COMPARATIVE STUDY OF ALUMINIUM AND STEEL FRAME CONSTRUCTIONS FOR SOLAR POWER PLANTS

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COMPARATIVE STUDY OF ALUMINIUM AND STEEL FRAME CONSTRUCTIONS FOR SOLAR POWER PLANTS

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ABSTRACT

COMPARATIVE STUDY OF ALUMINIUM AND STEEL FRAME CONSTRUCTIONS FOR SOLAR POWER PLANTS

There is an increasing need for solar power plants as a solution to green energy. Structural frames made of aluminium or steel profiles are commonly used to support solar panels. Although the fact that both materials are hundred percent recyclable makes both of them preferable in terms of clean energy, there are differences between the two materials from the standpoint of sustainability. Obviously, a comparative study must be carried out in order to make an accurate and environment friendly choice when investing structural systems used in solar energy industry, which is one of the cleanest energy types of today. In this thesis, frame structures that support solar panels are designed with aluminium and steel, using related design codes to compare the sustainable properties of the two materials.

ÖZET

GÜNEŞ ENERJİSİ SANTRALLERİ İÇİN ALÜMİNYUM VE ÇELİK ÇERÇEVE KONSTRÜKSİYONLARIN KARŞILAŞTIRMALI ÇALIŞMASI

Günümüzde çevre dostu enerjiye bir çözüm olarak güneş enerjisi santrallerine artan bir ihtiyaç vardır. Alüminyum veya çelik profiller kullanılarak inşa edilen yapısal çerçeveler, güneş panellerini desteklemek için yaygın olarak kullanılır. Her iki malzemenin de yüzde yüz geri dönüştürülebilir olması, her ikisini de temiz enerji açısından tercih edilebilir kılsa da, iki malzeme arasında sürdürülebilirlik açısından farklılıklar vardır. Günümüzün en temiz enerji türlerinden biri olan güneş enerjisine yatırım yaparken doğru ve çevre dostu bir seçim yapmak için karşılaştırmalı bir çalışma yapılmasının gerekliliği ortadadır. Bu tezde, güneş panellerini destekleyen çerçeve yapıları, sürdürülebilir özelliklerini karşılaştırmak amacı ile ilgili tasarım kodları kullanılarak alüminyum ve çelik olmak üzere iki şekilde tasarlanmıştır.

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LIST OF SYMBOLS/ABBREVIATIONS

A	Cross sectional area
A_{eff}	Effective area of cross section
A_g	Area of gross cross-section
A_v	Shear area
A_{vy}	Shear area in y-y axis
A_{vz}	Shear area in z-z axis
BKS	Structure utilization class
BYS	Structural height class
$C_{p,p}$	Wind pressure coefficient
$C_{p,s}$	Wind suction coefficient
D	Overstrength related factor
DD-2	An earthquake ground motion where spectral magnitudes have a 10 percent probability of exceedance in 50 years and a corresponding recurrence period of 475 years.
DD-3	An earthquake ground motion where spectral magnitudes have a 50 percent probability of exceedance in 50 years and a corresponding recurrence period of 72 years.
DTS	Earthquake design class
E	Modulus of elasticity
E_d	Design value of effect of actions
EQ	Earthquake load
EQ_x	Earthquake load in x-x axis
EQ_y	Earthquake load in y-y axis
F_1	Local soil impact coefficient for one second period
F_S	Local soil impact coefficient for short period
G	Shear modulus
G_k	Characteristic value of a permanent action
I	Structure importance factor
I_{yy}	Moment of inertia with respect to y-y axis
I_{zz}	Moment of inertia with respect to z-z axis

I_t	Torsion constant
I_w	Warping constant
L	Member length
$M_{b,Rd}$	Design buckling resistance moment
M_{cr}	Elastic critical moment for lateral-torsional buckling
$M_{c,Rd}$	Design resistance for bending
M_{Ed}	Design bending moment
$M_{y,Ed}$	Design bending moment in y-y axis
$M_{z,Ed}$	Design bending moment in z-z axis
$N_{b,Rd}$	Design buckling resistance of the compression member
$N_{c,Ed}$	Design compression force of a member
N_{cr}	Elastic critical force for the relevant buckling mode
$N_{c,Rd}$	Design resistance to normal forces of the cross-section
N_{Ed}	Design value of the axial force
$N_{o,Rd}$	Design value of resistance to general yielding of a member
$N_{pl,Rd}$	Design plastic resistance to normal forces of the cross section
$N_{t,Ed}$	Design tension force of a member
$N_{t,Rd}$	Design value of the resistance to tension force
$Q_{k,1}$	Characteristic value of the leading variable action
$Q_{k,i}$	Characteristic value of the accompanying variable action
R	Ductility factor
$R_a(T)$	Earthquake load reduction factor
S	Snow load
S_1	Spectral acceleration coefficient for one second period
$S_{ae}(T)$	Lateral elastic design spectral acceleration
$S_{aR}(T)$	Reduced design spectral acceleration under lateral earthquake effect
S_{D1}	Design spectral acceleration coefficient for one second period
S_{DS}	Design spectral acceleration coefficient for short period
S_i	Snow load for inner purlins
S_o	Snow load for outer purlins
S_S	Spectral acceleration coefficient for short period
T_a and T_b	Corner periods of the lateral design spectrum
T_{Ed}	Design value of torsional moment

$V_{c,Rd}$	Design shear resistance
V_{Ed}	Design shear force
$V_{pl,Rd}$	Design plastic shear resistance
V_{Rd}	Design shear resistance
$V_{y,Ed}$	Design shear force in y-y axis
$V_{z,Ed}$	Design shear force in z-z axis
W	Wind loading
W_{el}	Elastic modulus of the gross section
$W_{el,min}$	Minimum elastic section modulus
$W_{el,y}$	Elastic modulus of the gross section in y-y axis
$W_{el,z}$	Elastic modulus of the gross section in z-z axis
W_p	Wind pressure load
$W_{p,i}$	Wind pressure load for inner purlins
$W_{p,o}$	Wind pressure load for outer purlins
W_s	Wind suction load
$W_{s,i}$	Wind suction load for inner purlins
$W_{s,o}$	Wind suction load for outer purlins
W_y	Section modulus
f_0	Characteristic value of 0.2 percent proof strength
f_u	Characteristic value of ultimate tensile strength
f_y	Characteristic value of yielding tensile strength
h	Height of the structure
t	Thickness
α	imperfection factor
α	shape factor
α_{LT}	imperfection factor
$\gamma_{aluminium}$	Specific weight of aluminium
γ_f	Partial factor for actions
γ_G	Partial factor for permanent actions
γ_M	General partial factor
γ_{M0}	Partial factor for resistance of cross-sections for any class

γ_{M1}	Partial factor for resistance of members to instability assessed by member checks
$\gamma_{Q,1}$	Partial factor for leading variable actions
$\gamma_{Q,i}$	Partial factor for accompanying variable actions
γ_{steel}	Specific weight of steel
$\delta_{elastic}$	Elastic displacement
$\delta_{seismic}$	Displacements caused by an earthquake
δ_{max}	Maximum displacement
κ	Factor to allow for the weakening effect
λ	The empirical coefficient used to limit the relative storey drifts
$\bar{\lambda}$	Relative slenderness
$\bar{\lambda}_0$	Limit of the horizontal plateau of the buckling curves
$\bar{\lambda}_{0,LT}$	Plateau length of the lateral torsional buckling curve
$\bar{\lambda}_{LT}$	Non-dimensional slenderness for lateral torsional buckling
ϕ	Value to determine the reduction factor χ
ϕ_{LT}	Value to determine the reduction factor χ_{LT}
χ	Reduction factor for relevant buckling mode
χ_{LT}	Reduction factor for lateral-torsional buckling
$\psi_{0,i}$	Combination factor for the accompanying load
$\psi_{0,s}$	Combination coefficient for snow load where wind is the leading variable action
$\psi_{0,w}$	Combination coefficient for wind load where snow is the leading variable action
ω_x and $\omega_{x,LT}$	Factor allowing for the location of the design section along the member

1. INTRODUCTION

1.1. SOLAR POWER PLANTS FOR GREEN ENERGY

Energy need in the world has increased exponentially in recent years, depending on population and industry. Renewable energy sources have become the most important option after it was understood that meeting this demand would not be possible with fossil fuels.

Although there are many types of renewable energy sources, solar energy, which is one of the energy sources with the highest investment and production gains, has managed to stand out compared to other alternatives. This energy source, which can be used wherever the sun rays reach, has become the symbol of green energy in terms of both residential and industrial use.

It can be said that the acquisition of energy by solar rays in completely natural ways, without any intervention in nature, is an indication that solar energy will be a pioneer in building a clean future.

1.2. STATEMENT OF THE PROBLEM

Many parameters come into play in the usage areas for the efficient use of solar energy. In addition to the chemical efficiency of solar panels; The panels should be supported by structural elements that can stand at the right angle with the sun, are self-sufficient and provide uninterrupted energy flow. The correct production and use of these structural elements is as important as the chemical adequacy of the panels.

The most common choices for these structural members today are aluminium and steel. In these structures, where the metal industry meets with the sustainable energy sector, it is necessary to ensure that the objectives of them do not conflict. The energy spent in the construction and maintenance phase of a field established for clean energy production should not overshadow the benefits provided by the green energy to be produced. For example, the release of tons of carbon dioxide during the production of a solar power plant field established to provide green energy will create a problem in the solution. So how should a decision be made between these options? Before answering this question, it is necessary to compare the two options.

Many variables can be taken into account when making comparisons. There are many known advantages and disadvantages in the aluminium and steel production of today's metal industry. According to Saevarsdottir, G. (2019), the key benefits of aluminium -other than being infinitely recyclable- are its lightweight and high strength to weight ratio with a great flexibility and formability into complex shapes, but aluminium production is also categorized as an activity at very high risk of carbon leakage [1]. Whereas, steel is also infinitely recyclable, costing less than aluminium and emitting carbon, it is about 3 times heavier; which is a factor that will affect all steps from production to transportation.

However, some of the variables must be kept constant in order to make a simple and effective comparison. Therefore, in this thesis; the answer to this question is sought: What kind of results are achieved if an aluminium solar power plant project was to be built as steel by keeping the structural variables constant?

1.3. THESIS OUTLINE

This thesis consists of three main topics: design of metal frame constructions, analysis results and comparison. Many parameters are evaluated under these main headings.

The loads to be applied on the structure are calculated using the necessary standards and how they will be modelled on the structure are shown.

Using the SAP2000 program [2], the model was defined and the main structure for the two materials was determined.

The profiles to be used for aluminium frame in this model are taken from a project that was actually built in İzmir Tire. Profiles to be used for steel were selected later according to their suitability.

The standards and equations to be used to calculate the strengths of the selected profiles are specified. The adequacy of the cross-sectional properties of structural elements for the standards used under the determined loads is shown in the analysis results.

The conformity of checks has been approved and these structures, which provide adequate conditions under the applied loads, have been compared in terms of sustainability, based on the necessary outputs from the SAP2000 program [2]. Although it is known that cost analysis can be included within the scope of sustainability work, it is not included in this thesis. The reason for this is that the costs vary according to the countries and regions, as well as the high differences according to the cross-sections of the structural elements. For this reason, cost analysis is excluded from the scope of this thesis. Based on the results of these comparisons, a sustainability assessment is conducted and recommendations for future study are given.

2. DESIGN CONSIDERATIONS FOR METAL FRAME CONSTRUCTIONS OF SOLAR POWER PLANTS

2.1. TS13891 CERTIFICATION CRITERIA

Structural design of metal constructions that are to be built on land to produce photovoltaic energy should comply with section 5 of TS13891 certification criteria [3]. According to Clause 5.1 of TS13891 [3], steel constructions with a thickness of greater than 4 millimetres, $t>4$, may be designed as per TS648-Building Code for Steel Structures [4]. However, steel constructions with a thickness of less than 4 millimetres, $t<4$, and aluminium constructions, regardless of thickness value, must be designed according to TS EN 1993-1-3 Eurocode 3: Design of steel structures [5] and TS EN 1999-1-1 Eurocode 9: Design of aluminium structures Part 1-1 [6], respectively. Snow and wind loading on the structure must be taken from TS498-Design loads for Buildings [7] since the frames are built in Turkey. As per Clause 5.1 of TS13891 [3] certification criteria, earthquake resistant design of metal constructions must be carried out according to TS EN 1998-1 Eurocode 8: Design of structures for earthquake resistance- Part 1 [8].

2.2. LOAD COMBINATIONS IN EUROCODE

Solar energy plants are important investments, and they are exposed to the climatic conditions at the installation site. Wind, snow and earthquake loads must be resisted by these structures. Metal constructions for solar panel plants can be regarded as civil engineering structures and indicative design working life category for these structures may be assumed as Category 5 (Table 2.1 of EN1990 [9]). The deformations and deflections of these structures must be limited so as not to interrupt the energy production. Hence, the strength and the deflections of the metal constructions must be checked under possible load combinations. Load combinations used for the ultimate limit state, seismic design and serviceability limit state verifications are given in following chapters.

2.2.1. Load Combinations for Ultimate Limit State

Since the soil properties of the region must be taken into consideration to avoid loss of static equilibrium and geotechnical instability, these limit states are not considered in the checks. In this thesis, only ultimate limit state of internal failure of structural members is verified under combinations of loads. Equation 6.10 of EN1990 [9] is used to determine load combinations for the effect of permanent and variable actions (Eq 2.1). Load effects are calculated considering the possibility of unfavourable deviations from the characteristic load values by applying partial load factors, γ_f, and also accounting for the possibility of the most unfavourable leading live load event and the most unfavourable accompanying live loads will occur at the same time, by applying ψ factors,

$$E_d = \gamma_G G_k + \gamma_{Q,1} Q_{k,1} + \gamma_{Q,i} \psi_{0,i} Q_{k,i} \tag{2.1}$$

where G_k is the permanent load, $Q_{k,1}$ is the leading live load, $Q_{k,i}$ is the accompanying live load and $\psi_{0,i}$ is the combination factor for the accompanying load.

Values for γ_f factors are chosen from Table A1.2(B) of EN1990 [9] and given as:

$\gamma_G = 1.35$ for unfavourable permanent loads (own weight of the structure)

$\gamma_G = 0.9$ for favourable permanent loads (in combination with wind suction)

$\gamma_Q = 1.5$ for live loads (wind and snow)

Values for ψ factors are chosen from Table A1.1 of EN1990 [9]. Combination coefficient for wind load where snow is the leading variable action is taken as, $\psi_{0,w} = 0.6$ and combination coefficient for snow load where wind is the leading variable action is taken as $\psi_{0,s} = 0.7$. Equation (2.2) gives the load combinations for permanent and live loads.

$$\begin{gathered} 1.35G + 1.5S + 0.9W_p \\ 1.35G + 1.05S + 1.5W_p \\ 0.9G + 1.5W_s \end{gathered} \tag{2.2}$$

2.2.2. Load Combinations for Seismic Design

To calculate the effects of seismic loading on the metal constructions of solar panels, load combinations are determined based on Article 6.4.3.4 of EN1990 [9] and EN1998 [8]. The partial load factors for seismic design situations are taken as γ_f =1.0 according to Appendix A of EN1990 [9]. Only snow load is assumed to act simultaneously with seismic actions. The general form of the seismic load combinations is given in Equation ((2.3):

$$E_d = G_k + EQ + \psi_{2,i} Q_{k,i} \tag{2.3}$$

where G_k is the permanent load, EQ is the earthquake load as given in EN1998 [8], $Q_{k,i}$ is the accompanying live load and $\psi_{2,i}$ is the combination factor for the accompanying load in seismic design situation. Combination coefficient for snow load during earthquake action is taken as, $\psi_{2,s}$ = 0.2 as per Table A1.1 of EN1990 [9]. As a result, the Equation (2.3) deducts to:

$$E_d = G_k + EQ + 0.2S \tag{2.4}$$

The earthquake load effect due to the combination of possible horizontal components of the calculated seismic action is calculated in accordance with Article 4.3.3.5.1 of EN1998 [8].

$$\begin{aligned} EQ_{1,2} &= \pm EQ_x + 0.3EQ_y \\ EQ_{3,4} &= \pm EQ_x - 0.3EQ_y \\ EQ_{5,6} &= \pm EQ_y + 0.3EQ_x \\ EQ_{7,8} &= \pm EQ_y - 0.3EQ_x \end{aligned} \tag{2.5}$$

where EQ_x represents the earthquake load along the chosen horizontal x axis of the structure and EQ_y represents that of the chosen horizontal y axis of the structure.

Under seismic effects deflections must be checked so as to ensure that the energy production is not interrupted in the case of an earthquake. To calculate realistic values of displacements caused by an earthquake, the elastic displacements is multiplied by the ductility factor over importance factor as specified in both EN1998 [8] and Equation (4.33) of TBSC-2018 [10].

$$\delta_{\mathrm{seismic}} = \frac{R}{I}\delta_{\mathrm{elastic}} \tag{2.6}$$

The values of ductility factor, R, and importance factor, I, are given in Chapter 2.3.5.

2.2.3. Serviceability Limit State

The deformations of the solar panel must be limited such that the integrity of the structure and production of energy are not affected. The deflection criteria are considered as specified in Clause 5.9 of TS13891 [3]. These criteria are given in order to check whether the solar panels' own structure provides resistance against incoming forces. Notice that solar panels are already supplied after verifying that they meet these criteria. Therefore, it is assumed that all panels meet the deformation criteria.

When checking serviceability limit states of the structural system, the partial factors for loads must be taken as one except if specified differently in the related standards, i.e., EN1991 to EN1998.

Table A1.4 of EN1990 [9] gives the design values of loads acting simultaneously and can be expressed as:

$$G_k + Q_{k,1} + \psi_{0,i}Q_{k,i} \tag{2.7}$$

where, $\psi_{0,i}$ factors are determined as in Equation (2.7).

Consequently, the load combinations used to perform appropriate deflection checks are given as in Equation (2.8).

$$\begin{aligned}
&G_k + S + 0.6W_p \\
&G_k + S + 0.6W_s \\
&G_k + W_p + 0.7S \\
&G_k + W_s + 0.7S
\end{aligned} \tag{2.8}$$

Maximum deflections, that are calculated using the serviceability load combinations given in Equation (2.8), must be smaller than certain ratios as follows:

For purlins, upper and lower beams' deflections must be smaller than L/360 at spans and L/180 at cantilever ends.

For shorter front and longer rear columns, deflections must be smaller than L/180.

These maximum deflections that occur under the applied load combination must be selected as relative to beam ends, because these are the deflection values that will affect the integrity of the solar panels.

2.3. DESIGN LOADS FOR SOLAR POWER PLANTS

2.3.1. Self-Weight

The self-weight of the metal frame constructions is calculated automatically by the SAP2000 program [2] and the specific weights of the materials are given in Table 2.1. In this calculation, the weight of the connection elements (bolts etc.) have been ignored for the general purpose of the research.

Table 2.1. Specific weight of materials

Specific weight (γ)	Weight	Mass
γ_{steel}	76.97 kN/m^3	7.85 kg/m^3
$\gamma_{aluminium}$	27 kN/m^3	2.75 kg/m^3

2.3.2. Weight of Solar Panels

The total weight of the 48 pieces of solar panels, which includes the panel load and the total weight of the panel fasteners, is given in Table 2.2.

Table 2.2. Total weight of solar panels

Component	Mass/piece	Total
Panel weight	19.75 kg/48	948 kg
Panel fasteners	0.3 kg/48	14.4 kg
		962.4 kg

2.3.3. Wind Loading (W)

TS13891 Certification Criteria [3] specifies the use of TS498 [7] to calculate the wind loads on the metal constructions. In many design codes wind load on any structure is calculated by multiplying the reference wind pressure of the region where the structure is built, by a factor. This factor generally represents the effects of the change in wind speed with the height of structure, surrounding terrain or topography, gust, aerodynamic shape of the structure, orientation of the surface with respect to the wind flow, dynamic properties of the structure etc. As can be understood from the previous sentence, it is usually not an easy task to determine the wind load on a given structure and approximate methods that are given in many design codes (TS498 [7], EN 1991-1-1 [11]) can be used to calculate the variation of wind loading on a structure. It is also important to note that, wind load calculation for a single solar panel construction neglects the effect of the other solar panel constructions around. Several studies in literature have emphasized the difficulty in determining the wind load effects on solar power plants.

Barata [12] studied the effects of wind loads on solar panels by using Wind Tunnel Testing techniques. A method is suggested to both developing wind tunnel tests and analysing the data to more accurately predict the wind loads on solar panels. The study considered two different arrangements of solar panels and two different mounting locations. The results of the study provided reliable information on pressure coefficient values and observed major variations in pressure coefficients for ground mounted solar plants. The longitudinal distance between the solar arrays, lateral gap between the panels, support heights, panel inclination and wind angle of attack are determined to be influencing parameters. Pressure measurements were obtained from taps mounted on the top and bottom surface of the panel at the same point. For each point a net pressure coefficient value is calculated by adding the readings from the top and bottom taps at the same point. Later a statistical analysis is performed to determine maximum, minimum and average pressure coefficients. It was found that maximum wind pressure coefficients may be as high as +7 for pressure and minimum values may be -6 for suction.

Another study by Bogdan et al. [13] stated that the code based wind load calculations neglect the group effect of solar panels and that using results obtained from wind tunnel tests would be the most valuable and safe procedure to determine wind loads on solar power plants.

It must be noted that although several studies show that wind tunnel tests are more reliable to determine wind pressure coefficients, there is no generalization of the results to be included in a design code.

Therefore, in this study, wind loads are calculated according to TS498 [7] as specified in TS13891 Certification Criteria [3]. The peak velocity pressure is taken as 1.1 kN/m^2 since the solar panel plants are located on open terrain and wind pressures may be high depending on the terrain orography.

Based on Table 6 of TS498 [7] solar panel plants are designated as open structures. The wind direction is taken as horizontal as suggested in TS498 [7]. The wind pressure and suction for inclined panel surfaces are calculated in Chapter 2.3.3.1 and 2.3.3.2. Wind loads, applied as uniformly distributed loads on purlins as shown in Figure 2.5 and Figure 2.6, are calculated in Chapter 2.4.2.

2.3.3.1. Wind Pressure (W_p)

The panels are placed on the front and rear columns at an angle of 25 degrees as shown in Figure 2.1. Wind pressure coefficient, $C_{p,p}$, must be calculated according to related formulas from Figure 1 and Table 6 of TS498 [7] as given in Equation (2.9).

$$\mathrm{C_{p,p}} = 1.2\sin(25) - 0.4$$
$$\mathrm{C_{p,p}} = 0.107 \tag{2.9}$$

2.3.3.2. Wind Suction (W_s)

The value of wind suction coefficient, $C_{p,s}$, is recommended to be taken as -0.4 according to Figure 1 of TS498 [7].

2.3.4. Snow Loading (S)

Snow load is considered to be site specific type of load and it depends on the region and the altitude that the structure is located where in Tire, İzmir and region's altitude from the sea level is approximately 200 meters. Area number of the region is considered as Area III according to the table given in Appendix 1 of TS498 [7]. Snow load for areas with area number of III and for an altitude of less than 200 meters above sea level must be taken as 0.75 kPa according to Table 4 of TS498 [7].

2.3.5. Earthquake Loading (EQ)

The earthquake load of the structure is also site specific and it is based on seismic parameters and soil properties of the region. The earthquake load must be calculated according to Turkish Building Seismic Code-2018 [10] since the frames are built in Turkey. To calculate the earthquake load, parameters of the region that the frames are built on are taken from the AFAD website.

Based on the soil investigation report of the site, local soil classification of the region is taken as ZD (layers of medium dense-dense sand, gravel or stiff clay) according to the Table 16.1 in TBSC-2018 [10].

Earthquake ground motion level is taken as DD-2 according to Clause 2.2 of TBSC-2018 [10] which corresponds to an infrequent earthquake ground motion where spectral magnitudes have a 10 percent probability of exceedance in 50 years and a corresponding recurrence period of 475 years.

The parameters of the region, taken from the report of AFAD, are given in Table 2.3.

Table 2.3. The outcomes of the AFAD report

Spectral acceleration values for DD-2	
S_S (Spectral acceleration coefficient for short period)	0.724
S_1 (Spectral acceleration coefficient for one second period)	0.194

Local soil impact coefficients for short period (F_S) and for one second period (F_1) are given in Table 2.1 and 2.2 in TBSC-2018 [10] and both are calculated using interpolation. Values of F_S and F_1 are 1.221 and 2.212, respectively.

Design spectral acceleration coefficient for short period (S_{DS}) and for one second period (S_{D1}) of the region are calculated according to Equation 2.1 of TBSC-2018 [10] and the values are 0.884 and 0.429, respectively.

Corner periods of the lateral design spectrum, T_A and T_B, equal to 0.097 secs and 0.485 secs respectively according to Equation 2.3 of TBSC-2018 [10]. The value of long-period transition period is six secs according to Clause 2.3.4.1 of TBSC-2018 [10].

The natural periods of the steel and aluminium metal frame structures, are taken from SAP2000 [2] calculations. The natural period of the steel frame structure is 0.409 secs whereas the natural period of the aluminium structure is 0.231 secs. Since both natural period values are between the values of corner periods of the lateral design spectrum, T_A and T_B, lateral elastic design spectral acceleration ($S_{ae}(T)$) of the region is calculated as design spectral acceleration coefficient for short period (S_{DS}) and equals to 0.884 according to Equation 2.2 of TBSC-2018 [10].

Structure utilization class (BKS) and structure importance factor (I) depend on the purpose of the building according to Table 3.1 in TBSC-2018 [10]. Since solar panel plants are an energy producing and distributing facility, structure utilization class (BKS) and structure importance factor (I) for the metal constructions are one and 1.5, respectively.

According to Table 3.2 and Table 3.3 in TBSC-2018 [10], Earthquake design class (DTS) of the structure is DTS=3a and structural height class (BYS) of the structure is BYS=8.

According to Table 4.1 in TBSC-2018 [10] ductility related factor (R) and Overstrength related factor (D) are equal to R=4 and D=2.5 since the structure's framing system is "C31. Structures in which all of the earthquake effects are met with moment-transferring steel frames of limited ductility level".

According to Equation 4.1b of TBSC-2018 [10], earthquake load reduction factor ($R_a(T)$) equals to 2.641 and 2.58 for aluminium and steel frame structures, respectively.

Reduced design spectral acceleration under lateral earthquake effect ($S_{aR}(T)$) is calculated according to Equation 4.8 of TBSC-2018 [10] and equals to 0.1844 and 0.1848 for aluminium and steel frame structures respectively.

Earthquake load patterns of the aluminium and steel structures are defined according to the parameters given in Chapter 2.3.5 by "TSC-2018 Auto-Lateral load pattern" module in SAP2000 program [2] in both X and Y directions according to corresponding periods.

2.4. STRUCTURAL MODELLING FOR ANALYSIS

The structural model of the metal frames is taken from an actual project prepared by manufacturer. In the design of solar power plant, the solar panels sit on purlins as shown in Figure 2.2 and the spacing of the purlins is determined by the solar panel provider in order to prevent deformations of panels.

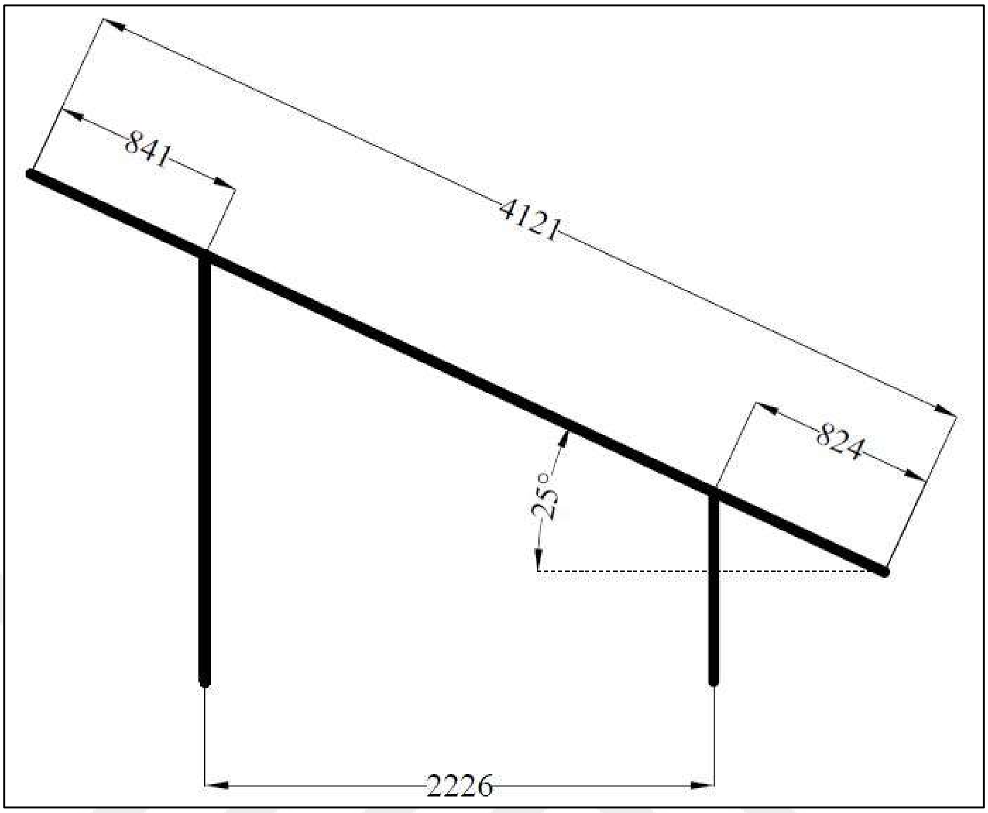

Figure 2.1. Structural detail in X-Z plane

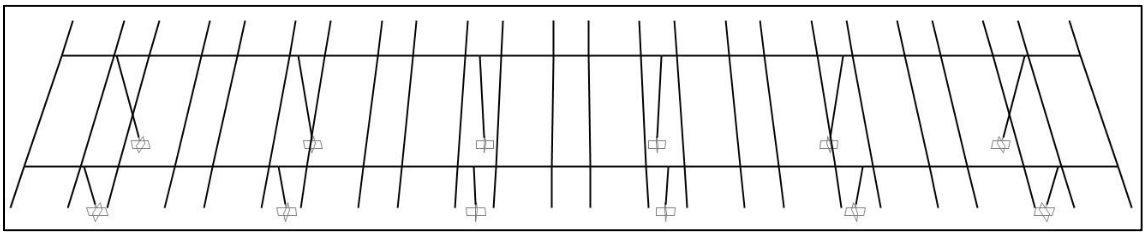

Figure 2.2. Structural detail: 3D view

2.4.1. Joint Modelling

The type of joints to be modelled is one of the issues to be carefully determined in order to analyse a structure correctly. Joints can be modelled in different ways, depending on whether they are constrained against horizontal and vertical movement and rotation in general.

In this thesis, joints of structures are designed as pin type joints. This approach is considered for purlins to transfer only loads to beams without transferring any moment. Therefore, these are joints where movements are restricted in horizontal and vertical direction but free in rotation.

2.4.2. Application of Loads

The total area of 48 pieces of solar panels is given as 79.2 m^2 by the manufacturer. Since the total weight is applied as a uniform load on the purlins as shown in Figure 2.4, the mass and the weight per area (m^2) of the solar panels can be calculated as:

$$962.4\ \mathrm{kg}/79.2\ \mathrm{m}^2 = 12.15\ \mathrm{kg}/\mathrm{m}^2 = 0.120\ \mathrm{kN}/\mathrm{m}^2 \tag{2.10}$$

In order to determine the tributary width of the panel area which is applied on purlins, a continuous beam with 23 spans is modelled in SAP2000. A uniform load of 1 kN/m is applied on the beam as shown in Figure 2.3.

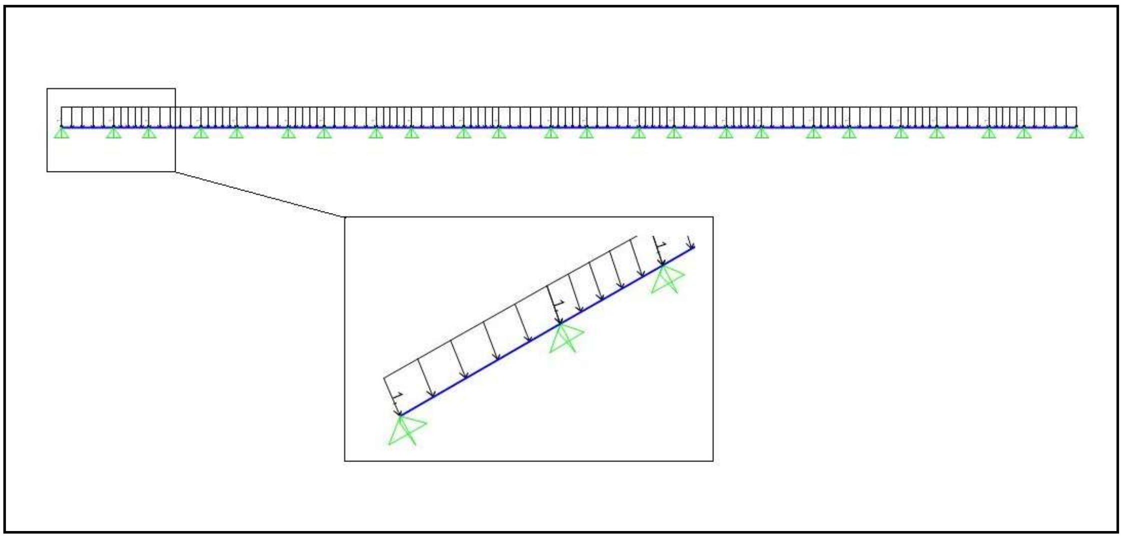

Figure 2.3. Uniform load of 1 kN/m applied on a continuous beam with 23 spans

From the analysis results of this model, tributary width of the panel area is determined from the reactions of the joints. Given in Table 2.4, tributary width of the panel area are 0.5 meters for the outer purlins and 0.84 for the inner purlins.

Table 2.4. Joint reactions of the model to determine tributary width of the panel area

Joint	**OutputCase**	**CaseType**	**F1**	**F2**	**F3**	**M1**	**M2**	**M3**
Text	Text	Text	kN	kN	kN	kN-m	kN-m	kN-m
1	DEAD	LinStatic	0	0	0.5	0	0	0
2	DEAD	LinStatic	0	0	0.84	0	0	0
3	DEAD	LinStatic	0	0	0.84	0	0	0
4	DEAD	LinStatic	0	0	0.84	0	0	0
5	DEAD	LinStatic	0	0	0.84	0	0	0
6	DEAD	LinStatic	0	0	0.84	0	0	0
7	DEAD	LinStatic	0	0	0.84	0	0	0
8	DEAD	LinStatic	0	0	0.84	0	0	0
9	DEAD	LinStatic	0	0	0.84	0	0	0
10	DEAD	LinStatic	0	0	0.84	0	0	0
11	DEAD	LinStatic	0	0	0.84	0	0	0
12	DEAD	LinStatic	0	0	0.84	0	0	0
13	DEAD	LinStatic	0	0	0.84	0	0	0
14	DEAD	LinStatic	0	0	0.84	0	0	0
15	DEAD	LinStatic	0	0	0.84	0	0	0
16	DEAD	LinStatic	0	0	0.84	0	0	0
17	DEAD	LinStatic	0	0	0.84	0	0	0
18	DEAD	LinStatic	0	0	0.84	0	0	0
19	DEAD	LinStatic	0	0	0.84	0	0	0
20	DEAD	LinStatic	0	0	0.84	0	0	0
21	DEAD	LinStatic	0	0	0.84	0	0	0
22	DEAD	LinStatic	0	0	0.84	0	0	0
23	DEAD	LinStatic	0	0	0.84	0	0	0
24	DEAD	LinStatic	0	0	0.5	0	0	0

The weight of solar panels, wind pressure, wind suction and snow loads are applied on purlins as uniformly distributed load. To determine the magnitude of these loads, tributary width of the panel area is multiplied with the load itself, and the uniformly distributed load which is applied on purlins is calculated.

The value of the weight of the solar panel load, which is applied on inner and outer purlins, is calculated according to Table 2.4 and Chapter 2.3.2.

For outer purlins, the weight of solar panel load equals to:

$$0.5\text{m x } 0.120\,\text{kN/m}^2 = 0.06\,\text{kN/m} \tag{2.11}$$

For inner purlin, the weight of solar panel load equals to:

$$0.84\text{ m x } 0.120\;\text{kN/m}^2 = 0.1008\,\text{kN/m} \tag{2.12}$$

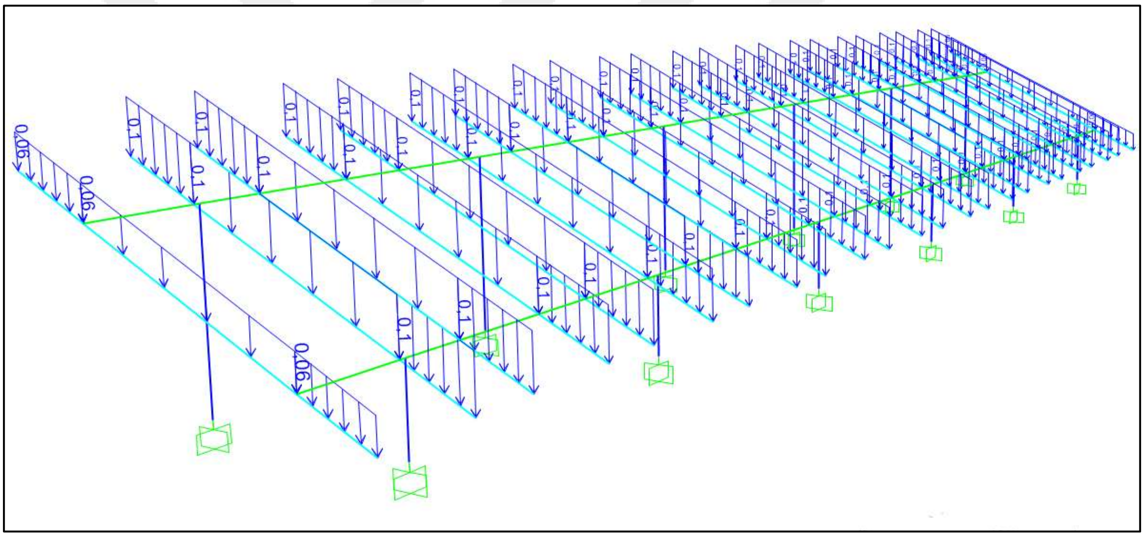

Figure 2.4. Weight of the solar panels applied on purlins

Wind pressure and wind suction loads on purlins are applied in local axes, as the wind is acting perpendicular to the solar panels, as shown in Figure 2.5 and Figure 2.6.

The value of the wind pressure load (W_p) which is applied on inner and outer purlins is calculated according to Table 2.4 and Chapter 2.3.3.1 as:

$$W_p = 0.107\text{x}1.1\,\text{kN/m}^2 = 0.118\,\text{kN/m}^2 \tag{2.13}$$

For outer purlins, wind pressure load equals to:

$$W_{p,o} = 0.5x0.118 = 0.06 \ kN/m \tag{2.14}$$

For inner purlins, wind pressure load equals to:

$$W_{p,i} = 0.84x0.118 = 0.1 \ kN/m \tag{2.15}$$

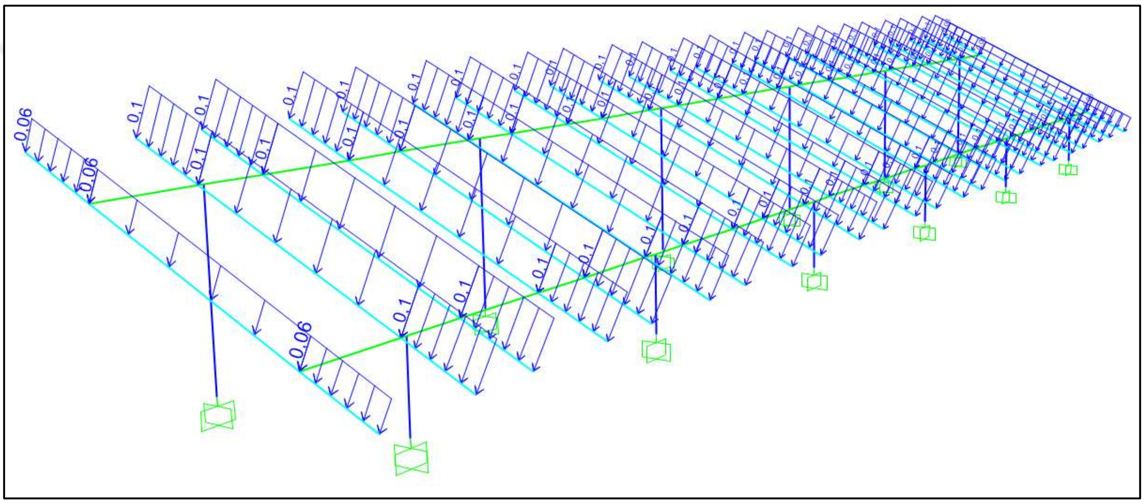

Figure 2.5. Wind pressure load applied on purlins

The value of the wind suction load, which is applied on inner and outer purlins, is calculated according to Table 2.4 and Chapter 2.3.3.2 as:

$$W_s = (-0.4)x1.1 \, kN/m^2 = -0.44 \, kN/m^2 \tag{2.16}$$

For outer purlins, wind suction load equals to:

$$W_{s,o} = 0.5x(-0.44) = -0.22 \, kN/m \tag{2.17}$$

For inner purlins, wind suction load equals to:

$$W_{s,i} = 0.84x(-0.44) = -0.37\,\mathrm{kN/m} \tag{2.18}$$

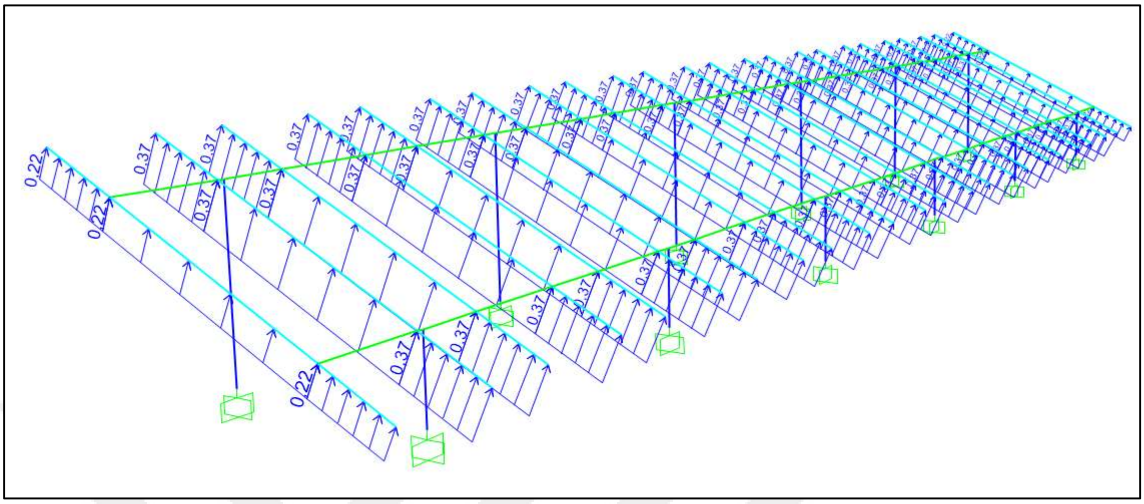

Figure 2.6. Wind suction load applied on purlins

The value of the snow load, which is applied on inner and outer purlins, is calculated according to Table 2.4 and Chapter 2.3.4.

For outer purlins, snow load equals to:

$$S_o = 0.5x0.75 = 0.38\,\mathrm{kN/m} \tag{2.19}$$

For inner purlins, snow load equals to:

$$S_i = 0.84x0.75 = 0.63\,\mathrm{kN/m} \tag{2.20}$$

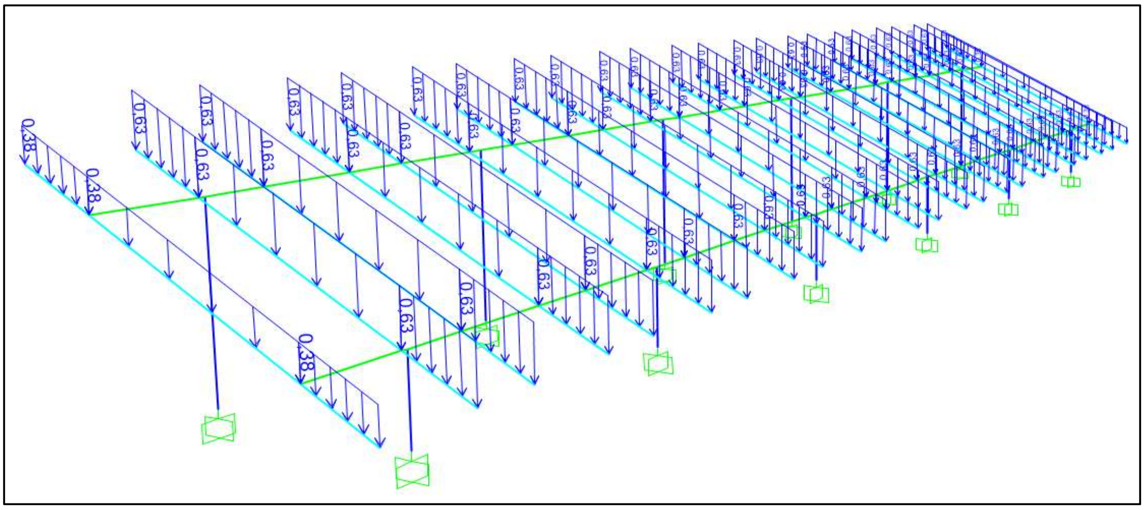

Figure 2.7. Snow load applied on purlins

2.4.3. Ground Structure Interaction

In case of modelling, the type of supports is determined according to the ground conditions. In line with the information obtained from the ground survey, the support that provides the most suitable condition for the soil is modelled. In this thesis, since the comparison of structures of two different materials is taken into consideration, fixed support was chosen because it includes a fictitious modelling. This choice could also be considered as pin support depending on the characteristics of the soil, but for the purpose of this thesis, the interaction of the structures with the ground is modelled with fixed support.

2.4.4. First Order Analysis

First order analysis is made considering the given initial dimensions and the loading of the structure. Second order analysis, on the other hand, is made by considering the loads affecting the structure and the deformations. The analysis type performed in this thesis is first order analysis.

3. DESIGN OF STEEL FRAME CONSTRUCTIONS ACCORDING TO EN1993-1-3

Steel material of the structure has EN-10210-1 standard and steel grade of S 235 H. Design values of material constants are taken from Article 3.2.6 of EN1993-1-3 [5]. Further material data is shown in Table 3.1.

Table 3.1. Steel material data

Steel grade	**Nominal thickness**	**f_y(MPa)**	**f_u(MPa)**
S 235 H	t ≤40mm	235.0	360.0

In an ultimate state method partial factor for the material resistance (γ_M) takes into account the scattering of the strength properties and the geometry of the cross-section. Partial safety factors for ultimate limit states', γ_{M0} and γ_{M1}, numerical values are recommended to be taken as one according to Article 6.1 NOTE 2B of EN1993-1-3 [5]. Member classification is Class 3 for all members.

Connections on all members are not checked by the program. The members ensure sufficient conditions throughout their length, and since there are no additions in the members, the net cross-section area is equal to the gross cross-section area.

Since a structure made of aluminium material is re-modelled using steel material, it is necessary to create the most suitable structure by trying different options in new steel modelling in order to be a fair comparison. That's why the design of the steel structure was carried out in two options.

The first of these options is the same with the system with 12 columns, two beams and 24 purlins formed in an aluminium frame. In the second option, the number of columns was reduced to 10 as five long and five short columns. Considering the deflection limits of the solar panels, the number of beams and purlins is kept the same as in Steel Frame Option-1.

Steel Frame Option-2's design checks were carried out under the same conditions as Steel Frame Option-1, which are given in Chapter 3. The design results of the Steel Frame Option-2 are given in Chapter 5.3.

3.1. DESIGN CHECKS OF BEAMS AND COLUMNS

Members are subjected to sectional checks which are tension, compression, bending, shear and combined bending, axial force and shear force as well as stability checks which are flexural, torsional and lateral torsional buckling.

3.1.1. Tension Check of Members

Tension check of members must be performed according to Equation (6.5) given in Article 6.2.3 of EN1993-1-3 [3].

$$\frac{N_{Ed}}{N_{t,Rd}} \leq 1.0 \tag{3.1}$$

The design tension resistance of gross cross-sections must be calculated according to Equation (6.6) given in Article 6.2.3 of EN1993-1-3 [3].

$$N_{pl,Rd} = Af_y/\gamma_{M0} \tag{3.2}$$

3.1.2. Compression Check of Members

Compression check of members must be performed according to Equation (6.9) given in Article 6.2.4 of EN1993-1-3 [3].

$$\frac{N_{Ed}}{N_{c,Rd}} \leq 1.0 \tag{3.3}$$

The design compression resistance of cross-sections with uniform compression must be calculated according to Equation (6.10) given in Article 6.2.4 of EN1993-1-3 [3].

$$N_{c,Rd} = Af_y/\gamma_{M0} \tag{3.4}$$

3.1.3. Bending Check of Members

Bending check of members must be performed according to Equation (6.12) given in article 6.2.5 of EN1993-1-3 [3].

$$\frac{M_{Ed}}{M_{c,Rd}} \leq 1.0 \tag{3.5}$$

Bending resistance of metal frame sections must be calculated according to Equation (6.14) given in Article 6.2.5 of EN1993-1-3 [3] considering all members are assumed to be Class 3.

$$M_{c,Rd} = M_{el,Rd} = W_{el,min}f_y/\gamma_{M0} \tag{3.6}$$

Bending check of members must be performed in both axes of the member.

3.1.4. Shear Check of Members

Shear check of members must be performed according to Equation (6.17) given in Article 6.2.6 of EN1993-1-3 [3].

$$\frac{V_{Ed}}{V_{c,Rd}} \leq 1.0 \tag{3.7}$$

The design shear resistance of sections, V_{Rd}, must be calculated according to Equation (6.18) given in Article 6.2.6 of EN1993-1-3 [3].

$$V_{pl,Rd} = \frac{A_v(f_y/\sqrt{3})}{\gamma_{M0}} \tag{3.8}$$

where A_v is the shear area and the value of it is obtained from the SAP2000 program.

In this thesis all steel members satisfy the non-slenderness criteria and hence, the design shear resistance is calculated as given in Equation (3.8).

Shear check of members must be performed in both axes of the member.

3.1.5. Combined Bending, Axial Force and Shear Force Check of Members

Combined bending, axial force and shear force check of members must be performed according to Article 6.2.10 of EN1993-1-3 [3], where indicated as: "… the design value of shear force, V_{Ed}, does not exceed 50 percent of the design plastic shear resistance, $V_{pl,Rd}$, no reduction of the resistances need be made." Since there aren't any members exceeding the 50 percent design plastic shear resistance, no other calculation need to be made.

3.1.6. Flexural Buckling and Torsional Buckling Check of Members

Flexural and torsional buckling check of members must be performed according to Equation (6.46) given in Article 6.3.1.1 of EN1993-1-3 [3].

$$\frac{N_{Ed}}{N_{b,Rd}} \leq 1.0 \tag{3.9}$$

The design buckling resistance of a Class 3 member must be calculated according to Equation (6.47) given in Article 6.3.1.1(3) of EN1993-1-3 [3].

$$N_{b,Rd} = \frac{\chi A f_y}{\gamma_{M1}} \tag{3.10}$$

The reduction factor, χ, can be calculated according to Equation (6.49) given in Article 6.3.1.2(1) of EN1993-1-3 [3].

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}^2}} \tag{3.11}$$

Where ϕ equals to:

$$\phi = 0.5\left(1 + \alpha\left(\bar{\lambda} - 0.2\right) + \bar{\lambda}^2\right) \tag{3.12}$$

The value of imperfection factor, α, must be obtained from Table 6.1 and Table 6.2 of EN1993-1-3 [3] and equals to 0.49 for cold-formed hollow sections of S 235 steel members.

For Class 3 cross-sections, $\bar{\lambda}$ equals to:

$$\bar{\lambda} = \sqrt{\frac{A f_y}{N_{cr}}} \tag{3.13}$$

Where the value of N_{cr}, elastic critical force, for each member can be obtained directly from the from the SAP2000 program's steel design check module in accordance with EN1993-1-3.

3.1.7. Lateral Torsional Buckling Check of Members

Lateral torsional buckling check of members must be performed according to Equation (6.54) given in article 6.3.2.1 of EN1993-1-3 [3].

$$\frac{M_{Ed}}{M_{b,Rd}} \leq 1.0 \tag{3.14}$$

The design buckling resistance moment of a member must be calculated according to Equation (6.55) given in Article 6.3.2.1(3) of EN1993-1-3 [3].

$$M_{b,Rd} = \chi_{LT} W_y \frac{f_y}{\gamma_{M1}} \tag{3.15}$$

Where W_y is the section modulus and equals to $W_{el,y}$ for Class 3 cross-sections.

The reduction factor for lateral torsional buckling, χ_{LT} , must be smaller or equal to one and calculated according to Equation (6.56) given in Article 6.3.2.2(1) of EN1993-1-3 [3].

$$\chi_{LT} = \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \bar{\lambda}_{LT}^2}} \tag{3.16}$$

Where ϕ_{LT} equals to:

$$\phi_{LT} = 0.5\left(1 + \alpha_{LT}\left(\bar{\lambda}_{LT} - 0.2\right) + \bar{\lambda}_{LT}^2\right) \tag{3.17}$$

The value of imperfection factor, α_{LT}, must be obtained from Table 6.3 and Table 6.4 of EN1993-1-3 [3] and equals to 0.76 for other cross-sections of steel members.

Where $\bar{\lambda}_{LT}$ equals to:

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}} \tag{3.18}$$

Where the value of M_{cr}, elastic critical moment, for each member can be obtained directly from the SAP2000 program's steel design check module in accordance with EN1993-1-3.

3.2. SEISMIC DESIGN CHECK OF THE STRUCTURE

Seismic design check of the steel structure must be performed according to Equation (4.34b) of TBSC-2018 [3]. Following equation applies for the steel structure in this thesis, since there are no filling material and the metal frame structure are independent.

$$\lambda \frac{\delta_{\text{max}}}{\text{h}} \leq 0.016\kappa \tag{3.19}$$

Where the value of maximum deflection, δ_{max}, is calculated according to Equation (2.6) for maximum deflection of the load patterns in both X and Y directions, EQ_x and EQ_y.

λ is the ratio of the values of lateral elastic design spectral acceleration ($S_{ae}(T)$) for ground motion level DD-3 and ground motion level DD-2, which was calculated before in Chapter 2.3.5, for the same location according to Clause 4.9.1.4 of TBSC-2018 [3]. Lateral elastic design spectral acceleration of the structure for ground motion level DD-3 equals to 0.472. Therefore, the value of λ equals to 0.534.

Height of the structure, h, is taken as the long rear column's height which equals to 1.865 meter.

The value of the coefficient, κ, equals to 0.5 for steel structures according to Clause 4.9.1.4 of TBSC-2018 [3].

4. DESIGN OF ALUMINIUM FRAME CONSTRUCTIONS ACCORDING TO EN1999-1-1

Aluminium material of the structure has EN-AW6063-T66 alloy designation which chemical symbol of the material is EN AW-Al Mg0,7Si. EN-AW6063 has durability rating of B. For T66 temper of the EN-AW6063 with less than 10 millimetres thickness, the buckling class is A and design values of material constants are taken from Article 3.2.5 of EN1999-1-1 [6] and further material data are shown in Table 4.1.

Table 4.1. Aluminium material data

Alloy EN-AW	**Temper**	**f_0(MPa)**	**f_u(MPa)**	**Buckling Class**
6063	T66	200.0	245.0	A

In an ultimate state method partial factor for the material resistance (γ_M) takes into account the scattering of the strength properties and the geometry of the cross-section. Partial safety factors for ultimate limit state's, γ_{M1} for resistance of cross-sections in any class, numerical value are recommended to be taken as 1.10 according to Article 6.1.3 NOTE 1 of EN1999-1-1 [6]. Member classification is Class 3 for all members.

Connections on all members are not checked by the program. The members ensure sufficient conditions throughout their length, and since there are no additions in the members, the net cross-section area is equal to the gross cross-section area.

Design checks of aluminium members has been discussed under two main headings as beams and columns due to structural and modelling differences between members.

4.1. DESIGN CHECK OF BEAMS AND COLUMNS

Members are subjected to sectional checks which are tension, compression, bending, shear and combined bending, axial force and shear force as well as stability checks which are flexural, torsional and lateral torsional buckling.

4.1.1. Tension Check of Members

Tension check of members must be performed according to Equation (6.17) given in Article 6.2.3 of EN1999-1-1 [6].

$$\frac{N_{Ed}}{N_{t,Rd}} \leq 1.0 \tag{4.1}$$

All frame members are assumed to have neither unfilled holes nor transverse weld. Therefore, the design tension resistance of such sections must be calculated according to Equation (6.18) given in Article 6.2.3 of EN1999-1-1 [6].

$$N_{o,Rd} = A_g f_o / \gamma_{M1} \tag{4.2}$$

4.1.2. Compression Check of Members

Compression check of members must be performed according to Equation (6.20) given in Article 6.2.4 of EN1999-1-1 [6].

$$\frac{N_{Ed}}{N_{c,d}} \leq 1.0 \tag{4.3}$$

All frame members are assumed to have neither unfilled holes nor transverse weld. Therefore, the design compression resistance of such sections must be calculated according to Equation (6.22) given in Article 6.2.4 of EN1999-1-1 [6].

$$N_{o,Rd} = A_{eff} f_0 / \gamma_{M1} \tag{4.4}$$

4.1.3. Bending Check of Members

Bending check of members must be performed according to Equation (6.23) given in Article 6.2.5 of EN1999-1-1 [6].

$$\frac{M_{Ed}}{M_{Rd}} \leq 1.0 \tag{4.5}$$

Bending resistance of members must be calculated according to Equation (6.25) given in Article 6.2.5 of EN1999-1-1 [6] considering all members are assumed to have neither holes nor transverse weld.

$$M_{o,Rd} = \alpha W_{el} f_o / \gamma_{M1} \tag{4.6}$$

Where the value of shape factor, α, alternatively be taken as one according to 6.2.5.1(2) and Table 6.4 of EN1999-1-1 [6].

Bending check of members must be performed in both axes of the member.

4.1.4. Shear Check of Members

Shear check of members must be performed according to Equation (6.28) given in Article 6.2.6 of EN1999-1-1 [6].

$$\frac{V_{Ed}}{V_{Rd}} \leq 1.0 \tag{4.7}$$

The design shear resistance of sections, V_{Rd}, for non-slender sections must be calculated according to Equation (6.29) given in Article 6.2.6 of EN1999-1-1 [6],

$$V_{Rd} = A_v \frac{f_o}{\sqrt{3}\gamma_{M1}} \tag{4.8}$$

where A_v is the shear area and the value of it is obtained from the SAP2000 program.

In this thesis all aluminium members satisfy the non-slenderness criteria and hence, the design shear resistance is calculated as given in Equation (4.8).

Shear check of beam members must be performed in both axes of the member.

4.1.5. Combined Bending, Axial Force and Shear Force Check of Members

Combined bending, axial force and shear force check of members must be performed according to Article 6.2.10 of EN1999-1-1 [6], where indicated as: "… the design value of shear force, V_{Ed}, does not exceed 50 percent of the design plastic shear resistance, V_{Rd}, no reduction of the resistances defined for bending and axial force need be made."

Since there aren't any members with exceedance of the 50 percent design shear resistance, no other calculation need to be made.

4.1.6. Flexural Buckling and Torsional Buckling Check of Members

Flexural and torsional buckling check of members must be performed according to Equation (6.48) given in Article 6.3.1.1 of EN1999-1-1 [6].

$$\frac{N_{Ed}}{N_{b,Rd}} \leq 1.0 \tag{4.9}$$

All frame members are assumed to have neither transverse welds nor heat affected zone. Therefore, the flexural buckling check of such sections must be calculated according to Equation (6.49a) given in Article 6.3.1.1 of EN1999-1-1 [6].

$$N_{b,Rd} = k\chi\omega_x A_{eff} f_0/\gamma_{M1} \tag{4.10}$$

The factor to allow for the weakening effect, k, equals to one since there is no weld in the section and since there are only axial forces acting on sections, the factor allowing for the location of the design section along the member, ω_x, equals to one according to Article 6.3.1.1(2) of EN1999-1-1 [6].

The reduction factor, χ, can be calculated according to Equation (6.50) given in Article 6.3.1.2 of EN1999-1-1 [6].

$$\chi = \frac{1}{\phi + \sqrt{(\phi^2 - \bar{\lambda}_0^{\,2}}} \tag{4.11}$$

Where ϕ equals to:

$$\phi = 0.5(1 + \alpha(\bar{\lambda} - \bar{\lambda}_0) + \bar{\lambda}^2 \tag{4.12}$$

The value of imperfection factor, α, must be obtained from Table 6.6 of EN1999-1-1 [6] and equals to 0.20 for Class A members.

Where $\bar{\lambda}$ equals to:

$$\bar{\lambda} = \sqrt{\frac{A_{eff} f_0}{N_{cr}}} \tag{4.13}$$

For the beams and purlins of the aluminium frame, the compression force is such that flexural and torsional buckling effects may be ignored according to Article 6.3.1.2(4) of EN1999-1-1 [6] where indicated as:

“For relative slenderness the buckling effects may be ignored and only cross-sectional check apply.”

$$\bar{\lambda} \leq \bar{\lambda}_0 \text{ or for } N_{Ed} \leq \bar{\lambda}_0^2 N_{cr} \tag{4.14}$$

The value of the limit of the horizontal plateau, $\bar{\lambda}_0$, is taken as 0.10 according to Table 6.6 of EN1999-1-1 [6].

N_{cr} is the critical axial force and can be calculated according to Equation (I.14) and (I.15) given in Annex I Article I.3 of EN1999-1-1 [6].

$$N_{cr} = \frac{\pi^2 EI}{k^2 L^2} \tag{4.15}$$

The value of buckling length factor, k, is taken as one according to Article 6.3.1.3(2) Table 6.8 of EN1999-1-1 [6].

Notice that the purlins are partially stiffened from many points on the member which are the auxiliary elements that fix solar panels to purlins. Therefore, the flexural and torsional buckling is not allowed for purlins and its check can be neglected due to these conditions.

4.1.7. Lateral Torsional Buckling Check of Members

Lateral torsional buckling check of members must be performed according to Equation (6.54) given in Article 6.3.2.1 of EN1999-1-1 [6].

$$\frac{M_{Ed}}{M_{b,Rd}} \leq 1.0 \tag{4.16}$$

The design buckling resistance moment of a member must be calculated according to Equation (6.55a) given in Article 6.3.2.1(3) of EN1999-1-1 [6].

$$M_{b,Rd} = \chi_{LT}\omega_{xLT}\alpha W_{el,y}\frac{f_0}{\gamma_{M1}} \tag{4.17}$$

Where α equals to one as indicated in Chapter 4.1.3.

Since there are only axial forces acting on sections, the factor allowing for the location of the design section along the member, ω_x, equals to one according to Article 6.3.2.1(2) of EN1999-1-1 [6].

The reduction factor for lateral torsional buckling, χ_{LT} , must be calculated according to Equation (6.56) given in Article 6.3.2.2(1) of EN1999-1-1 [6].

$$\chi_{LT} = \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \bar{\lambda}_{LT}^2}} \tag{4.18}$$

Where ϕ_{LT} equals to:

$$\phi_{LT} = 0.5\left(1 + \alpha_{LT}\left(\bar{\lambda}_{LT} - \bar{\lambda}_{0,LT}\right) + \bar{\lambda}_{LT}^2\right) \tag{4.19}$$

The value of imperfection factor, α_{LT}, and the limit of the horizontal plateau, $\bar{\lambda}_{0,LT}$, must be taken as 0.20 and 0.40 respectively for Class 3 sections according to Article 6.3.3.2(2) of EN1999-1-1 [6].

Where $\bar{\lambda}_{LT}$ equals to:

$$\bar{\lambda}_{LT} = \sqrt{\frac{\alpha W_{el,y} f_0}{M_{cr}}} \tag{4.20}$$

Where α equals to 0.2 according to Table 6.4 of EN1999-1-1 [6].

For the beams and purlins of the aluminium frame, the moment is such that lateral torsional buckling effects may be ignored according to Article 6.3.2.2(4) of EN1999-1-1 [6] where indicated as:

"For slenderness the buckling effects may be ignored and only cross-sectional check apply."

$$\bar{\lambda}_{LT} \leq \bar{\lambda}_{0,LT} \text{ or for } M_{Ed} \leq \bar{\lambda}_{0,LT}^2 M_{cr} \tag{4.21}$$

The value of the imperfection factor, α_{LT}, and the limit of the horizontal plateau, $\bar{\lambda}_{0,LT}$, is taken as 0.20 and 0.40 for Class 3 sections respectively according to Article 6.3.2.2(2) of EN1999-1-1 [6].

M_{cr} is the elastic critical moment and can be calculated according to Equation (I.1) given in Annex I Article I.1.1 of EN1999-1-1 [6].

$$M_{cr} = \frac{\pi\sqrt{EI_z GI_t}}{L}\sqrt{1 + \frac{\pi^2 EI_w}{L^2 GI_t}} \tag{4.22}$$

Where:

I_w is the warping constant and conservatively be taken as zero according to Article J.3(1) of Annex J of EN1999-1-1 [6].

I_t is the torsional constant and can be obtained from the SAP2000 program [2].

G is the shear modulus and equals to 27000 MPa according to Article 3.2.5 of EN1999-1-1 [6].

Notice that the purlins are partially stiffened from many points on the member, which are the auxiliary elements that fix solar panels to purlins. Therefore, the lateral torsional buckling is not allowed for purlins and its check can be neglected due to these conditions according to Article 6.3.2.1 NOTE b) of EN1999-1-1 [6].

4.2. SEISMIC DESIGN CHECK OF THE STRUCTURE

Seismic design check of the aluminium structure must be performed as indicated in Chapter 3.2, similar to steel structures. The value of the coefficient, κ, is taken also as 0.5 for aluminium structures because there is no other specification for aluminium structures in TBSC-2018 [10].

5. RESULTS OF ANALYSIS

In this thesis, SAP2000 program results are checked for four types of members. These members are: upper main beam, lower main beam, purlins and columns. The sectional and stability checks of these members are carried out with the relevant equations given in Chapter 3 and Chapter 4, considering the material data and maximum internal forces of the members.

5.1. ALUMINIUM FRAME DESIGN

All aluminium members are created in SAP2000 program. Column members are modelled as tube sections with a dimensions of 45 millimetres in width, 78 millimetres in height and 2.3 millimetres in thickness. In order to model the sections of the beam members (upper and lower main beams and purlins) given by manufacturer, section designer module in the SAP2000 program is used. Section designer details of the beam members are shown in Figure 5.1.

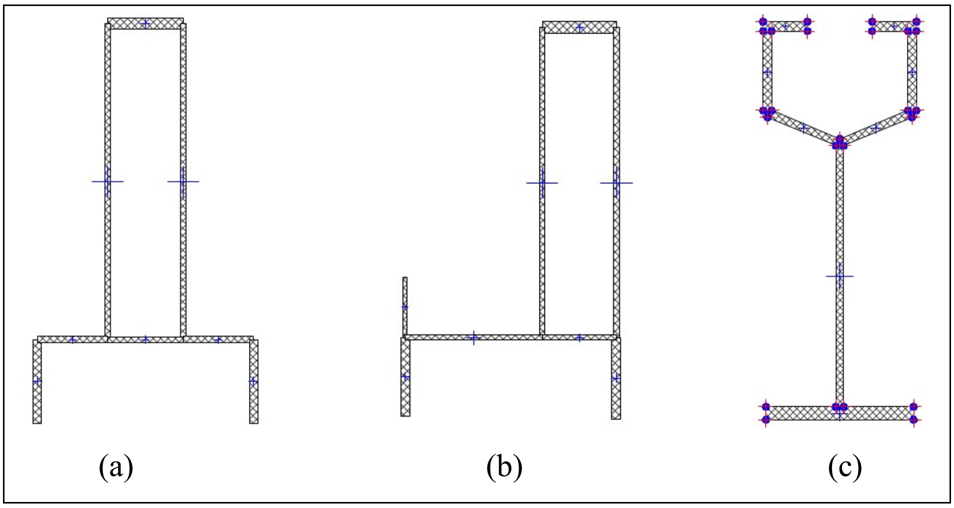

Figure 5.1. Section designer images of beam members (a) Upper main beam, (b) Lower main beam, (c) Purlins.

Section properties of the modelled members are taken from the SAP2000 program and given in Table 5.1.

Table 5.1. Material data of aluminium members

Material Data	Purlin	Lower main beam	Upper main beam	Column
A (m^2)	2.30E-04	9.75E-04	9.31E-04	5,01E-04
A_{vy}(m^2)	1.17E-04	2.32E-04	2.97E-04	2,07E-04
A_{vz} (m^2)	1.05E-04	5.55E-04	5.34E-04	3,12E-04
I_y (m^4)	1.57E-07	2.38E-06	2.20E-06	4,29E-07
I_z (m^4)	1.49E-08	7.50E-07	4.94E-07	1,71E-07
$W_{el,y}$ (m^3)	4.44E-06	2.96E-05	2.78E-05	1,10E-05
$W_{el,z}$ (m^3)	1.10E-06	1.33E-05	1.19E-05	7,59E-06

Maximum internal forces of the structural members are taken from the SAP2000 program analysis result and given in Table 5.2.

Table 5.2. Internal forces of aluminium members

Internal Forces	Purlin	Lower main beam	Upper main beam	Column
$N_{t,Ed}$ (kN)	0.84	0.14	0.20	2.33
$N_{c,Ed}$ (kN)	0.62	0.48	0.24	5.85
$V_{y,Ed}$ (kN)	0.03	2.16	0.56	0.48
$V_{z,Ed}$ (kN)	0.68	2.91	2.92	1.75
T_{Ed} (kNm)	0.00	0.33	0.13	0.06
$M_{y,Ed}$ (kNm)	0.27	1.46	1.61	0.35
$M_{z,Ed}$ (kNm)	0.02	0.43	0.28	0.22

5.1.1. Tension Check

Tension check of all members is performed according to Equations (4.1) and (4.2), Table 5.1 and Table 5.2. Values of unity checks are given in Table 5.3.

Table 5.3. Tension check of aluminium members

	Purlin	Lower main beam	Upper main beam	Column
Area (mm^2)	230	975	931	501
$N_{t,Rd}$ (kN)	42	177	169	91
N_{Ed} (kN)	0.84	0.14	0.20	2.33
Unity check	0.02	0.00	0.00	0.03

5.1.2. Compression Check

Compression check of all members is performed according to Equations (4.3) and (4.4), Table 5.1 and Table 5.2. Values of unity checks are given in Table 5.4.

Table 5.4. Compression check of aluminium members

	Purlin	Lower main beam	Upper main beam	Column
Area (mm^2)	230	975	931	501
$N_{c,Rd}$ (kN)	42	177	169	91
N_{Ed} (kN)	0.62	0.48	0.24	5.85
Unity check	0.01	0.00	0.00	0.06

5.1.3. Bending Check

Bending check of all members is performed according to Equations (4.5) and (4.6), Table 5.1 and Table 5.2. Values of unity checks in both Y and Z axes of cross-section are given in Table 5.5 and Table 5.6.

Table 5.5. Bending check of aluminium members in Y axis

	Purlin	Lower main beam	Upper main beam	Column
$W_{el,y}$ (mm^3)	4442	29610	27810	10990
M_{Rd} (kNm)	0.81	5.38	5.06	2.00
M_{Ed} (kNm)	0.27	1.46	1.61	0.35
Unity check	0.33	0.27	0.32	0.18

Table 5.6. Bending check of aluminium members in Z axis

	Purlin	Lower main beam	Upper main beam	Column
$W_{el,z}$ (mm^3)	1103	13310	11880	7589
M_{Rd} (kNm)	0.20	2.42	2.16	1.38
M_{Ed} (kNm)	0.02	0.43	0.28	0.22
Unity check	0.10	0.18	0.13	0.16

5.1.4. Shear Check

Shear check of all members is performed according to Equations (4.7) and (4.8), Table 5.1 and Table 5.2. Values of unity checks in both Y and Z directions of cross- section are given in Table 5.7 and Table 5.8.

Table 5.7. Shear check of aluminium members in Y axis

	Purlin	Lower main beam	Upper main beam	Column
A_{vy} (mm^2)	117	232	297	207
$V_{y,Rd}$ (kN)	12	24	31	22
V_{Ed} (kN)	0.03	2.16	0.56	0.48
Unity check	0.00	0.09	0.02	0.02

Table 5.8. Shear check of aluminium members in Z axis

	Purlin	Lower main beam	Upper main beam	Column
A_{vz} (mm^2)	105	555	534	312
$V_{z,Rd}$ (kNm)	11	58	56	33
V_{Ed} (kNm)	0.68	2.91	2.92	1.75
Unity check	0.06	0.05	0.05	0.05

5.1.5. Flexural Buckling and Torsional Buckling Check

As indicated in Chapter 4.1.6, flexural and torsional buckling effects may be ignored for the purlins. Beams, with low compression forces, must be checked if the design resistance moment is adequate enough to be neglected of the flexural buckling check. Also the section with the smallest elastic critical force value among beams is the upper main beam. Therefore, it should satisfy the Equation (4.14) by calculating N_{cr} according to Equation (4.15) as follows:

$$N_{cr} = \frac{\pi^2 x70000\text{ MPa x }2.20x10^{-6}m^4}{1.0x(3.470\text{ m})^2} = 126.23\text{ kN}$$
$$N_{Ed} = 0.24\text{ kN} \leq 1.26\text{ kN} \tag{5.1}$$

which indicates that only cross-sectional check applies for the beam members.

For column members, the design buckling resistance must be calculated using equations given in Chapter 4.1.6. According to Equation (4.15), critical axial force N_{cr} of a longer rear column member equals to:

$$N_{cr} = \frac{\pi^2 x70000\text{ MPa x }4.29x10^{-7}m^4}{1.0x(1.865\text{ m})^2} = 85.21\text{ kN} \tag{5.2}$$

According to Equation (4.13), $\bar{\lambda}$ equals to:

$$\bar{\lambda} = \sqrt{\frac{5.1\text{x}10^{-4}\ \text{m}^2\ \text{x}\ 200\ \text{MPa}}{85.21\ \text{kN}}} = 1.094 \tag{5.3}$$

According to Equation (4.12), ϕ equals to:

$$\phi = 0.5\text{x}(1 + 0.2\text{x}(1.094 - 0.1) + 1.094^2) = 2.396 \tag{5.4}$$

According to Equation (4.11), χ equals to:

$$\chi = \frac{1}{2.396 + \sqrt{2.396^2 - 1.094^2}} = 0.144 \tag{5.5}$$

Therefore, flexural and torsional buckling check of columns satisfies the relevant condition in Equation (4.9) and (4.10) as given in Equation (5.6).

$$\begin{gathered} \text{N}_{\text{b,Rd}} = 1.0\text{x}0.144\text{x}1.0\text{x}5.01\text{x}10^{-4}\text{m}^2\text{x}200\ \text{MPa}/1.10 = 13.12\ \text{kN} \\ \frac{\text{N}_{\text{Ed}}}{\text{N}_{\text{b,Rd}}} = \frac{5.85\ \text{kN}}{13.12\ \text{kN}} = 0.46 \leq 1.0 \end{gathered} \tag{5.6}$$

5.1.6. Lateral Torsional Buckling Check

As indicated in Chapter 4.1.7, flexural and torsional buckling effects may be ignored for the purlins. Beams, which due to low compression forces, must be checked if the design resistance moment is adequate enough to be neglected of the flexural buckling check. Also the section with the smallest elastic critical moment value among beams is the upper main beam. Therefore, it should satisfy the Equation (4.21) by calculating M_{cr} according to Equation (4.22) as follows:

$$M_{cr} = \frac{\pi\sqrt{70000\ \text{MPa x}\ 4.94\text{x}10^{-7}\text{m}^4\text{x}27000\ \text{MPa x}\ 3.18\text{x}10^{-7}\text{m}^4}}{3.470\ \text{m}} \tag{5.7}$$

$$M_{cr} = 15.6\ \text{kNm}$$

$$M_{Ed} = 1.61\ \text{kNm} \leq 2.50\ \text{kNm}$$

which indicates that only cross-sectional check applies for the beam members.

For column members, the design buckling resistance must be calculated using Equations given in Chapter 4.1.7. According to Equation (4.22), elastic critical moment, M_{cr}, of a longer rear column member equals to:

$$M_{cr} = \frac{\pi\sqrt{70000\ \text{MPa x}\ 1.71\text{x}10^{-7}\text{m}^4\text{x}27000\ \text{MPa x}\ 3.75\text{x}10^{-7}\text{m}^4}}{1.865\ \text{m}} \tag{5.8}$$

$$M_{cr} = 18.54\ \text{kNm}$$

According to Equation (4.20), $\bar{\lambda}_{LT}$ equals to:

$$\bar{\lambda}_{LT} = \sqrt{\frac{0.2\ \text{x}\ 1.10\text{x}10^{-5}\ \text{m}^3\ \text{x}\ 200\ \text{MPa}}{32.12\ \text{kNm}}} = 0.154 \tag{5.9}$$

According to Figure 6.13 of EN1999-1-1 [6], χ_{LT} equals to 1.

Therefore, lateral torsional buckling check of columns satisfies the relevant condition of Equation (4.17) and Equation (4.16) as given in Equation (5.10).

$$M_{b,Rd} = 1.0\text{x}1.0\text{x}0.2\text{x}1.10\text{x}10^{-5}\text{m}^3\text{x}200\ \text{MPa}/1.10 = 0.4\ \text{kNm}$$

$$\frac{M_{Ed}}{M_{b,Rd}} = \frac{0.22\ \text{kNm}}{0.4\ \text{kNm}} = 0.55 \leq 1.0 \tag{5.10}$$

5.1.7. Seismic Deflections Check

Seismic deflections check of the aluminium structures is performed as defined in Chapter 3.2 and as indicated in Chapter 4.2.

Values of the maximum displacements of joints in both directions X and Y under the earthquake loads, which are EQ_X and EQ_Y, taken from the SAP2000 program analysis, are given in Table 5.9.

Table 5.9. Maximum seismic displacement values of aluminium structure

Load case	Maximum displacement (m)
EQ_X	0.001981
EQ_Y	0.007964

According to Equation (2.6), the maximum deflection in X direction for seismic criteria equals to:

$$\delta_{seismic,x} = \frac{4.0}{1.5} 0.001981 \text{ m} = 0.0053 \text{ m} \tag{5.11}$$

And the maximum deflection in Y direction for seismic criteria equals to:

$$\delta_{seismic,y} = \frac{4.0}{1.5} 0.007964 \text{ m} = 0.0212 \text{ m} \tag{5.12}$$

According to Equation (3.19), maximum deflections must be lesser than 0.016 times the coefficient κ, which equals to 0.008.

Therefore, the deflection check of the displacements in X direction satisfies the required criteria, which is indicated in Chapter 3.2 and Equation (3.19) as follows:

$$0.534\text{x}\frac{0.0053\ \text{m}}{1.865\ \text{m}} = 0.0015 \leq 0.008 \tag{5.13}$$

The deflection check of the displacements in Y direction satisfies the required criteria, which is indicated in Chapter 3.2 and Equation (3.19) as follows:

$$0.534\text{x}\frac{0.0212\ \text{m}}{1.865\ \text{m}} = 0.0061 \leq 0.008 \tag{5.14}$$

5.1.8. Serviceability Check

Serviceability checks of the aluminium frame are performed according to Chapter 2.2.3 and Equation (2.8).

For purlins, maximum deflection relative to beam ends at spans is 5.74 millimetres which is smaller than L/360 for L being equal to 2456 millimetres, and maximum deflection at cantilever ends is minus 0.41 millimetres which is smaller than L/180 for L being equal to 841 millimetres.

For upper main beam, maximum deflection relative to beam ends at spans is 7.23 millimetres which is adequate L being equal to 3470 millimetres and at cantilever ends is minus 0.48 millimetres also adequate L being equal to 1069 millimetres.

For lower main beam, maximum deflection relative to beam ends at spans is 6.34 millimetres which is adequate and at cantilever ends is minus 0.40 millimetres also adequate for same lengths of upper main beam for both span and cantilever ends.

For columns, maximum deflection at shorter front column is 4.32 millimetres which is smaller than L/180 for L being equal to 827 millimetres and at longer rear column is 4.56 for L being equal to 1865 millimetres.

5.2. STEEL FRAME OPTION-1 DESIGN

All members for Steel Frame Option-1 are modelled as tube sections, cold formed hollow sections, with dimensions of:

30 mm x 30 mm x 3mm for purlins,

80 mm x 40 mm x 3 mm for lower and upper main beams and

60 mm x 60 mm x 3 mm for columns in SAP2000 program.

Section properties of the modelled members are taken from the SAP2000 program and given in Table 5.10.

Table 5.10. Material data of steel members

Material Data	Purlin	Lower main beam	Upper main beam	Column
A (m^2)	3.24E-04	6.84E-04	6.84E-04	6.84E-04
A_{vy} (m^2)	1.80E-04	2.40E-04	2.40E-04	3.60E-04
A_{vz} (m^2)	1.80E-04	4.80E-04	4.80E-04	3.60E-04
I_y (m^4)	3.98E-08	5.58E-07	5.58E-07	3.71E-07
I_z (m^4)	3.98E-08	1.84E-07	1.84E-07	3.71E-07
$W_{el,y}$ (m^3)	2.66E-06	1.40E-05	1.40E-05	1.24E-05
$W_{el,z}$ (m^3)	2.66E-06	9.20E-06	0.92E-05	1.24E-05

Maximum internal forces of the structural members are taken from the SAP2000 program analysis result and given in Table 5.11.

Table 5.11. Internal forces of steel members

Internal Forces	Purlin	Lower main beam	Upper main beam	Column
$N_{t,Ed}$ (kN)	0.61	0.53	0.40	2.88
$N_{c,Ed}$ (kN)	1.37	1.16	0.22	6.08
$V_{y,Ed}$ (kN)	0.07	2.16	0.73	1.34
$V_{z,Ed}$ (kN)	1.31	3.37	3.38	2.15
T_{Ed} (kNm)	0,00	0.28	0.33	0.12
$M_{y,Ed}$ (kNm)	0.49	1.83	1.86	0.92
$M_{z,Ed}$ (kNm)	0.08	0.68	0.44	0.41

5.2.1. Tension Check

Tension check of all members is performed according to Equations (3.1) and (3.2), Table 5.10 and Table 5.11. Values of unity checks are given in Table 5.12.

Table 5.12. Tension check of steel members

	Purlin	Lower main beam	Upper main beam	Column
Area (mm^2)	324	684	684	684
$N_{t,Rd}$ (kN)	84	177	177	177
N_{Ed} (kN)	0.61	0.53	0.40	2.88
Unity check	0.01	0.00	0.00	0.02

5.2.2. Compression Check

Compression check of all members is performed according to Equations (3.3) and (3.4), Table 5.10 and Table 5.11. Values of unity checks are given in Table 5.13.

Table 5.13. Compression check of steel members

	Purlin	Lower main beam	Upper main beam	Column
Area (mm^2)	324	684	684	684
$N_{c,Rd}$ (kN)	76	160	160	160
N_{Ed} (kN)	1.37	1.16	0.22	6.08
Unity check	0.02	0.01	0.00	0.04

5.2.3. Bending Check

Bending check of all members is performed according to Equations (3.5) and (3.6), Table 5.10 and Table 5.11. Values of unity checks in both Y and Z directions of cross-section are given in Table 5.14 and Table 5.15.

Table 5.14. Bending check of steel members in Y axis

	Purlin	Lower main beam	Upper main beam	Column
$W_{el,y}$ (mm^3)	2660	14000	14000	12400
$M_{c,Rd}$ (kNm)	0.63	3.29	3.29	2.91
M_{Ed} (kNm)	0.49	1.83	1.86	0.92
Unity check	0.78	0.56	0.57	0.32

Table 5.15. Bending check of steel members in Z axis

	Purlin	Lower main beam	Upper main beam	Column
$W_{el,z}$ (mm^3)	2660	9200	9200	12400
$M_{c,Rd}$ (kNm)	0.63	2.16	2.16	2.91
M_{Ed} (kNm)	0.08	0.68	0.44	0.41
Unity check	0.13	0.31	0.20	0.14

5.2.4. Shear Check

Shear check of all members is performed according to Equations (3.7) and (3.8), Table 5.10 and Table 5.11. Values of unity checks in both Y and Z directions of cross- section are given in Table 5.16 and Table 5.17.

Table 5.16. Shear check of steel members in Y axis

	Purlin	Lower main beam	Upper main beam	Column
A_{vy} (mm^2)	180	240	240	360
$V_{c,Rd}$ (kN)	24	33	33	49
V_{Ed} (kN)	0.07	2.16	0.73	1.34
Unity check	0.00	0.07	0.02	0.03

Table 5.17. Shear check of steel members in Z axis

	Purlin	Lower main beam	Upper main beam	Column
A_{vz} (mm^2)	180	480	480	360
$V_{c,Rd}$ (kN)	24	65	65	49
V_{Ed} (kN)	1.31	3.37	3.38	2.15
Unity check	0.05	0.05	0.05	0.04

5.2.5. Flexural Buckling and Torsional Buckling Check

Flexural buckling and torsional buckling check of all members is performed according to Equations given in Chapter 3.1.6, Table 5.10 and Table 5.11. Unity check values of the most critical direction of the cross-section are given in Table 5.18.

Table 5.18. Flexural and torsional buckling check of steel members in critical axis

	Purlin	Lower main beam	Upper main beam	Column
N_{cr} (kN)	13.7	382	382	55.3
$\bar{\lambda}$	2.36	0.65	0.65	1.70
α	0.49	0.49	0.49	0.49
ϕ	3.81	0.82	0.82	2.32
χ	0.15	0.76	0.76	0.26
$N_{b,Rd}$ (kN)	11.4	122	122	41.3
N_{Ed} (kN)	1.37	1.16	0.22	6.08
Unity Check	0.12	0.01	0.00	0.15

5.2.6. Lateral Torsional Buckling Check

Lateral torsional buckling check of all members is performed according to equations given in Chapter 3.1.7, Table 5.10 and Table 5.11. Unity check values of the most critical direction of the cross-section are given in Table 5.19.

Table 5.19. Lateral torsional buckling check of steel members in critical axis

	Purlin	Lower main beam	Upper main beam	Column
M_{cr} (kNm)	9.04	113	113	246
$\bar{\lambda}_{LT}$	0.26	0.17	0.17	0.11
α_{LT}	0.56	0.76	0.76	0.76
ϕ_{LT}	3.81	0.50	0.50	0.47
χ_{LT}	0.95	1.00	1.00	1.00
$M_{b,Rd}$ (kNm)	0.60	3.29	3.29	2.91
M_{Ed} (kNm)	0.49	1.83	1.86	0.92
Unity Check	0.82	0.55	0.56	0.32

5.2.7. Seismic Deflection Check

Seismic design check of the steel structures is performed according to Chapter 3.2.

Values of the maximum displacements of joints in both directions X and Y under the earthquake loads, which are EQ_X and EQ_Y, taken from the SAP2000 program analysis, are given in Table 5.20.

Table 5.20. Maximum seismic displacement values of steel structure

Load case	Maximum displacement (m)
EQ_X	0.001851
EQ_Y	0.003144

According to Equation (2.6), maximum deflection in X direction for seismic criteria equals to,

$$\delta_{\text{seismic,x}} = \frac{4.0}{1.5} 0.001851 \text{ m} = 0.005 \text{ m} \tag{5.15}$$

And maximum deflection in Y direction for seismic criteria equals to,

$$\delta_{\text{seismic,y}} = \frac{4.0}{1.5} 0.003144 \text{ m} = 0.0084 \text{ m} \tag{5.16}$$

According to Equation (3.19), maximum deflections must be lesser than 0.016 times the coefficient κ, which equals to 0.008.

Therefore, the deflection check of the displacements in X direction satisfies the required criteria, which is indicated in Chapter 3.2 and Equation (3.19) as follows:

$$0.534\text{x}\frac{0.005\text{ m}}{1.865\text{ m}} = 0.0014 \leq 0.008 \tag{5.17}$$

The deflection check of the displacements in Y direction satisfies the required criteria, which is indicated in Chapter 3.2 and Equation (3.19), as follows:

$$0.534\text{x}\frac{0.0084\text{ m}}{1.865\text{ m}} = 0.0024 \leq 0.008 \tag{5.18}$$

5.2.8. Serviceability Check

Serviceability checks of the steel frame is performed according to Chapter 2.2.3 and Equation (2.8).

For purlins, maximum deflection relative to beam ends at spans is 3.04 millimetres which is smaller than L/360 for L being equal to 2456 millimetres, and maximum deflection at cantilever ends is minus 0.85 millimetres which is smaller than L/180 for L being equal to 841 millimetres.

For upper main beam, maximum deflection relative to beam ends at spans is 9.12 millimetres which is adequate L being equal to 3470 millimetres and at cantilever ends is minus 0.64 millimetres also adequate L being equal to 1069 millimetres.

For lower main beam, maximum deflection relative to beam ends at spans is 8.46 millimetres which is adequate and at cantilever ends is minus 0.61 millimetres also adequate for same lengths of upper main beam for both span and cantilever ends.

For columns, maximum deflection at shorter front column is 2.19 millimetres which is smaller than L/180 for L being equal to 827 millimetres and at longer rear column is 2.03 for L being equal to 1865 millimetres.

5.3. STEEL FRAME OPTION-2 DESIGN

All members for Steel Frame Option-2 are modelled as tube sections, cold formed hollow sections, with dimensions of:

30 mm x 30 mm x 3 mm for purlins,

100 mm x 40 mm x 3 mm for lower and upper main beams and

70 mm x 70 mm x 3 mm for columns in SAP2000 program.

All checks which are subjected to Steel Frame Option-1 given in Chapter 3 and Chapter 5.2 are also valid for Steel Frame Option-2.

In this frame, where the total number of columns was reduced to 10, the beam and column dimensions were increased in order to provide sufficient deflection and strength values.

The sectional strength resulting from these dimensions meets the requirements of all design, seismic and serviceability checks given in Chapter 3 and Chapter 5.2, therefore no further calculations will be presented.

6. COMPARISON OF DESIGNS

Aluminium and steel design options, that are determined to be structurally adequate under the given loads by modelling with the analysis program, are reduced to two headings, which are material quantities and CO_2 emissions, in order to be compared in terms of sustainability.

To have a comparison with two subjects, one of the steel option must be selected to be compared with aluminium frame. Steel Frame Option-2 has fewer columns as indicated in Chapter 5.3 but it's heavier than Steel Frame Option-1 thus leaving it at disadvantage in terms of cost and CO_2 emissions. Therefore, Steel Frame Option-1 is selected for the comparison of metal frames.

The program outputs of the aluminium and steel frame models that will be examined under these two headings are given in following chapters.

6.1. MATERIAL QUANTITY

Material quantities of the both aluminium and steel frame are given in details in Table 6.1 and Table 6.2. As can be understood from Tables 6.1 and 6.2 the weight of the aluminium frame is about one third of that of steel frame.

Table 6.1. Material quantities of aluminium metal frame

Section	Total length (m)	Total weight (kN)	Total mass (kg)
Column	16.15	0.218	22.23
Upper main beam	19.48	0.490	49.97
Lower main beam	19.48	0.513	52.31
Purlins	98.88	0.613	62.51
		1.834	187.02

Table 6.2. Material quantities of steel metal frame

Section	Total length (m)	Total weight (kN)	Total mass (kg)
Column	16.15	0.850	86.68
Upper main beam	19.48	1.026	104.62
Lower main beam	19.48	1.026	104.62
Purlins	98.88	2.466	251.46
		5.368	547.38

6.2. CO_2 EMISSIONS

Construction materials have an embodied (energy/carbon) impact that arose from the extraction, processing, transportation and fabrication of the material which, according to BSRIA ICE Guide [14], defined as the total primary energy consumed from direct and indirect processes associated with a product or service and within the boundaries of cradle-to-gate, including all activities from material extraction, manufacturing, transportation and right through to fabrication processes until the product is ready to leave the final factory gate [14].

In this thesis, the embodied carbon values provided by Table 1 of BSRIA ICE Guide [14] were used to compare the carbon dioxide emissions of the materials. Using the cradle-to-gate results provided by these values, the carbon dioxide values produced by the materials at all stages are compared in Table 6.3.

Table 6.3. Embodied carbon coefficients and total CO_2 emissions of materials

Materials	Embodied carbon coefficients ($kgCO_2/kg$)	Total masses (kg)	Total CO_2 emissions (kg)
Aluminium (Extruded-Virgin)	11.8	187.02	2,206.84
Steel (Section-Virgin)	2.82	547.38	1,543.61

7. CONCLUSIONS

Humanity continues to evolve and learn as it always has. One of the most important consequences of this ongoing situation is the need for energy.

Considering the predictions put forward so far, today's methods will begin to be insufficient to meet the energy needs of humanity. Therefore, unless a permanent solution is offered in the energy generation sector, one of the best things to do is to keep the amount of energy consumed at levels that can be met by production.

Because of all these, this thesis draws attention to the differences in the construction phase of solar power generation plants.

7.1. SUSTAINABILITY ASPECT OF DIFFERENT DESIGNS

In the two models created for the same structure, the structural elements were designed as aluminium and steel, allowing the two materials to be compared. The structures formed by both materials were checked in accordance with the relevant standards and it was seen that the requirements were met.

CO_2 emission values were calculated in order to compare them in terms of sustainability by using the program outputs of the structures that provide sufficient values. Although the weight of the steel structure is approximately three times more than the weight of the aluminium structure, it has been observed that the amount of carbon dioxide released by the aluminium structure to the environment during the cradle-to-gate phase is 43 percent higher than that of the steel structure. Despite the small amount of aluminium, the fact that it has a much larger carbon footprint than steel, reveals one of the biggest differences between the two materials in terms of sustainability.

It is important to mention the maintenance differences of steel and aluminium shapes as the structure in question is built in an open environment. Aluminium is incredibly resistant to corrosion therefore, after it is extruded there is no need of additional treatments. Furthermore, aluminium does not rust, and does not require any coatings. On the other hand, steel is susceptible to corrosion. To protect steel shapes from corrosions it is required that a coating of paint or other finish treatment is applied and maintained throughout the life of the structure. This is especially necessary if the steel will be located in an environment that is wet or moist, as in the case of solar panel plants. Hence, it can be stated that metal constructions for solar panels will continuously need care against corrosion during their operation time.

Although a cost comparison of the two metals is not carried out in this thesis, it is inevitable not to mention the cost differences of the two metals. Steel is usually less expensive than aluminium. However, when designing a custom shape or profile, creating a custom aluminium extrusion is very cheap due to affordable mould and die costs.

7.2. FUTURE STUDY

In this thesis, aluminium and steel frame constructions are designed to resist applied loads in field installations. Among these loadings wind loading was the most uncertain loading to calculate. In many design codes wind load on any structure is calculated by multiplying the reference wind pressure of the region where the structure is built, by a factor. This factor generally represents the effects of the change in wind speed with the height of structure, surrounding terrain or topography, gust, aerodynamic shape of the structure, orientation of the surface with respect to the wind flow, dynamic properties of the structure etc. As can be understood, it is usually not an easy task to determine the wind load on a given structure and approximate methods that are given in many design codes It is also important to note that, wind load calculation for a single solar panel construction neglects the effect of the other solar panel constructions around.

Although several studies show that wind tunnel tests are more reliable to determine wind pressure coefficients, there is no generalization of the results to be included in a design code. Studies on the pressure coefficients, for solar panel plants, using the wind tunnel tests could be carried out to present a safe procedure which would clarify the issue about what coefficients must be used for a safe design.

Considering the maintenance aspect of steel structures new studies could also be carried out to reduce the carbon footprint of aluminium production.

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