	285.6	0.0
		Member 2-bfl 1	S350GD	4.0	LE1	295.7	0.0

(a)

		Plate 9	S450JR	5.0	LE1	162.8	0.0
		Plate 10	S450JR	5.0	LE1	163.2	0.0
>		Plate 11	S450JR	12.0	LE1	400.4	2.2
		Plate 12	S450JR	10.0	LE1	157.3	0.0
		Plate 13	S450JR	10.0	LE1	245.3	0.0
		Plate 14	S450JR	12.0	LE1	397.2	0.6
		Plate 15	S450JR	12.0	LE1	397.3	0.6

(b)

Figure 5.3: The created strain in the connection components of type (1) model of Eave joint; (a): Column flange which are connected to rafter (member 1-tfl 1); (b): End plate (plate 11).

- The created stress

The created stress in the connection components of considered Eave joint is shown in Figures 5.4 and 5.5.

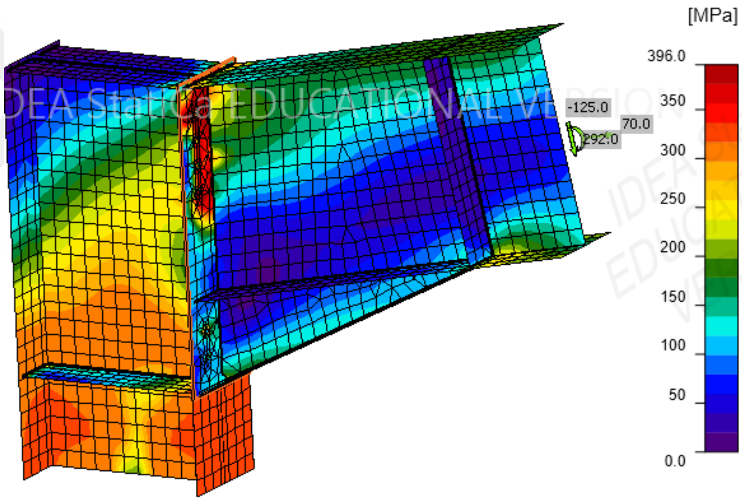
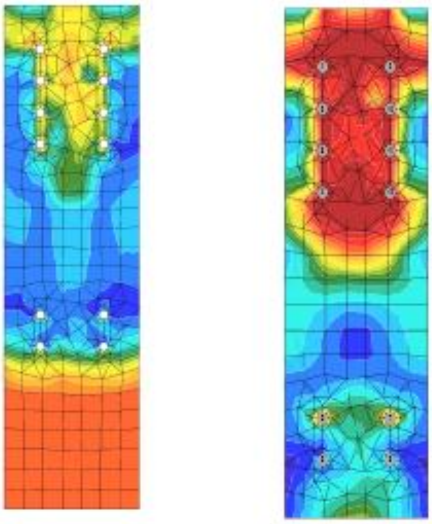


Figure 5.4: The created stress in the connection components of type (1) model of Eave joint.



(a) (b)

Figure 5.5: The distribution of created stress in the connection components of type (1) model of Eave joint; (a): Column flange which are connected to rafter; (b): End plate.

The largest created stress are 319.9 N/mm^2 and 400.4 N/mm^2 which are created in column flange that are connected to rafter and in the end plate respectively. It is shown in Figure 5.6.

The created stress in column flange is 319.9 N/mm^{2}is smaller than the yield stress of column steel material (F_{yc} = 350 N/mm^2)

And also, the created stress in end plate is 400.4 N/mm^{2}is smaller than the yield stress of end plate material (F_{yep} = 440 N/mm^2)

	Status	Item	Grade	Th [mm]	Loads	σ,Ed [MPa]	ε,Pl [%]
>	✓	Member 1-tfl 1	S350GD	4.0	LE1	319.9	2.4
	✓	Member 1-bfl 1	S350GD	4.0	LE1	318.7	1.9
	✓	Member 1-w 1	S350GD	8.0	LE1	317.9	1.5
	✓	Member 2-tfl 1	S350GD	4.0	LE1	285.6	0.0
	✓	Member 2-bfl 1	S350GD	4.0	LE1	295.7	0.0

(a)

	✓	Plate 9	S450JR	5.0	LE1	162.8	0.0
	✓	Plate 10	S450JR	5.0	LE1	163.2	0.0
>	✓	Plate 11	S450JR	12.0	LE1	400.4	2.2
	✓	Plate 12	S450JR	10.0	LE1	157.3	0.0
	✓	Plate 13	S450JR	10.0	LE1	245.3	0.0
	✓	Plate 14	S450JR	12.0	LE1	397.2	0.6
	✓	Plate 15	S450JR	12.0	LE1	397.3	0.6

(b)

Figure 5.6: The created stress in the connection components of type (1) model of Eave joint; (a): Column flange which are connected to rafter (member 1-tfl 1); (b): End plate (plate 11).

5.1.2 Type (2) model of Eave joint

The largest strain and the largest stress are created in the tension region of the end plate and also the column flange and the column web which are exposed to compression force in this model too.

- The created strain

The created strain in the connection components of Eave joint with supplementary web plate column stiffener is shown in Figures 5.7 and 5.8.

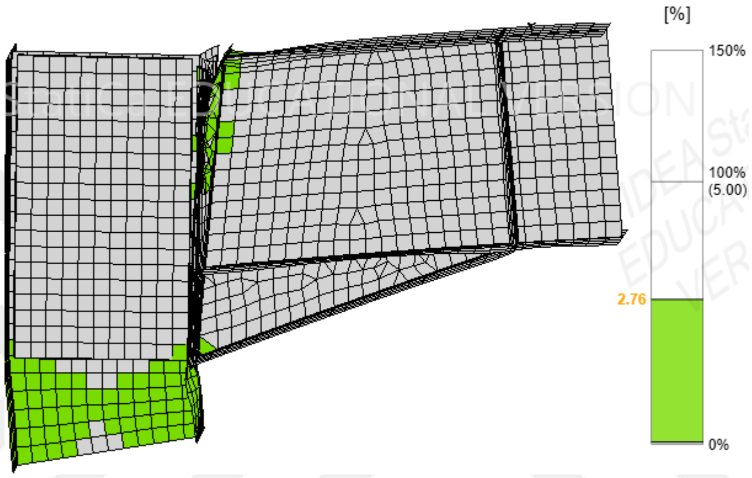


Figure 5.7: The created strain in the connection components of type (2) model of Eave joint.

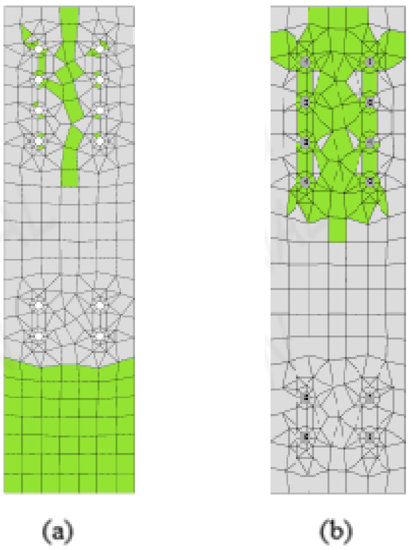


Figure 5.8: The distribution of created strain in the connection components of type (2) model of Eave joint; (a): Column flange which are connected to rafter; (b): End plate.

The largest created strain are 2.8 and 2.7 which are created in the column flange that are connected to the rafter and in the end plate respectively.

Status	Item	Grade	Th [mm]	Loads	σ,Ed [MPa]	ε,Pl [%]
	Member 1-tfl 1	S350GD	4.0	LE1	320.5	2.8
	Member 1-bfl 1	S350GD	4.0	LE1	319.7	2.4
	Member 1-w 1	S350GD	8.0	LE1	318.7	1.9
	Member 2-tfl 1	S350GD	4.0	LE1	291.7	0.0
	Member 2-bfl 1	S350GD	4.0	LE1	296.8	0.0

(a)

	Plate 9	S450JR	5.0	LE1	160.9	0.0
	Plate 10	S450JR	5.0	LE1	163.3	0.0
	Plate 11	S450JR	12.0	LE1	401.4	2.7
	Plate 12	S450JR	10.0	LE1	269.4	0.0
	Plate 13	S450JR	10.0	LE1	292.8	0.0
	Plate 14	S450JR	12.0	LE1	397.2	0.6
	Plate 15	S450JR	12.0	LE1	397.0	0.5

(b)

Figure 5.9: The created strain in the connection components of type (2) model of Eave joint; (a): Column flange which are connected to rafter (member 1-tfl 1); (b): End plate (plate 11).

- The created stress

The created stress in the connection components of considered joint is shown in Figures 5.10 and 5.11.

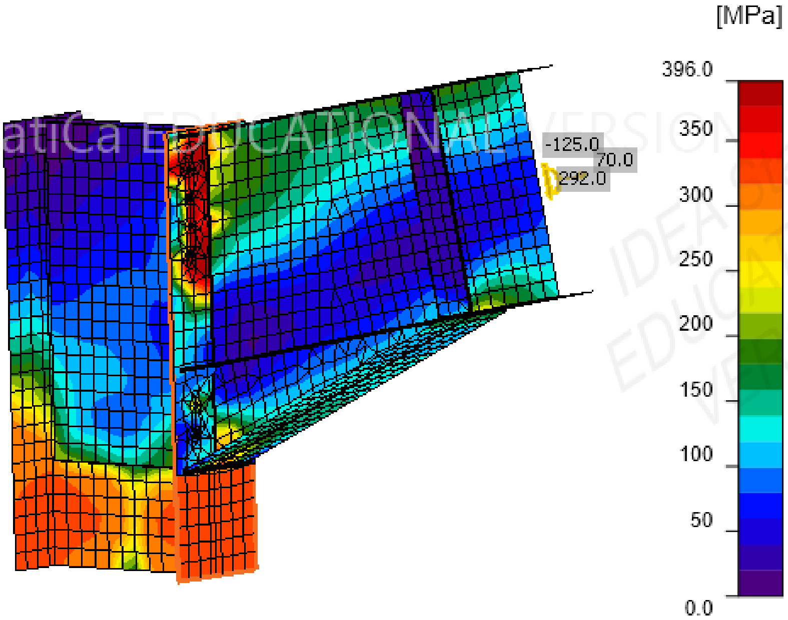


Figure 5.10: The created stress in the connection components of type (2) model of Eave joint.

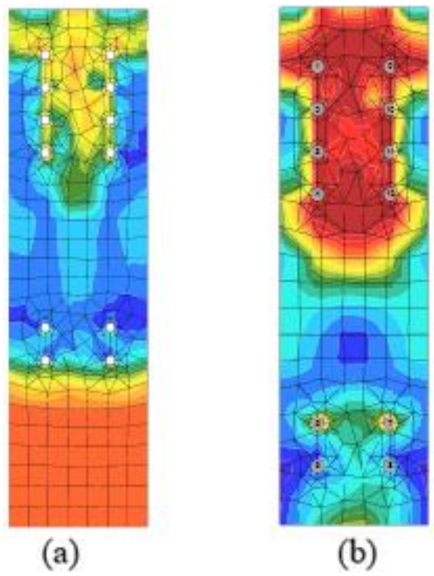


Figure 5.11: The distribution of created stress in the connection components of type (2) model of Eave joint; (a): Column flange which are connected to rafter; (b): End plate.

The largest created stress are 320.5 N/mm^2 and 401.4 N/mm^2 which are created in the column flange that are connected to the rafter and in the end plate respectively. It is shown in Figure 5.12.

The created stress in column flange is 320.5 N/mm^{2}is smaller than the yield stress of column steel material (F_{yc} = 350 N/mm^2).

And also, the created stress in end plate is 401.4 N/mm^{2}is smaller than the yield stress of end plate material (F_{yep} = 440 N/mm^2).

	Status	Item	Grade	Th [mm]	Loads	σ,Ed [MPa]	ε,Pl [%]
>	✓	Member 1-tfl 1	S350GD	4.0	LE1	320.5	2.8
	✓	Member 1-bfl 1	S350GD	4.0	LE1	319.7	2.4
	✓	Member 1-w 1	S350GD	8.0	LE1	318.7	1.9
	✓	Member 2-tfl 1	S350GD	4.0	LE1	291.7	0.0
	✓	Member 2-bfl 1	S350GD	4.0	LE1	296.8	0.0

(a)

	✓	Plate 9	S450JR	5.0	LE1	160.9	0.0
	✓	Plate 10	S450JR	5.0	LE1	163.3	0.0
	✓	Plate 11	S450JR	12.0	LE1	401.4	2.7
	✓	Plate 12	S450JR	10.0	LE1	269.4	0.0
	✓	Plate 13	S450JR	10.0	LE1	292.8	0.0
	✓	Plate 14	S450JR	12.0	LE1	397.2	0.6
	✓	Plate 15	S450JR	12.0	LE1	397.0	0.5

(b)

Figure 5.12: The created stress in the connection components of type (2) model of Eave joint; (a): Column flange which are connected to rafter (member 1-tfl 1); (b): End plate (plate 11).

5.1.3 Type (3) model of Eave joint

The largest strain and the largest stress are created in the tension region of the end plate and also the column flange and the column web which are exposed to compression force in this model too.

- The created strain

The created strain in the connection components of Eave joint with K diagonal column stiffener is shown in Figures 5.13 and 5.14.

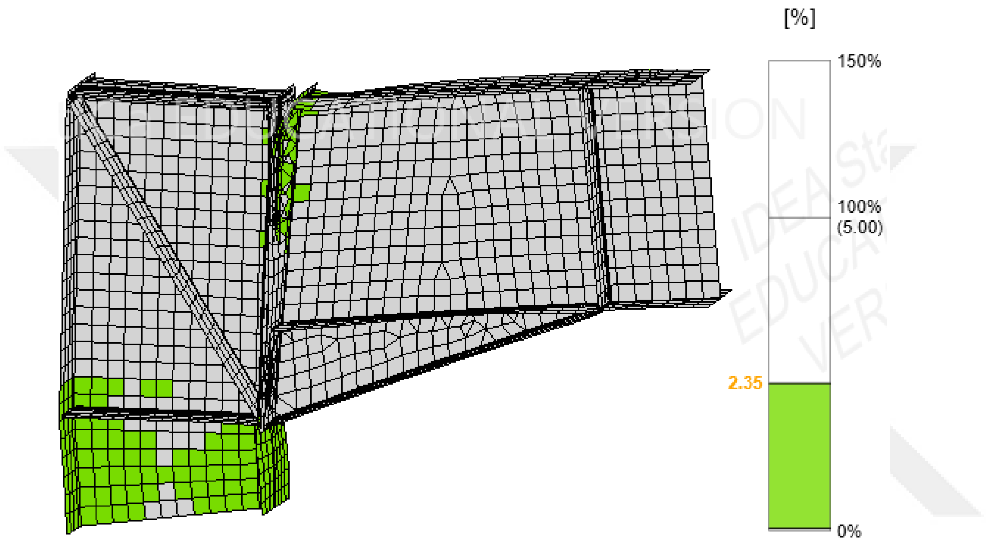


Figure 5.13: The created strain in the connection components of type (3) model of Eave joint.

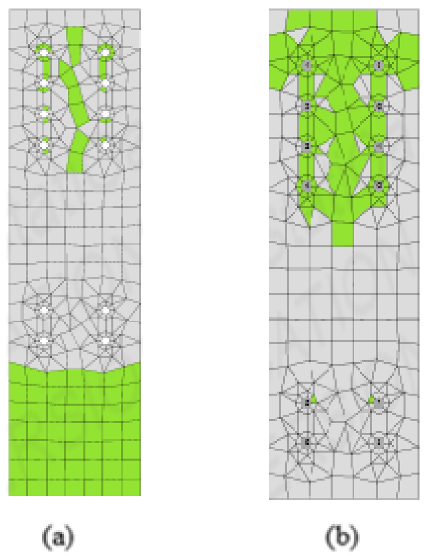


Figure 5.14: The distribution of created strain in the connection components of type (3) model of Eave joint; (a): Column flange which are connected to rafter; (b): End plate.

The largest created strain are 2.4 and 1.8 which are created in the column flange that are connected to the rafter and in the end plate respectively.

	Status	Item	Grade	Th [mm]	Loads	σ,Ed [MPa]	ε,Pl [%]
>	✓	Member 1-tfl 1	S350GD	4.0	LE1	319.7	2.4
	✓	Member 1-bfl 1	S350GD	4.0	LE1	318.7	1.8
	✓	Member 1-w 1	S350GD	8.0	LE1	318.0	1.5
	✓	Member 2-tfl 1	S350GD	4.0	LE1	282.9	0.0
	✓	Member 2-bfl 1	S350GD	4.0	LE1	286.8	0.0

(a)

	✓	Plate 8	S450JR	5.0	LE1	396.1	0.1
	✓	Plate 9	S450JR	5.0	LE1	140.6	0.0
	✓	Plate 10	S450JR	5.0	LE1	141.2	0.0
>	✓	Plate 11	S450JR	12.0	LE1	399.7	1.8
	✓	Plate 12	S450JR	12.0	LE1	131.9	0.0
	✓	Plate 13	S450JR	12.0	LE1	207.8	0.0
	✓	Plate 14	S450JR	12.0	LE1	397.0	0.5

(b)

Figure 5.15: The created strain in the connection components of type (3) model of Eave joint; (a): Column flange which are connected to rafter (member 1-tfl 1); (b): End plate (plate 11).

- The created stress

The created stress in the connection components of considered joint is shown in Figures 5.16 and 5.17.

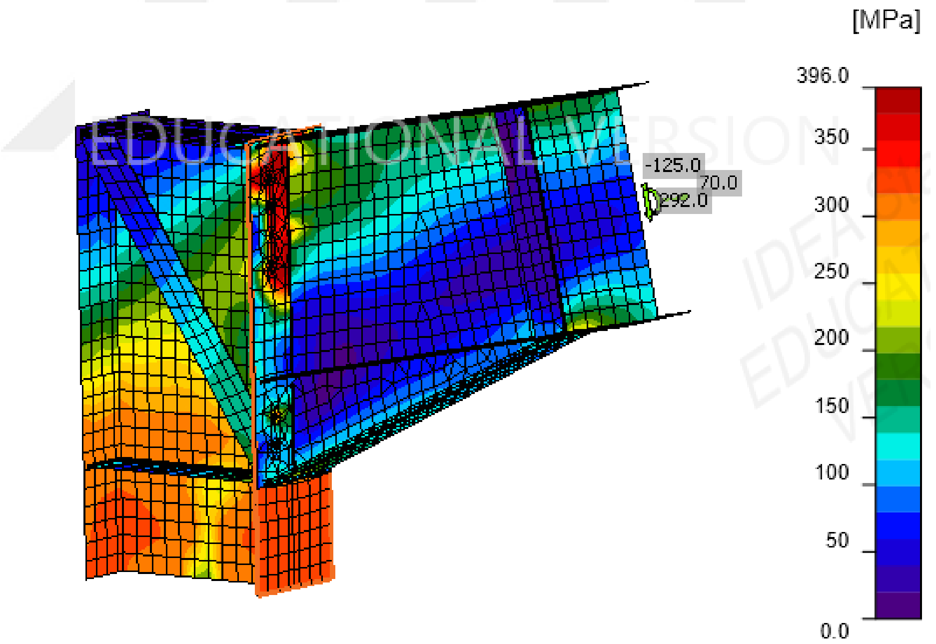
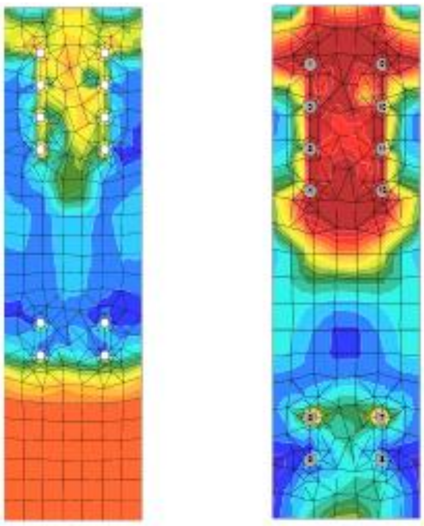


Figure 5.16: The created stress in the connection components of type (3) model of Eave joint.



(a) (b)

Figure 5.17: The distribution of created stress in the connection components of type (3) model of Eave joint; (a): Column flange which are connected to rafter; (b): End plate.

The largest created stress are 319.7 N/mm^2 and 399.7 N/mm^2 which are created in the column flange that are connected to the rafter and in the end plate respectively. It is shown in Figure 5.18.

The created stress in column flange is 319.7 N/mm^{2}is smaller than the yield stress of column steel material (F_{yc} = 350 N/mm^2).

And also, the created stress in end plate is 399.7 N/mm^{2}is smaller than the yield stress of end plate material (F_{yep} = 440 N/mm^2).

	Status	Item	Grade	Th [mm]	Loads	σ,Ed [MPa]	ε,Pl [%]
>		Member 1-tfl 1	S350GD	4.0	LE1	319.7	2.4
		Member 1-bfl 1	S350GD	4.0	LE1	318.7	1.8
		Member 1-w 1	S350GD	8.0	LE1	318.0	1.5
		Member 2-tfl 1	S350GD	4.0	LE1	282.9	0.0
		Member 2-bfl 1	S350GD	4.0	LE1	286.8	0.0

(a)

		Plate 9	S450JR	5.0	LE1	140.6	0.0
		Plate 10	S450JR	5.0	LE1	141.2	0.0
		Plate 11	S450JR	12.0	LE1	399.7	1.8
		Plate 12	S450JR	12.0	LE1	131.9	0.0
		Plate 13	S450JR	12.0	LE1	207.8	0.0
		Plate 14	S450JR	12.0	LE1	397.0	0.5

(b)

Figure 5.18: The created stress in the connection components of type (3) model of Eave joint; (a): Column flange which are connected to rafter (member 1-tfl 1); (b): End plate (plate 11).

5.2 Connection Characteristics

In traditional analysis and design of steel frame structures, the performance of column-to-beam connectionsis idealized into two groups [91]:

- Fully-restrained moment connection (rigid connection)
- Simple connection (pinned connection)

Fully-restrained moment connections are capable of transferring any created internal forces, vertical, horizontal and moment with negligible rotation between connected members [82].

Simple connections are capable of transferring any created internal forces, vertical and horizontal but not a moment. These type of connection have adequate rotation capacity to resist resulting rotation under the design loads [92].

But in reality, some of the connections are not treated as fully-restrained moment connections nor simple connections. These type of connections are capable of transferring any created internal forces, vertical, horizontal and moment but the rotation between connected members are not negligible and could accept resulting rotation [82].

These type of connections show semi-rigid performance that leads to arise a third category, the so-called Partially Restrained connections.

It is evident that increasing connections flexibility result in considerable second-order (P-Δ) in the frame and also the rotational distortion of the connections affect the displacements of the frame and cause redistribution of moments between beams and columns so determination of the actual flexibility of steel column-to-beam connections are a critical need in the analysis and design of steel structures [91].

In this way, AISC 360-10 specification has introduced its classification system. Considering the joint as a part of a structure is the idea of this classification. This means that not only the type of joint impress the structure behavior, but the type of structure (braced, or unbraced) has impress on the joint behavior as well. According to this phenomenon three types of connections are classified as below [82]:

- Fully-restrained moment connection (rigid connection)
- Partially restrained moment connection (Semi rigid connection)

- Simple connection (pinned connection)

Classifying connections are done by considering the moment–rotation (M–θ) curve characteristics of connections.

The moment-rotation curve describes the connection behavior. From M–θ curve, the main characteristics of connections can be explained, these characteristics are listed as below:

- Moment resistance (strength)
- Rotational stiffness (rigidity)
- Rotation capacity (ductility)

The moment-rotation curve of semi rigit connection is shown in Figure 5.19.

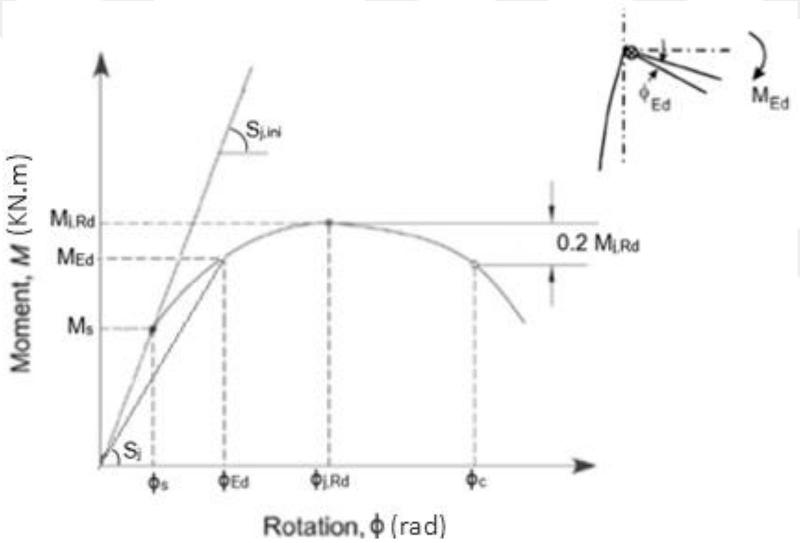


Figure 5.19: Moment-rotation characteristic for a semi rigid connection [74].

The connections behave linearly in the first stage followed by non-linear behavior and then the stiffness is decreased gradually with the increase in rotation [91].

The moment-rotation curve consists of 3 parts, the moment resistance of the connection increases linearly by increasing of connection rotation. This linear increasing continues up to 2/3 maximum moment resistance of connection ($M_{j,Rd}$). The slope of this line indicates the initial stiffness ($S_{j,ini}$) of connection, $S_{j,ini}$ is calculated by the equation 5.1.

$S_{j,ini} = M_s / \phi_s$ (Eq.5.1)

Where,

M_S is the moment at service loads which is supposed to be equal to %66 of moment resistance ($M_{j,Rd}$) of connection.

ϕs is the rotation at service loads.

The experimental test and analytical methods show that the thickness of the end-plate and the thickness of the column flange affect the connection initial rotational stiffness significantly but any increasing in the end-plate width with the same thickness do not change initial stiffness of connection [35].

From 2/3 $M_{j,Rd}$ up to $M_{j,Rd}$ the curve continues nonlinearity and after the moment resistance of connection have reached the yielding limit, the moment resistance of connection decreases by the increasing of rotation nonlinearity.

Moment resistance (strength): Strength of connection, $M_{j,Rd}$ is the maximum moment that the connection can be carried out. If the moment-rotation curve does not demonstrate a peak moment, the moment at a rotation of 0.02 rad (20mrad) is considered to be the maximum strength of connection [93].

The results of experiments show that the higher the initial stiffness of the connection the higher the moment resistance [91].

Rotational stiffness (rigidity): Stiffness of connection, S_j is the secant stiffness as it is indicated in the Figure 5.19. $M_{j,Ed}$ is the applied bending moment to a joint and ϕ_{Ed} is the corresponding rotation between the connected members. Generally, the higher the moment resistance of the connection, the stiffer the connections.

Rotation capacity (ductility): Ductility, ϕc is defined as the rotation at the point where the connection resistance has dropped to 0.8 $M_{j,Rd}$ or the deformation is more than 0.03 rad. The second principle is used for connections which there is no obvious reduction in strength capacity until a very large deformation occurs. This characteristic shows the amount of energy absorption of connection which is the critical factor for structures which are located in the seismic areas [91].

Based on AISC 360-10 specification, connection classification boundaries is defined as below:

If the ratio of connection stiffness to connected beam stiffness is more than 20, the connection is considered to be rigid (FR) and if this ratio is smaller than 2 the connection is considered to be pinned connection (simple), it means that:

To have rigid connection the equation 5.2 have to be provided.

K_s / (EI / L) ≥ 20 (Eq.5.2)

To have pinned connection the equation 5.3 have to be provided.

K_s / (EI / L) < 2 (Eq.5.3)

Where,

K_s = Initial stiffness of connection

E = Elasticity modulus

I = Moment of inertia of beam

L = Length of the beam

Any connections that the mentioned ratio is between the rigid connection and the pinned connection boundaries is categorized as semi-rigid connection (PR) [82].

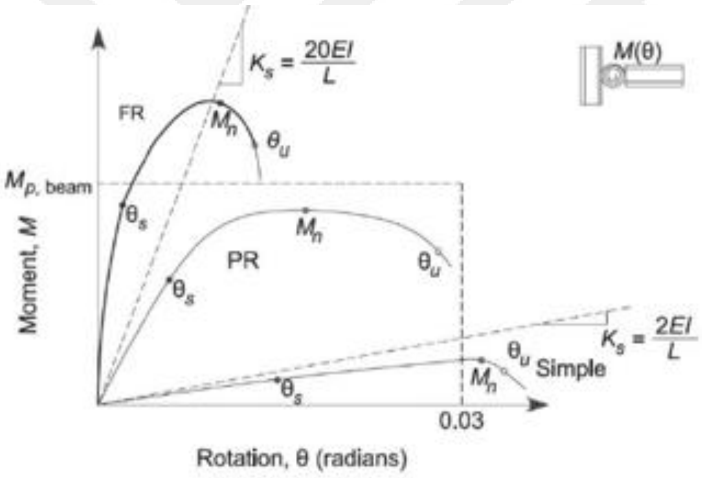


Figure 5.20: Connection classification boundaries according to AISC 360-10 specification.

The limit value for rigid and pinned connection for designed Eave joints are:

Elasticity modulus (E) = 200000

Length of the rafter (L) = 10.2 m

Rafter moment of inertia about Y axis (I) = 0.0004268 m^4

$S_{j,R}$ = 20EI / L = (20 x 20000 x 0.0004268) / 10.2 = 167.37 MNm/rad

$S_{j,p}$= 2EI / L = (2 x 20000 x 0.0004268) / 10.2 = 16.737 MNm/rad

The moment-rotation curve of joints are plotted in 5% strain limit in IDEA StatiCa program [94].

5.2.1 Type (1) model of Eave joint

The moment-rotation curve for Eave joint with full depth horizontal column stiffener is shown in Figure 5.21.

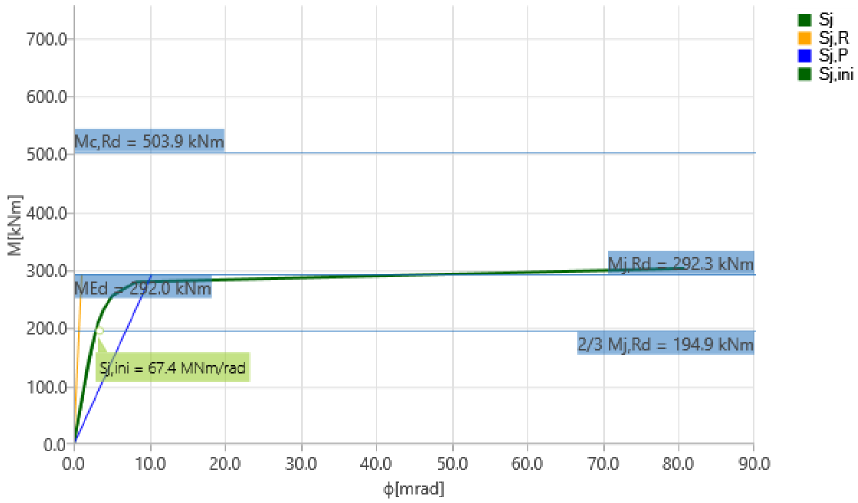


Figure 5.21: The moment-rotation curveof type (1) model of Eave joint.

As it can be seen the initial stiffness of joint ($S_{j,ini}$) is 67.4 MNm/rad

The performance of type (1) model of Eave joint is sumurrized in Table 5.1.

Table 5.1: The stiffness analyzed result of type (1) model of Eave joints.

Name	Comp.	Loads	Mj,Rd [kNm]	Sj,ini [MNm/rad]	Φc [mrad]	L [m]	Sj,R [MNm/rad]	Sj,P [MNm/rad]	Class.
Member 2	My	LE1	292.3	67.4	80.7	10.20	167.4	16.7	Semi-rigid

Name	Comp.	Loads	M [kNm]	Sjs [MNm/rad]	Φ [mrad]
Member 2	My	LE1	292.0	18.3	15.9

- The strength of connection

The maximum moment that the joint can be carried out (Mj,Rd) is 292.3 KN.m at the rotation of 20 mrad.

- The rotational stiffness of connection

The rotational stiffness of joint (S_{js}) under the applied moment (M_{Ed} = 292 KN.m) is 18.3 mrad which is between the rigid joint ($S_{j,R}$) and pinned joint ($S_{j,P}$) stiffness boundaries that this makes the joint class as semi-rigid joint.

- The ductility of connection

The ductility of considered joint (ϕc) is 80.7 mrad.

5.2.2 Type (2) model of Eave joint

The moment-rotation curve for Eave joint with supplementary web plate column stiffener is shown in Figure 5.22.

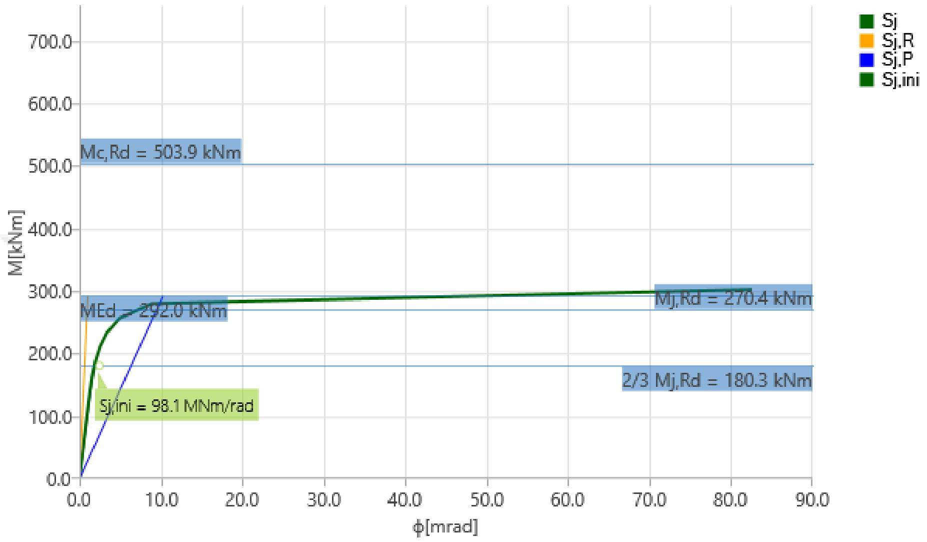


Figure 5.22: The moment-rotation curveof type (2) model of Eave joint.

As it can be seen the initial stiffness of joint ($S_{j,ini}$) is 98.1 MNm/rad

The performance of type (2) model of Eave joint is sumurrized in Table 5.2

Table 5.2: The stiffness analyzed result of type (2) model of Eave joints.

Name	Comp.	Loads	Mj,Rd [kNm]	Sj,ini [MNm/rad]	Φc [mrad]	L [m]	Sj,R [MNm/rad]	Sj,P [MNm/rad]	Class.
Member 2	My	LE1	270.4	98.1	82.6	10.20	167.4	16.7	Semi-rigid

Name	Comp.	Loads	M [kNm]	Sjs [MNm/rad]	Φ [mrad]
Member 2	My	LE1	292.0	16.6	17.6

- The strength of connection

The maximum moment that the joint can be carried out (Mj,Rd) is 270.4 KN.m at the rotation of 20 mrad.

- The rotational stiffness of connection

The rotational stiffness of joint (S_{js}) under the applied moment (M_{Ed} = 292 KN.m) is 16.6 mrad which is between the rigid joint ($S_{j,R}$) and pinned joint ($S_{j,P}$) stiffness boundaries that this makes the joint class as semi-rigid joint.

- The ductility of connection

The ductility of considered joint (ϕc) is 82.6 mrad.

5.2.3 Type (3) model of Eave joint

The moment-rotation curve for Eave joint with K diagonal column stiffener is shown in Figure 5.23.

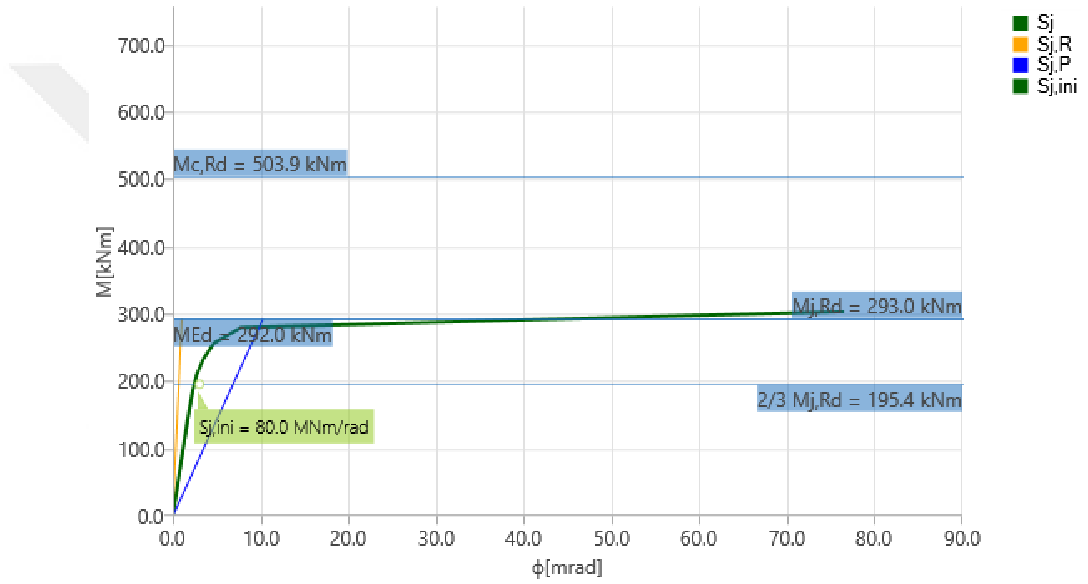


Figure 5.23: The moment-rotation curveof type (3) model of Eave joint.

As it can be seen the initial stiffness of joint ($S_{j,ini}$) is 80.0 MNm/rad.

The performance of type (3) model of Eave joint is sumurrized in Table 5.3.

Table 5.3: The stiffness analyzed result of type (2) model of Eave joints.

Name	Comp.	Loads	Mj,Rd [kNm]	Sj,ini [MNm/rad]	Φc [mrad]	L [m]	Sj,R [MNm/rad]	Sj,P [MNm/rad]	Class.
Member 2	My	LE1	293.0	80.0	76.5	10.20	167.4	16.7	Semi-rigid

Name	Comp.	Loads	M [kNm]	Sjs [MNm/rad]	Φ [mrad]
Member 2	My	LE1	292.0	19.4	15.1

- The strength of connection

The maximum moment that the joint can be carried out (Mj,Rd) is 293 KN.m at the rotation of 20 mrad.

- The rotational stiffness of connection

The rotational stiffness of joint (S_{js}) under the applied moment (M_{Ed} = 292 KN.m) is 19.4 mrad which is between the rigid joint ($S_{j,R}$) and pinned joint ($S_{j,P}$) stiffness boundaries that this makes the joint class as semi-rigid joint.

- The ductility of connection

The ductility of considered joint (ϕc) is 76.5 mrad.

6 CONCLUSIONS

The conclusions which are obtained from the comparison results of 3 modeled types of Eave joints according to their connection components characteristics and connection charecteristics are listed as below:

- The largest strain and the largest stress are created in the tension region of the end plate and also in the column flange and in the column web which are exposed to compression force in all 3 models.
- The column web in type (1) model, Eave joint with full depth horizontal column stiffener, is exposed to more stresses than the other two models so it is expected that if the column web thickness decreases, the column web in this model can not endure the created stress.
- Type (3) of model has a maximum moment resistance (strength) whereas type (2) model has a minimum moment resistance.
- Despite of the all 3 modeled Eave joints are expected to be rigid joints, they show semi rigit joint behavior so the decreased rigidity of designed connections should be taken into account in the designing of this type of structures as well as eave joints which control the overall deformation, stress distribution and ductility of the bearing system drasticallys.It is believed that this result is very significant outcome for the design purposes in practice.
- It is clear that the stiffness behavior as well as their values of the curves are close to each other therefore instead of demonstrate these curves in one diagram, it is preffered to give them in separately diagrams (Figures 5.21, 5.22, 5.23)
- Type (3) of model has a maximum rigidity (stifness) whereas type (2) model has a minimum rigidity.
- The ductility of all 3 models are almost equal but type (2) of model has a maximum deformation capacity (ductility) whereas type (3) model has a minimum deformation capacity.

- For different span lengths of cold formed portal frames, the ductilities of joints can be studied in details.
- According to the results of my study, The type 3 model, Eave joint with K diagonal stiffeners, is the most appropriate configuration for Eave joint in cold formed portal frames.

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