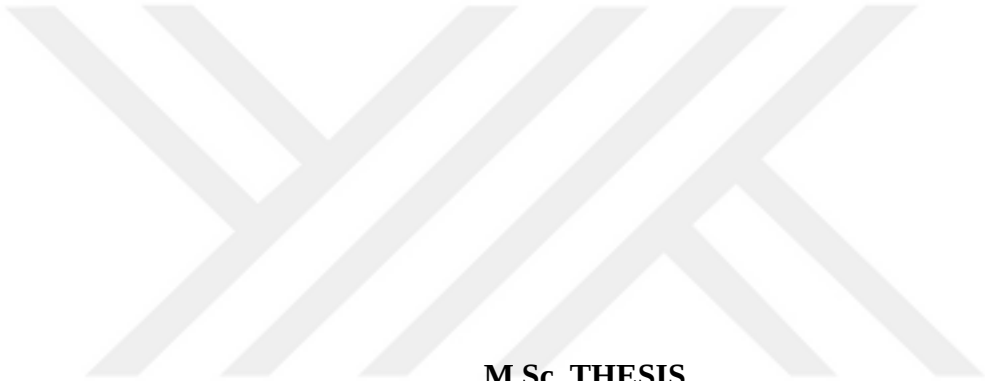


ISTANBUL TECHNICAL UNIVERSITY ★ GRADUATE SCHOOL

**RESPONSE OF A FIXED OFFSHORE PLATFORM TO SEISMIC
EXCITATIONS AND FATIGUE DAMAGE ASSESSMENT**



M.Sc. THESIS

Mahmoud KABBANI

Department of Civil Engineering

Structural Engineering Program

JUNE 2025

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Structural Engineering Program

Thesis Advisor: Prof. Dr. Abdul HAYIR

JUNE 2025

İSTANBUL TEKNİK ÜNİVERSİTESİ ★ LİSANSÜSTÜ EĞİTİM ENSTİTÜSÜ

**SABİT OFFSHORE PLATFORMUNUN SİSMİK UYARILARA KARŞI
TEPKİSİ VE YORGUNLUK HASAR DEĞERLENDİRMESİ**

YÜKSEK LİSANS TEZİ

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To my family,



FOREWORD

I would like to extend my heartfelt gratitude to Prof. Dr. Abdul HAYIR for his invaluable guidance, unwavering support, and kind mentorship throughout the course of my studies. It has been both an honor and a true privilege to work under his supervision. His exceptional character and inspiring dedication made my thesis journey not only enriching but also genuinely enjoyable.

June 2025

Mahmoud KABBANI
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ABBREVIATIONS

FEA	: Finite Element Analysis
PGA	: Peak Ground Acceleration
SCF	: Stress Concentration Factors
CPS	: Cathodic Protection System
API	: American Petroleum Institute
PSD	: power spectrum density
ISO	: International Organization for Standardization
SACS	: Structural Analysis Computer System
TBEC	: Turkish Building Earthquake Code
SRC	: Risk Category
PEER	: Pacific Earthquake Engineering Research Center
S–N	: Stress–Number



SYMBOLS

ρ	: Density
ν	: Poisson's ratio
σ_y	: Yield strength of the steel material
p	: Perimeter of the pile shaft in contact with the soil
F_a	: Allowable axial compressive stress
E	: Modulus of elasticity
K	: Effective length factor
l	: Unbraced length
R	: Radius of gyration
F_{xe}	: Elastic local buckling stress
C	: Critical elastic buckling coefficient.
D	: Outside diameter
t	: Wall thickness
F_b	: The allowable bending stress
f_v	: Shear stress
V	: Shear force,
A	: Cross-sectional area
F_{vt}	: Max. torsional shear stress
M_t	: Applied torque
I_p	: Polar moment of inertia
e	: Relative surface roughness
Rm	: Reynolds number
K	: Keulegan-carpenter number
r	: Current/wave velocity ratio
S_s	: Short-period spectral acceleration
S_1	: 1-Second period spectral acceleration
C_v	: Short-period site coefficient
C_a	: Long-period site coefficient
$S_{a,site}$	: Site-adjusted spectral acceleration
$S_{a,map}$	: Mapped spectral acceleration
V_{s30}	: Average shear wave velocity in the top 30 meters

Rjb	: Joyner-boore distance
Rrup	: Rupture distance
N	: Number of cycles to failure.
C	: Fatigue strength constant.
S	: Applied stress amplitude.
m	: Fatigue exponent.
D	: Cumulative fatigue damage.
n_i	: The number of cycles experienced at stress level i.
N_i	: The number of cycles to failure at the same stress level i.
t	: The thickness for the studied member.
t_{ref}	: The reference thickness



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RESPONSE OF A FIXED OFFSHORE PLATFORM TO SEISMIC EXCITATIONS AND FATIGUE DAMAGE ASSESSMENT

SUMMARY

The expansion of offshore structures (both reinforced concrete and steel) has seen significant growth since the mid-20th century, driven by the increasing global demand for energy and the need to exploit marine energy resources. These structures have been developed at varying sea depths, ranging from shallow coastal waters to deeper offshore locations.

Given the harsh and complex marine environment, offshore platforms are subjected to numerous external forces throughout their Service life. These include wave loads, wind pressures, ocean currents, seismic activity, and fatigue damage. As a result, these platforms must be designed as elevated, resilient, and robust systems capable of withstanding these diverse and dynamic loading conditions.

The cumulative effect of these external forces presents substantial risks to the integrity of structural elements, potentially leading to partial or total failure of the platform. Such structural failures can result in serious environmental hazards and significant economic losses.

This thesis presents a comprehensive static and dynamic analysis of a hypothetical fixed offshore platform situated in the Mediterranean Sea, off the coast of Türkiye. The structural model was developed and analyzed using the SACS V16 finite element software.

The study primarily investigates the effects of seismic and wave-induced loads on the platform and evaluates its structural performance under these combined actions. The objective is to assess the platform's safety, durability, and long-term reliability through detailed numerical simulations, contributing to the broader understanding of offshore structural behavior in challenging marine environments.

The ability of offshore structures to withstand seismic loading and resist fatigue damage is a critical design consideration universally recognized by engineers and researchers. These factors play a pivotal role in the structural assessment and design of offshore platforms, where the intensity of such demands is largely influenced by the platform's geographical setting—particularly when located in seismically active regions or environments subjected to severe and recurrent wave action.

To evaluate the seismic response of the fixed offshore structure, a nonlinear dynamic time history analysis was performed using three ground motion records in accordance with ISO 19901-2:2004 standards. The selected ground motions were carefully chosen based on the criteria outlined in the same standard and were spectrally matched to the target elastic response spectrum with 5% viscous damping using the SeismoMatch software. The time history analysis was conducted employing the implicit time integration method to ensure accurate representation of the structure's dynamic behavior under seismic excitation.

A comprehensive report was conducted to evaluate the stress distribution in the tubular joints and the associated fatigue damage in the fixed offshore structure under the influence of environmental and wave loads from all directions. This assessment contributes to estimating the platform's theoretical Service life. The numerical fatigue analysis was based on the S–N curve methodology in accordance with API standards, employing the spectral wave analysis approach to assess cumulative fatigue damage. The findings offer valuable insights into the structural integrity, safety, and long-term reliability of fixed offshore platforms.



SABİT OFFSHORE PLATFORMUNUN SİSMİK UYARILARA KARŞI TEPKİSİ VE YORGUNLUK HASAR DEĞERLENDİRMESİ

ÖZET

20. yüzyılın ortalarından itibaren, özellikle büyük sanayileşmiş ülkeler tarafından yönlendirilen küresel enerji talebindeki ciddi artış, derin denizlerde enerji kaynaklarının araştırılmasını teşvik etmiştir. Bu gereklilik, farklı derinliklerde ve çeşitli kıyı bölgelerinde açık deniz yapılarının inşasının yaygınlaşmasına yol açmıştır. Bu offshore platformlar, zorlu çevresel koşullara dayanabilecek şekilde yüksek ve sağlam yapılar olarak tasarlanmıştır.

Hizmet ömürleri boyunca, açık deniz yapıları dalga kuvvetleri, rüzgar basıncı ve deniz akıntıları gibi çevresel yüklerin yanı sıra, kendi ağırlıkları ve tekrarlanan döngüsel yüklemelerden kaynaklanan yorulma hasarları gibi çeşitli yüklemelere maruz kalmaktadır. Bu yüklemelere karşı yeterli dayanım gösterememeleri durumunda, kısmi veya tamamen yapısal çökme meydana gelebilir ve bu da ciddi çevresel ve ekonomik sonuçlar doğurabilir.

Bu tez çalışması, Türkiye'nin güney kıyılarına 65.54 metre su derinliğinde yerleştirilmesi varsayılan sabit bir açık deniz platformunun dinamik performansını değerlendirmeyi amaçlamaktadır. Çalışma kapsamında sismik ve çevresel yüklerin etkileri incelenmiştir. Yapısal model, sonlu elemanlar yöntemiyle oluşturulmuş ve analizler SACS V16 yazılımı kullanılarak gerçekleştirilmiştir.

Sabit açık deniz yapısı, eşit düşey aralıklarla yerleştirilmiş dört kattan oluşmaktadır. Çeket yapısının toplam yüksekliği 83.3 metre, tüm bileşenler dahil olmak üzere platformun toplam yüksekliği ise 86 metredir. Platformun bulunduğu bölgedeki su derinliği 65.54 metre olup, yapının toplam ağırlığı 1116 ton olarak hesaplanmıştır.

Yapının sismik tepkisi, ISO 19901-2:2004 standardına uygun olarak seçilmiş üç yer hareketi kaydıyla doğrusal olmayan zaman tanım alanında analiz yöntemi kullanılarak değerlendirilmiştir. Bu sismik kayıtlar, %5 sönüm oranına sahip elastik tepki spektrumuna uygun hale getirilmiş ve Türk deprem yönetmeliğine göre SEISMOMATCH yazılımı aracılığıyla eşleştirilmiştir. Zaman tanım alanı analizleri, örtük zaman integrasyon yöntemi kullanılarak gerçekleştirilmiştir.

Çalışma ayrıca, su üstü ve su altındaki kaynaklı boru bağlantılarının yorulma ömrü değerlendirmesini de içermektedir. Yorulma gerilmelerinin hesaplanması, açık deniz yapılarının işletme süresince sürdürülebilirliğinin sağlanması açısından büyük önem taşımaktadır. Bu amaçla, API standartlarına göre tanımlanan S-N eğrileri yöntemi kullanılmış ve uzun dönemli gerilmelerin değerlendirilmesi ile yorulma ömrünün tahmini için spektral yorulma analizi gerçekleştirilmiştir.

Doğrusal olmayan dinamik zaman tanım alanı analiz sonuçlarına göre aşağıdaki gözlemler elde edilmiştir:

İncelenen yer hareketleri arasında, El Centro depremi yapıda en yüksek taban kesme kuvveti olan 5066.53 kN değerini üretmiş ve bu da yapıya olan etkisinin yüksekliğini göstermiştir. Buna karşılık, Chi-Chi depremi yalnızca 4.188 kN'lık oldukça düşük bir taban kesme kuvveti üretmiş; Kocaeli depremi ise yaklaşık 10.73 kN değerinde bir kuvvetle daha sınırlı etki göstermiştir. Bu farklar, sismik olayların açık deniz yapılarına olan etkisinin ne kadar değişken olabileceğini ortaya koymaktadır.

Sismik tepki analizleri, El Centro yer hareketinin, Kocaeli ve Chi-Chi depremlerine kıyasla yapıda en yüksek yer değiştirme değerlerine neden olduğunu göstermektedir. Bu durumun temel nedeni, rezonans veya sismik tepkinin büyütülmesi gibi olayların meydana gelmesi olup, bu yer hareketine yapının özellikle hassas olduğunu göstermektedir.

Sabit deniz yapısının her katındaki düğümler için maksimum göreceli deplasmanlar hesaplanmış olup, her bir deprem hareketi için üç doğrultuda (X, Y, Z) olmak üzere toplam sekiz nokta değerlendirilmiştir.

Başka bir ifadeyle, El Centro depreminin frekans içeriği yapının doğal frekansı ile büyük ölçüde örtüşmekte, bu da rezonans etkisine neden olarak yapısal tepkinin önemli ölçüde artmasına yol açmaktadır. Ayrıca, El Centro depreminin sismik dalga süresi görece daha uzun olduğundan, yapının titreşim süresince daha fazla sismik enerji birikimine maruz kalmasına neden olmuştur.

Buna karşın, Kocaeli ve Chi-Chi depremleri yüksek enerji seviyelerine sahip olmalarına rağmen, süresel olarak kısa olmaları ve baskın frekans içeriklerinin yapının doğal frekansı ile uyumlu olmaması nedeniyle daha düşük yapısal tepkilere yol açmıştır.

Yorulma gerilmesi çalışması sonuçlarına göre aşağıdaki çıkarımlar yapılmıştır:

Airy doğrusal dalga teorisi, dalga hareketlerinden kaynaklanan gerilme dağılımlarını temsil eden transfer fonksiyonlarının oluşturulması sayesinde yorulma ömrünün doğru tahminine önemli katkı sağlamaktadır.

Kaynaklı bağlantılar için kullanılan S-N eğrisi, 0.016 metre referans kalınlık temelinde tanımlanmıştır. Malzeme kalınlığı bu değeri aştığında, ISO standardına göre kaynaklı bağlantılar (As-welded) için özel olarak geliştirilmiş uygun kalınlık düzeltme formülünün uygulanması gerekmektedir.

Gerilme yoğunlaşma katsayılarının (SCF) belirlenmesinde, boru tipi birleşimlerin geometrik özelliklerini esas alan ve yarı-ampirik bir yöntem sunan Efthymiou yaklaşımı tercih edilmiştir.

Yorulma hasarı, gerilme aralığı dağılımının yorulma dayanımına oranı olarak tanımlanmıştır. Elde edilen sonuçlar, bazı yapısal bağlantıların tahmini yorulma ömrünün 40 yılın altında olduğunu göstermiştir. Bu durum, mevcut açık deniz platformlarının güvenliği ve sürdürülebilirliğini sağlamak adına yorulma dayanımı değerlendirmesinin önemini vurgulamaktadır.

Yapılan analiz sonucunda, tasarlanan hizmet 6mr6n6 ařan toplam 24 adet ana eleman ile diyagonal elemanın birleřim noktası (CHORD–BRACE birleřim noktası) tespit edilmiřtir. Bu noktaların konumları ve yorulma hasarına iliřkin gerilme deęerleri belirlenmiř olup, etkilenen birleřimlerin 6ng6r6len hizmet 6mr6 kriterlerini karřılayacak řekilde yeniden tasarlanması hedeflenmiřtir.





1. INTRODUCTION

1.1 Background

The expansion of industry and the rapid urbanization of modern cities have led to a substantial and continuously increasing demand for energy. To ensure the availability of the necessary energy to drive economic development, efforts have been directed toward exploring new sources, including the discovery of oil and gas fields in offshore marine environments.

The deployment of fixed offshore platforms initially began near coastal areas and gradually extended into shallow waters, eventually reaching greater ocean depths. These fixed platforms are typically constructed from steel or reinforced concrete. With increasing water depths and environmental challenges, the industry has also seen the development and implementation of floating moored structures, which are better suited for deeper marine conditions. Additionally, foundation systems have evolved from traditional shallow-water designs to more advanced anchoring technologies capable of ensuring stability at greater depths.

Offshore platforms exist in a variety of forms and dimensions, depending on their intended operational purpose and the environmental conditions of their locations. Common structural configurations include CPS platforms, monopile platforms, and jacket-type platforms. This study focuses on jacket platforms, which are characterized by their fixed steel framework composed of vertical legs and horizontal and diagonal bracings. These structures are anchored to the seabed using tubular steel members to provide the necessary resistance against environmental loads and the platform's self-weight.

Ensuring that the structure is designed to withstand seismic loading and resist fatigue damage is a fundamental consideration agreed upon by engineers and researchers alike. These factors are particularly crucial in the structural assessment and design of offshore platforms. The severity of such considerations varies significantly based on

the platform's geographical location, especially whether it is situated in a seismically active zone or an area exposed to strong and frequent wave action.

1.2 Historical Review

The creation of the first fixed offshore platform in the early 20th century marked a significant milestone and breakthrough in offshore structural engineering. This platform was constructed to support oil and gas exploration activities in the shallow waters off the coast of the Gulf of Mexico.

The first fixed offshore platform, known as Kerr-McGee, was built in 1947 in the Gulf of Mexico. It served as the true beginning of fixed offshore structures. Since then, hundreds of thousands of fixed offshore platforms have been constructed around the world, at varying water depths, becoming a fundamental component of global energy production.

Although offshore structures have been widely developed since the mid-20th century, the first edition of the API (American Petroleum Institute) offshore design standard, known as API RP 2A, was not released until later. Prior to the release of this guideline, fixed offshore platform design was primarily based on adapting conventional industrial design practices used for onshore steel structures.

Following its initial release, API RP 2A underwent numerous updates and enhancements over the years, evolving into a key reference in the design and construction of fixed offshore platforms to this day.

Consequently, some earlier editions of design standards that were previously adopted have become outdated or no longer reflect the latest developments. When designing most fixed offshore platforms, it is essential to consider their operational lifespan, which typically ranges between 20 to 40 years. For some low-production platforms, the operational life may be as short as 10 years.

During the operational phase, offshore structures are exposed to a variety of environmental and operational factors that can affect their performance and structural integrity. Ensuring structural safety under extreme or unexpected conditions has become a central topic in many recent studies.

Historically, there have been catastrophic progressive collapses of offshore platforms due to the failure of a single structural component. A notable example is the Alexander L. Kielland platform, which sank after the failure of one of its braces located below the waterline. The failure was traced back to small cracks and defects that ultimately led to the platform's collapse and submersion.

Furthermore, hurricanes and severe storms, particularly in coastal and gulf regions where offshore platforms are concentrated, have been major causes of damage or collapse of fixed offshore structures. A significant number of these failures occurred in the Gulf of Mexico, providing real-world examples of structural vulnerability. One such collapsed platform is illustrated in Figure 1.1.



Figure 1.1 : An offshore platform sustained significant damage during a hurricane near louisiana in the gulf of mexico.

Major oil and gas fields are frequently located in regions where tectonic plates converge and fault lines are plates and facilitates the accumulation of hydrocarbons. These tectonic movements also lead to the occurrence of earthquakes.

Since fixed offshore structures present. This is due to the geological structure of the Earth, which drives the movement of these are anchored directly to the seabed using long piles driven to specific depths, they are inherently affected by seismic forces. Therefore, the structural design of such platforms must account for seismic loads to ensure their structural integrity and long-term resilience.

An illustration of the seismic risks in offshore regions can be seen in two significant historical earthquakes: one that struck the Bohai Gulf in 1969 with a magnitude of 7.4 MW, and another in the Gulf of Mexico in 2018, with a magnitude of 8.2 MW. These events highlight the importance of seismic design considerations for offshore platforms located in tectonically active areas.

1.3 Literature Review

In 2021, Elsayed M. Abdel Aal conducted a seismic response analysis of fixed offshore structures using power spectrum density (PSD)-driven inputs. His study focused on the ability of fixed offshore platforms to maintain structural integrity when subjected to seismic loads, and on evaluating the platform's performance under such conditions. The results demonstrated that the structures were capable of withstanding high-intensity seismic loads. Moreover, the research highlighted the significant role of pile foundations in influencing the dynamic response of the structure and in identifying potential failure modes.

In 2014, Qindan Huang studied the ultimate limit state of a fixed offshore structure, with a particular focus on evaluating base shear capacity and seismic demand. The research involved developing a detailed three-dimensional finite element model using the OpenSees platform to assess structural performance under seismic loading.

The findings of the study emphasized the importance of considering the influence of environmental loads (such as waves, wind, and storms) which were found to have a greater impact on the structural design than seismic loads, particularly when the peak ground acceleration (PGA) is less than 0.5g.

Additionally, the study highlighted that the yield strength of steel plays a critical and noticeable role in the variability of the limit state function, making it a key parameter in the structural reliability assessment under combined loading conditions.

The study conducted by Sirous Yasserli focused on the seismic design of subsea spools based on ISO standards. The analysis was performed using seven three-component earthquake ground motion records for a specific offshore site. The pile foundations were modeled at various penetration depths, and the seismic records were modified to match the target design spectrum within the fundamental frequency range of the offshore structure. The study employed the finite element method to accurately assess the structural response under seismic loading conditions.

In 2017, Abdolrahim Taheri conducted a fatigue stress analysis on a real fixed offshore platform using a spectral method for assessing long-term fatigue stresses. The study was based on the S–N approach to estimate the fatigue life of tubular joints within the structure, following the guidelines provided by API standards. The results revealed that a significant number of tubular joints had fatigue lives of less than 40 years, highlighting the critical importance of fatigue life assessment in ensuring the continued operational integrity and safety of offshore platforms throughout their Service life.

In general, these studies - whether focused on the design of offshore structures or conducted on actual implemented platforms - highlight the importance of performing dynamic analyses that encompass both seismic and environmental loads, as well as fatigue stress assessments, to ensure the sustainability and structural safety of fixed offshore platforms.

1.4 Aim of Study

This thesis emphasizes the critical importance of incorporating seismic loads as a fundamental consideration in the design of fixed offshore structures, particularly when they are located in seismically active regions. In addition to seismic effects, the study also accounts for various environmental and operational loads. Special attention is given to the potential for fatigue damage in tubular joint elements due to dynamic forces acting on the structure, especially those resulting from wave and wind actions.

Advanced analytical tools specifically tailored for offshore structures, such as SACS V16, were employed to model and analyze the platform. These tools facilitated the evaluation of structural performance and helped derive results that support the design of a structure capable of withstanding long-term operational and environmental stresses without compromising structural integrity.

The design and analysis conducted in this study were guided by internationally recognized standards, namely API RP 2A-WSD and ISO 19901-2:2004, ensuring compliance with industry best practices and enhancing the reliability of the findings.



2. MODELING OF FIXED OFFSHORE PLATFORM

2.1 Structure Information

In this thesis, a fixed offshore platform was assumed to be located off the Turkish coast in the Mediterranean Sea. The fixed structure consists of four floors positioned at equal vertical intervals. The total height of the jacket structure is 83.3 meters, while the overall height of the offshore platform, including all components, reaches 86 meters. The water depth at the platform's location is 65.54 meters, and the total weight of the structure is 1116 tons.

The horizontal cross-section of the platform is square-shaped, with external dimensions of 8.2 meters at the top level. At the base—where the structure rests on the seabed, the square dimensions expand to 22.16 meters. Horizontal braces are fixed at the ends. The structural joints are designed as rigid connections (Rigid Connections), ensuring high structural stability.

The platform's foundations are designed to penetrate 40 meters below the seabed. The design and analysis were carried out in accordance with the API RP 2A-WSD and ISO 19901-2:2004 standards.

The vertical cross-section of the platform illustrates the vertical bracing members that connect the different levels of the structure.

Both static and dynamic analyses were conducted, including nonlinear dynamic analysis using the SACS software, version 16.

Both the cross-sectional and longitudinal views are illustrated in (Figure 2.1) to provide a comprehensive understanding of the structural configuration and geometry.

- Density (ρ): 7.850 tonnes/m³
- Poisson's Ratio (ν): 0.3
- Young's Modulus (E): 20,000 kN/cm²
- Yield Strength (σ_y): 34.5 kN/cm²

These material properties were used as the basis for the structural design and ensured compliance with offshore engineering standards.

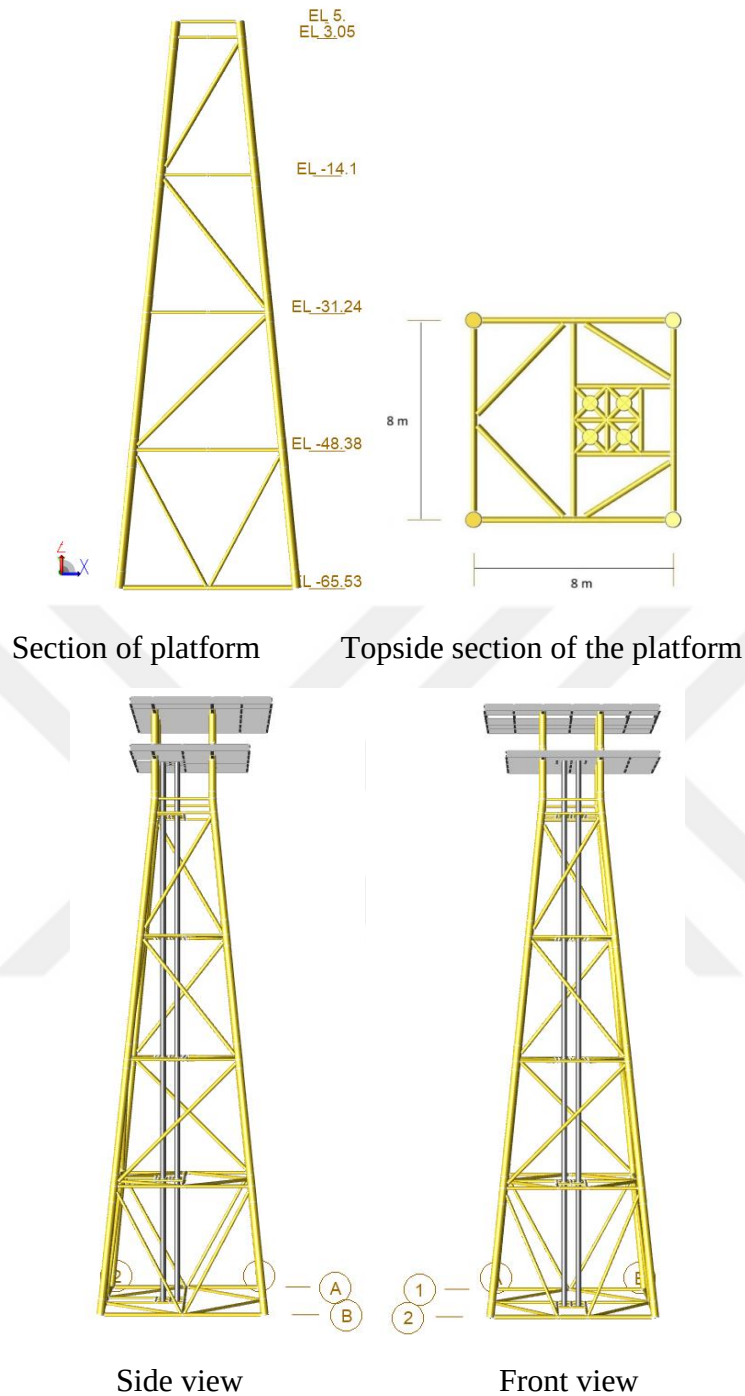


Figure 2.1 : The steel jacket offshore platform from several sections and views.

The Figure 2.2 also highlights the geometric properties of the structural elements, including member diameters and wall thicknesses, which are essential for the structural analysis and design.

According to the API standards adopted in this study, the material properties of the designed structure were defined based on the following parameters.

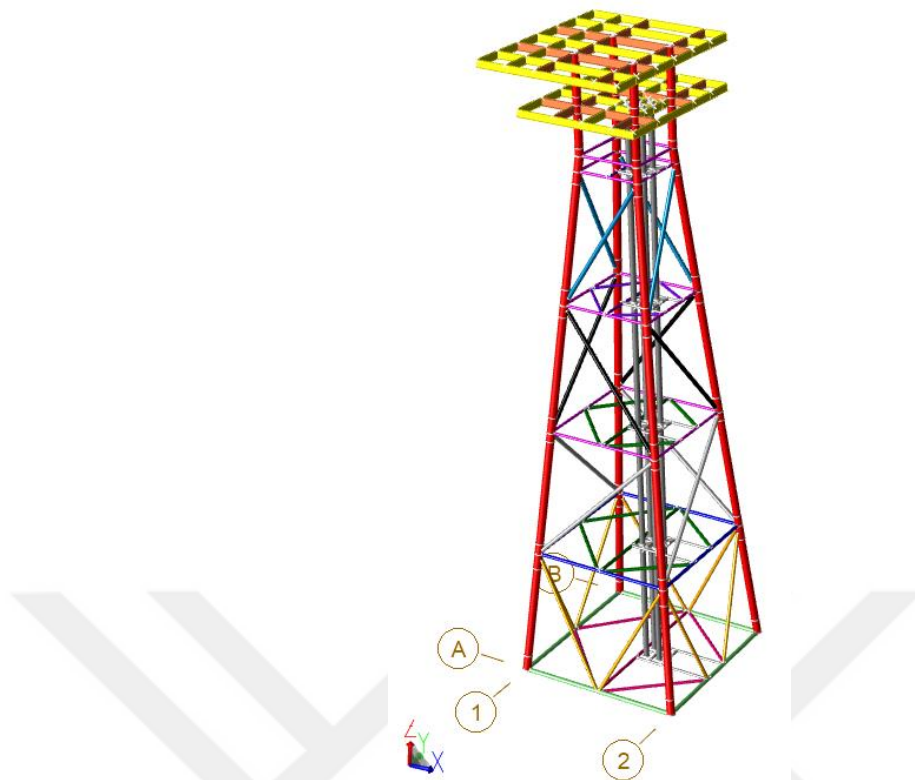


Figure 2.2 : The detailed physical structure of an offshore platform.

The fixed offshore structure comprises 13 structural members, which are categorized in the accompanying Table 2.1.

Table 2.1 : Specifications of structural members of jacket platform.

Member's Group	The Member Thickness (m)	Outside Member Diameter (m)	The Colour
LG1, to LG7	0.0382	1.0669	Red
Bl1	0.0098	0.3239	Grey
Bl3	0.0233	0.45719	Orange
Bl4	0.0129	0.45719	Green
Bl5	0.02539	0.5589	Light Grey
Bl6	0.02031	0.5589	Black
Bl7	0.0129	0.5589	Blue
Con	0.02539	0.7619	Dark Grey
Hl1	0.0159	0.4058	Magenta
Hl3	0.0129	0.45719	Dark Blue
Hl4	0.0098	0.3559	Bright Magenta
Hl5	0.0129	0.3559	Purple
Hl6	0.02539	0.5589	Light Green

2.2 Foundation (Pile/Structure Interaction Analysis)

2.2.1 Soil properties

In this study, the soil profile consists of multiple stratified layers, and its mechanical properties are determined based on site-specific geotechnical investigation and soil test data.

The soil's mechanical properties are assumed to be linear, where plastic deformations are stable and irreversible.

2.2.2 Loads and actions transfer mechanism from pile to soil

The primary function of foundations is to transfer the forces and loads acting on the structure to the underlying soil layers. This is achieved either through side friction along the perimeter walls of the foundations or by direct bearing on competent soil strata.

To determine the value of the frictional resistance per unit area at any depth z can be determined using the following equation (1.1).

$$f(z) = \frac{\Delta Q \cdot z}{p \cdot \Delta z} \quad (1.1)$$

Where

p = Perimeter of the pile shaft in contact with the soil.

2.2.3 P-y curve, t-z curve and q-z curve

Lateral forces acting on the structure arise from environmental conditions such as waves, wind, ocean currents, and seismic ground motions, in addition to the applied loads on the structure, including its self-weight and vertical forces. Therefore, the pile foundation must be designed to resist the combined effect of these loads while ensuring the overall stability of the structure.

2.2.3.1 P-y curve - lateral load-displacement relationship

The p-y curve represents the nonlinear relationship between (y – in cm) and (p – in kN/cm) per 30 cm pile

The accompanying tables present the values used in the study for lateral soil resistance of the designed piles and the corresponding lateral displacements at various depths.

These values highlight the role of piles in maintaining structural stability through their interaction with the surrounding soil. Table 2.2 illustrates the adopted P-Y curve values for analyzing the interaction between the soil and the structural system.

Table 2.2 : P-Y data.

	0.0	5.47	9.22	11.04	16.00	16.40	19.14	21.87	22.27	48.50
Y	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
P	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Y	0.334	0.631	0.331	0.331	0.331	0.383	0.323	0.331	0.673	0.673
P	5.02	16.76	16.76	12.57	12.57	18.27	18.27	25.15	23.27	26.27
Y	0.701	0.496	0.496	0.496	0.496	0.583	0.583	0.496	1.012	1.012
P	5.02	17.0	17.0	12.57	12.57	18.27	18.27	25.15	23.27	26.27
Y	0.74	0.894	0.594	0.594	0.594	0.583	0.709	0.594	1.213	1.213
P	5.02	16.76	13.23	12.57	12.57	18.27	20.86	25.15	23.27	26.27
Y	2.16	2.16	0.709	0.709	0.709	0.709	2.16	2.16	2.16	2.16
P	5.02	17.57	12.67	12.57	12.57	18.27	20.86	25.15	23.27	26.27

Figure 2.3 illustrates the p-y curve for a pile with an outer diameter of 1.1176 meters, a wall thickness of 0.025 meters for the first 10 meters of the pile, and 0.015 meters for the remaining 30 meters of its total length.

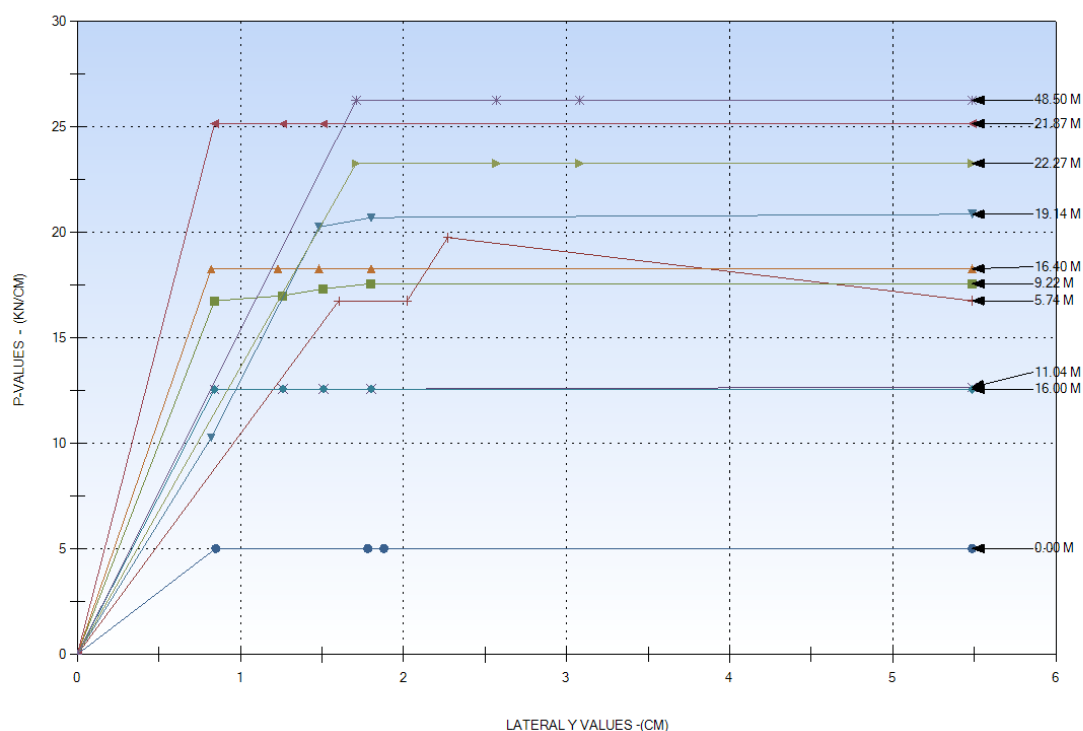


Figure 2.3 : The typical p-y curve derived for the complete soil profile.

2.2.3.2 Axial T-Z Curve - Axial Shaft Friction-Displacement Relationship

The t-z curve illustrates the relationship between ($t - \text{kN/cm}^2$) and ($z - \text{cm}$) per 30 cm pile.

This curve is crucial for evaluating the axial behaviour of piles and understanding how shear stress develops with increasing vertical displacement along the pile shaft.

The accompanying Table 2.3 presents the values used in this study for the shear stress applied to the designed piles, along with the corresponding vertical displacements at various depths.

Table 2.3 : T-Z data.

0.0		5.42		11		16.4	
Z	T	Z	T	Z	T	Z	T
0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.118	0.202	0.118	0.268	0.118	0.225	0.118	0.202
0.236	0.404	0.236	0.536	0.236	0.45	0.236	0.404
0.394	0.673	0.394	0.894	0.394	0.75	0.394	0.673
0.591	0.673	0.591	0.894	0.591	0.75	0.591	0.673
18.09		21.82		22.27		48.5	
Z	T	Z	T	Z	T	Z	T
0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.118	0.359	0.118	0.072	0.118	0.202	0.118	0.202
0.236	0.718	0.236	0.144	0.236	0.404	0.236	0.404
0.394	1.197	0.394	0.239	0.394	0.637	0.394	0.637
0.591	1.197	0.591	0.239	0.591	0.637	0.591	0.637

Figure 2.4 displays the t-z curve for a pile with an outer diameter of 1.1176 meters, and a wall thickness of 0.025 meters for the top 10 meters of the pile, and 0.015 meters for the remaining 30 meters of the total length.

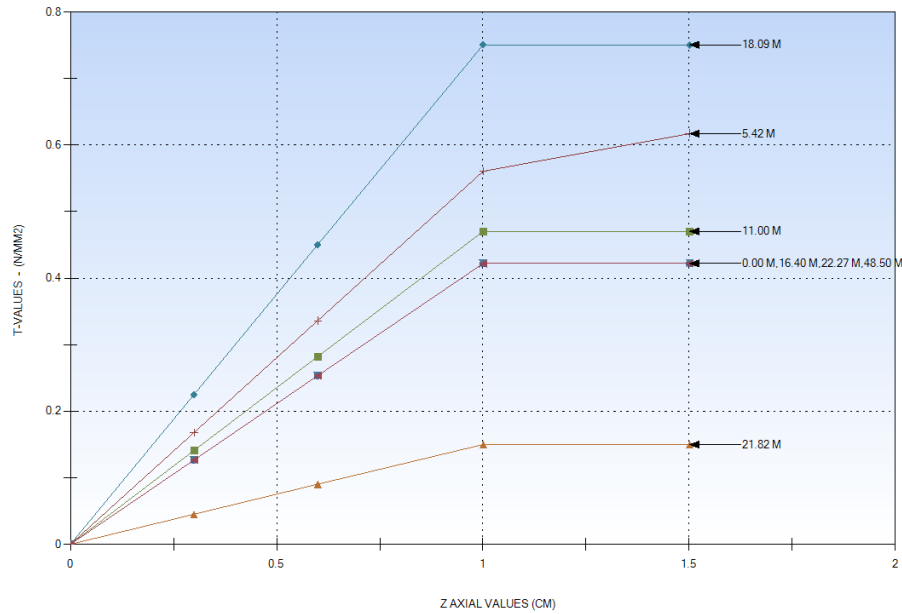


Figure 2.4 : The typical t-z curve derived for the complete soil profile.

2.2.3.3 End Bearing Q-Z Curve

The q-z curve illustrates the relationship between ($q - \text{kN/cm}^2$) and ($z - \text{cm}$) per 30 cm pile.

This curve is demonstrating how the end-bearing resistance evolves as the pile tip undergoes displacement

The accompanying Table 2.4 presents the values used in this study.

Table 2.4 : Q-Z data.

22.5		48.5	
Z	Q	Z	Q
0.00	0.00	0.00	0.00
39.37	1	39.37	1.25

Figure 2.5 displays the q-z curve for a pile with an outer diameter of 1.1176 meters, and a wall thickness of 0.025 meters for the top 10 meters of the pile, and 0.015 meters for the remaining 30 meters of the total length.

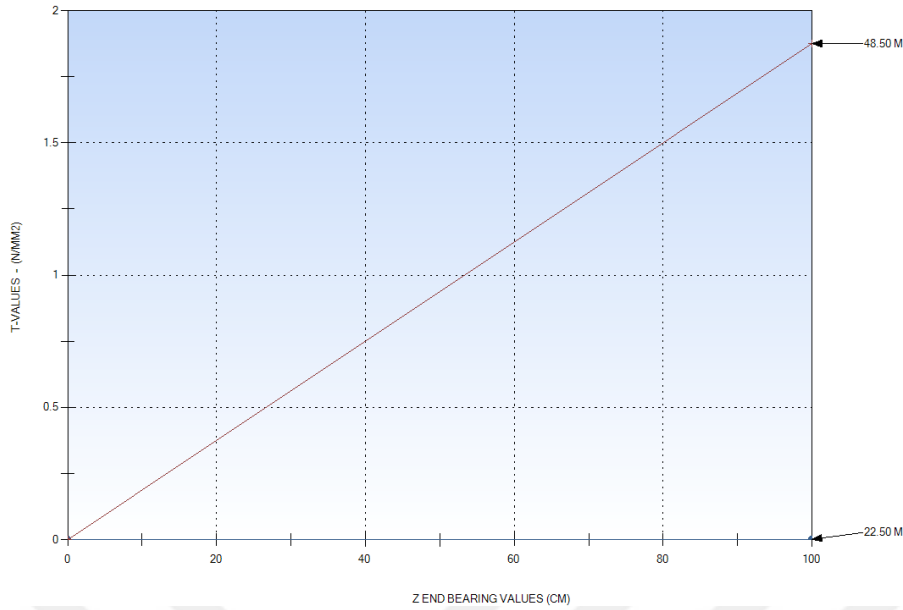


Figure 2.5 : The typical Q-Z curve derived for the complete soil profile.

2.3 Allowable Stresses for Cylindrical Members

2.3.1 Axial tension

The permissible axial tensile stress for cylindrical structural elements is determined based on the yield strength of the material from which they are fabricated.

This value is typically calculated using the equation (2.2) :

$$F_t = 0.6 \times \sigma_y \quad (2.2)$$

Where

σ_y = yield strength of the steel material (mpa).

2.3.2 Axial compression

2.3.2.1 Column buckling

Cylindrical structural members subjected to axial compressive loads are especially susceptible to buckling, owing to their slender geometry and extended length.

The provided equations (2.3) to (2.5) calculates the allowable axial compressive stress F_a for columns, considering buckling effects under specific conditions.

$$F_a = \frac{\left[1 - \frac{(Kl/r)^2}{2C_c^2}\right] F_y}{\frac{5}{3} + \frac{3(Kl/r)}{8C_c} - \frac{(Kl/r)^3}{8C_c^3}} \quad (2.3)$$

for $Kl/r < C_c$

$$F_a = \frac{12\pi^2 E}{23(Kl/r)^2} \quad (2.4)$$

for $(Kl/r)^2 \geq C_c$

Where

$$C_c = \sqrt{\frac{2\pi^2 E}{F_y}} \quad (2.5)$$

E = Modulus of elasticity of the material, representing its stiffness and ability to resist deformation under load.

K = A dimensionless coefficient that accounts for the boundary conditions of the column

l = The distance between points along the column that are braced against lateral movement (m)

R = A geometric property that describes the distribution of the column's cross-sectional area around its centroidal axis.

2.3.2.2 Local buckling

2.3.2.2.1 Elastic local buckling stress

The elastic local buckling stress can be found from the equation (2.6):

$$F_{xe} = 2CE t/D \quad (2.6)$$

Where

C = critical elastic buckling coefficient.

D = outside diameter, (meter)

t = wall thickness, (meter).

The value of C = 0.6.

2.3.2.2 In-elastic local buckling stress

The inelastic local buckling stress, F_{xc} and Allowable axial compressive stress, F_{xe} should be determined from the equations (2.7) and (2.8) :

$$F_{xc} = F_y \times \left[1.64 - 0.23 \left(\frac{D}{t} \right)^{1/4} \right] \quad (2.7)$$

For $\left(\frac{D}{t} \right) \leq 60$

$$F_{xe} = F_y \quad (2.8)$$

2.3.3 Bending

Equations (2.9) through (2.17) provide the allowable bending stress for three cases, categorized according to the diameter-to-thickness ratio $\left(\frac{D}{t} \right)$ of the tubular section.

Case 1:

$$\frac{D}{t} \leq \frac{1500}{F_y} \quad (2.9)$$

$$F_b = 0.75 \cdot F_y \quad (2.10)$$

For *SI* Units

$$\frac{D}{t} \leq \frac{10,340}{F_y} \quad (2.11)$$

Case 2:

$$\frac{1500}{F_y} < \frac{D}{t} \leq \frac{3000}{F_y} \quad (2.12)$$

$$F_b = \left[0.72 - 0.58 \frac{F_y D}{Et} \right] F_y \quad (2.13)$$

For *SI* Units

$$\frac{10,340}{F_y} < \frac{D}{t} \leq \frac{20,680}{F_y} \quad (2.14)$$

Case 3:

$$\frac{3000}{F_y} < \frac{D}{t} \leq 3000 \quad (2.15)$$

$$F_b = \left[0.72 - 0.58 \frac{F_y D}{Et} \right] \cdot F_y \quad (2.16)$$

For *SI* Units

$$\frac{20,680}{F_y} < \frac{D}{t} \leq 300 \quad (2.17)$$

2.3.4 Shear

2.3.4.1 Beam Shear

The permissible shear stress in beams with cylindrical cross-sections can be defined as a function of both the material's yield strength and the geometric characteristics of the cross-section.

The maximum value is from the equation (2.18).

$$f_v = \frac{V}{0.5A} \quad (2.18)$$

Where

f_v : Shear stress (N/m²).

V = shear force, (MN).

A = Cross-sectional area, (m²).

2.3.4.2 Torsional Shear

The torsional shear stress in cylindrical members can be represented using equation (2.19).

$$F_{vt} = \frac{M_t \cdot \left(\frac{D}{2}\right)}{I_p} \quad (2.19)$$

Where

F_{vt} = max. torsional shear stress (MPa)

M_t = Applied torque (MN-m)

I_p = Polar moment of inertia (m^4)

2.4 Environmental Load Data and Load Combinations

The structure is exposed to various types of loads, including dead loads, live loads, and environmental loads. Dead loads primarily consist of the self-weight of the structural elements along with the permanent weight of machinery and equipment installed on the platforms. Live loads refer to operational weights such as those caused by maintenance activities and the movement of equipment across the platform. Environmental loads encompass natural forces including wave action, storm conditions, continuous underwater currents, and wind pressure acting on the parts of the structure above the water surface.

These loads can generally be classified as either distributed or concentrated, depending on how they are applied to the structure.

The total applied loads on the platform are summarized in Table 2.5. In the context of this thesis, the characteristics and effects of these loads were clearly defined. A comprehensive mechanical and static analysis was then performed using SACS V16 software to evaluate the structural behaviour and ensure its stability under the combined influence of these loading conditions.

Table 2.5 : Total applied loads on the platform.

Applied Load	The Load Case	Sum of weights (- Z direction) /KN/
Area load	Main deck area	345 (0.750 KN/M2)
	Cellar deck area	115 (0.50 KN/M2)
Dead load	Main deck dead load	145
	Cellar deck dead load	40.49
Live load	Main deck live load	962 (5.00 KN/M2)
	Cellar deck live load	202.49 (2.50 KN/M2)
Equipment load	First skd weight	1112.06
	Second skd weight	667.229
	third skd weight	444.83
	Forth skd weight	155.5869
Crane load,	Crane vessel weight	88.965
Fire wall	Blast wall weight	46
load,	The walkway weight	124.7849
walk way		
load		
Anode load	Anode	5849

The load combinations used in the analysis are listed in Table 2.6.

Table 2.6 : Load combinations considered in the analysis.

Operation1	$1.00 \times \text{Total dead loads} + 0.75 \times \text{Total live loads} + 1.00 \times \text{P0000 wave load}$
Operation2	$1.00 \times \text{Total dead loads} + 0.75 \times \text{Total live loads} + 1.00 \times \text{P0000 wave load}$
Operation3	$1.00 \times \text{Total Area loads} + 0.75 \times \text{Total live loads} + 1.00 \times \text{P0000 wave load}$
Storm1	$1.00 \times \text{Total Area loads} + 1.00 \times \text{Total live loads} + 1.20 \times \text{S0000 wave load}$
Storm2	$1.00 \times \text{Total Area loads} + 1.00 \times \text{Total live loads} + 1.20 \times \text{S0000 wave load}$
Storm3	$1.00 \times \text{Total Area loads} + 1.00 \times \text{Total live loads} + 1.20 \times \text{S0000 wave load}$

2.4.1 Water depth

The platform is located in - 65.53 m depth under water.

2.4.2 Wind

The wind loads are calculated based on the API RP 2A, using following- wind directions

0.00, 45.00 , 90.00 , 135.00 , 180.00 , 225.00 , 270.00 , 315.00, as listed in Table 2.7.

Table 2.7 : Wind load details.

Wind Details	Speed (meter/second)	Direction (Degree)	Seabed level (meter)	Wave height (meter)	variable's degree
P0000	25.725	000.000	-65.529	10.50	01/8.0
P0045	25.725	045.000	-65.529	10.50	01/8.0
P0090	25.725	090.000	-65.529	10.50	01/8.0
P0135	25.725	135.000	-65.529	10.50	01/8.0
P0180	25.725	180.000	-65.529	10.50	01/8.0
P0225	25.725	225.000	-65.529	10.50	01/8.0
P0270	25.725	270.000	-65.529	10.50	01/8.0
P0315	25.725	315.000	-65.529	10.50	01/8.0
S0000	46.17	000.000	-65.529	10.50	01/8.0
S0045	46.17	045.000	-65.529	10.50	01/8.0
S0090	46.17	090.000	-65.529	10.50	01/8.0
S0135	46.17	135.000	-65.529	10.50	01/8.0
S0180	46.17	180.000	-65.529	10.50	01/8.0
S0225	46.17	225.000	-65.529	10.50	01/8.0
S0270	46.17	270.000	-65.529	10.50	01/8.0
S0315	46.17	315.000	-65.529	10.50	01/8.0

2.4.3 Wave and current

Wave height with associated period for 1 year wave conditions and 100 years

Conditions and the wave properties considered in this study are as shown in Tables 2.8 and Tables 2.9:

Table 2.8 : Wave load details.

On Year			Hundred Year		
Wave height (meter)	Wave Period (second)	waves speed (meter/second)	Wave height (meter)	Wave Period (second)	waves speed (meter/second)
06.12	12.50	19.750	12.18	15.50	23.380

Table 2.9 : Wave load details.

Wave load	Wave speed (meter/sec ond)	Wave Position Step size (degree)	Period (Sec)	The Wave Directions (Degree)	Waves Peak Height (meter)
P0000	25.725	20.00	12.50	000.000	6.15
P0045	25.725	20.00	12.50	045.000	6.15
P0090	25.725	20.00	12.50	090.000	6.15
P0135	25.725	20.00	12.50	135.000	6.15
P0180	25.725	20.00	12.50	180.000	6.15
P0225	25.725	20.00	12.50	225.000	6.15
P0270	25.725	20.00	12.50	270.000	6.15
P0315	25.725	20.00	12.50	315.000	6.15
S0000	45.175	20.00	15.50	000.000	12.18
S0045	45.175	20.00	15.50	045.000	12.18
S0090	45.175	20.00	15.50	090.000	12.18
S0135	45.175	20.00	15.50	135.000	12.18
S0180	45.175	20.00	15.50	180.000	12.18
S0225	45.175	20.00	15.50	225.000	12.18
S0270	45.175	20.00	15.50	270.000	12.18
S0315	45.175	20.00	15.50	315.000	12.18

2.4.4 Hydrodynamic coefficients (drag and inertia coefficients)

The majority of the variations in the drag (C_d) and inertia (C_m) coefficients can be attributed to their dependence on the following parameters:

Relative surface roughness $e = k/D$

Reynolds number $R_m = U_m D / \nu$

Keulegan-Carpenter number $K = 2U_m T^2 / D$

Current/wave velocity ratio $r = V_1 / U_{mo}$

The basic drag and inertia coefficients utilized in this study for evaluating wave forces on cylindrical members are as shown in Tables 2.10.

Table 2.10 : Drag and inertia coefficients used in the analysis.

Element outer diameter (cm)	Drag (coefficient)	Inertia (Coefficient)
2.60	0.6500	1.2500
260.00	0.6500	1.2500

2.4.5 Marine growth profile

Marine growth increases the self-weight of the structure and enhances the hydrodynamic forces acting upon it, including the added hydrodynamic mass. This, in turn, negatively affects the hydrodynamic stability of the structure.

For structural design purposes, the total hydrodynamic forces acting on the structure are typically estimated using Morison's equation, which accounts for both inertial and drag components. The specific weight of marine growth in air is generally assumed to be 1.4 kN/m³.

In this thesis, the total weight of marine growth is calculated to be 1258.146 kN.

The sea state characteristics corresponding to the global marine zone are summarized in Table 2.11.

Table 2.11 : Sea state global marine zone description.

Area above the seabed level (meter)	Member Thickness (meter)	Material densities (kg/m ³ .)	Marine status
0.00 to 65.00	0.025	1.401	CONS.
65.00 to 65.529	0.05	1.401	CONS.

2.5 Model Analysis Results

The designed structure was analyzed dynamically, and the analysis results revealed up to 100 modes, in addition to identifying the maximum displacement for each mode along the three principal axes. These outcomes are a result of the structure's mass and the applied loads.

This type of analysis is instrumental in understanding the dynamic behavior of the structure, thereby contributing to the enhancement of its performance under expected environmental loads.

Table 2.12 presents the results of the modal analysis, including the natural frequencies and mode shapes of the structure.

As presented in Figure 2.6, the deformation patterns represent the dynamic response of the structure.

Table 2.12 : Model analysis results.

Mode- shape no	Max. Deflec. X direction (centimetre)	Max. Deflec Y direction (centimetre)	Max. Deflec Z direction (centimetre)	structure Period (Second)	Frequencies (HZ)
1	0.872	2.850	-0.622	1.8659225	0.852499
2	2.187	-0.525	-0.921	1.76531578	0.624978
3	-2.678	2.501	-0.996	0.7964568	1.266329
4	-2.070	2.055	0.763	0.0657545	1.223210
5	2.821	-0.881	0.779	0.7895666	2.722642
6	-1.897	2.951	-1.752	0.9782665	2.783252
7	0.726	0.520	2.995	0.7686487	2.872365
8	-0.182	-0.186	2.742	0.8968578	2.765387
9	0.782	0.598	2.795	0.1787387	3.887223
10	-0.987	0.591	2.757	0.8798729	3.386556
20	0.638	0.186	2.289	0.7237189	4.761256
30	2.227	1.959	2.752	0.9838296	6.054577
40	0.786	-0.419	2.592	0.782322	7.988656
50	2.546	-0.943	0.357	0.731296	8.985678
60	-0.845	-0.179	2.442	0.9375678	10.878239
70	0.758	-0.989	2.979	0.9982368	11.897298
80	-1.127	1.849	2.962	0.9789834	13.787257
90	-0.057	-1.847	2.762	0.1065449	15.998678
100	0.457	0.086	2.886	0.0781245	18.273782

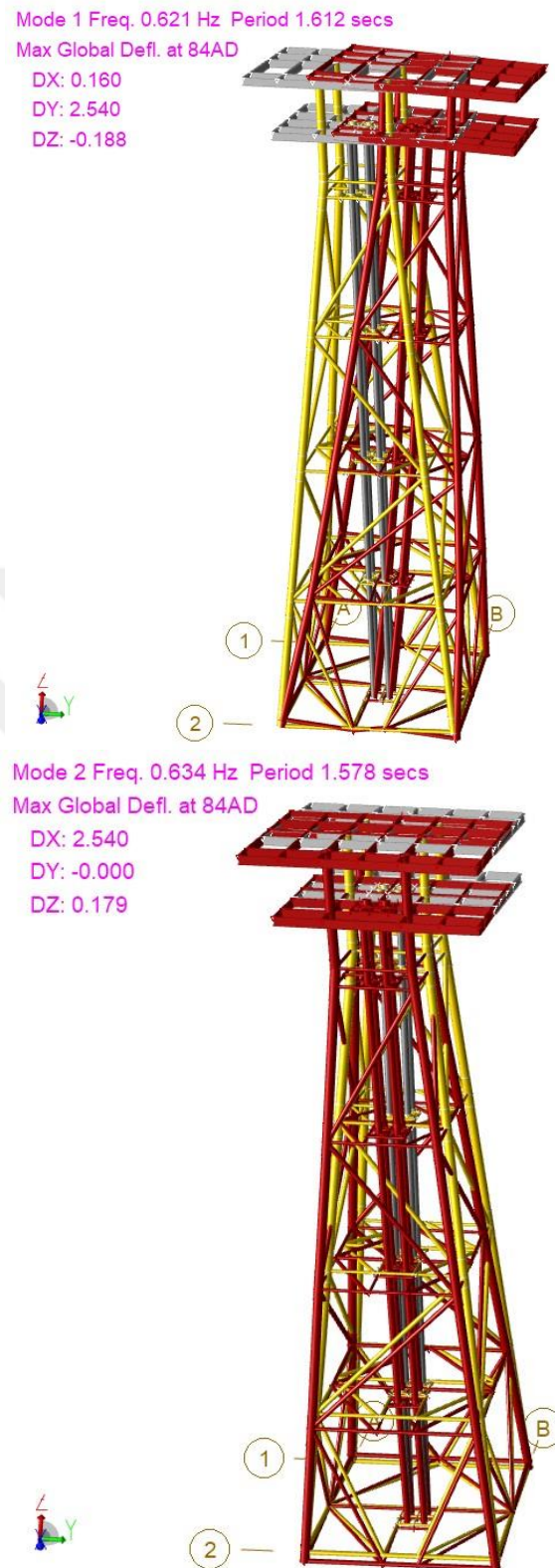


Figure 2.6 : Mode shapes of the fixed offshore structure.

Mode 3 Freq. 1.164 Hz Period 0.859 secs
Max Global Defl. at 81AD
DX: -2.395
DY: 2.540
DZ: 0.042



Mode 4 Freq. 1.858 Hz Period 0.538 secs
Max Global Defl. at 84FD
DX: -2.009
DY: 2.540
DZ: -0.297



Mode 5 Freq. 2.106 Hz Period 0.475 secs
Max Global Defl. at 81AD
DX: 2.540
DY: -0.186
DZ: 0.355



Mode 6 Freq. 2.385 Hz Period 0.419 secs
Max Global Defl. at 84AD
DX: 0.728
DY: 2.540
DZ: -0.831



Figure 2.6 (continued) : Mode shapes of the fixed offshore structure.

Mode 7 Freq. 2.442 Hz Period 0.409 secs
Max Global Defl. at 0007

DX: -0.058
DY: -0.002
DZ: 2.540



Mode 8 Freq. 2.510 Hz Period 0.398 secs
Max Global Defl. at 0016

DX: -0.011
DY: 0.005
DZ: 2.540



Mode 9 Freq. 3.120 Hz Period 0.321 secs
Max Global Defl. at 87AD

DX: 0.635
DY: 0.356
DZ: 2.540



Mode 10 Freq. 3.285 Hz Period 0.304 secs
Max Global Defl. at 84AD

DX: -0.076
DY: 0.098
DZ: 2.540



Figure 2.6 (continued) : Mode shapes of the fixed offshore structure.

3. NONLINEAR TIME HISTORY ANALYSIS AND MODAL RESPONSE SPECTRUM ANALYSIS

Among the fundamental principles in the design of fixed offshore structures is the necessity of accounting for the potential impact of seismic activity, which may significantly affect the structural integrity, especially when the design is intended for a site with a known seismic history and located in a seismically active region.

As part of the analytical study of the structure, a dynamic analysis was conducted using the SACS software, in accordance with international ISO standards, to ensure the structure's resistance to seismic loads.

A time-history analysis was performed using actual ground motion records from three significant historical earthquakes. These real seismic acceleration data were applied to the designed structure to accurately simulate its response to seismic events. The structural responses and resulting data were carefully recorded and analyzed as part of the study.

Figure 3.1 illustrates the time history records of the three ground motions considered in this study.

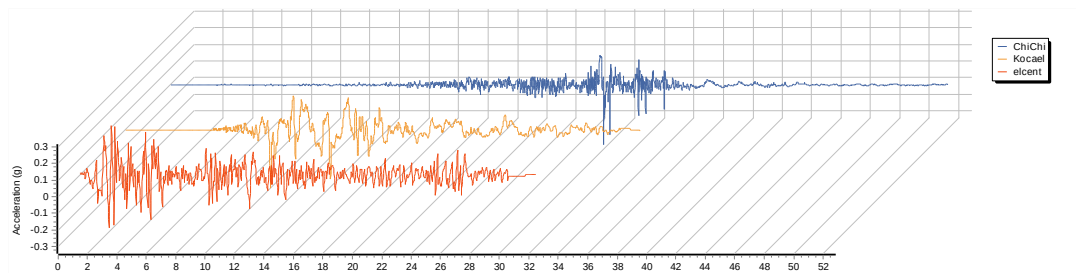


Figure 3.1 : Time history of the three ground motions.

The seismic acceleration time histories were calibrated at 5% viscous damping to match the elastic response spectrum. This calibration is essential to ensure that the

applied seismic forces on the fixed structure align with the expected structural response under realistic and actual site conditions.

Such an approach is critical for ensuring the stability and safety of the designed structure, particularly in seismically active regions. Matching the spectral characteristics of the ground motion data to the design response spectrum ensures that the dynamic behavior of the structure is accurately captured and that the design remains robust against potential seismic excitations.

3.1 Seismic Design Methodology According to ISO Standards.

3.1.1 Extreme level earthquakes design

This method is part of the structural design procedures intended to ensure the structure's resistance against extreme seismic loads. As a result of the structural analysis, the structure exhibits localized and limited nonlinear behaviour, causing yielding at specific points. This approach is essential to ensure the structure's ability to withstand seismic forces without causing structural damage or collapse.

3.1.2 Abnormal level earthquakes design

This method is part of the structural design procedures intended to ensure the structure's resistance against seismic loads that exceed normal levels. In this case, the pile foundations of the structure reach their axial capacity, and the structural elements exhibit plastic (inelastic) behavior. This analysis is conducted to verify the structure's ability to withstand extreme loads applied to it, demonstrating its capacity to resist both moderate and severe seismic events.

Seismic analysis procedures for structures, such as time history and spectral analysis, are carried out to ensure resistance against maximum expected seismic loads, while pushover analysis is used to evaluate the ultimate capacity of the structural elements under exceptional conditions.

3.2 Spectral Acceleration Data

To determine the spectral accelerations for the area where the structure is to be designed, detailed seismic hazard maps are required—such as the maps provided in Annex B of the ISO 2004 standard.

The maps include the essential values and parameters required for the design of offshore structures and to ensure their ability to withstand seismic events and hazards.

According to the Turkish Building Earthquake Code (TBEC), the fundamental values required for seismic design were obtained using the Dlubal software.

The values of the ground motion parameters S_s and S_1 , as well as the peak ground acceleration (PGA) for a return period of 2,475 years.

The location of the designed offshore structure is assumed to be situated off the coast as specified on the seismic hazard maps. The spectral acceleration values presented are expressed in terms of g and correspond to 5% damped spectral accelerations on bedrock outcrops, classified as Site Class A / B based on standard site classification criteria.

Figures 3.2 to 3.4 present the seismic hazard parameters associated with a return period of 2,475 years. Specifically, Figure 3.2 shows the Peak Ground Acceleration (PGA), while Figures 3.3 and 3.4 illustrate the spectral acceleration parameters S_s and S_1 , respectively.

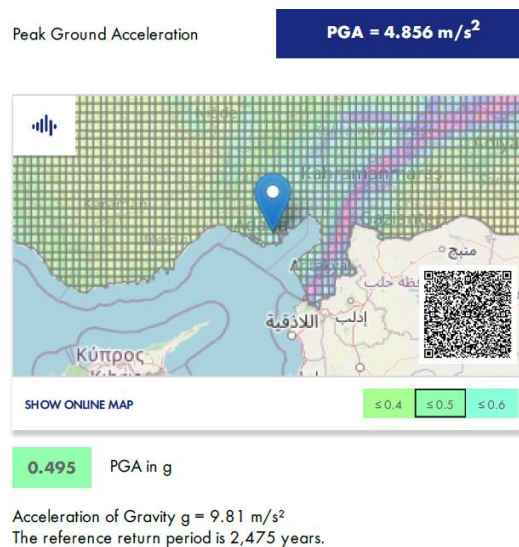
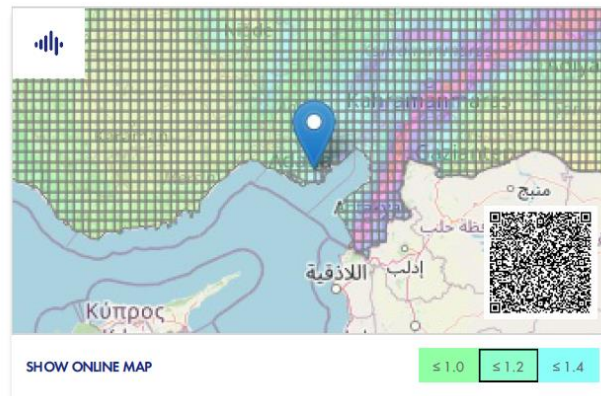


Figure 3.2 : Peak ground acceleration for return period 2,475 years.

Ground Motion Parameter

$$S_S = 11.674 \text{ m/s}^2$$



1.19 S_S in g

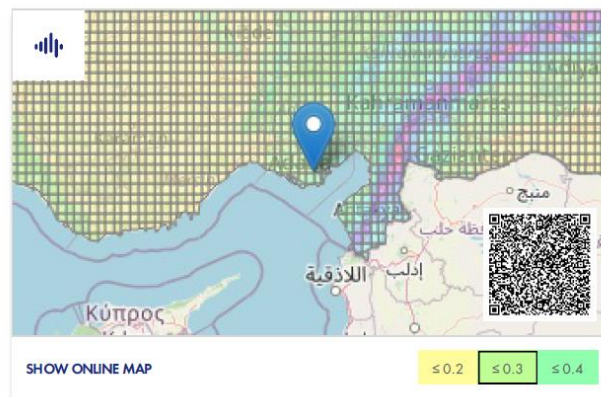
Gravitational Acceleration $g = 9.81 \text{ m/s}^2$, T...

The reference return period is 2,475 years.

Figure 3.3 : Ground motion parameters (S_S) for return period 2,475 years.

Ground Motion Parameter

$$S_1 = 2.933 \text{ m/s}^2$$



0.299 S_1 in g

Gravitational Acceleration $g = 9.81 \text{ m/s}^2$, T...

The reference return period is 2,475 years.

Figure 3.4 : Ground motion parameters (S_1) for return period 2,475 years.

3.3 Determine Seismic Risk Category

The seismic zone for the area where the structure is to be designed is determined based on seismic hazard maps, with a specific focus on the One-Second Spectral Acceleration values, which range between 0.26 g and 0.45 g.

Accordingly, the design is carried out assuming Seismic Zone 3.

The exposure level of the designed structure is also classified as L3.

Given that the site falls within Seismic Zone 3, the corresponding Seismic Risk Category (SRC) is determined to be SRC 2.

The seismic zone classification of the site is presented in Table 3.1.

Table 3.1 : The site seismic zone.

One-second spectral acceleration	0.26 g to 0.45 g
The site seismic zone for the platform	3

3.4 Simplified Seismic Action Procedure

3.4.1 Soil classification and spectral shape

The soil type at the design site is determined along with the appropriate spectral shape to represent ground motion characteristics. The soil classification influences the damping capacity of seismic waves, which in turn affects the shape of the design spectrum used in structural analysis.

a) Determine the site class

The soil properties at the site where the structure is to be designed—specifically the upper 30 meters of the effective seabed—are identified and evaluated. These properties are assessed in conjunction with the spectral acceleration values obtained from the site-specific seismic hazard maps, corresponding to a return period of 2,745 years.

Based on the studied seabed soil conditions, the site is classified as A/B (hard rock), with the thickness of soft sediments being less than 5 meters. The foundations of the offshore structure were designed using deep pile foundations.

The site soil classification was considered based on a 30-meter depth of soil located directly beneath the point of pile resistance, which is typically at the centroidal depth of lateral resistance forces (P-Y) or vertical resistance forces (T-Z).

- b) Determine the site coefficients C_a and C_v for deep pile foundations for the soil class A-B are 1.0 for C_a and 0.8 for C_v as shown in Table 3.2.

Table 3.2 : The site coefficients.

Site class	C_v	C_a
A-B	0.8	1.0

- c) To determine the site-specific horizontal acceleration spectrum for a 2,745-year return period,

The expressions given by ISO for representing the horizontal elastic response spectrum are given in the equations (3.1) to (3.4).

As shown in Figure 3.5, the seismic acceleration spectrum corresponding to 5% damping is presented

For periods T less than or equal to **0.2** seconds:

$$S_{a,site}(T) = (3T + 0.4) \cdot C_a \times S_{a,map}(0.2) \quad (3.1)$$

For periods T greater than 0.2 seconds:

$$S_{a,site}(T) = C_v \times S_{a,map}(1.0)/T \quad (3.2)$$

except that

$$S_{a,site}(T) \leq C_a \times S_{a,map}(0.2) \quad (3.3)$$

For periods T greater than 4.0 seconds:

$$S_{a,site}(T) = 4C_v \times S_{a,map}(1.0)/T^2 \quad (3.4)$$

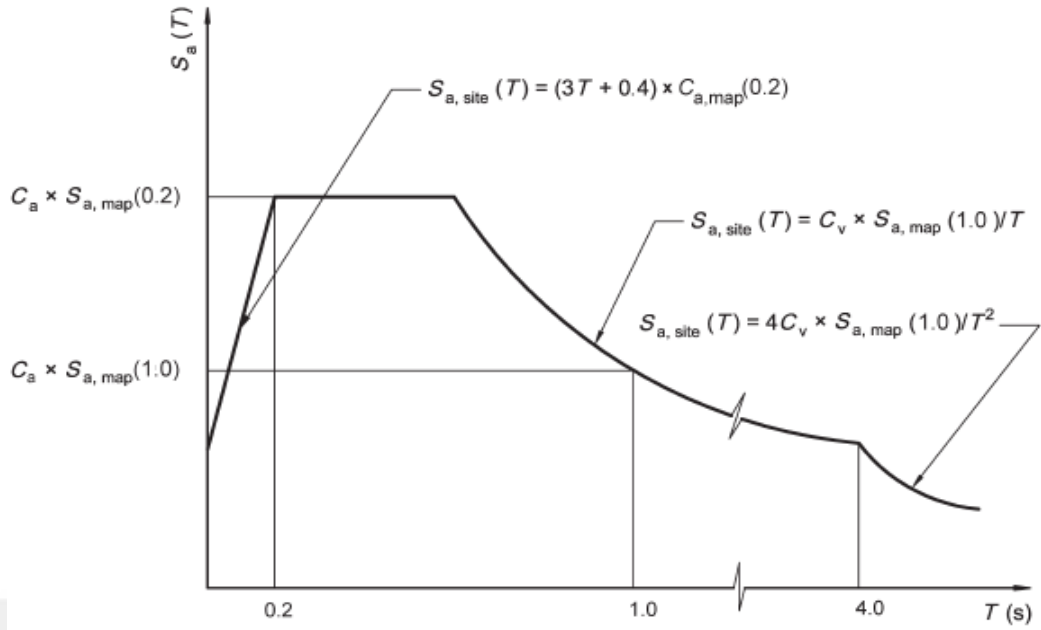


Figure 3.5 : Seismic Acceleration Spectrum for 5 % Damping.

3.4.2 Normalization of spectral acceleration

We perform the normalization of the spectral acceleration related to seismic response, which is originally measured in meters per second squared (m/s^2), into dimensionless relative values that express the acceleration as a ratio to gravitational acceleration. This process allows for a consistent comparison between the Turkish seismic code and the earthquake records applied to the designed structure.

Figure 3.6 illustrates the response spectra, which are normalized to a peak ground acceleration of 1.0 gravity.

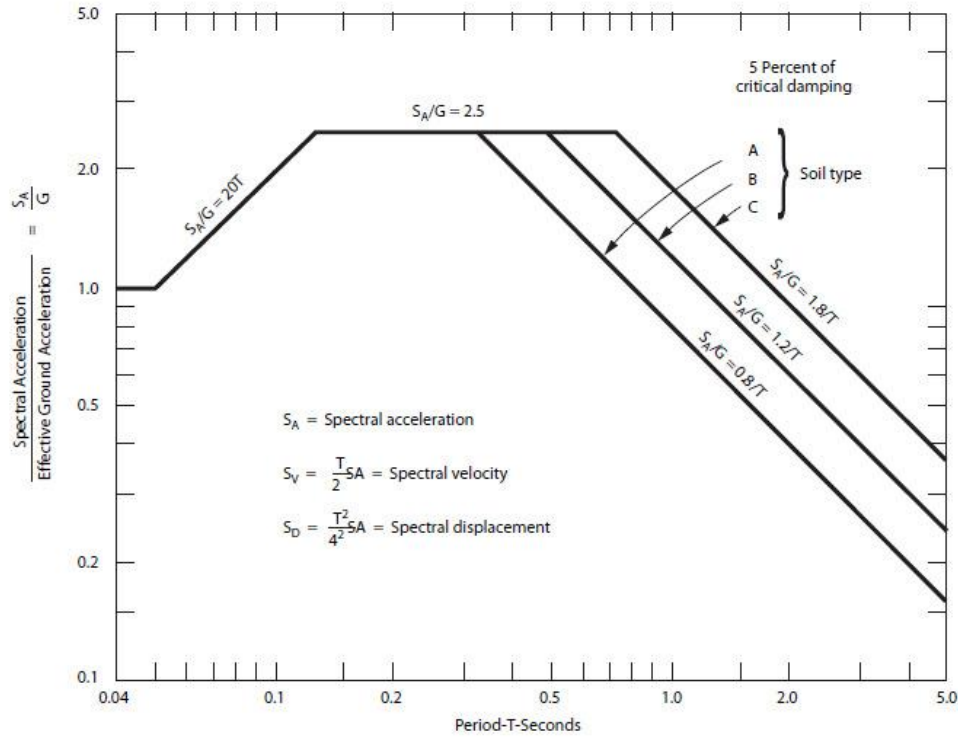


Figure 3.6 : Response spectra - spectra normalized to 1.0 gravity.

In this thesis, the platform is located in Seismic Zone 3, with Type A soil (rock) near the surface and Type A/B soil underlying the deeper sections of the pile foundation.

3.5 Selecting Ground Motions

Three real strong ground motions were selected in accordance with ISO requirements from the PEER Ground Motion Database. These ground motions were utilized to perform a nonlinear dynamic time history analysis of the structure. The time-stepping interval used in the earthquake ground motion records is 0.05 seconds. The details of the selected ground motions are provided in Table 3.3.

Table 3.3 : Selected ground motions details.

No	Earthquake	Station	MW	Vs30 (m/s)	Rjb (km)	Rrup (km)
1	El Centro 1940	El Centro Array #9 station	6.9	~760 m/s	~0 km	~0–5 km
2	Kocaeli 1999	Arcelik station	7.4	~523 m/s	~13.5 km	~13.5 km
3	Chi-Chi 1999	CHY024 station	7.6	~428 m/s	~19.7 km	~26.7 km

3.5.1 Ground motion matching

In this study, the spectral matching method was employed using SeismoMatch software. A comparison was conducted to assess the degree of matching between the spectra of the three selected ground motions and the elastic response spectrum.

The elastic response spectrum adopted for the matching process was selected in accordance with the Turkish Code. (TBDY)

Figure 3.7 presents a comparison of the Turkish code elastic response spectrum and the spectra generated to match the selected ground motions.

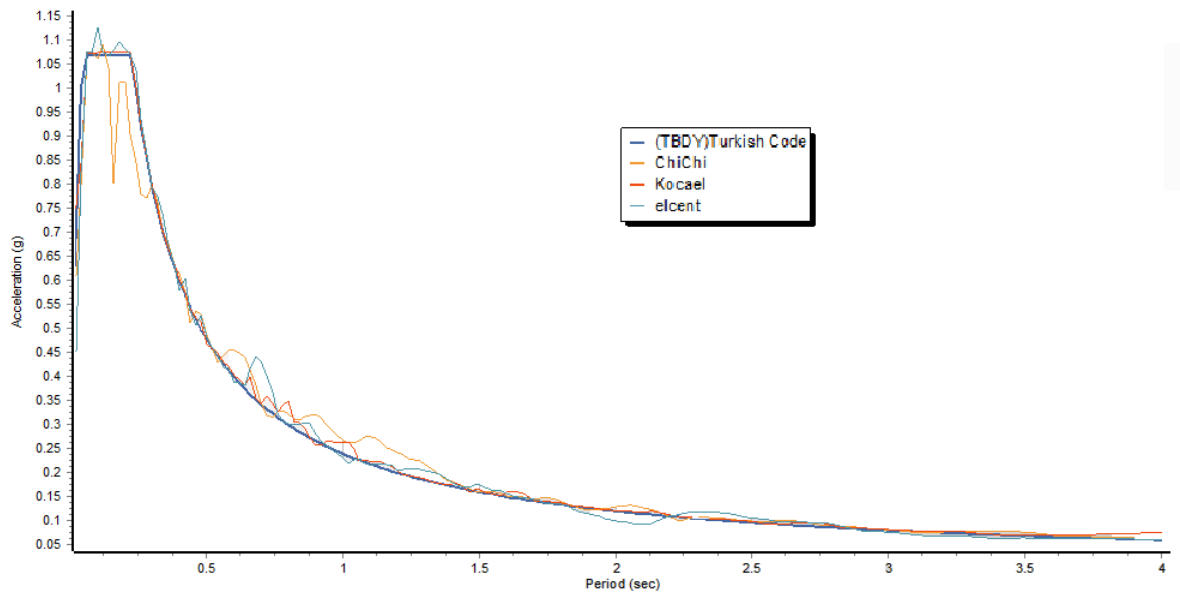


Figure 3.7 : Comparison of the elastic response spectrum of Turkish code with the matched spectra of selected ground motions.



4. SPECTRAL WAVE FATIGUE ANALYSIS

4.1 Fatigue Analysis

The continuous dynamic motion of waves is the primary cause of fatigue stresses affecting the structure of offshore installations, particularly in the tubular joint areas. In our study, waves were generated from eight different directions to ensure a comprehensive evaluation of the fatigue impact on the structure. The structural dynamic analysis relies on the application of Stress Concentration Factors (SCFs) and the calculation of stress under wave influence to assess the overall structural behaviour.

4.2 Stress Concentration Factors (SCF)

The tubular joints forming the structural elements of fixed offshore platforms are significantly affected by fatigue stresses. These stresses are highly concentrated in the welded areas due to elevated stress concentration factors in these regions. SACS software was used to determine the stress values in the tubular joints through the Finite Element Analysis (FEA) method. This approach provides a detailed analysis of the stress concentrations and their locations within the tubular joints.

The Stress Concentration Factor (SCF) for each brace loading type is calculated using the following equation (4.1) :

$$\text{Stress Concentration Factors} = \frac{\text{hot spot stress range}}{\text{Nominal brace stress range}} \quad (4.1)$$

The Stress Concentration Factor (SCF) in tubular joints is determined based on specific factors such as axial loads, type of joint, in-plane bending, and out-of-plane bending.

4.3 S-N Curves For Tubular Connections

4.3.1 Basic S-N curves

Through the spectral fatigue analysis method, the stress responses of the structure to marine loads such as waves and currents are calculated. Using these S-N curves, the cumulative fatigue damage can be estimated by applying the Palmgren-Miner rule.

The S-N curve is used to estimate the fatigue life of each tubular joint in the studied structure. The horizontal axis represents the number of failure cycles, while the vertical axis represents the stress amplitude. Through this curve, the endurance of the joint under repeated wave-induced loading can be assessed before failure occurs.

The fatigue life estimation is based on the equation (4.2).

$$N = \left(\frac{C}{S^m} \right) \quad (4.2)$$

Where

N: Number of cycles to failure.

C: Fatigue strength constant.

S: Applied stress amplitude.

m: Fatigue exponent.

Table 4.1 summarizes the basic representative S–N curves for steel components exposed to air and seawater with adequate corrosion protection, in accordance with the specifications of ISO 19902. These curves form the basis for fatigue life assessment and are typically expressed using logarithmic equations.

Table 4.1 : Basic representative S–N curves for air and sea water (with adequate corrosion protection).

Curve	Air		Sea Water + Corrosion Protection	
	$\log_{10}(k_1)$	m	$\log_{10}(k_1)$	m
Tubular joints (TJ)	12.48	3.0 for $N \leq 10^7$	12.18	3.0 for $N \leq 1.8 \times 10^6$
	16.13	5.0 for $N > 10^7$	16.13	5.0 for $N > 1.8 \times 10^6$
Cast joints (CJ)	15.17	4.0		
Other joints (OJ)				
B	15.01	4.0 for $N \leq 10^7$	14.61	4.0 for $N \leq 10^5$
	17.01	5.0 for $N > 10^7$	17.01	5.0 for $N > 10^5$
C	13.63	3.5 for $N \leq 10^7$	13.23	3.5 for $N \leq 4.68 \times 10^5$
	16.47	5.0 for $N > 10^7$	16.47	5.0 for $N > 4.68 \times 10^5$
D	12.18	3.0 for $N \leq 10^7$	11.78	3.0 for $N \leq 10^6$
	15.63	5.0 for $N > 10^7$	15.63	5.0 for $N > 10^6$
E	12.02	3.0 for $N \leq 10^7$	11.62	3.0 for $N \leq 10^6$
	15.37	5.0 for $N > 10^7$	15.37	5.0 for $N > 10^6$
F	11.80	3.0 for $N \leq 10^7$	11.40	3.0 for $N \leq 10^6$
	15.00	5.0 for $N > 10^7$	15.00	5.0 for $N > 10^6$
F ₂	11.63	3.0 for $N \leq 10^7$	11.23	3.0 for $N \leq 10^6$
	14.71	5.0 for $N > 10^7$	14.71	5.0 for $N > 10^6$
G	11.40	3.0 for $N \leq 10^7$	11.00	3.0 for $N \leq 10^6$
	14.33	5.0 for $N > 10^7$	14.33	5.0 for $N > 10^6$
W ₁	10.97	3.0 for $N \leq 10^7$	10.57	3.0 for $N \leq 10^6$
	13.62	5.0 for $N > 10^7$	13.62	5.0 for $N > 10^6$

4.3.2 The member thickness effect

When the wall thickness exceeds the reference thickness, the appropriate thickness correction equation is applied. This equation (4.3) is specifically designated for as-welded joints according to the ISO standard.

$$Stress = Stress_0 \times \left(t_{ref}/t \right)^{\frac{1}{4}} \quad (4.3)$$

Where

t = the thickness for the studied member.

t_{ref} = the reference thickness 16 Millimetre.

$Stress_0$ = the allowable stress range from the S-N curve.

$Stress$ = allowable stress range.

4.4 Determining Long-Term Stress Range Distribution by Spectral Analysis

4.4.1 Stress transfer functions

Stress transfer functions are one of the essential steps in the spectral analysis method, as they define the relationship between wave motion and the structural response to that motion. The structural response to a full wave cycle is analyzed to accurately evaluate dynamic behavior.

To study wave-induced loads, several wave theories are available, such as Airy, Stokes, and Solitary wave theories. In this analysis, the Airy Linear Wave Theory was used due to its simplicity and its suitability for representing waves with small to moderate heights.

Figure 4.1 illustrates a comparison between wave profiles generated using Airy wave theory and Dean's wave theory, highlighting the differences in wave shape representation under regular wave conditions.

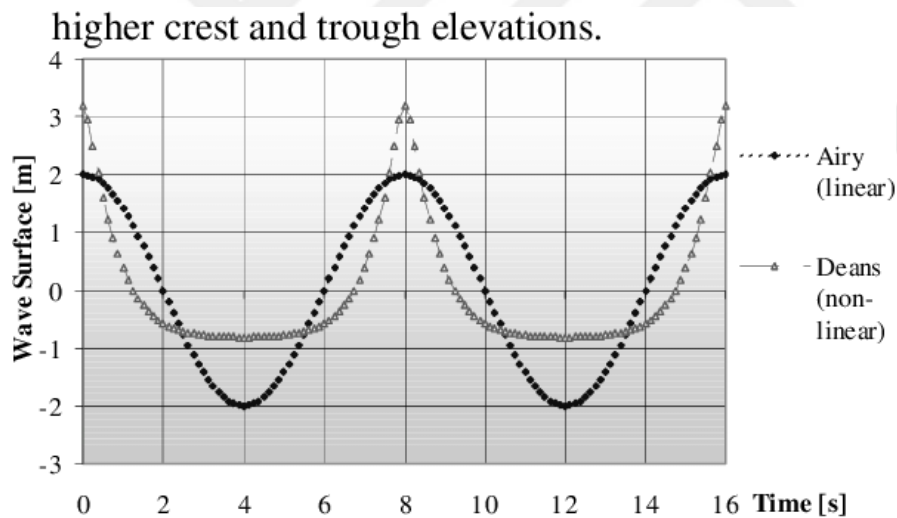


Figure 4.1 : Comparison of wave profile for regular Waves: airy (linear) vs. deans (non-linear).

The Airy linear wave theory contributes to accurate fatigue life prediction by generating transfer functions that precisely represent the stress distributions resulting from wave motion.

4.4.1.1 Selection of wave frequencies

To accurately determine the transfer functions associated with regular ocean waves, wave frequencies must be taken into account. Numerical calculations are performed to ensure that these transfer functions are consistent with the expected wave conditions.

In this study, the wave frequencies that the fixed offshore structure may encounter are identified. A transfer function graph is developed for each wave frequency and direction.

The figure 4.2 illustrates the relationship between wave-induced forces from each possible direction and the response of the offshore structure.

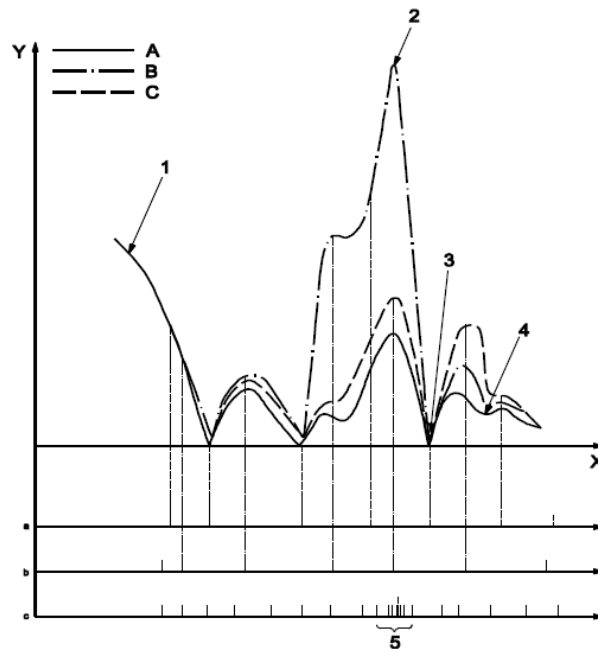


Figure 4.2 : Representative load transfer function for wave-induced forces.

Where :

A – Represents the quasi-static transfer function

B – Represents the dynamic transfer function at the peak

C – Represents the dynamic transfer function at the valley

Y-axis – Normalized applied wave action at wave frequency

X-axis – Typically represents wave frequency

1 – Represents long waves, low-frequency with no cancellation effects

2 – represents the natural frequency of the structure corresponding to a peak in the static transfer function

3 – represents the natural frequency of the structure corresponding to a valley in the static transfer function

4 – Peaks and valleys result from the interaction between structural geometry and wave length

5 – represents the fine grid around natural frequency

a : Underestimates the structural response (Poor)

b : Overestimates the structural response (Poor)

c : Accurately captures the characteristics of the transfer function (Good)

4.4.1.2 Selection of wave heights

When determining the transfer functions, the wave heights affecting the structure should be selected based on a constant wave steepness, which provides a simple relationship between wave frequency and wave height. Typical wave steepness values range from 1:15 - 1:25.

4.4.2 Short-term stress range statistics

This statistical analysis is used to determine the variation of stress caused by wave loads over a short period of time, typically measured in hours. It is based on the relationship between the wave characteristics (wave spectrum) affecting the structure and the dynamic response of the structure, known as transfer functions.

Short-term stress range analysis for each specific wave direction is a necessary step in calculating the long-term stress range, due to the repeated occurrence of these wave conditions and their cumulative impact on the structure.

This information is represented in what is known as the wave scatter diagram.

Tables 4.2 through 4.4 present the wave scatter diagrams corresponding to wave approach angles of 0.0°, 45.0°, and 90.0°, respectively.

Table 4.2 : the wave scatter diagram (0-degree direction).

Dominate Period (sec)	Significant wave height (M)		
	0.00 – 0.6	0.6-1.4	1.4-2.6
1.0-2.0	0.15	0.10	0.10
2.0-4.0	0.10	0.19	0.11
4.0-6.0	0.05	0.08	0.05
6.0-10.0	0.02	0.03	0.02

Table 4.3 : the wave scatter diagram (45-degree direction).

Dominate Period (sec)	Significant wave height (M)		
	0.00 – 0.6	0.6-1.4	1.4-2.6
1.0-2.0	0.10	0.13	0.08
2.0-4.0	0.15	0.13	0.10
4.0-6.0	0.08	0.08	0.07
6.0-10.0	0.03	0.02	0.03

Table 4.4 : the wave scatter diagram (90-degree direction).

Dominate Period (sec)	Significant wave height (M)		
	0.00 – 0.6	0.6-1.4	1.4-2.6
1.0-2.0	0.13	0.10	0.08
2.0-4.0	0.13	0.15	0.10
4.0-6.0	0.06	0.09	0.08
6.0-10.0	0.03	0.03	0.02

4.5 Palmgren-Miner Damage Rule

This analytical method is used to estimate the cumulative damage caused by fatigue in tubular joints as a result of repeated loading, such as wave-induced loads.

$$D = \sum_{i=1}^n \frac{n_i}{N_i} \quad (4.5)$$

D: Cumulative fatigue damage.

n_i: The number of cycles experienced at stress level *i*.

N_i: The number of cycles to failure at the same stress level *i*, determined from the S-N curve.

5. RESULTS AND DISCUSSION

To evaluate the behaviour of the fixed offshore structure, eight key points were identified and selected for analysis. These points are strategically located at various heights corresponding to different joints of the structure. Additionally, one of the selected points is situated at the mudline level to assess the foundation's response. The heights and joint names of the selected points are detailed in Table 5.1, while their specific locations are visually represented in Figure 5.1.

This comprehensive selection ensures a thorough assessment of the structure's performance across its critical components.

Table 5.1 : Joint name and the level of the selected key points.

The point name	The Joint Level (meter)	The Joint name
1. point	-65.558	104L Joint
2. point	-48.3798	204L Joint
3. point	-31.2379	304L Joint
4. point	-14.104	404L Joint
5. point	3.0498	504L Joint
6. point	4.89	604L Joint
7. point	11.357	704L Joint
8. point	17.6795	804L Joint

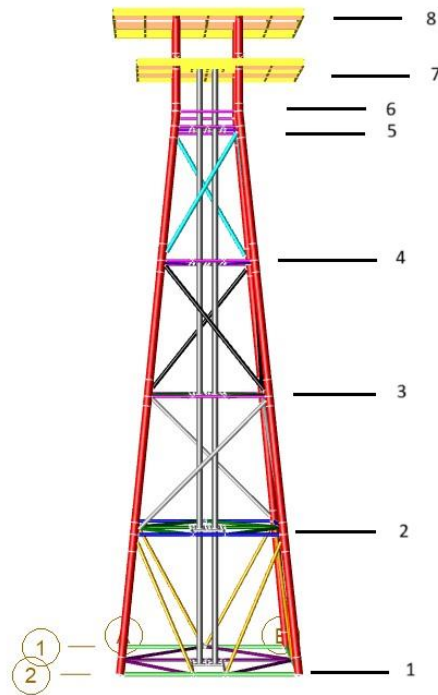


Figure 5.1 : The levels of the chosen key joints.

5.1 Modelling of Fixed Offshore Platform

Figures 5.2 to 5.9 illustrate the statically analysed stresses acting on the fixed offshore structure under external and environmental loads.

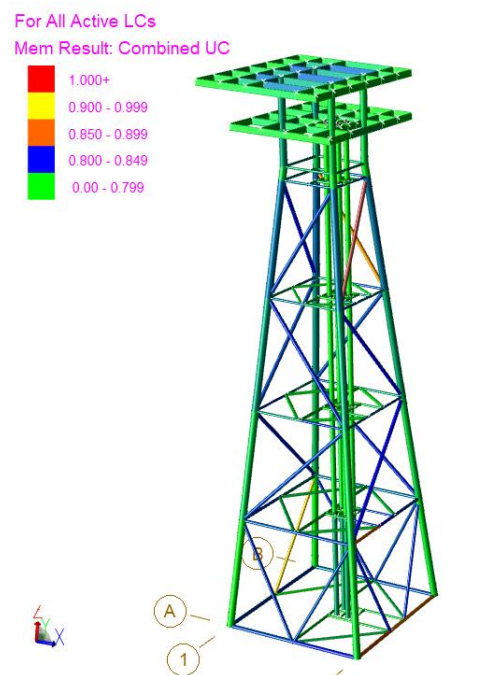


Figure 5.2 : maximum LC- unity check of the fixed platform.

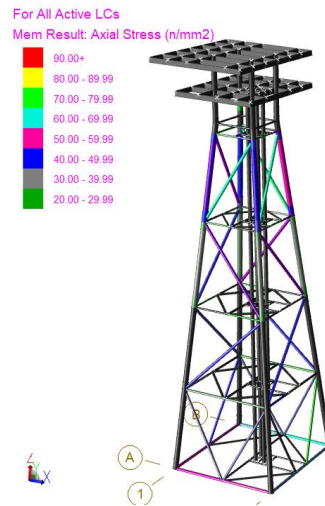


Figure 5.3 : Axial stress distributions of the fixed platform.

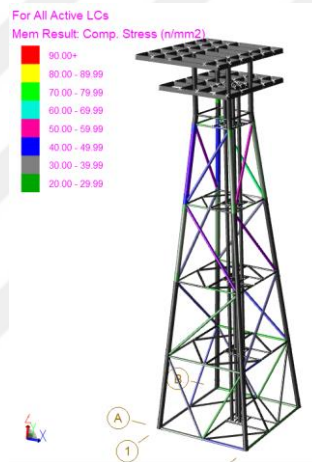


Figure 5.4 : Compression stress distributions of the fixed platform.

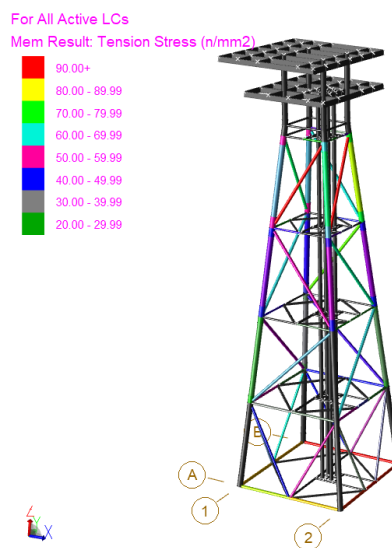


Figure 5.5 : Tension stress distributions of the fixed platform.

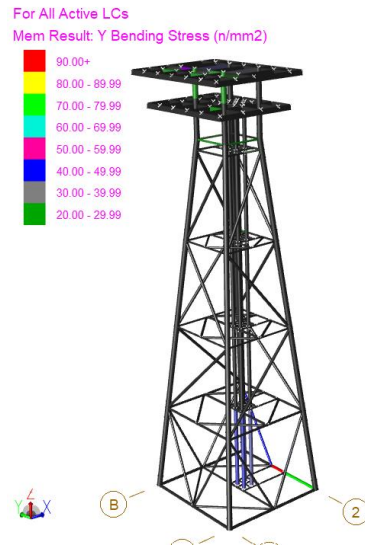


Figure 5.6 : Bending stress distribution around the Y-axis for the fixed platform.

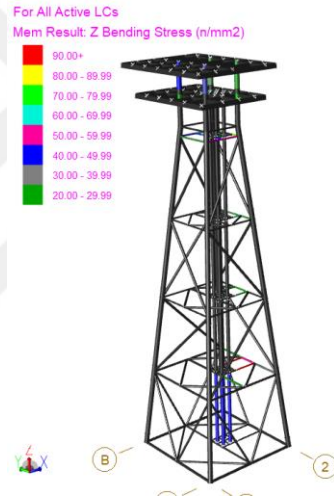


Figure 5.7 : Bending stress distribution around the Z-axis for the fixed platform.

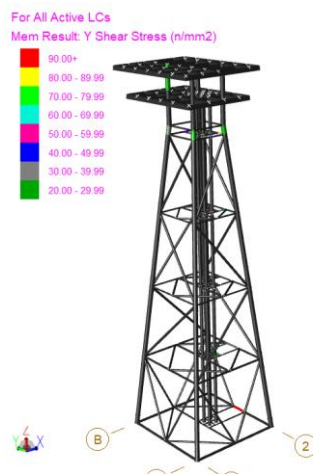


Figure 5.8 : Shear stress distribution around the Y-axis for the fixed platform.

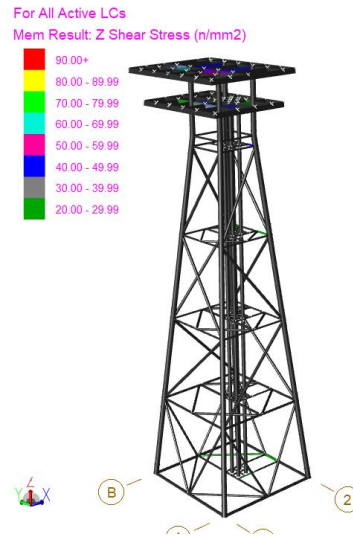


Figure 5.9 : Bending stress distribution around the Z-axis for the fixed platform.

5.2 Nonlinear Time History Analysis And Modal Response Spectrum Analysis

Table 5.2 presents the absolute displacement values recorded at selected structural points under the influence of the chosen ground motions. These values reflect the dynamic response of the structure to seismic loading scenarios.

Table 5.2 : Selected ground motions Absolute Displacements at selected points.

Point	Maximum Absolute Displacement (CM)		
	El Centro	Kocaeli	Chi-Chi
1. point	1.201	1.58	1.58
2. point	9.41	1.63	1.63
3. point	14.15	1.72	1.72
4. point	18.42	1.88	1.87
5. point	22.91	1.96	1.93
6. point	23.88	2.002	1.97
7. point	27.06	2.38	2.34
8. point	29.83	3.25	3.21

Table 5.3 shows the relative displacement values at selected structural points resulting from the applied ground motions. These displacements represent the differential movement between joints and are critical for evaluating the structural integrity and potential for damage during seismic events.

Table 5.3 : Selected ground motions relative displacements at selected points.

Joints	Axis	Maximum relative displacement (CM)		
		Kocaeli	Chi-chi	El Centro
804L	X	-1.006	-1.004	-2.643
804L	Y	0.223	0.223	1.566
804L	Z	-0.004	-0.004	-0.016
704L	X	-0.495	-0.491	-3.015
704L	Y	0.057	0.056	2.365
704L	Z	-0.011	-0.011	-0.050
604L	X	-0.060	-0.059	0.903
604L	Y	-0.007	-0.006	-0.751
604L	Z	-0.010	-0.010	-0.088
504L	X	-0.266	-0.217	-4.973
504L	Y	-0.051	-0.009	-4.088
504L	Z	0.111	-0.304	0.949
404L	X	-0.373	-0.332	-4.894
404L	Y	0.159	0.192	4.481
404L	Z	0.032	-0.305	0.862
304L	X	-0.392	-0.457	-5.558
304L	Y	0.090	0.015	4.658
304L	Z	0.021	0.767	-0.837
204L	X	-0.341	-0.191	-5.901
204L	Y	-0.026	0.113	7.598
204L	Z	-0.033	-1.430	-0.877

Figure 5.10 presents the absolute displacement time histories of the selected ground motions, recorded at specific critical points on the structure. This chart illustrates the variation in displacement response under different seismic inputs.

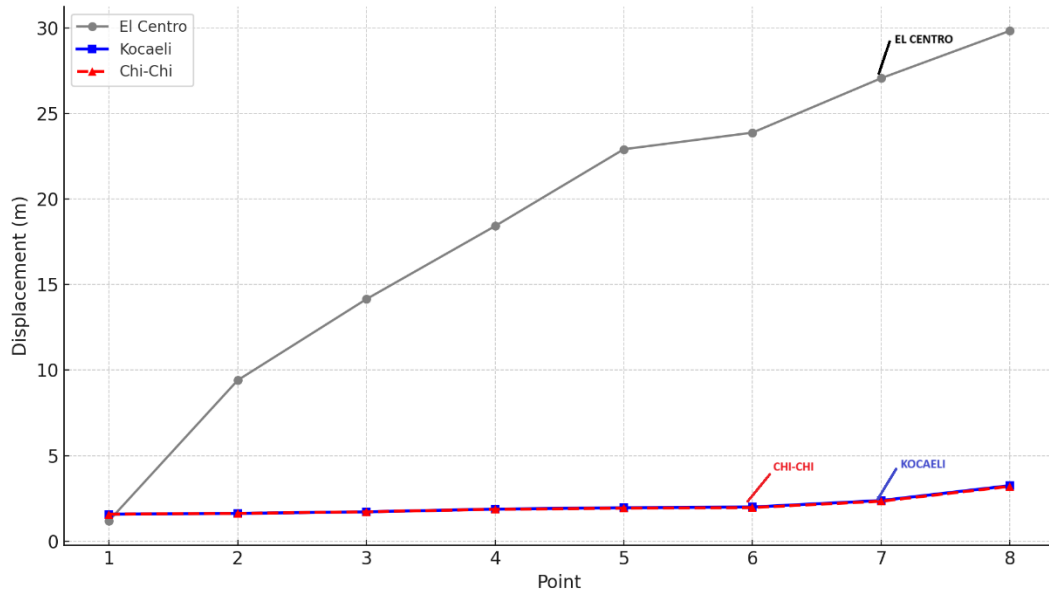


Figure 5.10 : Chart of selected ground motions absolute displacements at selected points.

Figures 5.11 to 5.18 illustrate the displacements of the selected joints at each story under the El Centro seismic excitation. Table 5.4 presents the strength report of the selected key joints subjected to El Centro seismic motion. Furthermore, Figures 5.19 to 5.21 present the base shear resulting from the three seismic motions (Chi-Chi, El centro and Kocaeli), providing a comparative view of the structural response under different earthquake scenarios.

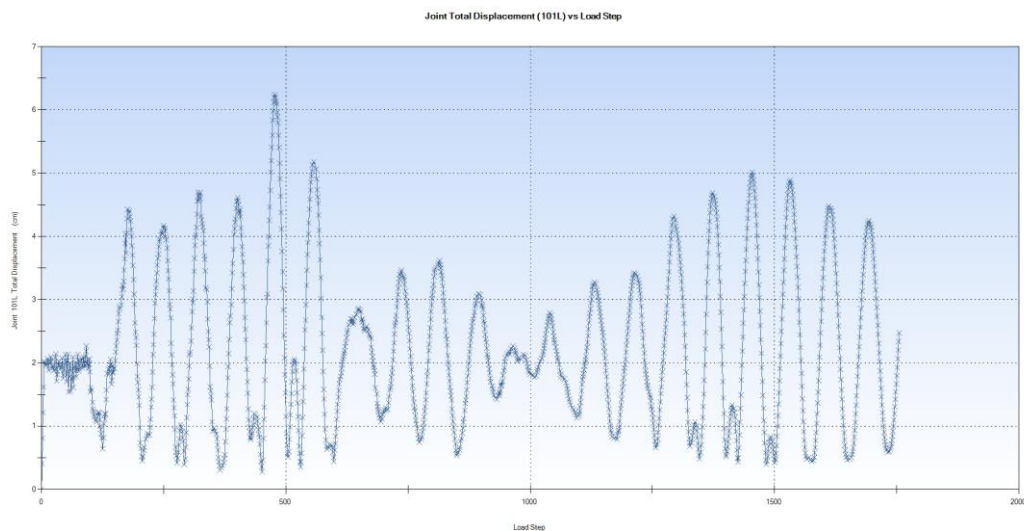


Figure 5.11 : Joint (104L) seismic load steps and displacements for El Centro seismic movement.

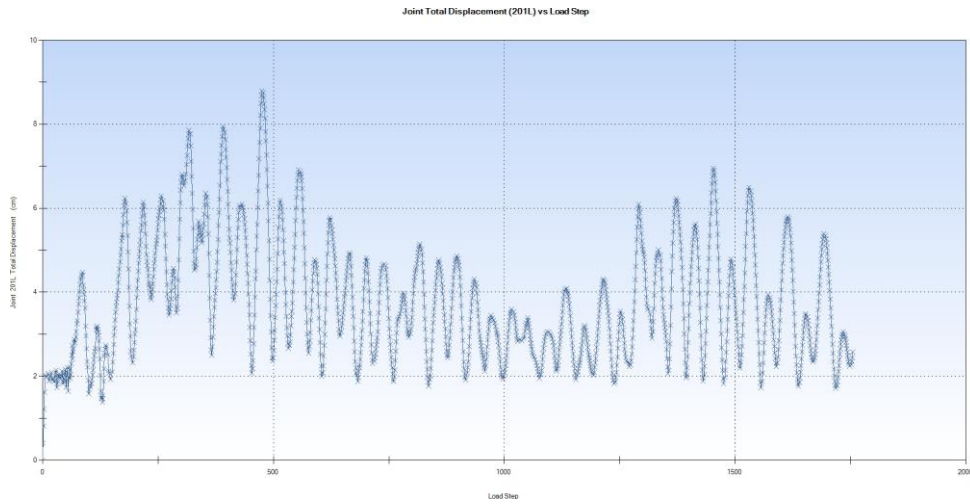


Figure 5.12 : Joint (204L) seismic load steps and displacements for El Centro seismic movement.

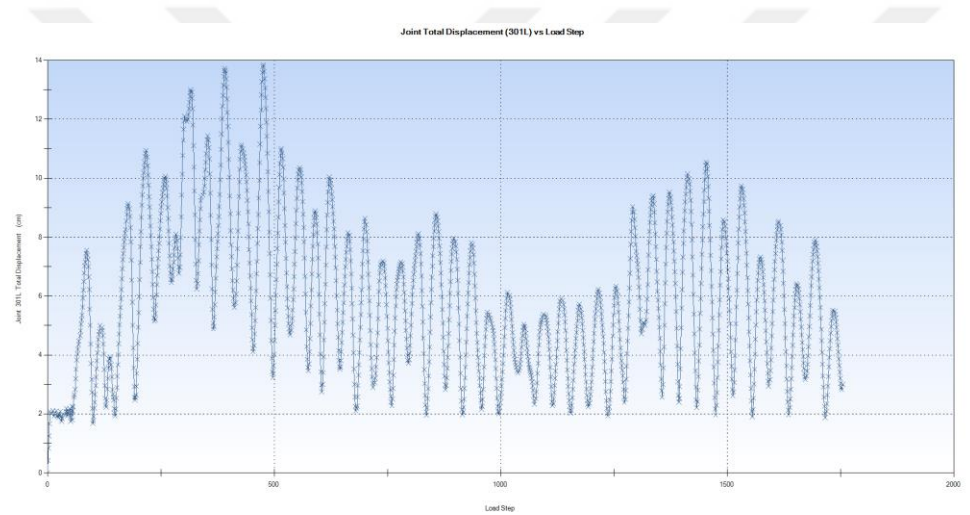


Figure 5.13 : Joint (304L) seismic load steps and displacements for El Centro seismic movement.

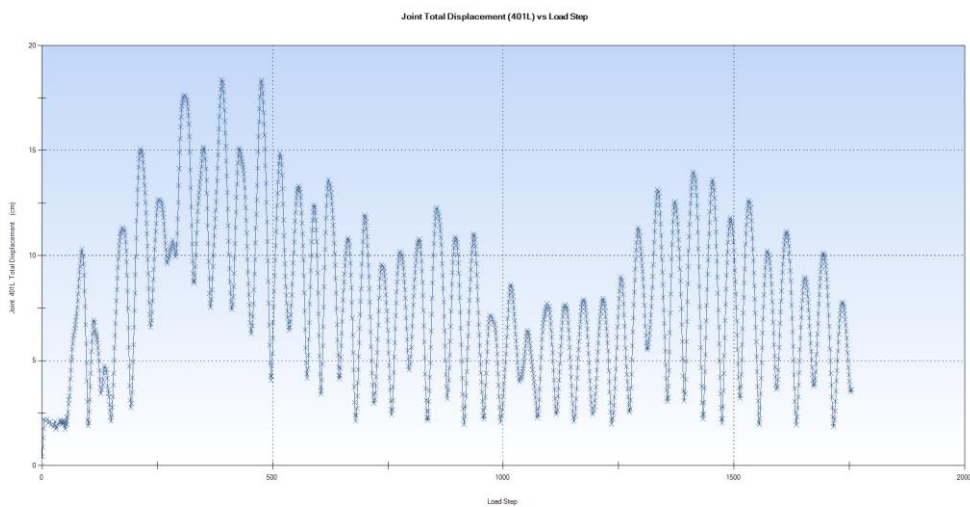


Figure 5.14 : Joint (404L) seismic load steps and displacements for El Centro seismic movement.

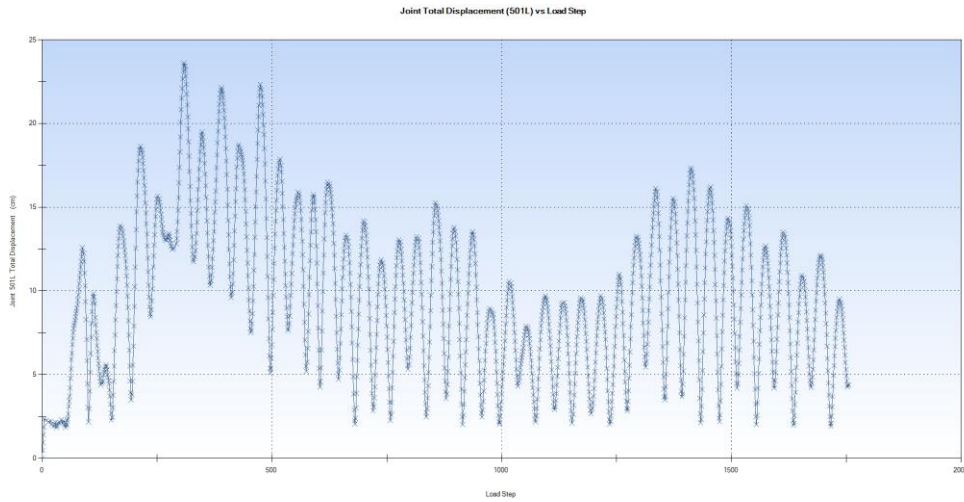


Figure 5.15 : Joint (504L) seismic load steps and displacements for El Centro seismic movement.

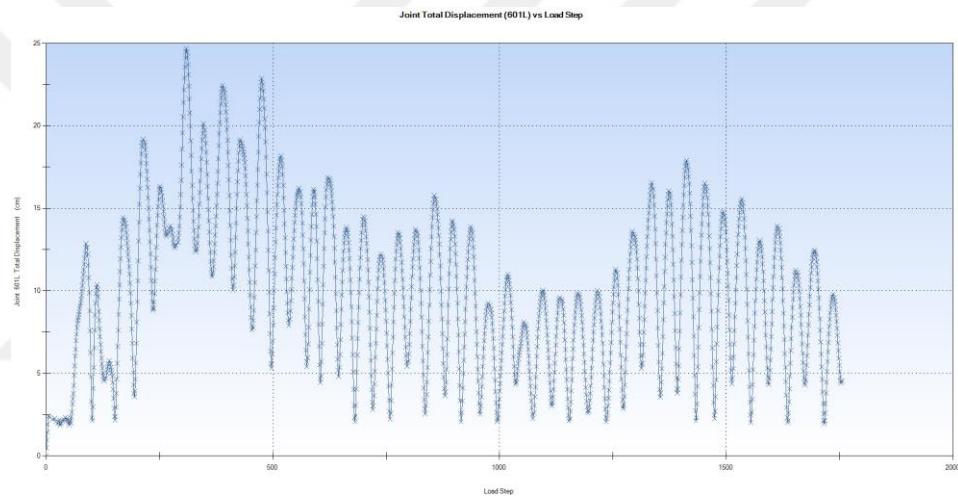


Figure 5.16 : Joint (604L) seismic load steps and displacements for El Centro seismic movement.

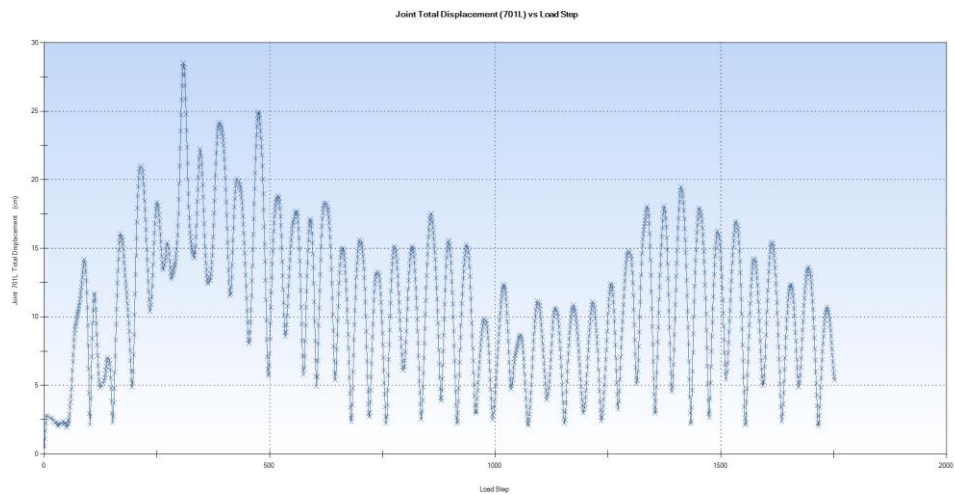


Figure 5.17 : Joint (704L) seismic load steps and displacements for El Centro seismic movement.

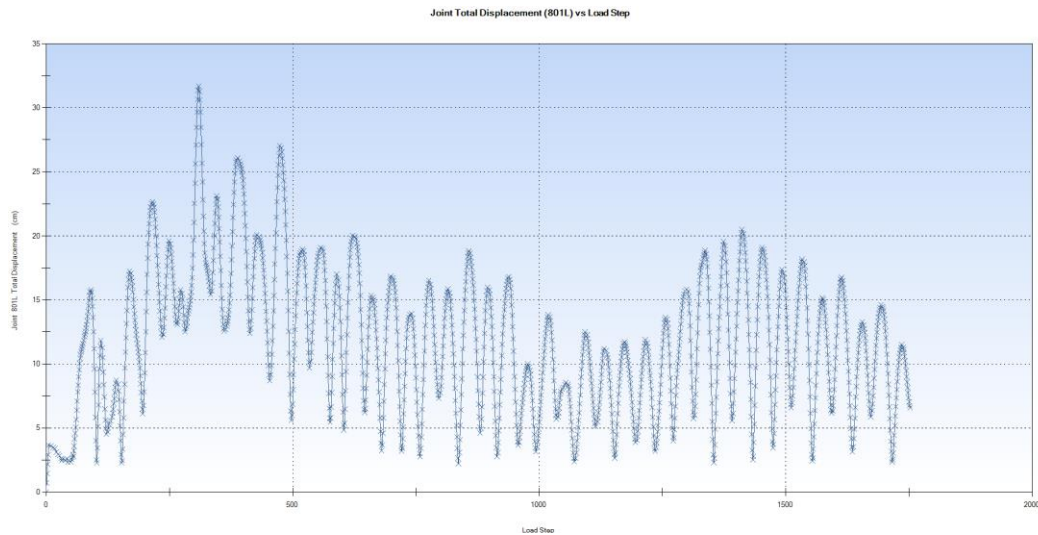


Figure 5.18 : Joint (804L) seismic load steps and displacements for El Centro seismic movement.

Table 5.4 : Strength report of selected key joints under El Centro seismic excitation.

The Joint name	Applied stress			ASD (allowable stress design)			UC ratio
	The Axial stress	Out of plane stress	In plane stress	The Axial stress	Out of plane moment	In plane moment	
Joint 104L	5.36	0.698	0.381	312.19	796.21	1284.62	0.00105
Joint 204L	-3.19	-0.210	-3.68	943.31	1932.99	3641.32	0.00121
Joint 304L	-3.38	0.532	1.51	1282.12	1425.29	3698.80	0.00107
Joint 404L	-7.78	-0.819	-0.83	1185.58	1883.56	2885.01	0.00161
Joint 504L	-18.44	-2.198	9.12	1387.82	2730.97	4392.92	0.00315
Joint 604L	-12.87	1.297	4.59	1287.81	2962.84	4759.74	0.00186

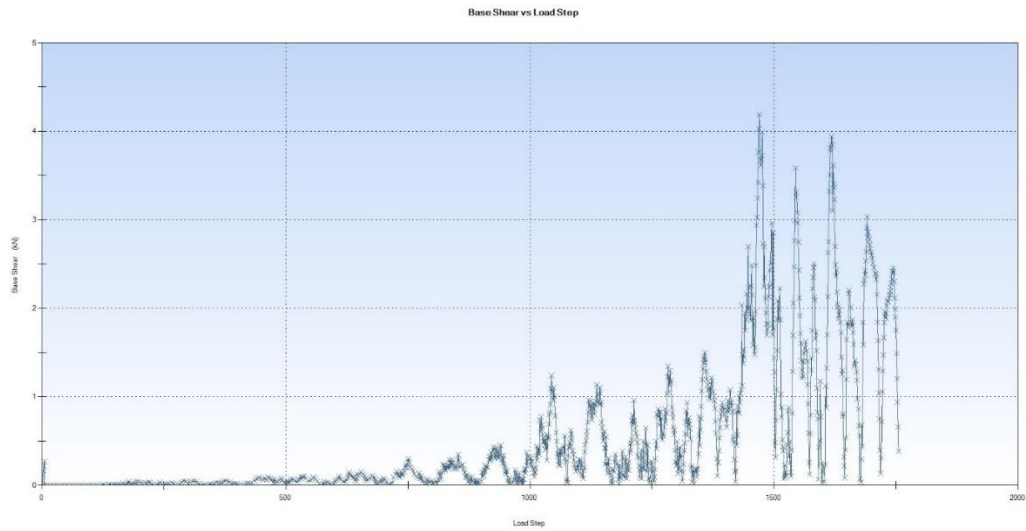


Figure 5.19 : Load steps and base shear of the seismic motion (Chi-Chi).

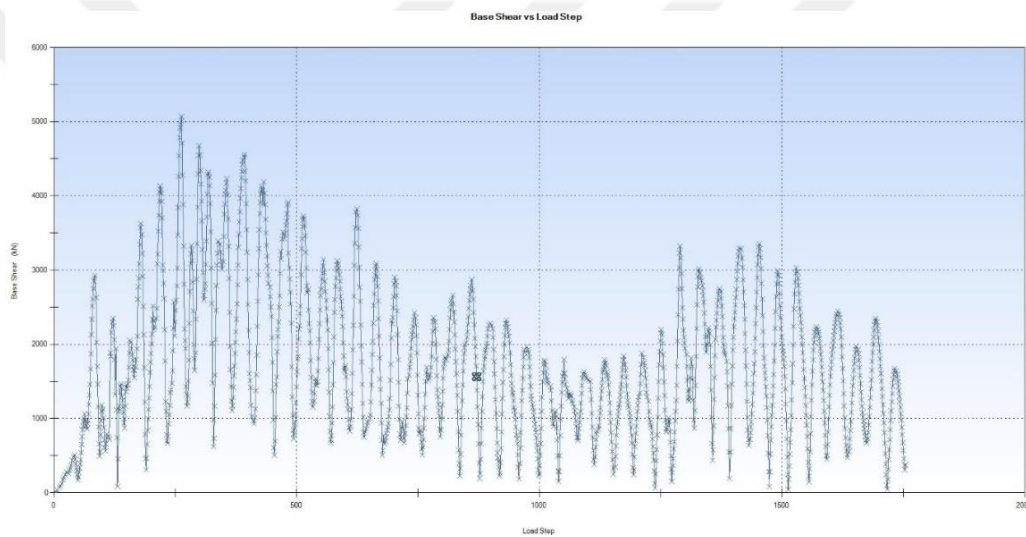


Figure 5.20 : Load steps and base shear of the seismic motion (El Centro).

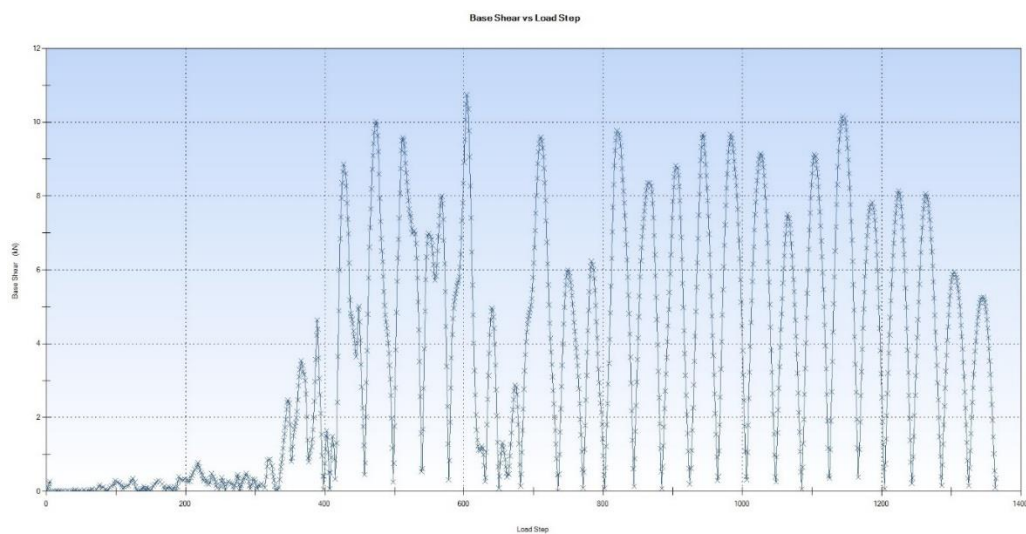


Figure 5.21 : Load steps and base shear of the seismic motion (kocaeli).

5.3 Spectral Wave Fatigue Analysis

Figure 5.22 presents the fatigue damage affecting the joints in the fixed offshore structure under wave action. Figures 5.23 to 5.88 illustrate the joints subjected to fatigue, Stress vs. Cycles Curve for Braces and the transfer Function Analysis for Braces while Tables 5.5 to 5.28 provide damage value, and the expected Service life for each joint.

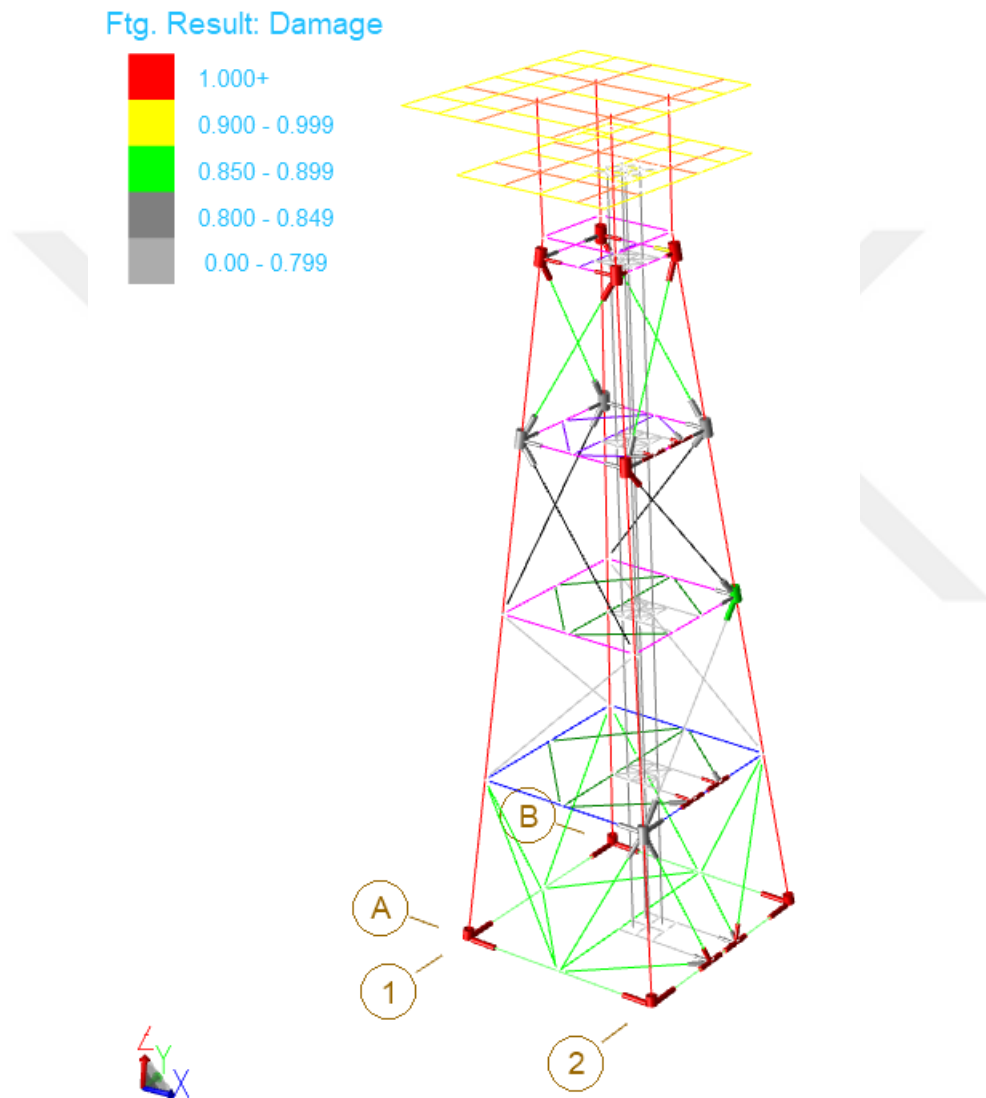


Figure 5.22 : The Joints that experienced fatigue due to environmental loads.

The following figures and tables present a detailed assessment of the fatigue performance and damage evaluation for joint 504L and its associated braces. Figure 5.23 provides a general view of joint 504L, while Table 5.5 summarizes its estimated damage levels and remaining service life. Fatigue behavior is further examined through the stress versus cycle (S–N) curves and transfer function analyses for braces 504L-0064 and 402L–504L, as illustrated in Figures 5.24 through 5.27. Table 5.6 complements these findings by providing additional data on damage accumulation and service life prediction for joint 504L under various loading conditions.

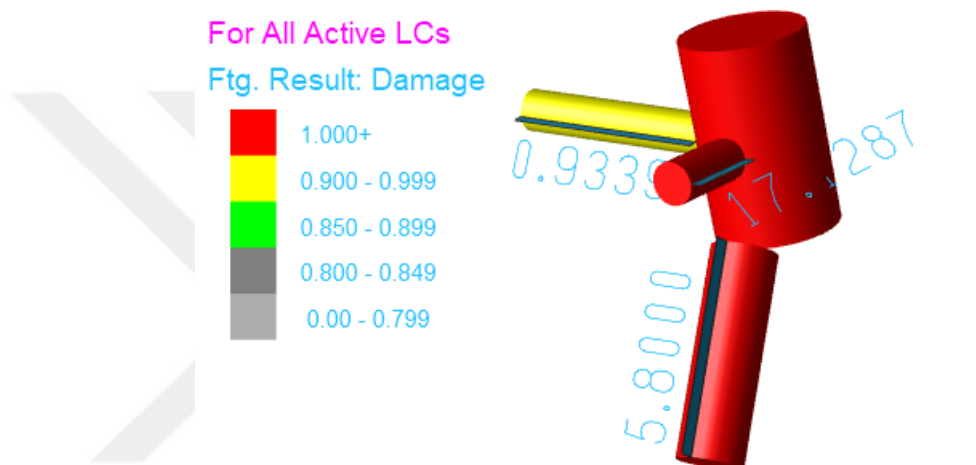


Figure 5.23 : Joint 504L

The structural elements examined include both chord and brace members. The chord member, spanning between joints 604L and 504L, has a diameter of 106.68 cm and a thickness of 3.81 cm. The stress concentration factors (SCFs) for this member were calculated as 2.602 at the crown and 2.955 at the saddle. The SCFs due to in-plane bending and out-of-plane bending were found to be 1.500 and 2.178, respectively.

In contrast, the brace member, connecting joints 504L and 0064, has a diameter of 35.60 cm and a thickness of 1.00 cm. This member exhibited relatively higher SCF values, with 3.412 at the crown and 4.287 at the saddle. The SCFs due to in-plane and out-of-plane moments were 2.146 and 3.278, respectively.

Table 5.5 : Damage and service life of joint (504L).

Damage point	The CHORD element		The BRACE element	
	Damage value	Service life	Damage value	Service life
Top	0.360788	110.869	0.520935	76.785
Top-Left	4.326626	9.245	8.501579	4.705
Left	8.216753	4.868	17.128681	2.335
Bottom-Left	1.995485	20.045	4.216455	9.487
Bottom	0.037215	1074.848	0.046150	866.730
Bottom-Right	0.583598	68.540	1.498296	26.697
Right	2.173978	18.399	5.542091	7.217
Top-Right	0.635019	62.990	1.486678	26.906
Max. value	8.216753	4.868	17.128681	2.335

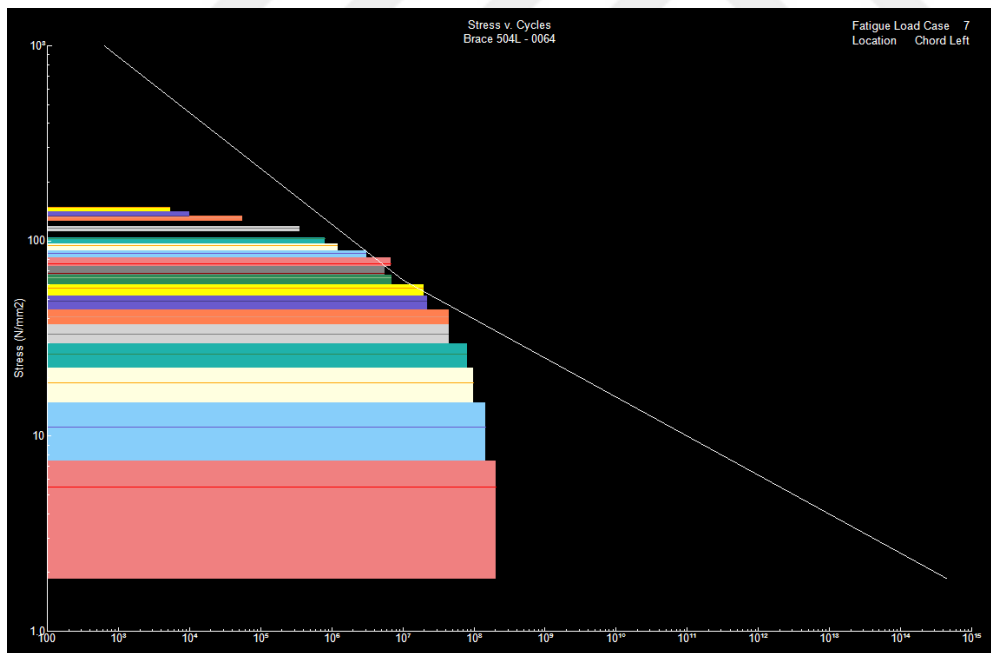


Figure 5.24 : Stress vs. cycles curve for brace 504L -0064.

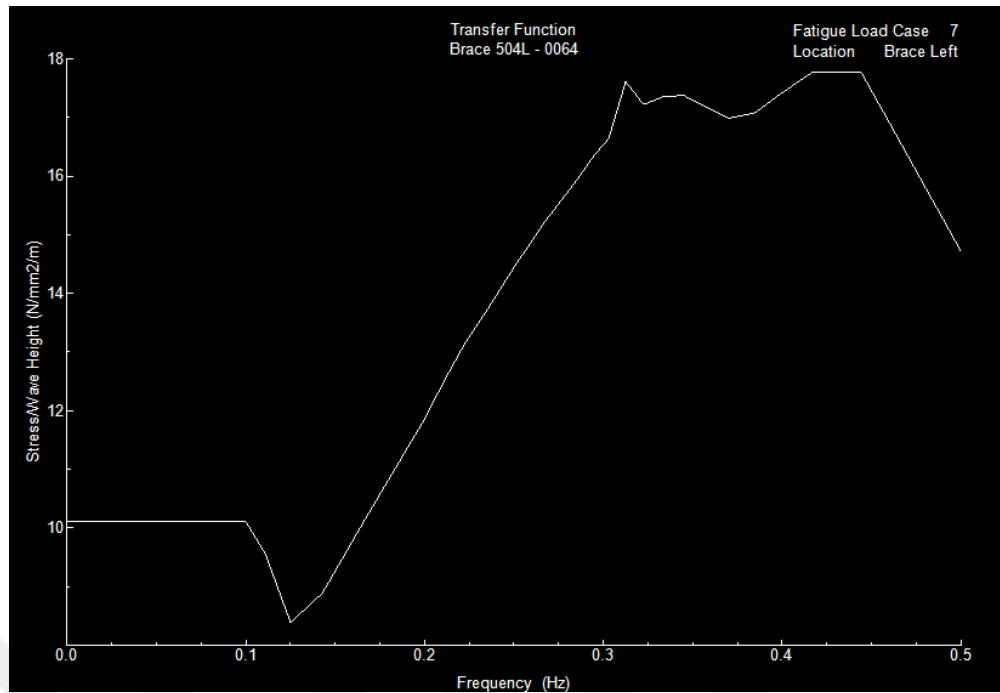


Figure 5.25 : Transfer function analysis for brace 504L -0064.

The structural evaluation also included a chord member between joints 404L and 504L, with a diameter of 106.68 cm and a wall thickness of 3.81 cm. The stress concentration factors (SCFs) for this member were determined as 2.381 at the crown and 2.435 at the saddle. The SCFs resulting from in-plane and out-of-plane bending moments were 1.500 and 1.921, respectively.

Additionally, the brace member connecting joints 402L and 504L has a diameter of 55.90 cm and a thickness of 1.30 cm. The SCFs for this brace were found to be 2.568 (crown) and 2.454 (saddle), while the values due to in-plane and out-of-plane bending were 2.606 and 2.255, respectively.

Table 5.6 : Damage and service life of joint (504L).

Damage point	The CHORD element		The BRACE element	
	Damage value	Service life	Damage value	Service life
Top	3.257841	12.278	5.800015	6.897
Top-Left	1.663043	24.052	2.242803	17.835
Left	0.511857	78.147	0.237798	168.210
Bottom-Left	0.281681	142.004	0.333840	119.818
Bottom	0.444698	89.949	1.008631	39.658
Bottom-Right	0.791557	50.533	0.716938	55.793
Right	1.602221	24.965	0.791953	50.508
Top-Right	2.993438	13.363	3.490257	11.460
Max. value	3.257841	12.278	5.800015	6.897

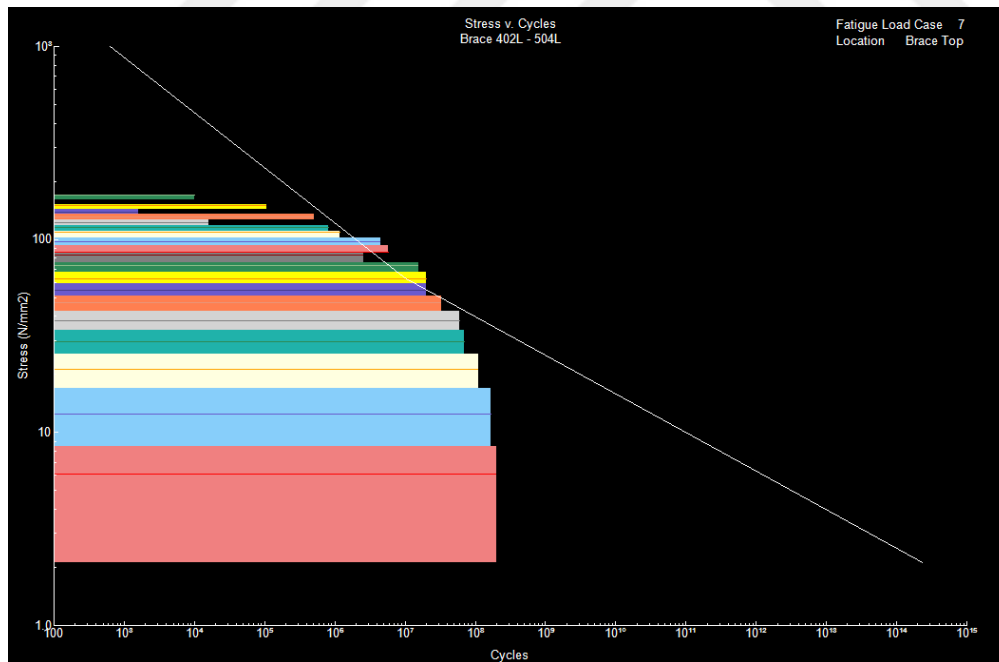


Figure 5.26 : Stress vs. cycles curve for brace 402L - 504L.

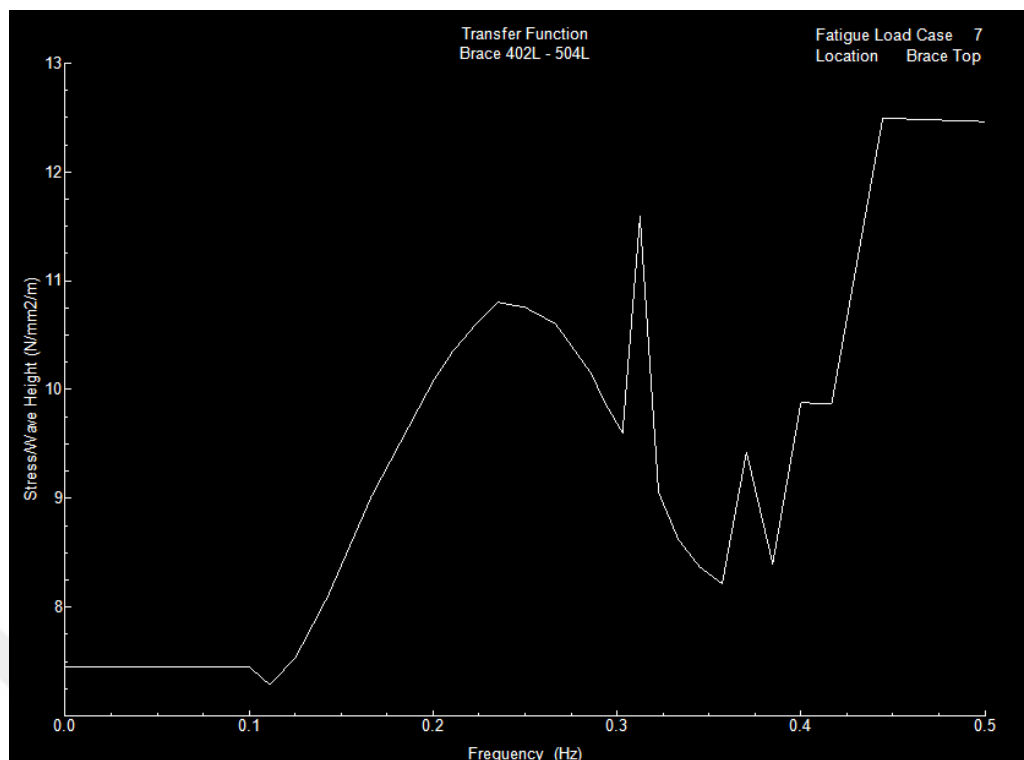


Figure 5.27 : Transfer function analysis for brace 402L - 504L.

The following figures and table provide a comprehensive evaluation of the structural performance of joint 0013 and its connecting brace. Figure 5.28 illustrates the overall configuration of joint 0013, while Table 5.7 presents the calculated fatigue damage and estimated service life. The fatigue response of brace 0040–0013 is further analyzed through the stress versus cycles (S–N) curve shown in Figure 5.29, along with the corresponding transfer function analysis depicted in Figure 5.30.

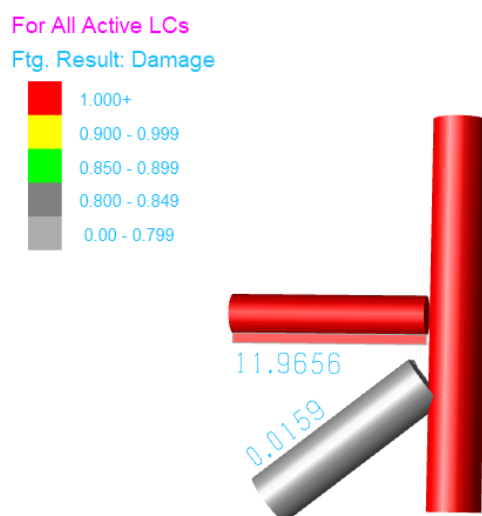


Figure 5.28 : Joint 0013

Another chord member, spanning between joints 0012 and 0013, has a diameter of 45.72 cm and a thickness of 1.30 cm. The stress concentration factors (SCFs) for axial loading were found to be 6.000 at both the crown and saddle positions. The SCFs due to in-plane bending and out-of-plane bending were 3.632 and 6.000, respectively.

The corresponding brace member, connecting joints 0040 and 0013, has a diameter of 32.40 cm and a wall thickness of 1.00 cm. The SCFs under axial load were recorded as 5.149 at the crown and 5.222 at the saddle. For bending effects, the SCFs were 2.975 (in-plane) and 6.000 (out-of-plane).

Table 5.7 : Damage and service life of joint (0013).

Damage point	The CHORD element		The BRACE element	
	Damage value	Service life	Damage value	Service life
Top	11.965578	3.343	4.625713	8.647
Top-Left	2.935378	13.627	1.144314	34.955
Left	0.001121	35.68E3	569.68E-6	70.21E3
Bottom-Left	0.321191	124.536	0.110169	363.080
Bottom	2.721158	14.700	0.962078	41.577
Bottom-Right	0.353847	113.043	0.123918	322.793
Right	449.50E-6	88.99E3	197.33E-6	202.7E3
Top-Right	2.685530	14.895	1.027118	38.944
Max. value	11.965578	3.343	4.625713	8.647

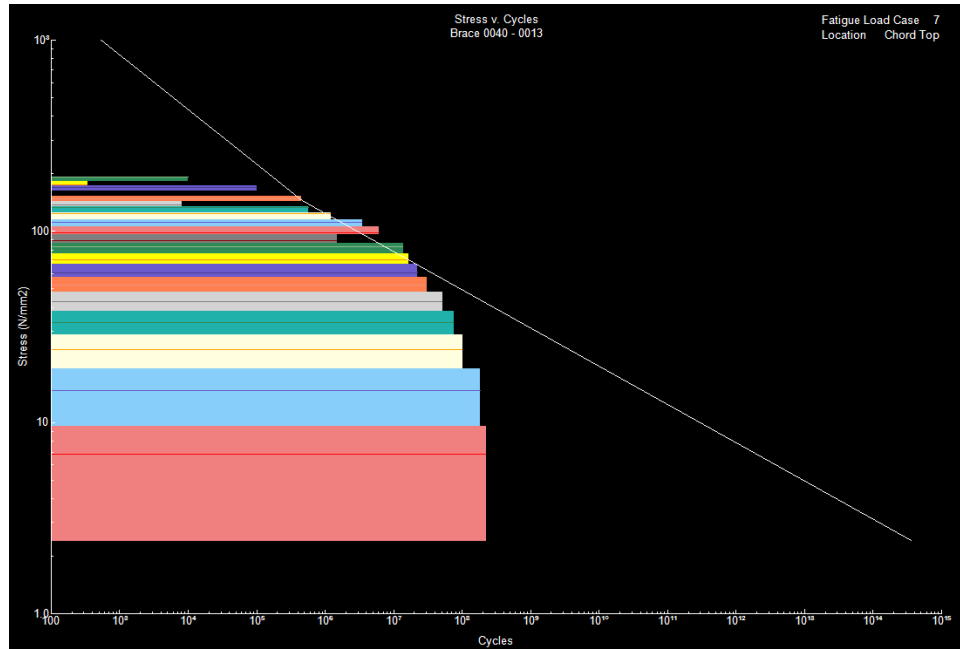


Figure 5.29 : Stress vs. cycles curve for brace 0040 – 0013.

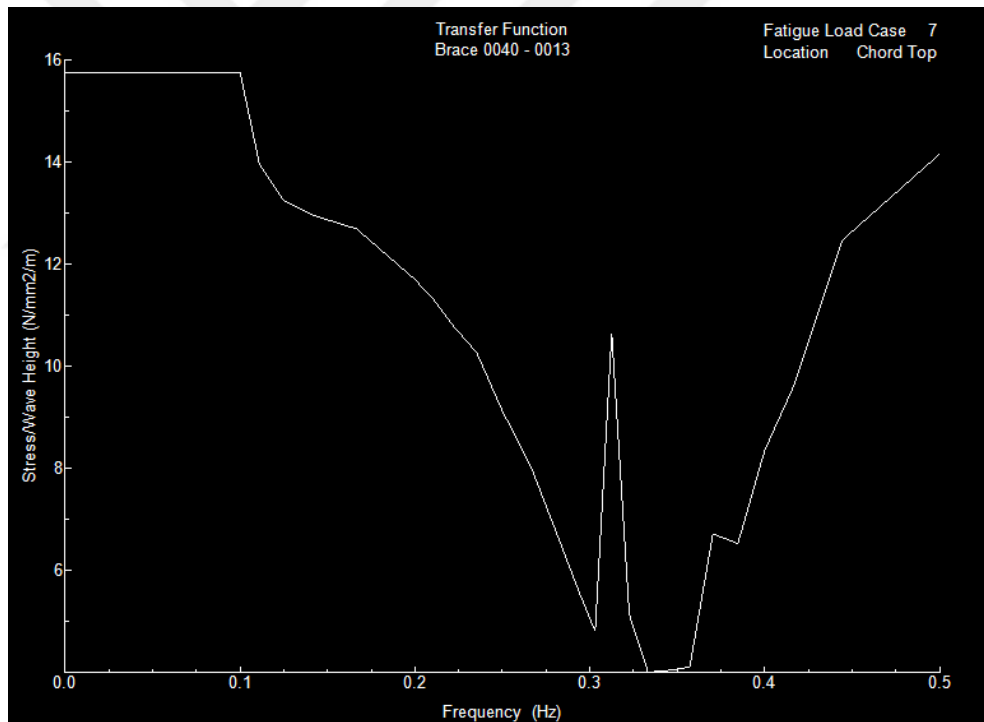


Figure 5.30 : Transfer function analysis for brace 0040 – 0013.

This section presents an in-depth analysis of joint 502L and its associated braces under fatigue loading conditions. Figure 5.31 provides a visual representation of the joint, followed by Table 5.8, which outlines the estimated fatigue damage and service life. The performance of brace 0073–502L is examined through the S–N curve in Figure 5.32 and the transfer function analysis in Figure 5.33. Additional evaluations for joint 502L are shown in Table 5.9, accompanied by fatigue analysis results for braces 502L–

0046 and 502L–401L. Figures 5.34 through 5.37 illustrate the stress versus cycles relationships and transfer function responses for these braces, while Tables 5.9 and 5.10 provide corresponding damage and service life estimates.

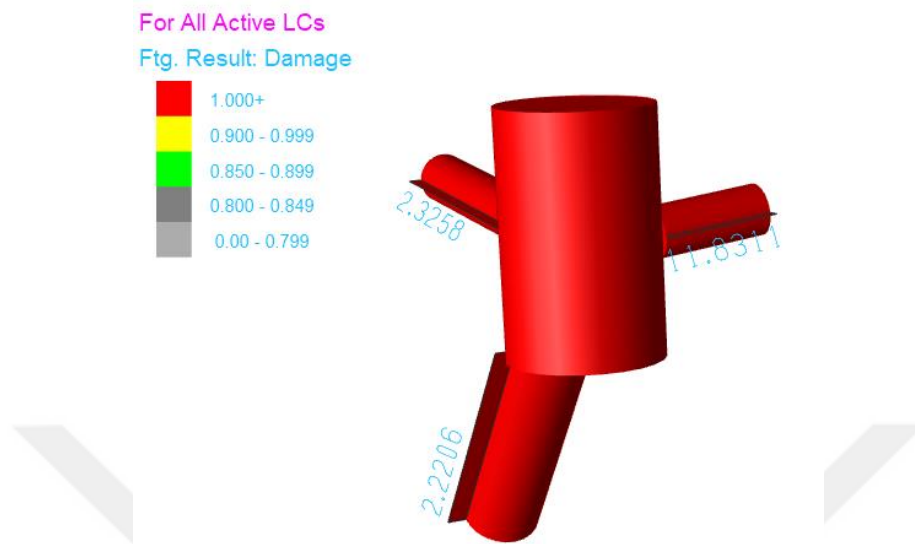


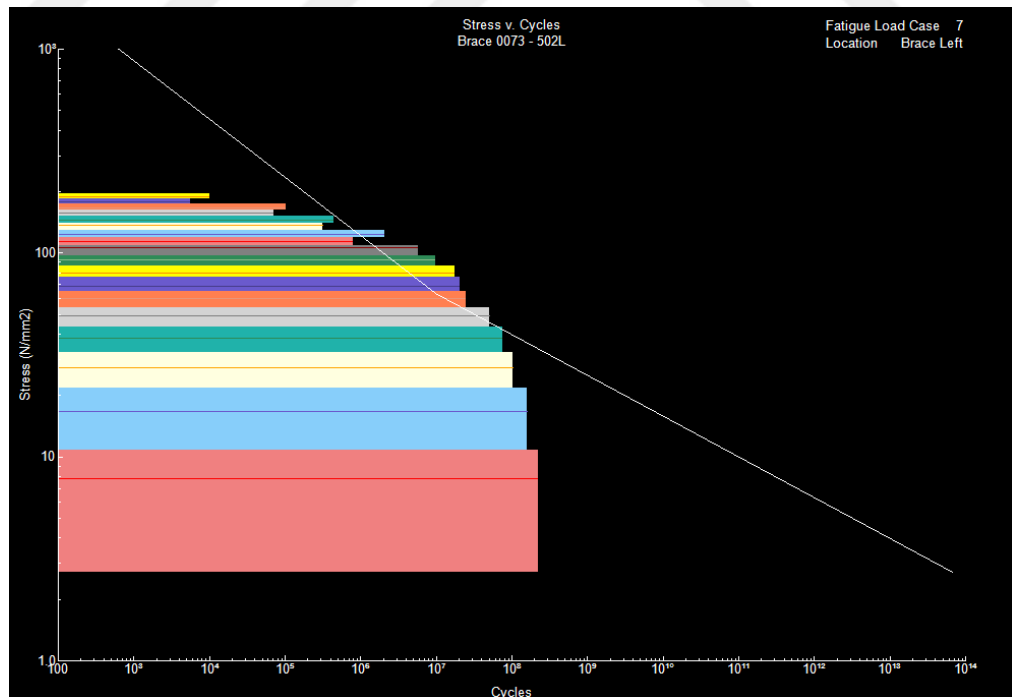
Figure 5.31 : Joint 502L.

The chord member between joints 602L and 502L has a diameter of 106.68 cm and a thickness of 3.81 cm. The stress concentration factors (SCFs) due to axial loading were calculated as 2.243 at the crown and 3.218 at the saddle. The SCFs resulting from in-plane and out-of-plane bending moments were 1.500 and 2.021, respectively.

The corresponding brace member, extending between joints 0073 and 502L, features a diameter of 35.60 cm and a wall thickness of 1.00 cm. The SCFs for axial loading were 3.049 (crown) and 5.461 (saddle), while those related to in-plane and out-of-plane bending were 2.146 and 3.042, respectively.

Table 5.8 : Damage and service life of joint (502L).

Damage point	The CHORD element		The BRACE element	
	Damage value	Service life	Damage value	Service life
Top	0.070835	564.694	0.118329	338.041
Top-Left	0.792626	50.465	2.234603	17.900
Left	4.001954	9.995	10.278523	3.892
Bottom-Left	1.600897	24.986	4.008020	9.980
Bottom	0.038240	1046.035	0.062384	641.188
Bottom-Right	0.819285	48.823	2.275655	17.577
Right	4.683672	8.540	11.831148	3.381
Top-Right	2.114774	18.915	5.175107	7.729
Max. value	4.683672	8.540	11.831148	3.381

**Figure 5.32 : Stress vs. cycles curve for brace 0073 - 502L**

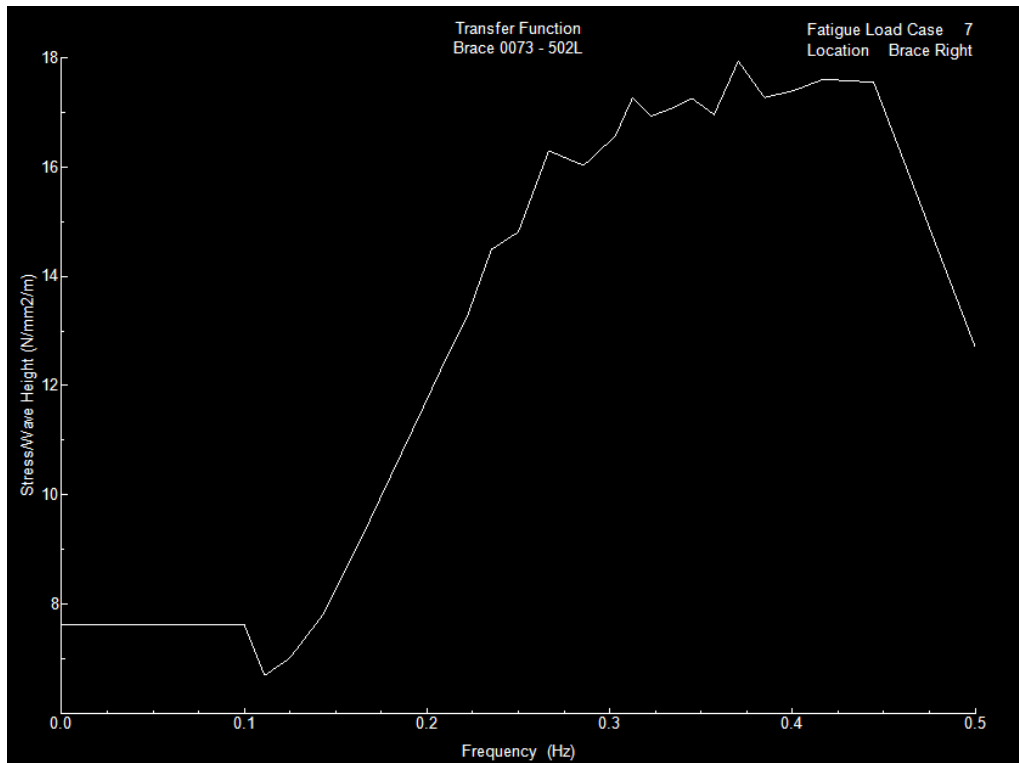


Figure 5.33 : Transfer Functions Analysis for Brace 0073 - 502L.

The chord member connecting joints 602L and 502L is characterized by a diameter of 106.68 cm and a thickness of 3.81 cm. The stress concentration factors (SCFs) for axial loading are 2.560 at the crown and 2.986 at the saddle. Regarding bending, the SCFs were found to be 1.500 for in-plane and 2.159 for out-of-plane moments.

The associated brace member, spanning from joint 502L to 0046, has a diameter of 35.60 cm and a wall thickness of 1.00 cm. Its axial SCFs were calculated as 3.370 at the crown and 4.425 at the saddle, while the SCFs due to in-plane and out-of-plane bending were 2.146 and 3.251, respectively.

Table 5.9 : Damage and service life of joint (502L).

Damage point	The CHORD element		The BRACE element	
	Damage value	Service life	Damage value	Service life
Top	0.026739	1495.941	0.037987	1052.991
Top-Left	0.471304	84.871	0.978775	40.867
Left	1.038289	38.525	2.325762	17.199
Bottom-Left	0.180771	221.275	0.398422	100.396
Bottom	0.001201	33.29E3	0.001339	29.87E3
Bottom-Right	0.092239	433.654	0.230490	173.543
Right	0.397820	100.548	0.995949	40.163
Top-Right	0.090987	439.622	0.207364	192.898
Max. value	1.038289	38.525	2.325762	17.199

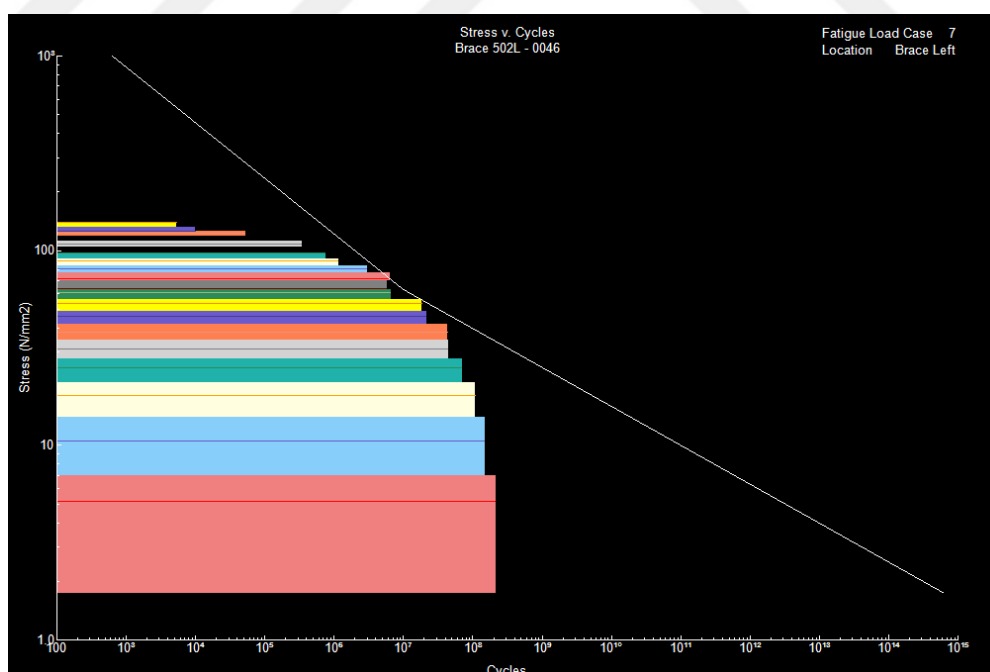


Figure 5.34 : Stress vs. cycles curve for brace 502L – 0046.

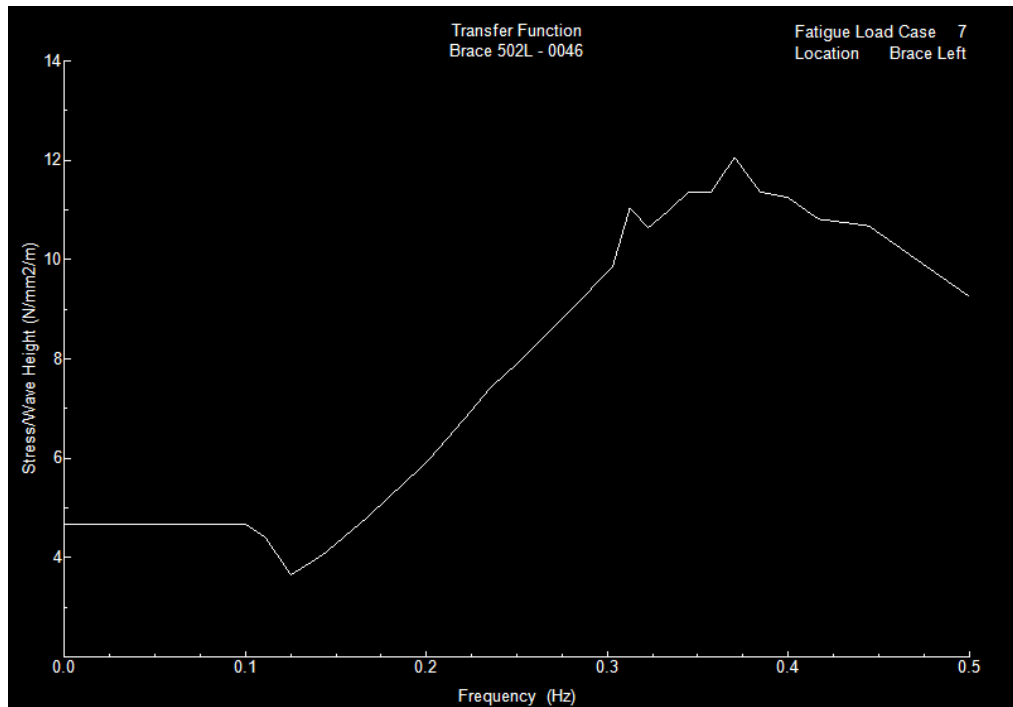


Figure 5.35 : Transfer function analysis for brace 502L – 0046.

The chord member between joints 402L and 502L has a diameter of 106.68 cm and a thickness of 3.81 cm. The stress concentration factors (SCFs) due to axial forces were determined to be 2.441 at the crown and 2.511 at the saddle. The SCFs corresponding to in-plane and out-of-plane bending were calculated as 1.500 and 1.849, respectively.

The corresponding brace member, connecting joints 502L and 401L, is designed with a diameter of 55.90 cm and a wall thickness of 1.30 cm. The SCFs under axial loading were 2.701 (crown) and 2.554 (saddle), while the SCFs for in-plane and out-of-plane moments were 2.606 and 2.171, respectively.

Table 5.10 : Damage and service life of joint (502L).

Damage point	The CHORD element		The BRACE element	
	Damage value	Service life	Damage value	Service life
Top	1.116148	35.838	1.962236	20.385
Top-Left	1.389946	28.778	1.471101	27.191
Left	1.655963	24.155	1.027264	38.938
Bottom-Left	0.614897	65.051	0.555214	72.044
Bottom	0.225515	177.372	0.458024	87.332
Bottom-Right	0.951801	42.026	0.816499	48.990
Right	2.220588	18.013	1.383535	28.911
Top-Right	1.808230	22.121	1.867059	21.424
Max. value	2.220588	18.013	1.962236	20.385

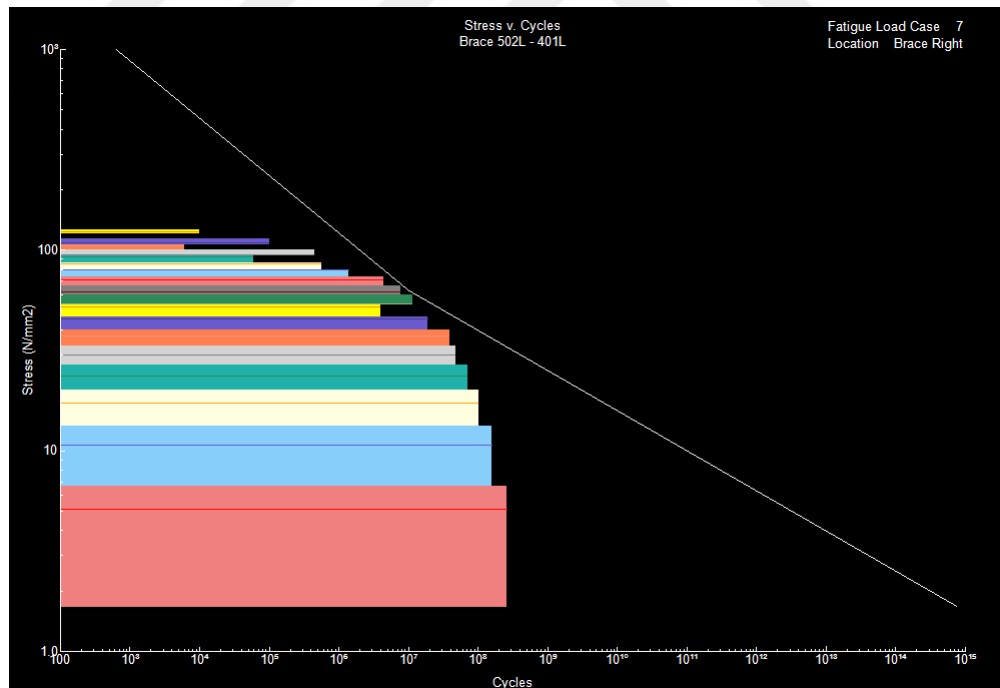


Figure 5.36 : Stress vs. cycles curve for brace 502L - 401L.

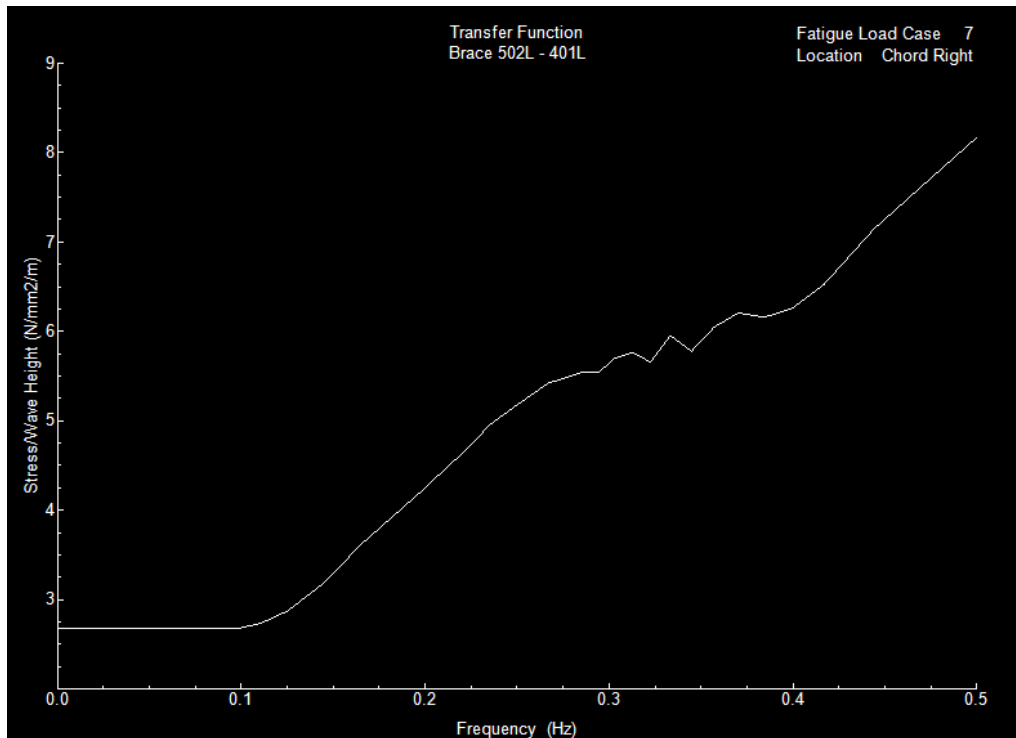


Figure 5.37 : Transfer function analysis for brace 502L - 401L.

The structural performance of joint 0012 is assessed through the following figures and table. Figure 5.38 displays the joint configuration, while Table 5.11 presents the corresponding damage values and estimated service life. The fatigue behavior of brace 0039–0012 is further investigated using the stress versus cycles (S–N) curve shown in Figure 5.39 and the transfer function analysis illustrated in Figure 5.40.



Figure 5.38 : Joint 0012.

The chord member between joints 0012 and 0013 has a diameter of 45.72 cm and a thickness of 1.30 cm. The stress concentration factors (SCFs) due to axial forces were

determined to be 6.000 at the crown and 6.000 at the saddle. The SCFs corresponding to in-plane and out-of-plane bending were calculated as 3.632 and 6.000, respectively. The corresponding brace member, connecting joints 0039 and 0012, is designed with a diameter of 32.40 cm and a wall thickness of 1.00 cm. The SCFs under axial loading were 5.148 (crown) and 5.224 (saddle), while the SCFs for in-plane and out-of-plane moments were 2.975 and 6.000, respectively.

Table 5.11 : Damage and service life of joint (0012).

Damage point	The CHORD element		The BRACE element	
	Damage value	Service life	Damage value	Service life
Top	9.691558	4.127	3.722421	10.746
Top-Left	2.131376	18.767	0.812898	49.207
Left	0.00025329	157900	0.00010999	363700
Bottom-Left	0.310882	128.666	0.109250	366.132
Bottom	2.346816	17.044	0.831377	48.113
Bottom-Right	0.281291	142.202	0.096761	413.388
Right	0.00076078	52580	0.00039255	101900
Top-Right	2.345224	17.056	0.913114	43.806
Max. value	9.691558	4.127	3.722421	10.746

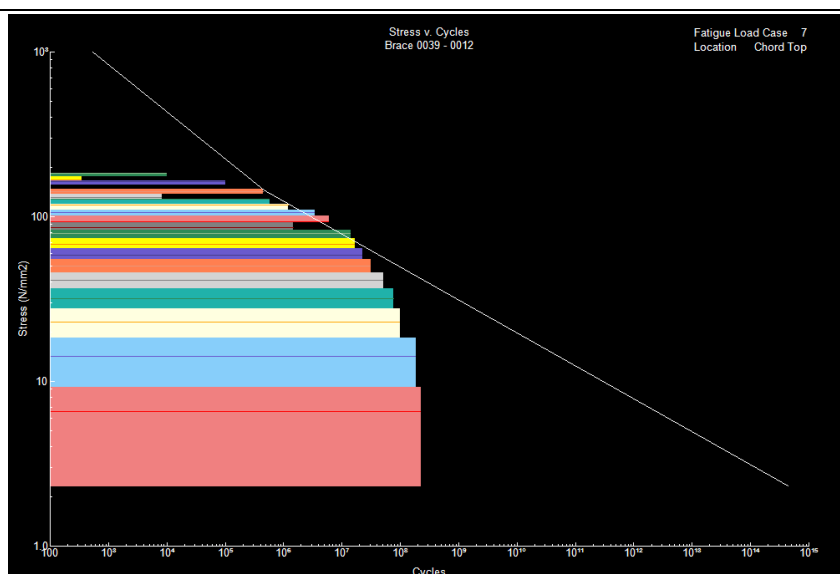


Figure 5.39 : Stress vs. cycles curve for brace 0039 – 0012.

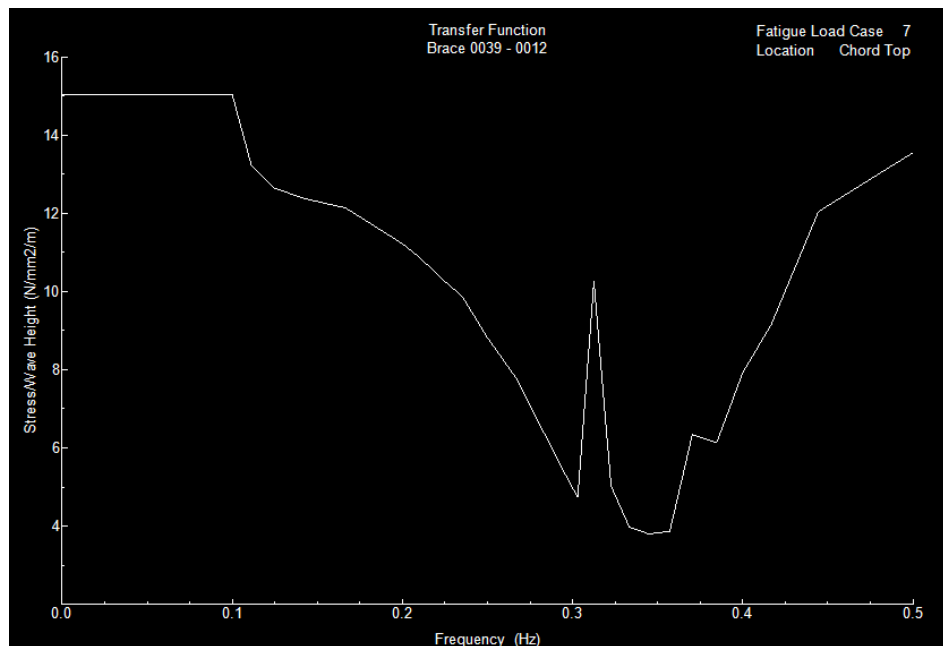


Figure 5.40 : Transfer function analysis for brace 0039 – 0012.

The fatigue performance of joint 102L is examined through a series of figures and tables detailing its structural response. Figure 5.41 illustrates the geometry of joint 102L, and Table 5.12 presents the initial assessment of damage and service life. Brace 102L–0001 is evaluated using the stress versus cycles (S–N) curve in Figure 5.42 and the transfer function analysis in Figure 5.43. Further data on joint degradation are summarized in Table 5.13, alongside fatigue evaluations for brace 0006–102L, which are depicted in Figures 5.44 and 5.45.

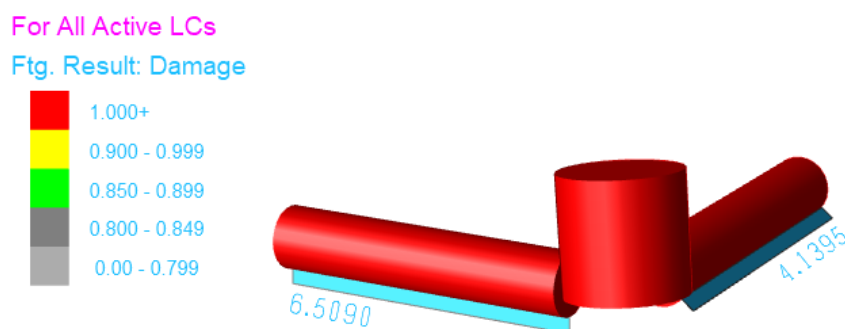


Figure 5.41 : Joint 102L.

The chord member between joints 202L and 102L has a diameter of 106.68 cm and a thickness of 3.81 cm. The stress concentration factors (SCFs) due to axial forces were determined to be 6.000 at the crown and 6.000 at the saddle. The SCFs corresponding to in-plane and out-of-plane bending were calculated as 2.933 and 6.000, respectively.

The corresponding brace member, connecting joints 102L and 0001, is designed with a diameter of 55.88 cm and a wall thickness of 2.54 cm. The SCFs under axial loading were 5.109 (crown) and 6.000 (saddle), while the SCFs for in-plane and out-of-plane moments were 2.775 and 6.000, respectively.

Table 5.12 : Damage and service life of joint (102L).

Damage point	The CHORD element		The BRACE element	
	Damage value	Service life	Damage value	Service life
Top	6.509009	6.145	1.797537	22.253
Top-Left	6.083941	6.575	2.543608	15.726
Left	5.334290	7.499	3.216678	12.435
Bottom-Left	4.713492	8.486	1.956871	20.441
Bottom	4.540714	8.809	1.200526	33.319
Bottom-Right	4.894660	8.172	2.037977	19.627
Right	5.613400	7.126	3.387038	11.810
Top-Right	6.305307	6.344	2.644448	15.126
Max. value	6.509009	6.145	3.387038	11.810

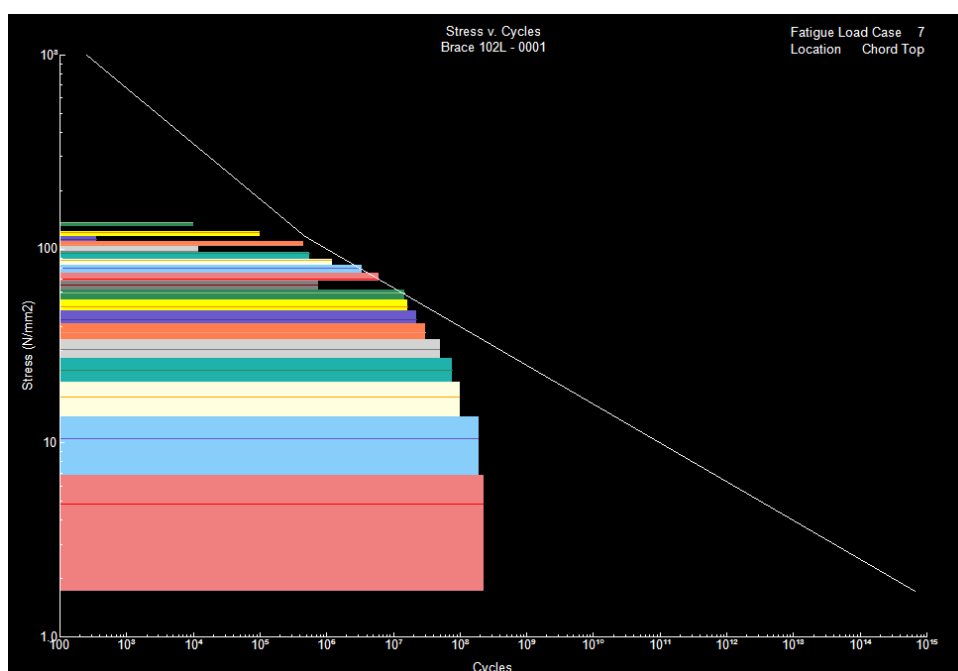


Figure 5.42 : Stress vs. cycles curve for brace 102L – 0001.

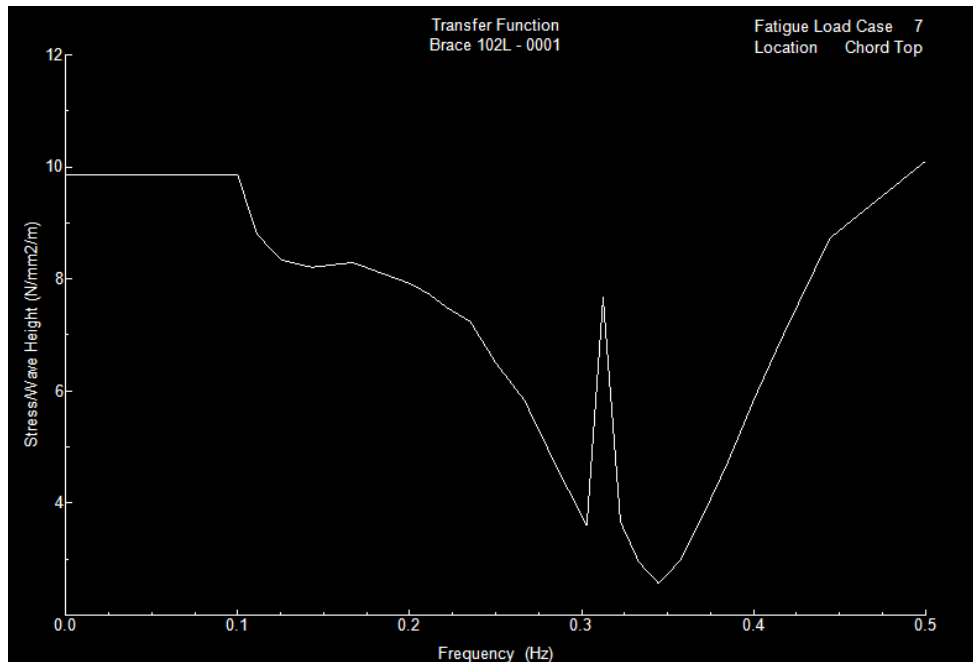


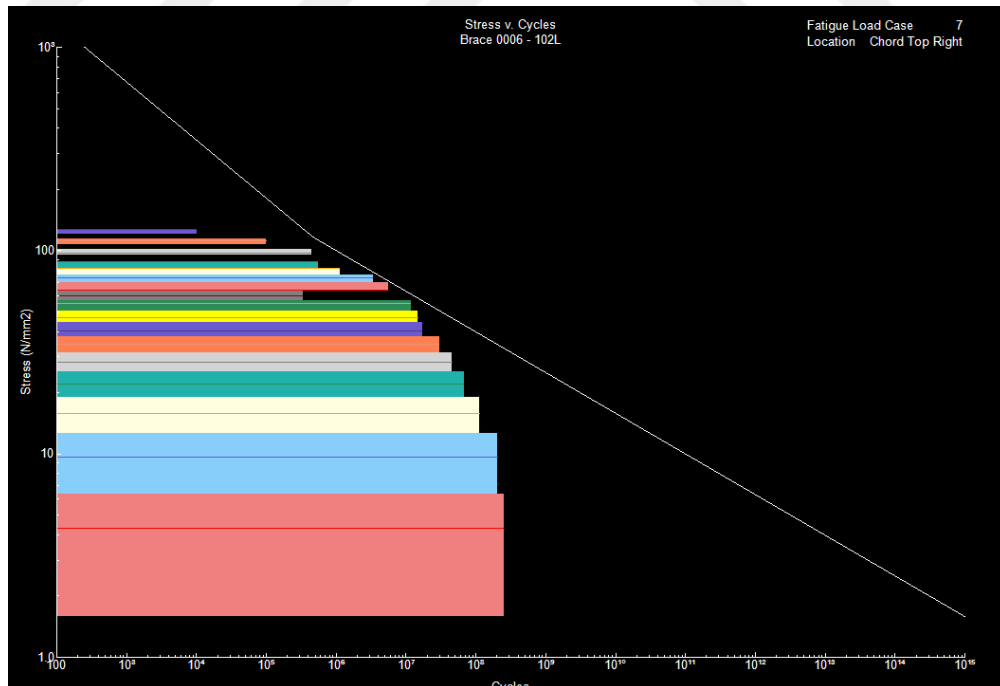
Figure 5.43 : Transfer function analysis for brace 102L – 0001.

The chord member between joints 202L and 102L has a diameter of 106.68 cm and a thickness of 3.81 cm. The stress concentration factors (SCFs) due to axial forces were determined to be 6.000 at the crown and 6.000 at the saddle. The SCFs corresponding to in-plane and out-of-plane bending were calculated as 2.933 and 6.000, respectively.

The corresponding brace member, connecting joints 0006 and 102L, is designed with a diameter of 55.88 cm and a wall thickness of 2.54 cm. The SCFs under axial loading were 5.109 (crown) and 6.000 (saddle), while the SCFs for in-plane and out-of-plane moments were 2.775 and 6.000, respectively.

Table 5.13 : Damage and service life of joint (102L).

Damage point	The CHORD element		The BRACE element	
	Damage value	Service life	Damage value	Service life
Top	3.625244	11.034	1.039100	38.495
Top-Left	2.595182	15.413	1.068839	37.424
Left	2.730711	14.648	1.645079	24.315
Bottom-Left	3.800512	10.525	1.613387	24.793
Bottom	4.029515	9.927	1.164287	34.356
Bottom-Right	3.372028	11.862	1.410880	28.351
Right	3.487962	11.468	2.102471	19.025
Top-Right	4.139530	9.663	1.775788	22.525
Max. value	4.139530	9.663	2.102471	19.025

**Figure 5.44 : Stress vs. cycles curve for brace 0006 - 102L.**

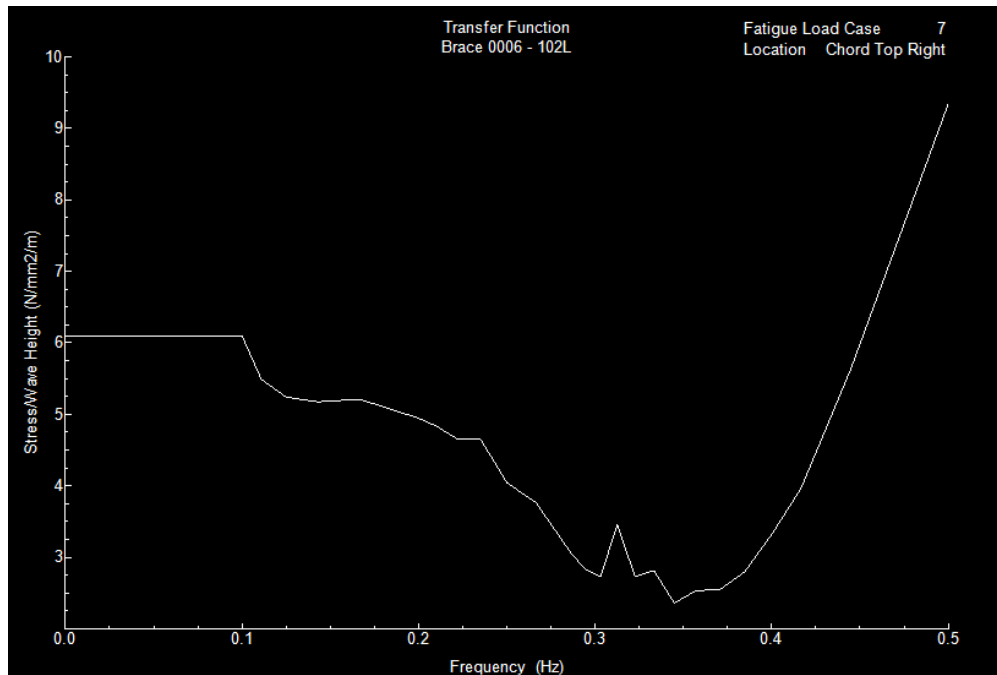


Figure 5.45 : Transfer function analysis for brace 0006 - 102L.

Joint 104L and its connecting braces are analyzed to evaluate fatigue behavior and service life. Figure 5.46 shows the configuration of joint 104L, with Table 5.14 presenting the corresponding damage assessment and life expectancy. The behavior of brace 0000–104L is explored through the S–N curve in Figure 5.47 and the transfer function in Figure 5.48. Additional results are provided in Table 5.15, which support the evaluation of brace 104L–0005. Its fatigue performance is illustrated in Figures 5.49 and 5.50.



Figure 5.46 : Joint 104L.

The chord member between joints 104L and 204L has a diameter of 106.68 cm and a thickness of 3.81 cm. The stress concentration factors (SCFs) due to axial forces were

determined to be 6.000 at the crown and 6.000 at the saddle. The SCFs corresponding to in-plane and out-of-plane bending were calculated as 2.933 and 6.000, respectively. The corresponding brace member, connecting joints 0000 and 104L, is designed with a diameter of 55.88 cm and a wall thickness of 2.54 cm. The SCFs under axial loading were 5.109 (crown) and 6.000 (saddle), while the SCFs for in-plane and out-of-plane moments were 2.775 and 6.000, respectively.

Table 5.14 : Damage and service life of joint (104L).

Damage point	The CHORD element		The BRACE element	
	Damage value	Service life	Damage value	Service life
Top	6.467860	6.184	1.790266	22.343
Top-Left	6.064289	6.596	2.537937	15.761
Left	5.263263	7.600	3.173835	12.603
Bottom-Left	4.555040	8.781	1.892208	21.139
Bottom	4.307096	9.287	1.135486	35.227
Bottom-Right	4.634483	8.631	1.927789	20.749
Right	5.400841	7.406	3.257250	12.280
Top-Right	6.181379	6.471	2.591510	15.435
Max. value	6.467860	6.184	3.257250	12.280

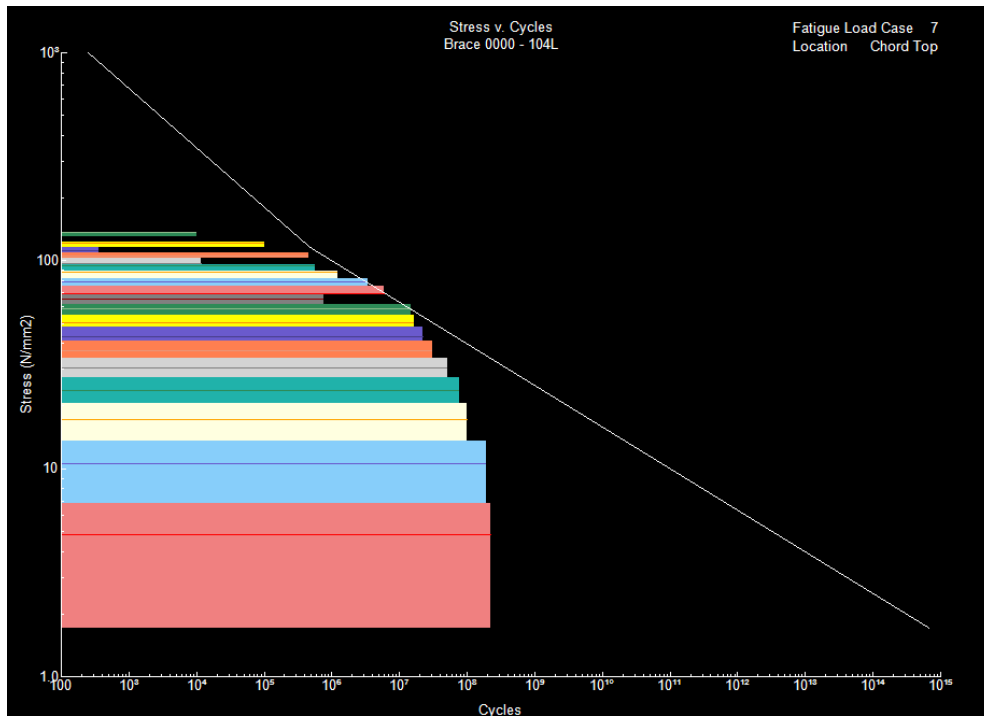


Figure 5.47 : Stress vs. cycles curve for brace 0000 - 104L.

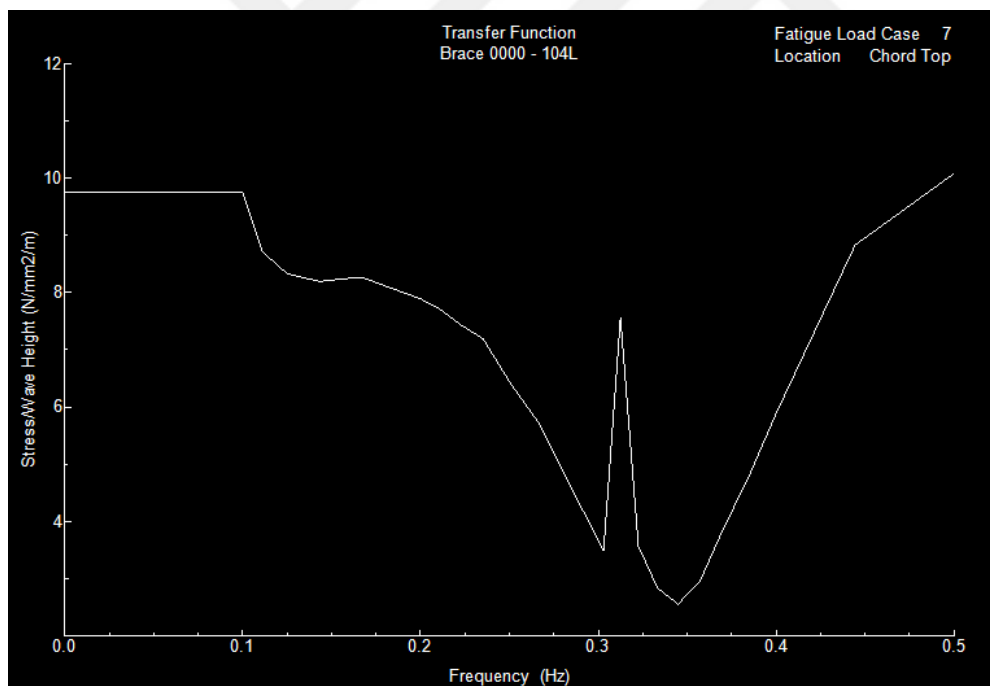


Figure 5.48 : Transfer function analysis for brace 0000 - 104L.

The structural elements examined include both chord and brace members. The chord member, spanning between joints 104L and 204L, has a diameter of 106.68 cm and a thickness of 3.81 cm. The stress concentration factors (SCFs) for this member were calculated as 6.000 at the crown and 6.000 at the saddle. The SCFs due to in-plane bending and out-of-plane bending were found to be 2.933 and 6.000, respectively.

In contrast, the brace member, connecting joints 104L and 0005, has a diameter of 55.88 cm and a thickness of 2.54 cm. This member exhibited relatively high SCF values, with 5.109 at the crown and 6.000 at the saddle. The SCFs due to in-plane and out-of-plane moments were 2.775 and 6.000, respectively.

Table 5.15 : Damage and service life of joint (104L).

Damage point	The CHORD element		The BRACE element	
	Damage value	Service life	Damage value	Service life
Top	3.244111	12.330	0.915914	43.672
Top-Left	3.458251	11.567	1.468141	27.245
Left	3.104488	12.885	1.870833	21.381
Bottom-Left	3.540498	11.298	1.474070	27.136
Bottom	4.711518	8.490	1.373070	29.132
Bottom-Right	4.585385	8.723	1.955272	20.458
Right	3.210278	12.460	1.934678	20.675
Top-Right	2.702669	14.800	1.120023	35.714
Max. value	4.711518	8.490	1.955272	20.458

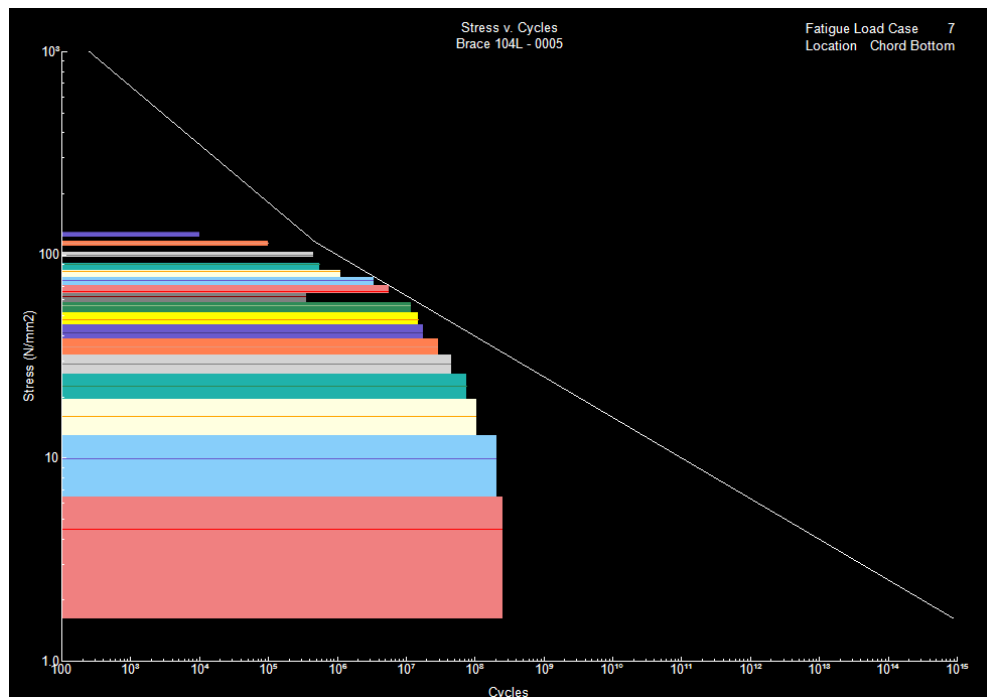


Figure 5.49 : Stress vs. cycles curve for brace 104L – 0005.

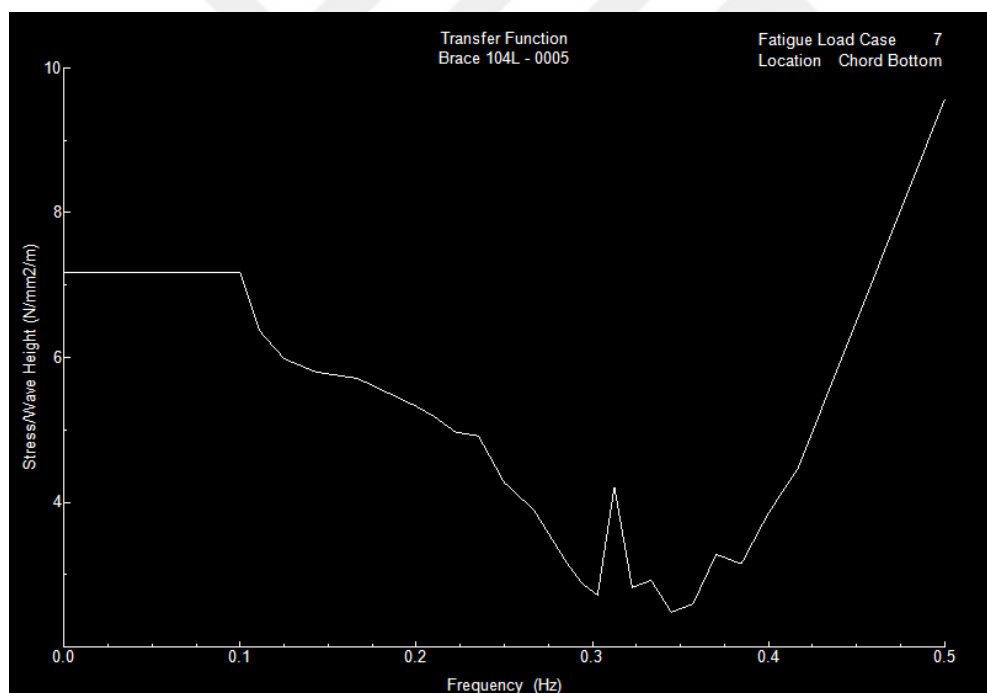


Figure 5.50 : Transfer function analysis for brace 104L – 0005.

The final case in this series focuses on joint 0030. Figure 5.51 illustrates the joint layout, followed by the S–N curve for brace 0044–0030 in Figure 5.52, which highlights its fatigue response. Table 5.16 presents the calculated damage levels and estimated service life, while Figure 5.53 shows the corresponding transfer function analysis for the same brace.

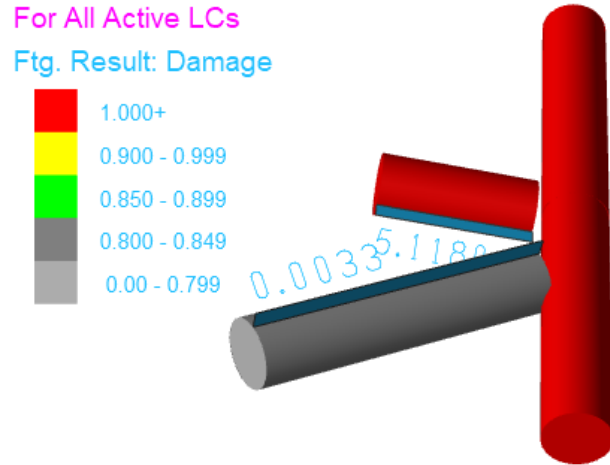


Figure 5.51: Joint 0030.

The chord member between joints 0030 and 0029 has a diameter of 35.60 cm and a thickness of 1.00 cm. The stress concentration factors (SCFs) due to axial forces were determined to be 6.000 at the crown and 6.000 at the saddle. The SCFs corresponding to in-plane and out-of-plane bending were calculated as 3.954 and 6.000, respectively. The corresponding brace member, connecting joints 0044 and 0030, is designed with a diameter of 32.40 cm and a wall thickness of 1.00 cm. The SCFs under axial loading were 4.713 (crown) and 4.713 (saddle), while the SCFs for in-plane and out-of-plane moments were 2.814 and 6.000, respectively.

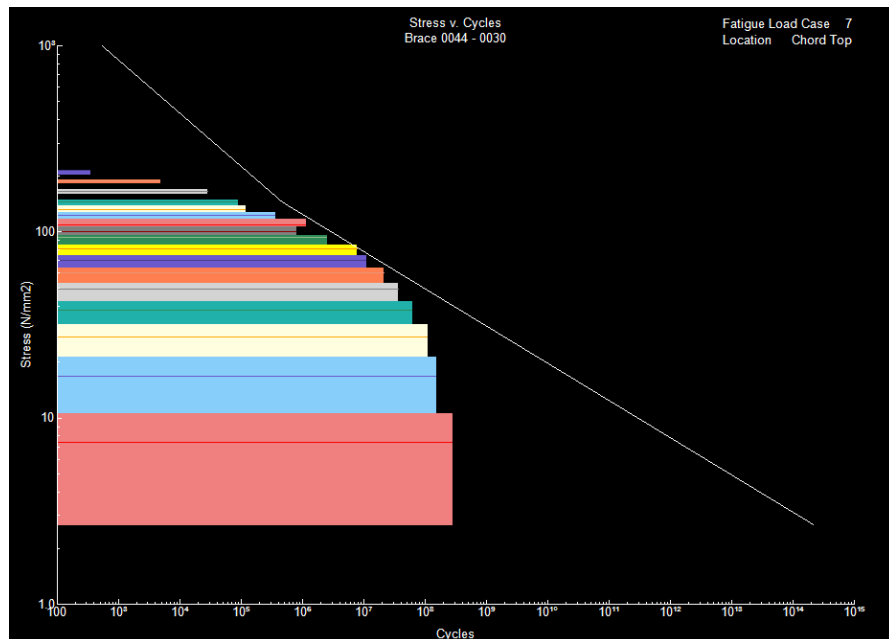


Figure 5.52 : Stress vs. cycles curve for brace 0044 – 0030.

Table 5.16 : Damage and service life of joint (0030).

Damage point	The CHORD element		The BRACE element	
	Damage value	Service life	Damage value	Service life
Top	5.117993	7.816	0.998697	40.052
Top-Left	1.131968	35.337	0.218118	183.387
Left	0.001478	27060	0.00052814	75740
Bottom-Left	0.181890	219.913	0.033458	1195.530
Bottom	1.180735	33.877	0.206601	193.610
Bottom-Right	0.169414	236.108	0.031546	1267.982
Right	0.006490	6163.020	0.002562	15610
Top-Right	1.449346	27.599	0.306428	130.536
Max. value	5.117993	7.816	0.998697	40.052

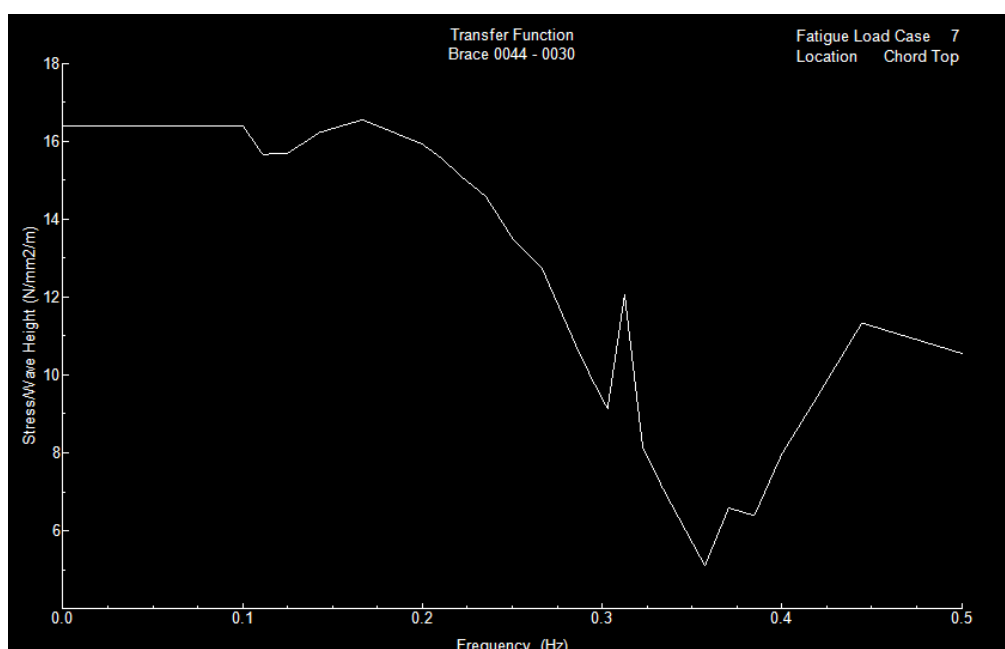


Figure 5.53 : Transfer function analysis for brace 0044 – 0030.

Joint 101L is analyzed through a combination of visual inspection, fatigue assessment, and frequency-domain evaluation. Figure 5.54 illustrates the joint's geometry, while

Table 5.17 presents the initial fatigue damage and estimated service life. The performance of brace 101L–0001 is shown through the S–N curve in Figure 5.55 and the transfer function analysis in Figure 5.56. Further details regarding the joint’s fatigue behavior are provided in Table 5.18, along with additional evaluations for brace 101L–0002, depicted in Figures 5.57 and 5.58.

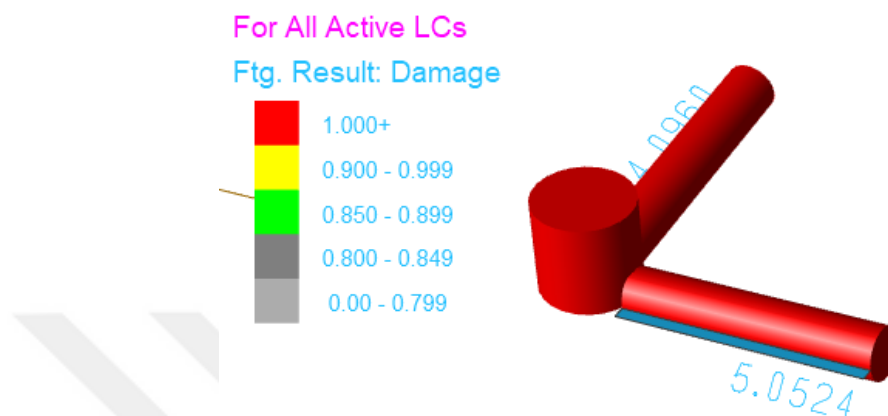


Figure 5.54 : Joint 101L.

The chord member between joints 201L and 101L has a diameter of 106.68 cm and a thickness of 3.81 cm. The stress concentration factors (SCFs) due to axial forces were determined to be 6.000 at the crown and 6.000 at the saddle. The SCFs corresponding to in-plane and out-of-plane bending were calculated as 2.933 and 6.000, respectively. The corresponding brace member, connecting joints 101L and 0001, is designed with a diameter of 55.88 cm and a wall thickness of 2.54 cm. The SCFs under axial loading were 5.109 (crown) and 6.000 (saddle), while the SCFs for in-plane and out-of-plane moments were 2.775 and 6.000, respectively.

Table 5.17 : Damage and service life of joint (101L).

Damage point	The CHORD element		The BRACE element	
	Damage value	Service life	Damage value	Service life
Top	3.440759	11.625	0.934792	42.790
Top-Left	2.441254	16.385	0.991687	40.335
Left	2.032306	19.682	1.224262	32.673
Bottom-Left	2.229318	17.943	0.903665	44.264
Bottom	3.044722	13.137	0.815764	49.034
Bottom-Right	4.265554	9.377	1.818981	21.990
Right	5.052446	7.917	3.046735	13.129
Top-Right	4.631601	8.636	1.977871	20.224
Max. value	5.052446	7.917	3.046735	13.129

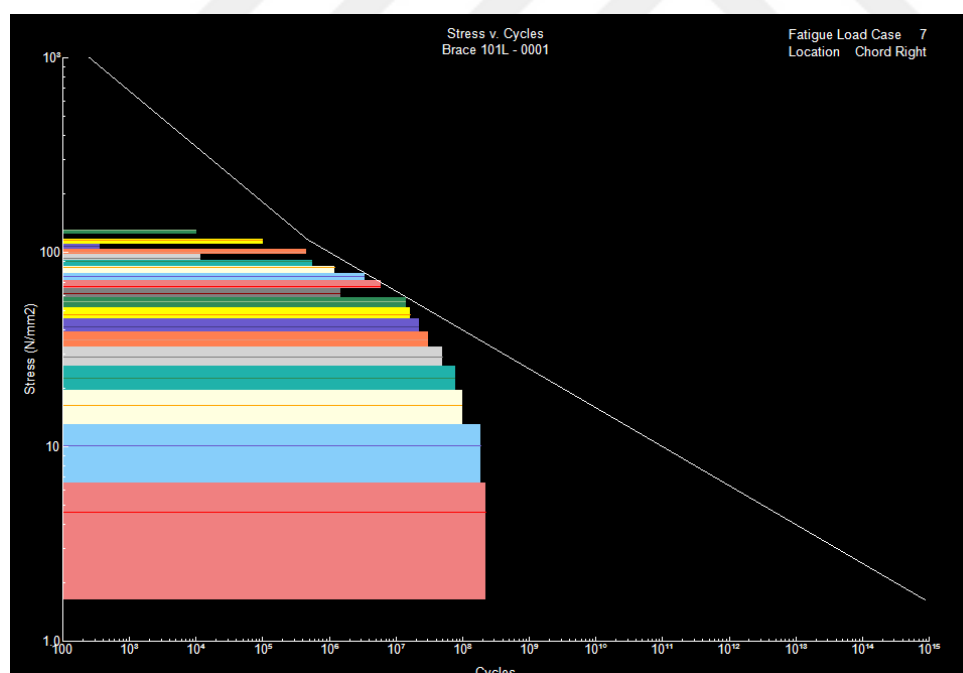


Figure 5.55 : Stress vs. cycles curve for brace 101L – 0001.

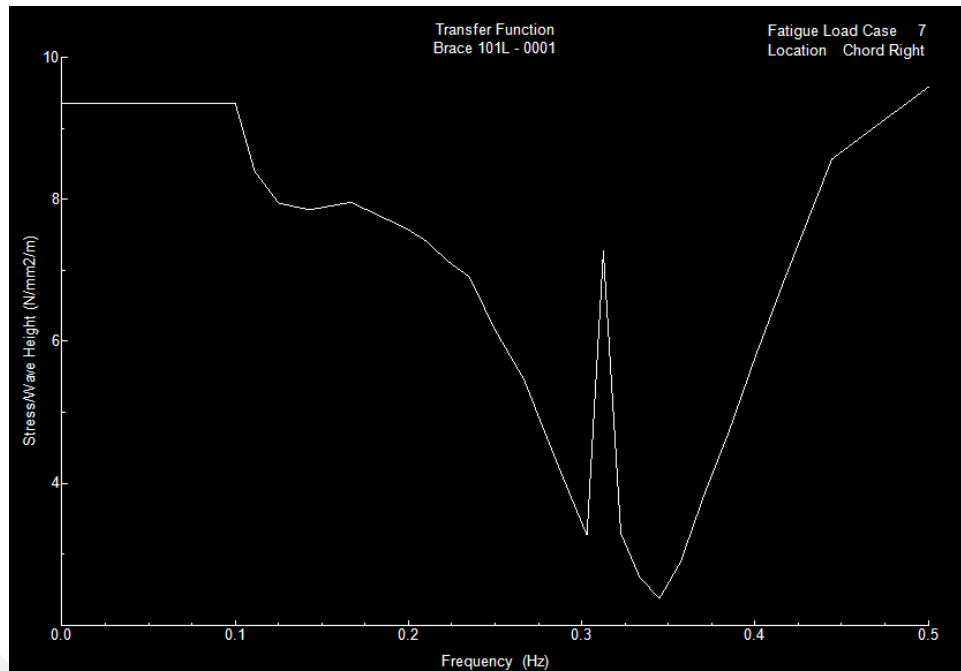


Figure 5.56 : Transfer function analysis for brace 101L – 0001.

The chord member between joints 201L and 101L has a diameter of 106.68 cm and a thickness of 3.81 cm. The stress concentration factors (SCFs) due to axial forces were determined to be 6.000 at the crown and 6.000 at the saddle. The SCFs corresponding to in-plane and out-of-plane bending were calculated as 2.933 and 6.000, respectively. The corresponding brace member, connecting joints 101L and 0002, is designed with a diameter of 55.88 cm and a wall thickness of 2.54 cm. The SCFs under axial loading were 5.109 (crown) and 6.000 (saddle), while the SCFs for in-plane and out-of-plane moments were 2.775 and 6.000, respectively.

Table 5.18 : Damage and service life of joint (101L).

Damage point	The CHORD element		The BRACE element	
	Damage value	Service life	Damage value	Service life
Top	3.605547	11.094	0.984046	40.649
Top-Left	4.096031	9.766	1.729273	23.131
Left	4.078102	9.808	2.458974	16.267
Bottom-Left	3.566819	11.214	1.501917	26.633
Bottom	2.946666	13.575	0.785930	50.895
Bottom-Right	2.572086	15.552	1.055385	37.901
Right	2.585112	15.473	1.557273	25.686
Top-Right	2.980715	13.420	1.227304	32.592
Max. value	4.096031	9.766	2.458974	16.267

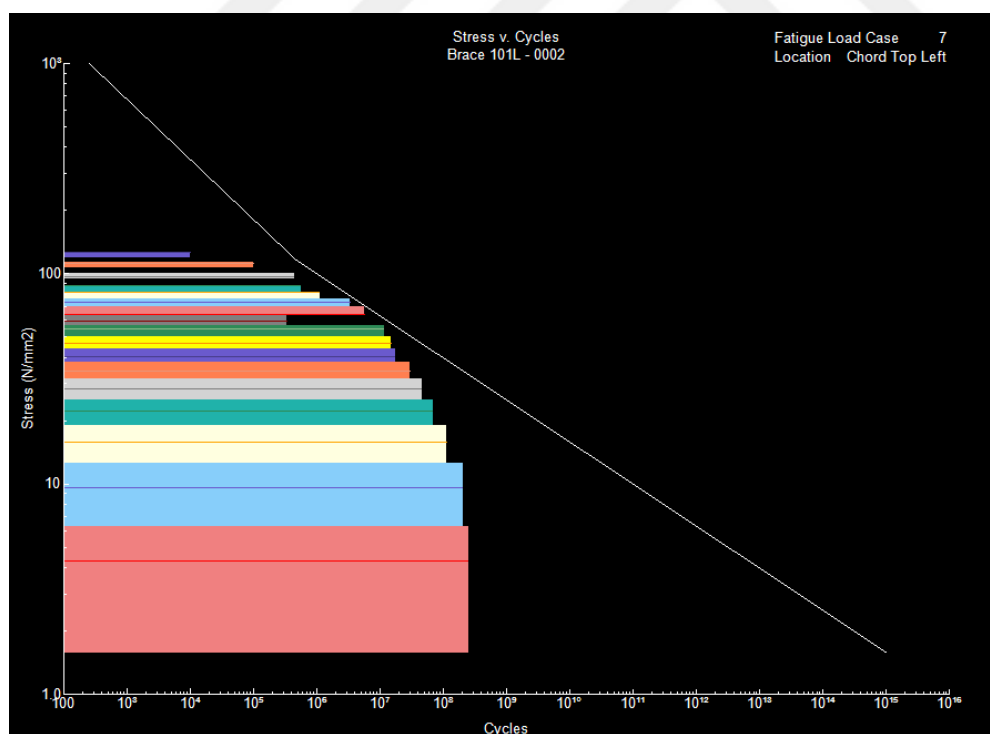


Figure 5.57 : Stress vs. cycles curve for brace 101L – 0002.

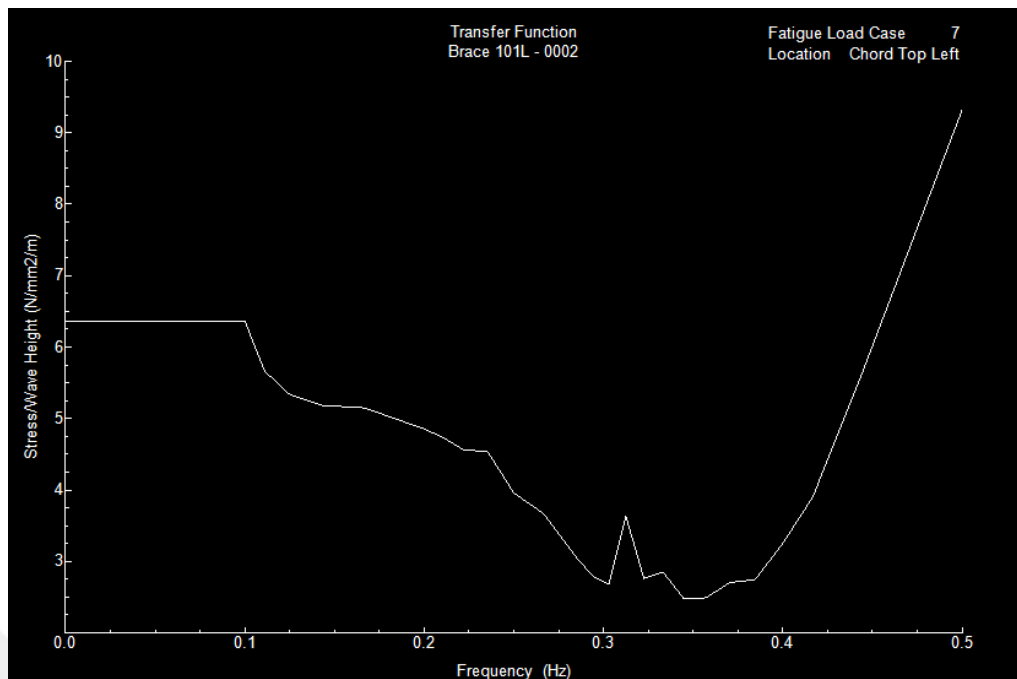


Figure 5.58 : Transfer function analysis for brace 101L – 0002.

Joint 103L is evaluated to determine its fatigue response and expected service life. Figure 5.59 presents the joint's layout, followed by the S–N curve for brace 103L–0000 in Figure 5.60. Table 5.19 summarizes the corresponding fatigue damage and life estimates. The transfer function analysis for the same brace is shown in Figure 5.61. Additional evaluation results for joint 103L are provided in Table 5.20, including further fatigue analysis for brace 0002–103L, depicted in Figures 5.62 and 5.63.



Figure 5.59 : Joint 103L.

The chord member between joints 103L and 203L has a diameter of 106.68 cm and a thickness of 3.81 cm. The stress concentration factors (SCFs) due to axial forces were determined to be 6.000 at the crown and 6.000 at the saddle. The SCFs corresponding

to in-plane and out-of-plane bending were calculated as 2.933 and 6.000, respectively. The corresponding brace member, connecting joints 103L and 0000, is designed with a diameter of 55.88 cm and a wall thickness of 2.54 cm. The SCFs under axial loading were 5.109 (crown) and 6.000 (saddle), while the SCFs for in-plane and out-of-plane moments were 2.775 and 6.000, respectively.

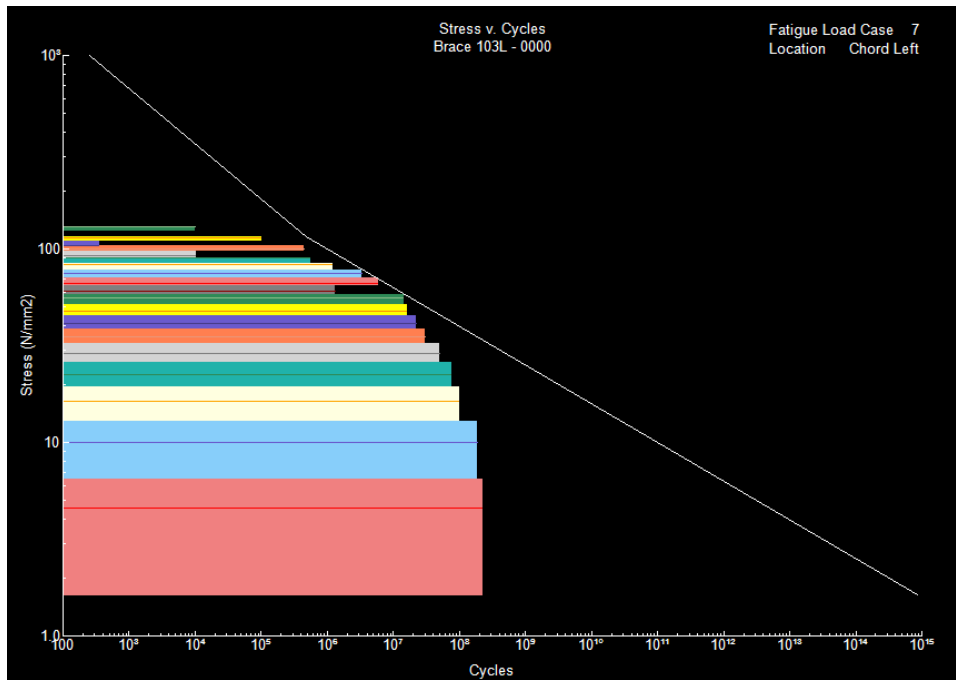
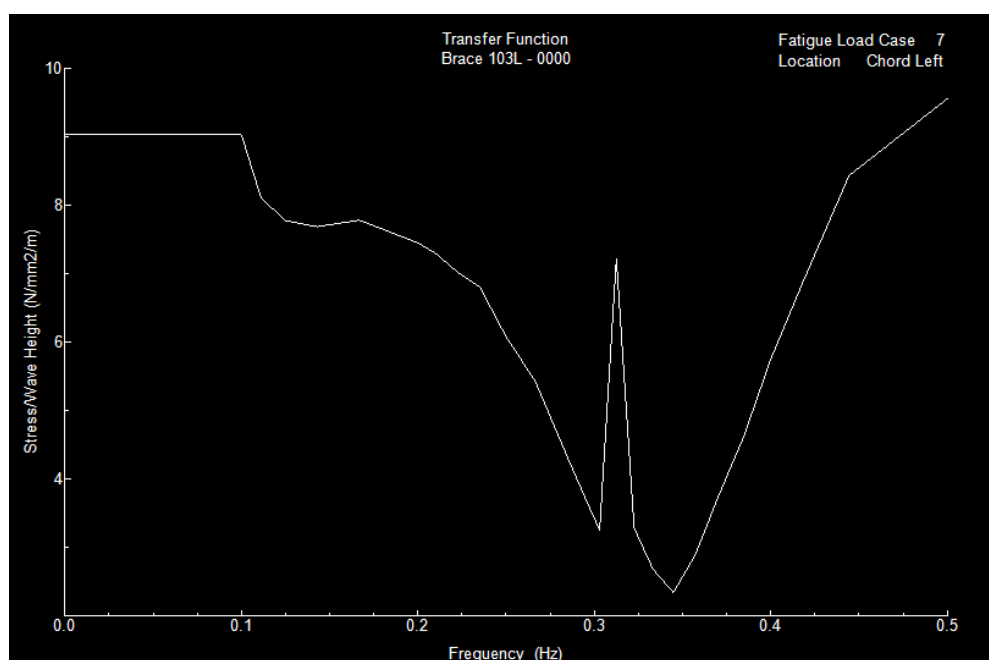


Figure 5.60 : Stress vs. cycles curve for brace 103L – 0000

Table 5.19 : Damage and service life of joint (103L).

Damage point	The CHORD element		The BRACE element	
	Damage value	Service life	Damage value	Service life
Top	3.488848	11.465	0.943719	42.386
Top-Left	4.510199	8.869	1.917587	20.860
Left	4.955405	8.072	2.988249	13.386
Bottom-Left	4.392552	9.106	1.866944	21.425
Bottom	3.346565	11.953	0.900956	44.397
Bottom-Right	2.563258	15.605	1.044296	38.303
Right	2.325732	17.199	1.401022	28.551
Top-Right	2.649042	15.100	1.080242	37.029
Max. value	4.955405	8.072	2.988249	13.386

**Figure 5.61** : Transfer function analysis for brace 103L – 0000.

The chord member between joints 103L and 203L has a diameter of 106.68 cm and a thickness of 3.81 cm. The stress concentration factors (SCFs) due to axial forces were

determined to be 6.000 at the crown and 6.000 at the saddle. The SCFs corresponding to in-plane and out-of-plane bending were calculated as 2.933 and 6.000, respectively. The corresponding brace member, connecting joints 0002 and 103L, is designed with a diameter of 55.88 cm and a wall thickness of 2.54 cm. The SCFs under axial loading were 5.109 (crown) and 6.000 (saddle), while the SCFs for in-plane and out-of-plane moments were 2.775 and 6.000, respectively.

Table 5.20 : Damage and service life of joint (103L).

Damage point	The CHORD element		The BRACE element	
	Damage value	Service life	Damage value	Service life
Top	3.638169	10.995	0.995103	40.197
Top-Left	2.985528	13.398	1.229445	32.535
Left	2.552781	15.669	1.537797	26.011
Bottom-Left	2.503288	15.979	1.026814	38.955
Bottom	2.857581	13.998	0.760365	52.606
Bottom-Right	3.489001	11.465	1.469362	27.223
Right	4.055042	9.864	2.445097	16.359
Top-Right	4.119987	9.709	1.740567	22.981
Max. value	4.119987	9.709	2.445097	16.359

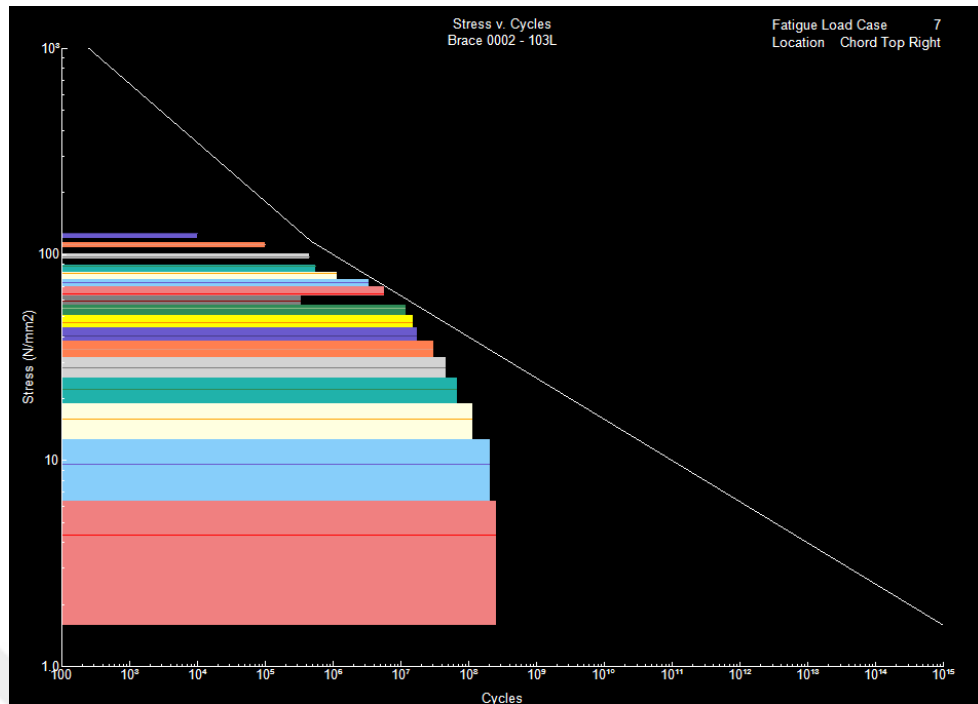


Figure 5.62 : Stress vs. cycles curve for brace 0002 - 103L.

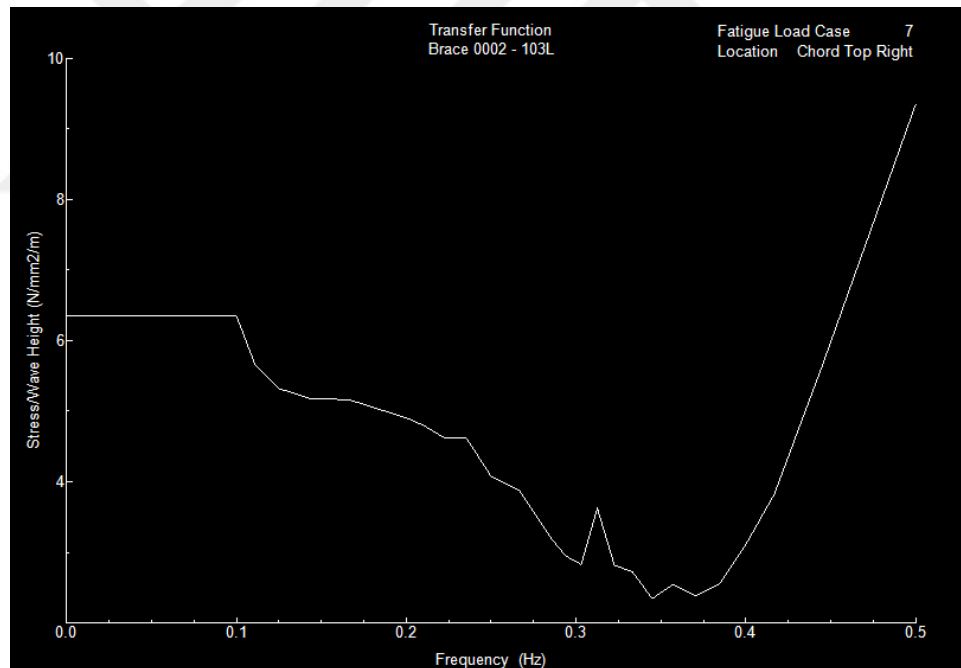


Figure 5.63 : Transfer function analysis for brace 0002 - 103L.

Joint 501L is investigated to assess its fatigue performance and structural integrity under cyclic loading. Figure 5.64 provides a representation of the joint configuration, while the fatigue behavior of brace 403L–501L is detailed through the S–N curve in Figure 5.65 and the corresponding transfer function in Figure 5.66. Table 5.21 summarizes the damage and estimated service life. Additional results are presented in

Table 5.22 for brace 0046–501L, whose fatigue characteristics are illustrated in Figures 5.67 and 5.68.

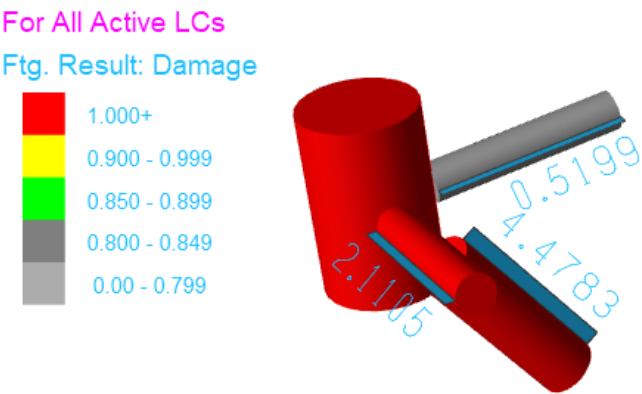


Figure 5.64 : Joint 501L.

The chord member between joints 401L and 501L has a diameter of 106.68 cm and a thickness of 3.81 cm. The stress concentration factors (SCFs) due to axial forces were determined to be 2.409 at the crown and 2.471 at the saddle. The SCFs corresponding to in-plane and out-of-plane bending were calculated as 1.500 and 1.887, respectively. The corresponding brace member, connecting joints 403L and 501L, is designed with a diameter of 55.90 cm and a wall thickness of 1.30 cm. The SCFs under axial loading were 2.631 (crown) and 2.501 (saddle), while the SCFs for in-plane and out-of-plane moments were 2.606 and 2.216, respectively.

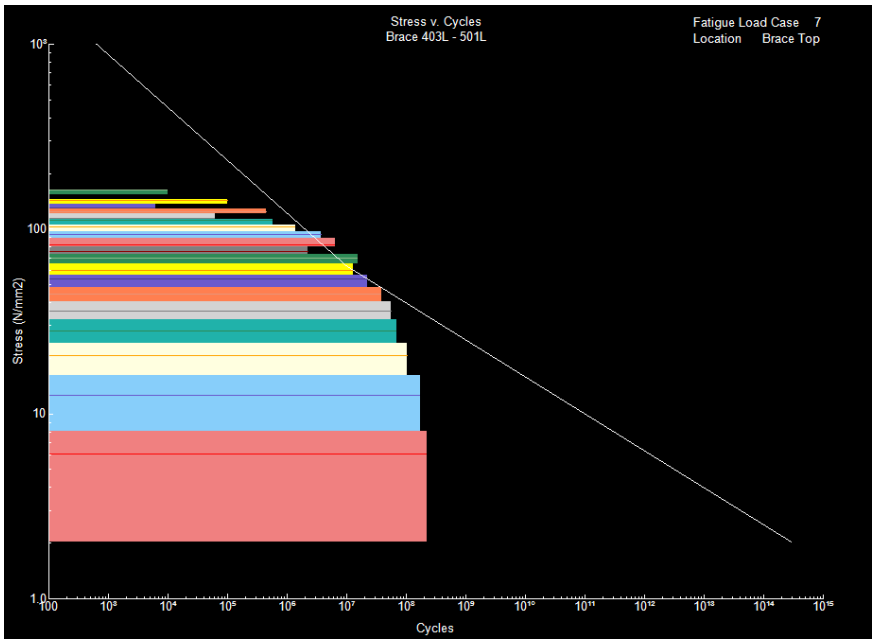
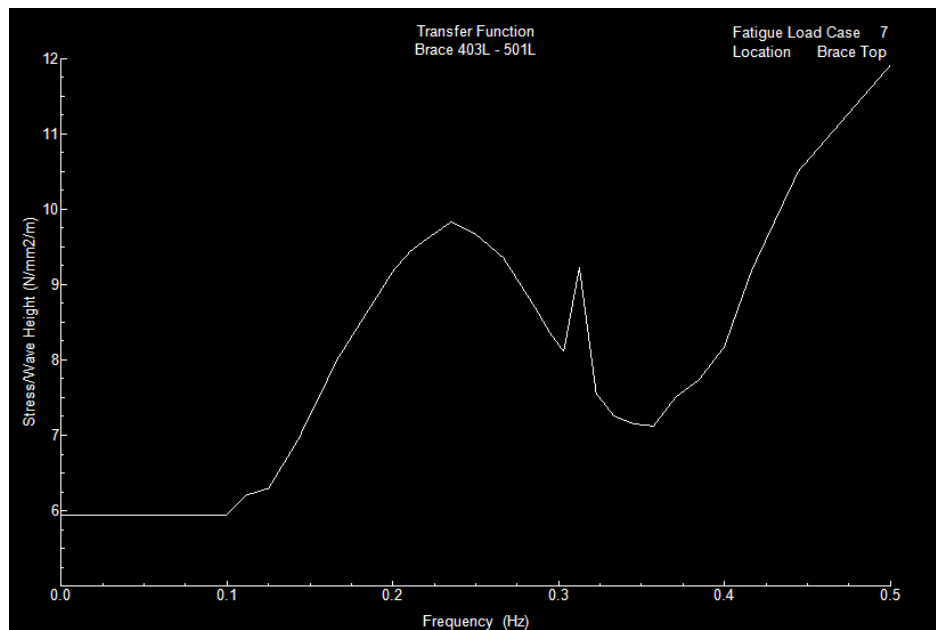


Figure 5.65 : Stress vs. cycles curve for brace 403L - 501L

Table 5.21 : Damage and service life of joint (501L).

Damage point	The CHORD element		The BRACE element	
	Damage value	Service life	Damage value	Service life
Top	2.309119	17.323	4.478338	8.932
Top-Left	1.507207	26.539	1.992115	20.079
Left	0.606855	65.914	0.311814	128.281
Bottom-Left	0.358698	111.514	0.462947	86.403
Bottom	0.299904	133.376	0.878616	45.526
Bottom-Right	0.289047	138.386	0.347436	115.129
Right	0.619438	64.575	0.312890	127.840
Top-Right	1.648172	24.269	2.242212	17.840
Max. value	2.309119	17.323	4.478338	8.932

**Figure 5.66** : Transfer function analysis for brace 403L - 501L

The chord member between joints 601L and 501L has a diameter of 106.68 cm and a thickness of 3.81 cm. The stress concentration factors (SCFs) due to axial forces were

determined to be 2.243 at the crown and 3.218 at the saddle. The SCFs corresponding to in-plane and out-of-plane bending were calculated as 1.500 and 2.021, respectively. The corresponding brace member, connecting joints 0046 and 501L, is designed with a diameter of 35.60 cm and a wall thickness of 1.00 cm. The SCFs under axial loading were 3.049 (crown) and 5.461 (saddle), while the SCFs for in-plane and out-of-plane moments were 2.146 and 3.042, respectively.

Table 5.22 : Damage and service life of joint (501L).

Damage point	The CHORD element		The BRACE element	
	Damage value	Service life	Damage value	Service life
Top	0.001391	28750	0.002297	17420
Top-Left	0.089129	448.789	0.256019	156.238
Left	0.721878	55.411	1.975529	20.248
Bottom-Left	0.277958	143.907	0.741022	53.979
Bottom	0.006298	6351.575	0.010767	3715.161
Bottom-Right	0.130734	305.964	0.372215	107.465
Right	0.787865	50.770	2.110481	18.953
Top-Right	0.248441	161.004	0.652618	61.292
Max. value	0.787865	50.770	2.110481	18.953

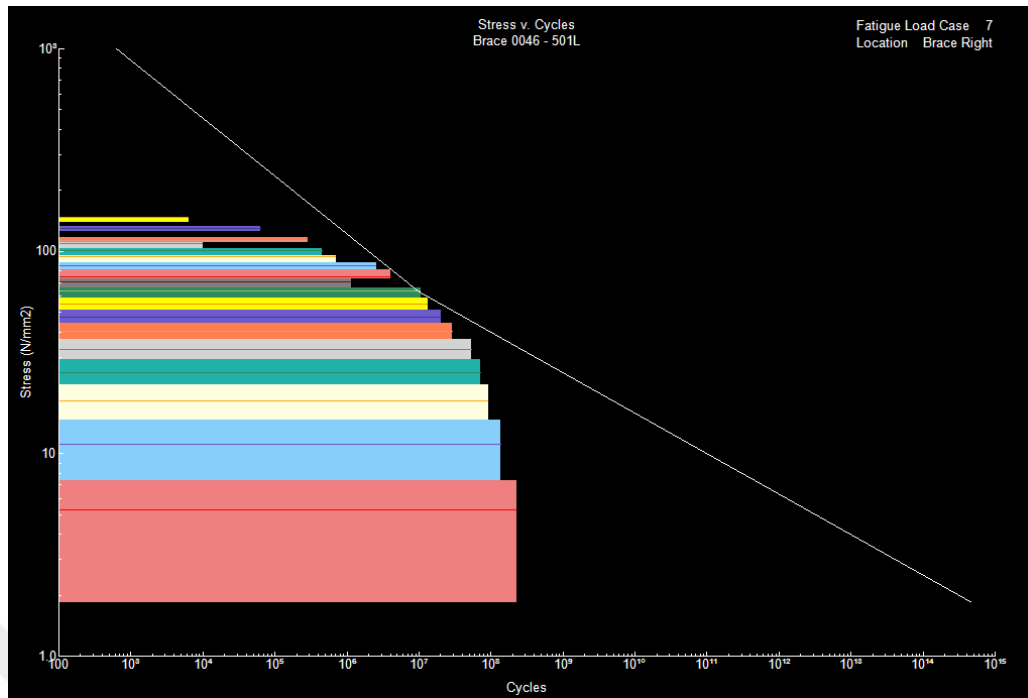


Figure 5.67 : Stress vs. cycles curve for brace 0046 - 501L.

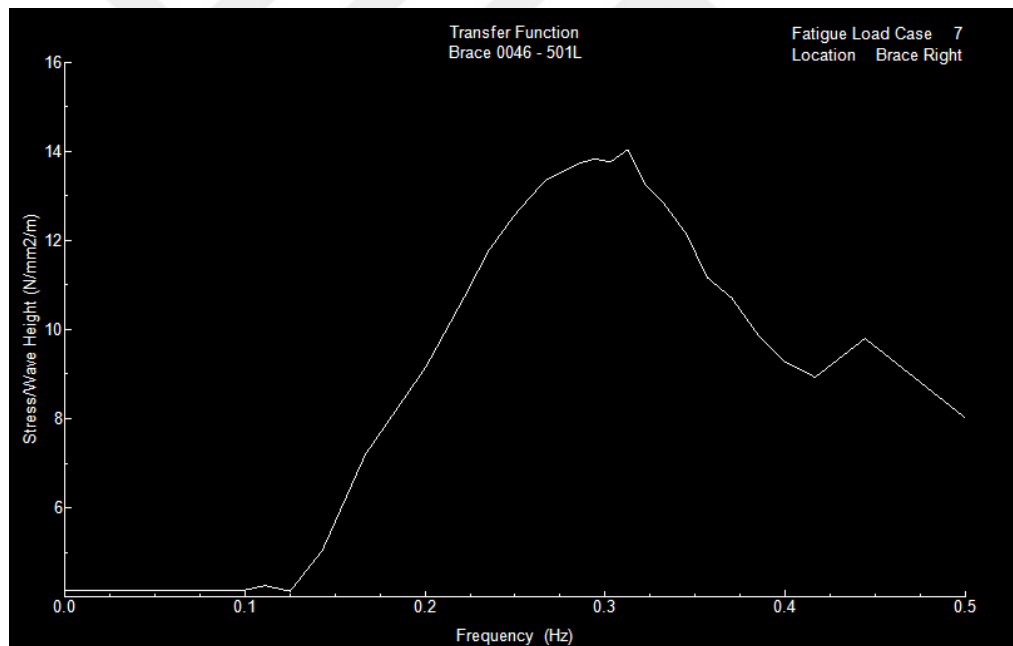


Figure 5.68 : Transfer function analysis for brace 0046 - 501L.

The structural performance of joint 0029 is examined through the following analyses. Figure 5.69 illustrates the joint layout, while the stress versus cycles (S–N) curve for brace 0043–0029 is shown in Figure 5.70. Table 5.23 presents the fatigue damage and service life estimation, and Figure 5.71 provides the transfer function analysis for the same brace, offering insights into its dynamic response under cyclic loading.

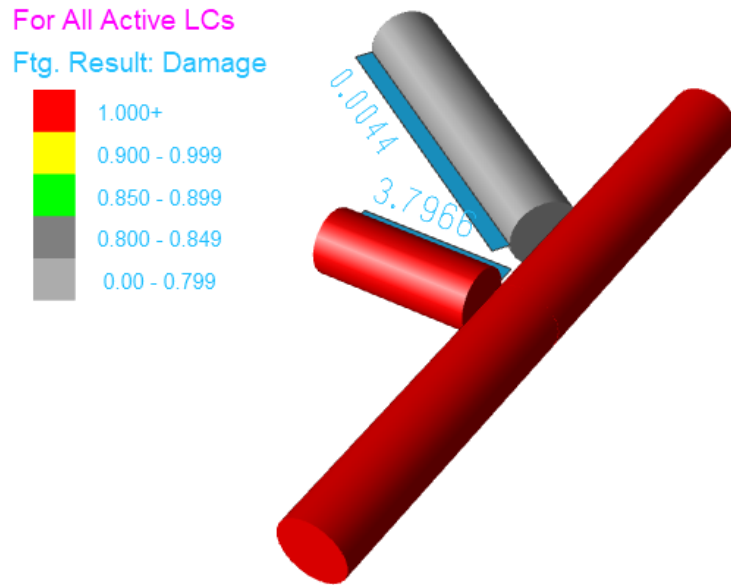


Figure 5.69 : Joint 0029.

The chord member between joints 0030 and 0029 has a diameter of 35.60 cm and a thickness of 1.00 cm. The stress concentration factors (SCFs) due to axial forces were determined to be 6.000 at the crown and 6.000 at the saddle. The SCFs corresponding to in-plane and out-of-plane bending were calculated as 3.954 and 6.000, respectively. The corresponding brace member, connecting joints 0043 and 0029, is designed with a diameter of 32.40 cm and a wall thickness of 1.00 cm. The SCFs under axial loading were 4.692 (crown) and 4.692 (saddle), while the SCFs for in-plane and out-of-plane moments were 2.814 and 6.000, respectively.

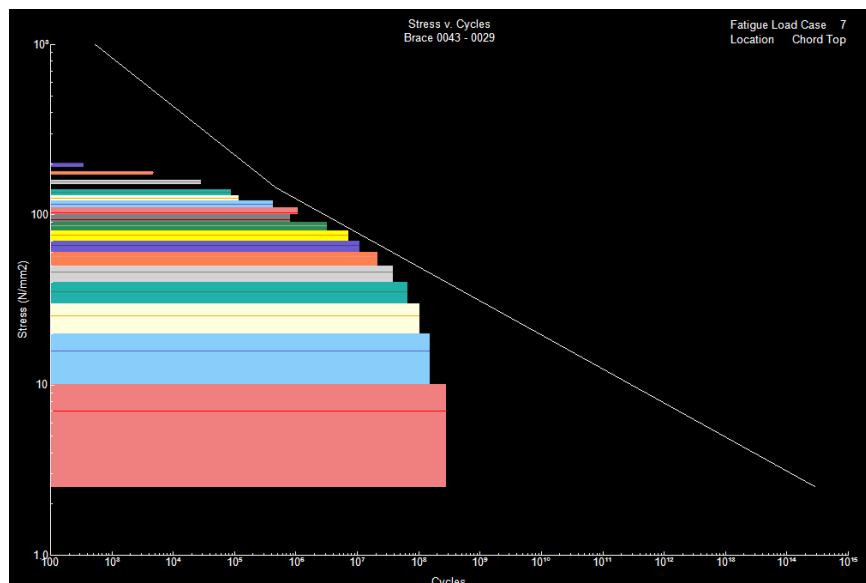


Figure 5.70 : Stress vs. cycles curve for brace 0043 – 0029.

Table 5.23 : Damage and service life of joint (0029).

Damage point	The CHORD element		The BRACE element	
	Damage value	Service life	Damage value	Service life
Top	3.796617	10.536	0.744403	53.734
Top-Left	1.229763	32.527	0.271820	147.156
Left	0.009122	4384.767	0.003735	10710
Bottom-Left	0.084361	474.150	0.015101	2648.870
Bottom	0.689363	58.025	0.120710	331.374
Bottom-Right	0.119212	335.536	0.023665	1690.270
Right	0.001172	34130	0.000369	108400
Top-Right	0.787450	50.797	0.146801	272.478
Max. value	3.796617	10.536	0.744403	53.734

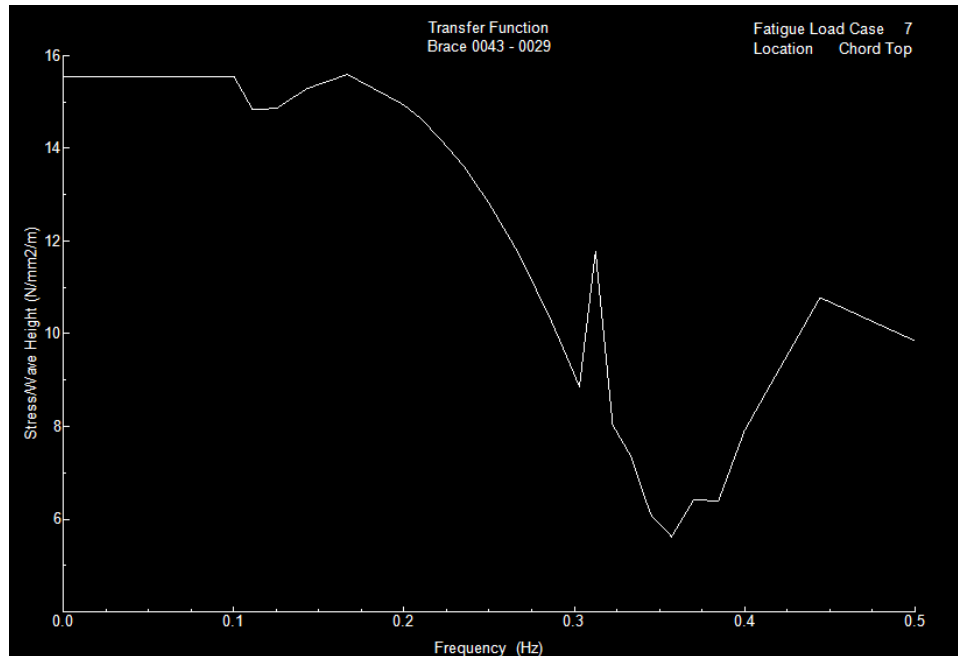


Figure 5.71 : Transfer function analysis for brace 0043 – 0029.

Joint 503L is assessed to determine its fatigue behavior and structural durability. Figure 5.72 presents the joint configuration, while Figure 5.73 shows the S–N curve

for brace 0045–503L. Table 5.24 summarizes the corresponding fatigue damage and estimated service life. The dynamic behavior of the same brace is analyzed through the transfer function in Figure 5.74. Additional fatigue data are provided in Table 5.25, related to brace 404L–503L, whose stress-cycle response and transfer function analysis are illustrated in Figures 5.75 and 5.76, respectively.



Figure 5.72 : Joint 503L.

The chord member between joints 603L and 503L has a diameter of 106.68 cm and a thickness of 3.81 cm. The stress concentration factors (SCFs) due to axial forces were determined to be 2.597 at the crown and 2.959 at the saddle. The SCFs corresponding to in-plane and out-of-plane bending were calculated as 1.500 and 2.176, respectively. The corresponding brace member, connecting joints 0045 and 503L, is designed with a diameter of 35.60 cm and a wall thickness of 1.00 cm. The SCFs under axial loading were 3.408 (crown) and 4.303 (saddle), while the SCFs for in-plane and out-of-plane moments were 2.146 and 3.275, respectively.

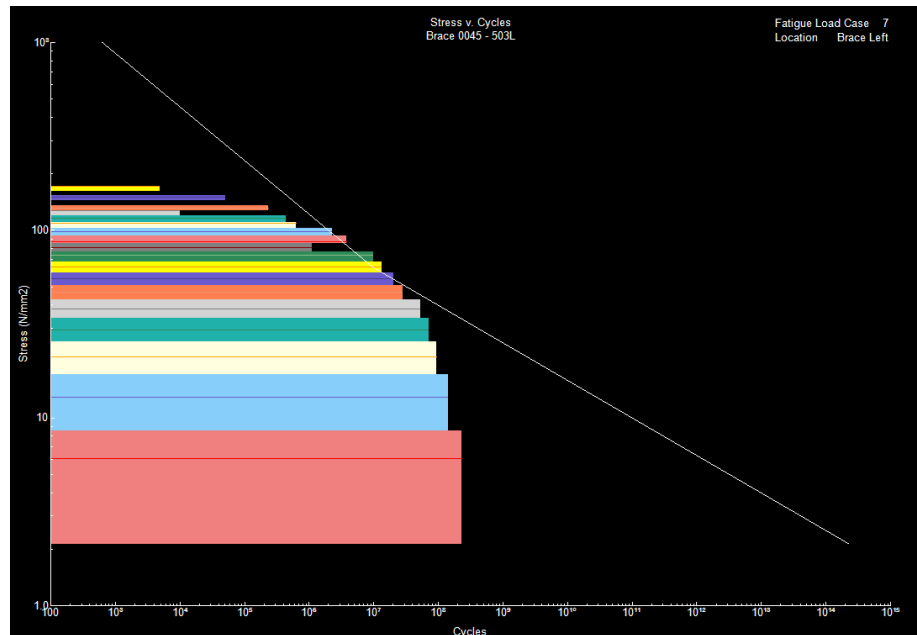


Figure 5.73 : Stress vs. cycles curve for brace 0045 - 503L.

Table 5.24 : Damage and service life of joint (503L).

Damage point	The CHORD element		The BRACE element	
	Damage value	Service life	Damage value	Service life
Top	0.043830	912.617	0.063119	633.721
Top-Left	0.731172	54.707	1.588598	25.179
Left	1.663734	24.042	3.761566	10.634
Bottom-Left	0.249604	160.254	0.598376	66.848
Bottom	0.002123	18850	0.002559	15630
Bottom-Right	0.331216	120.767	0.764604	52.315
Right	1.149015	34.812	2.597897	15.397
Top-Right	0.268506	148.973	0.585739	68.290
Max. value	1.663734	24.042	3.761566	10.634

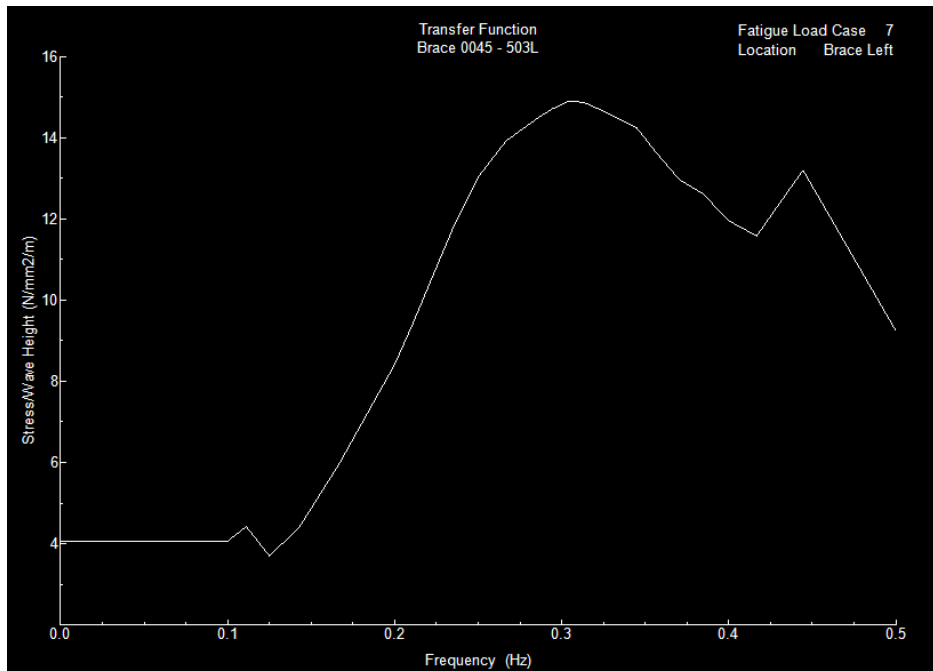
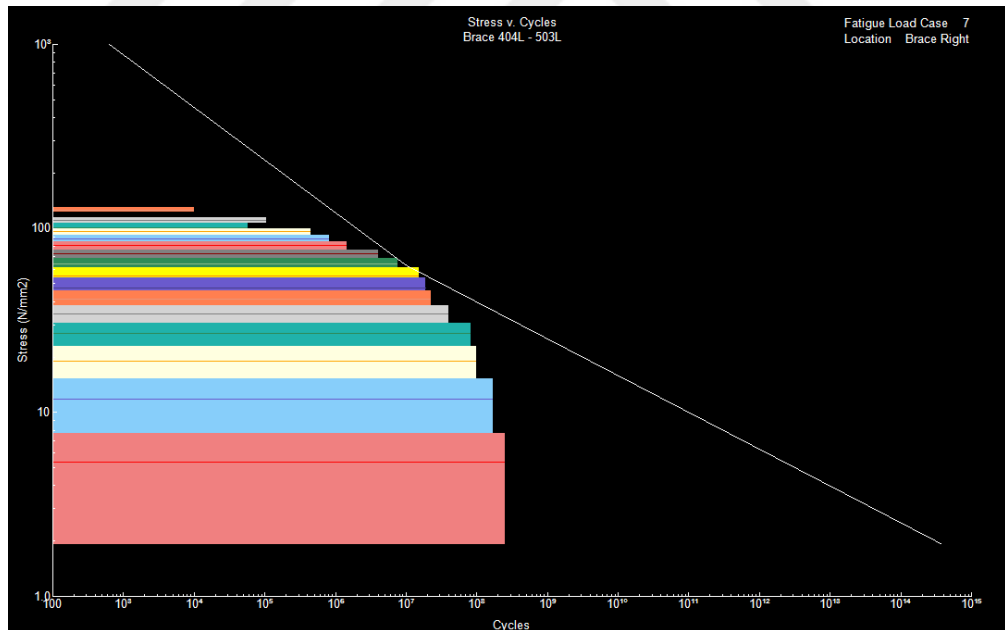


Figure 5.74 : Transfer function analysis for brace 0045 - 503L.

The chord member between joints 403L and 503L has a diameter of 106.68 cm and a thickness of 3.81 cm. The stress concentration factors (SCFs) due to axial forces were determined to be 2.354 at the crown and 2.401 at the saddle. The SCFs corresponding to in-plane and out-of-plane bending were calculated as 1.500 and 1.953, respectively. The corresponding brace member, connecting joints 404L and 503L, is designed with a diameter of 55.90 cm and a wall thickness of 1.30 cm. The SCFs under axial loading were 2.509 (crown) and 2.409 (saddle), while the SCFs for in-plane and out-of-plane moments were 2.606 and 2.293, respectively.

Table 5.25 : Damage and service life of joint (503L).

Damage point	The CHORD element		The BRACE element	
	Damage value	Service life	Damage value	Service life
Top	2.057674	19.439	3.465660	11.542
Top-Left	2.419096	16.535	2.567863	15.577
Left	2.157637	18.539	1.344259	29.756
Bottom-Left	0.414415	96.522	0.314472	127.197
Bottom	0.096781	413.304	0.204256	195.833
Bottom-Right	0.740235	54.037	0.599248	66.750
Right	2.595742	15.410	1.619946	24.692
Top-Right	2.485531	16.093	2.662921	15.021
Max. value	2.595742	15.410	3.465660	11.542

**Figure 5.75 : Stress vs. cycles curve for brace 404L - 503L.**

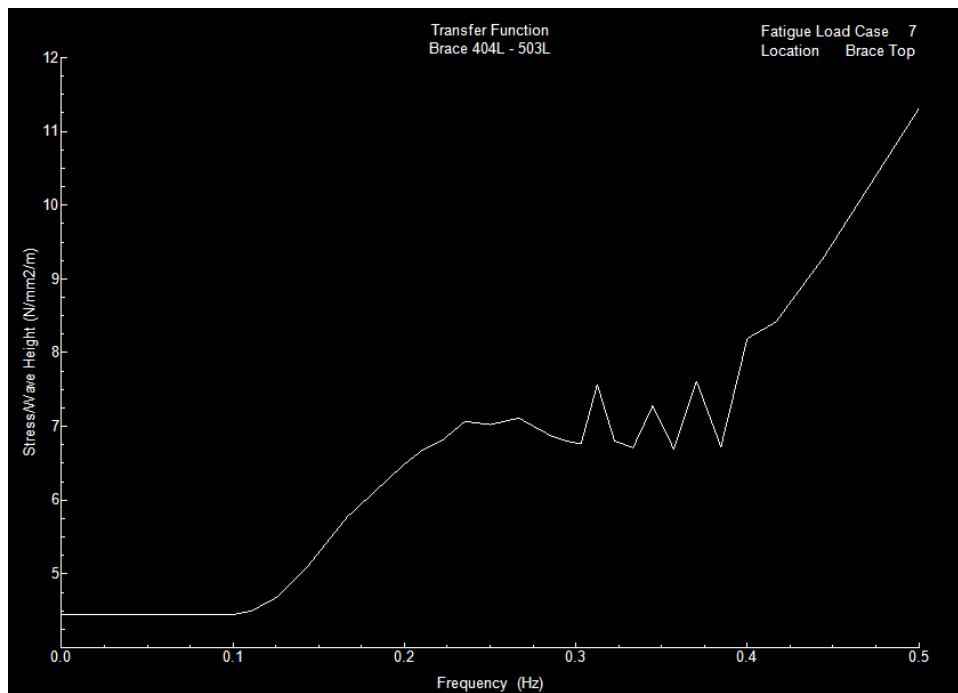


Figure 5.76 : Transfer function analysis for brace 404L - 503L.

The fatigue performance of joint 0006 is presented through a series of analyses. Figure 5.77 illustrates the joint configuration, while Figure 5.78 depicts the stress versus cycles (S–N) curve for brace 0006–202L. Table 5.26 summarizes the damage assessment and estimated service life. The transfer function analysis for this brace is shown in Figure 5.79, providing insight into its dynamic response under cyclic loading.

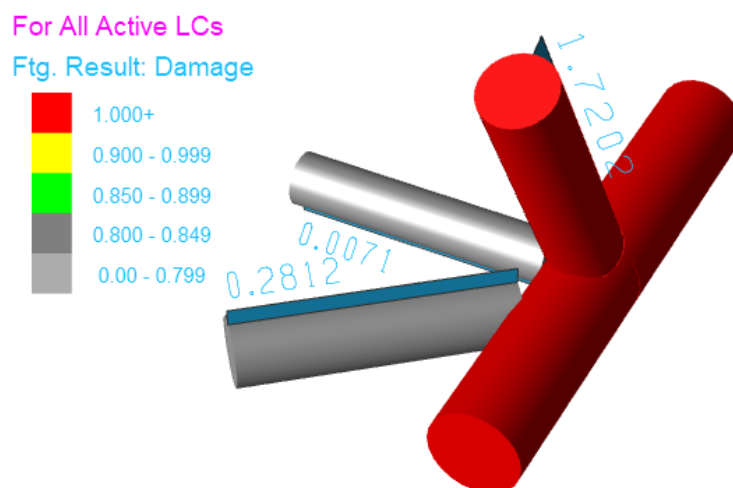


Figure 5.77 : Joint 0006.

The chord member between joints 0006 and 102L has a diameter of 55.88 cm and a thickness of 2.54 cm. The stress concentration factors (SCFs) due to axial forces were determined to be 6.000 at the crown and 6.000 at the saddle. The SCFs corresponding

to in-plane and out-of-plane bending were calculated as 2.682 and 6.000, respectively. The corresponding brace member, connecting joints 0006 and 202L, is designed with a diameter of 45.72 cm and a wall thickness of 2.03 cm. The SCFs under axial loading were 5.739 (crown) and 5.206 (saddle), while the SCFs for in-plane and out-of-plane moments were 2.528 and 4.546, respectively.

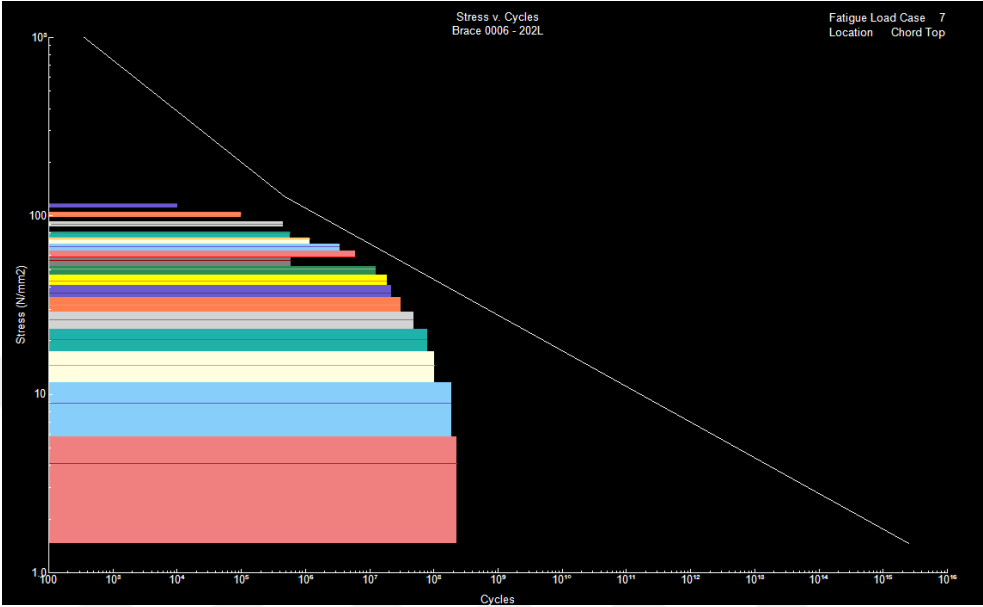
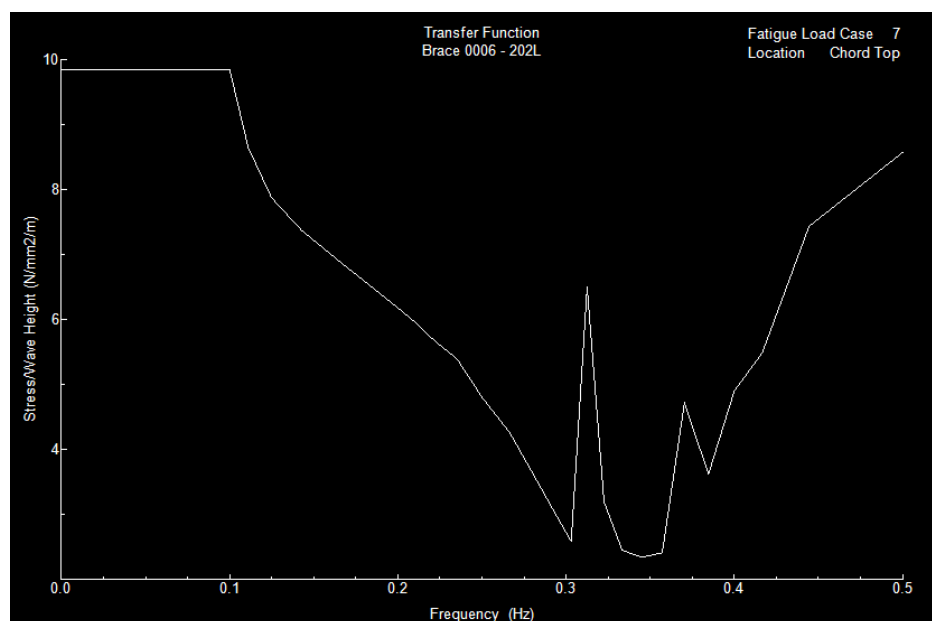


Figure 5.78 : Stress vs. cycles curve for brace 0006 - 202L.

Table 5.26 : Damage and service life of joint (0006).

Damage point	The CHORD element		The BRACE element	
	Damage value	Service life	Damage value	Service life
Top	1.720209	23.253	1.000703	39.972
Top Left	0.871514	45.897	0.434425	92.076
Left	0.048229	829,369	0.016717	2,392,828
Bottom Left	3.9023E-6	10,250,000	4.1270E-7	96,920,000
Bottom	6.4969E-5	615,700	2.8570E-5	1,400,000
Bottom Right	0.000000	Infinite	0.000000	Infinite
Right	0.013754	2,908,227	0.005597	7,146,810
Top Right	0.550241	72.695	0.298388	134.054
Maximum	1.720209	23.253	1.000703	39.972

**Figure 5.79 : Transfer function analysis for brace 0006 - 202L.**

Joint 0005 is evaluated to understand its fatigue characteristics and durability. Figure 5.80 depicts the joint layout, while Figure 5.81 shows the stress versus cycles (S–N)

curve for brace 0005–204L. Table 5.27 presents the calculated damage and estimated service life. The transfer function analysis for this brace is illustrated in Figure 5.82, highlighting its dynamic fatigue response.

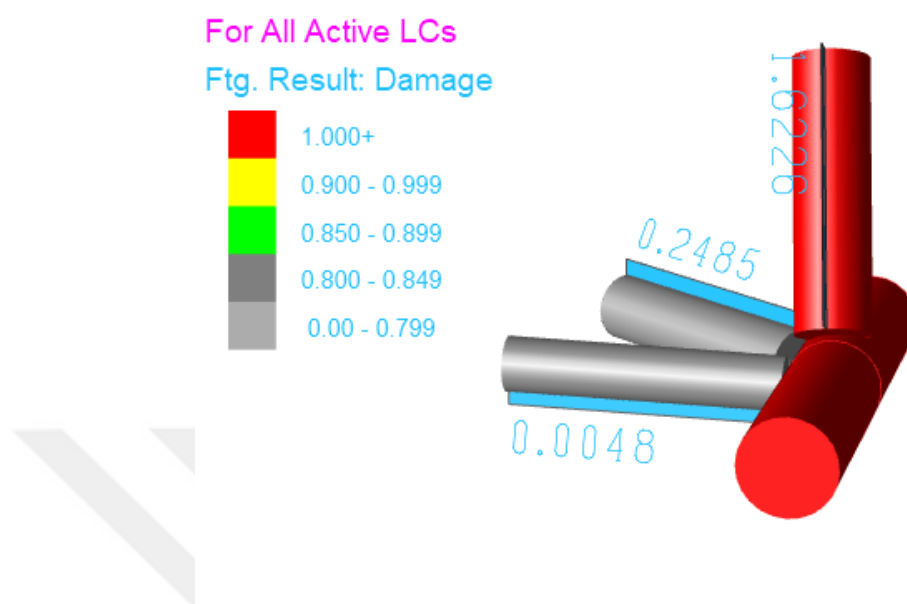


Figure 5.80 : Joint 0005.

The chord member between joints 104L and 0005 has a diameter of 55.88 cm and a thickness of 2.54 cm. The stress concentration factors (SCFs) due to axial forces were determined to be 6.000 at the crown and 6.000 at the saddle. The SCFs corresponding to in-plane and out-of-plane bending were calculated as 2.682 and 6.000, respectively. The corresponding brace member, connecting joints 0005 and 204L, is designed with a diameter of 45.72 cm and a wall thickness of 2.03 cm. The SCFs under axial loading were 5.739 (crown) and 5.206 (saddle), while the SCFs for in-plane and out-of-plane moments were 2.528 and 4.546, respectively.

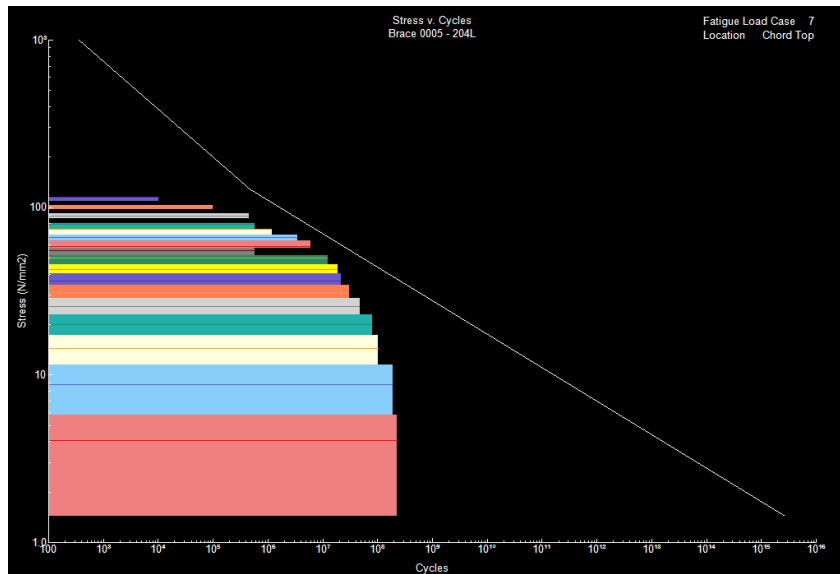


Figure 5.81 : Stress vs. cycles curve for brace 0005 - 204L.

Table 5.27 : Damage and service life of joint (0005).

Damage point	The CHORD element		The BRACE element	
	Damage value	Service life	Damage value	Service life
Top	1.622591	24.652	0.944156	42.366
Top Left	0.557407	71.761	0.298369	134.062
Left	0.016503	2,423.789	0.006527	6,128.401
Bottom Left	1.3690E-8	2,922,000,000	1.7251E-9	23,190,000,000
Bottom	4.6361E-5	862,800	2.0057E-5	1,994,000
Bottom Right	2.8605E-6	13,980,000	3.2279E-7	123,900,000
Right	0.040745	981.727	0.014368	2,783.991
Top Right	0.780198	51.269	0.392707	101.857
Maximum	1.622591	24.652	0.944156	42.366

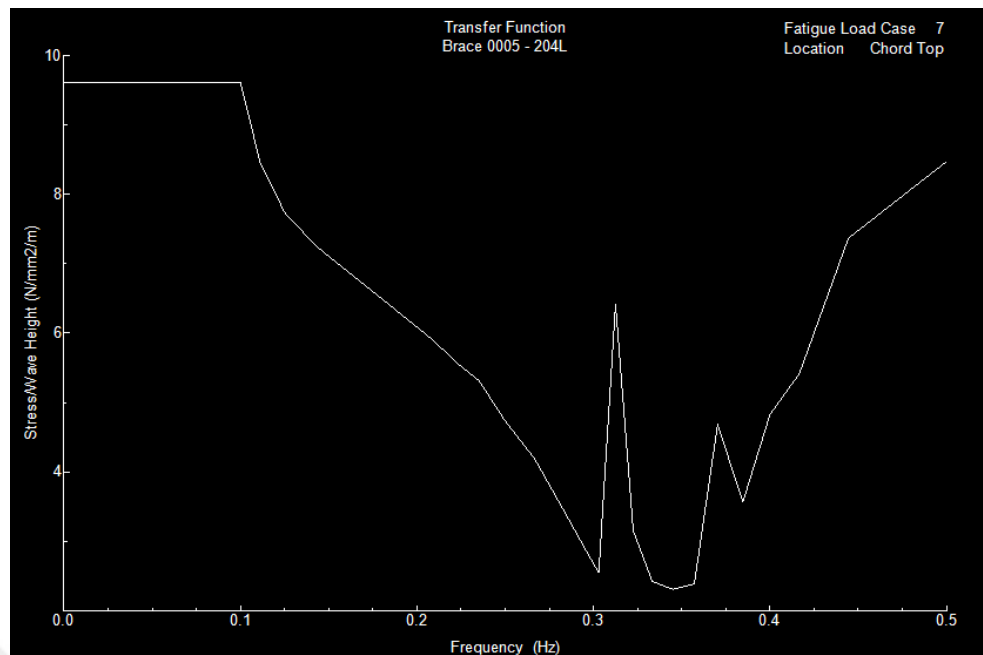


Figure 5.82 : Transfer function analysis for brace 0005 - 204L.

Joint 402L is analyzed to assess its fatigue behavior and expected service performance. Figure 5.83 shows the layout of the joint, while the stress versus cycles (S–N) relationship for brace 304L–402L is depicted in Figure 5.84. Table 5.28 summarizes the estimated fatigue damage and remaining service life. The transfer function analysis for the same brace is provided in Figure 5.85, offering a clearer understanding of its dynamic response characteristics.

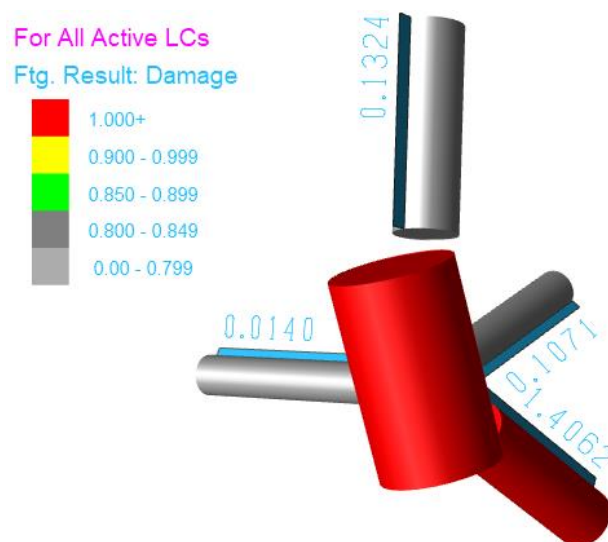


Figure 5.83 : Joint 402L.

The chord member between joints 302L and 402L has a diameter of 106.68 cm and a thickness of 3.81 cm. The stress concentration factors (SCFs) due to axial forces were

determined to be 4.292 at the crown and 4.292 at the saddle. The SCFs corresponding to in-plane and out-of-plane bending were calculated as 1.885 and 3.487, respectively. The corresponding brace member, connecting joints 304L and 402L, is designed with a diameter of 55.90 cm and a wall thickness of 2.03 cm. The SCFs under axial loading were 3.403 (crown) and 3.413 (saddle), while the SCFs for in-plane and out-of-plane moments were 2.823 and 3.217, respectively.

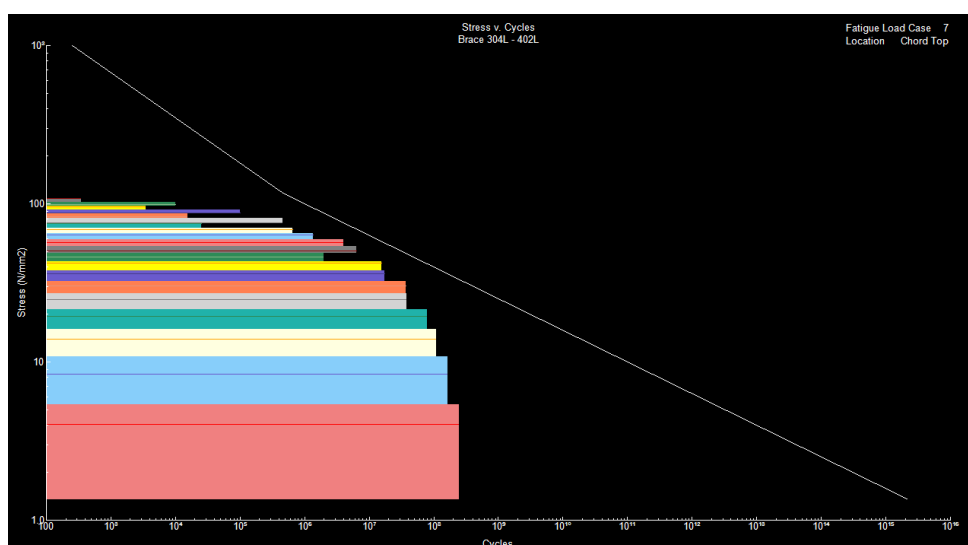


Figure 5.84 : Stress vs. cycles curve for brace 304L - 402L.

Table 5.28 : Damage and service life of joint (402L).

Damage point	The CHORD element		The BRACE element	
	Damage value	Service life	Damage value	Service life
Top	1.406221	28.445	0.334660	119.524
Top Left	1.186616	33.709	0.249939	160.039
Left	0.792959	50.444	0.120830	331.044
Bottom Left	0.511376	78.220	0.052249	765.561
Bottom	0.419323	95.392	0.034832	1148.359
Bottom Right	0.515030	77.665	0.051706	773.606
Right	0.815173	49.069	0.124730	320.692
Top Right	1.219908	32.789	0.259332	154.242
Maximum	1.406221	28.445	0.334660	119.524

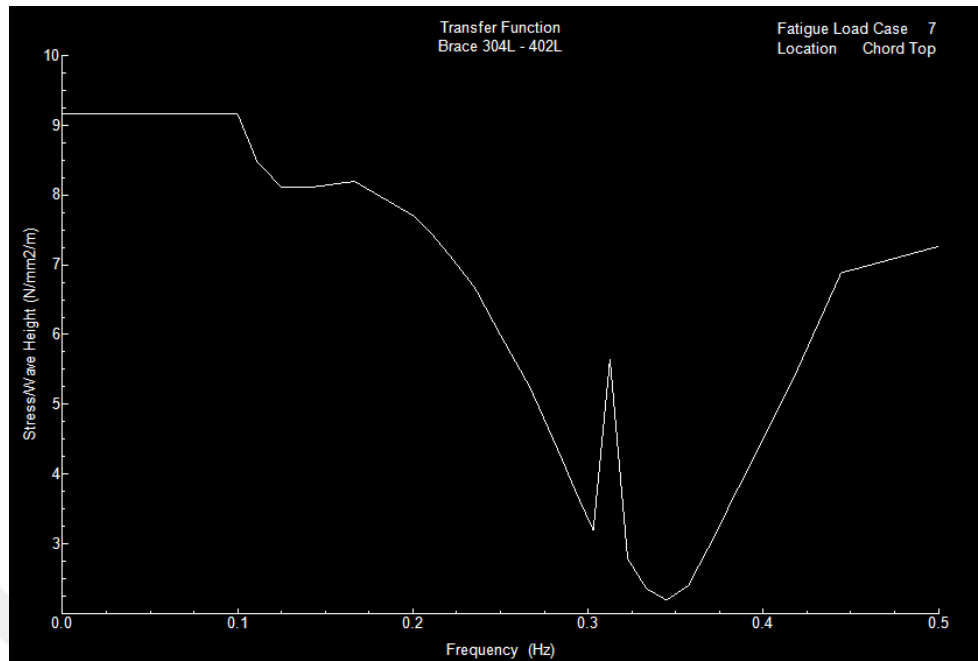


Figure 5.85 : Transfer function analysis for brace 304L - 402L.

5.4 Results Discussion

The structural response under seismic loading is calculated as one of the most critical outcomes of the dynamic seismic analysis conducted using the SACS software. A key result is the base shear, which represents the total horizontal force transmitted from the entire structure—subjected to seismic excitation—to the foundation level at the seabed. This value serves as a significant indicator of the earthquake’s intensity and its impact on the structural system.

Among the different seismic records analyzed, the El Centro earthquake produced the highest base shear value, reaching 5066.53 kN, indicating its relatively higher intensity and impact on the structure. In contrast, the Chi-Chi earthquake resulted in a significantly lower base shear value of 4.188 kN, while the Kocaeli earthquake yielded a base shear of 10.73 kN. These variations highlight the differences in seismic demands and their influence on offshore structural response.

The results of the fatigue analysis reveal both the magnitude and the location of the maximum fatigue damage, as well as the estimated Service life of the joints. A design Service life of 40 years was assumed. The analysis identified 24 CHORD–BRACE intersection points that exceeded the assumed design life. The locations and corresponding stress values of the fatigue damage were determined in order to enable

redesign of the affected joints to meet the intended Service life requirements. The S–N curves adopted in this study are based on the provisions of ISO 19902, which provides fatigue resistance data for different welded joint types used in offshore structures. These curves relate the stress range to the number of cycles to failure, allowing for the estimation of fatigue life under cyclic loading. For the calculation of Stress Concentration Factors (SCFs), the Efthymiou method was employed. This method offers a semi-empirical approach to estimate SCFs in tubular joints, taking into account the geometric parameters of the connection. The results of the fatigue analysis reveal both the magnitude and the location of the maximum fatigue damage, as well as the estimated Service life of the joints. A design Service life of 40 years was assumed. The analysis identified 24 CHORD–BRACE intersection points that exceeded the assumed design life. The locations and corresponding stress values of the fatigue damage were determined in order to enable redesign of the affected joints to meet the intended Service life requirements. The S–N curves adopted in this study are based on the provisions of ISO 19902, which provides fatigue resistance data for different welded joint types used in offshore structures. These curves relate the stress range to the number of cycles to failure, allowing for the estimation of fatigue life under cyclic loading. For the calculation of Stress Concentration Factors (SCFs), the Efthymiou method was employed. This method offers a semi-empirical approach to estimate SCFs in tubular joints, taking into account the geometric parameters of the connection.

6. CONCLUSION

This thesis investigates the behaviour of a fixed offshore structure subjected to diverse environmental and seismic forces, with an emphasis on evaluating fatigue damage.

The primary goal is to enhance understanding of the structural response of fixed offshore platforms under such conditions and to assess the impact of fatigue damage. The study focuses on a platform situated in the Mediterranean Sea near Turkey's coastline, a region recently noted for oil field discoveries.

To achieve these objectives, the SACS V16 software was utilized to model and analyse the fixed offshore structure under seismic, operational, and environmental loading scenarios. The seismic response of the platform was assessed using dynamic nonlinear time history analysis, modal response spectrum analysis, adhering to API/ISO-19901 standards.

Dynamic nonlinear time history analysis indicated that the platform could withstand ground motion from the 1940 El Centro earthquake without collapsing.

Base shear calculations demonstrated that the seismic loads on the platform's foundation (specifically the piles) were within safe capacity limits, with safety factors remaining below 1.

It is evident that the El Centro ground motion resulted in the highest displacement values of the structure when compared to the other two earthquakes, Kocaeli and Chi-Chi. This can be attributed to several factors, primarily the occurrence of resonance or amplification of the seismic response, indicating that the structure responded more significantly to this particular ground motion.

In other words, the frequency content of the El Centro earthquake closely aligns with the natural frequency of the structure, leading to a resonant effect that amplified the structural response.

Moreover, the El Centro earthquake is characterized by a relatively longer duration of seismic waves, which facilitated the accumulation of seismic energy within the structure over the course of the ground shaking.

In contrast, although the Kocaeli and Chi-Chi earthquakes possess high energy levels, their shorter durations and the mismatch between their dominant frequencies and the structure's natural frequency resulted in a comparatively lower seismic response.

A spectral wave fatigue analysis was performed on the platform's finite element model to determine the stress transfer function. This analysis involved calculating wave loads under specific loading and environmental conditions, including external and internal wave pressures as well as inertial forces.

The Rayleigh distribution model was applied to describe the short-term stress range, while the S-N curve represented the fatigue capacity. Fatigue damage was quantified as the ratio of stress range distribution to fatigue capacity. Results revealed that many joints exhibited a fatigue lifespan of less than 40 years, offering valuable insights into the fatigue integrity of existing offshore platforms.

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