

ISTANBUL TECHNICAL UNIVERSITY ★ GRADUATE SCHOOL

**THE EFFECTS OF MOISTURE CONTENT AND LOAD DURATION ON THE
BEHAVIOUR OF GLUED LAMINATED TIMBER THREE-PINNED PORTAL
FRAMES WITH TAPERED MEMBERS**



M.Sc. THESIS

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Department of Civil Engineering

Structure Engineering Programme

FEBRUARY 2023

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İSTANBUL TEKNİK ÜNİVERSİTESİ ★ LİSANSÜSTÜ EĞİTİM ENSTİTÜSÜ

**RUTUBETİN VE YÜK ETKİ SÜRESİNİN DEĞİŞKEN KESİTLİ, ÜÇ
MAFSALLI TUTKALLI LAMİNE AHŞAP ÇERÇEVENİN DAVRANIŞINA
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To my family and friends,



FOREWORD

I would like to express my appreciation to my advisor, Prof. Dr Güliz Bayramoğlu, for her encouragement, support and patience during this entire Master's study. Undoubtedly, I could not have completed this study if I had not had her guidance.

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ABBREVIATIONS

EC0 : Eurocode - Basis of structural design

EC1- Part 1.3: Eurocode 1: Actions on Structures - Part 1-4: General actions - Wind

EC1- Part 1.4: Eurocode 1: Actions on structures - Part 1-3: General actions – Snow Load

EC5 : Eurocode 5: Design of timber structures - Part 1-1: General - Common rules and rules for buildings

EC8 : Eurocode 8: Design of structures for earthquake resistance

ULS : Ultimate limit state

SLS : Serviceability limit state



SYMBOLS

a_g	: Design ground acceleration
a_{gR}	: Reference peak ground acceleration
β_c	: Straightness factor
C_{pe}	: External pressure coefficient
d_r	: Design interstorey drift
E_{mean}	: Mean value of modulus of elasticity
E_d	: Design value of modulus of elasticity
$E_{0.05}$	: Fifth percentile value of modulus of elasticity
$E_{0,ort}$	: Mean value of modulus of elasticity parallel to the grain
$E_{90,ort}$	: Mean value of modulus of elasticity perpendicular to the grain
$f_{m,k}$	: Characteristic bending strength
$f_{c,0,k}$	: Characteristic compressive strength along the grain
$f_{c,90,k}$	: Characteristic compressive strength perpendicular to grain
f_{inst}	: Instant deflection
f_{fin}	: Final deflection
f_i	: Horizontal seismic force at storey i
$f_{t,0,k}$	: Characteristic tensile strength along the grain
$f_{t,90,k}$	: Characteristic tensile strength perpendicular to the grain
$f_{v,k}$	: Characteristic design shear strength
F_b	: Lateral earthquake load
$G_{0.05}$	: Fifth percentile value of shear modulus
G_{mean}	: Mean value of value of shear modulus
G_d	: Design value of shear modulus
h	: Interstorey height
k_{crit}	: Factor used for lateral buckling
$k_{c,z}$	: Instability factor
k_{mod}	: A modification factor taking into account the effect of the duration of load and moisture content
k_{def}	: Deformation factor
m_i	: Mass of storey i

n	: Damping correction factor
q	: Behaviour factor
R_d	: Design value of load-carrying capacity
R_k	: Characteristic value of load-carrying capacity
s_i	: Displacement of mass m_i in the fundamental mode shape of a building
$S_d(T)$	: Ordinate of the design spectrum
$S_d(T_1)$	: Ordinate of the design spectrum at period T_1
S_{De}	: Spectral displacement
$S_e(T)$	: Elastic response spectrum
α	: Angle between the x-direction and the force for a punched metal plate; Angle between a force and the direction of grain; Angle between the direction of the load and the loaded edge (or end)
T_1	: Fundamental period of vibration
w_{inst}	: Instantaneous deflection
w_{creep}	: Creep deflection
w_{fin}	: Final deflection
W_y	: Section modulus about the strong axis y
$\sigma_{m,y,d}$	: Design bending stress about the principal y-axis
$\sigma_{c,0,d}$	: Design compressive stress along the grain
$\sigma_{m,kr}$	: Critical bending stress
$\sigma_{c,\alpha,d}$	: Compressive stress at an angle α to the grain
λ	: Correction factor
λ_y	: Slenderness ratio corresponding to bending about the y-axis
λ_z	: Slenderness ratio corresponding to bending about the z-axis
$\lambda_{rel,y}$	: Relative slenderness ratio corresponding to bending about the y-axis
$\lambda_{rel,z}$	: Relative slenderness ratio corresponding to bending about the z-axis
ψ_0	: Factor for combination value of a variable action
ψ_2	: Factor for quasi-permanent value of a variable action
ν	: Reduction factor which takes into account the lower return period of the seismic action associated with the damage limitation requirement.
γ_M	: Partial factor for a material property

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SUMMARY

The early 1900s were the beginning of the lengthy history of glued laminated timber's patent of the construction method was taken in Switzerland. Since then, the technics of lamination, adhesive properties and manufacturing facilities have developed, and this helped glued laminated timber to spread around the World.

Glued laminated timber, which is usually treated as an orthotropic material in engineering (Mascia & Lahr, 2006), is made of two or more layers of processed lumbers parallel to each other with structural adhesives. The material's strength and stiffness differ based on the direction of the material and the direction of the loading therewithal.

There are many motivations to increase the usage of timber, which are included but not limited to: being a light structural material that provides easier constructability, less seismic load due to light mass, reducing carbon emission by acting as a carbon store, being sustainable material, performing slow burn and being predictable during fire and more.

The three-pinned portal frame, one of the most common glued laminated timber structures, is the subject of this thesis. The thesis aims to understand the effect of the moisture, load duration and the service classes of the glued laminated timber on three pinned portal frame members along with the glued laminated timber material's structural behaviour.

The span and spacing of the frames have been chosen with consideration of commonly used construction products. Therefore, the strength classes considered are; GL24, GL28, GL30 and GL32. All service classes and the applicable load durations have been considered for the analysis. The symmetrical tapered portal frame was designed with 24 m of spanning with an assumed 5 m of frame spacing. To analyse the portal frame's maximum of 1 capacity utilisation capacity, the design of the portal frame has been determined based on the most critical material, loading term and the service class, which are GL24 class timber, medium-term loading and service class 3.

There are three service classes, all of which have been included in this study. However, only two of the five load durations have been included in this study. The service classes differ from 1 to 3 depending on the material's moisture content and the surrounding environment's humidity. The class is higher where the moisture content is higher (British Standards Institution, 2004). On the other hand, the load durations are specified in EC5 according to the load type and the accumulated load duration range. For the study case, the only load that can have different load durations is snow, which the considered load durations are medium-term and short-term load durations.

The analysis and design of the glued laminated timber members have been done primarily based on Eurocode standards, and where geographical information is

required but not available, the Turkish standards have also been considered. So, the characteristic snow load has been taken from TSE 498 (Turkish Standards Institution, 1997) due to the lack of Turkey details in EC1-Part 1.3.

The seismic load has been carried out per EC8, spectral type 1 (higher seismic), and the ground and the seismic zone parameters have been chosen according to the structure's location, which is in Turkey, Marmara region. The reference peak ground acceleration is taken from the Turkey 475 return period seismic map (TMMOB Chamber of Geology Engineers, 2022). The fundamental period of vibration (T_1) was first calculated with the Rayleigh method with the required fictitious of 1 kN horizontal load to be applied at the top of the columns in the Sap2000 model to determine the deflection. Additionally, the T_1 period has been checked with modal response spectrum analysis to compare the results. It has been seen that both results are only slightly different, and they are both between T_B and T_C . So that the base shear force is found per the first period and the ordinate of the design spectrum ($S_{d(T)}$).

This thesis study worked with the programs: 2-D SAP2000 analysis, AutoCAD, TEDDS for Word and MS Office. The frame members, the joint releases, and material properties have been assigned along with the specified loading and the combinations per EC0 and EC5 in the SAP2000. This software provided the moment bending, shear and axial force body diagrams in addition to the initial joint deflection. According to the first design, the tapered beams and columns are designed with 1.00 capacity utilisation capacity: the beams' thickness is from apex to the end, 0.4 m to 1.225 m, and the columns' are from pinned to the other end, 0.5 m to 1.3 m. The width of the beams and columns is 0.25 m. The details have been provided in section 4 with drawings.

To run the analysis of beam and column design multiple times under different loadings and different materials strength, the calculation sheet has been prepared with the program TEDDS for Word. In these formula sheets, the most critical governing strength and deformation combinations are considered. The results showed that the ULS govern case is $1.35G + 1.5S$ for both columns and beams, whereas the SLS govern case is $G + S$ for beams, and the governing case of the column is $G + W$.

The capacity utilisation results have been gathered and plotted with MS Excel charts initially based on the material strength class, limit state, and whether it is a beam or a column. The interpretations of these graphs are provided in the conclusion, and additional graphs are included. The interpretation of results has been undertaken for ULS and SLS separately for all considered materials. Additionally, a combined ULS and SLS chart has been presented for the GL24 class material.

The combined ULS and SLS graphs of GL24 material for both beam and column showed that the governing limit state is the ultimate limit state compared to the serviceability limit state with a capacity utilisation difference of more than %40.

The overall governing case is the strength of the beam compared to the column with the capacity utilisation of 1.00 where the considerations are; service class 3 and load-duration class of snow is medium term. Also, one of the main interpretations is that the columns are governed by the stability capacity, whereas the beams are governed by stress capacity. It can also be seen that the ULS utilisation of both services, class 1 and 2 are equal due to the same k_{mod} coefficient. However, this differs for service class 3 due to different load-duration classes of snow. Additionally, it is noted that the strength class of beams and columns can be selected differently for economical

solutions because the capacity utilisation of columns is always less than the beams' capacity utilisation.

The serviceability limit state has been calculated for portal frames' vertical and horizontal displacement limitations. During the limitation consideration, it is noted that the portal frame does not have any crane and the roof to support non-flexible or brittle parts (Steel Construction Institute, 2006). One of the findings is that the capacity usage difference between the vertical and horizontal deflections is significant in the serviceability limit state capacity utilisation. It demonstrates that vertical displacement is more critical than horizontal deflection. Vertical displacement has the highest serviceability capacity utilisation of 0.524, while horizontal displacement has 0.19. Additionally, the portal frame's vertical deflection is governed by the initial deflection, whereas the horizontal deflection limitation is governed by the final deflection. Thus, regardless of the service class difference, the SLS capacity utilisation is the same for vertical deflection. Therefore, no service class and moisture content impact can be observed on the portal frame's vertical deflection. These results can be used in future designs, making the design more efficient.





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ÖZET

1900'lerin başları, yapıştırılmış lamine ahşabın uzun tarihinin başlangıcıdır. Tutkallı lamine ahşap için inşaat yöntemi patenti ilk olarak İsviçre'de alındı. O zamandan beri, laminasyon teknikleri, yapışkan özellikleri ve üretim tesisleri gelişti ve bu, yapıştırılmış lamine ahşabın Dünya'ya yayılmasına yardımcı oldu.

Tutkallı lamine ahşap, yapısal yapıştırıcılarla birbirine paralel iki veya daha fazla işlenmiş kereste katmanından oluşan, mühendislikte ortotropik malzeme olarak kullanılmakta olan yapısal bir malzemedir. Malzemenin mukavemeti ve sertliği, malzemenin yönüne ve bununla ilgili yükleme yönüne göre değişmektedir.

Ahşabın kullanımının artması için birçok motivasyon vardır; daha kolay inşa edilebilirlik sağlayan hafif bir yapı malzemesi olması, hafif kütlesi nedeniyle daha az sismik yük gerçekleşmesi, karbon deposu görevi görerek karbon emisyonunu azaltan sürdürülebilir bir malzeme olması gibi. Bunlarla sınırlı olmamak üzere ahşap aynı zamanda yangın durumunda yavaş yanma ve öngörülebilir olması ile de dikkat çeken yangın performansı yüksek bir yapı malzemesidir.

En yaygın tutkallı lamine ahşap yapılarından biri olan üç mafsallı moment çerçevesi bu tezin konusunu oluşturmaktadır. Tez, tutkallı lamine ahşabın nem, yük etki süresi ve kullanım sınıflarının üç mafsallı portal çerçeve elemanı üzerindeki etkisini ve yapıştırılmış lamine ahşap malzemenin yapısal davranışını anlamayı amaçlamaktadır.

Çerçeve açıklıkları ve aralıkları, yaygın olarak kullanılan yapı malzemeleri dikkate alınarak seçilmiştir. Bu nedenle dikkate alınan mukavemet sınıfları; GL24, GL28, GL30 ve GL32. Analiz için tüm kullanım sınıfları ve uygulanabilir yük etki süreleri dikkate alınmıştır. Simetrik üç mafsallı portal çerçevesi, varsayılan 5 m çerçeve aralıkları ve 24 m açıklıkla tasarlanmıştır. Moment çerçevesinin boyut tasarımı, maksimum kapasitesi kullanımında analiz etmek için, en kritik malzeme, yükleme süresi ve kullanım sınıfı olan GL24 sınıfı ahşap, orta süreli yükleme ve kullanım sınıfı 3 olan değişkenlere göre belirlenmiştir ve kapasite kullanımı 1 de olacak şekilde boyutlandırılmıştır.

Tüm kullanım sınıflarının üçü de bu çalışmaya dahil edilmiştir. Ancak, beş yük etki süresinden sadece ikisi dahil edilebilmiştir. Yük etki süreleri ise yük tipine ve birikmiş yük süresi aralığına göre EC5'te belirtilmiştir. Bu çalışma için, farklı yük etki sürelerine sahip olabilecek tek yük türü, orta ve kısa süreli yük etki süreleri olan kardır. Kullanım sınıfları, malzemenin nem içeriğine ve çevredeki ortamın nemine bağlı olarak 1 ile 3 arasında değişmektedir ve nem içeriğinin yüksek olduğu yerlerde kullanım sınıfı daha yüksektir (British Standards Institution, 2004).

Tutkallı lamine ahşap elemanların analiz ve tasarımı Eurocode standartlarına göre yapılmış olup, coğrafi bilginin gerekli olduğu ancak mevcut olmadığı durumlarda Türk standartları da dikkate alınmıştır. Bu nedenle, EC1-Bölüm 1.3'te Türkiye ayrıntılarının bulunmaması nedeniyle, karakteristik kar yükü TSE 498'den (Turkish Standards Institution, 1997) alınmıştır.

Deprem yük analizi, EC8 spektral tip 1'e (yüksek sismik sınıfı) göre yapılmış ve yapının zemin ve deprem bölgesi parametreleri Türkiye Marmara bölgesindeki konumuna göre seçilmiştir. Referans pik yer ivmesi, Türkiye 475 dönüş periyodu sismik haritasından (TMMOB Chamber of Geology Engineers, 2022) alınmıştır. Sap2000 modelinde yatay deformasyonu belirlemek için kolonların tepesine toplam 1 kN yatay fiktif yük uygulanmış ve ilk olarak Rayleigh yöntemi ile yapının doğal titreşimin periyodu (T_1) hesaplanmıştır. Ek olarak, T_1 periyodu, sonuçları karşılaştırmak için SAP2000 modal spektrum analizi ile de bulunmuş ve karşılaştırılmıştır. Her iki sonucun da çok az farklı olduğu ve her ikisinin de TB ile TC arasında olduğu görülmüştür. Böylece taban kesme kuvveti, birinci periyoda ve tasarım spektrumunun ordinatına ($S_d(T)$) göre bulunmuştur.

Bu tez çalışması, 2 boyutlu SAP2000 analizi, AutoCAD, TEDDS for Word ve MS Office programları ile çalışılmıştır. SAP2000'de EC0 ve EC5'e göre belirtilen yükleme ve kombinasyonlarla birlikte çerçeve elemanları, bağlantı serbestlikleri ve malzeme özellikleri atanmıştır. SAP2000 programı analiz sonucu olarak, moment, kesme ve eksenel kuvvet diyagramlarını sağladı. Kapasite kullanımı 1'de boyutlandırmaya göre, değişken kesitli kirişler ve kolonlar; kirişlerin enine kesitleri mahyadan diğer uca 0.4 m' den 1.225 m' ye ve kolonların mafsallardan diğer uca 0.5 m'den 1.3 m' ye artmaktadır. Kiriş ve kolon kesitlerinin genişlikleri de 0.25 m'dir. Detaylar 4. bölümde çizimlerle birlikte verilmiştir.

Farklı yükleme ve farklı malzeme dayanımları altında kiriş ve kolon tasarımı için birden fazla analiz yapmak üzere, TEDDS programı ile hesaplama tablosu hazırlanmıştır. Bu formül sayfalarında, en kritik taşıma sınır durumu ve kullanılabilirlik sınır durumu kombinasyonları dikkate alınmıştır. Sonuç olarak, taşıma kullanım sınıfının kritik olan kombinasyonunun kiriş ve kolonlar içinde 1.35G + 1.5S olduğu görülmüştür. Kullanılabilirlik sınır durumu için ise; düşey yerdeğiştirme için G + S olduğu, yatay yerdeğiştirme için G + W olduğu gözlenmiştir.

Kapasite kullanım sonuçları, başlangıçta malzeme dayanım sınıfı, kullanım sınıfı ve kritik kesitin kiriş mi kolon mu olduğuna göre MS Excel ile toplanmış ve grafikler ile çizilmiştir. Bu grafiklerin yorumlarına sonuç kısmında yer verilmiştir. Sonuçların yorumlanması, dikkate alınan tüm materyaller için taşıma sınır durumu ve kullanılabilirlik sınır durumu için ayrı ayrı yapılmıştır. Ek olarak, GL24 sınıfı malzeme için bir grafikte taşıma sınır durumu ve kullanılabilirlik sınır durumu ile sunulmuştur.

GL24 malzemesinin hem kiriş hem de kolon için birleştirilmiş sınır durumları karşılaştırma grafikleri, taşıma sınır durumunun kullanılabilirlik sınır durumundan %40'tan fazla bir farka sahip olduğunu göstermektedir.

Genel karşılaştırma yapıldığında, 1 kapasite kullanımıyla, kolona kıyasla kirişin mukavemeti kritik olduğu görülmektedir. 1 kapasitede tasarım varsayımları; kullanım sınıfı 3, GL24 malzeme ve kar yükü yük etki süre sınıfı orta vadelidir. Ayrıca, çalışma sonucunda görülmüştür ki, kolonların stabilite kapasitesi kritik iken, kirişlerin ise gerilme kapasitesi kritiktir. Ayrıca, aynı k_{mod} katsayısı dolayısı ile kullanım sınıfları 1 ve 2'nin taşıma gücü kapasite kullanımının farklı yük etki süre sınıflarına göre eşit olduğunu, ancak bunun, kullanım sınıfı 3 için farklı olduğu görülebilir. Ayrıca, ekonomik çözümler için kiriş ve kolonların dayanım sınıflarının farklı seçilebileceği, çünkü kolonların kullanımının her zaman kirişlerin kullanımından daha az olduğu belirtilmiştir.

Kullanılabilirlik sınır durumu, moment çerçeveleri düşey ve yatay yer değiştirme sınırlamalarına göre hesaplanmıştır. Sınırlama değerlendirmesi sırasında, portal çerçevesinin vince sahip olmadığı ve esnek olmayan, kırılğan parçaları destekleyebilecek bir çatı olduğu varsayımı yapılmıştır (Steel Construction Institute, 2006). Bulgulardan biri, dikey ve yatay sapmalar arasındaki kapasite kullanım farkının, bu sınır durumunda yüksek olmasıdır. Dikey yer değiştirmenin yatay yerdeğiştirmelerden daha kritik olduğunu gösterir. En yüksek düşey yer değiştirme kapasite kullanımı 0.524 iken, yatay yer değiştirme için 0.19 dir. Ek olarak, portal çerçevesinin düşey yerdeğiştirmesi ani sehimde kritik iken, yatay deplasmanı nihai deformasyon ile yönetilir. Bundan dolayı kullanım sınıfları ve nem içerik etkisi düşey deplasman kullanılabilirlik sınır durumunda gözlenemez. Bu sonuçlar tasarımın daha efektif olmasını sağlayacak şekilde ileriki tasarımlarda kullanılabilir.



1. INTRODUCTION

The thesis study of “The Effects of Moisture Content and Load Duration on The Behaviour of Glued Laminated Timber Three-Pinned Portal Frames with Tapered Members” has been investigated in five sections.

The purpose of the thesis has been provided in the first chapter, along with the literature studies. The previous thesis and articles related to glued laminated timber study have been explained.

Section two provides the beginning and the history of glued laminated timber with some of the recent studies completed in the last five years.

The definition and general knowledge of the structural timber material have been provided in the third chapter. Also, the structural behaviour of the material has been detailed with schematic figures and Eurocode coefficients depending on the material classifications.

The system layouts and section details, the mechanical properties of the considered structural system, loading and the design details have been provided in section four. The design is carried out mainly according to Eurocode. The study details have been provided with AutoCAD drawings, flowcharts and formulas. Additionally, the SAP2000 structural analysis process has been explained, and the govern case’s design outputs are provided with captures from the program.

Finally, the capacity utilisation findings depending on the different variables have been detailed in this section with MS Excel graphs, and their interpretation has been provided in this section.

1.1 Purpose of Thesis

This study aims to understand the effect of the moisture, load duration and the service classes of the glued laminated timber on three pinned portal frame members, along with the glued laminated timber material’s structural behaviour. According to EC5, the duration of the load and the moisture content has an impact on the strength and

stiffness of wood and wood-based elements, and these factors must be considered when designing for mechanical resistance and serviceability. The consequences of variations in the timber's moisture content must be considered while designing (British Standards Institution, 2004)

This study aims to understand these effects better on glued laminated timber three-pinned structures design and improve the usage of this sustainable structural material. To see these effects, this study included a portal frame detailed in section 4.2. The capacity utilisations of GL24, GL28, GL30 and GL32 timber materials are analysed for classes 1, 2, 3 and the load durations of short and medium. Because the most critical case of the considered options is service class 3 and the medium-term load duration, the portal frame is sized at 1 capacity for the most critical case. Then, the capacity utilisation changes aim to observe and interpret the effects.

1.2 Literature Review

For the purpose of this thesis, the previous studies on timber material and timber structures have been searched. The search and the found studies show that the glued laminated structures, including three-pinned frames, and the moisture content of timber material have been worked separately; however, it was never considered for a structure with different materials strength and load duration classes.

The information on researched theses on timber and glued laminated timber material are given below.

“Numerical and experimental studies on mechanical properties of glued laminated timber beams made from European beech wood” is a research paper that used three different combined grade beams, GL40c, GL48c and GL55c, with a total of 4 different beam depths, 200 mm, 400 mm, 600mm and 800 mm. The study has observed the failure orders and locations such as finger joint failure, and local buckling failure with the stress value at the failure point. The result has been compared with the FE-Model, and seen that the modulus of elasticity had high accuracy, whereas the actual beam bending strength did not have between the methods comparisons. Additionally, the study used an electric moisture meter to determine the moisture content. According to results, it has been seen that the actual moisture content is mostly at 8 %, and the

others' values are still between 7% and 11 % moisture content (Ehrhart, Palma, Steiger, & Frangi, 2018) .

Kaplan (2013), the thesis “Seismic Analysis of Traditional Ottoman Timber Mansion and Reinforcement Techniques” is a structural investigation of traditional Ottoman timber structure and specifies the required remediation work. The analysis was carried out in the SAP2000 program in 2-D and 3-D models (Kaplan, 2013)

Olga (2017), “Three-Pinned Glued-Laminated Frame” is mainly focused on the curved glued laminated three-pinned frames design under Eurocodes and building a Mathcad calculation document for this unique frame type (Olga, 2017).

Jockwer (2015), “Structural behaviour of glued laminated timber beams with unreinforced and reinforced notches”; this thesis focuses on the requirements of considering the significant load-carrying capacity reduction in the construction of glued laminated unreinforced notches. Also, the design of reinforced glued laminated timber notched beams and suggestions have been provided with experimental and finite element methods (Jockwer, 2015).

The study “Deformability of Glued Laminated Beams with Combined Reinforcement” was carried out under experimental works to increase the rigidity and durability of glued laminated timber under two different load durations with reinforcement in the tension and stretch zones of beams (Sobczak-Piastka, et al., 2020).

“Laminated Wood Beams and Their Applications” provides general information on different types of timber material and focuses on glued laminated timber. The production and application developments and requirements of glued timber have been provided in addition to a finite element model of a gable beam for transverse stresses (Ćehić & Omer, 2016).

“The Effect of Span Lengths on the Bending Strength Properties of Glued Laminated Timber Beam” provides a study on the bending strength related to beam span length and carries out test samples. Based on the results, the failure seems to happen mainly at the finger joints. Also, it was observed that the longer span had a higher modulus of elasticity (Kazeroon & Ahmad, 2018).



2. THE USE OF GLUED LAMINATED TIMBER

2.1 The History of Glued Laminated Timber

In 1901, German architect Otto Hetzer received the first patent for the glued laminated construction method in Switzerland, which was a 10 m long straight beam made up of adhesive-bonded laminations (Rhude, 1996). It was followed by his curved beam invention, which glued several layers of laminates that joined insolvably under pressure (TEMTIS, 2008).

The production of glued material was relatively small until the early 1960s, but it has risen steadily since then, owing primarily to developments in the manufacturing process (TEMTIS, 2008). The tree-pinned frames are a common way of using glued laminated timber material for structural purposes. The most commonly used way of these frames has been shown below.

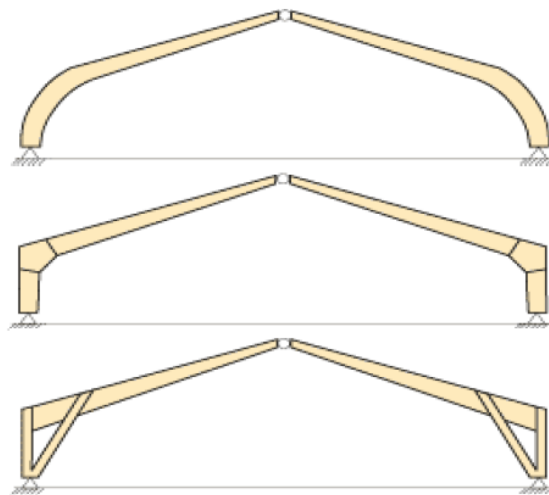


Figure 2.1 : Three-pinned frame examples with curved, jointed and haunches (TEMTIS, 2008).

2.2 The Recent Timber Structures

The use of glued laminated timber has continued to gain momentum, and some current projects from around the globe are presented here.

25 King St Timber Tower, Brisbane, Australia (2018); a mid to high-rise mass timber structure is unique for Australia with a 52 m tall and 10-story building. The primary engineered timber structure comprises cross laminated timber (CLT) walls, roof, and floor plates in European spruce, supported by glued laminated timber columns and beams. One of the rewards that the structure received is the “CTBUH Award of Excellence, Best Tall Building Under 100 Metres, Category Winner, 2021” (Wood Solutions).



Figure 2.2 : 25 King St Timber Tower (Wood Solutions).

The Soto, San Antonio, United States (2019); is a 6-story office building that it's the first mass timber structure in the city. The building has five storeys of timber structure over one storey of concrete. Dowel-laminated timber (DLT) panels were used for the exposed floor and roof decks, supported by a glued laminated timber post and beam frame (Architizer).



Figure 2.3 : The Soto, San Antonio (Architizer).

Melbourne Showgrounds Visitors Centre, Melbourne Australia, (2019); GL18 tapered columns used for the roof portal frame, spanning up to 12.5m portals timbers.



Figure 2.4 : Melbourne Showgrounds Visitors Centre, 2019 (Vicbeam).



3. GLUED LAMINATED TIMBER AND STRUCTURAL IMPORTANCE

In this section, the glued laminated timber has been described; also the structural behaviours and efficiencies have been discussed, along with the importance of these properties on why they are structurally significant.

3.1 The Definition of Glued Laminated Timber

Glued Laminated timber is made of two or more layers of lumber that have been glued together with the grain running parallel to the length of all laminations (Cai & Stark, 2021). Lamination is a modern technic with structural adhesives that provides adequate strength and resistance, also the production of glued laminated timber should meet the requirements of EN 14080 (British Standard Institution, 2013) with layers whose thicknesses differ from 6mm to 45 mm (Ćehić & Omer, 2016).

This form can be made either straight or curved by joining the lumbers, all layers parallel to the longitudinal axis of the member (Cai & Stark, 2021).

For the rated strength and stiffness of timber, characteristic strength values are those with a probability of 5% at a load time of 300 seconds at 65% relative humidity and 20°C temperature. Values with a probability of 5% for ULS (Ultimate Limit State) or average values for SLS (Serviceability Limit State) are defined as characteristic stiffness values under the same test conditions (Bayramoglu & Ozgen).

3.2 The Mechanical behaviours of Timber and Glued Laminated Timber

Timber is commonly accepted as anisotropic material; however, it is usually treated as an orthotropic material in engineering (Mascia & Lahr, 2006). There are three different directions for orthotropic material: parallel to the grain, radial (running from the core to the bark) and tangential (running parallel to the annual rings) (Çaliskan, Meric, & Yunculer, 2019). Due to its orthotropic feature, it can affect the physical properties of wood and its mechanical properties (Mascia & Lahr, 2006). This shows that the elasticity and strength of wood depend on its direction. While the modulus of elasticity

value is highest in the direction parallel to the fibres, it decreases in the radial direction and will decrease in the tangential direction (Mascia & Lahr, 2006). For instance, the compression capacity for a perpendicular loading is way smaller due to crushing the grains, and the tension capacity for a perpendicular tension is assumed to be zero because it only relies on the pectocellulosic component (TEMTIS, 2008). The mentioned direction of loading and the timber strength is explained with a capture from Handbook 1- Timber Structures, First Educational Materials for Designing and Testing of Timber Structures (TEMTIS, 2008), shown below.

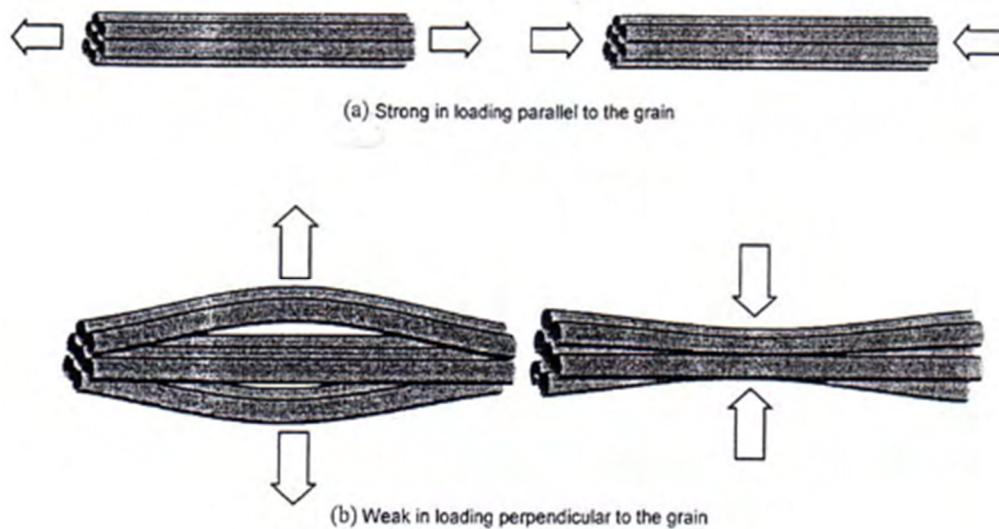


Figure 3.1 : Orthotropic nature of the wood fiber (TEMTIS, 2008).

There are several benefits of using glued laminated timber as a structural member instead of solid timbers and other structural members. Firstly, the glued laminated timber method enables the production of large members from smaller sawn lumber which explains the trend towards small-diameter harvesting (Cai & Stark, 2021). Additionally, the stress-rated glued laminated timber can be manufactured with the lower grade numbers included where the high stresses less likely to happen at the centre of the member, which is called combined glued laminated timber (Cai & Stark, 2021).

It is common knowledge that the amount of moisture affects how stiff and strong lumbers are when bent, and the reduced moisture content is accompanied by an increase in strength and stiffness (Dissemination of IT for the Promotion of Material Science). The relationship between compressive strength and the moisture content of

a timber material in Figure 3.2 below shows the reduction of strength with an increase of the moisture content (Dissemination of IT for the Promotion of Material Science).

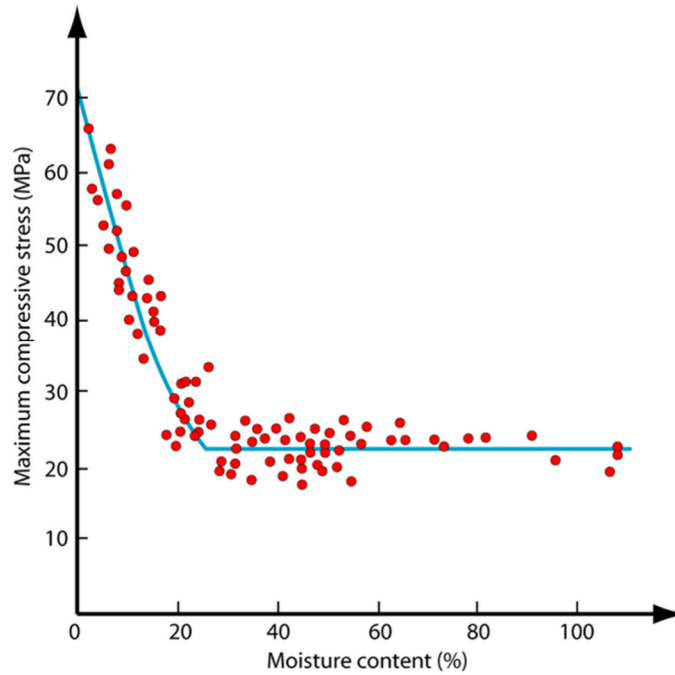


Figure 3.2 : Longitudinal compressive strength vs moisture content (Dissemination of IT for the Promotion of Material Science).

After a certain period, the humidity of the surrounding environment changes, causing the timber material to change in size, either lengthen or shorten (Turkish Standards Institution, 1979). However, as part of manufacturing requirements, the material requires to be seasoned or dried before using it, which allows for minimising the drying defects and maximising design capacities compared to unseasoned timber (Cai & Stark, 2021). The required moisture content of lumbers is $\sim 12 \pm 3\%$, and the components of the glued laminated timber member should not have more than $3\sim 5\%$ moisture difference (Bayramoglu & Ozgen). In addition to the strength and stability benefits of lower moisture content, reducing the moisture content decreases the risk of fungal attack (Bayramoglu & Ozgen; British Standards Institution, 2004).

According to EC5, the modification factor, k_{mod} (a modification factor taking into account the effect of the duration of load and moisture content), provides the consideration of the duration of load effect and moisture content effect on ultimate limit state design. The load duration classes and the different values of k_{mod} are shown in Table 3.1 and Table 3.2 for glued laminated timber and solid timber.

Table 3.1 : Load duration classes (British Standards Institution, 2004).

Load-duration class	Order of accumulated duration of characteristic load	Examples of loading
Permanent	more than 10 years	self-weight
Long-term	6 months – 10 years	storage
Medium-term	1 week – 6 months	imposed floor load, snow
Short-term	less than one week	snow, wind
Instantaneous	-	wind, accidental load

Table 3.2 : k_{mod} values (British Standards Institution, 2004).

Material	Service class	Load-duration class				
		Permanent action	Long term action	Medium term action	Short term action	Instantaneous action
Glued	1	0.60	0.70	0.80	0.90	1.10
Laminated and	2	0.60	0.70	0.80	0.90	1.10
Solid Timber	3	0.50	0.55	0.65	0.70	0.90

The service classes differ from 1 to 3 depending on the material's moisture content and the surrounding environment's humidity. The service class is higher where the moisture content is higher (British Standards Institution, 2004).

The service classes' classification has been shown in Table 3.3 below per EC5 definitions.

Table 3.3 : Service classes' classification.

Service class	Moisture content in the corresponding materials temperature	Relative humidity of the surrounding air for a few weeks per year	Average moisture content in most softwoods
1	20°	> 65 %	≤ 12 %
2	20°	> 85 %	≤ 20 %
3	20°	higher moisture contents than in service class 2.	higher moisture contents than in service class 2.

The k_{def} values provides the consideration of the moisture effect on serviceability limit state design shown in Table 3.4 below for different service classes.

Table 3.4 : k_{def} values (British Standards Institution, 2004).

Material	Service class		
	1	2	3
Glued Laminated & Solid Timber	0.6	0.8	2.0

The R_d (design value of load-carrying capacity) is to be calculated with the formula below with R_k (characteristic value of load-carrying capacity), k_{mod} and γ_m (partial factor for a material property) according to EC5:

$$R_d = k_{mod} \cdot \frac{R_k}{\gamma_m} \quad ; \quad \gamma_m = 1.3 \text{ for Glued laminated timber}$$



4. THE STRUCTURAL ANALYSIS

4.1 The Structural System

This study has chosen the glued laminated timber structural system as a three-pinned portal frame with tapered beams and columns. Timber sections are smaller at the column base and the apex, where the minimal bending moments are. Similarly, the sections are more significant at the beam-column connections where the significant bending moments are. This provides efficiency on material and ease of construction due to lighter structure. In this study, the tapered beams and columns are sections' volume is ~30% less in volume compared to the non-tapered sections.

Firstly, the study was taken under a portal frame with span and spacing of the portal frames are 24 m and 5 m, respectively. Also, the height of the column is 6.48 m, and the height to the apex is 6.78 m. The reason for these agreed values is that the study aimed to work on common usage of framing sizes. The considered frame layout has been detailed in Figure 4.1.

4.1.1 The system layouts and details

The system layout in Figure 4.1 shows half of the symmetric portal frame that is part of this study, which has a 24 m of spanning. The beam sections' lengths are from 0.4 m minimum to 1.225 m maximum, and the columns sections are from 0.5 m minimum to 1.3 m maximum. Additionally, these sections' width is 0.25 m for beams and columns.

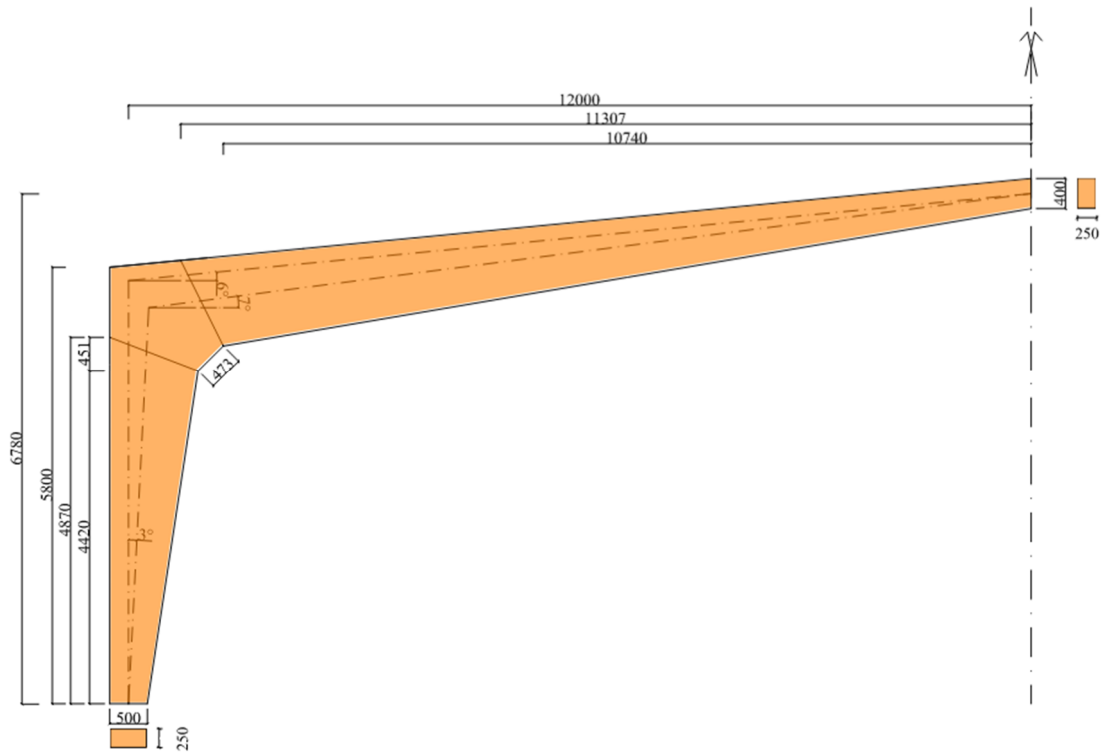


Figure 4.1 : Symmetrical Portal Frame Layout.

One of the typical corner details, finger-jointed moment resistance connection detail, has been provided schematically and in a practical example in Figure 4.2 and Figure 4.3, respectively.

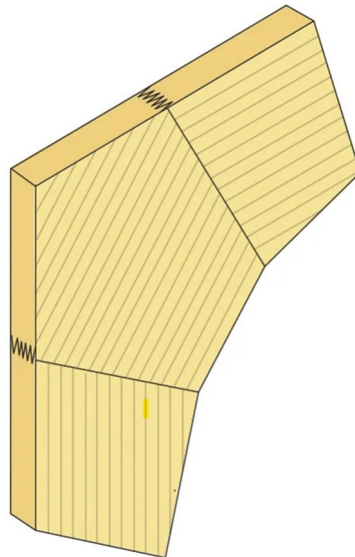


Figure 4.2 : Schematically Finger-jointed Corner Detail (Wood Magazine - Issue 4, 2016).



Figure 4.3 : Finger-jointed Corner Detail (DERIX GROUP).

The considered beam and column sections are provided in Figure 4.4, and 65% lengths are also given because of tapered members' capacity analysis input requirement where the critic section depth and stress occur.

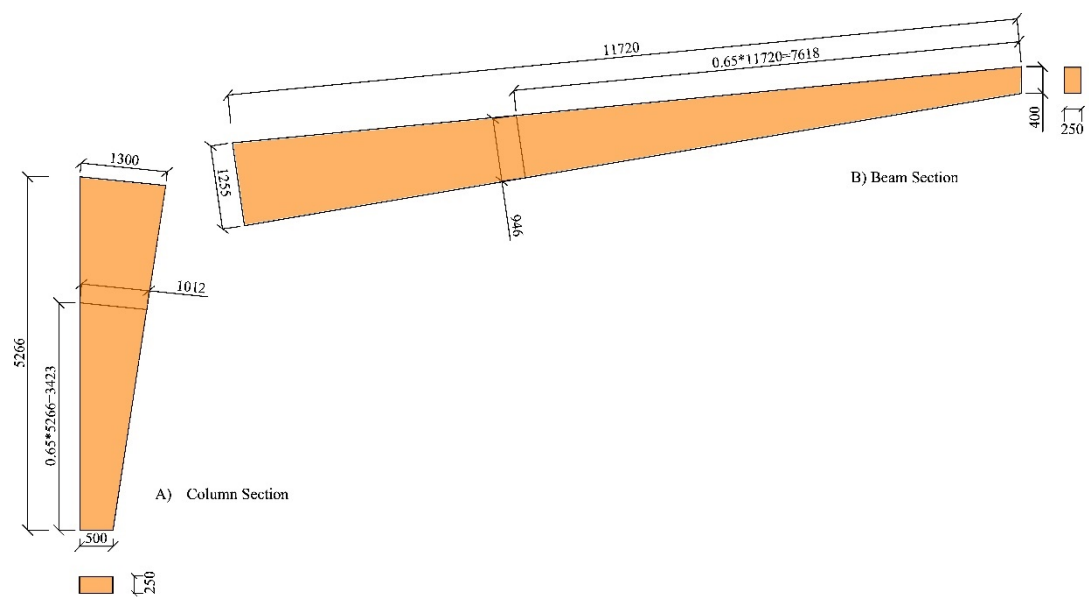


Figure 4.4 : Beam & Column Design Sections.

4.1.2 The structural material properties

The structural material is glued laminated timber, and the study has included four different strength classes, which the characteristic properties of these materials have been provided in Table 4.1 with GL24, GL28, GL30 and GL32 (British Standard Institution, 2013) and C24 (TEMTIS, 2008). The design has used combined glued laminated timber instead of homogenous because the combined technique is more common to the chance of using the lower grade timber in the core.

The solid timber of strength class C24 material properties has been listed below to enable the comparison between GL24 glued laminated timber class. It can be seen that their mechanical properties are almost the identical for these materials; however, solid timber has limitations in size, whereas glued laminated timber can be manufactured from smaller lumbers.

Table 4.1 : Characteristic strength, stiffness and density values (British Standard Institution, 2013; TEMTIS, 2008).

Properties	C24	GL24	GL28	GL30	GL32
$f_{m,k}$ (Bending) (N/mm ²)	24	24	28	30	32
$f_{t,0,k}$ (Lif // tension) (N/mm ²)	14	17	19.5	19.5	19.5
$f_{c,0,k}$ (Lif //compression) (N/mm ²)	21	21.5	24	24.5	24.5
$E_{0,ort}$ (Lif // E modulus) (N/mm ²)	11000	11000	12500	13000	13500
G_{ort} (Shear modulus) (N/mm ²)	690	650	650	650	650
γ (Density) (N/m ³)	3432	3579	3825	3825	3923
$f_{t,90,k}$ (Lif $\perp$ tension)	0.5	0.5	0.5	0.5	0.5
$f_{c,90,k}$ (Lif $\perp$ compression)	2.5	2.5	2.5	2.5	2.5
$E_{90,ort}$ (Lif $\perp$ E modulus)	370	300	300	300	300
$f_{v,k}$ (Shear and torsion) (N/mm ²)	2.5	3.5	3.5	3.5	3.5

Note: The definitions of the properties in the table above are given under symbols section.

4.2 Design Loads

The details of the design loads of dead, wind, snow and seismic have been provided in this section.

4.2.1 Dead load

The dead load has been auto-calculated for the material weight in the SAP2000 program. In addition to the material weight, the roof super-dead load has been considered 0.1 kPa per EC0.

4.2.2 Wind load

The wind load has been calculated per EC1- Part 1.4. The 2D analysis of the portal frame wind loading has been only calculated towards the frame spanning, as shown in Figure 4.5 below.

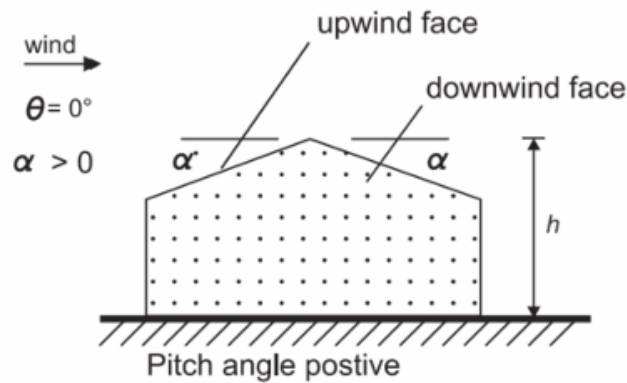


Figure 4.5 : Wind calculation load direction (British Standards Institution, 2005).

The external pressure coefficients (C_{pe}) are to differ based on the zones on the structure per the Eurocode, and these zones are shown in Figure 4.6 below for the wind direction 0 degrees.

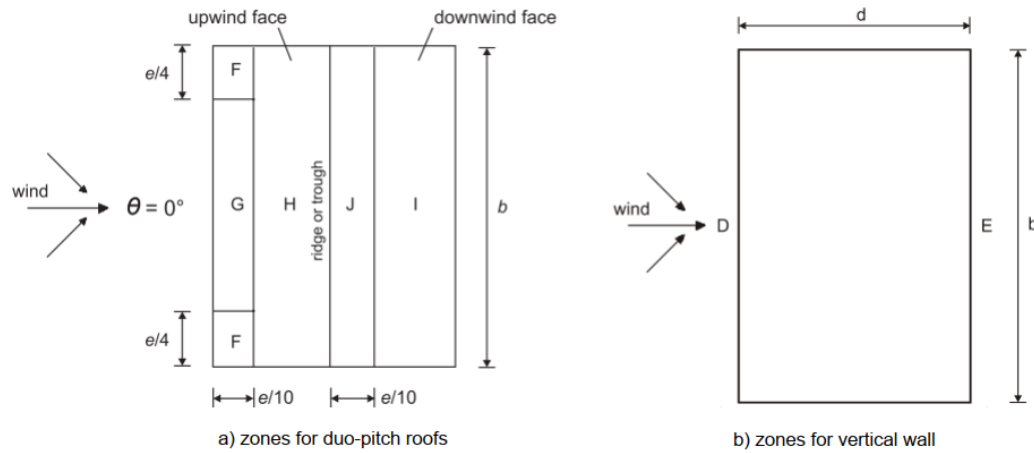


Figure 4.6 : The pressure coefficients zones (British Standards Institution, 2005).

These coefficients for vertical walls and the duo-pitch roofs of the portal frame have been calculated with interpolations and provided in Figure 4.7 below, along with the direction of the wind force. Note that the analysis has disregarded the $C_{pe,1}$ (for loaded areas A of 1 m² and 10 m²) because this study is related to the design of overall load-bearing structure (EC1-Part 1.4, Section 7.2.1, Note 1).

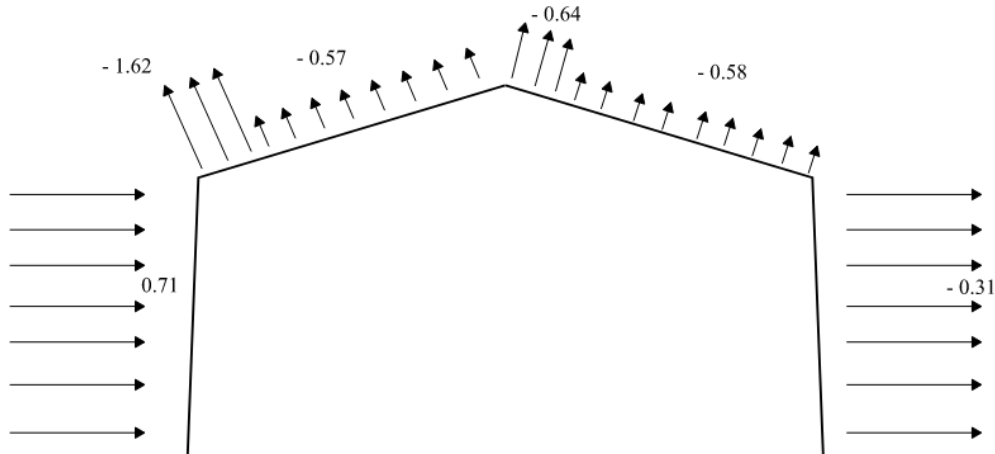


Figure 4.7 : The external pressure coefficient (C_{pe}) values on the frame.

The wind calculations have been provided in Figure 4.8 below as a flow chart according to EC1-4.

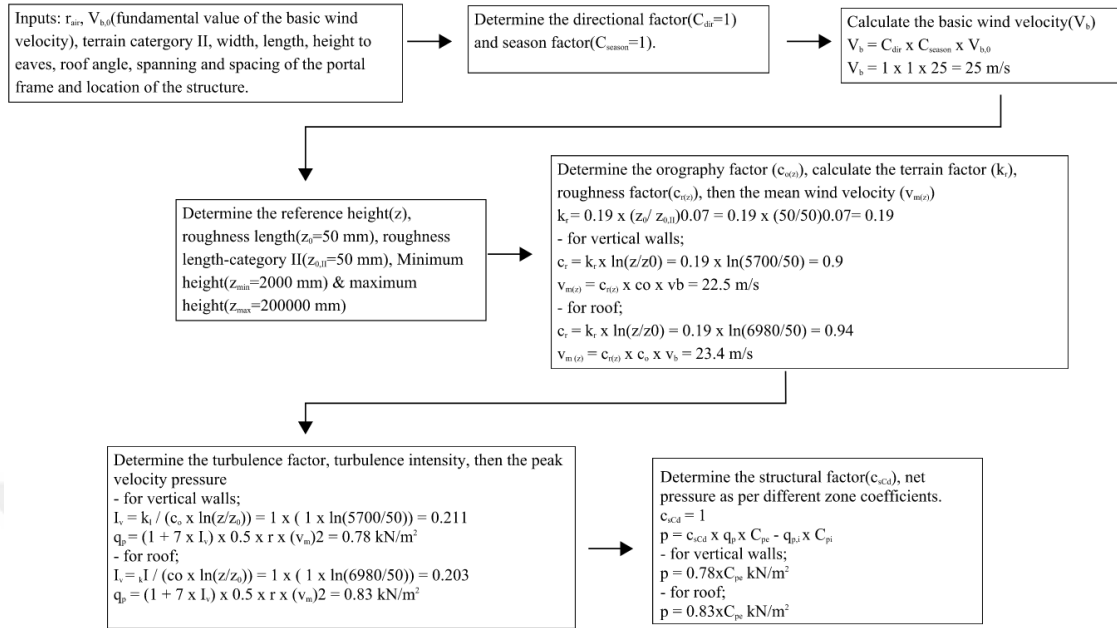


Figure 4.8 : Wind load calculation flow chart.

The net pressures have been applied as a uniform distributed loading on the frame members in SAP2000 software, refer to Figure 4.9 for the representation of the uniform loading.

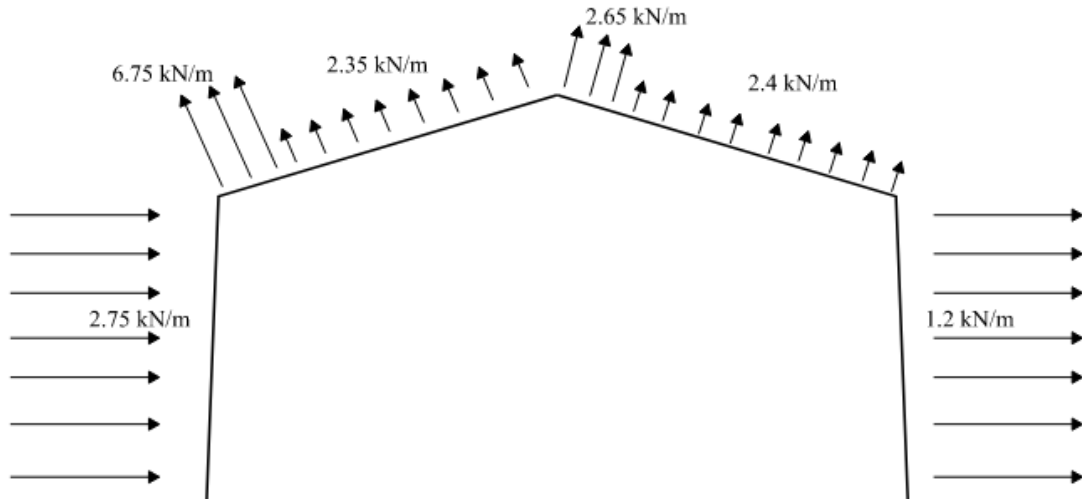


Figure 4.9 : SAP2000 Schematic wind loading on the portal frame.

4.2.3 Snow load

The snow load has been calculated according to EC1-3 ((British Standards Institution, 2013)). The characteristic snow load has been taken from TSE 498 (Turkish Standards Institution, 1997) due to the lack of Turkey's climatic snow region map and altitude-snow load relationships in the EC1-3. Note that the drifted and undrafted snow loading arrangements have been considered according to EC1-3 for the pitched roof, and the undrafted loading arrangement was found to govern the loading scenario.

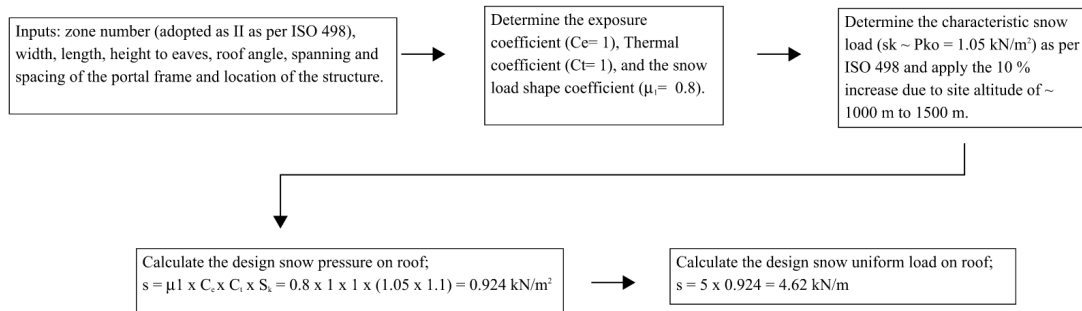


Figure 4.10 : Snow load calculation flow chart.

The snow loading has been applied as a uniform distributed loading on the roof members in SAP2000 software, refer to Figure 4.11.

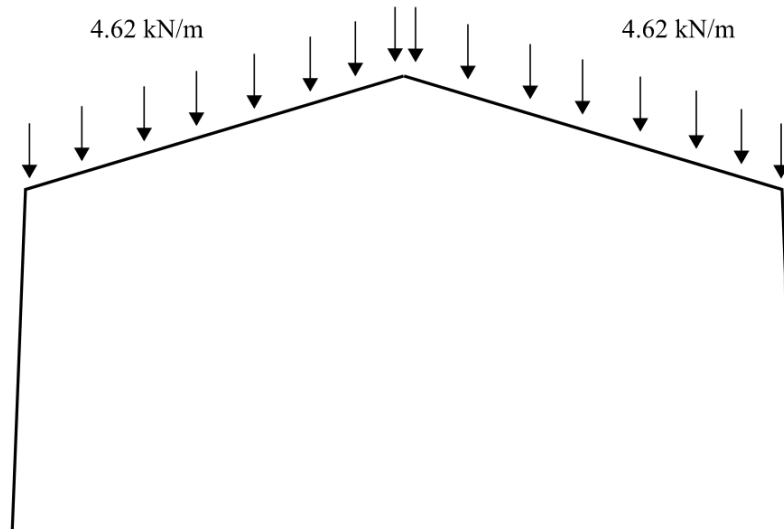


Figure 4.11 : SAP200 Schematic Snow loading on the roof members.

4.2.4 Seismic load

The seismic load has been carried out according to EC8 (British Standards Institution, 2004). The ground and seismic zone parameters have been chosen according to the structure's location in Turkey Marmara region. The ground type is taken C, whose stratigraphic profile description is deep deposits of dense or medium-dense sand, gravel or stiff clay. The important class II per EC8 and the importance factor $\gamma_I = 1$. The reference peak ground acceleration on type A ground a_{gR} is taken as $0.4g$ per the Turkey 475 return period seismic map (TMMOB Chamber of Geology Engineers, 2022), therefore the design ground acceleration is calculated as $a_g = \gamma_I \times a_{gR} = 0.4g$ per EC8. The behaviour factor is chosen as 2, considering the medium capacity to dissipate energy (DCM) per EC8 table 8.1.

According to EC8, there are two spectral types: type 1 and type 2. Because type 1 reflects higher local seismicity, such as most in the Mediterranean (The Institution of Structural Engineers, 2010), this analysis has considered the horizontal response spectrum parameters below according to type 1 spectral type for 5 % damping.

Table 4.2 : The horizontal response spectrum parameters (EC8 table 3.2).

Ground type	Type 1 (Higher seismicity)			
	S (soil factor)	TB (sec)	TC (sec)	TD (sec)
C	1.15	0.2	0.6	2.0

The T_1 (fundamental period of vibration) was calculated with two different methods to compare and have a sanity check. Firstly, T_1 is calculated according to the Rayleigh method by the equation below.

$$T_1 = 2\pi \sqrt{\frac{\sum_{i=1}^n (m_i \cdot s_i^2)}{\sum_{i=1}^n (f_i \cdot s_i)}}$$

In order to use this method, a fictitious 1 kN horizontal load is applied at top of the columns in the SAP2000 model, and the horizontal deflection is found as 0.00032 mm at the top of the column. As a result of the Rayleigh formula, the T_1 is equal to 0.227 s. Secondly, with the dead load and the 30 % of the snow participation of total mass

load, the T1 period has been calculated with modal response spectrum analysis in SAP2000, which is found at 0.27 s. It shows that both first periods are between T_B and T_C , so that the elastic response spectrum ($S_e(T)$), spectral displacement (S_{De}), and ordinate of the design spectrum ($S_d(T)$) formulas have been selected for the values between $T_B \leq T \leq T_C$ according to EC8.

$$T_B \leq T \leq T_C : s_{e(T)} = a_g \cdot S \cdot n \cdot 2.5 = 1.15g$$

$$T_B \leq T \leq T_C : s_{De(T)} = S_{e(T)} \cdot \left(\frac{T}{2\pi}\right)^2 = 0.015 \text{ m}$$

$$d_s = q_d \cdot d_e = 0.03 \text{ m}$$

The limitation of interstory drift has been checked for building that either without structural elements or with non-structural elements fixed in a way that is not interfere the structure (British Standards Institution, 2004), limitations and the check are provided below.

$$\frac{d_r}{h} \leq \frac{0.010}{v} = 0.02 \quad (v = 0.5 \text{ for important classes I and II})$$

$$\frac{d_r}{h} = \frac{d_s}{h} = \frac{0.03}{5.3} = 0.0057 \leq 0.02$$

The lateral seismic load has been calculated per lateral force method, which is along the portal frame spanning (short direction of the structure) in order to determine the base shear force that will be horizontally applied at top of the columns in SAP2000 modelling. The $S_{d(T1)}$ (ordinate of the design spectrum at period T1) and F_b (base shear force) are calculated as below with λ (correction factor) and m (mass of structure).

$$T_B \leq T \leq T_C : s_{d(T1)} = a_g \cdot S \cdot \frac{2.5}{q} = 0.4 \cdot 1.15 \cdot \frac{2.5}{2} = 0.575g$$

$$F_b = \lambda \cdot m \cdot s_{d(T1)} = 35.4 \text{ kN} \quad (\lambda = 1, m = 61.5 \text{ kN})$$

The seismic load has applied equally at top of both columns as shown in Figure 4.12 below.

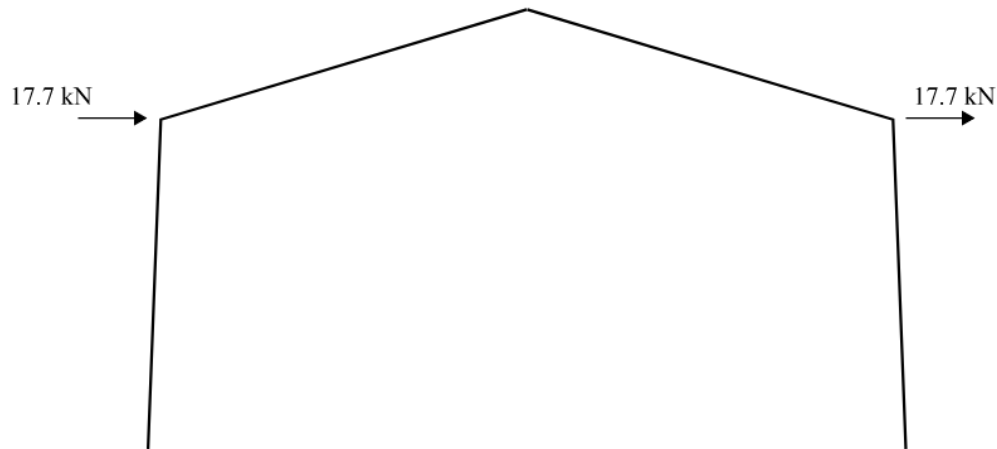


Figure 4.12 : SAP200 Schematic Earthquake loading at top of columns.

4.3 SAP2000 software 2 - D Analysis

The main purpose of the modelling is to determine the governing cases of the portal frame members with critical deflections, moment bending, shear and axial forces based on the Eurocode standard (EC0) combinations (British Standard Institution, 2002). The SAP2000 modelling and the results of the analysis have been detailed in the following sections.

4.3.1 The frame modelling

The portal frame has been modelled as 2-D with tapered sections under non-prismatic section properties per their axis that can be seen in Figure 4.13. Also, to use SAP2000 software for glued laminated timber material, the material properties provided in Table 4.1 has manually assigned for each strength class. The pin connections are provided at the base supports and the apex, and moment connections are provided where the column-beam connection is. A capture of the SAP2000 frame model can be seen with extruded members view in Figure 4.13.

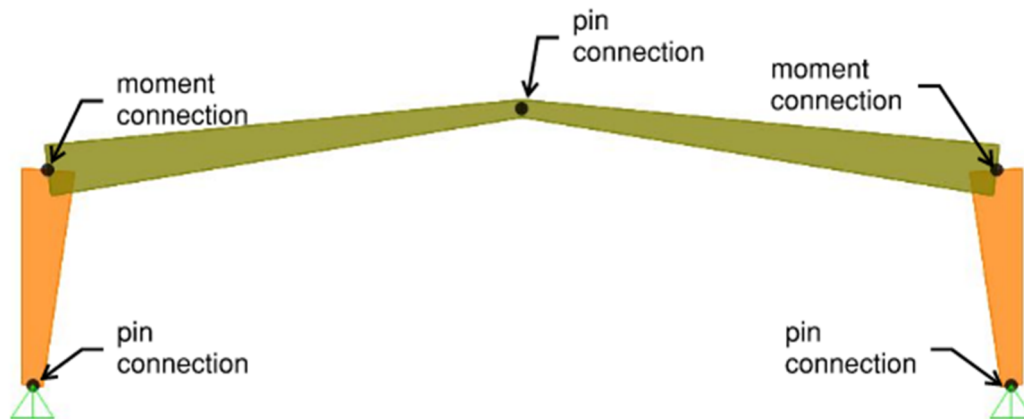


Figure 4.13 : SAP2000 Extruded Members View.

Once the model dimensions and materials are ready, the combinations (TEMTIS, 2008) below and the loading provided in section 4.2 have been assigned to the members.

Table 4.3 : Considered ULS and SLS combinations.

Combination type	Combinations
Ultimate limit state (ULS)	$1.35G + 1.5S$
	$1.35G + 1.5W$
	$1.35G + 1.5S + 0.9W$
	$1.35G + 1.5W + 1.05S$
	$G + E$
	$G + E + 0.2S$
Serviceability limit state (SLS)	$G + W$
	$G + S$
	$G + S + 0.6W$
	$G + W + 0.7S$

4.3.2 The SAP2000 2-D analysis results

According to the analysis results, it is understood that the combinations below are critical for the members' capacity check, which is the performance purpose provided in parenthesis.

- $1.35G + 1.5S$ (for ULS),
- $1.35G + 1.5W$ (for ULS)
- $G + S$ (for SLS)
- $G + W$ (for SLS)
- $G + E$ (for SLS)

Because of repeated analysis requirements due to changes in the service, strength, and load duration classes, the calculation sheet is prepared in TEDDS for Word. The combinations are included in the prepared TEDDS for Word calculation sheet, and a sample of the sheets can be seen in Appendix A. Also, the member analysis has been detailed in section 4.4.

SAP200 element forces have been provided in the Table 4.4 below where the maximum moments are for all cases and combinations are considered.

Table 4.4 SAP2000 outputs according to maximum bending moment.

Cases & Combinations	Beam			Column		
	P (Axial), kN	V (Shear), kN	M (Moment), kNm	P (Axial), kN	V (Shear), kN	M (Moment), kNm
G	-15	-14	-60	-16	12	-60
S	-56	-47	-240	-58	46	-240
W	28	32	149	37	-21	149
E	-6	-12	-142	-15	27	-142
G + S	-70	-61	-301	-73	57	-301
G + W	13	15	87	17	-9	87
G + E + 0.2S	-32	-35	-248	-43	47	-248
G + E	-21	-25	-201	-31	39	-201
G + S + 0.6W	-55	-47	-239	-57	43	-239
G + W + 0.7S	-27	-24	-130	-29	22	-130
1.35G + 1.5S	-103	-90	-437	-109	-84	-437
1.35G + 1.5W	22	30	142	34	16	142
1.35G + 1.5S + 0.9W	-80	-69	-349	-84	64	-349
1.35G + 1.5W + 1.05S	-39	-34	-186	-42	-31	-186

The example of moment bending diagram, shear, and axial force diagrams have been presented below for ultimate limit state combination of $1.35G + 1.5S$ in Figure 4.14, Figure 4.15, Figure 4.16.

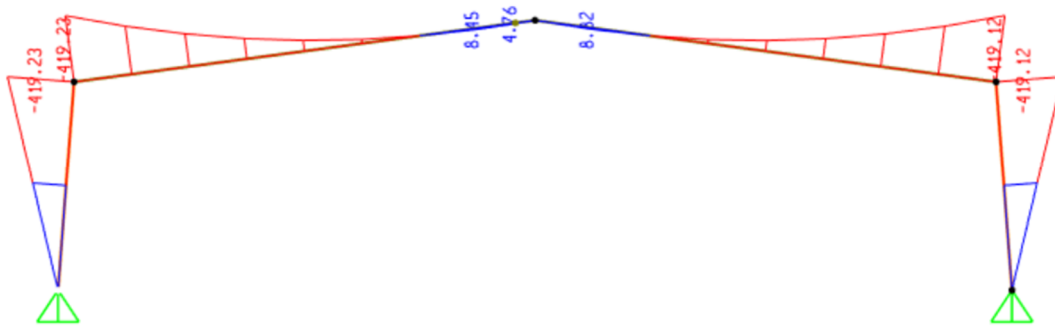


Figure 4.14 : $1.35G + 1.5S$ Moment Bending Diagram, kNm (SAP2000).

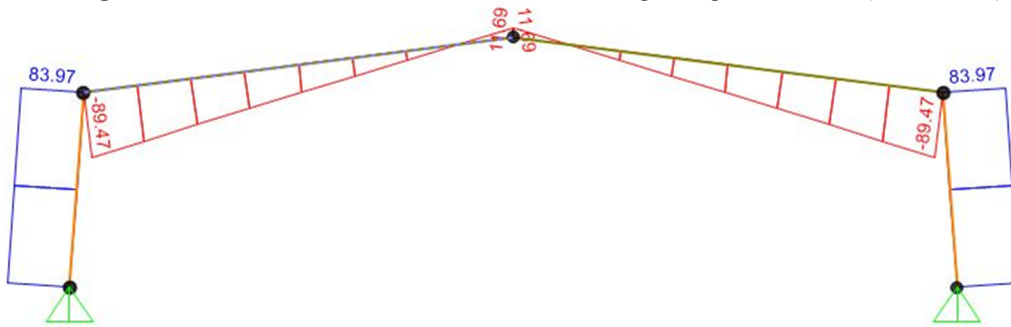


Figure 4.15 : $1.35G + 1.5S$ Shear Force Diagram, kN (SAP2000).

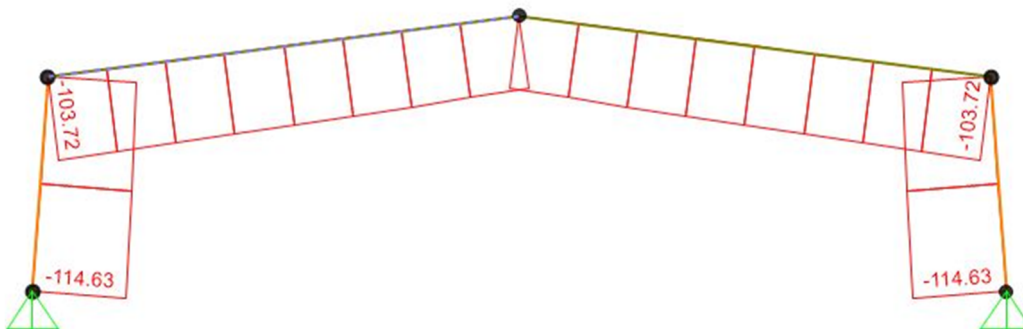


Figure 4.16 : $1.35G + 1.5S$ Axial Force Diagram, kN (SAP2000).

4.4 The Analysis of Sections' and Members' Capacities

There are two basic requirements that the performance of structures should meet: the ultimate limit state and serviceability limit state (TEMTIS, 2008). Ultimate limit state

is related to the safety, and it's related to load bearing capacity (TEMTIS, 2008). In this study, the capacity of the three-pinned portal frame's beams and columns has been analysed according to EC5.

4.4.1 Ultimate state design capacity design methods and formulas

Note that only single-axis bending was considered due to the concept of 2-D analysis, and the portal frame would have stabilised along the length of the structure. The ULS governing formulas are: $(1.35G + 1.5S)$ & $(1.35G + 1.5W)$.

The formulas used in the calculation sheets are presented below and some of the results have been provided from the calculation of GL24, service class 3 and medium-term snow loading. For more detail, Appendix A can be seen.

For bending:

$$\sigma_{m,d} \leq f_{m,d}$$

$$11.47 \text{ N/mm}^2 < 12.48 \text{ N/mm}^2$$

For compression:

$$\sigma_{c,0,d} \approx \sigma_{c,0,d} \leq f_{c,0,d}$$

$$0.44 \text{ N/mm}^2 < 11.18 \text{ N/mm}^2$$

For shear:

$$\tau_d \leq f_{v,d}$$

$$0.38 \text{ N/mm}^2 < 1.82 \text{ N/mm}^2$$

For combined stresses (bending + axial compression) :

$$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}} \right)^2 + \frac{\sigma_{m,d}}{f_{m,d}} \leq 1$$

For tapered beam stress capacity at pitch:

Top // to the grains;

$$\sigma_{m,d} \leq f_{m,d}$$

Top α to the grains;

$$\sigma_{m,a,d} \leq k_{m,a,c} \cdot f_{m,d} ; \quad \sigma_{m,0,d} \approx \sigma_{m,a,d} \approx \sigma_{m,d}$$

$$k_{m,a,c} = \frac{1}{\sqrt{1 + \left(\frac{f_{m,d}}{1.5 \cdot f_{v,d}} \cdot \tan \alpha \right)^2 + \left(\frac{f_{m,d}}{f_{c,90,d}} \cdot \tan^2 \alpha \right)^2}} = 0.909$$

For stability of members:

- For column stability and lateral-torsional stability:

$$\frac{\sigma_{c,0,d}}{k_{c,y} \cdot f_{c,0,d}} + \frac{\sigma_{m,d}}{f_{m,d}} \leq 1$$

$$\frac{0.44}{0.929 \cdot 11.18} + \frac{11.47}{12.48} = 0.813 \leq 1$$

$$\frac{\sigma_{c,0,d}}{k_{c,x} \cdot f_{c,0,d}} + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} \leq 1 ; k_m = 0.7 \text{ for rectangular sections, } 1 \text{ for otherwise}$$

$$\frac{0.44}{0.988 \cdot 11.18} + 0.7 \frac{11.47}{12.48} = 0.811 \leq 1$$

$$k_{c,i} = \frac{1}{k_i + \sqrt{k_i^2 - \lambda_{rel,i}^2}} ; k_{c,y} = 0.929$$

$$k_i = 0.5 \cdot (1 + B_c(\lambda_{rel,i} - 0.3) + \lambda_{rel,i}^2) ; B_c = 0.1 \text{ for GL timber} ; k_y = 0.771$$

$$\lambda_{rel,i} = \frac{\lambda_i}{\pi} \cdot \sqrt{\frac{f_{c,0,k}}{E_{0.05}}} ; \lambda_{rel,y} = 0.709$$

$$\lambda_i = \frac{l_{ef}}{\sqrt{\frac{I_i}{A}}} ; \lambda_y = 45.96$$

$$l_{ef,c} = 2 * hc * \text{sqrt}(1 + 0.4 * c) ; l_{ef,c,y} = 19.139 \text{ m}$$

$$c = \left(\frac{hc}{hb}\right)^3 * \frac{2 * L_b}{L_c} = 5.756$$

- For beam stability due to lateral buckling:

$$\frac{\sigma_{c,d}}{k_{c,z} \cdot f_{c,0,d}} + \left(\frac{\sigma_{m,d}}{k_{crit} \cdot f_{m,d}} \right)^2 \leq 1$$

$$\frac{0.44}{0.262 \cdot 11.18} + \left(\frac{11.47}{1 \cdot 12.48} \right)^2 = 0.99 \leq 1$$

$$k_{crit} = 1, \quad \text{if } \lambda_{rel,m} \leq 0.75;$$

$$k_{crit} = 1.56 - 0.75 \cdot \lambda_{rel,m}, \quad \text{if } 0.75 < \lambda_{rel,m} \leq 1.4$$

$$k_{crit} = \frac{1}{\lambda_{rel,m}^2}, \quad \text{if } 1.4 < \lambda_{rel,m}$$

$$k_{crit} = 1$$

$$\lambda_{rel,m} = \sqrt{\frac{f_{m,k}}{\sigma_{m,crit}}} = 0.672$$

$$\sigma_{m,crit} = \frac{\pi \cdot b^2 \cdot E_{0.05}}{l_{ef} \cdot h} \cdot \sqrt{\frac{G_{ort}}{E_{0,ort}}} = 53.1 \text{ N/mm}^2$$

4.4.2 Serviceability state design capacity design methods and formulas

Per EC5, the deflections have been analysed for initial and the final deflection status, which is the creep included, and there was not any precamber considered. The considered limitation, formulas and the deflections are shown below. The vertical and horizontal deflection limits for portal frames have been considered as below (Steel Construction Institute, 2006).

$$w_{inst} = w_{G,inst} + w_{Q,inst}$$

$$w_{h,inst} = 5.08 \text{ mm} \leq \frac{h_c}{150} = 35.3 \text{ mm}$$

$$w_{v,inst} = 35.9 \text{ mm} \leq \frac{L}{350} = 68.6 \text{ mm}$$

$$w_{fin} = w_{G,inst} + w_{Q,inst} \cdot (1 + k_{def})$$

$$w_{h,fin} = 6.72 \text{ mm} \leq \frac{h_c}{150} = 35.3 \text{ mm}$$

$$w_{v,fin} = 49.7 \text{ mm} \leq \frac{L}{250} = 96 \text{ mm}$$

4.4.3 Results

In this section, the findings of the results have been gathered and plotted with MS Excel charts based on the material class, limit state and it is whether beam or column. The interpretation of these graphs is provided in conclusion.

In order to interpret the results separately for ULS and SLS , the findings are separated into two and provided as graphs in the following sections 5.1 and 5.2.

Additionally, the ULS and SLS capacity utilisations are presented together for GL24 below.

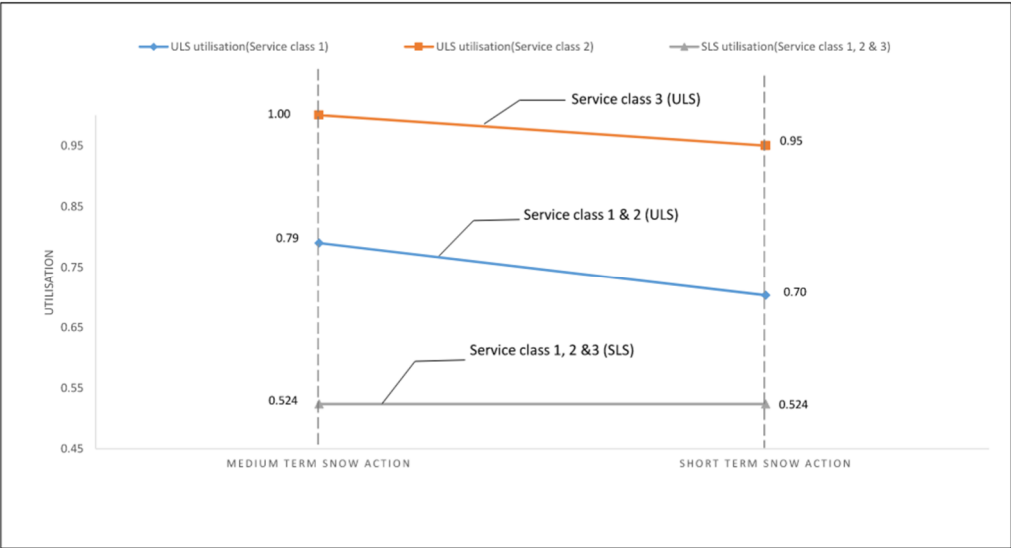


Figure 4.17 : GL24 Beam – ULS & SLS capacity utilisations.

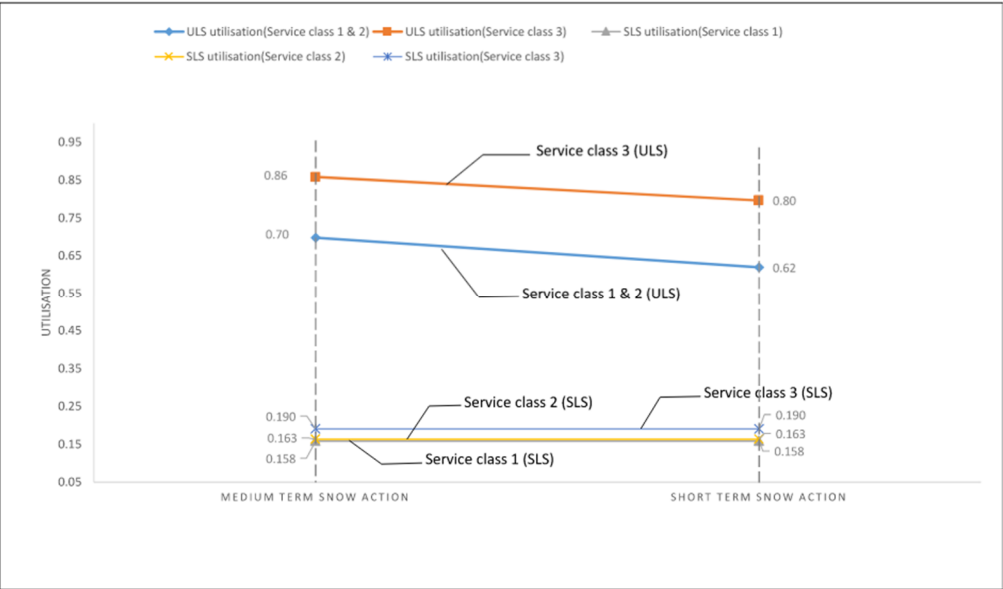


Figure 4.18 : GL24 Column – ULS & SLS capacity utilisations.

4.4.3.1 Ultimate limit state results

The portal frame ULS capacity utilisations have been presented with charts below for different material classes, service classes and the snow loading duration.

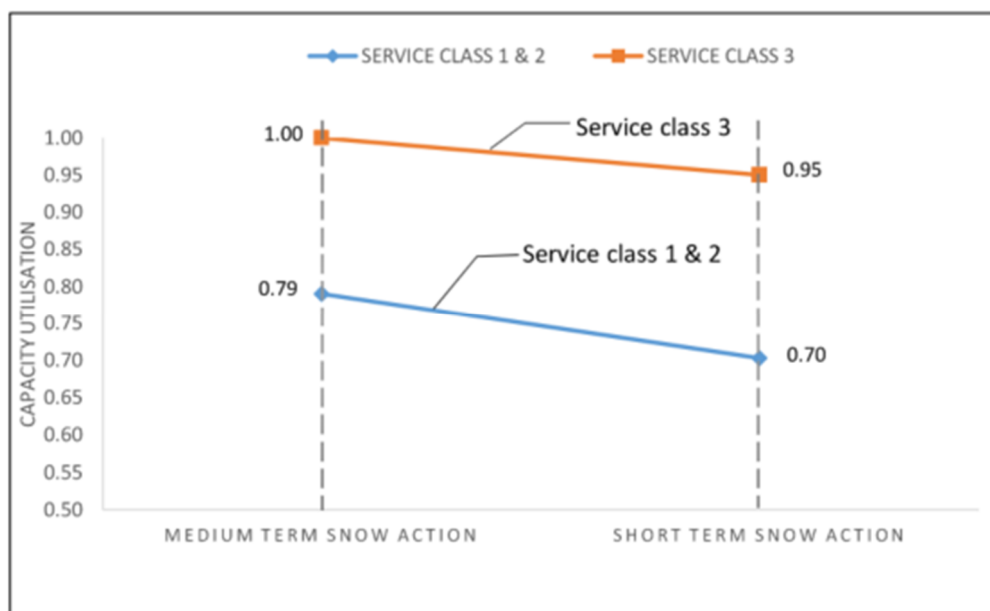


Figure 4.19 : GL24 Beam ULS Capacity Utilisation.

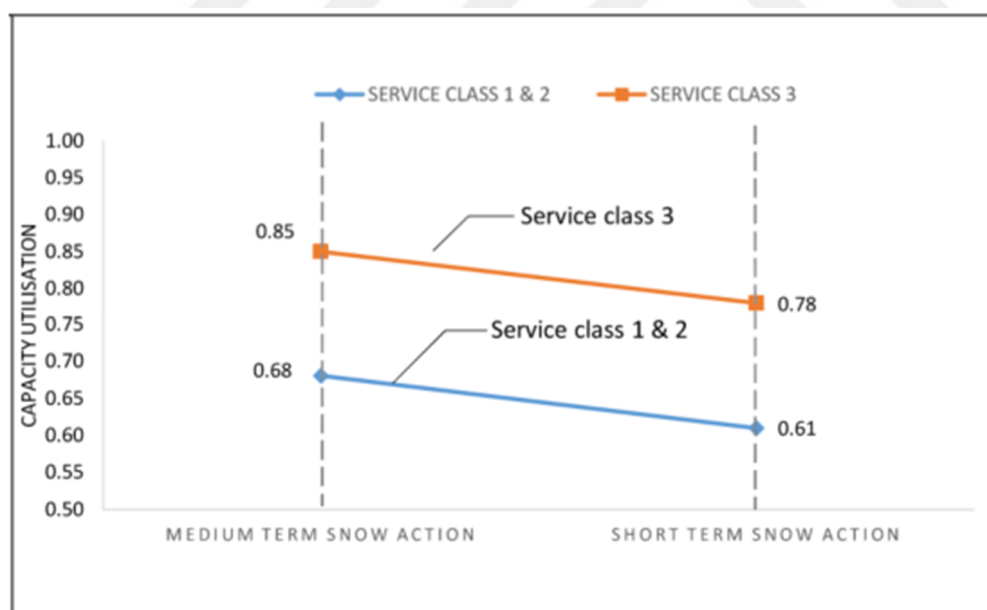


Figure 4.20 : GL28 Beam ULS Capacity Utilisation.

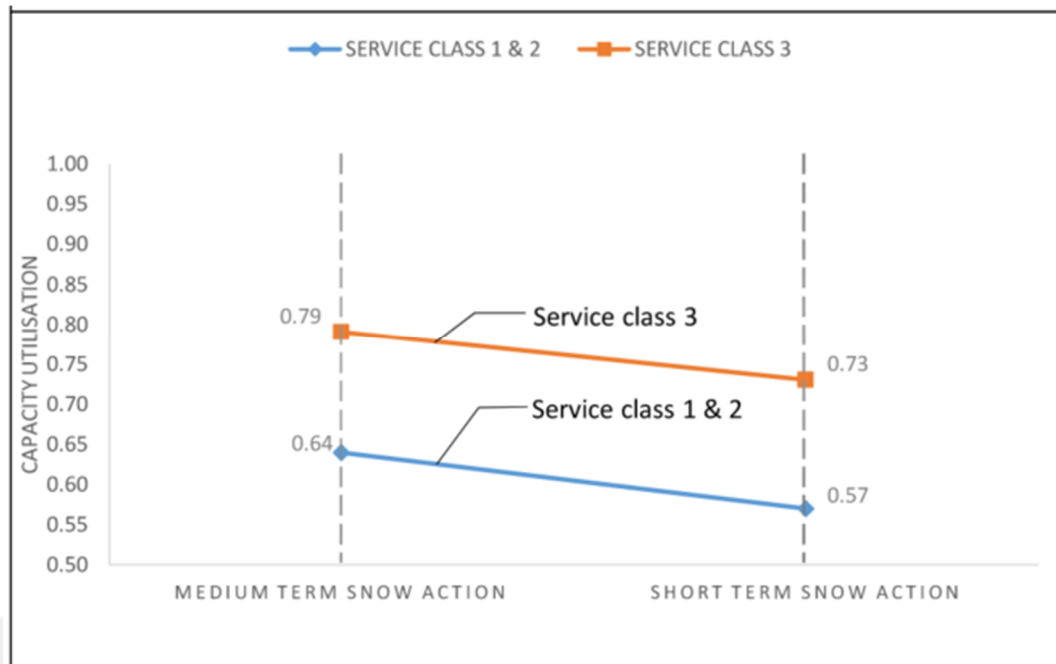


Figure 4.21 : GL30 Beam ULS Capacity Utilisation.

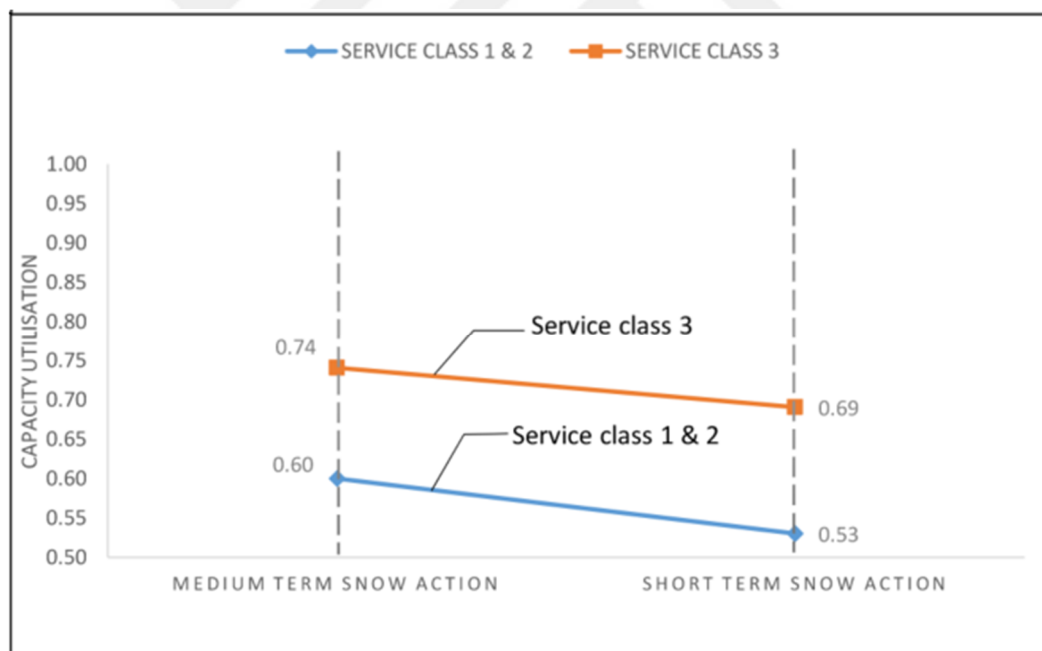


Figure 4.22 : GL32 Beam ULS Capacity Utilisation.

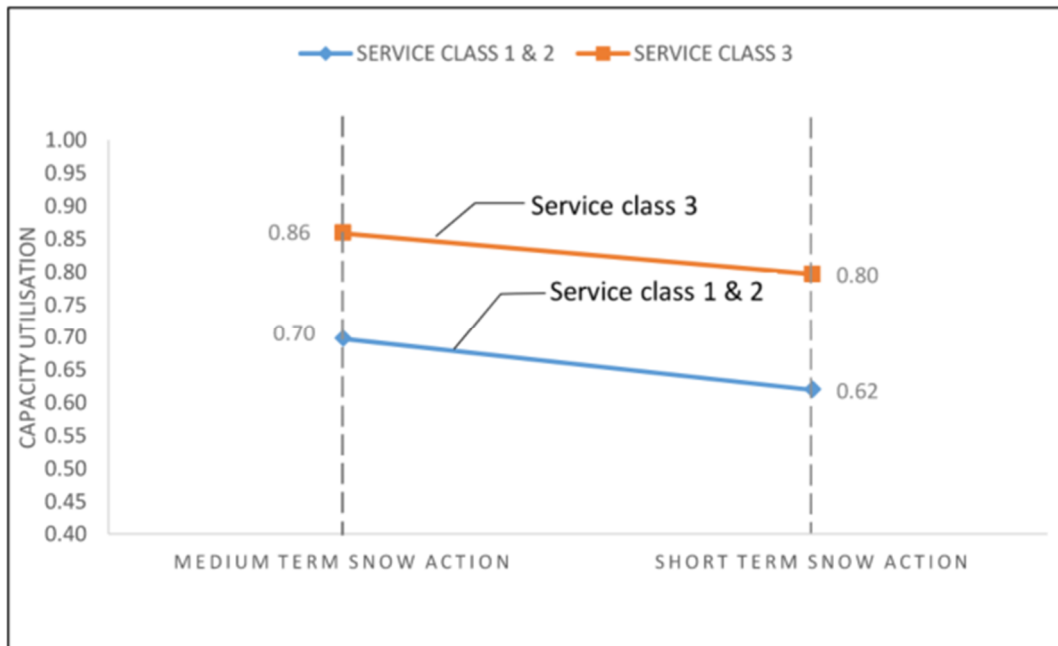


Figure 4.23 : GL24 Column ULS Capacity Utilisation.

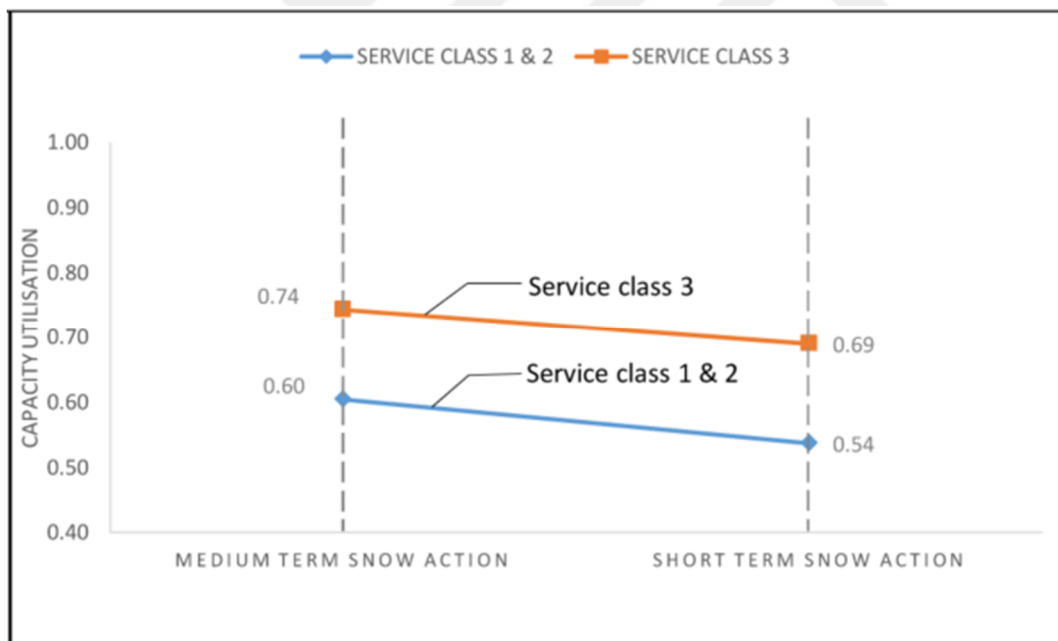


Figure 4.24 : GL28 Column ULS Capacity Utilisation.

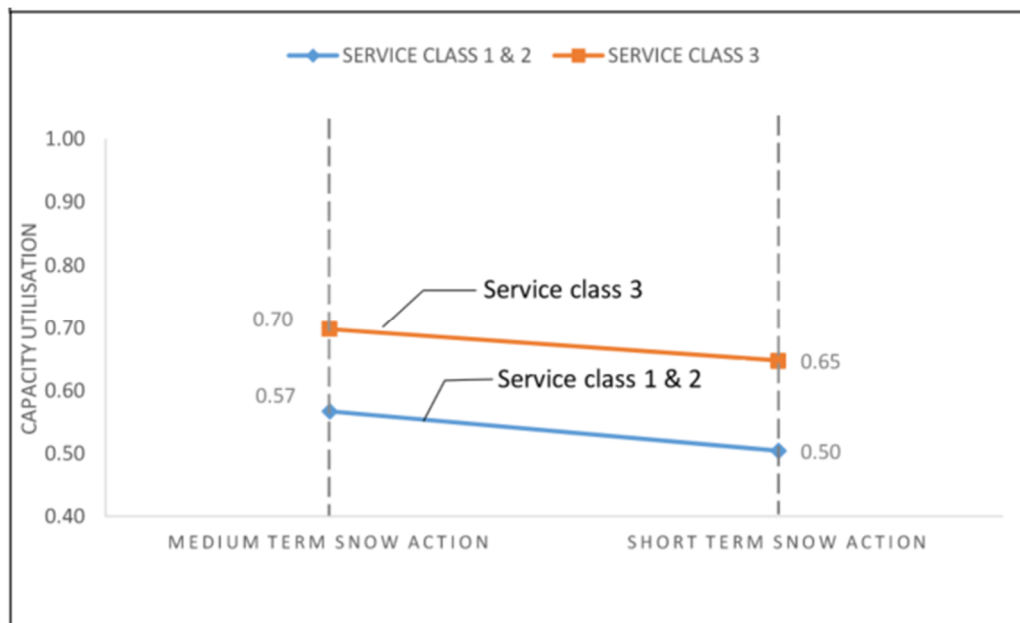


Figure 4.25 : GL30 Column ULS Capacity Utilisation.

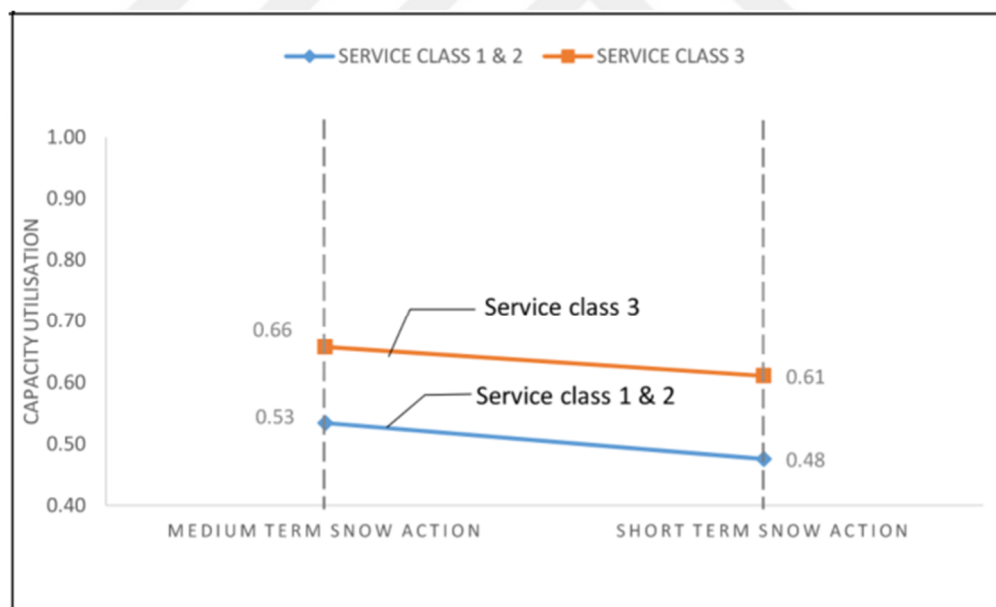


Figure 4.26 : GL32 Column ULS Capacity Utilisation.

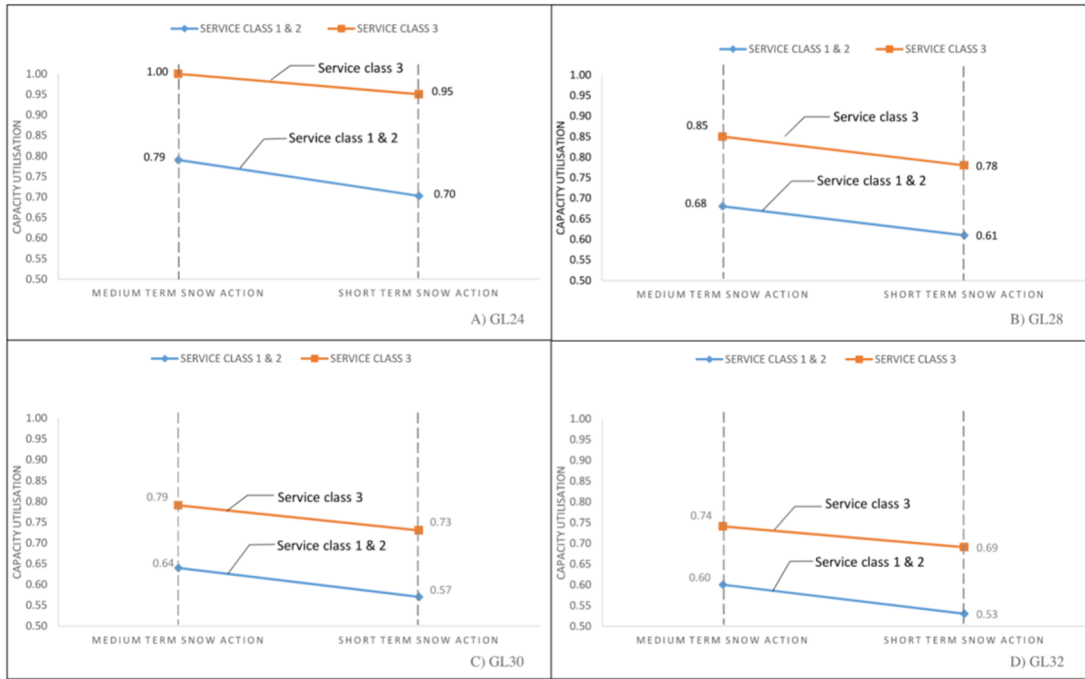


Figure 4.27 : Beam ULS Capacity Utilisations for GL24/28/30&32 Classes.

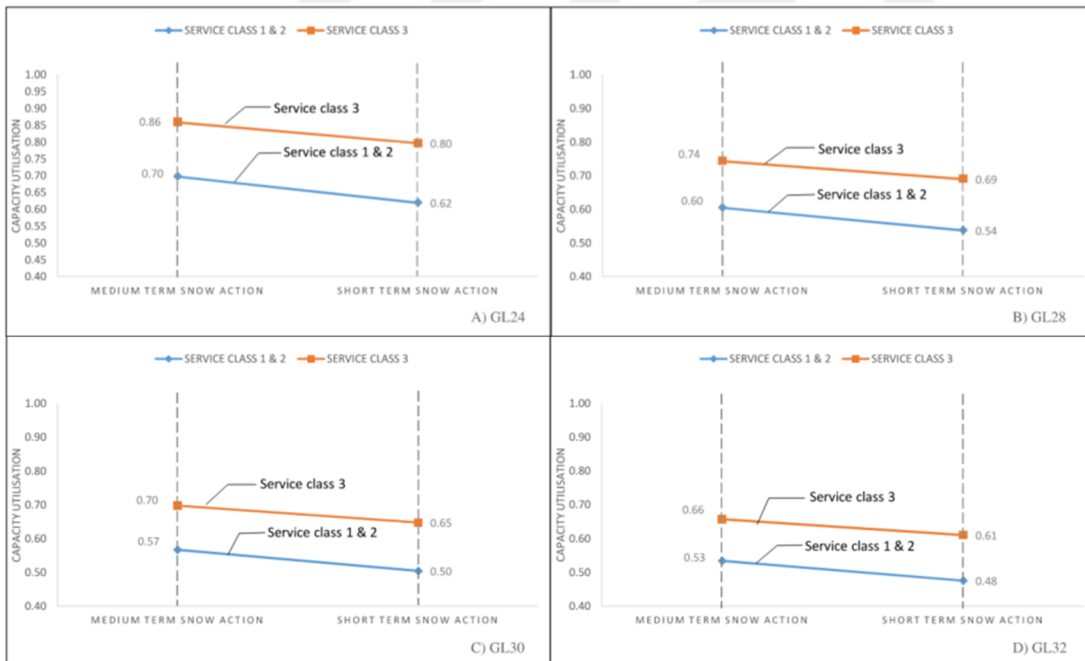


Figure 4.28 : Column ULS Capacity Utilisations for GL24/28/30&32 Classes.

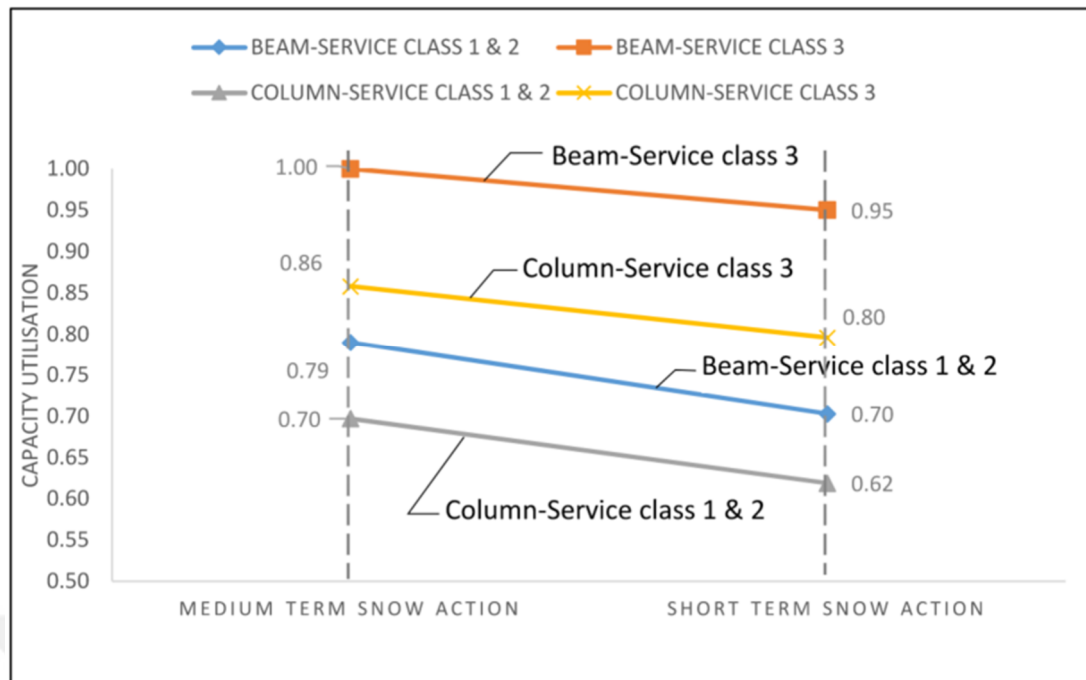


Figure 4.29 : Column and Beam ULS Capacity Utilisations for GL24 Class.

4.4.3.2 Serviceability limit state results

The portal frame SLS have been analysed for the portal frame's vertical and horizontal displacements, and the displacements are shown in Figure 4.30 and Figure 4.31. Additionally, the vertical displacement and horizontal displacement capacity utilisation results have been presented in Figure 4.32 and Figure 4.33 for the displacement limits of 68.6 mm and 35.3 mm, respectively.

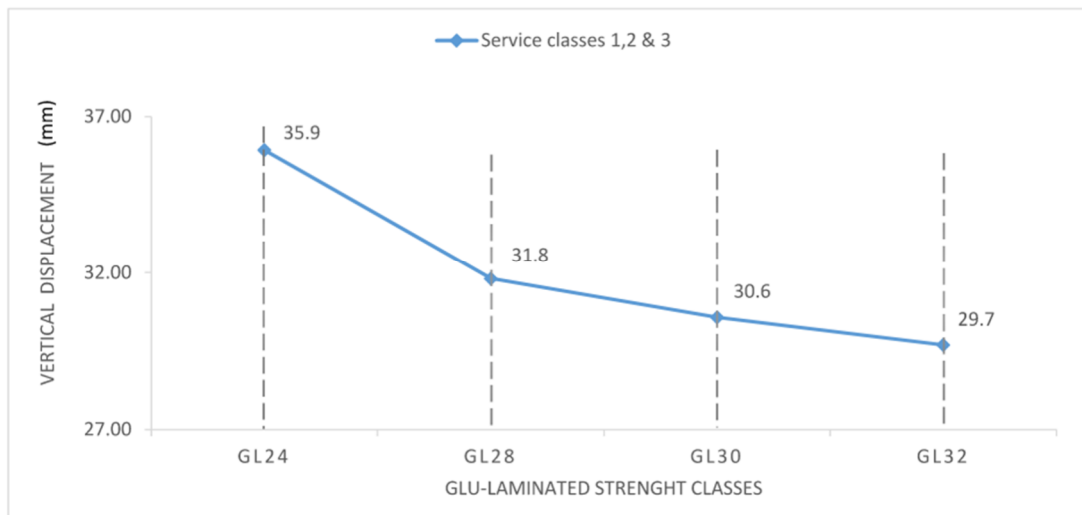


Figure 4.30 : Portal Frame SLS Vertical Displacement, mm.

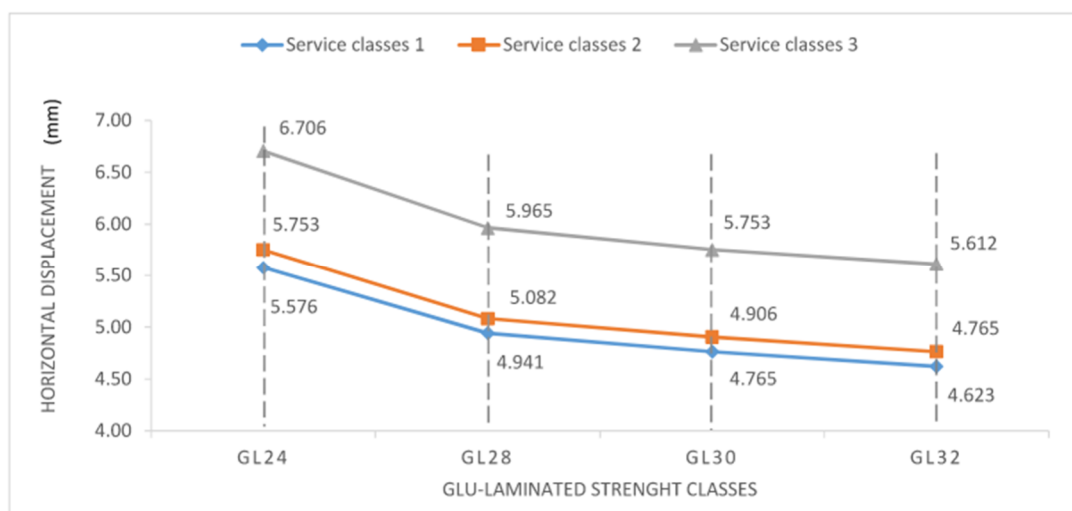


Figure 4.31 : Portal Frame SLS Vertical Displacement, mm.

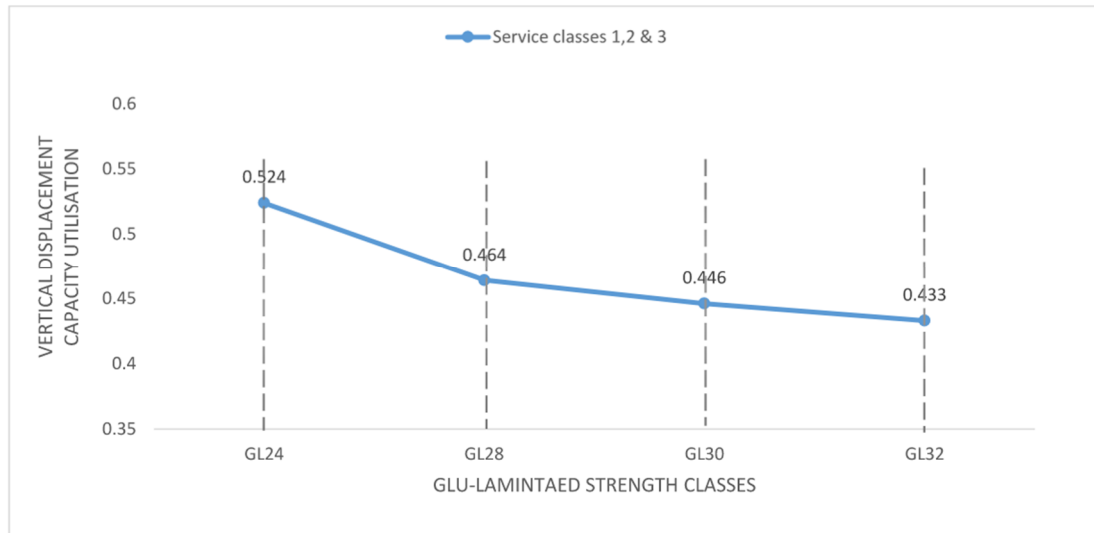


Figure 4.32 : Portal Frame Vertical Displacement SLS Capacity Utilisation.



Figure 4.33 : Portal Frame Horizontal Displacement SLS Capacity Utilisation.



5. CONCLUSION

It shows that ULS governing case is $1.35G + 1.5S$ for both columns and beams, whereas SLS governing case is $G+S$ for beams and $G+W$ for columns.

Also, the seismic analysis shows the more minor effect of earthquakes on timber structures because of being a light structure which is one of the many benefits of timber structures.

Figure 4.17 and Figure 4.18 below show that the strength has more effect than the deflection for all considered parameters.

5.1 The Effect on The Ultimate Limit State Capacity

- The overall governing case is the strength of the beam, as it can be seen in Figure 4.27 with a capacity utilisation of 1.00 where the considerations are; service class 3 and load-duration class of snow is medium term for GL24.
- The capacity utilisation of service classes 1 and 2 are equal due to same k_{mod} coefficient, however, this differs for class 3 due to different load-duration classes of snow, and the effect of the k_{mod} difference can be seen in Figure 4.27.
- The columns govern by the stability capacity whereas the beams govern by stress capacity.
- The reduce of the capacity utilisation usage show that the service class change has a higher effect than the load duration changes for all materials except GL32. The comparison between these two changes is more significant at the beginning, and the percentage difference is getting lesser from GL24 to GL28 and GL30.
- It can be seen that if the strength of GL32 is to be used instead of GL24, ~ %35 of the capacity would be not used.
- The capacity utilisations of the GL24 class beam and column are presented in Figure 4.29 to show that the usage of the column's strength is approximately

10-20 % less than the beam capacity utilisation. It shows that the material class of beams and columns can be selected differently, as the capacity usage different shows in Figure 4.27 and Figure 4.28, to have economical solutions.

5.2 The Effect on The Serviceability Limit State Capacity

The serviceability limit state analysis has been carried out for the vertical and horizontal displacement of the portal frame. Because the deflection criteria do not have the k_{mod} coefficient according to EC5, the load duration effect cannot be seen for the serviceability limit state analysis. The ULS is affected by the k_{def} coefficients based on the service classes are detailed in section 3.2.

- The serviceability limit state capacity utilisation difference between the beam and column shows that the capacity utilisation of the beam governs similar to the ultimate limit state govern member. The highest serviceability capacity utilisation of the beams is 0.524, whereas it is 0.19 for columns.
- The serviceability limit state governs by initial deflection for the beams and final deflection for the columns. As a result of these, as seen in Figure 4.32, the capacity utilisations of the beams are the same regardless of the service classes. So, the initial deflection is not affected by any service classes, therefore, any moisture content. However, the columns' serviceability governs by the final deflection, which is creep included, so the effect of the service classes can be seen in Figure 4.33.

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APPENDICES

APPENDIX A: Calculations



APPENDIX A



CALCULATION SHEET

GL24 - SERVICE CLASS 3 - MEDIUM TERM SNOW LOADING CALCULATION SHEET

Analysis inputs;

Material properties for GL24;

$f_{m_k} = 24 \text{ N/mm}^2$;

$f_{c_0_k} = 21.5 \text{ N/mm}^2$;

$f_{t_0_k} = 17 \text{ N/mm}^2$;

$f_{c_{90_k}} = 2.5 \text{ N/mm}^2$;

$f_{v_k} = 3.5 \text{ N/mm}^2$;

$b = 250 \text{ mm}$;

$\gamma_M = 1.25$;

$\beta_c = 0.1$; (for laminated timber);

$E_{0_{ort}} = 11000 \text{ N/mm}^2$

$E_{0.05} = 5/6 * E_{0_{ort}} = \mathbf{9166.67 \text{ N/mm}^2}$

$G_{ort} = 650 \text{ N/mm}^2$

Combinations	Beam			Column		
	P (Axial), kN	V (Shear), kN	M (Moment), kNm	P (Axial), kN	V (Shear), kN	M (Moment), kNm
1.35G + 1.5S	-103	-90	-437	-109	-84	-437
1.35G + 1.5W	22	30	142	34	16	142
G + E	-21	-25	-201	-31	39	-201

Note: The table above shows the forces based on maximum bending moment values that occur at beam-column connection. The calculations below also include the forces that occur where the section depth is critic (near to apex).

CALCULATION SHEET

Ultimate limit state capacity check for beam;

1-BEAM ULTIMATE LIMIT STATE CAPACITY CHECK FOR POSITIVE MOMENT1

Combination 1,35G+1,5S;

$k_{mod}=0.65$;

$f_{m,d}=(f_{m,k}*k_{mod})/\gamma_M=12.48 \text{ N/mm}^2$;

$f_{c,0,d}=(f_{c,0,k}*k_{mod})/\gamma_M=11.18 \text{ N/mm}^2$;

$f_{t,0,d}=(f_{t,0,k}*k_{mod})/\gamma_M=8.84 \text{ N/mm}^2$;

$f_{c,90,d}=(f_{c,90,k}*k_{mod})/\gamma_M=1.30 \text{ N/mm}^2$;

$f_{v,d}=(f_{v,k}*k_{mod})/\gamma_M=1.82 \text{ N/mm}^2$;

-Column buckling length;

$h_{c,max}=1.3\text{m}$;

$h_{c,min}=0.5\text{m}$;

max column length; $L_c=5.266\text{m}$;

$h_{b,max}=1.225\text{m}$;

$h_{b,min}=0.4\text{m}$;

max beam length; $L_b=11.72\text{m}$;

Length of l positive; $L_{p1}=2.92\text{m}$

$h_{p1}=h_{b,min}+((h_{b,max}-h_{b,min})*L_{p1})/L_b=0.606\text{m}$;

the depth of column $0.65*L_{column}$ distance away from hinge;

$h_{c,0.65}=h_{c,min}+((h_{c,max}-h_{c,min})*0.65*L_c)/L_c=1.020\text{m}$;

the depth of beam at $0.65*L_{beam}$ distance away from hinge;

$h_{b,0.65}=h_{b,min}+((h_{b,max}-h_{b,min})*0.65*L_b)/L_b=0.936\text{m}$;

$h_{b,0.65,ss}=h_{p1}+((h_{b,max}-h_{p1})*0.65*(L_b-L_{p1}))/L_b=1.008\text{m}$;

(the above to check is there any considerable section depth change where the L of $0.65L$ as per the overall beam length or from the negative length only. It shows that the values almost equal)

$I_{c,0.65}=0.25\text{ m}*((h_{c,0.65})^3)/12=0.022\text{ m}^4$;

$I_{b,0.65}=0.25\text{ m}*((h_{b,0.65})^3)/12=0.017\text{ m}^4$;

$c=(h_{c,0.65}/h_{b,0.65})^3*(2*L_b/L_c)=5.756$;

$S_{k,c}=2*L_c*\sqrt{1+0.4*c}=19.139\text{m}$;

CALCULATION SHEET

$N_c=109 \text{ kN};$

$N_b=90 \text{ kN};$

-Beam buckling length;

$S_{k_b} = S_{k_c} \sqrt{((I_{b_0.65} N_c) / (I_{c_0.65} N_b))} = 18.522 \text{ m};$

The angle of beam; $\alpha=5.71;$

$k_{m_c} = 1 / (\sqrt{1 + (f_{m_d} / (1.5 * f_{v_d}) * \tan(\alpha))^2 + (f_{m_d} / f_{c_90_d}) * (\tan(\alpha)^2)^2}) = 0.909;$

Length of l positive; $L_{p1}=2.92 \text{ m};$

$h_{m1}=0.51 \text{ m};$

$h_{a1}=0.4 \text{ m};$

$x_1 = L_{p1} * (1 / (1 + (h_{m1} / h_{a1}))) = 1.28 \text{ m};$

$h_{x1} = 2 * h_{m1} / (1 + h_{m1} / h_{a1}) = 0.45 \text{ m};$

$M_{x_d1}=8.4 \text{ kNm};$

$W_{x1} = b * h_{x1}^2 / 6 = 8375800.02 \text{ mm}^3;$

$\sigma_{m_d1} = M_{x_d1} / W_{x1} = 1.00 \text{ N/mm}^2;$

a)Top side(// to edges);

$R_1 = \sigma_{m_d1} / (f_{m_d}) = 0.080;$

if($\sigma_{m_d1} \leq f_{m_d}$, "OK", "NOT OK")="OK";

b)Bottom side(with α to edges);

$R_2 = \sigma_{m_d1} / (k_{m_c} * f_{m_d}) = 0.088;$

if($\sigma_{m_d1} \leq k_{m_c} * f_{m_d}$, "OK", "NOT OK")="OK";

$R = \max(R_1, R_2) = 0.088;$

CALCULATION SHEET

2-BEAM ULTIMATE LIMIT STATE CAPACITY CHECK FOR POSITIVE MOMENT2

Combination 1,35G+1,5W;

$k_{mod}=0.7$;

$f_{m,d}=(f_{m,k}*k_{mod})/\gamma_M=13.44 \text{ N/mm}^2$;

$f_{c,0,d}=(f_{c,0,k}*k_{mod})/\gamma_M=12.04 \text{ N/mm}^2$;

$f_{t,0,d}=(f_{t,0,k}*k_{mod})/\gamma_M=9.52 \text{ N/mm}^2$;

$f_{c,90,d}=(f_{c,90,k}*k_{mod})/\gamma_M=1.40 \text{ N/mm}^2$;

$f_{v,d}=(f_{v,k}*k_{mod})/\gamma_M=1.96 \text{ N/mm}^2$;

-Column buckling length;

$h_{c,max}=1.3\text{m}$;

$h_{c,min}=0.5\text{m}$;

max column length; $L_c=5.294\text{m}$;

$h_{b,max}=1.225\text{m}$;

$h_{b,min}=0.4\text{m}$;

max beam length; $L_b=11.720\text{m}$;

the depth of column $0.65*L_{column}$ distance away from hinge;

$h_{c,0.65}=h_{c,min}+((h_{c,max}-h_{c,min})*0.65*L_c)/L_c=1.020 \text{ m}$;

the depth of beam at $0.65*L_{beam}$ distance away from hinge;

$h_{b,0.65}=h_{b,min}+((h_{b,max}-h_{b,min})*0.65*L_b)/L_b=0.936 \text{ m}$;

$I_{c,0.65}=0.25 \text{ m}*((h_{c,0.65})^3)/12=0.022 \text{ m}^4$;

$I_{b,0.65}=0.25 \text{ m}*((h_{b,0.65})^3)/12=0.017 \text{ m}^4$;

$c=(h_{c,0.65}/h_{b,0.65})^3*(2*L_b/L_c)=5.725$;

$Sk_c=2*L_c*\sqrt{1+0.4*c}=19.205\text{m}$;

$N_c=34 \text{ kN}$;

$N_b=30 \text{ kN}$;

-Beam buckling length;

$Sk_b=Sk_c*\sqrt{(I_{b,0.65}*N_c)/(I_{c,0.65}*N_b)}=17.980\text{m}$;

CALCULATION SHEET

The angle of beam; $\alpha=5.71$;

$km_{\alpha_c}=1/(\sqrt{1+(f_{m_d}/(1.5*f_{v_d})*\tan(\alpha))^2+(f_{m_d}/f_{c_{90_d}}*(\tan(\alpha)^2)^2)})=0.909$;

Length of l positive; $L_{p1}=11.7m$;

$hm_1=1.225 m$;

$ha_1=0.4 m$;

$x_1=L_{p1}*(1/(1+(hm_1/ha_1)))= 2.88 m$;

$hx_1=2*hm_1/(1+hm_1/ha_1)= 0.60 m$;

$M_{x_d1}=142 kNm$;

$W_{x1}=b*hx_1^2/6=15154240.63 mm^3$;

$\sigma_{m_d1}=M_{x_d1}/W_{x1}=9.37 N/mm^2$;

a)Top side(// to edges);

$R_1=\sigma_{m_d1}/f_{m_d}=0.697$;

if($\sigma_{m_d1} \leq f_{m_d}$, "OK", "NOT OK")="OK";

b)Bottom side(with α to edges);

$R_2=\sigma_{m_d1}/(km_{\alpha_c}*f_{m_d})=0.767$;

if($\sigma_{m_d1} \leq km_{\alpha_c}*f_{m_d}$, "OK", "NOT OK")="OK";

$R=\max(R_1,R_2)=0.767$;

CALCULATION SHEET

3-BEAM ULTIMATE LIMIT STATE CAPACITY CHECK FOR NEGATIVE MOMENT

;Combination 1,35G+1,5S;

$L_{p1} = \text{GetSectionVar}(2, "L_{p1}") = 2.92 \text{ m};$

$S_{k_b} = \text{GetSectionVar}(2, "S_{k_b}") = 18.52 \text{ m};$

$h_{b_0.65} = \text{GetSectionVar}(2, "h_{b_0.65}") = 0.94 \text{ m}$

$k_{mod} = 0.65;$

$f_{m_d} = (f_{m_k} * k_{mod}) / \gamma_M = 12.48 \text{ N/mm}^2;$

$f_{c_0_d} = (f_{c_0_k} * k_{mod}) / \gamma_M = 11.18 \text{ N/mm}^2;$

$f_{t_0_d} = (f_{t_0_k} * k_{mod}) / \gamma_M = 8.84 \text{ N/mm}^2;$

$f_{c_90_d} = (f_{c_90_k} * k_{mod}) / \gamma_M = 1.30 \text{ N/mm}^2;$

$f_{v_d} = (f_{v_k} * k_{mod}) / \gamma_M = 1.82 \text{ N/mm}^2;$

$l_{ef_z} = L_b - L_{p1} = 8.80 \text{ m};$

$l_{ef_y} = S_{k_b} = 18.52 \text{ m};$

$M_y = 419 \text{ kNm};$

$N = 103 \text{ kN};$

$I_z = (h_{b_0.65} * b^3) / 12 = 1219075520.83 \text{ mm}^4;$

$I_y = (h_{b_0.65}^3 * b) / 12 = 17097564656.58 \text{ mm}^4;$

$A = b * h_{b_0.65} = 234062.50 \text{ mm}^2;$

$i_z = \sqrt{I_z / A} = 72.17 \text{ mm};$

$i_y = \sqrt{I_y / A} = 270.27 \text{ mm};$

$W_y = (b * h_{b_0.65}^2) / 6 = 36523502.60 \text{ mm}^3;$

$\sigma_{m_y_d} = M_y / W_y = 11.47 \text{ N/mm}^2;$

$\sigma_{c_0_d} = N / A = 0.44 \text{ N/mm}^2;$

$\sigma_{m_kr} = \pi^2 * b^2 * E_{0.05} / l_{ef_z} / h_{b_0.65} * (\sqrt{G_{ort} / E_{0_ort}}) = 53.10 \text{ N/mm}^2;$

$\lambda_m = \sqrt{f_{m_k} / \sigma_{m_kr}} = 0.672;$

$k_{crit} = \text{ifelseif}(\lambda_m \leq 0.75, 1, \lambda_m > 1.4, 1 / \lambda_m^2, 1.56 - 0.75 * \lambda_m) = 1.000;$

$-k_{c_z};$

$\lambda_z = l_{ef_z} / i_z = 121.94 ;$

$\text{if}(\lambda_z < 150, "OK", "NOT OK") = "OK";$

$\lambda_{rel_z} = (\lambda_z / \pi) * \sqrt{f_{c_0_k} / E_{0.05}} = 1.880;$

$k_{z_0.5} = 0.5 * (1 + \beta_c * (\lambda_{rel_z} - 0.3) + \lambda_{rel_z}^2) = 2.346;$

$k_{c_z} = 1 / (k_{z_0.5} + \sqrt{(k_{z_0.5})^2 - \lambda_{rel_z}^2}) = 0.267;$

$\text{if}(k_{c_z} < 1, "OK", "NOT OK") = "OK";$

CALCULATION SHEET

- k_{c_y} ;

$\lambda_y = l_{ef_y} / i_y = \mathbf{68.53}$;

if($\lambda_y < 150$, "OK", "NOT OK") = **"OK"**;

$\lambda_{rel_y} = (\lambda_y / \pi) * \sqrt{f_{c_0_k} / E_{0.05}} = \mathbf{1.056}$;

$k_{y_0.5} = 0.5 * (1 + \beta_c * (\lambda_{rel_y} - 0.3) + \lambda_{rel_y}^2) = \mathbf{1.096}$;

$k_{c_y} = 1 / (k_{y_0.5} + \sqrt{(k_{y_0.5})^2 - \lambda_{rel_y}^2}) = \mathbf{0.721}$;

if($k_{c_y} < 1$, "OK", "NOT OK") = **"OK"**;

-stress check;

$R_1 = (\sigma_{c_0_d} / f_{c_0_d})^2 + (\sigma_{m_y_d} / f_{m_d}) = \mathbf{0.921}$;

if($(\sigma_{c_0_d} / f_{c_0_d})^2 + (\sigma_{m_y_d} / f_{m_d}) < 1$, "OK", "NOT OK") = **"OK"**;

stability check;

$(\sigma_{c_0_d} / (k_{c_z} * f_{c_0_d})) + (\sigma_{m_y_d} / (k_{crit} * f_{m_d}))^2 =$;

if($(\sigma_{c_0_d} / (k_{c_z} * f_{c_0_d})) + (\sigma_{m_y_d} / (k_{crit} * f_{m_d}))^2 < 1$, "OK", "NOT OK") = **"OK"**;

Shear force Check;

Design shear force; $F_{y_d} = 90 \text{ kN}$;

Design shear stress; $\tau_{y_d} = F_{y_d} / (b * h_{b_0.65}) = \mathbf{0.38 \text{ N/mm}^2}$;

Design shear strength; $f_{v_d} = \mathbf{1.820 \text{ N/mm}^2}$;

$\tau_{y_d} / f_{v_d} = \mathbf{0.21}$;;

CALCULATION SHEET

Ultimate limit state capacity check for column;

4-COLUMN ULTIMATE LIMIT STATE CAPACITY CHECK

$L_{p1} = \text{GetSectionVar}(2, "L_{p1}") = 2.92 \text{ m};$

$S_{k_b} = \text{GetSectionVar}(2, "S_{k_b}") = 18.52 \text{ m};$

$S_{k_c} = \text{GetSectionVar}(2, "S_{k_c}") = 19.14 \text{ m};$

$k_{mod} = 0.65;$

$f_{m_d} = (f_{m_k} * k_{mod}) / \gamma_M = 12.48 \text{ N/mm}^2;$

$f_{c_0_d} = (f_{c_0_k} * k_{mod}) / \gamma_M = 11.18 \text{ N/mm}^2;$

$f_{t_0_d} = (f_{t_0_k} * k_{mod}) / \gamma_M = 8.84 \text{ N/mm}^2;$

$f_{c_90_d} = (f_{c_90_k} * k_{mod}) / \gamma_M = 1.30 \text{ N/mm}^2;$

$f_{v_d} = (f_{v_k} * k_{mod}) / \gamma_M = 1.82 \text{ N/mm}^2;$

$M_y = 419 \text{ kNm};$

$N = 103 \text{ kN};$

$I_x = h_{c_0.65} * h_{c_min}^3 / 12 = 10625000000.00 \text{ mm}^4;$

$I_y = h_{c_0.65}^3 * h_{c_min} / 12 = 44217000000.00 \text{ mm}^4;$

$A = b * h_{c_0.65} = 255000.00 \text{ mm}^2;$

$i_x = \sqrt{I_x / A} = 204.12 \text{ mm};$

$i_y = \sqrt{I_y / A} = 416.41 \text{ mm};$

$W_y = b * (h_{c_0.65}^2) / 6 = 43350000.00 \text{ mm}^3;$

$\sigma_{m_y_d} = M_y / W_y = 9.67 \text{ N/mm}^2;$

$\sigma_{c_0_d} = N / A = 0.40 \text{ N/mm}^2;$

$l_{ef_x} = L_c = 5.29 \text{ m};$

$l_{ef_y} = S_{k_c} = 19.14 \text{ m};$

$-k_{c_x};$

$\lambda_x = l_{ef_x} / i_x = 25.94 ;$

$\text{if}(\lambda_x < 150, "OK", "NOT OK") = "OK";$

$\lambda_{rel_x} = (\lambda_x / \pi) * \sqrt{f_{c_0_k} / E_{0.05}} = 0.400;$

CALCULATION SHEET

$$k_x = 0.5 * (1 + \beta_c * (\lambda_{rel_x} - 0.3) + \lambda_{rel_x}^2) = \mathbf{0.585};$$

$$k_c = 1 / (k_x + \sqrt{(k_x)^2 - \lambda_{rel_x}^2}) = \mathbf{0.988};$$

$$\text{if}(k_c < 1, \text{"OK"}, \text{"NOT OK"}) = \mathbf{"OK"};$$

$$-k_c \gamma;$$

$$\lambda_y = l_{ef_y} / i_y = \mathbf{45.96};$$

$$\text{if}(\lambda_y < 150, \text{"OK"}, \text{"NOT OK"}) = \mathbf{"OK"};$$

$$\lambda_{rel_y} = (\lambda_y / \pi) * \sqrt{f_{c_0_k} / E_{0.05}} = \mathbf{0.709};$$

$$k_y = 0.5 * (1 + \beta_c * (\lambda_{rel_y} - 0.3) + \lambda_{rel_y}^2) = \mathbf{0.771};$$

$$k_c = 1 / (k_y + \sqrt{(k_y)^2 - \lambda_{rel_y}^2}) = \mathbf{0.929};$$

$$\text{if}(k_c < 1, \text{"OK"}, \text{"NOT OK"}) = \mathbf{"OK"};$$

-stress check;

$$R1 = (\sigma_{c_0_d} / f_{c_0_d})^2 + (\sigma_{m_y_d} / f_{m_d}) = \mathbf{0.776};$$

$$\text{if}((\sigma_{c_0_d} / f_{c_0_d})^2 + (\sigma_{m_y_d} / f_{m_d}) < 1, \text{"OK"}, \text{"NOT OK"}) = \mathbf{"OK"};$$

stability check;

1);

$$(\sigma_{c_0_d} / (k_c \gamma * f_{c_0_d})) + (\sigma_{m_y_d} / f_{m_d}) = \mathbf{0.813};$$

$$\text{if}((\sigma_{c_0_d} / (k_c \gamma * f_{c_0_d})) + (\sigma_{m_y_d} / f_{m_d}) < 1, \text{"OK"}, \text{"NOT OK"}) = \mathbf{"OK"};$$

2);

$$(\sigma_{c_0_d} / (k_c \gamma * f_{c_0_d})) + k_{crit} * (\sigma_{m_y_d} / f_{m_d}) = \mathbf{0.811};$$

$$\text{if}((\sigma_{c_0_d} / (k_c \gamma * f_{c_0_d})) + k_{crit} * (\sigma_{m_y_d} / f_{m_d}) < 1, \text{"OK"}, \text{"NOT OK"}) = \mathbf{"OK"};$$

$$R2 = \max((\sigma_{c_0_d} / (k_c \gamma * f_{c_0_d})) + (\sigma_{m_y_d} / f_{m_d}), (\sigma_{c_0_d} / (k_c \gamma * f_{c_0_d})) + k_{crit} * (\sigma_{m_y_d} / f_{m_d})) = \mathbf{0.813};$$

Shear force Check;

$$\text{Design shear force; } F_{y_d} = 115 \text{ kN};$$

$$\text{Design shear stress; } \tau_{y_d} = F_{y_d} / (b * h_{c_0.65}) = \mathbf{0.45 \text{ N/mm}^2};$$

$$\text{Design shear strength; } f_{v_d} = \mathbf{1.820 \text{ N/mm}^2};$$

$$\tau_{y_d} / f_{v_d} = \mathbf{0.25};$$

CALCULATION SHEET

5-COLUMN SERVICEABILITY LIMIT STATE CAPACITY CHECK

1-With the worst case G+W ;

-Instant deflection;

$f_{inst1}=0.82$ mm;

$f_{winst1}=4.26$ mm;

$f_{inst1}= f_{inst1}+ f_{winst1}= 5.080$ mm;

$limit1=L_c/150=35.293$ mm;

$ratio1= f_{inst1}/ limit1=0.144$;

$if(ratio1 \leq 1, "OK", "NOT OK")="OK"$;

-final deflection;

$ksi2_w=0$;

$k_{def}=2$;

$f_{gfin1}= f_{inst1}*(1+k_{def})= 2.460$ mm;

$f_{wfin1}= f_{winst1}*(1+ksi2_w*k_{def})= 4.260$ mm;

$f_{fin1}= f_{gfin1}+ f_{wfin1}=6.720$ mm;

$limit2=L_c/150=35.293$ mm;

$ratio2= f_{fin1}/ limit2=0.190$;

$if(ratio2 \leq 1, "OK", "NOT OK")="OK"$;

2-With the worst case G+E ;

-Instant deflection;

$f_{inst2}=0.82$ mm;

$f_{einst2}=0.935$ mm;

$f_{inst2}= f_{inst2}+ f_{einst2}= 1.755$ mm;

$ratio3= f_{inst2}/ limit1=0.050$;

$if(ratio3 \leq 1, "OK", "NOT OK")="OK"$;

-final deflection;

$ksi2_w=0$;

$k_{def}=2$;

$f_{gfin2}= f_{inst2}*(1+k_{def})= 2.460$ mm;

$f_{efin2}= f_{einst2}*(1+ksi2_w*k_{def})= 0.935$ mm;

$f_{fin2}= f_{gfin2}+ f_{efin2}=3.395$ mm;

CALCULATION SHEET

ratio4= ffin2/ limit2=**0.096**;

if(ratio4<=1,"OK","NOT OK")=**"OK"**;

ratioins=max(ratio1,ratio3)=**0.144**;

ratiofin=max(ratio2,ratio4)=**0.190**;



CALCULATION SHEET

6-BEAM SERVICEABILITY LIMIT STATE CAPACITY CHECK

With the worst case G+S ;

$L=24$ m;

$f_{ginst}=6.9$ mm;

$f_{sinst}=29$ mm;

$f_{inst} = f_{ginst} + f_{sinst} = 35.900$ mm;

$limit1 = L/350 = 68.571$ mm;

$ratio1 = f_{inst} / limit1 = 0.524$;

$if(f_{inst} \leq limit1, "OK", "NOT OK") = "OK"$;

-final deflection;

$ksi2_w=0$;

$kdef=2$;

$fgfin = f_{ginst} * (1 + kdef) = 20.700$ mm ;

$fsfin = f_{sinst} * (1 + ksi2_w * kdef) = 29.000$ mm;

$ffin = fgfin + fsfin = 49.700$ mm;

$limit2 = L/250 = 96.000$ mm;

$ratio2 = ffin / limit2 = 0.518$;

$if(ffin \leq limit2, "OK", "NOT OK") = "OK"$;

CALCULATION SHEET

;*Result;

GL24			
;1-BEAM ULTIMATE LIMIT STATE CAPACITY CHECK FOR POSITIVE MOMENT1;	; Combination 1,35G+1,5S;	0.088;	;Rbs=1;
;2-BEAM ULTIMATE LIMIT STATE CAPACITY CHECK FOR POSITIVE MOMENT2;	; Combination 1,35G+1,5W;	0.767;	
;3-BEAM ULTIMATE LIMIT STATE CAPACITY CHECK FOR NEGATIVE MOMENT;	; Combination 1,35G+1,5S;	;Stress check; 0.921;	
		;Stability check; 1;	
;6-BEAM SERVICEABILITY LIMIT STATE CAPACITY CHECK	;G+S;	;Initial; 0.524;	Rbd= 0.524;
		;final; 0.518;	
;4-COLUMN ULTIMATE LIMIT STATE CAPACITY CHECK;	; Combination 1,35G+1,5S;	;Stress check; 0.776;	Rcs= 0.813;
		;Stability check; 0.813;	
;5-COLUMN SERVICEABILITY LIMIT STATE CAPACITY CHECK;	; bigger of G+W;G+E;	;Initial; 0.144;	Rcd= 0.190;
		;final; 0.190;	

;http://unsworks.unsw.edu.au/fapi/datastream/unsworks:73295/SOURCE01?view=true;

CALCULATION SHEET



CURRICULUM VITAE

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- **B.Sc. : 2018, Istanbul Technical University, Faculty of Civil Engineering, Department of Civil Engineering**

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- Golder (WSP Golder), Structural engineer (2020 - Ongoing)
- Sevenhill Engineering, Junior Structural engineer (2019 - 2020)
- A-Teknik Design, Junior Structural engineer (2018 - 2019)