

Behaviour of Cable-Stayed Bridges with High Modulus CFRP Cables

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Author's Declaration

I hereby declare that I am the sole author of this thesis. This is a true copy of the thesis, including any required final revisions, as accepted by my examiners. I understand that my thesis may be made electronically available to the public.

Abstract

A number of cable-stayed bridges have undergone costly cable replacements as a result of cable corrosion. Additionally, steel cables are prone to fatigue due to stress variations under traffic loads. In light of these issues, an interest in alternative cable materials has developed, with carbon fiber-reinforced polymers (CFRP) being the top candidate due to its high strength, corrosion resistance, relatively high stiffness, fatigue resistance, and creep rupture resistance.

Aside from cost, the two main differences between steel and CFRP are that steel has a higher elastic modulus and a much higher coefficient of thermal expansion than CFRP. Over the last 3 decades, many papers have been published on the use of CFRP cables for cable-stayed bridges. The literature review revealed a general consensus that the lower elastic modulus of CFRP is somewhat disadvantageous as it results in larger vertical deck displacements under live loads and thus larger deck bending moments. Different studies have used different methods of sizing CFRP cables: some used the equivalent strength principle, while others used the equivalent stiffness principle. The literature review also revealed that there is little research on the behaviour of cable-stayed bridges with CFRP cables subjected to temperature loads. A parametric study is conducted and presented in this thesis to address these findings from the literature review.

The parametric study is carried out in two Phases. **Phase I** involves varying the elastic modulus of the CFRP material. Current literature cites the elastic modulus of CFRP as being lower than that of steel, but high modulus (HM) CFRP materials do exist. The purpose of **Phase I** is to study the effect of using HM CFRP for the cables of a cable-stayed bridge. **Phase II** involves varying the cable cross section area while keeping the elastic modulus constant. The purpose of **Phase II** is to study the effect of the cable sizing method on the behaviour of a cable-stayed bridge. To conduct the parametric study, a series of finite element models were created for a theoretical cable-stayed bridge which has two H-pylons, a semi-fan cable arrangement, a 250 m long main span, and 100 m long side spans.

The following results were obtained from the finite element models and discussed: cable forces; vertical deck displacements; longitudinal pylon tip displacements; axial force, shear, and bending moment in the deck and pylons; and the ratio of the 1st torsional frequency (T) to the 1st bending frequency (B). Only dead, live, and temperature loads were applied to the bridge, and only SLS-

1, ULS-1, and ULS-2 load combinations from CAN/CSA S6-19 were considered. The parametric study yielded the following findings:

- Decreasing the cable area in **Phase II** of the parametric study has the same effect as decreasing the elastic modulus of the CFRP in **Phase I**.
- Increasing the elastic modulus attracts slightly more live load to the cables, increasing the axial forces and decreasing the shear and bending moment in the deck and pylons.
- Temperature loads have a monumental effect on the performance of cable-stayed bridges with CFRP cables due to CFRP having a much lower coefficient of thermal expansion than steel and concrete.
- When considering the combined effects of all applied loads, it appears that the use of HM CFRP cables has benefits.
- Increasing the elastic modulus or the area of CFRP cables increases T and B but decreases the T:B ratio. Consequently, increasing the elastic modulus of the cables decreases the critical wind speed required to cause flutter.

For future work, the parametric study should be extended to include the effects of various bridge geometries, various boundary conditions, wind loads, seismic loads, and ice accretion. A comprehensive analysis of the bridge's dynamic behaviour should be conducted.

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List of Abbreviations

Acronym	Name
AADT	Average annual daily traffic
BFRP	Basalt fiber-reinforced polymer
CFRP	Carbon fiber-reinforced polymer
CHBDC	Canadian Highway Bridge Design Code
EMPA	Swiss Federal Laboratories for Materials Testing and Research
FLS	Fatigue Limit State
FRP	Fiber-reinforced polymer
GFRP	Glass fiber-reinforced polymer
HM	High modulus
LTM	Load transfer medium
MTO	Ontario Ministry of Transportation
MUP	Multi use pathway
SLS	Serviceability Limit State
TAC	Transportation Association of Canada
ULS	Ultimate Limit State

Chapter 1. Introduction

1.1. Background

Cable-stayed bridges are a popular solution for spanning long distances. Cable-stayed bridges typically consist of a deck which carries traffic, one or two pylons which transfer the loads into the supporting soil, and steel cables which transfer the loads from the deck to the pylons. Cable-stayed bridges represent a relatively efficient structural system as the deck, pylons and cables are subjected primarily to axial forces. The deck and pylons are typically constructed using either concrete or steel, with concrete being popular for shorter spans due to its lower cost and corrosion resistance and steel being used for longer bridges due to its higher strength-to-weight ratio.

With a few exceptions, steel is currently the only material which is being used for cables. The corrosion of steel cables in suspension and cable-stayed bridges is a significant problem. Several cable-stayed bridges have required costly cable replacements as a result of corrosion. Two examples of this are the replacement of the cables of the Köhlbrand Bridge in Germany in 1979 which cost \$6 million USD, and the replacement of the cables of the Maracaibo Bridge in Venezuela between 1979 and 1981, which cost \$50 million USD [1]. Additionally, steel cables are prone to fatigue as they are subjected to stress variations under traffic loads.

An interest in alternative cable materials has developed in light of these issues. Several fiber-reinforced polymers (FRPs) have been considered as an alternative to steel. Glass, aramid, basalt, and carbon FRPs have been studied, but the consensus is that carbon fiber-reinforced polymers (CFRP) is the best material to be used for cables due to its high strength, relatively high stiffness, good fatigue resistance, and high resistance to creep rupture. Research on the use of CFRP cables for cable-stayed bridges surfaced in the 1980s and gained traction in the 1990s. In the last 3 decades, several cable-stayed bridges were constructed with CFRP cables, including the Storchen Bridge which was constructed in 1996 and the Jiangsu University CFRP Footbridge which was constructed in 2005.

1.2. Problem Statement

The use of CFRP cables in cable-stayed bridges has been studied for several decades, and many published papers discuss the static and dynamic behaviour of such bridges. Aside from cost, the

two main differences between steel cables and CFRP cables are their elastic modulus and coefficient of thermal expansion. Steel cables have an elastic modulus of 190 to 200 GPa, whereas CFRP cables generally have an elastic modulus between 150 and 160 GPa. Steel has a coefficient of thermal expansion of 12×10^{-6} , while CFRP has a coefficient of thermal expansion of 0.6×10^{-6} , 20 times less than that of steel.

Many studies cite the lower elastic modulus of CFRP cables as being disadvantageous. The lower stiffness of CFRP cables results in higher vertical deck displacements under live loads and, subsequently, higher bending moments in the deck. To address this concern, some studies have used the equivalent axial stiffness principle to size CFRP cables, but other studies have sized CFRP cables based on the equivalent strength principle. A study has not been conducted on the performance of cable-stayed bridges with CFRP cables as a function of the cable sizing methodology. Another solution for this concern would be to use higher-modulus CFRP cables. Higher modulus CFRP materials do exist, but their application in cable-stayed bridges with CFRP cables has not been studied.

Additionally, there is very little research on the performance of cable-stayed bridges with CFRP cables under thermal loads. Since the coefficient of thermal expansion of CFRP is 20 times lower than that of steel, it is likely that bridges with CFRP cables perform very differently from bridges with steel cables when subjected to thermal expansion and contraction.

1.3. Objective and Scope

This thesis aims to study the static and dynamic behaviour of cable-stayed bridges with cables which are entirely made of CFRP. The objectives of this research are as follows:

- 1) To review the existing literature on the use of CFRP cables in cable-stayed bridges and address the research and practice gaps.
- 2) To explore the effects of using high modulus (HM) CFRP cables on the static and dynamic performance of cable-stayed bridges.
- 3) To explore the effects of the cable sizing methodology on the static and dynamic performance of cable-stayed bridges by varying the cable area using the equivalent stiffness and equivalent strength principles.

- 4) To comprehensively analyze the performance of cable-stayed bridges with CFRP cables under thermal loads.

The scope of this thesis is limited to studying the static behaviour of cable-stayed bridges under dead, live, and temperature loads. This thesis does not consider wind, seismic, ice, vessel collision, and other exceptional loads as they require much more complex analyses. As such, the Canadian Highway Bridge Design Code (CHBDC) load combinations are limited to SLS-1, ULS-1, and ULS-2. The study of the dynamic behaviour of cable-stayed bridges is limited to the comparison of the ratios of the 1st torsional frequency to the 1st bending frequency, which directly affects the critical wind speed and the resistance against flutter.

Only the global behaviour of cable-stayed bridges is considered in this thesis. Local behaviours, such as transverse bending in the deck, are not considered. The anchorage of CFRP cables is not studied, and it is assumed that the anchorages perform sufficiently to resist the cable loads. The bridges being studied in this thesis have a constant geometry. The span lengths, boundary conditions, cable arrangements, member sizes, pylon/deck types, and pylon/deck materials are not varied in the parametric study.

Finally, the properties of HM CFRP materials are not scrutinized in this thesis. It is assumed that HM CFRP materials possess the same mechanical properties as regular CFRP materials (those properties being the coefficient of thermal expansion, fatigue resistance, creep properties, and other such properties) with the exception of the elastic modulus.

1.4. Structure of Thesis

This thesis is organized in five chapters. Chapter 1 is the introduction. Chapter 2 presents the findings of the literature review. Chapter 3 outlines the methodology and assumptions behind the parametric study. Chapter 4 provides a detailed discussion of the results from the parametric study. Chapter 5 is the conclusion.

Chapter 1 introduces the research. The background explains the issues with the steel cables used for cable-stayed bridges and provides the highlights of the research performed to date on the use of CFRP cables. The problem statement describes the gaps in the current research which will be addressed in this thesis. The objective and scope define the objectives and limitations of the parametric study.

Chapter 2 presents the literature review. The literature review summarizes the available literature on the static performance, long-term properties, anchorage performance, dynamic performance, aerodynamic performance, and seismic performance of cable-stayed bridges with CFRP cables. It also briefly summarizes the available information on HM CFRP materials. The gaps in the existing research and useful assumptions which can be implemented in the parametric study are identified.

Chapter 3 describes the methodology and assumptions behind the parametric study. First, a description of the bridge being used for the parametric study is provided. Next, the finite element model and the applied loads are discussed. Finally, the methodology of the parametric study is presented.

Chapter 4 presents the results, discusses the likely reasons for these results being observed, and compares the results to the literature review findings. The parametric study is divided into two phases, **Phase I** and **Phase II**. **Phase I** involves varying the modulus of the CFRP material, while **Phase II** involves varying the cable area. As applicable, the displacements, axial loads, shear, and bending moments are presented for the cables, deck, and pylons under dead loads, live loads, thermal loads, and load combinations. The figures, which serve as the focal point of the discussion in this chapter, are located in Appendix A and Appendix B.

Chapter 5 summarizes the conclusions. The conclusions are presented in accordance with the research objectives identified in Section 1.3. Recommendations for future work are presented based on the scope and limitations of this thesis identified in Section 1.3.

Chapter 2. Literature Review

2.1. Overview

The purpose of this literature review is to (1) summarize the findings of a conglomerate of studies related to various aspects of the static and dynamic performance of CFRP cable-stayed bridges, (2) identify areas of agreement and disagreement between various studies for each aspect of static and dynamic performance, (3) identify gaps in the research that has been conducted so far which can be addressed in this thesis, and (4) identify useful assumptions that can be used to narrow down the scope of this thesis and simplify the parametric study. The findings of the literature review are organized by the following topics, which serve as the remaining sections of this chapter:

- Static Performance;
- Long Term Properties;
- Anchorage Performance;
- Dynamic Performance;
- Aerodynamic Stability;
- Seismic Performance; and
- Structural Applications of High Modulus CFRP.

2.2. Background

Interest in using CFRP as a material for the cables of cable-stayed bridges surfaced in the 1980s and gained traction in the 1990s. The earliest research on this topic originated from Europe, and several papers published in the 1980s and 1990s were authored by Urs Meier, a prominent researcher at the Swiss Federal Laboratories for Materials Testing and Research (EMPA) since 1969 [2]. In 1987 he proposed a CFRP cable-stayed bridge with a main span of 8400 m over the Strait of Gibraltar at its narrowest point [3]. His contributions in the anchorage of CFRP cables [4] and monitoring of CFRP cables led to the implementation of two CFRP cables in the Storchen bridge in Winterthur, Switzerland in 1996, as shown in Figure 1 [5].



Figure 1 – Storchen Bridge in Winterthur, Switzerland [6]

In the 2000s and the 2010s this field of study was dominated by China. Multiple Chinese papers were published on cable-stayed bridges' static and dynamic properties with CFRP cables. Many of these studies are theoretical and implemented finite element methods to compare the performance of bridges with CFRP cables to identical bridges with steel cables. The focus in the majority of these studies is overwhelmingly on long-span bridges (main span > 600 m); in several papers an existing long-span cable-stayed bridge such as the Sutong Bridge in China with a main span of 1088 m was redesigned with CFRP cables to compare the performance of steel and CFRP bridges; in others a theoretical bridge was designed to perform the investigation, with main spans ranging from 600 m to upwards of 2000 m and the most popular choice of main span being 1400 m. A notable accomplishment in China was the construction of the Jiangsu University CFRP Footbridge, a pedestrian cable-stayed bridge with fully composite cables with a main span of 30 m on the Jiangsu University campus. This bridge allowed researchers to study the behaviour of CFRP cables in real life, particularly the performance of the anchorages [7].

2.3. Static Performance

The consensus between all studies which discuss the static performance of cable-stayed bridges with CFRP cables is that CFRP cables offer an advantage over steel cables in that they are more efficient than steel cables as their high strength-to-weight ratio in comparison to steel greatly

reduces the sag effect [8, 9, 10, 11, 12, 13, 14, 15]. CFRP's density is only about 1/7 of that of steel [8], which results in a lower reduction in effective stiffness of the cable. While the type of CFRP which is commonly used in structural applications has a lower elastic modulus than steel (approximately 160 GPa for CFRP compared to 200 GPa for steel), once the cable's horizontal projection reaches a certain threshold, CFRP cables become stiffer than steel cables due to the decreased sag when using the Ernst equation to calculate effective elastic modulus [10]. Most of the studies, many of which have been published within the last 10 years, rely on the assumption that CFRP cables have a lower elastic modulus than steel cables, generally about 80% of that of steel or 160 GPa [10, 12, 16, 13, 17, 14, 18, etc.]. Hua et al. (2007) commented that some types of CFRP whose elastic modulus is equal to or greater than that of steel have been manufactured abroad [13]. The availability and current structural applications of HM CFRP are discussed in Section 2.8 of this literature review.

CFRP cables have greater tensile strength than steel, ranging from 2000 to 3000 MPa compared to 1860 MPa for high-strength steel [13]. An example of this is the CFCC tendons manufactured by a Japanese company called Tokyo Rope which have a guaranteed breaking strength between 2200 and 2500 MPa [19]. CFRP cables, while stronger in tension than steel cables, have a lower elastic modulus. Different studies have addressed this issue in different ways when redesigning a bridge with steel cables to a bridge with CFRP cables to compare the performance of the two. Most studies adopted what is called the principle of equivalent stiffness, which means that they sized the CFRP cables such that their axial stiffness EA (where E is the elastic modulus and A is the cross-sectional area of the cable) is equal to the axial stiffness of their steel counterparts [17, 18, 20, 21, 22, 23]. This approach ensures that the benefits of the steel cables (their stiffness) are not lost, but does it not make very efficient use of the CFRP material given its superior strength and fatigue performance as well as its significant cost per unit weight, which was reported to be approximately 10 times greater than that of steel in a 2022 paper [1]. Ren et al. (2019) and Zhang et. al (2008) sized CFRP cables based on the equivalent strength principle, meaning that they equated the tensile strength of the CFRP cables to that of the steel cables [9, 24]. This approach makes more efficient use of the expensive CFRP material but loses out in stiffness given the smaller elastic modulus of CFRP and the smaller area required to achieve the same tensile strength. Kuihua et. al (2017) used an approach which is a compromise between the two: the authors utilized the same cable area for the CFRP cables as for the steel cables, which results in a slightly lower

axial stiffness but a higher axial strength [12]. In another study, Xiong et al. (2011) determined the CFRP cable area based on allowable stress and a factor of safety [16]. It would be interesting to examine the effect of the method chosen to size the CFRP cables on the behaviour of a cable-stayed bridge, whether it is the equivalent strength, equivalent stiffness, or equivalent area principle.

Several researchers have entertained the possibility of using a hybrid arrangement of steel and CFRP cables or hybrid cables to obtain the combined benefits of both materials. Several papers examined the performance of bridges with hybrid steel/CFRP cables and varying ratios of steel to CFRP within the cables [25, 26, 27, 28]. An example of a hybrid steel/CFRP cable proposed by Cai and Aref (2015) [28] is shown in Figure 2. Wang and Wu (2010) proposed the use of hybrid B/CFRP cables made with basalt and carbon fibers and compared the performance of a bridge with pure BFRP cables, pure CFRP cables, and varying ratios of basalt and carbon fibers within a hybrid cable, concluding that using hybrid B/CFRP cables results in a lower cost than using pure CFRP, higher stiffness than using pure BFRP, and better aerodynamic stability than pure CFRP or BFRP due to designable internal damping [11]. Yang et al. (2020) and Wang and Wu (2010) proposed the use of a hybrid cable arrangement consisting of hybrid cables with varying proportions of high-strength steel, BFRP and CFRP within the cable cross-section for various cable length ranges [29, 30]. Sun et al. (2020) and Xiong et al. (2012) proposed hybrid stay cable systems which utilize pure steel for the shorter cables and pure CFRP for the longer cables to increase the efficiency of the longer cables while avoiding the premium cost of using CFRP for the shorter cables [31, 32]. This thesis focuses on the performance of cable-stayed bridges with pure CFRP cables; thus, hybrid cables and hybrid cable arrangements are outside the scope of this thesis.

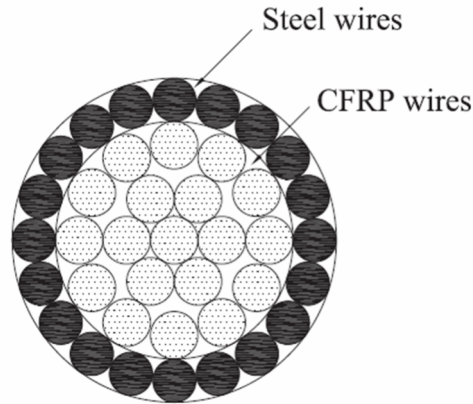


Figure 2 – Example of Hybrid Steel/CFRP Cable [28]

A notable property of CFRP is that it has a very low coefficient of thermal expansion compared to steel, approximately $1/20^{\text{th}}$ of that of steel or $0.6 \times 10^{-6}/^{\circ}\text{C}$ [8], and it can be expected that this property of CFRP will affect the performance of a cable-stayed bridge. Most literature acknowledged the difference in thermal expansion coefficients of steel and CFRP but did not comment much on the repercussions of this difference. Kuihua et al. (2017) studied the relationship between temperature and member forces for steel and CFRP cable-stayed bridges and found that the bending moment in the deck at the pylon and the vertical midspan displacement due to temperature is smaller for CFRP bridges than steel bridges, which is beneficial for the service performance of the whole bridge [12]. Ren et al. (2019) noted that for a cable-stayed bridge with CFRP cables and an ultra-high-performance concrete (UHPC) deck, displacements due to temperature changes in both the cables and the deck were found to be significantly lower than for a bridge with steel cables and a steel deck, particularly when cables were heated [9]. However, this study varies both the cable and deck material, making it difficult to isolate the effect of changing just the cable material on the thermal response of a cable-stayed bridge. The thermal response of a cable-stayed bridge with CFRP cables is not well discussed in the available literature, and this gap is addressed in this thesis.

Finally, it is important to discuss the available literature on the structural response of the other components of a cable-stayed bridge as a function of the cable material. The consensus is that the deck will experience more significant bending moments and vertical displacements in a bridge with CFRP cables than in a bridge with steel cables due to the lower stiffness of CFRP cables. In one study done by X. Xie et al. (2014), the maximum deck displacement under live load was found

to be 22% higher for CFRP than steel cables [8]. In another study by G. Xie et. al (2015), it was found that when considering the non-linear effects of cable sag, large deformations, and beam-column interaction, the vertical displacement of the main span is 10% higher, the vertical displacement of the side spans is 60% lower, and the longitudinal displacement of the pylon top is virtually the same for a bridge with CFRP cables when compared to a bridge with steel cables [10]. A third study by Kuihua et al. (2017) concluded that when considering all non-linear effects, most structural effects, including the deck bending moments and vertical displacements, are greater for the bridge with CFRP cables than the bridge with steel cables, which was attributed to the elastic modulus of CFRP being lower than that of steel, even when accounting for cable sag [12]. As for the compressive stress in the deck at the pylon, [8] and [12] agreed that it is lower in the bridge with CFRP cables; [8] reported this stress to be 9% lower for a CFRP bridge compared to a steel bridge. In terms of additional observations, Xiong et al. (2011) noted that CFRP cables slightly reduce the global stiffness of a bridge when compared to steel cables due to the CFRP material's lower elastic modulus [16], while X. Xie (2007) noted that CFRP cables can offer more lateral stiffness to the deck due to the lower sag effect [33]. It appears that most of these observations can be attributed to the lower elastic modulus of CFRP, regardless of whether or not linear or non-linear analysis is used (i.e., whether or not cable sag is accounted for). Using a high-modulus CFRP material for the cables of the bridge could potentially lower the bending moments and vertical displacements in the deck.

2.4. Long Term Properties

Creep is a concern for fiber-reinforced polymers as polymer resins and some types of fibers are prone to creep under sustained loads. Glass fiber-reinforced polymers (GFRP) have especially poor creep performance because glass fibers are susceptible to creep rupture and lose their strength over time [34]. Thus GFRP is not recommended as a substitute for steel cables. CFRP has been cited as having good creep performance compared with other FRPs, but few studies discuss the creep performance of CFRP in the context of cable-stayed bridges. The creep performance of the first cable-stayed bridge with CFRP cables in China (the Jiangsu University CFRP Footbridge) was discussed in a paper by Mei et al. (2015) which stated that the stresses in the cables did not decline significantly over time which indicates that creep deformation of the resin under service loads is not obvious [7]. However, results were only presented for the first 3 hours and thus did

not provide a full picture of the creep performance of the cables of this pedestrian bridge. Another paper by Hua et al. (2007) reviewed the research and application of CFRP cables for cable-stayed bridges described two tests that had been performed on CFRP tendons. The results of one test showed that the creep coefficient of CFRP is practically zero when loaded for 1000 hours at a stress of $0.60f_u$, while the relaxation was less than 1%. Another test found a creep coefficient of 0.01% after 3000 hours and a relaxation is equal to 0.8% after 1000 hours. The volumetric ratio of the fibers in the tested cables was not stated. [13]

There is also some literature that describes the creep performance of CFRP outside of the context of cable-stayed bridges. One paper written by Gajdosova et al. (2020) summarized the research and experimental studies on the long-term properties of CFRP to date and reported several estimations of the creep rupture stress for CFRP: Yamaguchi et al. (1997) extrapolated the creep rupture stress to be $0.93f_u$ for CFRP, while Ando et al. (1997) estimated the creep rupture stress to be $0.79f_u$ for CFRP [35]. Another study by Yang et al. (2018) examined the creep behaviour of CFRP tendons under high stress levels [36]. Test specimens were gradually loaded to stress levels of $0.69f_u$, $0.76f_u$, and $0.85f_u$, results were recorded over 1000 hours (h) and then extrapolated to obtain creep coefficients for 50 years and 1 million hours. The creep coefficient was found to be higher for higher stresses and varied from 1.08% to 1.16% for sustained applied stresses of $0.69f_u$ to $0.85f_u$ over 1000 h. The residual strength after creep testing was 4.54% lower, while the elastic modulus increased by 6.99%. Through extrapolation, it was determined that for 1 million hours, the creep coefficients under stress levels of $0.69f_u$ - $0.85f_u$ are 1.83-1.91%. It was noted that other studies reported that CFRP tendons subjected to stresses less than $0.60f_u$ did not exhibit any creep behaviour.

The studies available at this time present results for only 1000 h of load testing and use those results to extrapolate creep coefficients and creep rupture stresses. It would be useful to obtain long term results by monitoring CFRP tendons and cables which have been installed for real life applications, as this would provide more accurate creep data for practical applications. Based on these findings, limiting the stress in CFRP cables to $0.60f_u$ will dismiss creep rupture concerns, limit the creep coefficient to 0.01%, and limit the relaxation to 1%. To avoid the uncertainties associated with long-term effects, the cable stresses for the parametric study in this thesis will be limited to $0.60f_u$.

Fatigue is often cited as an issue for steel cables, leading to their premature deterioration and high maintenance costs for cable-stayed bridges with steel cables. CFRP cables are widely considered to have superior corrosion and fatigue performance compared to that of the steel cables, yielding lower maintenance costs over the lifetime of the bridge [8, 9, 7, 10, 11, 12, 13, 17, 14, 18, 37, etc.]. Several studies comment on the fatigue performance of CFRP cables in the context of cable-stayed bridges. The study by Mei et al. (2015) describing the Jiangsu University CFRP Footbridge suggests that bond-type anchorages prevent damage to the tendons and thus possess better fatigue performance [7]. This study raises an important concern regarding the low shear strength of CFRP cables and the importance of implementing a suitable anchorage system that does not damage the cables, reducing their fatigue capacity. Hua et al. (2007) stated that based on test results, the fatigue strength of CFRP is about three times greater than steel and that the tensile capacity of CFRP did not decrease after being subjected to 2 million load cycles [13]. Another paper by Feng et al. (2019) suggested that in order to meet a 100-year fatigue life, safety factors of 2.55 and 3.64 are recommended for CFRP and steel, respectively, confirming the superior fatigue performance of CFRP [14].

Song et al. (2019) extensively studied the fatigue life of CFRP tendons [38]. Data was collected for stress ranges of 200 MPa to 800 MPa and maximum stresses of $0.37f_u$ to $1.0f_u$. A new wedge-type anchorage system was used to test the tendons, and it was observed that all tendons ruptured in the middle, indicating that the anchorages worked well and did not affect the tendons' fatigue strength. Both the stress range and the maximum stress were found to affect the fatigue resistance of the tendons, with larger maximum stresses and larger stress ranges both decreasing the fatigue life of the tendons. The predictions obtained using Whitney's method suggested that at the 95% confidence level, the CFRP tendons could be subjected to two (2) million cyclic loads without fatigue failure when the maximum stresses were 63.9%, 53.0%, and 36.8% of f_u for the stress ranges of 200 MPa, 400 MPa, and 600 MPa, respectively.

The Stuttgart Stadtbahn Bridge in Germany, a 127 m long arch bridge with CFRP hangers which carries the light rail from Stuttgart-Fasanenhof to the airport over the A8 motorway, was erected in May 2020. This bridge, shown in Figure 3, used CFRP hangers, which were developed by Swiss company Carbo-Link, an EMPA spinoff. These hangers underwent rigorous testing at EMPA, including fatigue testing. The hangers were subjected to 11 million load cycles and successfully

withstood the fatigue test without failure, once again proving that CFRP has excellent fatigue resistance [39].



Figure 3 – Stuttgart Stadtbahn Bridge using Carbo-link CFRP Hangers [40]

In comparison, BBR® has performed fatigue tests on steel cables with a maximum stress of 45% of f_u for a stress range of 200 MPa [41]. It is clear from the available literature that CFRP's creep behaviour is satisfactory and that its fatigue performance is superior compared to that of steel.

2.5. Anchorage Performance

The anchorage of CFRP cables is particularly challenging due to the low transverse strength of CFRP. Generally, two types of anchorage systems have been employed for CFRP cables and tendons: clamp-type anchorages and bond-type anchorages. Bond-type anchorages are significantly more popular than clamp-type anchorages, as clamp-type anchorages increase the risk of damaging the cables that are weak and brittle in the transverse direction, particularly when notches or imperfections are present. Hua et al. (2007) briefly discuss the performance of clamp-type anchorages, noting that protective covers can be used between CFRP cables and clamps to prevent the breakage of the outer wires. This study also notes slippage tends to be significant under

long-term loading for clamp-type anchorages [13]. Nevertheless, most available studies focus on describing the performance of bond-type anchorages [7, 13, 37, 4, 42, 43].

One of the earliest successful applications of CFRP cable anchorages was on the Storchen Bridge in Switzerland in 1996, as described by Meier and Farshad (1996). A bond-type anchorage was developed by EMPA, featuring a load-transfer-medium (LTM) whose density and stiffness vary along the length of the anchorage. In the new anchorage, the LTM stiffness is very low at the top of the anchor and gradually increases as it reaches the back of the anchor. Figure 4 illustrates the variation of the LTM stiffness along the length of the anchorage. This solution prevents shear stress concentrations associated with an abrupt increase in LTM stiffness while avoiding creep within the anchorage, which could be caused by having a low stiffness LTM. [4]

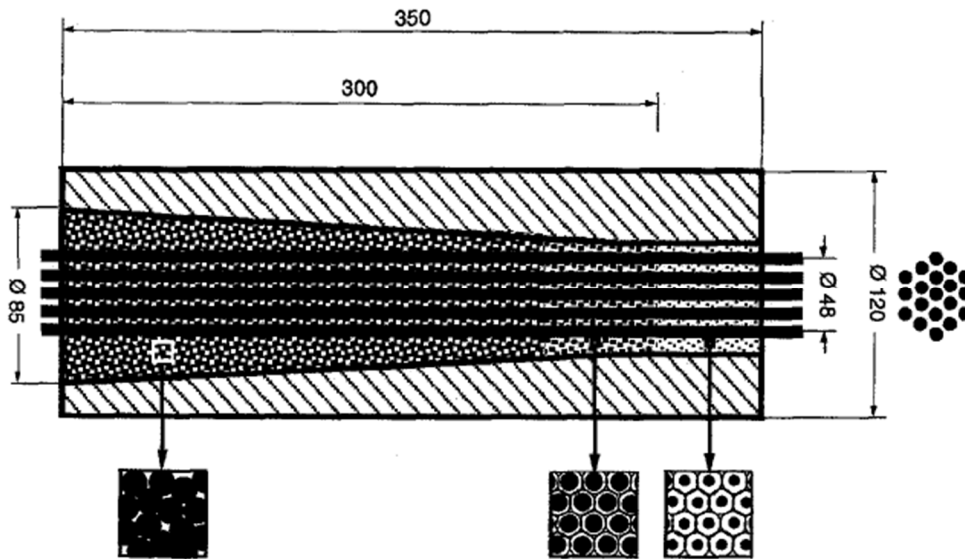


Figure 4 – Variation of LTM Density Within Anchorage Proposed by EMPA [4]

Significant work was done to develop the anchorages utilized in the Jiangsu University CFRP Footbridge, as described by Zhang et al. (2010). Laboratory tests were conducted to assess the performance of the CFRP anchorages that would be implemented in the pedestrian bridge. Tests were performed on 5 specimens: 3 specimens had cables with six wires, one specimen had a cable with 11 wires, and one had a cable with 16 wires. All cable/anchorage samples tested exhibited failure of the tendons, indicating that anchorages worked well. No slippage was observed. After four years results showed a slight increase in cable forces compared to when they were freshly anchored and no stress relaxation [37]. Mei et al. (2015) discussed the performance of the

anchorages after they were installed in the bridge. Bond-type anchorages were used for this bridge: half of them were filled with epoxy resin, and the other half with expansive cement. The anchorage sleeves were a hybrid between a straight pipe and a cone anchorage: the straight pipe portion was oriented at the loaded end to minimize shear stress concentrations associated with cone anchorages, while the cone portion was located at the unloaded end of the cable to minimize the anchorage length as straight pipe anchorages require a longer anchorage length to achieve the same anchorage force. The anchorage used for the pedestrian bridge is shown in Figure 5. Field tests performed for the pedestrian bridge showed that the anchorages worked well and that creep deformation of resin in the anchorages under service loads was not obvious [7].

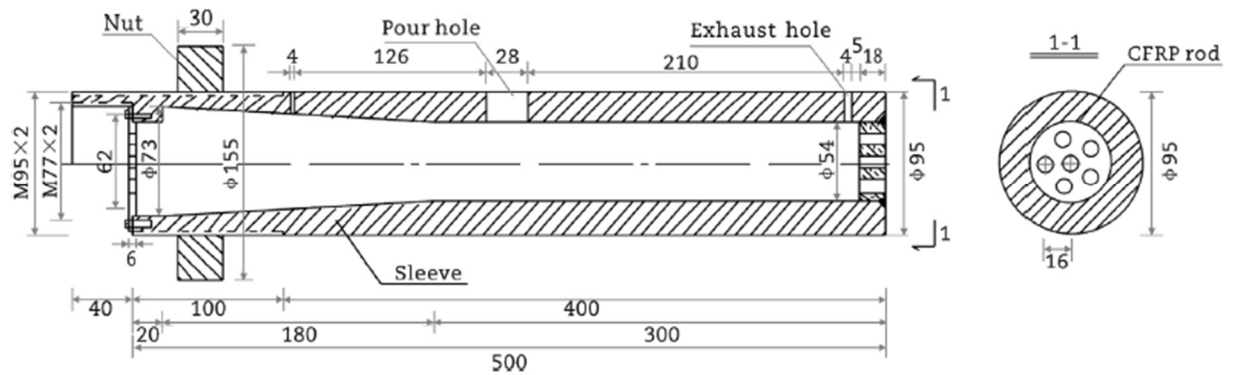


Figure 5 – Anchorage Used for Pedestrian Bridge in China [7]

The available literature on the anchorage of CFRP cables is limited to a handful of real-life applications. Both the Storchen Bridge in Switzerland and the Jiangsu University CFRP Footbridge in China are short-span cable-stayed bridges with main spans of 63 m and 30 m, respectively, which means that the cable loads for those bridges are not significant. There is no data available for the anchorage of CFRP cables with large diameters meant to withstand higher tensile loads associated with long-span cable-stayed bridges. Further studies should be conducted to determine anchorage capacity and optimal anchorage configuration for large cables. For the purposes of this thesis, it is assumed that the load capacity of the cables is governed by the cable cross-section rather than the anchorage, which is a reasonable assumption given that the anchorage/cable samples described by Zhang et al. (2010) exhibited failure of the tendons rather than failure of the anchorages [37].

2.6. Dynamic Performance

The dynamic performance of cable-stayed bridges depends on several parameters, including the vertical bending natural frequency of the bridge, the torsional natural frequency of the bridge, the natural frequency of the cables, and damping characteristics. An important consideration in the design of cable-stayed bridges is flutter which occurs when vertical bending and torsional vibrations coincide. Therefore, having a large gap between the vertical bending and torsional frequencies is beneficial. The studies conducted thus far agree that the vertical bending natural frequency is slightly lower for bridges with CFRP cables than bridges with steel cables, while the lateral bending and torsional frequencies are higher for CFRP bridges than for steel bridges [10, 12, 16]; this is summarized in Table 1 below. The increased ratio of torsional frequency to vertical bending frequency reduces the flutter risk; thus, bridges with CFRP cables have a better dynamic performance than bridges with steel cables. However, it is important to note that the vertical natural frequency for CFRP bridges is cited as being lower due to the lower elastic modulus of CFRP cables, and that when the span of the bridge increases past a certain threshold, the bridge with CFRP cables will have a higher vertical bending frequency than the bridge with steel cables according to Kuihua et al. (2017) [12]. Additionally, if CFRP cables are sized based on the principle of equivalent axial stiffness, the vertical bending frequency of the bridge may actually increase due to the decrease in the overall mass of the bridge associated with using lighter CFRP cables according to Xin-Jun and Lei-Dong (2007) [17]. Considering this in the context of HM CFRP cables when conducting the parametric study is important. The torsional natural frequency is cited by Kuihua et al. (2017) as being higher for CFRP bridges due to the smaller mass and moment of inertia of CFRP cables [12], which will remain true even if HM CFRP is used for the cables.

Table 1 – Summary of Natural Frequencies of Bridges with Steel and CFRP Cables

Source	Main Span (m)	Vertical Bending Frequency (Hz)		Torsional Frequency (Hz)	
		Steel	CFRP	Steel	CFRP
[16]	1400	0.1986	0.1771	0.4635	0.4727
[12]	1000	0.2080	0.2031	0.4883	0.4999
[10]	648	0.1280	0.1263	1.090	1.314

Regarding the natural frequency of cables themselves, there is a consensus that the natural frequency of CFRP cables is higher than that of steel cables [8, 13, 18, 44]. This is significant because the behaviour of a cable-stayed bridge as a whole is governed by low-frequency vibration, and increasing the vibration frequency of the cables makes bridge-cable resonance even less likely for bridges with CFRP cables than bridges with steel cables according to Yang et al. (2015) [44]. Additionally, Hua et al. (2007) note that rain-wind-induced vibration results in low-frequency high-amplitude vibrations, and thus having higher natural cable frequencies helps avoid rain-wind-induced resonance [13]. Several papers also comment on the dynamic response of CFRP cables subjected to parametric vibration. There is a consensus that parametric vibration has a lesser effect on CFRP cables than on steel cables. Particularly, the parametric vibration amplitude is cited to be smaller for CFRP cables than steel cables [8, 45, 46, 47, 48].

When it comes to bridge vibration due to traffic, there is a consensus that the vertical midspan displacements of a long-span cable-stayed bridge are dominated by static deflection and long-period vibration and that the short-period vibrations caused by traffic have little effect on the dynamic behaviour of the bridge as large amplitude cable-bridge coupling vibration due to traffic will not occur [8, 49, 50]. The influence of traffic speeds and vehicle models was found to have little effect on the dynamic response of the bridge [8, 49]. Most studies have been performed for long-span cable-stayed bridges, and these findings are expected to change if similar analyses are performed for short-span cable-stayed bridges.

Various conclusions have been drawn regarding the damping characteristics of CFRP cables. Xie et al. (2014) calculated damping ratios for steel and CFRP cables using theoretical equations and compared them to the results of a cable vibration test conducted for steel and CFRP cables of various diameters, concluding that for both steel and CFRP cables a damping ratio of 0.05 can be used [8]. Yang et al. (2015) conducted test on reduced-scale cable models. It determined that damping ratios of in-plane (vertical) vibration were higher for CFRP cables than steel cables. In comparison, out-of-plane (lateral) damping ratios were lower for CFRP cables [44]. In a third study, Li et al. (2015) examined the relationship between vibration amplitude and damping and found that CFRP cables have a smaller damping ratio than steel cables at the same vibration amplitude [51]. The results of these studies are quite scattered, and it would be beneficial to conduct additional studies to obtain more consistent results. However, cable vibration will not be

included in the scope of this thesis in order to retain the focus on the global performance of cable-stayed bridges.

2.7. Aerodynamic Stability

Cable-stayed bridges with CFRP cables have characteristics which contribute both to the increase and the decrease in critical wind speed compared to a bridge with steel cables. All studies agree that using CFRP cables increases the torsional natural frequency due to the lighter mass of the cables [10, 12, 16, 17, 18]. The increase in torsional natural frequency contributes to the increase in critical wind speed by increasing the flutter resistance of the bridge. When it comes to the vertical bending frequency, Kuihua et al. (2017) and Xiong et al. (2011) argue that it decreases due to the lower stiffness of the CFRP cables [12, 16], while Xin-Jun and Lei-Dong (2007) state that the vertical bending frequency may increase due to the decrease in mass associated with CFRP cables, particularly if they are sized based on the principle of equivalent axial stiffness [17]. Xiong et al. (2011) also note that the vertical bending frequency of a bridge may increase if a lighter material is used for the deck, or once the bridge span reaches a certain threshold at which the CFRP cables become stiffer than steel cables due to the sag effect [16]. An increase in vertical bending frequency acts to decrease the critical wind speed as it decreases flutter resistance.

The question arises whether the critical wind speed increases or decreases for a bridge with CFRP cables, all factors considered. G. Xie et al. (2015) and Kuihua et al. (2017) conclude that the ratio of the torsional to vertical bending frequencies increases for CFRP bridges, thus increasing the critical wind speed of CFRP bridges [10, 12]. Xin-Jun and Lei-Dong (2007) conclude that while CFRP bridges experience an increase in torsional frequency, there is still a decrease in the critical wind speed due to the higher vertical bending frequency associated with the lighter cables [17]. Xiong et al. (2011) state that for shorter spans and heavier decks, the critical wind speed of CFRP bridges sees an increase, while for longer spans and lighter decks, it decreases [16]. Cheng and Lau (2006) note that if cable vibration is neglected, critical wind speed is increased for CFRP cables since they increase the torsional frequency, but if cable vibration is considered, the critical wind speed decreases for CFRP compared to steel since CFRP cable vibration has a lesser effect on the dynamic behaviour of the bridge, resulting in less energy dissipation due to cable vibration [18]. X. Xie et al. (2014) confirm that under lateral wind load, vertical vibration amplitudes of CFRP cables are smaller than those of steel cables and that CFRP bridges are less subjected to

wind fluctuating vibrations, which is consistent with the claim that CFRP cables contribute less to energy dissipation than steel cables. Additionally, the authors note that the transverse bending moment and deformation of the deck at midspan is slightly larger for a bridge with steel cables than a bridge with CFRP cables due to the smaller windward area of CFRP cables compared to steel cables [8].

In short, [8, 10, and 12] state that the use of CFRP cables can increase the critical wind speed, [17 and 18] state that CFRP cables are associated with a slight decrease in the critical wind speed, and [16] provides arguments for both outcomes. In all cases, the difference in aerodynamic performance between the two cable types is cited as being small. These papers generally only perform a comparison of CFRP cables and steel cables in the context of a single bridge and do not examine the effect of different span lengths on the aerodynamic stability of cable-stayed bridges. It is expected that the aerodynamic performance of a CFRP bridge compared to that of a steel bridge will vary depending on the span length, method of cable sizing, and stiffness of CFRP material.

2.8. Seismic Performance

Very limited information is available regarding the seismic performance of CFRP bridges compared to steel bridges. From the studies and abstracts which were accessible, the consensus is that the seismic response of bridges with CFRP cables is lower than that of bridges with steel cables [22, 24, 29]. Yang et al. (2020) compared the static and dynamic performance of bridges with various hybrid cable arrangements found that both the vertical midspan displacement of the deck and the longitudinal pylon tip displacement were smaller for a bridge with pure CFRP cables than for a bridge with pure high-strength steel cables under the same seismic loading [29]. Adanur and Gunaydin (2013) analyzed the forces and displacements of the deck and pylons subjected to the same seismic loading for bridges with steel cables, CFRP cables designed using the equivalent strength principle (CFRP-1), and CFRP cables designed using the equivalent stiffness principle (CFRP-2), and found that the response of the steel and CFRP-2 bridges under earthquake loads were practically the same, while the CFRP-1 bridge experienced higher deck and pylon displacements than the steel bridge [52]. Based on the available literature, the seismic performance of a bridge with CFRP cables appears to be the same or better than that of a bridge with steel cables. Seismic analysis is, therefore, not included in the scope of this thesis.

2.9. Availability and Structural Applications of High Modulus CFRP

Standard modulus CFRP products typically have an elastic modulus of 150 to 160 GPa. An example is the tendons manufactured by Tokyo Rope with an elastic modulus ranging from 165 GPa for uni-strand tendons to 150 GPa for 7-strand tendons to 145 GPa for larger tendons [19]. At present, various manufacturers also offer HM CFRP products. Typically these manufacturers provide the stiffness parameters for the carbon fibers rather than the composite CFRP material, but the composite material stiffness can easily be determined from the fiber stiffness by multiplying the stiffness of the fibers by the fiber volume fraction which is typically between 50 and 60% [34]. The elastic modulus of the matrix (typically polyester, epoxy, or vinylester resin) is generally very low, ranging between 2.0 and 4.0 GPa [34], so its contribution to the stiffness of the composite can be neglected, and thus the above method for determination of stiffness of the composite based on the fiber stiffness is a very good approximation.

Toray Composite Materials America offers a line of HM carbon fibers with an elastic modulus ranging between 343 GPa and 588 GPa which translates to a composite elastic modulus of 202 GPa to 360 GPa assuming a 60% fiber volume as per the product data sheets [53]. DragonPlate is another manufacturer that offers HM fibers with a stiffness which ranges from 235 GPa to 455 GPa, translating to a composite stiffness which ranges from 140 GPa to 270 GPa assuming a 60% fiber volume fraction [54]. Epsilon Composite also offers HM fibers with stiffnesses ranging from 230 GPa to 900 GPa, translating to a composite material stiffness of 140 GPa to 540 GPa [55]. While all of these fibers can be used for pultrusion, the HM fibers are considerably more expensive than normal modulus fibers, and thus at present, they may not be a financially viable option for large-scale structural applications. While there is limited information available on the cost of HM CFRP, one source cited the cost of ultra-high modulus CFRP fibers being 36 times higher than the cost of standard modulus CFRP fibers in 2010 [56]. However, they are already available on the market, and it is conceivable that the costs of CFRP, and in particular HM CFRP, will continue to decrease, making these materials more likely to be considered for bridge construction in the future.

When it comes to the use of HM CFRP in structural applications, it appears that the vast majority of this material's structural applications involve the strengthening of steel members. Schnerch et al. discuss strengthening of steel structures using carbon fibers manufactured by Mitsubishi Chemical Corporation with an elastic modulus of 640 GPa and a fiber volume fraction of 55%

[57]. Wu et al. (2012) studied the bond between HM CFRP laminates and steel using CFRP laminates with an elastic modulus of 460 MPa [58]. Shaat and Fam (2009) discuss the application of HM CFRP plates to strengthen slender steel columns to control buckling [59], and Dawood et al. (2007) discuss the use of HM CFRP plates to strengthen composite steel-concrete sections [60]. A search of the available databases showed many similar search results, and no studies discussed the use of HM CFRP for the tendons of prestressed concrete or for cables of a cable-supported structure.

Chapter 3. Methodology and Assumptions

This thesis aims to examine the effect of cable material on the static and dynamic performance of a cable-stayed bridge through a parametric study which is performed using a finite element model of the bridge. This chapter details the parameters, assumptions, and methodology used to carry out the parametric study. First, a case study for the bridge is presented. This is followed by a description of the finite element model, which was created to analyse the bridge, and a description of all of the assumptions adopted during the modelling process. Finally, the methodology used to carry out the parametric study is discussed.

3.1. A Case Study: Replacement of Alexandra Interprovincial Bridge by a Cable-Stayed Bridge with CFRP Cables

The Alexandra Interprovincial Bridge (Figure 6) spans over the Ottawa River in Ottawa, Canada and connects the provinces of Ontario and Quebec. This steel truss balanced cantilever bridge was constructed in 1901 and is reaching the end of its useful life. It is due for a replacement and its construction is currently planned for 2028 [61]. The proposal to replace the existing Alexandra Bridge serves as the case study is used to design a hypothetical bridge over the Ottawa River. This hypothetical cable-stayed bridge is the basis for the parametric study.



Figure 6 – Existing Alexandra Interprovincial Bridge

3.1.1. Overall Geometry

The Alexandra Bridge consists of trestle spans on the south side with a length of 17.770 m, two anchor spans with a length of 75.286 m, a main span of 169.393 m which includes two cantilever spans and the suspended span, two spans north of the main structure with span lengths of 75.286 m and 42.672 m, and north trestle spans with a combined length of 118.872 m. The total length of the bridge, including all trestle spans, is approximately 575 m. The spans of the existing bridge are illustrated in Figure 7.

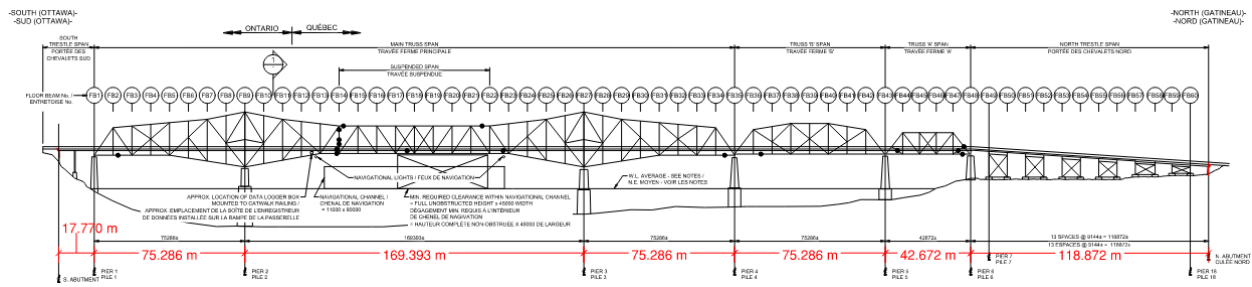


Figure 7 – Alexandra Bridge Spans

The proposed replacement bridge consists of a main span and two side spans with a combined length approximately equal to the combined length of the existing Alexandra Bridge between the trestle spans. The combined length of the truss spans is approximately 438 m, whereas the combined length of the main and side spans of the proposed bridge is 450 m. A similar configuration of approach spans is proposed beyond the side spans of the cable-stayed bridge. The approach spans of the new bridge consist of one 20 m long span at the south approach (comparable to the existing 17.770 m trestle span at the south approach) and six 20 m long spans at the north approach (comparable to the 18.288 m spans between centerlines of pylons at the north approach). The bridge deck of the cable-stayed bridge will be located at approximately the same elevation as the existing deck to avoid excessive excavation or grade raises.

The principles outlined in Cable-Stayed Bridges by Holger Svensson [62] were used to complete the preliminary sizing of the proposed bridge. Provided that the total length of the cable-stayed bridge is 450 m, the lengths of the main span and side spans were chosen in line with the side-to-main span ratio of 0.4 to be 250 m and 100 m respectively, as recommended by Svensson. The pylon height above the deck surface was determined to be 50 m based on Svensson's recommendation for the pylon height to be 20% of the main span length. A semi-fan cable

arrangement was selected as it allows for more efficient use of cable material than a harp arrangement while avoiding a congested anchorage zone at the top of the pylons, which would occur with a fan arrangement. There are many examples of cable-stayed bridges with cables spaced far apart and many examples of bridges with closely spaced cables. A relatively small cable spacing was chosen as it would reduce the bending moments in the deck between adjacent cables and reduce the forces in the cables. A cable spacing along the deck of 10 m was chosen for the main span, corresponding to 12 cables on each side of the pylon. A cable spacing of 8.5 m along the deck was chosen for the side spans. The cable spacing along the pylon was selected to be 1.2 m such that the cables are spread out over approximately one quarter ($\frac{1}{4}$) of the pylon height. The layout of the proposed cable-stayed bridge is shown in Figure 8.

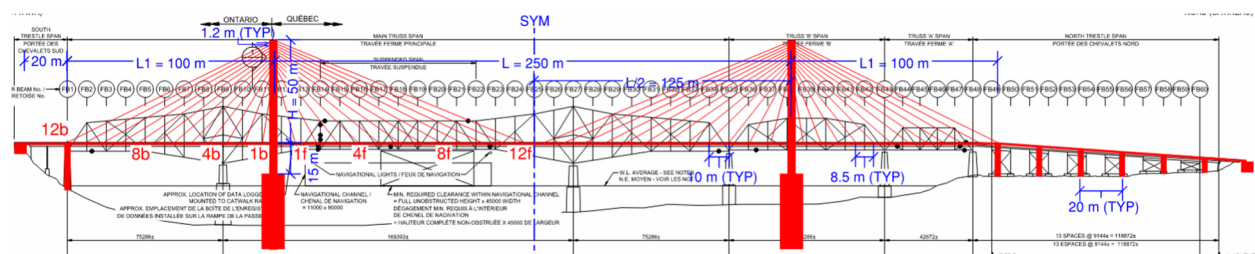


Figure 8 – Geometry of Proposed Bridge

3.1.2. Cross Section

The existing Alexandra Bridge accommodates one 4.877 m wide northbound lane, one 5.710 m wide southbound lane, and a 4.877 m wide multi-use pathway (MUP) along the west side of the bridge, which accommodates pedestrian and bicycle traffic, as shown in Figure 9.

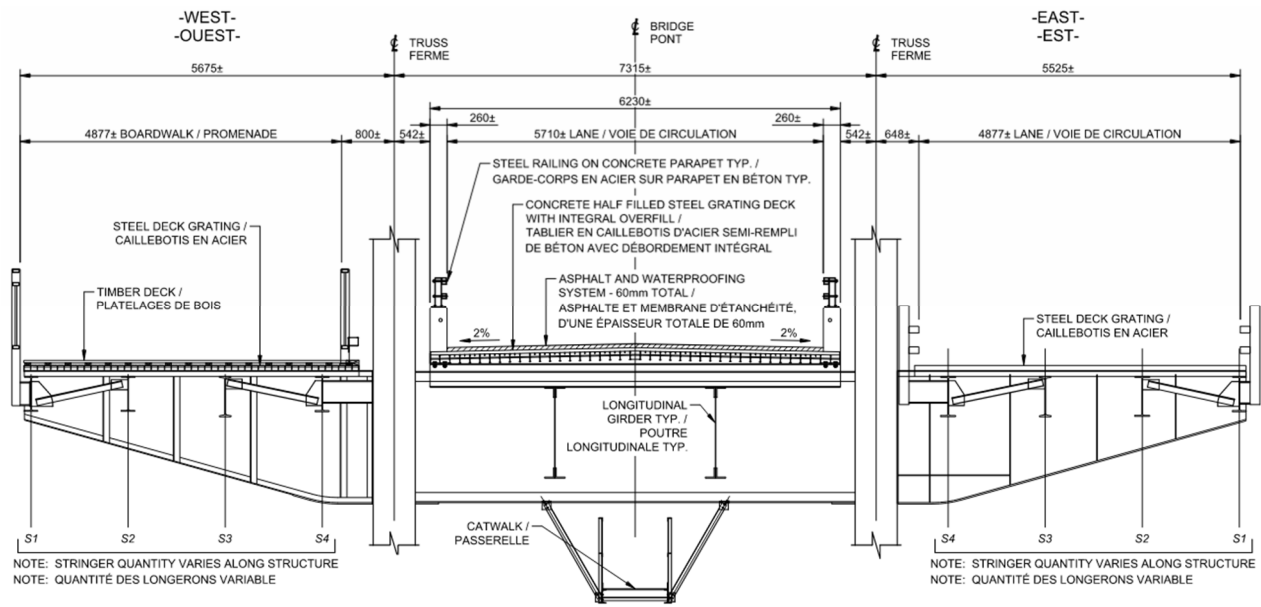


Figure 9 – Existing cross-section of Alexandra Bridge

For the design of the new bridge, it is proposed to maintain similar traffic capacity and have one lane of vehicular traffic as the opportunities to widen the approach roadways are limited. It is proposed to have a pedestrian sidewalk on one side of the bridge and a bicycle lane on the other side, as the existing configuration of having pedestrian and bicycle traffic together in one area creates unsafe conditions. The cross-section for the proposed replacement bridge is shown in Figure 10.

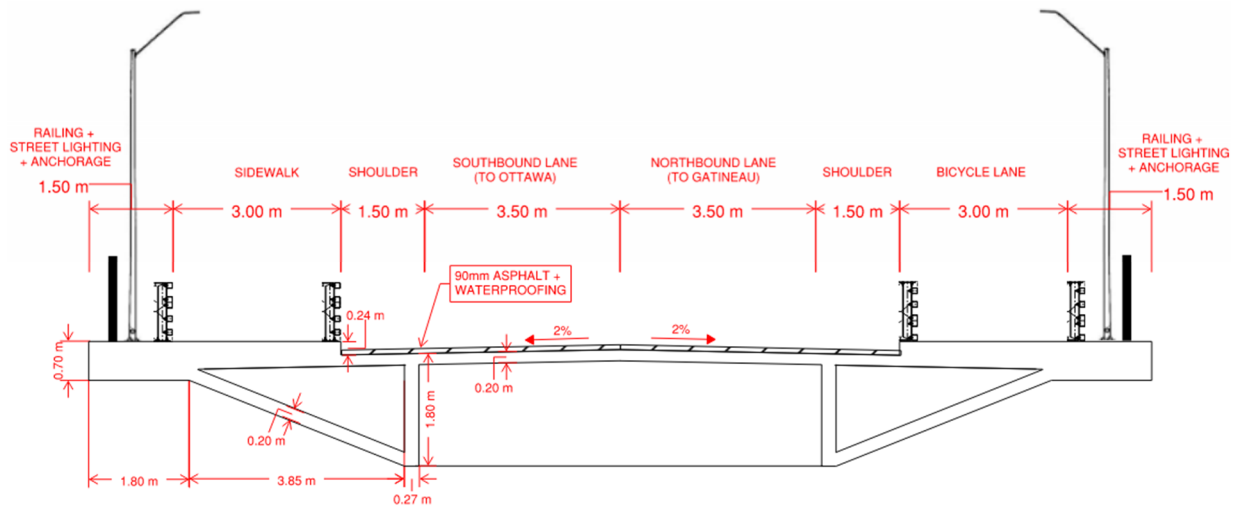


Figure 10 – Cross section of proposed cable-stayed bridge

The lane, shoulder and sidewalk clearances were determined using the 2017 Transportation Association of Canada (TAC) Geometric Design Guide and the Ontario Ministry of Transportation (MTO) Supplement to 2017 TAC Geometric Design Guide. A design speed of 70 km/h and an average annual daily traffic (AADT) of greater than 3000 was assumed when designing the cross-section.

3.1.3. Design and Analysis Assumptions

A preliminary design was completed in order to determine reasonable member sizes for the proposed cable-stayed bridge. As a starting point, the member sizes of an existing cable-stayed bridge with a similar span and deck width were adopted for the deck and pylons of the proposed bridge, modified slightly to suit the needs of the bridge. Preliminary sizing of the cables was performed via hand calculations guided by simplified design methods for the cables. While cable-stayed bridge design is a complex and iterative process, no iterations were performed for the design of the deck and pylons of the proposed bridge, as this thesis is not an exercise in the detailed design of cable-stayed bridges. It was assumed that the selected pylon and deck cross sections were adequate as they were used for an existing bridge with a larger span and wider deck. Only one design iteration was performed to size the cables. A model was run with the initial cable sizes, and the cable forces extracted from the model were used to resize the cables as necessary to optimize the use of material.

It should also be noted that certain assumptions were made regarding the detailing of the bridge. No detailed design was performed for the cable anchorages, reinforcing of the pylons and deck, the bearings, or the articulation of the bridge. It was assumed that the cable anchorages at the deck and pylon are not the limiting factor in the capacity of the cables. It was also assumed that adequate reinforcement was provided to meet the requirements of the Ultimate and Serviceability Limit States, and thus reinforcement design was not conducted. Additionally, the construction stages and construction loading are not examined; only the final state of the bridge is analyzed.

The CHBDC includes provisions for long-span bridges, including cable-stayed bridges in §5.10 and throughout the code. The Preface of the CHBDC states that “Section 3 does not specify limits on the span lengths for application of the truck and lane loads. Accordingly, long-span requirements have been developed and appear in Section 3 and elsewhere in [the CHBDC,

although these requirements] should not be considered comprehensive”. §1.1.1 (Scope of Code) states that “there is no limit on span length, but [the CHBDC] does not necessarily cover all aspects of design for every type of long-span bridge”. One requirement of the CHBDC is that refined methods of analysis must be used for long-span bridges as stated in §5.9, and one of the refined methods of analysis listed is finite element analysis which was used for the parametric study. There are also requirements which state that wind tunnel tests should be conducted for long-span bridges, but wind analysis is outside the scope of this thesis. As such, for the purposes of this thesis, it is acceptable to use the CHBDC for application of live loads and design of cable cross-sectional area.

The following load combinations outlined in Table 3.1 of CHBDC will be considered for the analysis and design of the proposed bridge in the parametric study, as presented in Table 2.

Table 2 – CHBDC Load Combinations used for Analysis

	ULS-1	ULS-2	SLS-1	FLS-1
$\alpha_{D,CIP}$ – dead load (cast-in-place)	1.20	1.20	1.00	1.00
$\alpha_{D,P}$ – dead load (precast)	1.10	1.10	1.00	1.00
$\alpha_{D,SDL}$ – superimposed dead load (barriers, light fixtures)	1.10	1.10	1.00	1.00
$\alpha_{D,WS}$ – wearing surface (90mm of asphalt)	1.50	1.50	1.00	1.00
α_L – live load (vehicular)	1.70	1.60	0.90	1.00
$\alpha_{L,PED}$ – live load (pedestrian)	1.36	1.28	-	-
α_K – thermal effects (expansion and contraction)	-	1.15	0.80	-

Wind, seismic, ice, and collision load analyses are not conducted for the bridge, and thus the CHBDC load combinations ULS-3 through ULS-9 are not examined in the parametric study. The parametric study is limited to analyzing the bridge under dead loads, live loads, and thermal effects. To conduct a proper wind analysis for long-span bridges, wind tunnel tests and/or advanced analysis methods are required as per §3.10.5 of CHBDC, which is outside the scope of the thesis. However, as part of the parametric study, a very basic aerodynamic stability check will be performed by observing the ratio of the first torsional frequency to the first bending frequency.

Since wind and seismic analyses are outside the scope of the thesis, only the longitudinal behaviour of the bridge is examined. Only the displacements, shear and bending moments in the pylons and deck due to longitudinal and vertical loads are noted in the parametric study.

3.1.4. Preliminary Member Sizing

Preliminary sizing of the pylons, deck and cables was conducted to ensure that the member sizes selected for the analysis were reasonable. The Pasco-Kennewick Bridge was used as a starting point due to its similar span length and deck width. The Pasco-Kennewick Bridge (Figure 11) was constructed in 1978 in Washington, USA, connecting the cities of Pasco and Kennewick across the Columbia River. This cable-stayed bridge has a main span of approximately 299 m and side spans which are approximately 124 m long. The total width of the deck is 24.33 m [62]. In comparison, the cable-stayed bridge proposed as part of the case study has a main span length of 250 m, 100 m long side spans, and a 19 m wide deck.



Figure 11 – Pasco-Kennewick Bridge [63]

The pylons of the Pasco-Kennewick Bridge reach a height of 57.0 m above the deck and 69 m above the foundations which consist of massive wall piers founded on spread footings. Each pylon consists of two legs connected by a cross frame at the top of the pylon. Each leg has dimensions of 3.35 m by 3.05 m at the top and 3.35 m by 4.57 m at the base [62]. A similar pylon geometry was adopted for the case study bridge. The pylon height of the proposed bridge is 50 m above the deck and 65 m above the foundation. Each leg has dimensions of 3.35 m by 4 m at the top and 3.35 m by 5.5 m at the base. The legs' depth was increased to reduce the effects of pylon deflection on the cable tightening procedure. Like the Pasco-Kennewick Bridge, the pylons of the case study bridge are also supported on massive wall pier foundations, which are assumed to provide a fixed condition at the base of the pylon legs.

The deck cross-section of the Pasco-Kennewick bridge (Figure 12) consists of precast segments post-tensioned on site [62]. It is assumed that the construction of the proposed bridge is similar to that of the Pasco-Kennewick Bridge. The pylons of the proposed bridge are assumed to be constructed using cast-in-place concrete, while the deck is assumed to consist of precast concrete segments post-tensioned on site. The effects of deck shrinkage are not considered, as the deck is comprised of mature precast segments, with most of the shrinkage occurring before they are installed. As per Svensson [62], concrete decks of cable-stayed bridges tend to creep towards the moments of the beam on rigid supports, and since the cables will be tensioned in such a way that the bending moments in the deck under dead loads in the SLS-1 load combination will be those of a beam on rigid supports, creep need not be considered for the preliminary design. It is assumed that the bridge is modelled in the final configuration after all the creep and shrinkage of the pylons and relaxation of the cables has already happened, and thus creep and shrinkage of the pylons is not considered in the preliminary design. Of course, these factors need to be considered when performing detailed calculations, but this is outside the scope of this thesis.

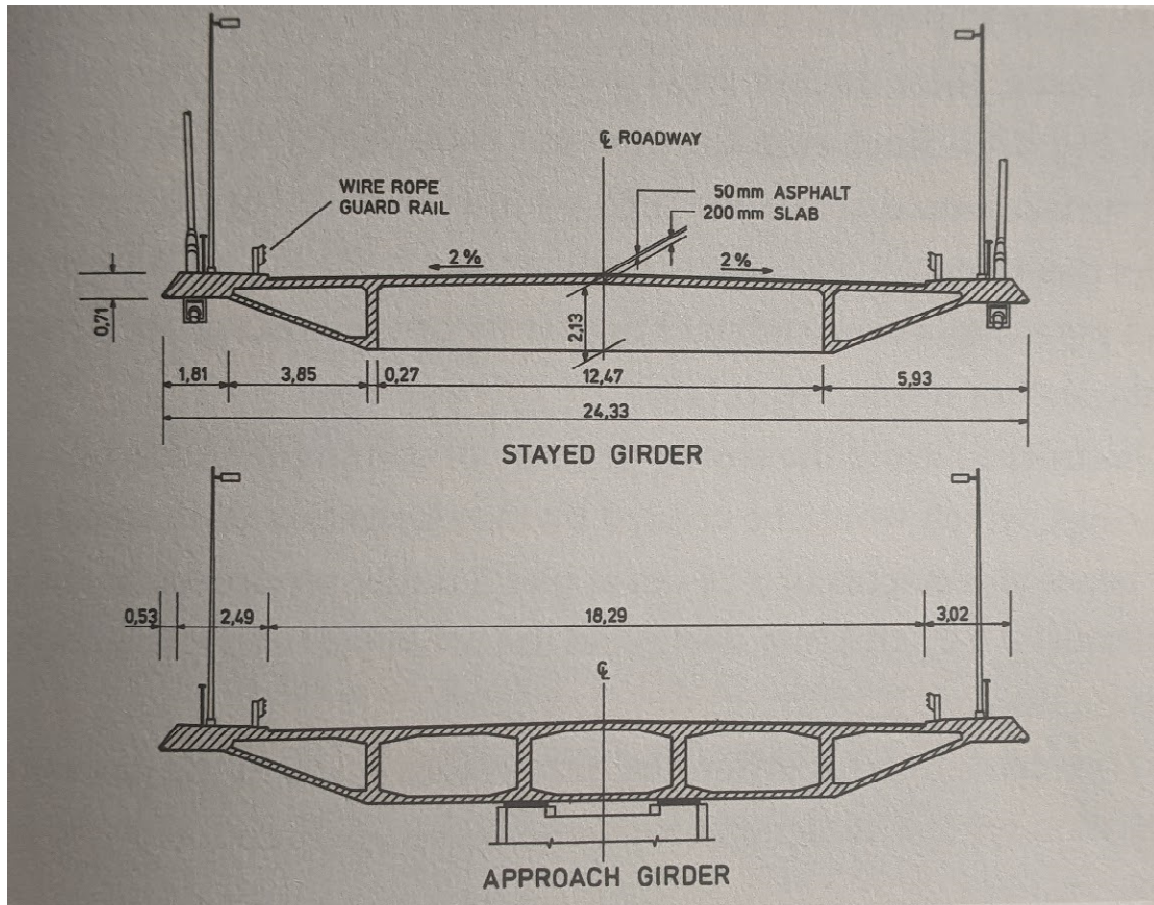


Figure 12 – Pasco-Kennewick Bridge Cross Section [62]

A similar deck cross-section was adopted for the proposed bridge, but the dimensions were adjusted to suit the width of the proposed bridge, as shown in Figure 10. A preliminary check was completed to determine the amount of prestressing required to satisfy stress requirements under the Serviceability Limit State to ensure that the selected deck cross-section has adequate capacity. To determine the bending moments in the deck under dead loads, it was assumed that the deck behaves as a beam on rigid supports located at the cable anchorages, a design assumption suggested by Svensson. The bending moments due to live loads were calculated using the procedure outlined by Svensson based on the assumption that the deck behaves as a beam on an elastic foundation under live loads.

The cables of the Pasco-Kennewick bridge have a fan arrangement. In this study, a semi-fan cable arrangement was selected for the proposed bridge to avoid issues with anchorage congestion at the top of the pylon while still providing a relatively efficient design. The backstays (the outermost cables of the bridge) are assumed to be anchored into the ground rather than attached to the deck.

This is because the backstays are typically used to balance the bridge, and the high tensile forces incurred in the backstays would essentially double the compression in the deck.

Preliminary sizing of the cables was conducted using the preliminary design guidelines outlined by Svensson [62]. First, the backstays were sized based on maximum load considerations. The dead load component in the backstays was calculated by determining the moment about the pylon due to the factored weight of the imbalanced portion of the central span, the length of which is equal to $0.5L - L_1$ (where L is the length of the main span and L_1 is the length of the side span), and then determining the required vertical force in the cable that results in a total sum of moments about the pylon of zero. The maximum backstay force due to live load is calculated by placing a factored CL-625-ONT lane load on the main span with the truck placed at the center of the main span and determining the required reaction in the backstay which would result in a total sum of moments about the pylon of zero. The required number of strands per backstay was determined by dividing the total force by the factored allowable stress of 1023 MPa that is calculated by multiplying the ultimate strength of 1860 MPa by a resistance factor of 0.55 as specified in §10.5.7 of CHBDC. The cable sizes were also chosen such that the stress range in the cables is limited to 200 MPa to align with the recommendations of [38] for CFRP cables and the experiments which were conducted by BBR® for steel cables [41]. The cable size was selected based on the cable sizes supplied by BBR® [41]. All load factors for the maximum backstay force calculations were conducted using the ULS-1 load combination.

The backstays were also sized based on fatigue considerations. A single CL-625-ONT truck with a load factor of 1.25 as per the FLS-1 load combination was placed along the bridge to determine the maximum range of stresses in the backstays. The maximum decrease in the backstay force due to the live load was calculated by placing the truck at a distance of $L_1/3$ from the end of the side span and determining the required reaction in the backstay, resulting in a total sum of moments about the pylon of zero. The maximum increase in the backstay force due to live load was determined by placing the truck at the midspan of the main span. The required number of strands and corresponding cable size were calculated based on a maximum allowable stress range of 200 MPa [41].

The remaining stay cables were sized based on the factored weight of the tributary length of deck they support. In addition, the conservative assumption is that each cable must develop a sufficient

force to support a supplementary load imposed by a full factored CL-625-ONT truck, with load factors as per the ULS-1 load combination. The cable force for fatigue considerations was assumed to be the weight of a full CL-625-ONT truck factored as per the FLS-1 load combination.

3.2. Finite Element Model

To conduct the parametric study, a finite element model was created for the proposed bridge. The model was built using the finite element software CSiBridge, created by Computers and Structures, Inc. This section discusses the modelling methodology and assumptions.

3.2.1. Model Structure

The bridge model is shown in Figure 13. The deck, pylons, and cables are all modelled using frame elements.

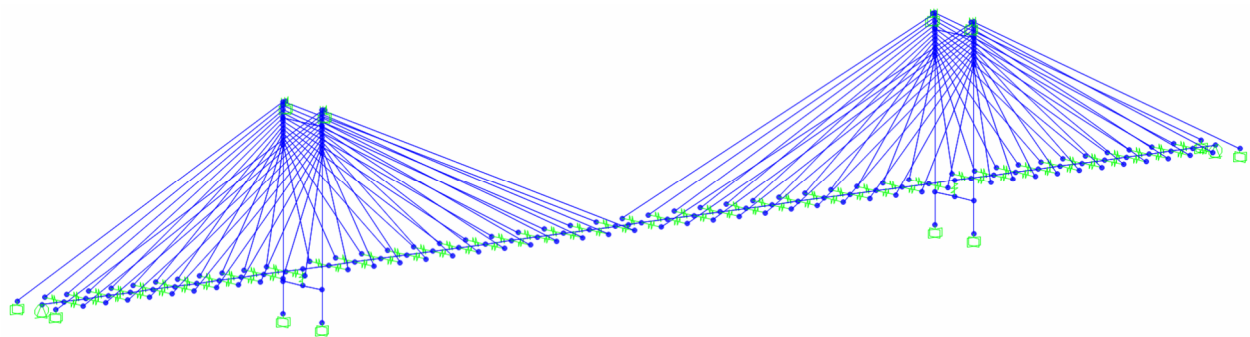


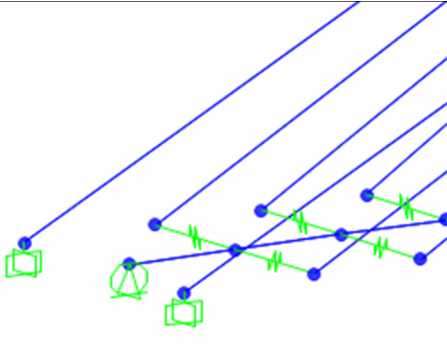
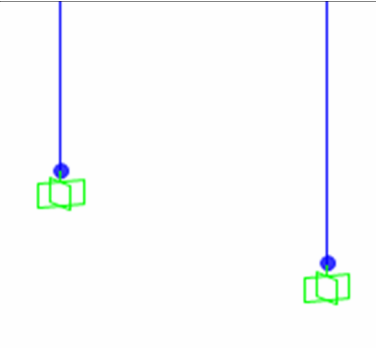
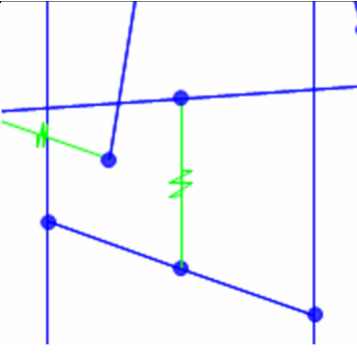
Figure 13 – Bridge Finite Element Model

The pylons are modelled using frame elements which have a variable cross-section along the length to reflect the tapered geometry of the pylons. The deck of cable-stayed bridges is modelled using shell elements in some cases, but for this thesis, frame elements were used because the transverse bending in the deck is not observed in this thesis; only the global behaviour is observed. The cables are modelled using frame elements despite the fact that CSiBridge has the option to model the cables with cable elements. This is because the cable elements did not act as desired during the modelling process, and there were issues with the convergence of the model. Frame elements can be used for cables, provided that they are always under tension in all load cases. The cables are extremely slender, so the fact that they have a small amount of bending stiffness is not a concern for the analysis. In the literature review it was found that the deck and pylons are typically modelled with frame elements, but different researchers have modelled cables in different

ways, including with bar elements [11, 17], truss elements [18, 31], and catenary cable elements [28, 46]. In this thesis, the frame elements used for the cables behave very similar to truss elements since moment releases are applied at the ends of the frame elements.

The pylon legs are fixed at the base as they are assumed to be founded on a wall pier, which is significantly (close to 50 times) stiffer in bending than the pylons. The ends of the deck are fixed in the transverse direction but are free to move in the longitudinal direction to allow the bridge to expand and contract under thermal effects. The deck is supported at the pylons on pinned bearings which allow rotation of the deck but not translation. While floating and roller connections between the deck and pylons of cable-stayed bridges are the most common, there are several examples of bridges which have a rigid connection between the deck and the pylons, including the Normandy Bridge and the Second Bridge across the Panama Canal [65]. A pinned connection between the deck and pylons was chosen to explore the effects of temperature loads since it is expected that temperature loads would have a larger effect on a cable-stayed bridge whose deck is restrained against translation relative to the pylons. The backstays are pinned to the ground rather than connected to the deck to avoid introducing excessive compression into the deck. The cables are connected to the deck via rigid links and connected to the pylons directly. Moment releases about the 2 and 3 axes (the one (1) axis being parallel to the member) are included at the ends of the cables to prevent bending moments from developing at the ends of the cables. The boundary conditions of the model are shown closer in Table 3.

Table 3 – Finite Element Model Boundary Conditions

		
Roller supports at the deck end, separate ground supports for backstays, and rigid links connecting cables to deck	Pylon legs on fixed supports	Deck supported by pylon using a rigid link with moment releases about the local 2 and 3 axes

The material properties used for the pylons, deck and steel cables are summarized in Table 4.

Table 4 – Material Properties of Bridge with Steel Cables

Parameter	Pylons	Deck	Cables
Material:	Concrete	Concrete	Steel
Unit weight, γ (kN/m ³):	24.0	24.5	77.0
Elastic modulus, E (MPa):	35,000	35,000	190,000*
Shear modulus, G (MPa):	14,580	14,580	-
Material strength, f'_c and f_y (MPa):	60	60	1860
Coefficient of thermal expansion, α (°C ⁻¹):	10×10^{-6}	10×10^{-6}	12×10^{-6}

*Although steel typically has an elastic modulus of 200 GPa, an elastic modulus of 190 GPa was assumed for the cables as per Svensson [62]

The cross-section properties of the pylons and deck are summarized in Table 5. The cross-section properties were calculated by hand based on the deck and pylon cross sections of the Pasco-Kennewick bridge, modified to suit the geometry of this bridge as described in Section 3.1.4. The deck cross-sectional properties do not account for the transverse ribs since the transverse ribs mainly affect the deck behaviour in the transverse direction and have little effect on the properties of the deck in the longitudinal direction. The weight of the ribs is, however, accounted for in the application of the dead loads.

Table 5 – Pylon and Deck Section Properties

Parameter	Pylons - top	Pylons - base	Deck
Area, A (m ²):	13.4	18.4	9.53
Moment of inertia, I_3 (m ⁴):	17.9	46.4	2.24
Moment of inertia, I_2 (m ⁴):	12.5	17.2	376
Torsional constant, J (m ⁴):	24.8	42.8	3.94

The cable properties are discussed in more detail in Section 3.2.4.

3.2.2. Applied Loads

The following loads were applied to the bridge inside the model:

- The software automatically applies the self-weight of members based on the material unit weights; applied under the DEAD load case.
- A uniformly distributed load (UDL) of 23.0 kN/m was applied along the deck to account for the weight of the transverse ribs, which are 0.2 m thick and spaced at 2.5 m; applied under the DEAD load case.

- 1 kN/m per bridge railings and 0.1 kN/m for street lighting, for a total superimposed dead load of 4.2 kN/m applied to the deck as a UDL. Applied under the DEAD load case.
- 90 mm of asphalt for a total of 21.2 kN/m was applied to the deck as a UDL; applied under the DEAD load case.
- Thermal expansion of +18.0°C, +18.9°C, and +40.0°C applied to the pylons, deck, and cables, respectively, calculated as per §3.9 of CHBDC; applied under the T+ load case.
- Thermal contraction of -43.0°C, -44.3°C, and -63.0°C applied to the pylons, deck, and cables, respectively, calculated as per §3.9 of CHBDC; applied under the T- load case.
- Pedestrian live load pressure of 1.6 kPa was applied to the sidewalks for a total UDL of 8.6 kN/m applied to the deck, calculated as per §3.8.9 of CHBDC; applied under the PED ENV load case.
- CL-625-ONT truck and lane loads applied as a moving load as per §3.8 of CHBDC to all spans, the main span only, and the side spans only in different load combinations. A dynamic load allowance (DLA) of 1.25 is applied to the CL-625-ONT truck load. A lane reduction factor of 0.9 is applied to the second lane out of two lanes. Applied under the LL ENV load case.

It should be noted that the temperatures applied to the deck, pylons and cables are vastly different. The temperatures were applied as per the requirements of the CHBDC. In the CHBDC, the effective temperature change assigned to an element depends on the size of the element and the material of the element. Smaller elements tend to be assigned higher temperature variations, as do steel elements versus concrete elements. As such, the cables have significantly larger temperature variations than the deck and pylons. Additionally, the temperature changes under T+ and T- are applied assuming a baseline temperature of 15°C at construction, and thus the temperature decreases under T- are larger in magnitude than the temperature increases under T+. It should also be noted that no thermal gradients were considered in the application of the temperatures.

3.2.3. Load Cases and Load Combinations

The dead loads and cable tensioning loads are applied to the model using non-linear load cases with the P-delta effect incorporated. The cable tensioning load cases take into account the stiffness and displacements of the structure at the end of the dead load case or previous cable tensioning

load case. Any loads applied to the bridge after cable tensioning (temperature and live loads) are applied in linear load cases, taking into account the stiffness and displacements of the structure at the end of the cable tensioning. Because of the way the software works, the temperature and live loads are applied in linear load cases so that the results of these load cases can be extracted separately without the results from the cable tensioning being included. A sensitivity analysis was performed to determine whether the result is affected by using linear analysis instead of non-linear analysis for these loads, and it was found that the difference is negligible. The load cases used for each load combination, applicable load factors and the order of load application is summarized in Table 6.

Table 6 – Model Load Application

Order of Application	SLS-1	ULS-1	ULS-2
1	Dead loads, DEAD	Dead loads, DEAD	Dead loads, DEAD
2	TENSION CABLES (1) through (6) or (8) (6-8 iterations)	TENSION CABLES (1) through (6) or (8) (6-8 iterations)	TENSION CABLES (1) through (6) or (8) (6-8 iterations)
3	RELEASE PYLONS	RELEASE PYLONS	RELEASE PYLONS
4	TENSION BACKSTAYS ($\alpha_D = 1.00$)	TENSION BACKSTAYS ($\alpha_D = 1.10$)	TENSION BACKSTAYS ($\alpha_D = 1.10$)
5	Thermal effects: none, thermal expansion T+, and thermal contraction T- ($\alpha_K = 0.80$)	Pedestrian load, PED ENV: all spans, main span only, side spans only ($\alpha_{L,PED} = 1.36$)	Thermal effects: thermal expansion T+ and thermal contraction T- ($\alpha_K = 1.15$)
6	Live load, LL ENV: all spans, main span only, side spans only ($\alpha_L = 0.90$)	Live load, LL ENV: all spans, main span only, side spans only ($\alpha_L = 1.70$)	Pedestrian load, PED ENV: all spans, main span only, side spans only ($\alpha_{L,PED} = 1.28$)
7			Live load, LL ENV: all spans, main span only, side spans only ($\alpha_L = 1.60$)

First, the dead and superimposed dead loads are applied in the DEAD load case. At this stage, the tops of the pylons are restrained against displacement in the longitudinal direction in order to facilitate the process of tensioning the cables. Next, all cables with the exception of the backstays, are tensioned in a series of iterations in the TENSION CABLES (1) through (8) load cases, which are described in more detail in Section 3.2.4, with the pylon tops still restrained against displacement in the longitudinal direction. After all cables except for the backstays have been tensioned, the pylon restraints are removed in the RELEASE PYLONS load case, and the pylons

are free to deflect inward towards the center of the bridge. The backstays are then tensioned to bring the tops of the pylons back to zero longitudinal displacement in the TENSION BACKSTAYS load case. The TENSION BACKSTAYS load case includes the effects from the dead loads, cable tensioning, releasing the pylons, and tensioning the backstays. It reflects the final state of the bridge under dead loads; therefore, the dead load factor is applied to the TENSION BACKSTAYS load case instead of the DEAD load case. Following this procedure, all the other loads (thermal loads and live loads) are applied to the bridge as required by each load combination being examined. The vehicular live load is applied under the LL ENV load case in an envelope, considering the worst case of live load on side spans only, live load on main span only, and live load on all spans. The pedestrian live load is applied under the PED ENV load case in an envelope, taking into account the worst case of live load on side spans only, live load on main span only, and live load on all spans. In this thesis, the term “envelope” refers to the fact that the full range of minimum and maximum values (displacements, axial forces, shear forces, or bending moments) is captured for a given load or load combination.

The results presented in this thesis from applying the load cases described above were combined into load combinations within the software using the load factors listed in Table 6. The resulting load combinations are summarized in Table 7.

Table 7 – Load Combinations

SLS-1	No thermal effects (SLS-1)
	Thermal expansion (SLS-1-T+)
	Thermal contraction (SLS-1-T-)
ULS-1	No thermal effects (ULS-1)
ULS-2	Thermal expansion (ULS-2-T+)
	Thermal contraction (ULS-2-T-)

The MODAL load case is also run to obtain the first ten frequencies of each bridge, including the first bending frequency and first torsional frequency. The finite element models used to model the bridges are lumped mass models as the mass of each element is automatically concentrated at joint locations during analysis.

3.2.4. Cable Tensioning and Sizing

Preliminary cable sizes were hand calculated during the preliminary design stage described in Section 3.1.4. The initial model was constructed with only two cable sizes: one size was used for the backstays, and all the other cables were made to be the same size as the largest cable. Cable sizes were determined based on the available cable sizes supplied by BBR® [41]. Cable tensions were applied to this initial model through a series of iterations to minimize the bending moments in the deck, and the ULS-1 load combination was then applied to determine the required cable sizes. The cables were then resized based on the required cable sizes, and these cable sizes were used for the base model with steel cables (referred to as the “base model” henceforth). The cables were re-tensioned for the base model, the required cable sizes were confirmed, and then the base model was ready for use in the parametric study. The number of strands chosen for the cables is summarized in Table 8. The nomenclature for the cables is as follows: the cables in the side spans are labelled b, the cables in the main span are labelled f, and the cable number represents how far away the cable is from the pylon. This means that the cables in the main span closest to the pylons are labelled 1f, and the cables in the side spans furthest away from the pylons are labelled 12b. The cable nomenclature is illustrated in Figure 8.

Table 8 – Number of Strands per Cable

Cables	Number of 7-wire strands per cable
12b (backstays)	121
11b-9b	37
8b-5b	27
4b-1b	24
1f-4f	24
5f-8f	31
9f-12f	37

Tensioning the cables proved to be the most challenging task during the modelling process. CSiBridge has a feature called the Load Optimizer which calculates the cable tensions that are required to meet a set of goals (in this instance vertical deck displacements). At first the Load Optimizer was used to obtain ideal cable forces, but there were issues with convergence and issues

with compression in the cables. After this approach failed, it was decided to tension the cables using manual iterations, which are based on obtaining vertical displacements of the deck from the model, inputting those displacements into an equation, and obtaining the resulting strains which need to be applied to the cables in order to bring the deck displacements closer to the target displacements.

The target deck vertical displacements were calculated based on the displacement of the deck at the location of the pylon under dead and superimposed dead loads only. This displacement, and a zero displacement at the end of the side span, were used to determine $\Delta_{x,t}$, the target deck displacement at cable x , assuming a parabolic equation.

The top of the pylon was fixed in the longitudinal direction inside the model to allow the cables to be tensioned separately from the backstays. The first iteration involves reading the value of $\Delta_{x,1}$, the deck displacement at cable x , from the model, and determining the required cable strain, $\varepsilon_{x,1}$, which would decrease the length of the cable enough to bring the deck displacement to $\Delta_{x,t}$. This strain at cable x is determined with Equation (1), where L_x is the length of cable x and θ_x is the angle of cable x to the horizontal.

$$\varepsilon_{x,1} = \frac{\Delta_{x,1} - \Delta_{x,t}}{L_x \sin(\theta_x)} \quad (1)$$

After applying the first iteration of strains to the cables, it was found that the deck did not displace by exactly $\Delta_{x,1} - \Delta_{x,t}$ but rather by a value that is larger; in other words, the applied cable strains resulted in the deck displacements to overshoot. To correct for this a second iteration was applied in which the second applied strain, $\varepsilon_{x,2}$, was calculated based on the percentage by which $\varepsilon_{x,1}$ overshoot the target deck displacement $\varepsilon_{x,t}$. The equation for the second strain at cable x is Equation (2).

$$\varepsilon_{x,2} = \frac{\varepsilon_{x,1}}{\left| \frac{(\Delta_{x,2} - \Delta_{x,t}) - (\Delta_{x,1} - \Delta_{x,t})}{\Delta_{x,1} - \Delta_{x,t}} \right|} \quad (2)$$

The second iteration brought the deck displacements closer to the target values, but the target values were still not reached. Subsequent iterations were applied, and the equation for each iteration took into account the deck displacements which were obtained in all prior iterations such

that all the errors from prior iterations are taken into account. The equations for iterations 3, 4 and 5 are shown in Equations (3), (4), and (5).

$$\varepsilon_{x.3} = \frac{\varepsilon_{x,1}}{\left| \frac{(\Delta_{x,2} - \Delta_{x,t}) + (\Delta_{x,3} - \Delta_{x,t}) - (\Delta_{x,1} - \Delta_{x,t})}{\Delta_{x,1} - \Delta_{x,t}} \right|} \quad (3)$$

$$\varepsilon_{x.4} = \frac{\varepsilon_{x,1}}{\left| \frac{(\Delta_{x,2} - \Delta_{x,t}) + (\Delta_{x,3} - \Delta_{x,t}) + (\Delta_{x,4} - \Delta_{x,t}) - (\Delta_{x,1} - \Delta_{x,t})}{\Delta_{x,1} - \Delta_{x,t}} \right|} \quad (4)$$

$$\varepsilon_{x.5} = \frac{\varepsilon_{x,1}}{\left| \frac{(\Delta_{x,2} - \Delta_{x,t}) + (\Delta_{x,3} - \Delta_{x,t}) + (\Delta_{x,4} - \Delta_{x,t}) + (\Delta_{x,5} - \Delta_{x,t}) - (\Delta_{x,1} - \Delta_{x,t})}{\Delta_{x,1} - \Delta_{x,t}} \right|} \quad (5)$$

After approximately six iterations were applied, it became evident that the deck displacements, although they improved significantly, did not achieve the target deck displacement, as seen in Figure 14.

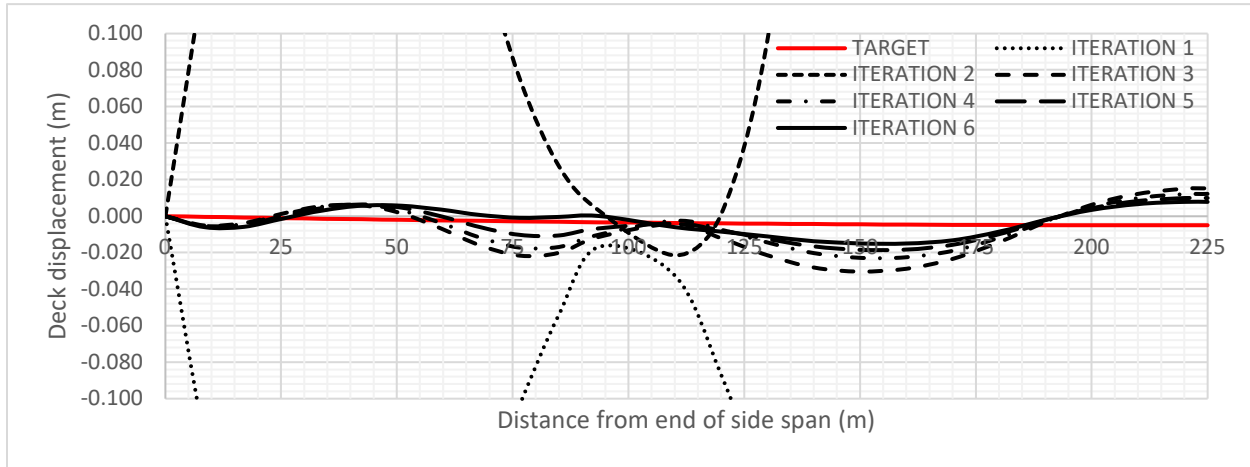


Figure 14 – Round 1 of Iterations for Cable Tensioning

It was decided to implement a second round of iterations consisting of applying secondary cable strains on top of the cable strains applied in the sixth iteration of the first round. The same equations were used for the iterations; five or six iterations were performed, and the best result was chosen to be the starting point for the next round. Round 3 of iterations was applied on top of the strains which were applied in Round 2, following the same procedure as that used in Rounds 1 and 2. With each round of iterations, the deck displacements got closer and closer to the target values. All in all, six to seven rounds of iterations were required before the deck displacements reached

values which were within two (2) mm of the target values. These rounds of iterations were applied in the load cases called TENSION CABLES (1) through (6), or TENSION CABLES (1) through (8) for some models inside the model.

After all, iterations were performed, the tops of the pylons were released in the next load case, and then the backstays were tensioned to bring the tops of the pylons back to zero displacement in the longitudinal direction. The bending moments in the deck were observed after the backstays were tensioned, and it was found that they were very close to being those of a beam on rigid supports, which was the ultimate goal of tensioning the cables. A snippet of the deck bending moments obtained after tensioning all the cables is shown in Figure 15.

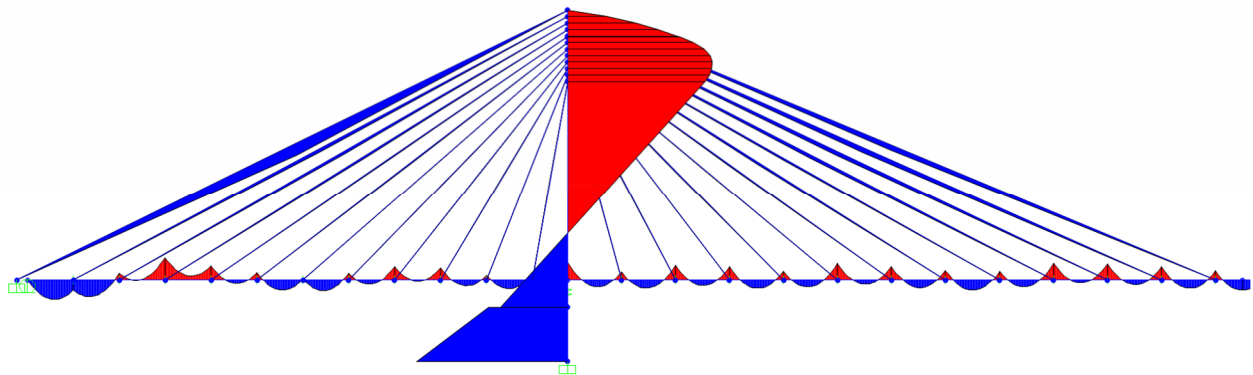


Figure 15 – Deck Bending Moments after Tensioning Cables

It is acknowledged that the results are not perfect and that some areas of the deck still do not exhibit beam-on-rigid-supports behaviour, particularly in the side spans. However, these results should be observed in contrast to the deck bending moments prior to tensioning the cables (Figure 16, bending moments shown using the same scale as in Figure 15). It is clear that the iterations accomplished the goal of minimizing the bending moments in the deck to a significant degree and bringing the vertical deck displacements to within 2 mm of the target values. The acquired cable tensions were compared to the cable forces calculated by hand under dead assuming beam-on-rigid-support behaviour, and it was found that most of the cable forces obtained from the model were within 10% of the values calculated by hand, confirming the validity of the cable tensioning procedure.

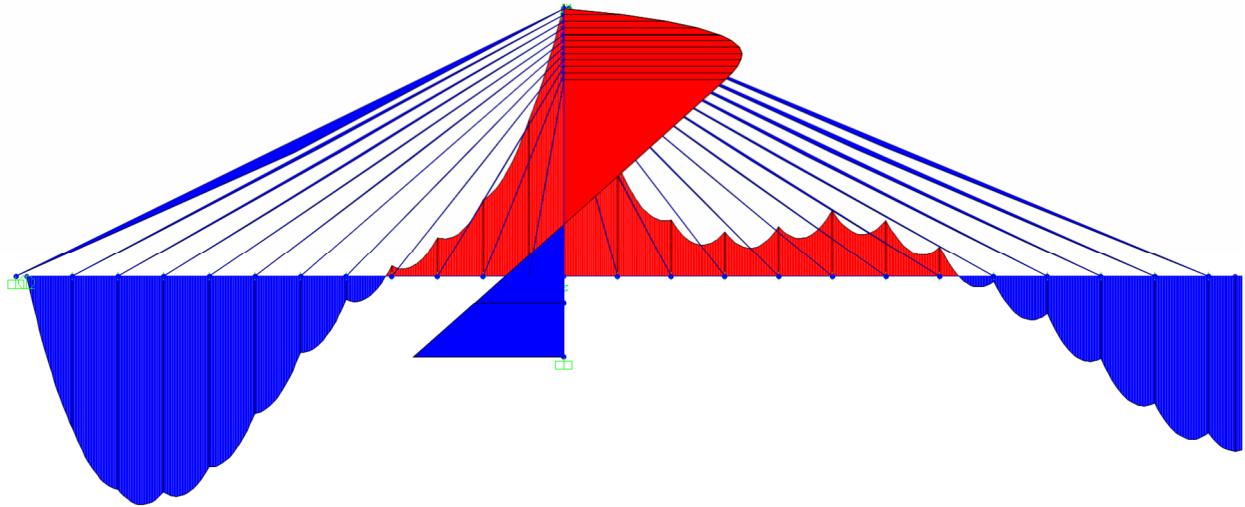


Figure 16 – Deck Bending Moments prior to Tensioning Cables

These iterations were performed for each new model described in Section 3.3 that is used to conduct the parametric study.

3.2.5. Cable Sag

A well-known phenomenon associated with long-span cable-supported bridges is cable sag. Cable sag occurs due to the self-weight of the cable. In an ideal world, the cables of a cable-stayed bridge would be straight; however, in reality, the cables assume a catenary shape due to their self weight, even if they are subjected to tension. The more tension is applied to a cable, the straighter it becomes, but it never reaches a fully straight condition; for that to happen, the tensile force would need to be infinitely large. Another factor which influences the effect of cable sag is the weight of the cable per unit length. A larger cable or a cable which is made of a denser material would experience more sag than a smaller cable or a cable which is made of a less dense material. [64] The focal point of this thesis is the comparison of the static behaviour of cable-stayed bridges with CFRP cables to the behaviour of bridges with steel cables. CFRP has a unit weight of approximately 19 kN/m^3 , whereas steel has a unit weight of 77 kN/m^3 . Thus it can be expected that the effect of cable sag in the bridges with CFRP cables will be significantly less than in the bridge with steel cables. It is important to consider this phenomenon when comparing the behaviour of the two types of bridges because cable sag reduces the effective stiffness of a cable, and cable stiffness is the focus of the parametric study.

The effective elastic modulus of a cable can be calculated using the Ernst equation (Equation (6)), which was developed by H.-J. Ernst [64]. The effective elastic modulus E_i , is a function of the elastic modulus of the cable material E_e , the density of the cable γ , the cable stress σ , and the horizontal cable span ($l = s \cos \alpha$ where s is the cable length and α is the angle between the cable and the horizontal).

$$E_i = \frac{E_e}{1 + (\gamma l)^2 E_e / 12 \sigma^3} \quad (6)$$

It should be noted that the effective modulus of a cable will change depending on how much load is applied to the cable, so the effective modulus calculated using Equation (6) is only applicable for a single value of cable tension. For instance, when a truck passes over the bridge, and the force in the cables increases or decreases; as a result, the effective modulus of the cables changes in that instant. To account for variations in cable forces, Ernst proposed a secant modulus which is calculated based on the lowest and highest cable stresses, σ_{min} and σ_{max} . The equivalent stress σ , which can be substituted into Equation (6) to determine this secant modulus, is calculated using Equation (7).

$$\sigma = \sigma_m \left(\frac{16\mu^2}{(1 + \mu)^4} \right)^{1/3} \quad (7)$$

where

$$\mu = \sigma_{min} / \sigma_{max}$$

and

$$\sigma_m = (\sigma_{min} + \sigma_{max}) / 2$$

Since only one frame element is used to model each cable, the models created in CSiBridge do not capture the effect of cable sag even though nonlinear analysis is used in the models. The secant effective modulus was calculated for each cable based on the maximum and minimum cable forces under SLS-1 obtained from the model to determine whether there is a need to consider the reduced effective modulus in the model. It was found that the reduction in effective modulus was less than 1% for all of the cables, even for the bridge with steel cables. This is likely due to the relatively short span of the bridge of 250 m. A brief sensitivity analysis was performed, and it demonstrated that increasing the span of the bridge results in a notable reduction in the effective modulus of the cables. As the reduction in effective modulus was negligible, it was concluded that cable sag does not need to be considered in the models used for the parametric study. However, the cable sag

would be more significant if the bridges examined in the parametric study had significantly longer spans.

3.3. Parametric Study Methodology

Two variables will be examined in the parametric study which serves as the basis for this thesis. The first variable (**Phase I**) is the performance of the bridge with CFRP cables which are composed of CFRP material with various elastic moduli. The selected stiffnesses of the CFRP will be: 150 GPa (normal elastic modulus of CFRP cables), 190 GPa (same elastic modulus as that of the steel cables in the base model), 300 GPa, and 400 GPa. These values for the elastic modulus of the CFRP cables were chosen based on the range of stiffnesses of HM CFRP materials which are currently available on the market, as described in Section 2.9 of this thesis. To conduct **Phase I** of the parametric study the steel cables of the base model will be replaced with CFRP cables in a new model, and individual models will be created for each of the CFRP stiffnesses listed above. It is assumed that, like the steel cables, the CFRP cables are also comprised of 7-wire strands with an area of 150 mm² per strand. The cable sizes will be kept the same for all bridges studied in **Phase I** of the study to avoid varying too many parameters at the same time. The cable properties for the different models are summarized in Table 9. The models are named based on their material and elastic modulus. STEEL-190 is the model with steel cables with an elastic modulus of 190 GPa, CFRP-150 is the model with CFRP cables with an elastic modulus of 150 GPa, and so on.

Table 9 – Parametric Study **Phase I**: CFRP Modulus

Parameter	STEEL-190	CFRP-150	CFRP-190	CFRP-300	CFRP-400
γ (kN/m ³):	77.0	19.0	19.0	19.0	19.0
E (GPa):	190	150	190	300	400
f_y/f_{br} (MPa):	1860	2550	2550	2550	2550
α (°C ⁻¹):	12×10^{-6}	0.6×10^{-6}	0.6×10^{-6}	0.6×10^{-6}	0.6×10^{-6}

The chosen values for the CFRP elastic moduli are fictional as there are currently no high-modulus CFRP cables available, although as discussed in Section 2.8, there are many high-modulus carbon fibers available on the market which could conceivably be used to produce high-modulus CFRP cables. The presumption is that higher modulus CFRP cables will eventually become a reality, and the aim of this parametric study is to determine how the behaviour of a cable-stayed bridge would change if higher modulus CFRP material were to be used for the cables. The unit weight of 19.0 kN/m³, the guaranteed breaking strength (f_{br}) of 2550 MPa, and the coefficient of thermal

expansion of 0.6×10^{-6} were selected based on the parameters of the 7-strand tendons offered by Tokyo Rope [19].

The second variable in the parametric study is the cable area. For this portion of the parametric study (**Phase II**) CFRP cables with a stiffness of 190 GPa will be selected to match the elastic modulus of the steel cables. The cable area will be varied, ranging from equivalent stiffness (same cable area as the steel cables) to comparable strength (same factored tensile resistance as the steel cables). During the literature review, it was found that when scholars studied the behaviour of cable-stayed bridges with CFRP cables, they implemented several different methods to size the CFRP cables; some used the equivalent strength principle [9, 24], and others used the equivalent area principle [12].

The purpose of **Phase II** is to examine the effect of cable sizing methodology (equivalent strength, stiffness, or area method) on the behaviour of a cable-stayed bridge while keeping the elastic modulus of the CFRP cables equal to that of the steel cables. The proposed cable sizes are summarized in Table 10, with the numbers representing the number of 7-wire strands that comprise each cable. It is assumed that both the steel and CFRP cables will be comprised of a number of 7-wire strands with a cross-sectional area of 150 mm^2 per strand. BBR offers cables comprised of these exact strands [41], while Tokyo Rope offers 7-wire tendons of various sizes, one of the sizes being 151.1 mm^2 , which is very close to 150 mm^2 [19]. The models are named based on the cable material and percentage of the original cable area. CFRP-100% has cables that are sized based on the equivalent strength principle and use the same number of strands per cable as STEEL-100%. CFRP-70% has cables that are approximately sized based on the equivalent strength principle, assuming an ultimate breaking strength of 2550 MPa for the CFRP material. It should be noted that in most cases, the closest cable sizes are chosen based on the cable sizes supplied by BBR®, so the sizing of the cables based on the equivalent strength principle may not be exact.

Table 10 – Parametric Study **Phase II**: Cable Area

Cables	Number of 7-wire strands (strand area = 150 mm ²)			
	STEEL-100%	CFRP-100%	CFRP-80%	CFRP-70%
12b (backstays)	121	121	97	85
11b-9b	37	37	31	27
8b-5b	27	27	22	19
4b-1b	24	24	19	17*
1f-4f	24	24	19	17*
5f-8f	31	31	24	22
9f-12f	37	37	31	27

*This cable size is not available from BBR; however, the next closest size would be the same as for CFRP-80%, so it is necessary to use a non-existent size for the purpose of this study

For both **Phase I** and **Phase II** of the parametric study, the geometry of the bridge and the material properties and member sizes of the pylons and deck will be kept the same throughout the study. Cable tensioning will be performed separately for each model to minimize the bending moments in the deck .

Chapter 4. Results and Discussion

In this chapter, the results of the two phases of the parametric study are presented and discussed. **Phase I** involves varying the elastic modulus of the cables while keeping the cable area constant. The bridge models being compared in **Phase I** are named STEEL-190, CFRP-150, CFRP-190, CFRP-300, and CFRP-400, where the first portion of the name represents the cable material (steel or CFRP), and the second portion of the name represents the elastic modulus of the cables in GPa. **Phase II** involves varying the cable area while keeping the elastic modulus of the cable material constant at 190 GPa. The bridge models being compared in **Phase II** are named STEEL-100% (same as STEEL-190), CFRP-100% (same as CFRP-190), CFRP-80%, and CFRP-70%, where the second portion of the name represents the percentage of the initially designed cable area being used for the cables.

In both phases, the static and dynamic behaviour of the different cable-stayed bridges examined in each phase is compared. A finite element model was created for each bridge, and all loads were applied inside the models as described in Chapter 3. The load combinations were created directly inside the software, and results were extracted from the model for each load case and load combination to compare the results for the different bridges. TENSION BACKSTAYS represents the state of the bridge under its own weight and after all of the cables have been tensioned to minimize deck bending moments. LL ENV is the live load envelope. T+ and T- represent thermal expansion and contraction, respectively. ULS-1, ULS-2-T+, ULS-2-T-, and SLS-1 represent the load combinations which consist of the loads described above. For a detailed description of the applied loads and load combinations, refer to Sections 3.2.2 and 3.2.3.

Results were collected for the following parameters for each bridge:

- Cable axial forces under the following separately applied loads: TENSION BACKSTAYS, LL ENV (min/max), T+, T-, ULS-1 (min/max), ULS-2-T+ (min/max), and ULS-2-T- (min/max);
- Vertical deck displacements under the loads: LL ENV (min/max), T+, T-, and SLS-1 (min/max);
- Longitudinal pylon tip displacements (horizontal pylon tip displacements in the direction of traffic movement) under the loads: LL ENV (min/max), T+, T-, and SLS-1 (min/max);

- Deck axial load, shear, and bending moment under the loads: LL ENV (min/max), T+, T-, ULS-1 (min/max), ULS-2-T+ (min/max), and ULS-2-T- (min/max);
- Pylon axial load, shear, and bending moment under the loads LL ENV (min/max), T+, T-, ULS-1 (min/max), ULS-2-T+ (min/max), and ULS-2-T- (min/max); and
- 1st bending and 1st torsional frequencies of the bridge.

These results were used to develop figures and tables which serve as the focus of the discussion in this chapter. Each figure compares the results for all the bridges considered in each phase. As there are 94 figures, they are all included in appendices, and the reader is invited to read this chapter in conjunction with the appendices. **Phase I** figures are included in Appendix A (labelled “Figure A-X”), and **Phase II** figures are included in Appendix B (labelled “Figure B-X”). The full list of figures can be found in the List of Appendix Figures at the beginning of the thesis on page x.

The bridge’s main span of 250 m exceeds the maximum span for which the use of CHBDC is suitable. Given the absence of a suitable Canadian standard for long-span bridges, to simplify the analysis, all loads which were applied to the bridge during the analysis are symmetrical about the midspan and about the centerline of the deck. Results are collected for only half of the bridge as both the bridge geometry and loading are symmetrical about the midspan. Results for the cables are only collected for one cable plane, and results for the pylons are only collected for one leg of one pylon since the bridge geometry and loading are symmetrical about the centroid of the deck cross-section as well. It is acknowledged that for the detailed design of a cable-stayed bridge, asymmetrical loading would need to be considered as well.

4.1. Phase I Results

4.1.1. Cables Forces

Cable forces were extracted from the models and plotted in bar charts to determine the effect of the cable material and elastic modulus on the forces that develop within the cables under various loads. Cable forces were extracted for the ULS load combinations ULS-1, ULS-2-T+ and ULS-2-T- to determine the effect of the cable material on the overall maximum cable loads. Cable forces were also extracted for the unfactored individual loads (DEAD, LL, T+ and T-), which the aforementioned ULS load combinations comprise, to examine the effect of each individual load

on the forces in the cables and determine how these individual loads affect the total cable forces in the ULS load combinations.

4.1.1.1. Dead Load and Cable Tensioning

The cable forces were compared under dead loads and cable tensioning loads only in Figure A-1. This figure shows the cable forces under dead/cable tensioning loads for each cable for each bridge. At first glance, there does not appear to be any definitive trend. The cable forces are similar for all five (5) bridges, with a few exceptions. Errors in the cable tightening iterations likely cause the slight variations in cable forces for most of the cables. The cable tensions under dead loads were determined through numerous iterations completed by hand rather than through the load optimizer function of the finite element software, and as such, there is a high possibility that the cable tensions for each bridge will be slightly different. For most of the cables, the differences in cable tensions are less than 10%, and there is no clear relationship between the stiffness of the cable material and the cable forces. The only visible pattern is that the tensile forces in approximately half of the cables of the STEEL-190 bridge are somewhat higher (approximately by 5%) than the cable tensions in the CFRP bridges. This is likely caused by the fact that the steel cables are considerably heavier than the CFRP cables, and the added weight of the steel cables would theoretically increase the maximum tension in the cables of the STEEL-190 bridge.

Cables 11b, 2b and 1b show a significant variance in cable forces between the five (5) bridges. Errors in the hand iterations can likely also explain the large variances in the cable tensions of these cables. It should be noted that cable 11b is very close to the end of the bridge, while cables 2b and 1b are very close to the pylons. Cables which are close to the pylons or the ends of the bridge are much more susceptible to variable results during the cable tightening iterations since the magnitude of the tensions in these cables does not have as much impact on the behaviour of the bridge deck. The proximity of these cables to the pylons or ends of the bridge means that the forces in these cables produce a relatively small lever arm, and thus, pulling on these cables has a lesser effect on the displacements of the pylons or deck. Therefore, it is possible for the final displacements in the pylons and deck to be very similar for the 5 bridges even with such significant variations in the cable forces of cables 11b, 2b and 1b.

4.1.1.2. Live Load Envelope

In Figure A-2, the maximum increase and decrease in cable forces under live load were compared for the 5 bridge models. The forces shown in these figures include the effects of live load only, and do not include the effects of the dead loads and cable tensioning. Live load primarily causes tension in the cables, but some reduction in tensile force can also be created in the cables depending on the positioning of the live load. There is a clear evident pattern showing the cable tension due to live loads is directly related to the cables' stiffness. The cable tensions in the CFRP-150 bridge are 5-10% lower than those in the STEEL-190 bridge because the CFRP-150 cables are less stiff than the STEEL-190 cables and thus attract less live load. The remainder of the live load goes into the deck via increased deck bending moments. The cable tensions in the CFRP-190 cables are virtually identical to the tensions in the STEEL-190 cables because the stiffness of the cables in these two bridges are the same. The cable tensions in the CFRP-300 and CFRP-400 cables are higher than in the other bridges, with the CFRP-400 cable tensions being generally 15-25% higher than the STEEL-190 cable tensions. This is because these cables are stiffer and attract more live load as a result. The backstays exhibit similar behaviour, with the backstay force in the CFRP-400 bridge being approximately 30% higher than the backstay force in the STEEL-190 bridge. The general observation is that the cable material alone (CFRP vs steel) does not affect the magnitude of the cable force under live loads. Still, the stiffness of the cable material affects the amount of live load that the cables take, with the stiffer cable taking more load.

4.1.1.3. Temperature Loads

The results for the change in cable forces under T+ and T- are presented in Figures A-3 and A-4, respectively. T+ involves a thermal expansion of +18.0°C, +18.9°C, and +40.0°C applied to the pylons, deck, and cables, respectively, while T- involves a thermal contraction of -43.0°C, -44.3°C, and -63.0°C applied to the pylons, deck, and cables respectively. A higher temperature increase/decrease is applied to the cables due to the requirements of CHBDC. The effects of T- are opposite to and larger than the effects of T+ because the magnitude of T- is larger than the magnitude of T+. Refer to Section 3.2.2 for a discussion of why these temperatures were applied.

Immediately from looking at these figures, it is clear that both the cable material and the cable stiffness significantly impact the cable force under temperature loads. There are two general trends

that can be observed in the cables' performance. The first trend is that among the CFRP bridges, the bridges with stiffer CFRP materials attract more loads into the cables under thermal loading. This is expected because stiffer cables will likely attract more loads. The second trend is that T+ tends to create higher tensile forces in CFRP cables than in steel cables, while T- tends to create lower tensile loads in CFRP cables than in steel cables. This effect is not present in all cables, but it occurs in the backstays, cables 11b-8b, cables 3b-3f, and cables 10f-12f. The forces due to thermal loads are relatively small in the rest of the cables. It is difficult to offer a definitive explanation for these observations, but they may be occurring for the following reasons:

- The coefficients of thermal expansion of concrete and steel are relatively similar, which means that under the same temperature change, concrete and steel will expand or contract to approximately the same degree. Since the temperature change prescribed to the cables is larger than the temperature changes prescribed to the pylon and deck, the cables of STEEL-190 will expand and contract to a considerably larger degree than the pylons and deck. This means that the steel cables are expected to develop some reduction in tensile force under T+ due to expanding to a larger extent than the concrete deck and pylons, and likewise, they are expected to develop some additional tension under T- due to undergoing more shortening than the concrete deck and pylons.
- The coefficient of thermal expansion of CFRP cables is approximately 20 times lower than that of steel cables. As a result, the CFRP cables are not expected to undergo any notable lengthening or shortening under thermal loads. Therefore, when the pylons and deck expand under T+, the CFRP cables would be placed under tension since they do not expand by any significant amount under T+, and the opposite effect would occur under T-.

There are few papers which discuss the thermal performance of cable-stayed bridges with CFRP cables, and the few studies which do discuss this do not provide an in-depth analysis of the possible explanations for the behavior of these bridges under thermal loads. The study conducted by Kuihua et al. [12] showed that the axial forces in the cables at the pylons when subjected to a temperature increase. The cable forces under T+ were reported to be positive for CFRP cables and negative for steel cables, which is consistent with the findings of this parametric study.

It is unclear why cables 7b through 4b and cables 4f through 9f exhibit the opposite trend. The steel cables exhibit more tensile force than the CFRP cables in these cables. Theoretically, this

should not happen because the steel cables are allowed to expand under T+ while the CFRP cables are not, and thus the upward expansion of the pylons should place all of the CFRP cables under tension. An interesting observation, however, is that these anomalies appear to occur in the locations where the curvature of the deck changes (i.e., around inflection points in the vertical deck deflection). Figure A-9 shows the deflected shape of the deck in the vertical direction under T+ loads. For the CFRP bridges, the deck exhibits negative curvature in the side span, positive curvature near the pylon, and again negative curvature at midspan. The locations where the direction of the curvature of the deck changes approximately correspond to the locations where the cables exhibit the unexpected behavior. The indeterminate nature of the bridge structure makes it difficult to offer definitive explanations for the observed effects, but the correlation between the cable forces and the curvature of the deck under T+ and T- may offer some insight.

In all cases, the backstays are subjected to a reduction in tensile force under T+ and increase in tensile force under T-. Under T+ the backstays of STEEL-190 undergo a considerable amount of thermal expansion, resulting in a reduction in tensile force, while the backstays of the CFRP bridges do not experience this effect. However, it is the case for all five bridge models that the deck within the main span expands, pushing the pylons outward towards the ends of the bridge, and this creates a reduction in tensile force within the backstays. These effects are reversed under T-. It is likely that these relatively large forces in the backstays due to thermal effects are driven by the fact that the connection between the deck and the pylons is pinned, meaning that the deck and the pylons must expand and contract in unison in the longitudinal direction. If the deck were to be on roller supports at the pylons or not have any supports at the pylons, it is expected that the axial forces in the backstays due to thermal effects would be considerably smaller.

4.1.1.4. ULS Load Combinations

The cable force envelopes under the ULS-1, ULS-2-T+, and ULS-2-T- load combinations are shown in Figures A-5 through A-7, respectively. In all 3 load combinations, the differences in the total cable forces are less than 10% for most cables. The largest component of the cable forces is attributed to the dead load, with the remainder of the loads making up less than 20% of the total cable forces. The dead load results in very similar cable forces when compared for the 5 bridges. The live and temperature loads exhibit a noticeable variation in cable forces between the 5 bridges, but they do not constitute a significant portion of the total cable loads. Therefore, the total cable

forces under ULS load combinations are very similar for the 5 bridges. Cables 11b, 2b and 1b exhibit large variations in cable forces for the reasons described in Section 4.1.1.1, but the force variations in these cables are not indicative of any relationship between the cable material or stiffness and the total cable force.

4.1.2. Deck and Pylon Tip Displacements

4.1.2.1. Vertical Deck Displacements

Figure A-8 shows the minimum and maximum vertical deck displacements for the 5 bridges under the live load envelope. Positive values indicate that the deck is deflecting upward, while negative values indicate that the deck is deflecting downward. There is a clear relationship between the stiffness of the cable material and the vertical deck displacement. The deck displacements of the CFRP-150 bridge are the largest because the CFRP-150 cables have the smallest elastic modulus. This is consistent with the findings of [8]. The deck displacements of the STEEL-190 and CFRP-190 bridges are identical since these bridges have cables with the same axial stiffness. The deck displacements of the CFRP-300 bridge are smaller than those of the CFRP-190 bridge, and the deck displacements of the CFRP-400 bridge are the smallest. The elastic modulus of the cables has a notable impact on the vertical deck displacements under live loads, but the cable material (steel versus CFRP) does not make a difference.

Figures A-9 and A-10 show the vertical deck displacements under unfactored T+ and T- loads respectively. Both figures show that the deck displacements of the CFRP bridges due to thermal loads are very similar, while the deck displacements of the STEEL-190 bridge are significantly larger under thermal loads. Under T+ the STEEL-190 deck undergoes a large downward displacement at midspan, which is likely attributed to the fact that the temperature increase in the cables causes them to expand, reducing the tension in the cables and thus creating extra downward displacement in the deck. The side spans of the STEEL-190 bridge experience an upward displacement, likely due to the significant downward displacement of the main span. Both the main and side spans of the CFRP bridges experience an upward displacement because the cables of the CFRP bridges do not undergo any significant lengthening under thermal expansion. As the pylon expands, the length of the CFRP cables remains relatively constant, and thus, the upward expansion of the pylon pulls the deck upwards. The downward displacement of the midspan of

STEEL-190 bridge is larger in magnitude than the upward displacement of the midspan of the CFRP bridges when subjected to T+, which is consistent with the findings of [12]. There is a small difference between the deck displacements of the various CFRP bridges. It appears that the CFRP-400 bridge has the largest upward displacement at the side spans, whereas the CFRP-150 bridge has the lowest upward displacement at the side spans. This is likely due to the fact that the CFRP-400 bridge develops considerably larger cable tension under T+ than the CFRP-150 bridge. The same effects can be observed in the opposite direction for the deck displacements under T-.

When considering the above loads in combination, the result is that the STEEL-190 bridge experiences the largest vertical deck displacements in the main span and the smallest displacements in the side spans under the SLS-1 envelope, as seen in Figure A-11. Although the STEEL-190 bridge does not see the largest deck displacements under live loads, it does see significantly larger deck displacements under thermal loads than the CFRP bridges, which is likely why it has the largest main span displacements. Out of the CFRP bridges, the CFRP-150 bridge has the largest deck displacements, while the CFRP-400 bridge has the smallest deck displacements. This is attributed to the displacements under live loads. The displacements under thermal loads are relatively similar for all the CFRP bridges.

4.1.2.2. Longitudinal Pylon Tip Displacements

Table 12 shows the minimum and maximum longitudinal pylon tip displacements for the 5 bridges under various loads. Positive values indicate that the pylon tips are displacing towards the midspan, while negative values indicate that the pylon tips are displacement away from the midspan and towards the ends of the bridge.

Table 11 – **Phase I** Longitudinal Pylon Tip Displacements

	LIVE LOAD ENV		T+	T-	SLS-1 ENV	
	MIN	MAX			MIN	MAX
STEEL-190	-21	37	23	-24	-38	52
CFRP-150	-23	42	-15	36	-32	67
CFRP-190	-21	37	-14	35	-30	61
CFRP-300	-16	28	-13	33	-25	52
CFRP-400	-14	23	-13	31	-23	46

The pylon tip displacements under the live load envelope follow the same trend as the deck displacements under the live load envelope. The pylon displacements of the STEEL-190 and CFRP-190 bridges are identical due to the cable stiffnesses being the same for those two bridges. The pylon displacements of the CFRP-150 bridge are the highest due to it having the least stiff cables. The displacements of the CFRP-400 bridge are the lowest due to this bridge having the stiffest cables out of all the bridges.

Under T+ the STEEL-190 pylons have a positive displacement (towards midspan), while the CFRP bridge pylons have negative displacements (away from midspan). The positive pylon displacement of the STEEL-190 bridge is caused by the loosening of the backstays under T+. The loosening of the backstays is considerably smaller for the CFRP bridges, and thus, the backstays of the CFRP bridges provide more restraint against the longitudinal displacement of the pylon tips towards midspan. For the CFRP bridges the pylon tip displacement is likely negative because the pylons are pushed outward by the expansion of the deck. This phenomenon is likely also present for the STEEL-190 bridge, but it is overcome by the loosening of the backstays. Amongst the CFRP bridges, the magnitude of the pylon tip displacements is the largest for the CFRP-150 bridge and smallest for the CFRP-400 bridge because the less stiff cables of the CFRP-150 bridge allow for larger movement of the pylons, while the stiffer cables of the CFRP-400 bridge act to restrain the movement of the pylons. All of these effects are similar but opposite under T- loading.

The result is that under the SLS-1 envelope, the pylons of the STEEL-190 bridge experience the largest overall negative longitudinal displacement, while the pylons of the CFRP-150 bridge experience the largest overall positive longitudinal displacement. The pylons of the CFRP-400 bridge undergo the least amount of movement in both directions.

4.1.3. Deck Forces

4.1.3.1. Live Load Envelope

Figures A-12 through A-14 show the minimum and maximum axial load, shear, and bending moment in the deck under the live load envelope.

The axial loads under live loads are the lowest for the CFRP-150 bridge and the highest for the CFRP-400 bridge. The axial loads for the STEEL-190 and CFRP-190 bridges are the same. These

results can be directly explained by the fact that the cable forces under live loads are the lowest for the CFRP-150 bridge, the highest for the CFRP-400 bridge, and identical for the STEEL-190 and CFRP-190 bridges. The higher the cable forces, the higher the deck axial load, because the horizontal component of the cable forces directly causes the axial load in the deck. These results are consistent with [8] and [12].

Both the shear and the bending moment in the deck are the highest for the CFRP-150 bridge (3% and 14% higher than STEEL-190 for shear and bending moment at the pylon, respectively) and the lowest for the CFRP-400 bridge (11% and 35% lower than STEEL-190 for shear and bending moment at the pylon, respectively), with the shear and bending moment being identical for the STEEL-190 and CFRP-190 bridges. This is consistent with the cable forces in the CFRP-150 bridge being the lowest due to the lower stiffness of those cables, resulting in more load being carried through the shear and bending moment in the deck. The shear and bending moment are the lowest for the CFRP-400 bridge because this bridge has the stiffest cables, which pick up additional load, thereby reducing the shear and bending moment in the deck. The stiffer cables also deflect to a smaller degree under live loads, resulting in lower vertical deck displacements which in turn results in lower shear and bending moment in the deck. These results are consistent with the findings of [12].

4.1.3.2. Temperature Loads

Figures A-15 to A-17 show the axial load, shear, and bending moment in the deck under T+. Figures A-18 to A-20 show the axial load, shear, and bending moment in the deck under T-.

Looking at the axial force in the deck under T+, on the side spans, there is a clear trend which shows that the axial force in the deck is lowest for the steel cables. The likely explanation is that the steel cables can also expand under T+ and thus, do not restrain the expansion of the side spans as much as the CFRP cables do. The bridges with the CFRP cables exhibit higher axial forces in the side spans because the CFRP cables do not expand much under T+, thus, they restrain the expansion of the side spans. CFRP-150 exhibits the lowest axial force in the side span of the bridges with CFRP cables. In contrast, CFRP-400 exhibits the highest axial force in the side span, and this is because the higher the stiffness of the cables, the more they restrain the expansion of the side spans, which increases the compression in the side spans. There is a significant jump in

the axial force in the main span due to the restraining action of the pylons. Since the connection between the pylons and the deck restrains the relative longitudinal movement between the deck and the pylons, the longitudinal expansion and contraction of the main span under temperature loads is restrained by the pylons. If the connection was a roller or if there was no connection between the deck and the pylons, this effect would not be present. There is a similar pattern of axial forces when looking at the CFRP bridges. The pylon tips expand vertically under T+ by approximately 12 mm for all of the bridges, but the cables of the CFRP bridges do not expand by much under T+. The upward expansion of the pylons pulls on the CFRP cables, and as a result, they place the deck under additional compression. CFRP-150 exhibits the lowest axial force in the main span since it has the least stiff cables, while CFRP-400 exhibits the highest axial force in the main span since it has the stiffest cables which are inclined to develop more force. The reason for the axial force in the main span of the STEEL-190 bridge not being the lowest is unclear. One would expect it to be lower than in the CFRP bridges because the cables within the main span of the STEEL-190 bridge are allowed to expand due to their higher coefficient of thermal expansion, and thus, they should not induce compression within the deck.

The bending moments in the deck under T+ show a close correlation to the deflected shape of the deck under T+. The STEEL-190 bridge exhibits a large amount of positive bending moment at the midspan and a large negative bending moment at the pylon. The positive midspan moment can be attributed to the large downward displacement of the deck under T+ due to the reduction in tension in the cables resulting from the thermal expansion of the cables. The bending moment is negative in the side spans of the STEEL-190 bridge because the deck deflects upward within the side span. There is a sharp uptick in negative moments at the pylon because, at the pylon, there is a point of inflection in the deflected shape of the deck. The bending moments in the CFRP bridges are negative in the main span and side spans because the deck deflects upward at both spans for the reasons explained in Section 4.1.2.1. The positive bending moment at the pylon is due to the positive curvature of the deck pylon, which is caused by the upward displacement of the main span. The positive bending moments of the CFRP bridges at the pylon are smaller in magnitude than the negative bending moment of the STEEL-190 bridge at the pylon. These results are consistent with the findings of [12]. Out of the CFRP bridges, CFRP-150 exhibits the lowest bending moments, while CFRP-400 exhibits the highest bending moments. This is due to the deck

of the CFRP-150 bridge having the least amount deflection, and the deck of the CFRP-400 bridge having the most amount of deflection.

The shear in the deck follows a similar pattern to the bending moment since shear is closely related to the bending moment, being the derivative of the bending moment. All of these effects are similar and in the opposite direction under T-. However, a key observation is that the magnitude of the negative bending moments of the CFRP bridges at the pylons is actually larger than the magnitude of the positive bending moment of the STEEL-190 bridge at the pylon. In the literature review [12] noted that under a positive temperature increase (T+) the magnitude of the positive bending moment at the pylons in CFRP bridges is smaller than the magnitude of the negative bending moment at the pylons in a bridge with steel cables. On the basis of this the study concluded that using CFRP cables is beneficial for decreasing the bending moments in the deck. However, this study did not discuss what would happen if a temperature decrease were to be applied to the bridge. T- appears to result in higher bending moments in the CFRP bridges than in the steel bridge overall. Furthermore, using HM CFRP cables exacerbates this effect.

4.1.3.3. ULS Load Combinations

Figures A-21 through A-29 show the axial force, shear, and bending moment envelopes in the deck under the ULS-1, ULS-2-T+, and ULS-2-T- load combinations.

For ULS-1, the axial load envelope shows a negligible difference in axial force between the five bridges. The maximum axial force does appear to be slightly lower for the CFRP-150 bridge and slightly higher for the CFRP-400 bridge due to the live load contribution, but otherwise, the results are very similar. The shear under ULS-1 is virtually the same for all 5 bridges with the exception of a few locations where the CFRP-150 bridge exhibits slightly higher shear due to the live load contribution. The bending moments under ULS-1 exhibit the largest variation, with CFRP-150 moments being the highest and CFRP-400 moments being the lowest due to live loads. The stiffer cables of the CFRP-400 bridge allow for less deck displacement under live loads, thus reducing the bending moments in the deck, while the less stiff cables of the CFRP-150 bridge result in the opposite effect. The midspan bending moment in the CFRP-150 deck is 10% higher than the midspan bending moment in STEEL-190, and the bending moment in CFRP-400 is 22% lower

than the bending moment in STEEL-190. This means that using CFRP cables with a higher stiffness positively affects the reduction of deck forces under ULS-1.

Under ULS-2-T+ the axial forces tend to be highest for CFRP-400 in all spans, lowest for STEEL-190 in the side spans, and lowest for CFRP-150 in the main span. This is consistent with the deck axial load being the lowest for STEEL-190 in the side spans and lowest for CFRP-150 in the side spans under T+. Additionally, since the deck axial load is the highest for CFRP-400 under both T+ and the live load envelope, CFRP-400 exhibits the highest overall axial load under ULS-2-T+. While the live load envelope and temperature loads cause these minor differences in the deck axial load under ULS-2-T+, the difference in the axial load between all five bridges is less than 10%. The shear in the deck is largely the same for all five bridges, with the only discrepancies in shear manifesting at the ends of the bridge and at the pylon. At the pylons, the shear is approximately 35% higher for the STEEL-190 bridge compared to the other bridges, and at the ends of the bridge, the positive shear is highest for the CFRP-400 bridge due to the temperature loads. The bending moments in the deck show the contribution of the live load envelope and the temperature loads. Overall, the bending moments in the main span tend to be the highest for CFRP-150 and the lowest for CFRP-400 due to the live loads. STEEL-190 exhibits a spike in negative moment at the pylon and an increase in positive bending moment at midspan due to the contribution of T+. In the side spans, CFRP-400 exhibits the lowest positive bending moments but the highest negative bending moments due to temperature effects. Meanwhile, CFRP-190 has the highest positive bending moments in the side spans, while STEEL-190 has the lowest negative bending moments in the side spans. Overall, the use of steel cables results in the highest bending moments in the main span and at the pylons, while the use of HM CFRP cables results in higher negative bending moments in the side spans due to temperature effects.

The axial loads and shear in the deck are relatively similar for the 5 bridges under ULS-2-T- loads. The bending moments in the deck are worth discussion as there is significant variation in the bending moments between the 5 bridges. In the ULS-2-T+ load combination there is a spike in negative bending moment at the pylons in STEEL-190. In the ULS-2-T- load combination there is a spike in negative bending moment at the pylons in the CFRP bridges, while the bending moments in STEEL-190 bridge are relatively uniform at the pylon. This is due to the reversal of the loads under T-. Under T+ the highest bending moments are in the STEEL-190 bridge, while under T-,

the highest bending moments are in the CFRP bridges. Out of the CFRP bridges, the pattern of CFRP-150, having the highest bending moments and CFRP-400, having the lowest bending moments, remains, although increasing the elastic modulus of the CFRP material appears to result in higher positive bending moments in the side spans due to the temperature effects.

When comparing the bending moments in the deck across all three ULS load combinations, the highest negative bending moment is present at the pylons in CFRP-150 under ULS-2-T-, while the highest positive bending moment is present in the side span of CFRP-300 under ULS-2-T-. Overall, the use of CFRP cables results in slightly higher bending moments in the deck. Stiffer CFRP cables decrease the bending moments due to live loads but increase the bending moments due to temperature loads.

4.1.4. Pylon Forces

4.1.4.1. Live Load Envelope

Figures A-30 through A-32 show the minimum and maximum axial load, shear, and bending moment in a single leg of one pylon under the live load envelope. The effects of live loads on the pylon are very similar to those on the deck. Similar to the deck, the pylons are subject to the smallest axial load for CFRP-150 and the largest axial load for CFRP-400, with the axial loads being the same for STEEL-190 and CFRP-190. This is because the forces in the cables under live loads increase when the stiffness of the cables increases. Increased cable forces result in larger axial loads in both the deck and the pylons.

The shear and bending moments in the pylons are the lowest for CFRP-150 and highest for CFRP-400 in the region where the cables are anchored to the pylons. CFRP-400 exhibits the highest cable forces due to live load, while CFRP-150 exhibits the lowest cable forces due to live loads. Higher cable forces result in higher shear and, subsequently, higher bending moments in the pylons, which is why the shear and bending moments are the highest for CFRP-400 at the tops of the pylons. Below the cable anchorages, however, the reverse effect occurs. The shear and bending moments are lowest in CFRP-400 and highest in CFRP-150. This is because the stiffer cables of CFRP-400 restrain the longitudinal movement of the pylons to a higher degree than the less stiff cables of CFRP-150. The higher the longitudinal displacement in the pylons, the higher the shear and bending moments will be at the base of the pylons.

4.1.4.2. Temperature Loads

Figures A-33 through A-38 show the axial load, shear, and bending moment in a single leg of one pylon under unfactored T+ and T- loads.

The axial load in the pylons increases for the CFRP bridges and decreases for the STEEL-190 bridge under T+. Under T+ the steel cables of STEEL-190 are allowed to expand due to their high coefficient of expansion, and because they are subject to a higher temperature increase than the pylons, the tension in the cables reduces under T+, thus reducing the axial compression in the pylons. On the other hand, the CFRP cables do not expand under T+, so when the pylons expand, the CFRP cables resist the expansion of the pylons, thus increasing the compressive force in the pylons. The compression in the pylons is the most prominent for CFRP-400 because it has the stiffest cables, which resist the expansion of the pylons to the largest extent. The opposite effect occurs under T-. When the pylons contract under T-, the steel cables also contract and place the pylons under compression since they are subject to a higher temperature decrease than the pylons. The CFRP cables do not contract under T-, and thus when the pylons contract, the tension in the cables reduces, which in turn reduces the axial force in the pylons.

The shear and bending moment in the pylons exhibit a very interesting behaviour under temperature loads. There is virtually no shear or bending moment above deck level in the CFRP bridges under temperature loads. This is consistent with the longitudinal pylon tip displacements of the CFRP bridges under T+. The CFRP bridges exhibit a negative longitudinal pylon tip displacement under T+, meaning that under T+ the pylon tips displace away from the midspan. The expansion of the deck due to a temperature increase will push the pylons away from the midspan due to the pinned connection between the pylons and the deck, resulting in a negative longitudinal pylon displacement at deck level. Based on the model results, for the CFRP bridges the rotations of the pylons in the longitudinal direction are very similar at the pylon tips and at deck level (within 10% of each other at -0.00024 rad and -0.00022 rad at the pylon tips and at deck level, respectively). Since bending moment is a derivative of rotation, having a constant value of rotation along the pylons between the tips and deck level means that the bending moment along the pylons should be close to zero, which is exactly the case for the CFRP bridges. On the other hand, the pylon tip displacement of the STEEL-190 bridge is positive under T+ due to the loosening of the backstays under T+. When combined with the negative displacement of the pylons

at deck level due to the expansion of the deck, the positive pylon tip displacement will create a positive bending moment within the pylon. In all of the bridges T+ causes the expansion of the deck which pushes the pylons apart at deck level. The pylons are fixed at the base, 10 m below the connection with the deck. This results in a significant bending moment in the pylons below deck level. The increase in negative bending moment below deck level is the same for STEEL-190 and the CFRP bridges. However, due to the bending moment in the CFRP bridges being close to zero and the bending moment in the STEEL-190 bridge having a positive value positive above deck level, the resulting negative bending moment at the base of the pylon is significantly larger for the CFRP bridges than for the STEEL-190 bridge. These results are consistent with the findings of [12], where the negative bending moments at the base of the pylons were found to be higher in bridges with CFRP cables than in bridges with steel cables. T- creates a similar but opposite effect on the shear and bending moment within the pylons of the bridges being considered. Under T-, a significant positive bending moment is created at the base of the pylons for the CFRP bridges. Under both T+ and T-, all four CFRP bridges exhibit similar shear and bending moments in the pylons, and the small differences in values between the CFRP bridges are not noteworthy.

4.1.4.3. ULS Load Combinations

Figures A-39 through A-47 show the axial load, shear, and bending moment envelopes for the pylons when subjected to the ULS-1, ULS-2-T+, and ULS-2-T- load combinations.

The differences in the pylon forces under ULS-1 are relatively small and are primarily attributed to the live load. The axial load is the smallest in the pylons of the CFRP-150 bridge and the largest in the pylons of the CFRP-400 bridge. Based on Figure A-39, the cable material and cable stiffness does not have a significant effect on the axial forces which develop in the pylons under ULS-1. The shear and bending moment in the pylons are highest for CFRP-150 and lowest for CFRP-400 due to the effects of the live load envelope which were discussed earlier.

The pylon axial forces under ULS-2-T+ and ULS-2-T- are also quite similar for all 5 bridges, as shown in Figures A-42 and A-45. The effect of the temperature loads on the axial forces in the pylons is relatively small when compared to the total axial force in the pylons. The STEEL-190 bridge exhibits the lowest compressive forces in the pylons under ULS-2-T+ and the highest compressive forces in the pylons under ULS-2-T- for the reasons discussed in Section 4.1.4.2.

Under ULS-2-T+ the negative shear and bending moments at the base of the pylons are significantly higher for the CFRP bridges than for the STEEL-190 bridge due to the effects of the temperature loads. The positive bending moments in the pylons are higher for the STEEL-190 bridge than they are for the CFRP bridges. Similarly, under ULS-2-T- the positive shear and bending moments at the base of the pylons are significantly higher for the CFRP bridges than for the STEEL-190 bridge, while the negative bending moments are highest for the STEEL-190 bridge. Out of the CFRP bridges, CFRP-150 exhibits the highest bending moments, while CFRP-400 exhibits the lowest bending moments. Based on these observations, the use of CFRP cables on a cable-stayed bridge that has a pinned or fixed connection between the pylons and the deck will significantly increase the shear and bending moments in the pylons under temperature loads; however, the use of higher modulus CFRP cables will decrease this effect.

4.1.5. Bridge Frequencies

Table 13 shows a comparison of the 1st torsional (T) to 1st bending (B) frequency ratios for the 5 bridges in **Phase I**.

Table 12 – **Phase I** Comparison of Bridge Frequencies

	1st Bending Frequency (B)	1st Torsional Frequency (T)	T:B Ratio	% Increase	‘T’ Mode Number
STEEL-190	0.4317	1.7130	3.97	-	12
CFRP-150	0.4021	1.7420	4.33	9%	14
CFRP-190	0.4357	1.7831	4.09	3%	13
CFRP-300	0.5127	1.8833	3.67	-7%	12
CFRP-400	0.5703	1.9629	3.44	-13%	12

It was found that both the first Bending Frequency (B) and the first Torsional Frequency (T) of the CFRP bridges increases as the stiffness of the CFRP cables increases. This makes sense because stiffer structures generally have higher frequencies. It was also noted that the mode number of T changes with the change in cable material and elastic modulus. Given the same elastic modulus, changing the cable material from steel to CFRP results in a slightly higher mode number for T. From there, decreasing the modulus of the CFRP material further increases the mode number of T, while increasing the modulus of the CFRP decreases the mode number of T. The mode number of B was 1 for all bridges. Data regarding the first 30 modes of each bridge can be found in Appendix C.

When comparing STEEL-190 and CFRP-190 it was found that CFRP-190 has slightly higher frequencies, which can likely be attributed to the lower mass of the bridge resulting from the lower density of the CFRP cables. This result agrees with the findings of [17]. CFRP-150 exhibits a lower B and T than STEEL-190. This conflicts with most studies, which have found that the use of CFRP cables increases the torsional frequency of a cable-stayed bridge due to the lighter mass of the cables [10, 12, 16, 17, 18]. It should be noted that the difference in the torsional frequencies of STEEL-190 and CFRP-150 is very small at less than 2%. It is possible that this discrepancy can be attributed to the relatively short span of the bridges being studied in this thesis. Most literature pertaining to the use of CFRP cables in cable-stayed bridges examines longer-span cable-stayed bridges, and it is expected that the results would be different if a longer bridge is studied. Overall, the T:B ratio is higher for CFRP-150 and CFRP-190 when compared to STEEL-190, and this ratio becomes lower as the stiffness of the cables is increased. The increase in the T:B ratio is very small for CFRP-150 and CFRP-190, which means that the aerodynamic stability of these bridges is comparable to that of STEEL-190. Looking at the T:B ratio alone, the use of HM CFRP cables appears to decrease the aerodynamic stability of cable-stayed bridges. The T:B ratio of CFRP-400 is 13% lower than that of STEEL-190. Of course, wind tunnel tests would need to be conducted to determine the true effects of the use of CFRP cables on the aerodynamic stability of cable-stayed bridges.

4.2. Phase II Results

In **Phase II** of the parametric study, the cross-sectional area of the cables is varied while keeping the elastic modulus of the material constant at 190 GPa. The four bridges compared in **Phase II** are STEEL-100%, CFRP-100%, CFRP-80%, and CFRP-70%. CFRP-100% has the same cable areas as STEEL-100%, resulting in cables which have a stiffness equivalent to that of the cables in the STEEL-100% bridge (equivalent strength principle); CFRP-80% has cables with areas which are 80% of those in the STEEL-100% bridge; and CFRP-70% has cables with areas which are 70% of those in the STEEL-100% bridge, resulting in cables, which have a strength equivalent to that of the cables in the STEEL-100% bridge (equivalent stiffness principle). Table 13 below shows the cable utilization ratios for the four bridges being compared in **Phase II**.

Table 13 – **Phase II** Comparison of Cable Utilization Ratios

Cable	Cable Utilization Ratio (max. stress / f_u) [-]			
	STEEL-100%	CFRP-100%	CFRP-80%	CFRP-70%
12b	0.46	0.31	0.38	0.42
11b	0.24	0.21	0.21	0.26
10b	0.32	0.24	0.29	0.31
9b	0.36	0.25	0.31	0.35
8b	0.37	0.26	0.32	0.37
7b	0.36	0.26	0.32	0.37
6b	0.34	0.26	0.30	0.35
5b	0.34	0.25	0.31	0.34
4b	0.35	0.24	0.29	0.33
3b	0.33	0.21	0.28	0.32
2b	0.26	0.17	0.26	0.28
1b	0.21	0.24	0.19	0.20
1a	0.30	0.21	0.26	0.32
2a	0.35	0.26	0.32	0.34
3a	0.38	0.28	0.34	0.38
4a	0.39	0.28	0.36	0.39
5a	0.39	0.28	0.36	0.40
6a	0.40	0.29	0.38	0.41
7a	0.43	0.30	0.38	0.43
8a	0.44	0.31	0.39	0.42
9a	0.42	0.31	0.38	0.43
10a	0.44	0.32	0.38	0.44
11a	0.46	0.33	0.39	0.44
12a	0.47	0.33	0.39	0.44

Various studies performed for cable-stayed bridges with CFRP cables have used different methods for sizing CFRP cables, as described in Chapter 2. In some studies, CFRP cables were sized based on the equivalent strength principle, resulting in cables with an area smaller than their steel counterparts [9, 24]. The equivalent strength principle results in cables that have a smaller stiffness. Other studies used the equivalent stiffness principle in which CFRP cables were sized to have the same axial stiffness as their steel counterparts [17, 18, 20, 21, 22, 23]. This resulted in the CFRP cables having a larger cross-sectional area than their steel counterparts due to the typical elastic modulus of CFRP being lower than the steel. The purpose of this portion of the parametric study is to study the effects of the cable sizing method on the static and dynamic performance of a cable-stayed bridge.

In **Phase I** of the parametric study, the elastic modulus of the cables was varied while maintaining a constant cable area for all bridges. An initial comparison of the **Phase II** results to the **Phase I** results shows that varying the cable area in **Phase II** has the same effect as decreasing the elastic modulus of the cable material in **Phase I**. **Phase I** results were presented in detail, including explanations of the possible reasons for the results, and given that varying cable area is similar to varying cable modulus, conducting a similar detailed discussion of the **Phase II** results would be redundant. Therefore, this section is limited to a brief summary of the key observations from **Phase II**. All **Phase II** results are presented in figures, which are included in Appendix B for **Phase II**.

4.2.1. Cables Forces

Cable forces were extracted from the models and plotted in bar charts to determine the effect of the cable material and elastic modulus on the forces which develop within the cables under various loads. Cable forces were extracted for the ULS load combinations ULS-1, ULS-2-T+ and ULS-2-T- to determine the effect of the cable material on the overall maximum cable loads. Cable forces were also extracted for the unfactored individual loads (DEAD, LL, T+ and T-), which the aforementioned ULS load combinations comprise, to examine the effect of each load alone on the forces in the cables and determine how these individual loads affect the total cable forces in the ULS load combinations.

Figure B-1 shows that cable forces appear to be relatively uniform under dead loads, with the minor variances likely attributed to errors in the iterative procedure used to determine the cable forces. Cable 1b is the only exception where CFRP-100% shows a much larger cable force than the rest of the bridges, and this is likely due to the iterative process behind determining the cable forces as explained in Section 4.1.1.1.

Under the live load envelope (Figure B-2), the pattern in the cable forces is similar to the one observed in **Phase I**. Reducing the area of the cables reduces the stiffness of the cables, and the stiffer cables attract more force due to live loads. The STEEL-100% and CFRP-100% bridges exhibit practically identical cable forces under live loads.

Under T+ (Figure B-3) the cables of the CFRP bridges exhibit more tension than the cables of the STEEL-100% bridge, similar to **Phase I**. Several cables (7b through 5b and 4f through 9f) do not exhibit this trend, and the reason for this is unknown. Like in **Phase I**, the stiffer cables are

subjected to higher loads under T+ while the more elastic cables attract less loads. CFRP-70% has the lowest cable forces out of the CFRP bridges, and CFRP-100% has the highest cable forces. The effects of T- (Figure B-4) are similar to the effects of T+ but in the opposite direction.

The variations in cable forces of the four bridges in load combinations ULS-1, ULS-2-T+ and ULS-2-T- (Figures B-5 through B-7) are small, like in **Phase I**. All cable forces are within 10% for each cable except for cable 1b, which shows a larger variation due to errors in the iterations used to determine cable forces under dead loads. Across all three load combinations, there is a general trend of CFRP-100% having the highest cable forces and CFRP-70% having the lowest cable forces among the CFRP bridges, which can be attributed to stiffer cables attracting more force under live and temperature loads. In ULS-1 and ULS-2-T- there appears to be a trend of the STEEL-100% bridge having higher cable forces than the CFRP bridges for many of the cables. It can be concluded that varying the cable area does not significantly impact the cable forces of a cable-stayed bridge.

4.2.2. Deck and Pylon Tip Displacements

The bridges with more elastic cables exhibit higher vertical deck displacements under live loads than the bridges with stiffer cables. Figure B-8 shows that STEEL-100% and CFRP-100% exhibit identical deck displacements under live loads, while CFRP-70% exhibits the largest deck displacements under live loads due to this bridge having the most elastic cables.

The patterns observed under T+ and T- are similar to those observed in the similar loading case in **Phase I**. Under T+ (Figure B-9) the CFRP bridges exhibit upward deck displacement due to the expansion of the pylon, while the STEEL-100% shows a downward displacement much larger in magnitude than the upward displacement of the CFRP bridges due to the elongation of the steel cables. The CFRP bridges exhibit a higher upward displacement in the side spans than the STEEL-100% bridge for similar reasons. Out of the CFRP bridges, CFRP-100% exhibits the largest displacements in the side spans, while CFRP-70% exhibits the lowest displacement in the side spans. This is similar to **Phase I**, since the CFRP-400 bridge (the stiffest bridge) exhibited the highest displacements in the side spans. In contrast, CFRP-150 (the most elastic bridge) exhibited the lowest displacements. The effects under T- (Figure B-10) are similar but in the opposite direction.

Under the SLS-1 envelope (Figure B-11), the STEEL-100% bridge exhibits the highest displacements in the main span and the lowest displacements in the side spans, mainly due to the effects of the temperature loads. CFRP-100% has the lowest displacements of all the bridges in the main span and the lowest displacements of the CFRP bridges in the side spans. There is a clear relationship between the area of the cables and the deck displacements under the SLS-1 envelope. The smaller the cable area, the higher the deck displacements.

The longitudinal pylon tip displacements are presented in Table 14.

Table 14 – **Phase II** Longitudinal Pylon Tip Displacements

	LIVE LOAD ENV		T+	T-	SLS-1 ENV	
	MIN	MAX			MIN	MAX
STEEL-100%	-21	37	23	-24	-38	52
CFRP-100%	-21	37	-14	35	-30	61
CFRP-80%	-23	42	-15	36	-33	66
CFRP-70%	-24	45	-15	36	-34	69

The pylon tip displacements under live load are identical for STEEL-100% and CFRP-100%, which is to be expected since these two bridges have cables with identical stiffnesses. The displacements become larger for CFRP-80% and are the largest for CFRP-70%. The pylon tip displacement of STEEL-100% is positive under T+, while the pylon tip displacements of the CFRP bridges are negative. The reverse effect is seen under T-. Under temperature loads, CFRP-70% has slightly larger pylon tip displacements than CFRP-100%. Under the SLS-1 envelope, STEEL-100% has the largest negative displacement, while CFRP-100% has the smallest negative displacement. STEEL-100% has the smallest positive displacement, while CFRP-70% has the largest positive displacement. Overall, decreasing the cable area results in an increase in the pylon tip displacements of the bridge.

4.2.3. Deck Forces

Under live loads, the axial forces in the deck are identical for STEEL-100% and CFRP-100% as shown in Figure B-12. The axial loads are highest for STEEL-100% and CFRP-100%, and lowest for CFRP-70% because the smaller cables of CFRP-70% attract less live load, resulting in less compression in the deck from the horizontal component of the cable tensions. The differences in axial forces are, however, very small. The differences in shear forces are also very small as seen

in Figure B-13, but it does appear that CFRP-70% shows slightly higher shear forces than the rest of the bridges, which is likely because there is more bending moment in the deck. The bending moments in the deck are the highest for CFRP-70% and lowest for STEEL-100% and CFRP-100% as shown in Figure B-14. Overall, the decrease in cable area decreases the axial forces and increases the shear and bending moments in the deck under live load.

Under T+ the deck of STEEL-100% exhibits the highest axial forces in the main span and lowest axial forces in the side spans (Figure B-15). Of the CFRP bridges, CFRP-100% has the highest axial forces, and CFRP-70% has the lowest axial forces in the deck, which is likely because the stiffer cables of CFRP-100% attract more load under temperature, thus putting the deck under more compression. STEEL-100% has the highest shear and bending moment at both the pylon and the main span, and the lowest shear and bending moment at the side spans, as shown in Figures B-16 and B-17. CFRP-100% exhibits the highest shear and bending moment in the side spans and at the pylon, while CFRP-70% exhibits the lowest. There is no clear trend in the shear and bending moment of the CFRP bridges in the main span. Similar to **Phase I**, STEEL-100% exhibits a large negative moment at the pylon, while the CFRP bridges show a positive moment at the pylon, which is smaller in magnitude. STEEL-100% has a positive moment in the main span due to the downward displacement of the deck, while the CFRP bridges have a negative moment in the main span due to the upward displacement of the deck, as explained in Section 4.1.3.2. The effects under T- are similar but in the opposite direction (Figures B-18 through B-20). Similar to **Phase I**, the magnitude of the negative bending moment at the pylon in the CFRP bridges is larger than the magnitude of the positive bending moment at the pylon of STEEL-100%.

Under ULS-1 the axial forces in the deck (Figure B-21) are almost the same for all bridges, as are the shear forces (Figure B-22). The bending moments appear to be the lowest for STEEL-100% and the highest for CFRP-70% (Figure B-23). In general, the bending moments appear to increase with the decrease in cable area. Under ULS-2-T+ the axial forces in the deck are very similar for all bridges, and the differences are insignificant as shown in Figure B-24. The shear in the deck is almost the same throughout the bridge with the exception of the pylon, where a spike in the shear of STEEL-100% is observed, as shown in Figure B-25. The positive bending moments in the deck are the highest for STEEL-100% at midspan and highest for CFRP-70% in the side span, as seen in Figure B-26. The negative bending moments are lowest for STEEL-100% at midspan and in the

side spans, but there is a spike in negative bending moment for STEEL-100% due to temperature effects. Among the CFRP bridges, there is a relatively consistent trend of CFRP-70% having the largest bending moments and CFRP-100% having the smallest bending moments. These effects are similar but in the opposite direction under ULS-2-T- (Figures B-27 through B-29).

4.2.4. Pylon Forces

The pylon axial loads are the highest for STEEL-100% and CFRP-100% under live loads and the lowest for CFRP-70%, as seen in Figure B-30. On the other hand, the shear and bending moments in the pylon (Figures B-31 and B-32) are the highest for CFRP-70% and the lowest for STEEL-100% and CFRP-100% under live loads.

T+ creates a slight positive axial force in the pylons for STEEL-100% and a smaller negative axial force in the pylons for the CFRP bridges (Figure B-33). CFRP bridges' largest negative force is attributed to CFRP-100%, while the smallest is attributed to CFRP-70%. The shear in the pylon under T+ is higher for STEEL-100% above deck but lower for STEEL-100% below deck, with the shears in the CFRP bridges being very similar, as shown in Figure B-34. Figure B-35 shows that the bending moment in the pylon is nearly zero above deck for the CFRP bridges for reasons explained in Section 4.1.4.2, and in the STEEL-100% bridge, there is a large positive moment in the pylon above deck level. Below deck level, all bridges experience a large negative jump in bending moment. Since STEEL-100% has a high positive bending moment above deck, the negative jump causes a much smaller negative bending moment at the base of the pylon of the STEEL-100% bridge than at the base of the pylons of the CFRP bridges. There is virtually no difference in the pylon bending moments among the CFRP bridges. T- creates similar effects but in the opposite direction as shown in Figures B-36 through B-38.

Under ULS-1, the pylon axial loads are relatively similar for all bridges, although the axial loads in STEEL-100% and CFRP-100% appear to be slightly higher as seen in Figure B-39. The shear (Figure B-40) seems to be slightly higher for the CFRP-70% bridge and lowest for the STEEL-100% and CFRP-100% bridges. The bending moments (Figure B-41) are also highest in CFRP-70% and lowest in STEEL-100% and CFRP-100%. Under ULS-2-T+, the axial load appears to be the highest in CFRP-100% and the lowest in STEEL-100%, as shown in Figure B-42. The positive shear is the highest in STEEL-100% and the lowest in CFRP-100%, as shown in Figure B-43. The

negative shear is the lowest in STEEL-100% and the highest in CFRP-70%. The positive bending moment is highest in STEEL-100% and lowest in CFRP-100%, as seen in Figure B-44. The negative bending moment is highest in CFRP-70% and lowest in STEEL-100%. CFRP-70% exhibits the highest shear and bending moment among the CFRP bridges, and CFRP-100% exhibits the lowest shear and bending moment. These effects are similar but in the opposite direction under ULS-2-T- (Figures B-45 through B-47).

4.2.5. Bridge Frequencies

Table 15 shows a comparison of the 1st torsional (T) to 1st bending (B) frequency ratios for the 4 bridges in **Phase II**.

Table 15 – **Phase II** Comparison of Bridge Frequencies

	1st Bending Frequency (B)	1st Torsional Frequency (T)	T:B Ratio	% Increase	T Mode Number
STEEL-100%	0.4317	1.7130	3.97	-	12
CFRP-100%	0.4357	1.7831	4.09	3%	13
CFRP-80%	0.4064	1.7484	4.30	8%	14
CFRP-70%	0.3894	1.7283	4.44	12%	14

It was found that decreasing the area of the cables increases the T:B ratio, thus improving the aerodynamic stability of the bridge. Of course, this would have to be confirmed by conducting wind tunnel tests, since the T:B ratio is a very simplistic method of determining the aerodynamic stability of a bridge. Decreasing the cable area decreases both B and T due to the reduced stiffness of the structure. Decreasing the cable area also results in a higher mode number for T. Changing the cable material from steel to CFRP without changing the cable area results in a slight increase in both B and T due to the decrease in the mass of the structure, and it also results in a slight increase in the T:B ratio. Data regarding the first 30 modes of each bridge can be found in Appendix D.

Chapter 5. Conclusions and Future Work

5.1. Conclusions

The possibility of using CFRP instead of steel for the cables of cable-stayed bridges has been studied for several decades. Among all the available literature, the unanimous observation regarding the use of CFRP cables in cable-stayed bridges is that CFRP cables are less stiff than steel cables, except in the case of very long spans where the cable sag effect significantly reduces the stiffness of steel cables. The first objective of this thesis was to review the existing literature on the use of CFRP cables in cable-stayed bridges, identify agreements and disagreements in the literature, and identify areas which have not been addressed in existing literature. The second objective of this thesis was to explore the effects of using HM CFRP cables on the static and dynamic performance of cable-stayed bridges. The third objective was to explore the effects of the cable sizing methodology on the static and dynamic performance of cable-stayed bridges by varying the cable area using the equivalent stiffness and equivalent strength principles. The fourth objective was to perform a comprehensive analysis of the performance of cable-stayed bridges with CFRP cables under thermal loads.

Upon completion of the literature review, the following observations were noted:

- The elastic modulus of CFRP is in the range of 150 to 160 GPa, which is lower than the elastic modulus of steel (around 190 to 200 GPa). This generally makes CFRP cables less stiff, except in the cases where the cable sag effect significantly reduces the stiffness of steel cables. The unit weight of CFRP is approximately 4 times lower than that of steel, which means that the cable sag effect is a lot lower for CFRP cables than for steel cables.
- The cable sizing methodology varies between studies. Some studies size CFRP cables based on the equivalent stiffness method by making the cable area of the CFRP cables slightly larger than their steel counterparts such that the axial cable stiffness EA is equal for both types of cables. Some studies size CFRP cables based on the equivalent strength method by which the area of the CFRP cables is chosen such that the ultimate strength of the CFRP cables is equal to that of the steel cables. This method results in the CFRP cables being less stiff. Another method which has been referenced is sizing CFRP cables by the equivalent area method which is a compromise between the two.

- The use of CFRP cables generally results in higher displacements and bending moments but lower axial compression in the deck under live loads.
- CFRP cables have good long term properties. CFRP exhibits virtually zero creep under stress levels below $0.6f_u$, and the fatigue performance of CFRP is superior to that of steel when stresses are not too high. Given that the maximum ultimate stress prescribed for cables by CHBDC is $0.55f_u$, creep is not a concern for CFRP cables, and their fatigue performance under these stress levels is better than the fatigue performance of steel cables.
- There is no consensus regarding the effect of the use of CFRP cables on the aerodynamic stability of cable-stayed bridges. The aerodynamic stability of cable-stayed bridges is governed by the ratio of the 1st torsional frequency (T) to the 1st bending frequency (B), and a low ratio of T to B may result in flutter. On one hand, the lower mass of CFRP cables increases both T and B. On the other hand, the lower stiffness of CFRP cables decreases both T and B. Some studies report that the critical wind speed increases when CFRP cables are used, while other studies report the opposite effect.

One of the main gaps in the existing literature pertains to the performance of cable-stayed bridges with CFRP cables under thermal loads. Currently there is only one study which has examined the effect of thermal loads on the behaviour of cable-stayed bridges [12]. This study only considered the effects of a temperature increase and conducted only a brief discussion of the implications of thermal loads on the behaviour of cable-stayed bridges with CFRP cables. Another gap in the literature pertains to the use of HM CFRP materials for cable-stayed bridges. While many studies acknowledged the fact that traditional CFRP materials are more elastic than steel, only one study mentioned that higher modulus CFRP materials exist [13], and none of the studies have examined the effect of using HM CFRP materials for cables.

In order to address these gaps in the existing literature, a parametric study was conducted in two phases. **Phase I** involved varying the stiffness of the CFRP material while keeping the cable area constant and comparing these results to a cable-stayed bridge with steel cables. **Phase II** involved varying the cable area between equivalent stiffness and equivalent strength while keeping the elastic modulus of the CFRP constant and comparing these results to a cable-stayed bridge with steel cables. A finite element model was created for each cable-stayed bridge being compared using the software CSiBridge. The applied loads included dead loads, initial cable tensioning,

positive and negative temperature loads, and live loads. Only CHBDC load combinations SLS-1, ULS-1, and ULS-2 were considered. Displacements, axial forces, shear, and bending moments were compared for the cables, deck, and pylons under separate loads and under the load combinations as applicable. The dynamic performance of the bridges was also briefly discussed; the T:B ratio was recorded for each bridge. The following conclusions were drawn after reviewing the results of the parametric study:

- Reducing the cable area of CFRP cables has the same effect as reducing the elastic modulus of CFRP cables. As such, the results obtained in **Phase I** and **Phase II** of the parametric study are very similar in nature.
- Increasing the modulus of CFRP cables attracts slightly more live load to the cables which increases the axial forces in the deck and pylons. However, increasing the stiffness also allows for less displacements in the pylons and deck under live loads, and thus the shear and bending moments in the pylons and deck are slightly lower under live loads when the modulus is increased. Decreasing the cable area has a similar but opposite effect.
- Temperature loads have a monumental effect on the performance of cable-stayed bridges with CFRP cables, and this effect cannot be ignored. CFRP has a much lower coefficient of thermal expansion than steel and concrete, which means that when the temperature changes, the deck and pylon expand and contract while the CFRP cables remain relatively constant in length. When a temperature increase is applied to the bridge, the pylon and deck expand, and the expansion of the pylons pulls the deck upwards since the CFRP cables do not expand. This creates a positive bending moment in the deck at the pylons and a negative bending moment in the deck at the main and side spans. This effect becomes more prominent when the stiffness of the CFRP or the cable area is increased. The steel bridge exhibits the opposite effect as the steel cables expand to a larger extent than the pylons. In a steel bridge the bending moment in the deck is negative at the pylons and positive in the main and side spans. The pylons exhibit a very unexpected behaviour. Under positive temperature loads the pylons of the steel bridge exhibit a high positive bending moment above deck level, and there is a sharp decrease in bending moment below deck level, resulting in a negative bending moment at the base of the pylons. The CFRP bridges, on the other hand, have virtually no bending moment above deck level when subjected to a temperature increase, but the decrease in moment below deck level is approximately the

same as for the steel bridge, and thus the resulting negative bending moment at the base of the pylons is much higher for the CFRP bridges than for the steel bridge. The stiffness of the CFRP material and the cable area do not have a notable effect this behaviour. All of these effects are similar but in the opposite direction when the bridges are subjected to a temperature decrease.

- When examining the cable, deck and pylon forces under load combinations which capture the effect of all applied loads acting together, it appears that the use of HM CFRP cables has benefits. Increasing the modulus of CFRP cables result in overall lower bending moments in the deck within the main span in both ULS-1 and ULS-2, which could allow bridge designers to use a slenderer cross section for the deck, decreasing the construction cost for the deck. It also results in lower bending moments in the pylons, which can also reduce costs by using a smaller cross section for the pylons. It is also noted that the axial loads in the deck and pylons are slightly higher in bridges with stiffer CFRP cables, but the differences are minor. The only drawback is higher modulus CFRP cables result in slightly higher bending moments in the side spans under ULS-2 load combinations due to the effect of thermal loads. To account for this, designers would need to use a slightly deeper deck cross section within the side spans.
- When considering the deflected shape of the deck and pylons under the SLS-1 load envelope, overall, both the deck and pylons undergo smaller deflections in bridges where a higher modulus of CFRP is used for the cables. Increasing the elastic modulus of the cable material would allow designers to use slenderer deck sections to meet the deflection requirements prescribed by codes.
- It should be noted that using CFRP cables in general results in considerably higher bending moments at the base of the pylons when compared to bridges with steel cables due to the effect of temperature loads, but this effect is reduced by using higher modulus CFRP cables. Additionally, the use of CFRP cables increases the bending moments in the side spans when compared to bridges with steel cables due to the effect of temperature loads, and this effect is somewhat exacerbated by the use of HM CFRP cables.
- Increasing the elastic modulus or the area of CFRP cables increases both the 1st torsional (T) and 1st bending (B) frequencies of a cable-stayed bridge, and it also decreases the ratio

of T to B. Consequently, increasing the elastic modulus of the cables decreases the critical wind speed required to cause flutter.

5.2. Limitations and Future Work

The scope of this thesis was limited to studying the static behaviour of cable-stayed bridges under the ULS-1 and ULS-2 load combinations, and the only dynamic behaviour that was examined in the thesis is the ratio of T to B. Wind loads, seismic loads, and complex dynamic behaviours were not considered in this thesis due to the complexity of these topics. The parametric study which was conducted in this thesis should be extended to include the effects of wind loads, seismic loads, and ice accretion, as well as collision loads and other exceptional loads. A more comprehensive analysis of the bridge's dynamic behaviour should also be conducted.

The parametric study should be expanded to include various bridge spans and deck-pylon connection types. It is likely that the results obtained in this thesis are largely influenced by the fact that the bridge which was studied has a relatively short main span of 250 m when compared to other cable-stayed bridges, and the fact that there is a pinned connection between the deck and the pylons. It is expected that the cable-stayed bridge which was studied would behave very differently if the connection between the deck and the pylon were to be different or nonexistent. The parametric study should also be performed for various bridge geometries which may entail:

- Different number of pylons (the bridge studied in this thesis has two pylons);
- Different pylon shapes such as A, Y, and diamond shapes (the bridge studied in this thesis has H shaped pylons);
- Different cable spacings;
- Harp and fan cable arrangements (the bridge studied in this thesis has semi-fan cable arrangement); and
- Different member sizes (e.g., varying the size of the pylons and deck).

Additionally, the dynamic behaviour of the bridge is likely affected by the span length, and increasing the span may yield different results pertaining to the critical wind speed of cable-stayed bridges with HM CFRP cables.

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Appendix A. Phase I Results

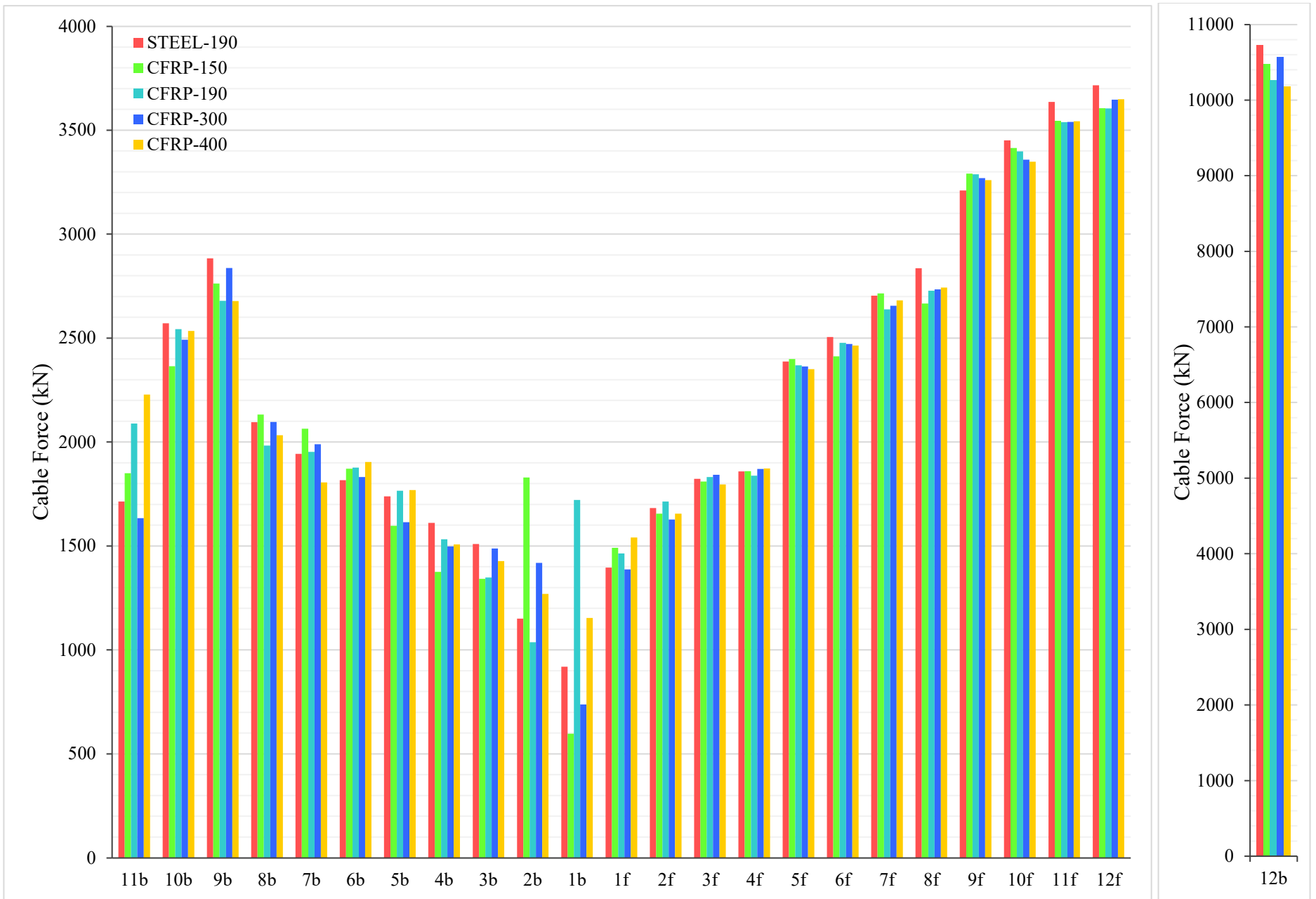


Figure A-1: Cable forces under SLS-1 TENSION BACKSTAYS

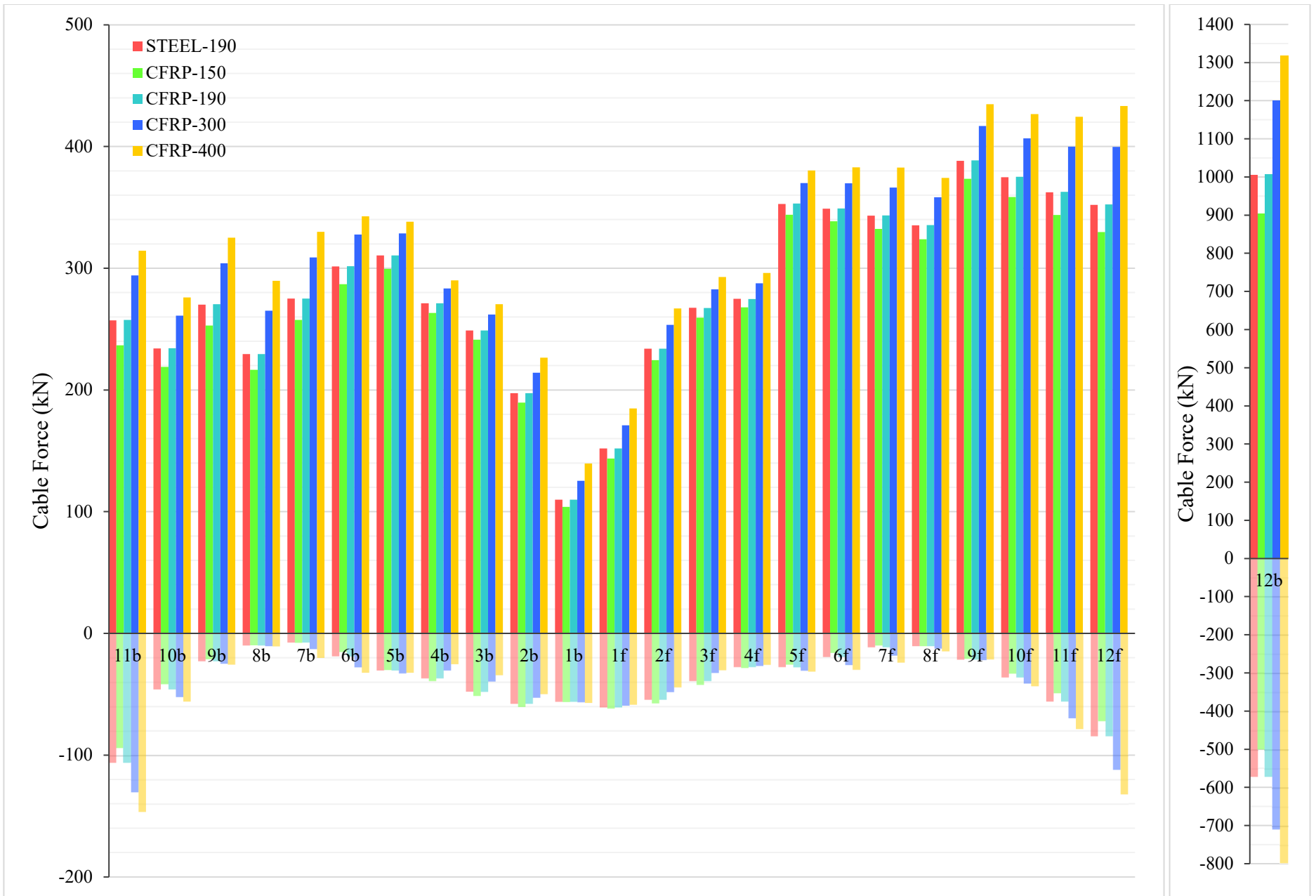


Figure A-2: Maximum increase and decrease in cable forces under live load envelope

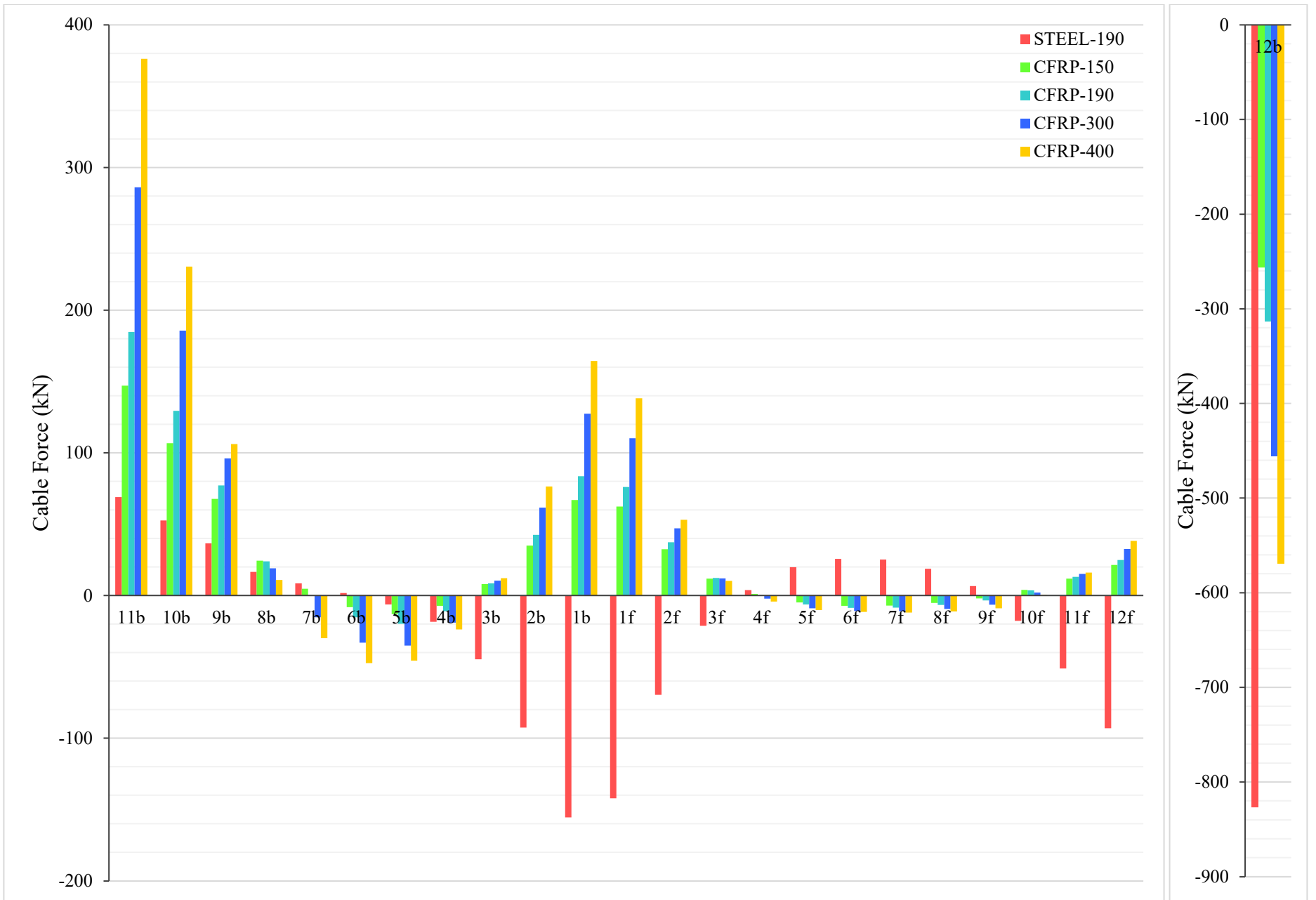


Figure A-3: Change in cable forces under T+

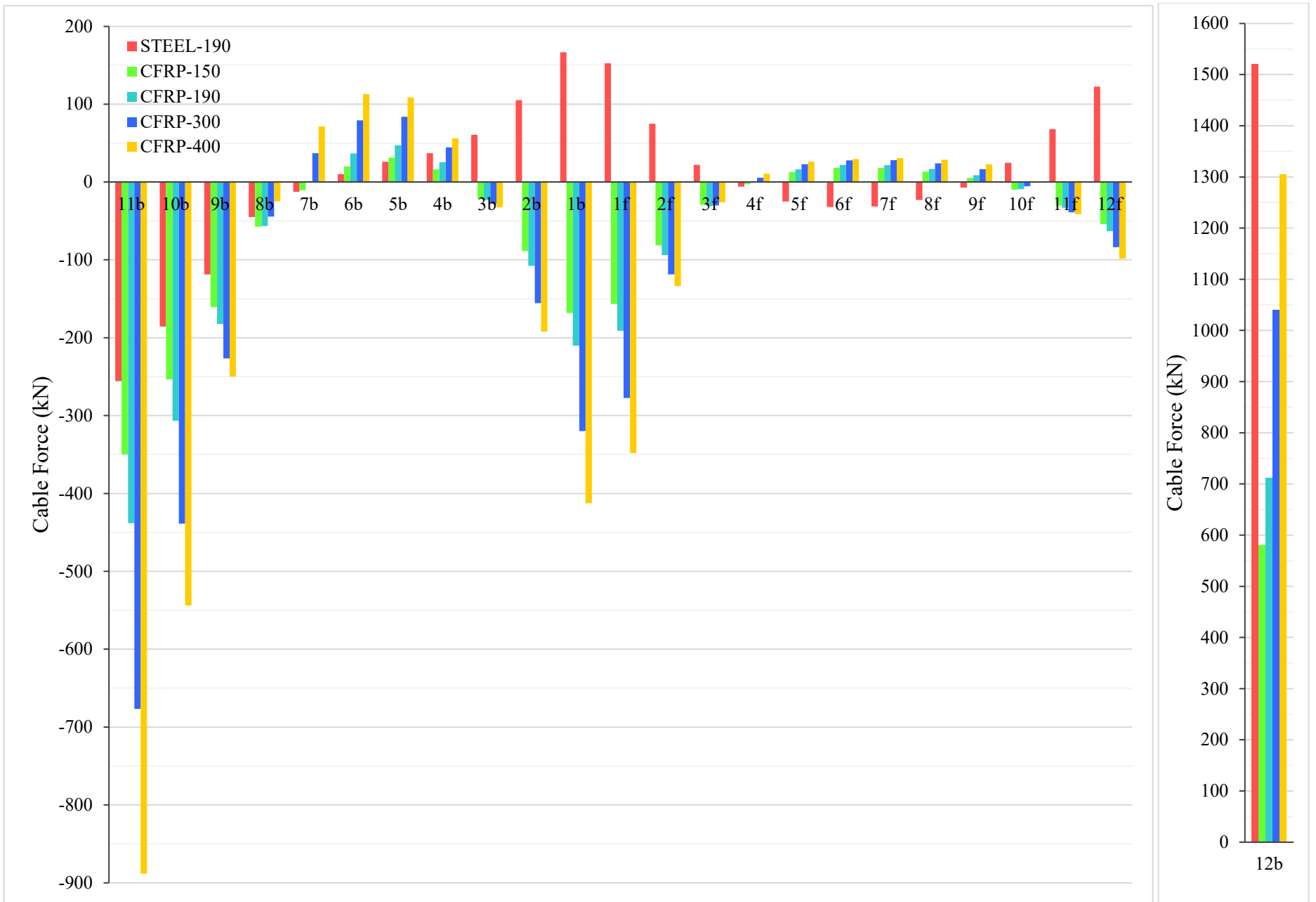


Figure A-4: Change in cable forces under T-

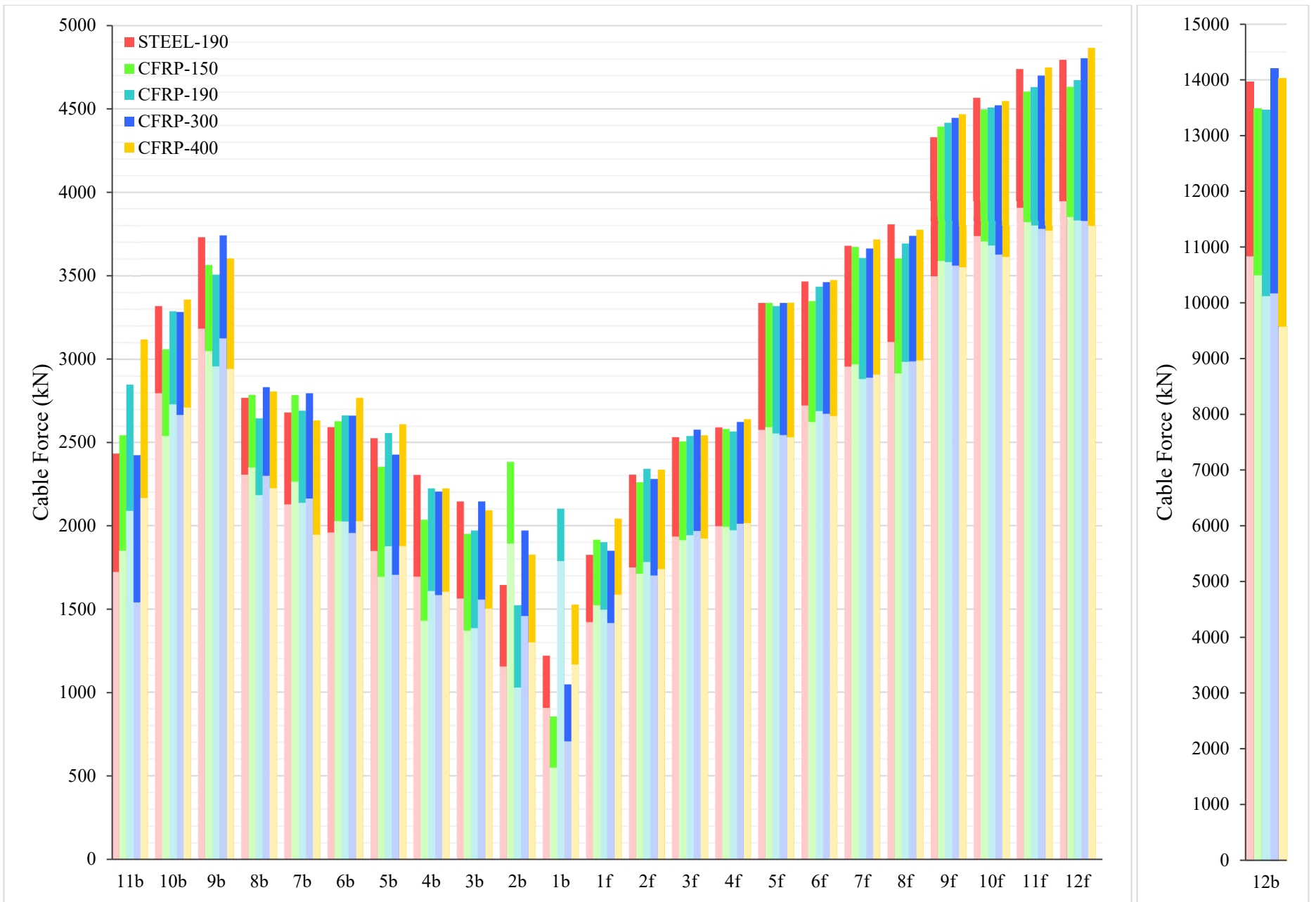


Figure A-5: Minimum and maximum cable forces under ULS-1 envelope

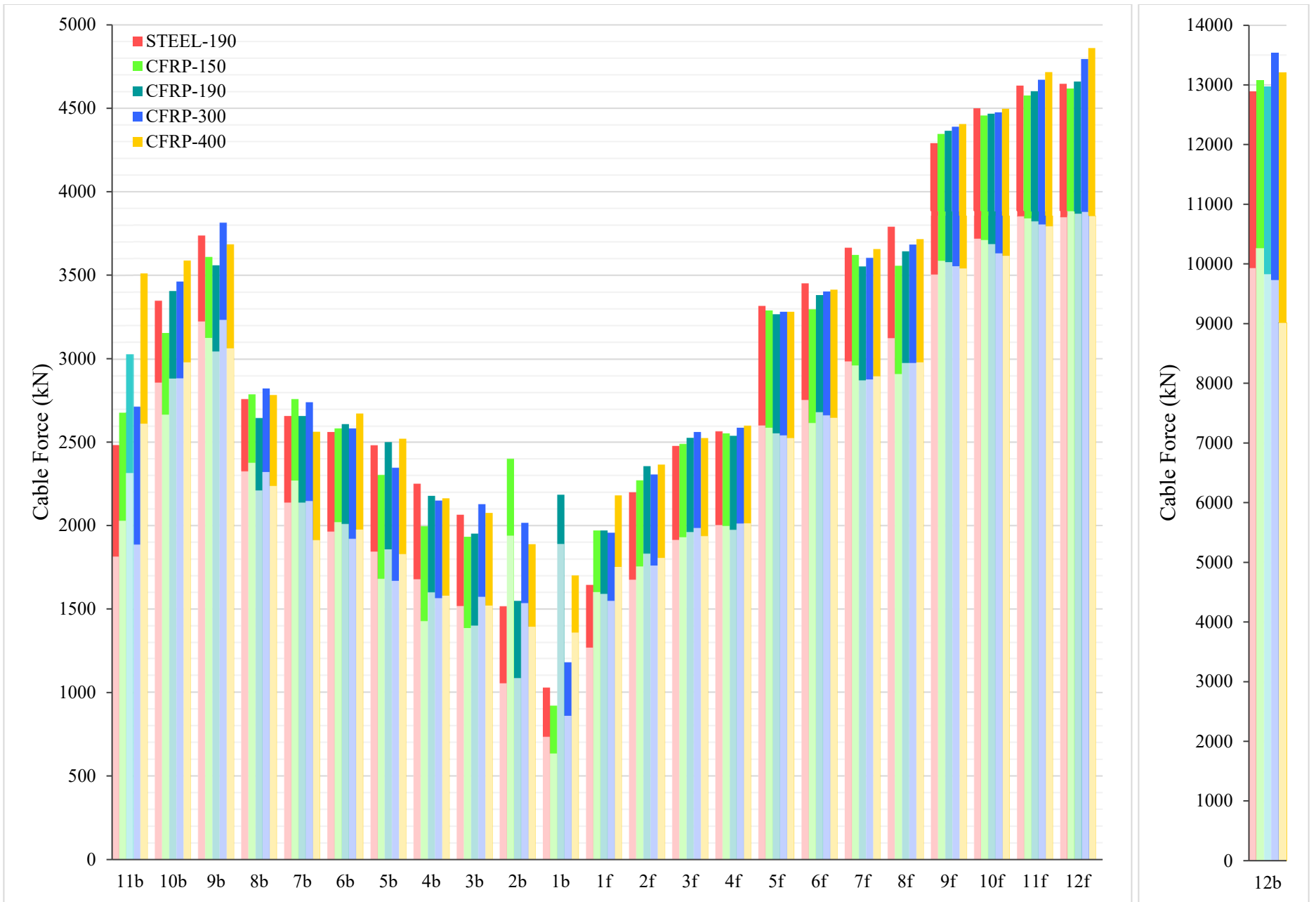


Figure A-6: Minimum and maximum cable forces under ULS-2-T+ envelope

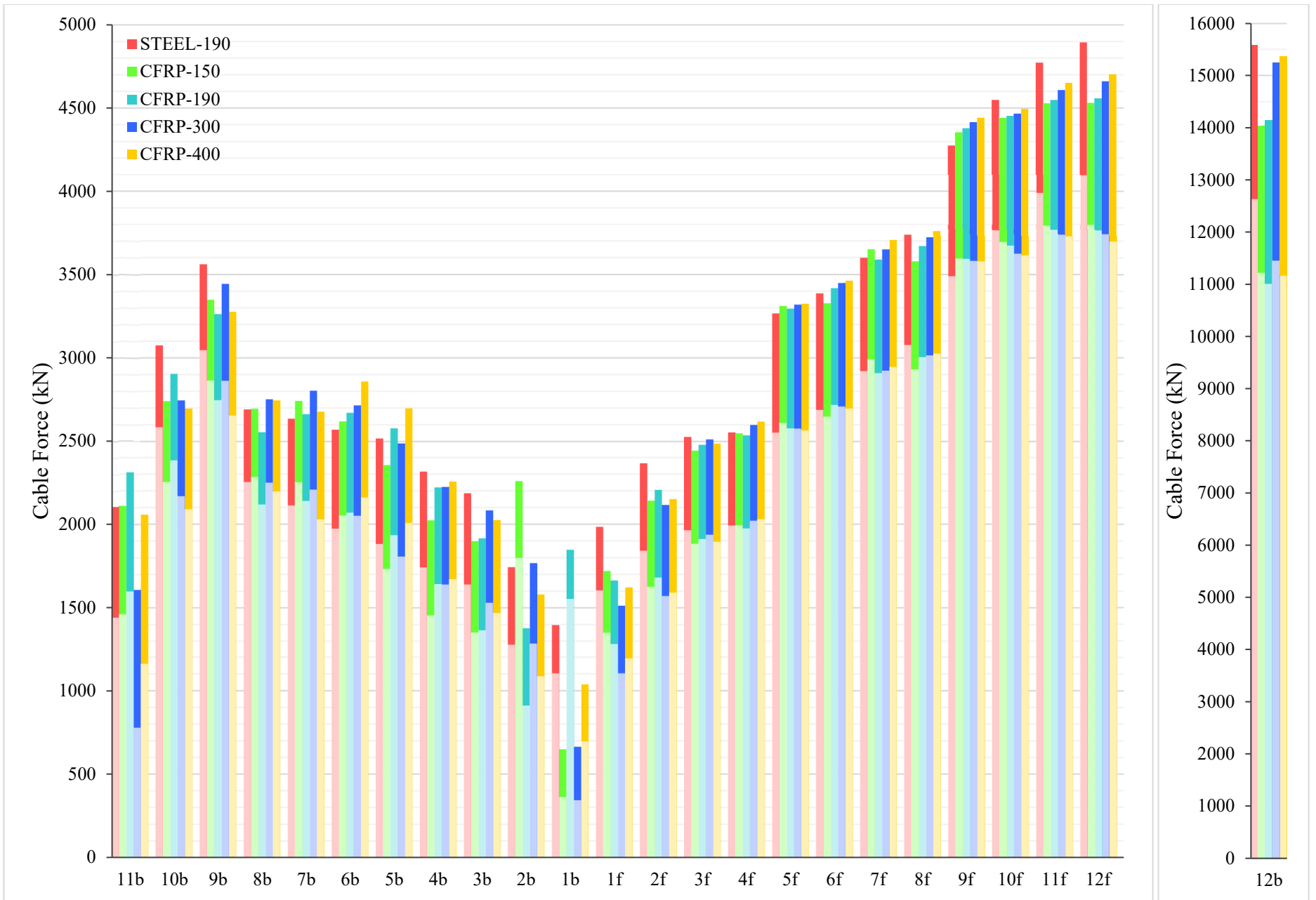


Figure A-7: Minimum and maximum cable forces under ULS-2-T- envelope



Figure A-8: Minimum and maximum vertical deck displacements under LL ENV

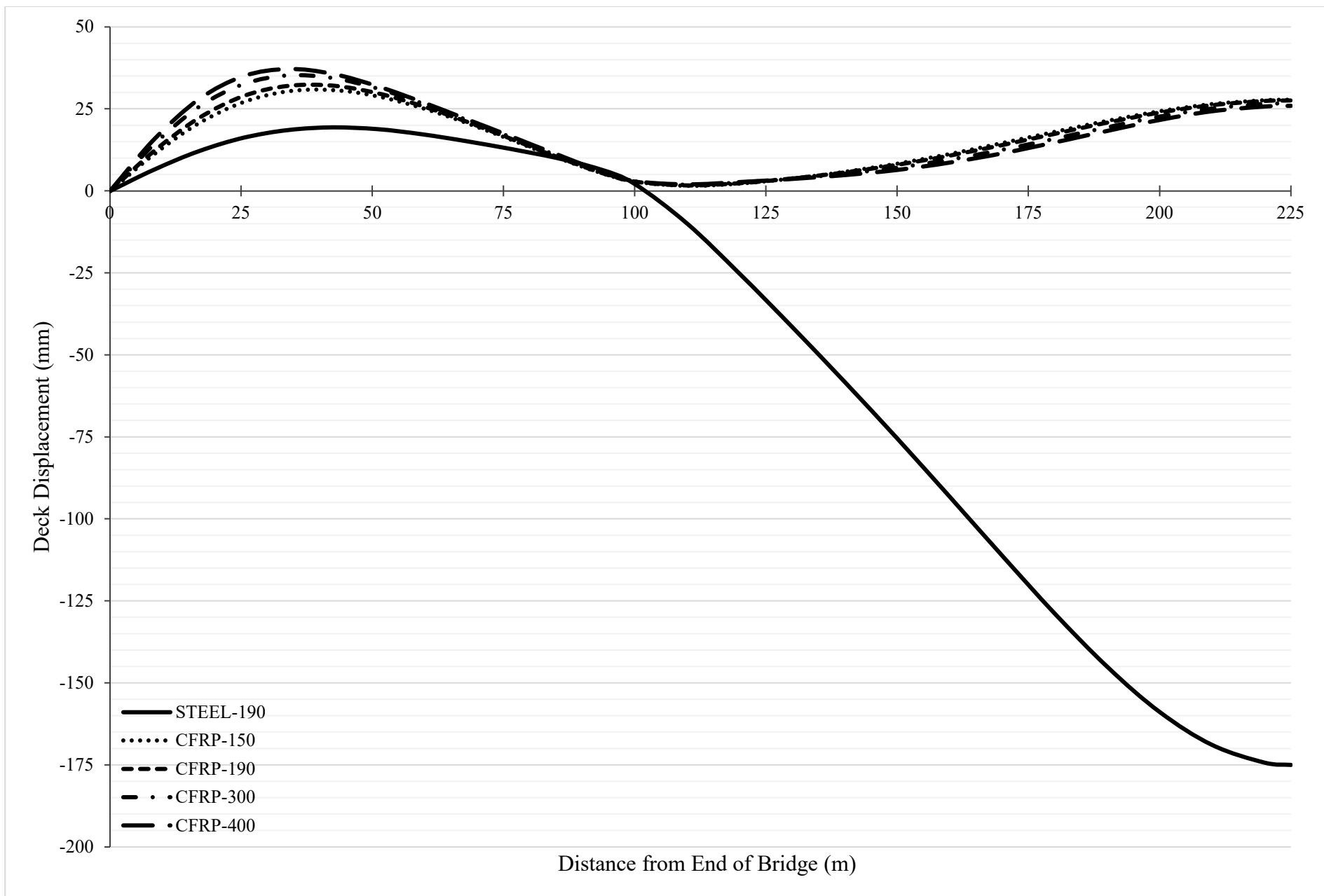


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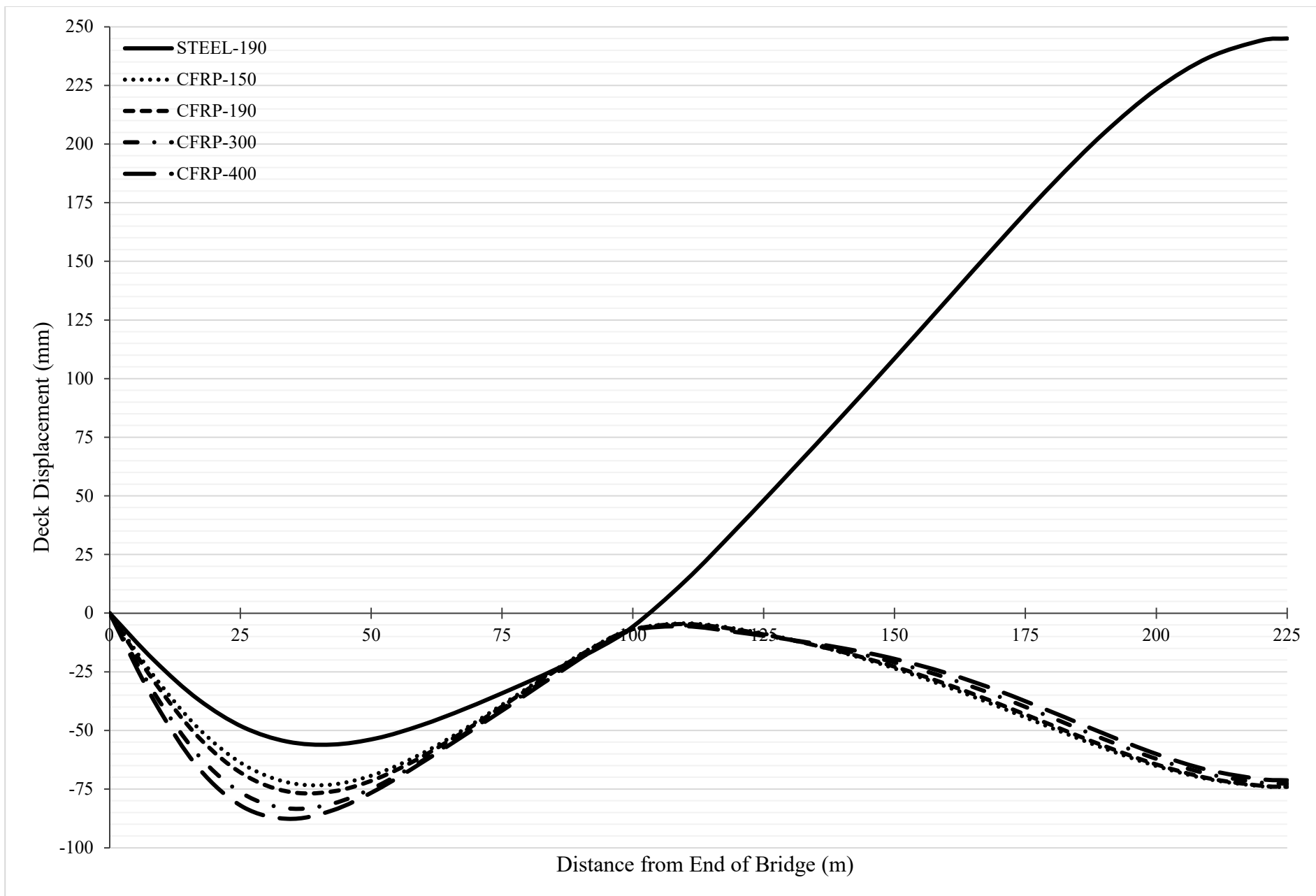


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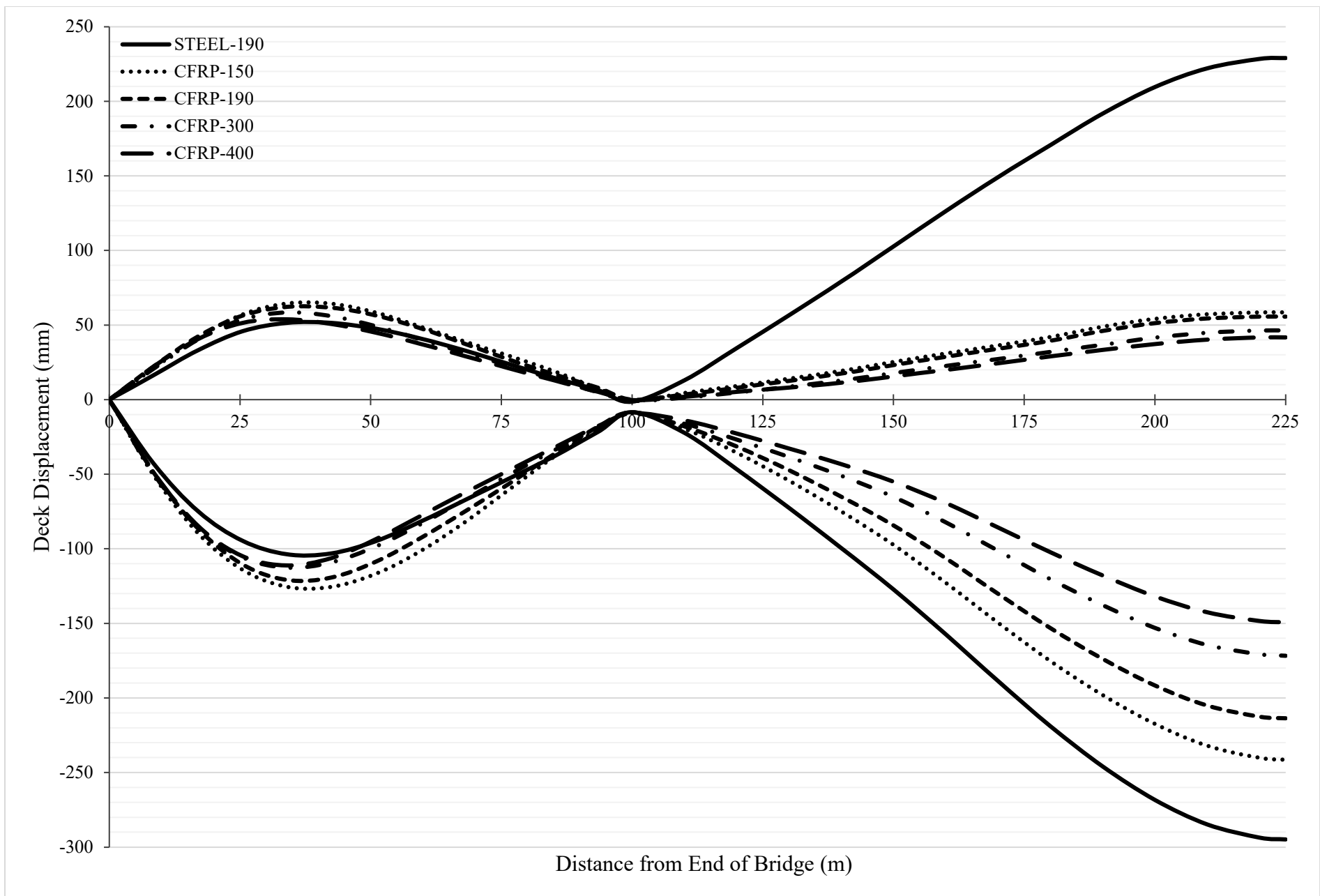


Figure A-11: Minimum and maximum vertical deck displacements under SLS-1 envelope

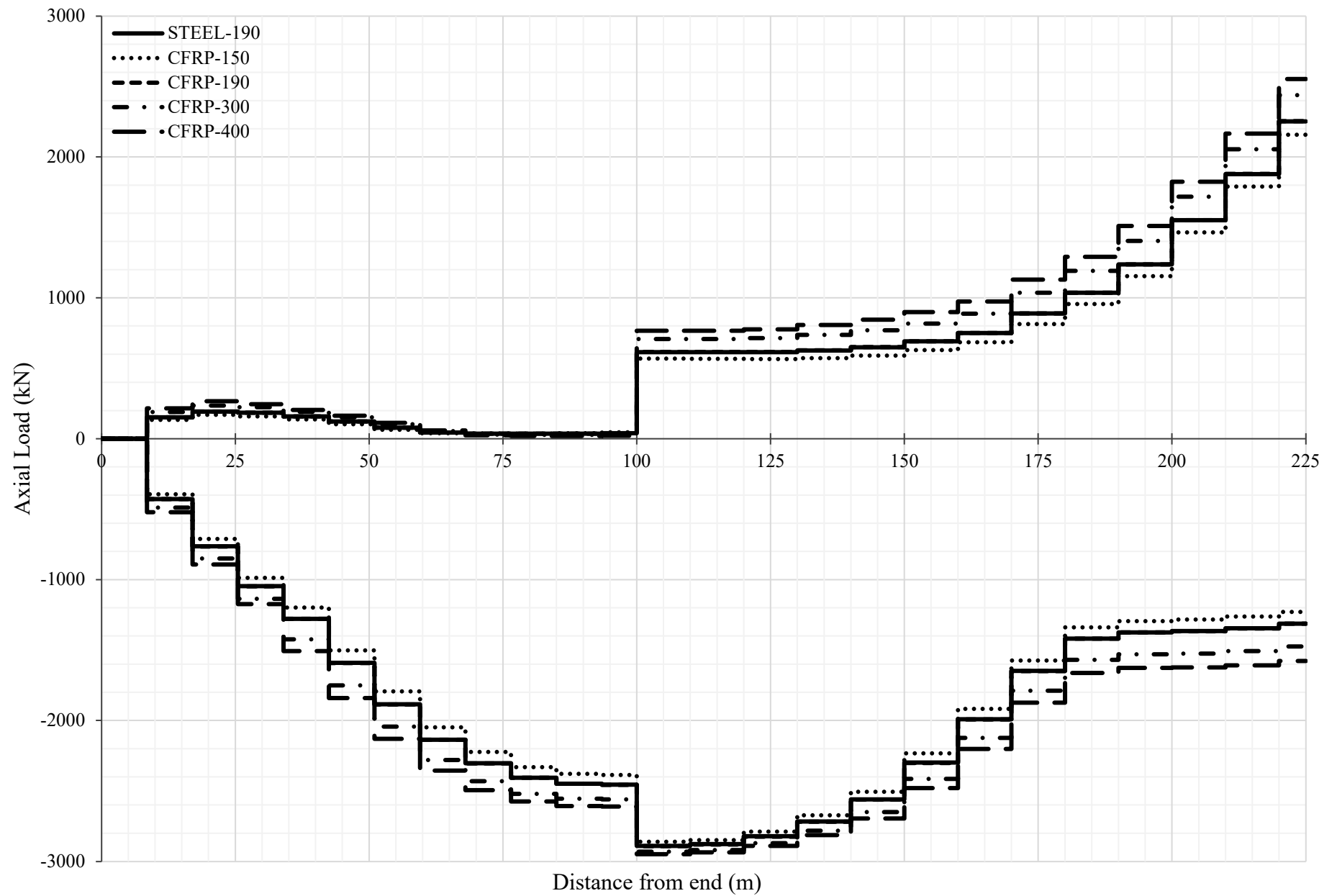


Figure A-12: Minimum and maximum deck axial loads under LL ENV

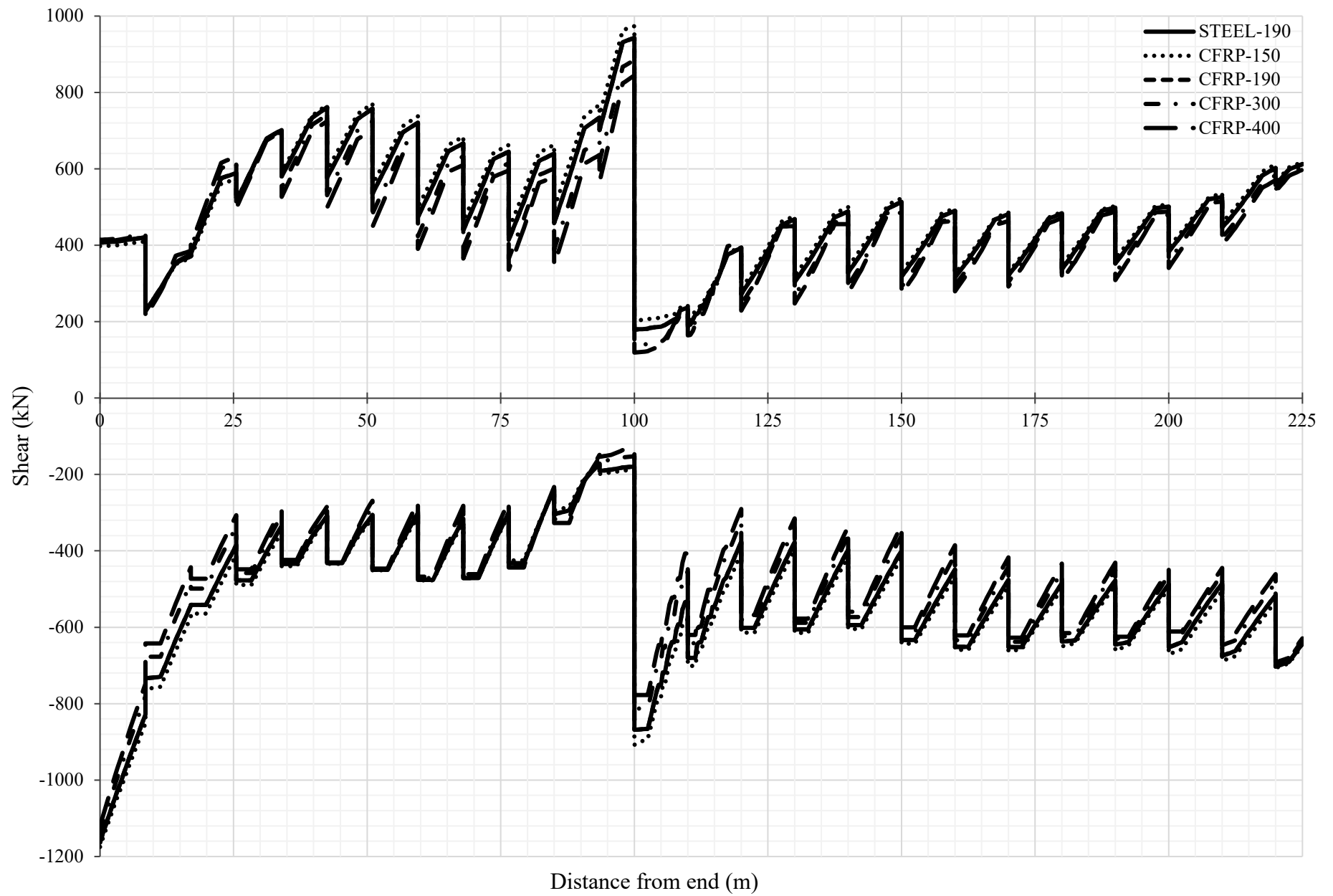


Figure A-13: Minimum and maximum deck shear under LL ENV

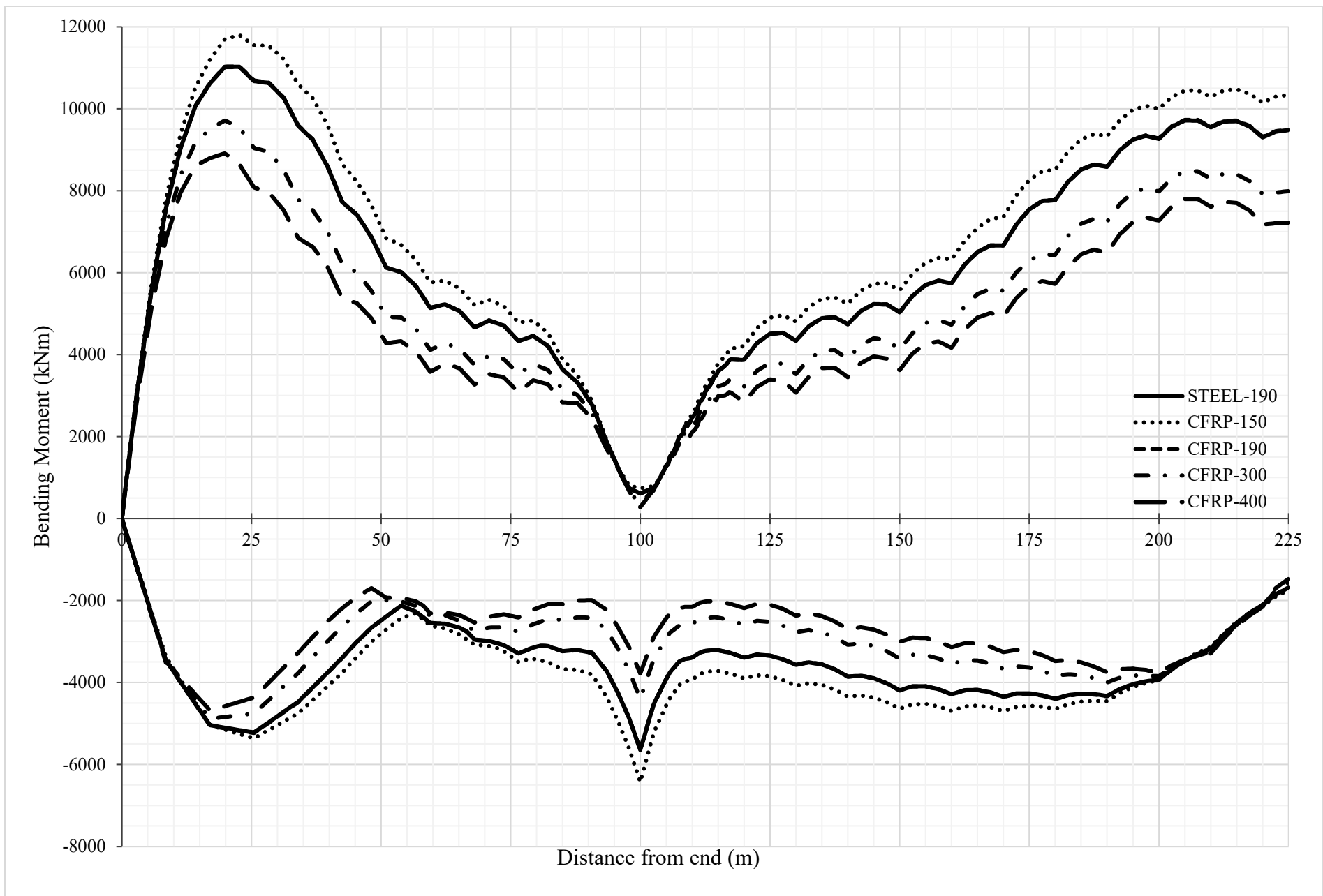


Figure A-14: Minimum and maximum deck bending moment under LL ENV

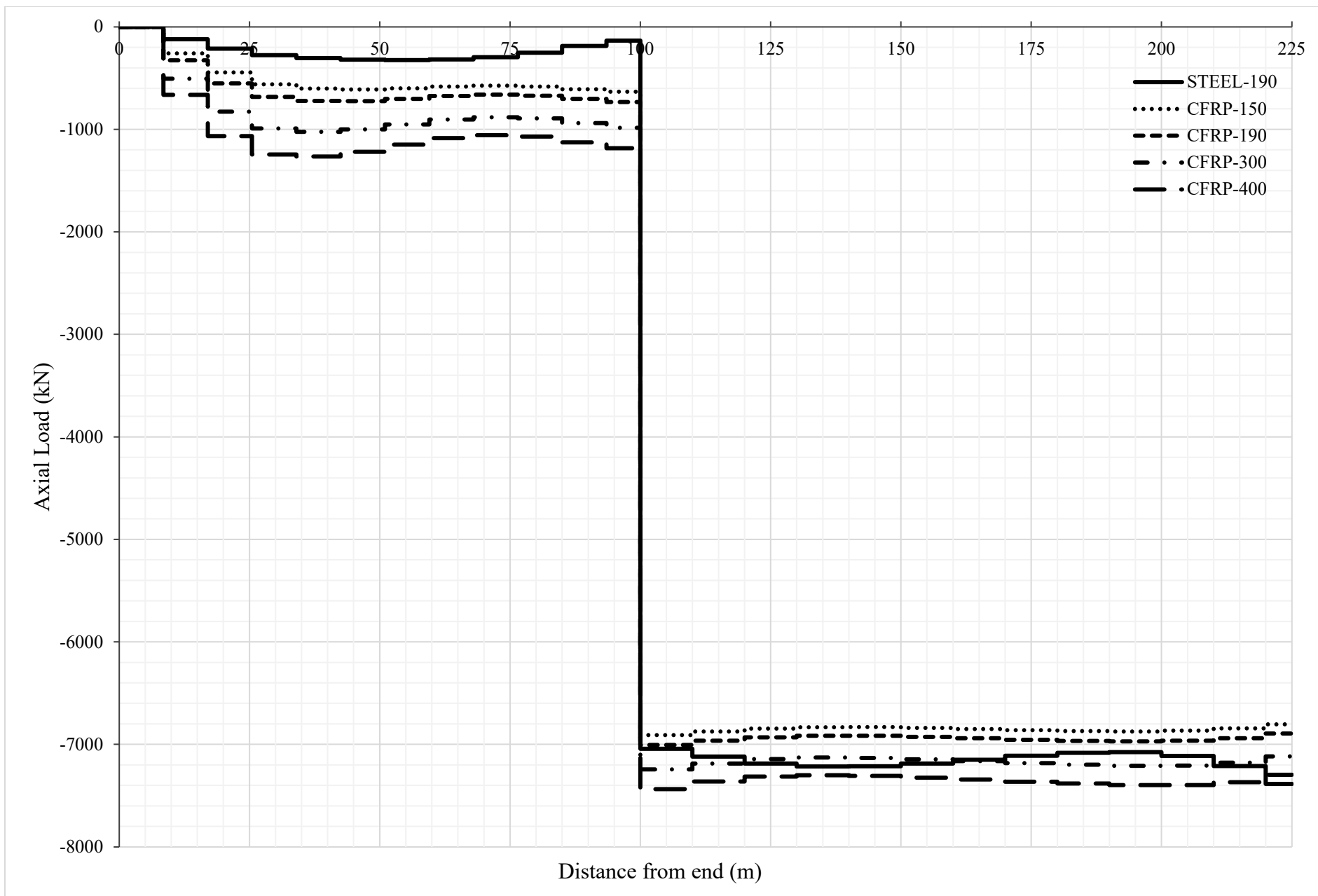


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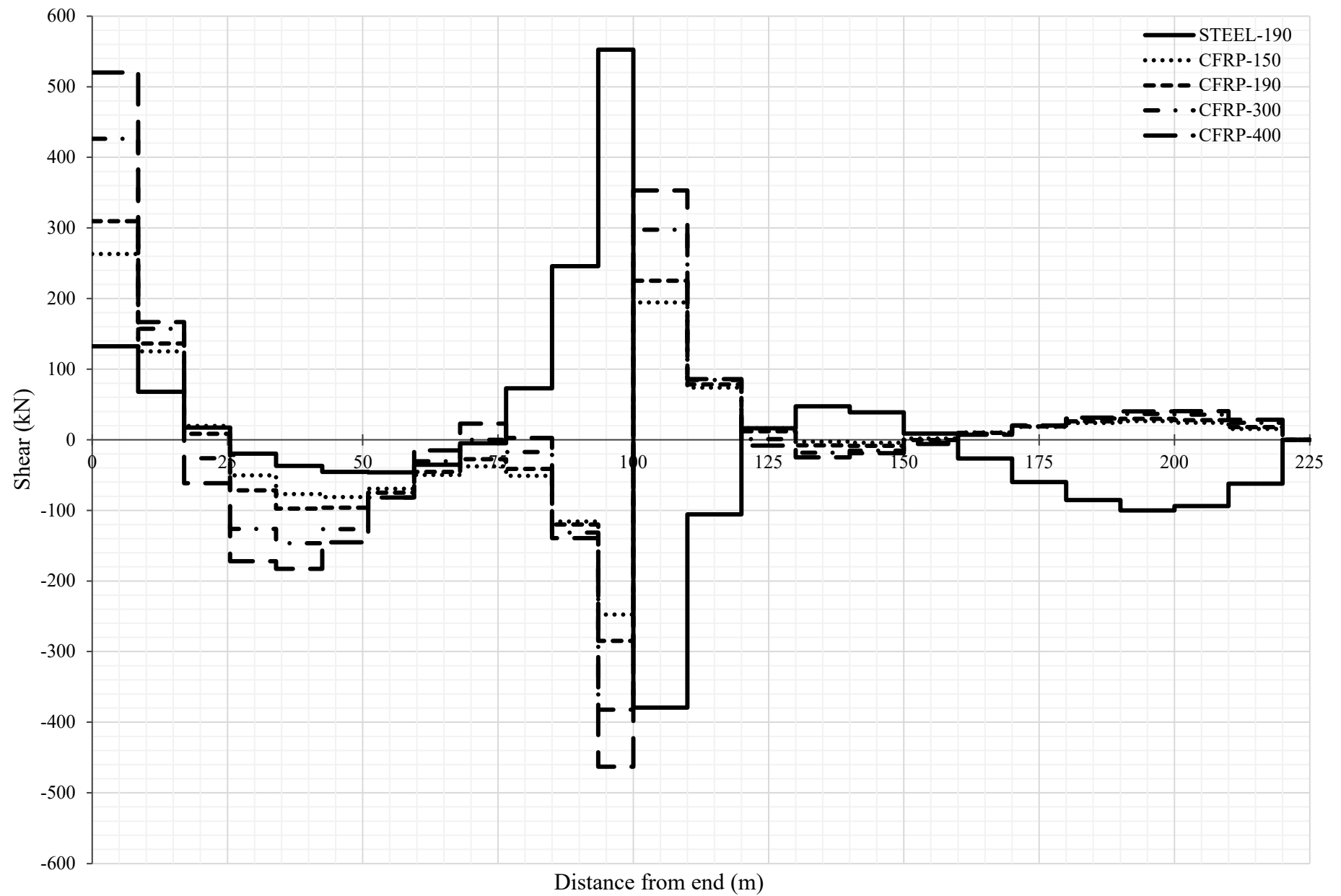


Figure A-16: Deck shear under T+

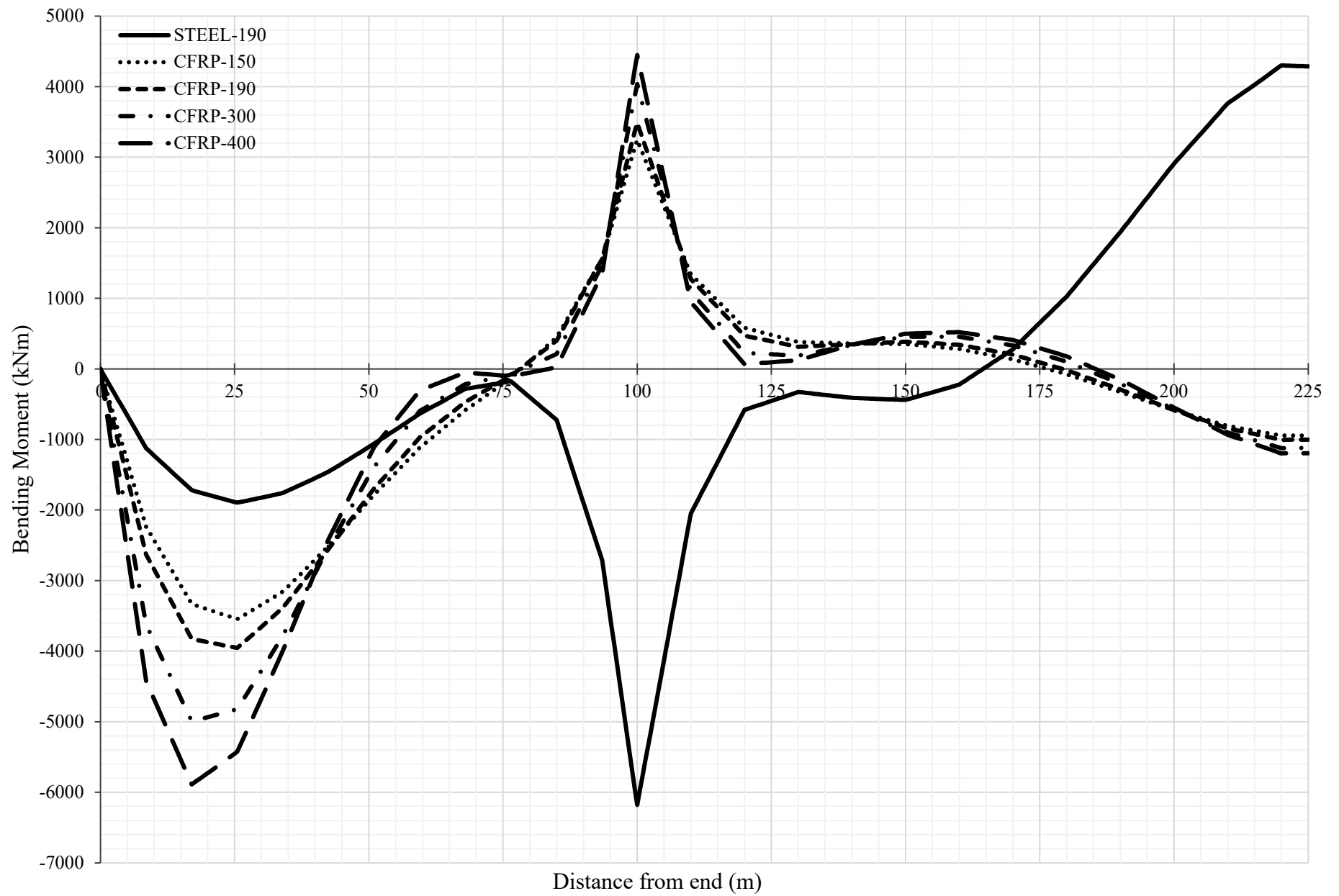


Figure A-17: Deck bending moment under T+

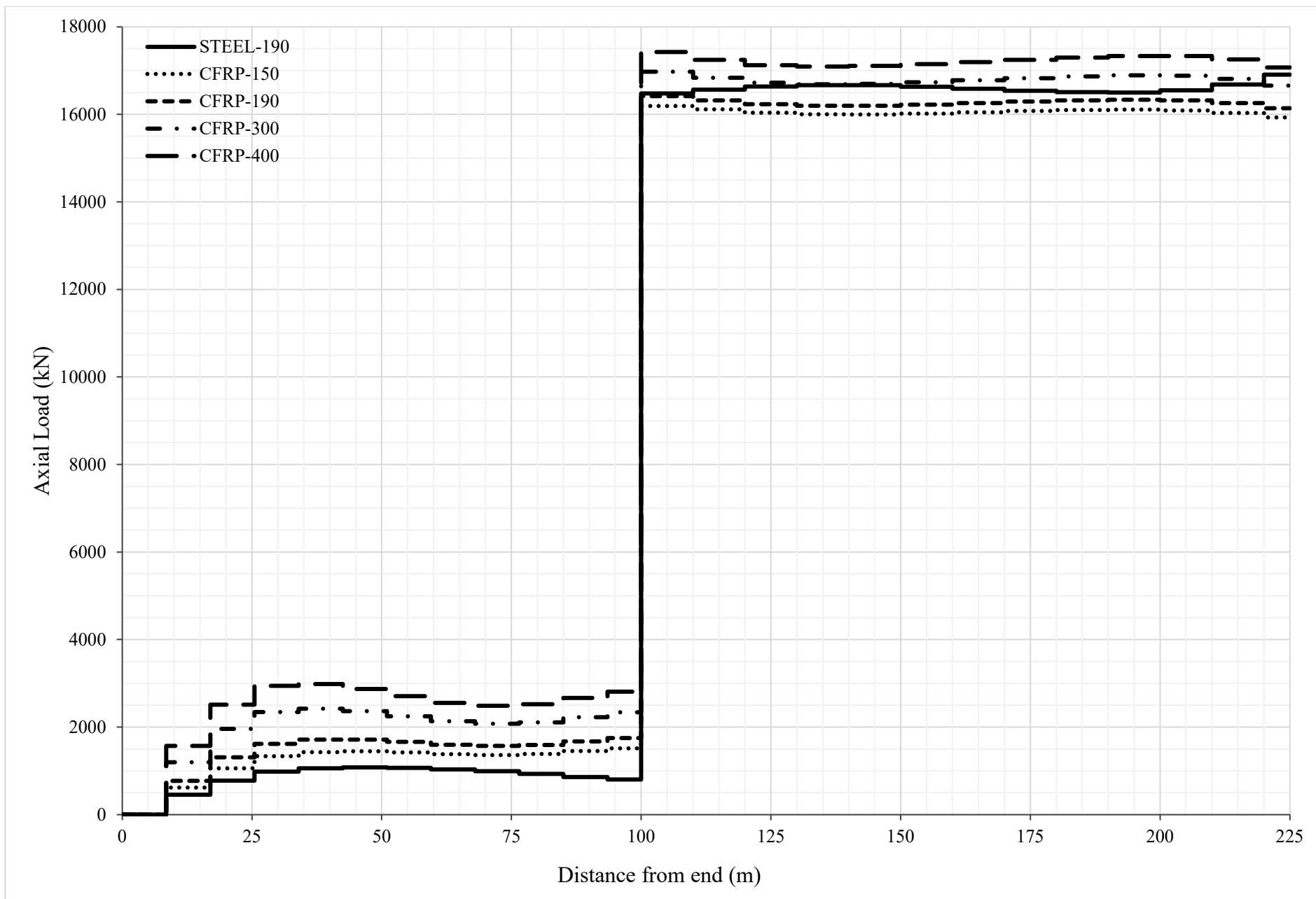


Figure A-18: Deck axial load under T-

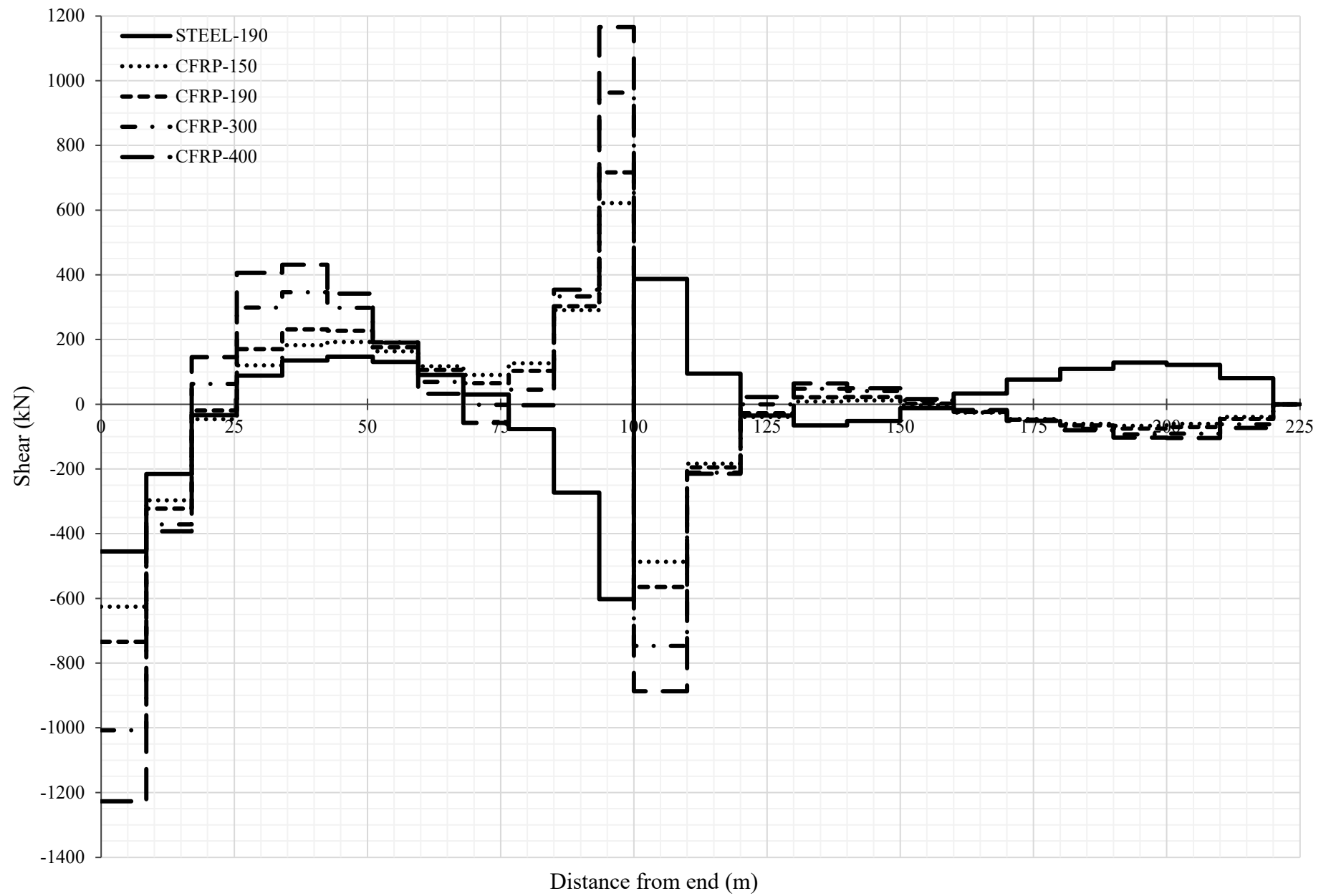


Figure A-19: Deck shear under T-

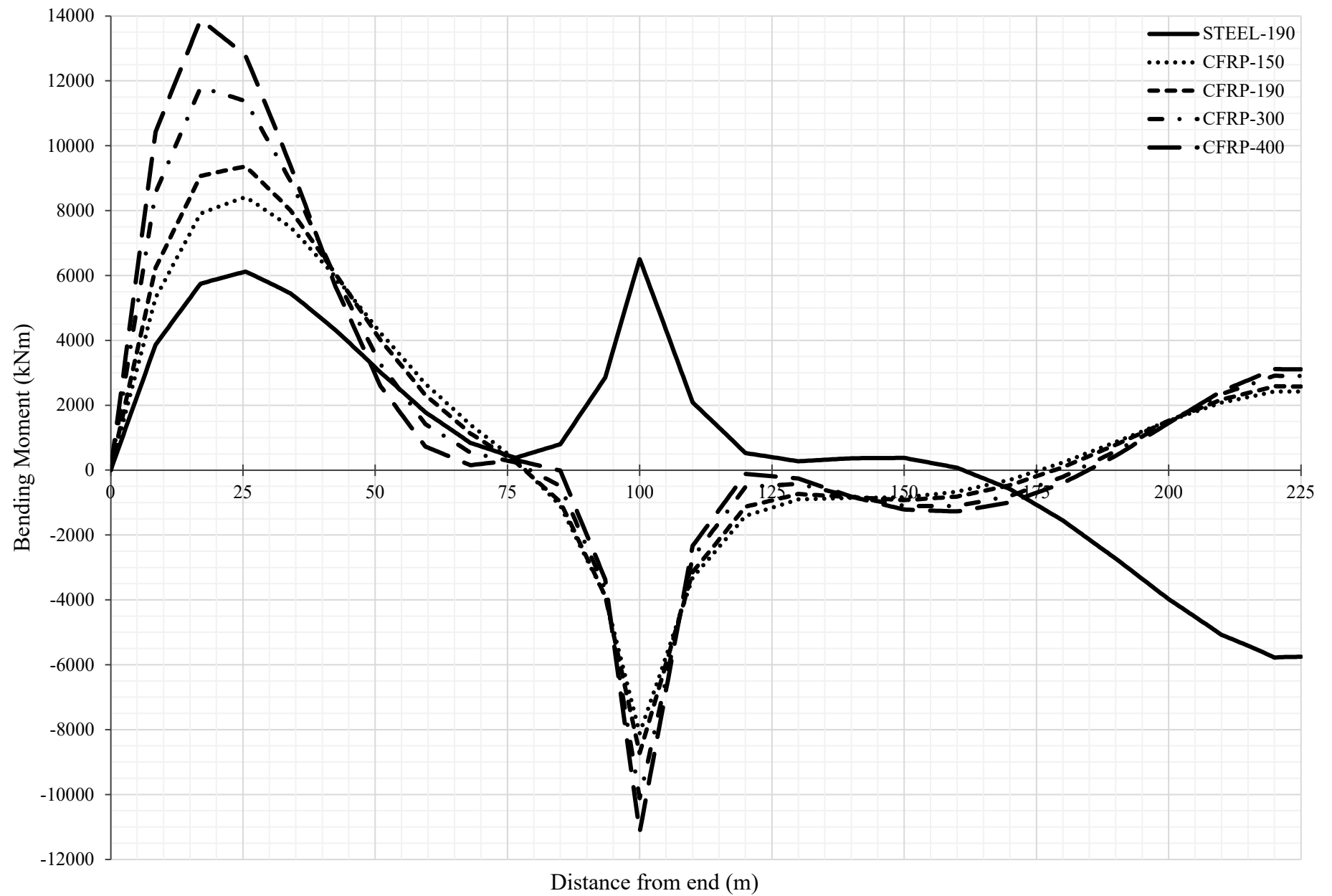


Figure A-20: Deck bending moment under T-

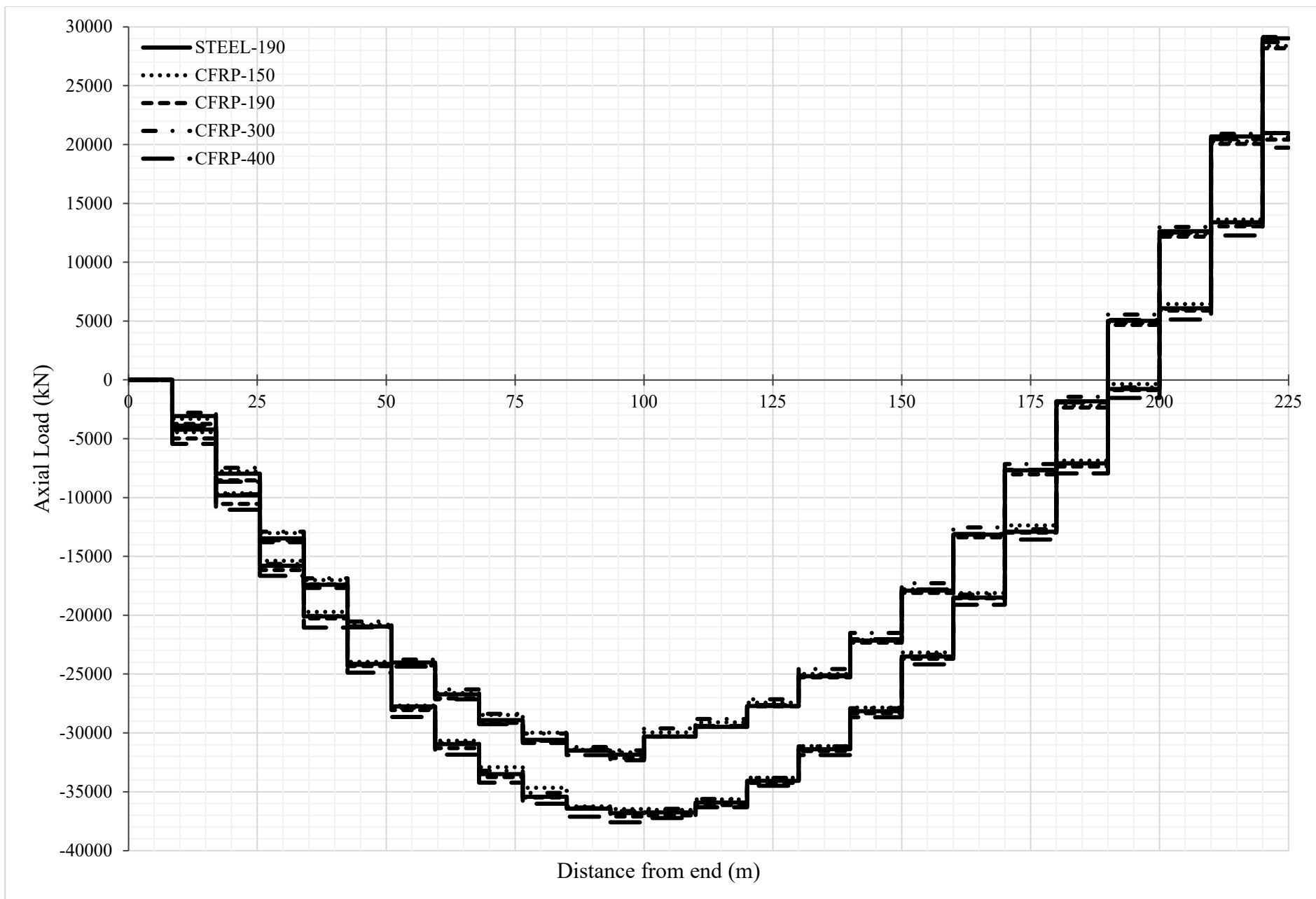


Figure A-21: Minimum and maximum deck axial load under ULS-1

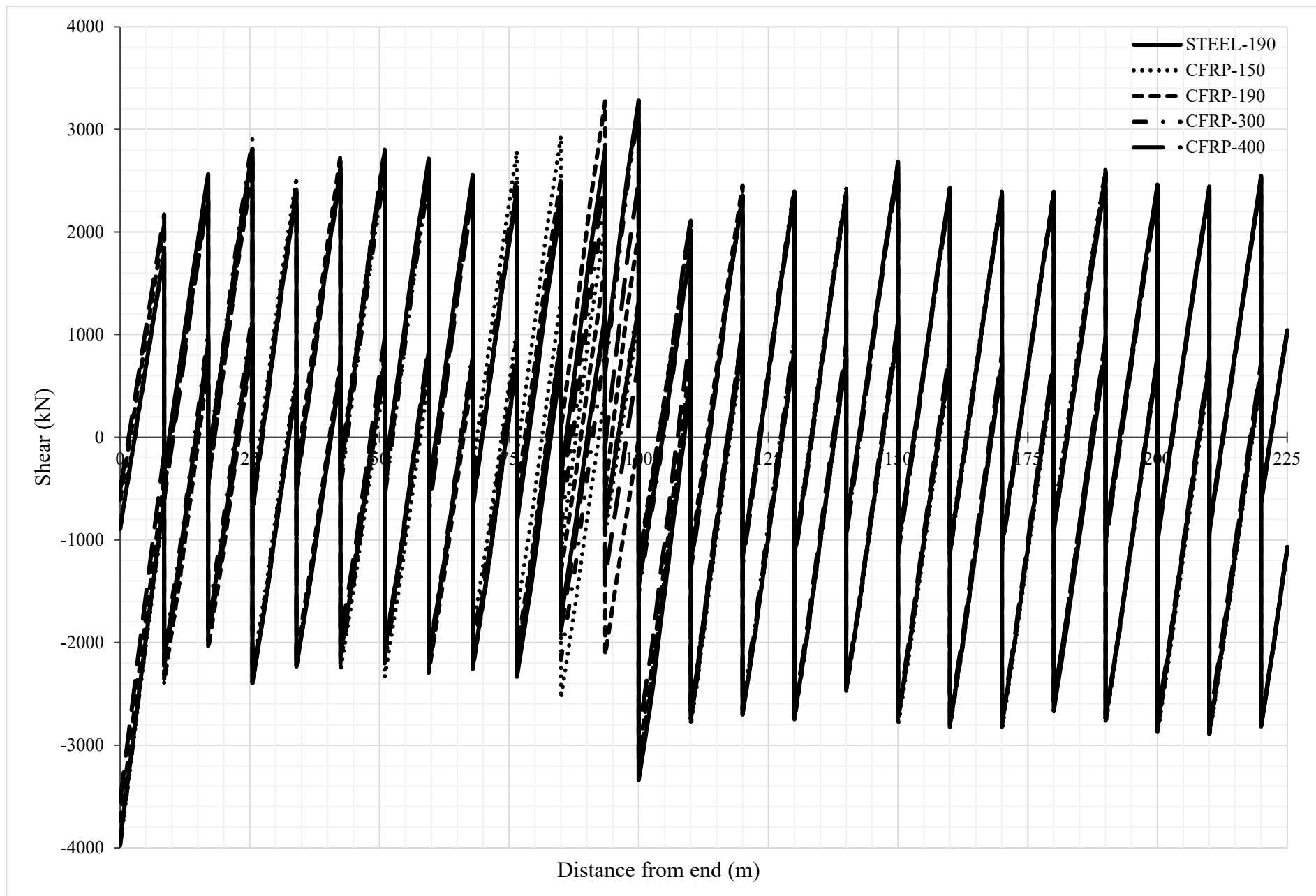


Figure A-22: Minimum and maximum deck shear under ULS-1

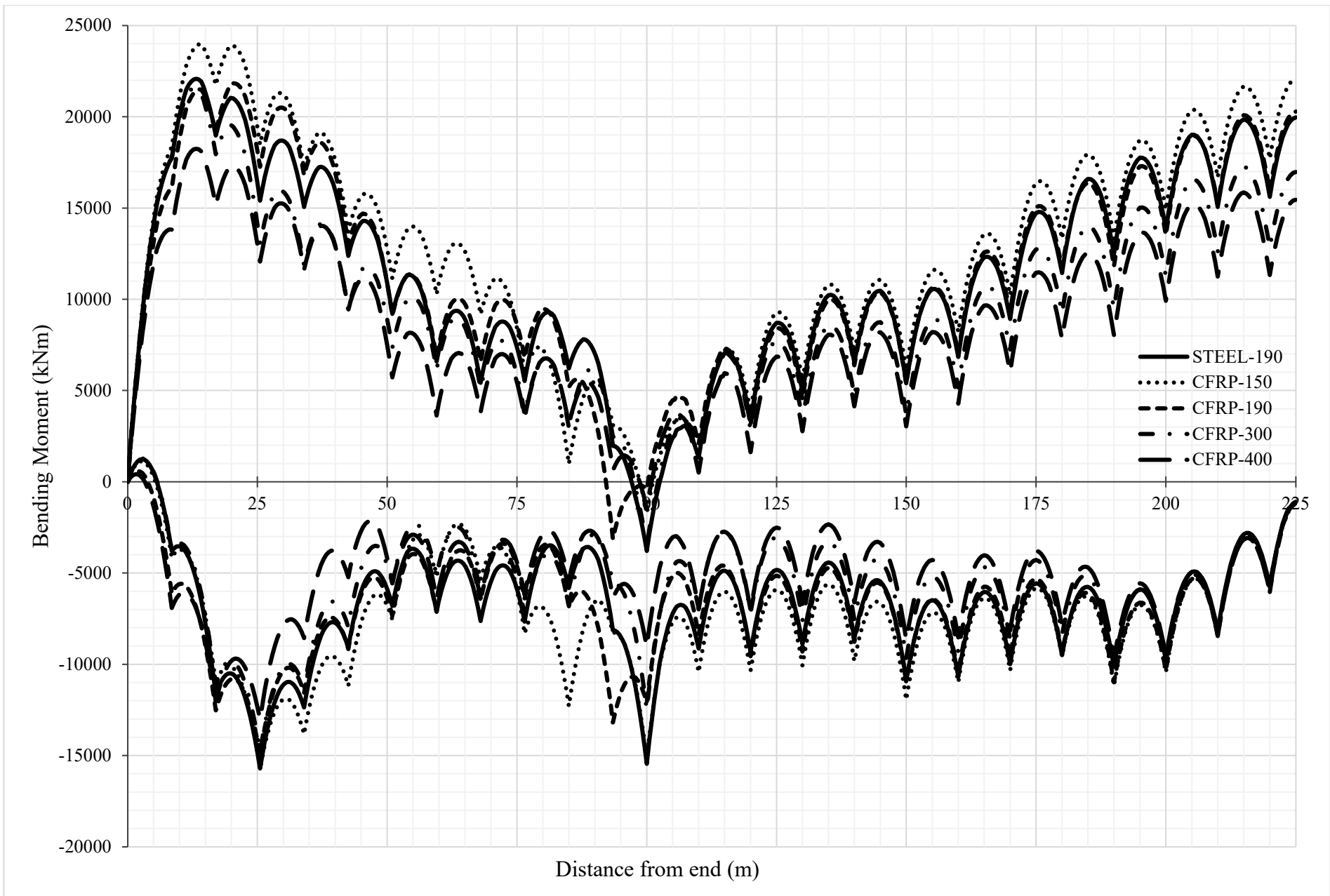


Figure A-23: Minimum and maximum deck bending moment under ULS-1

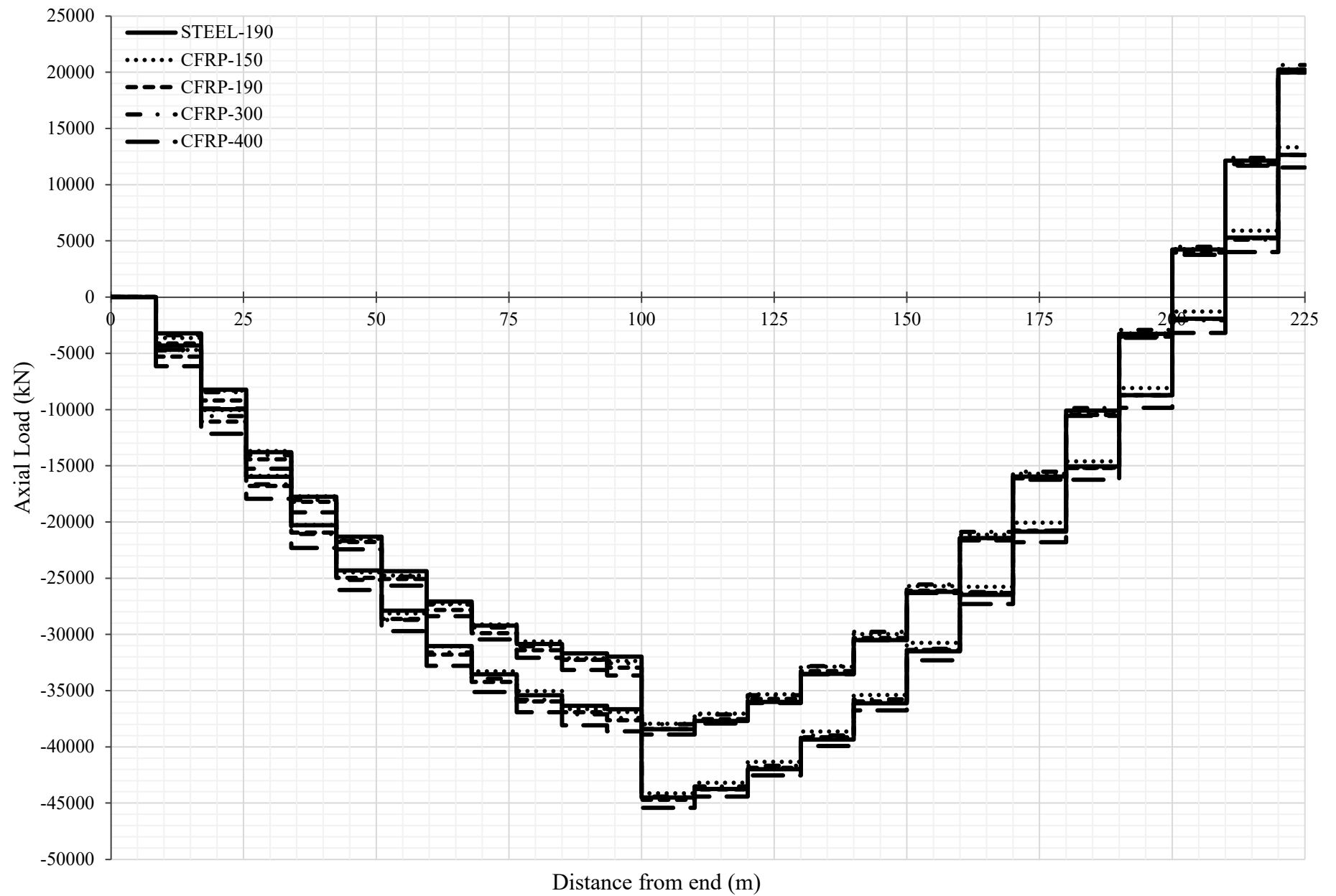


Figure A-24: Minimum and maximum deck axial load under ULS-2-T+



Figure A-25: Minimum and maximum deck shear under ULS-2-T+

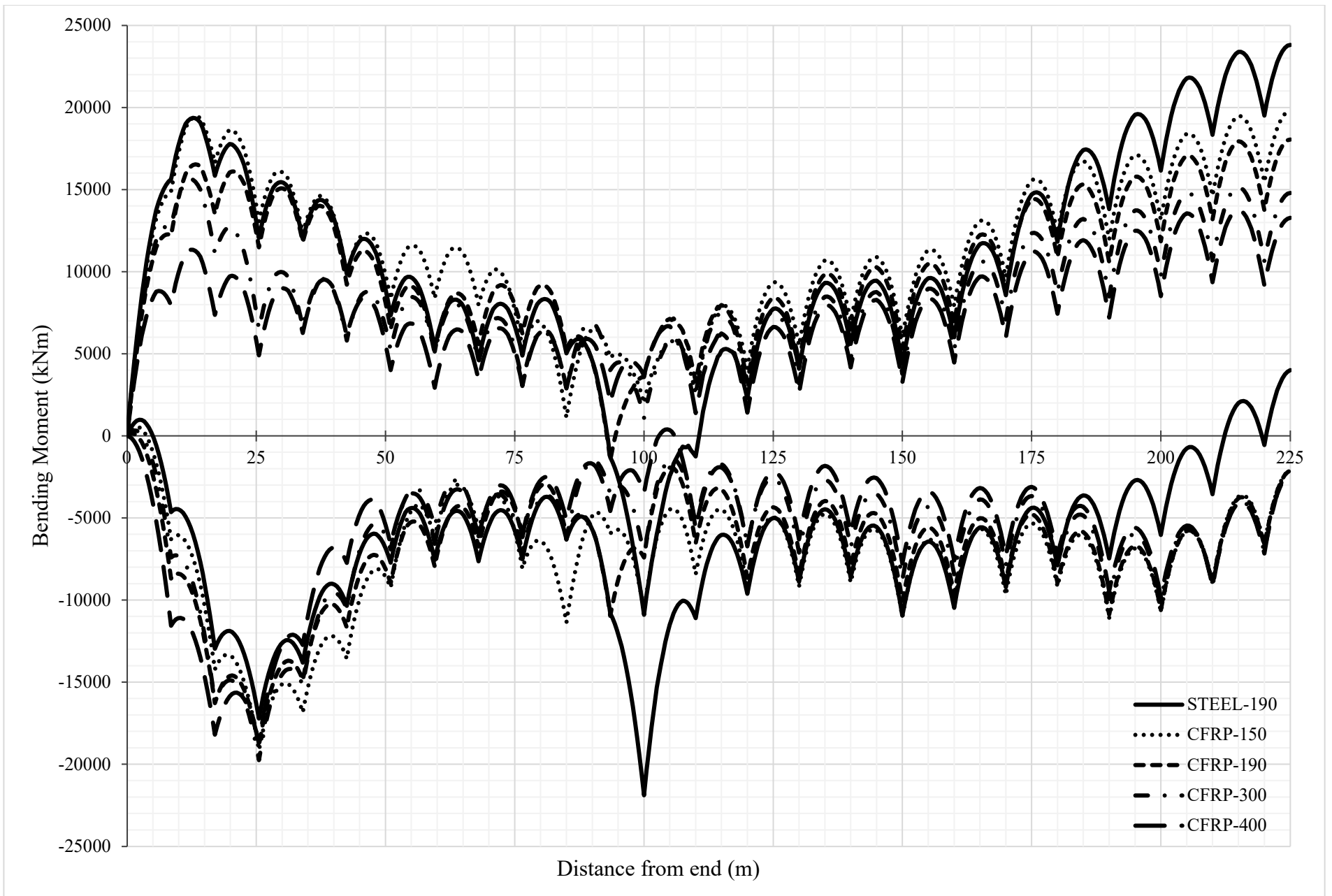


Figure A-26: Minimum and maximum deck bending moment under ULS-2-T+

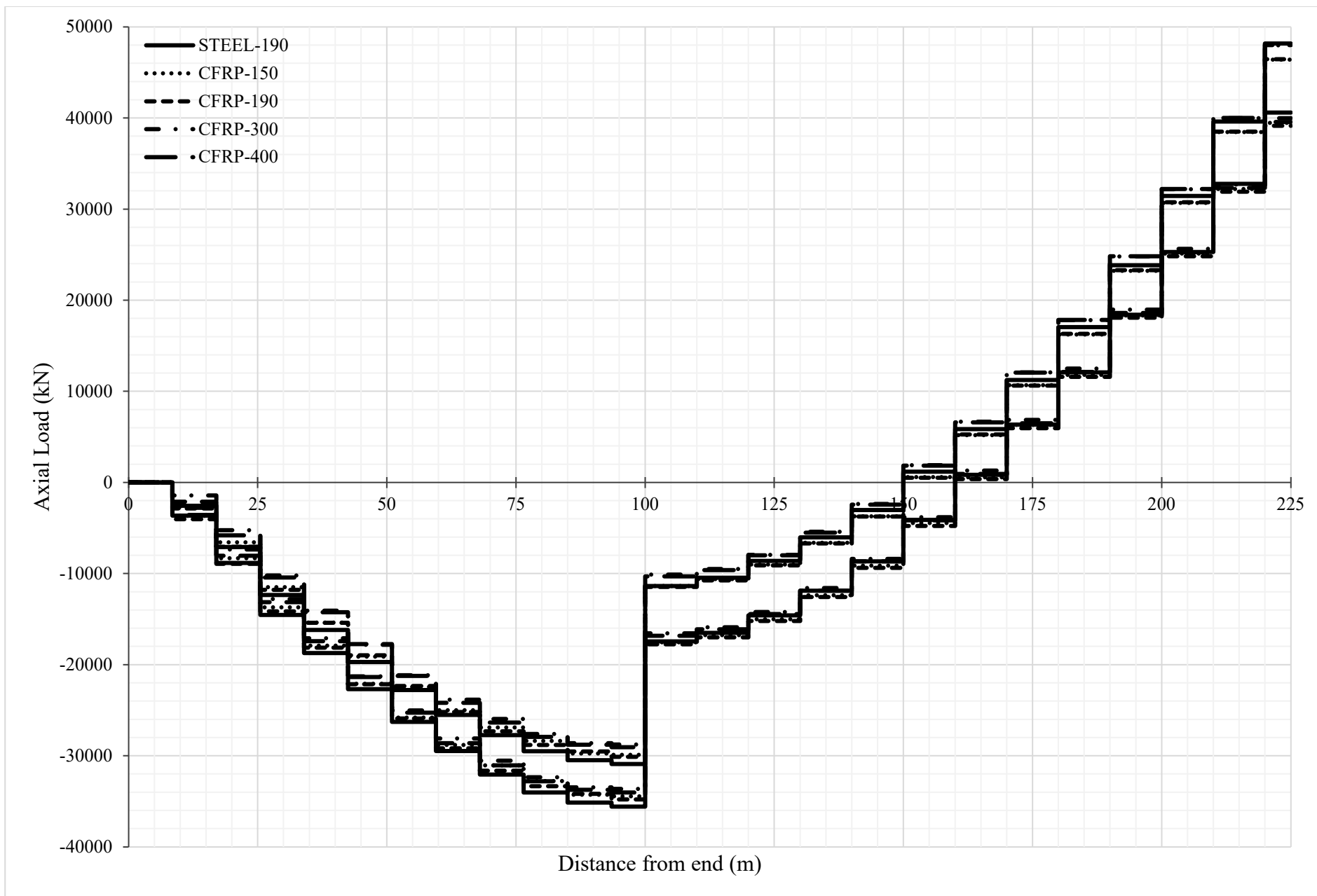


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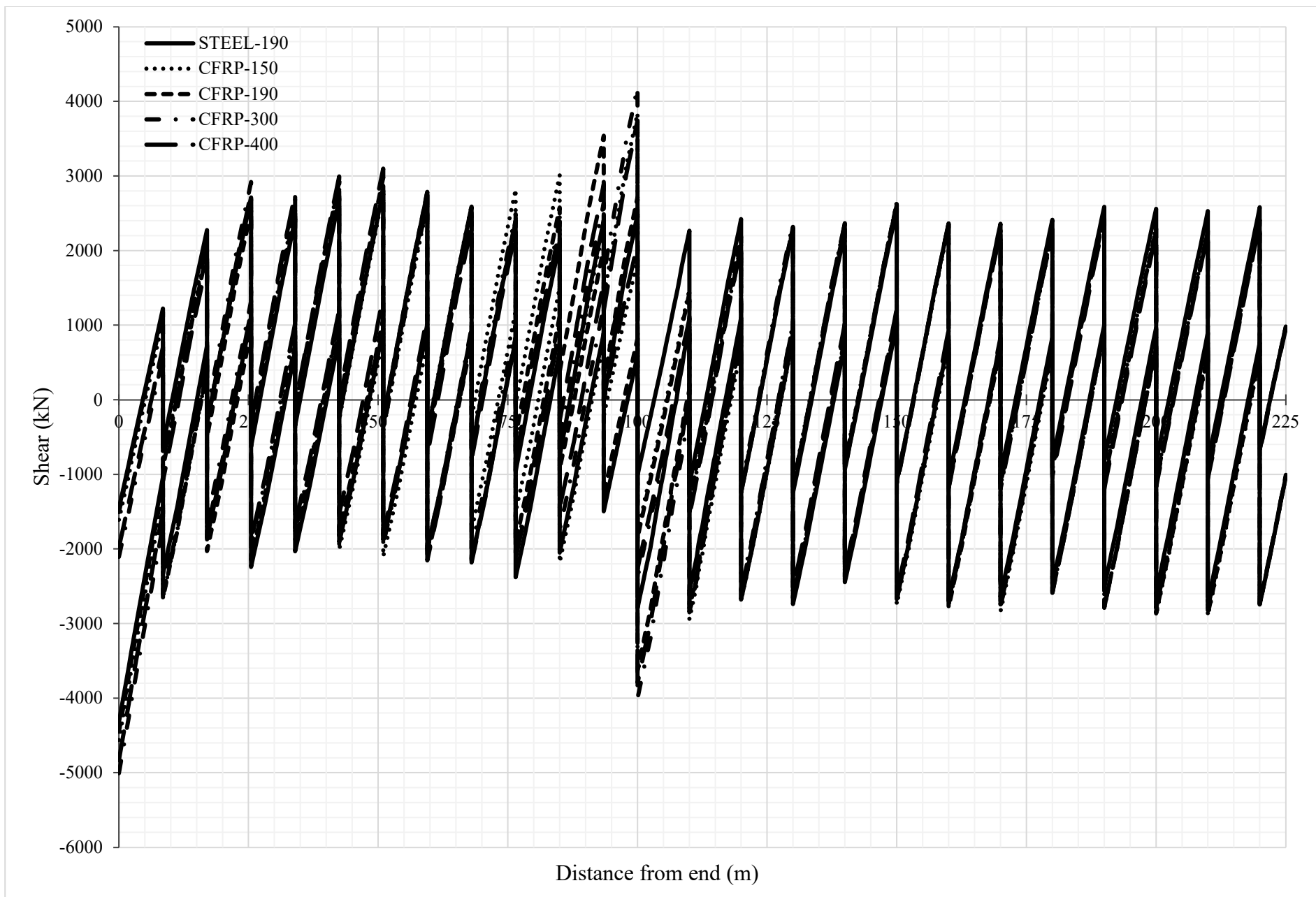


Figure A-28: Minimum and maximum deck shear under ULS-2-T-

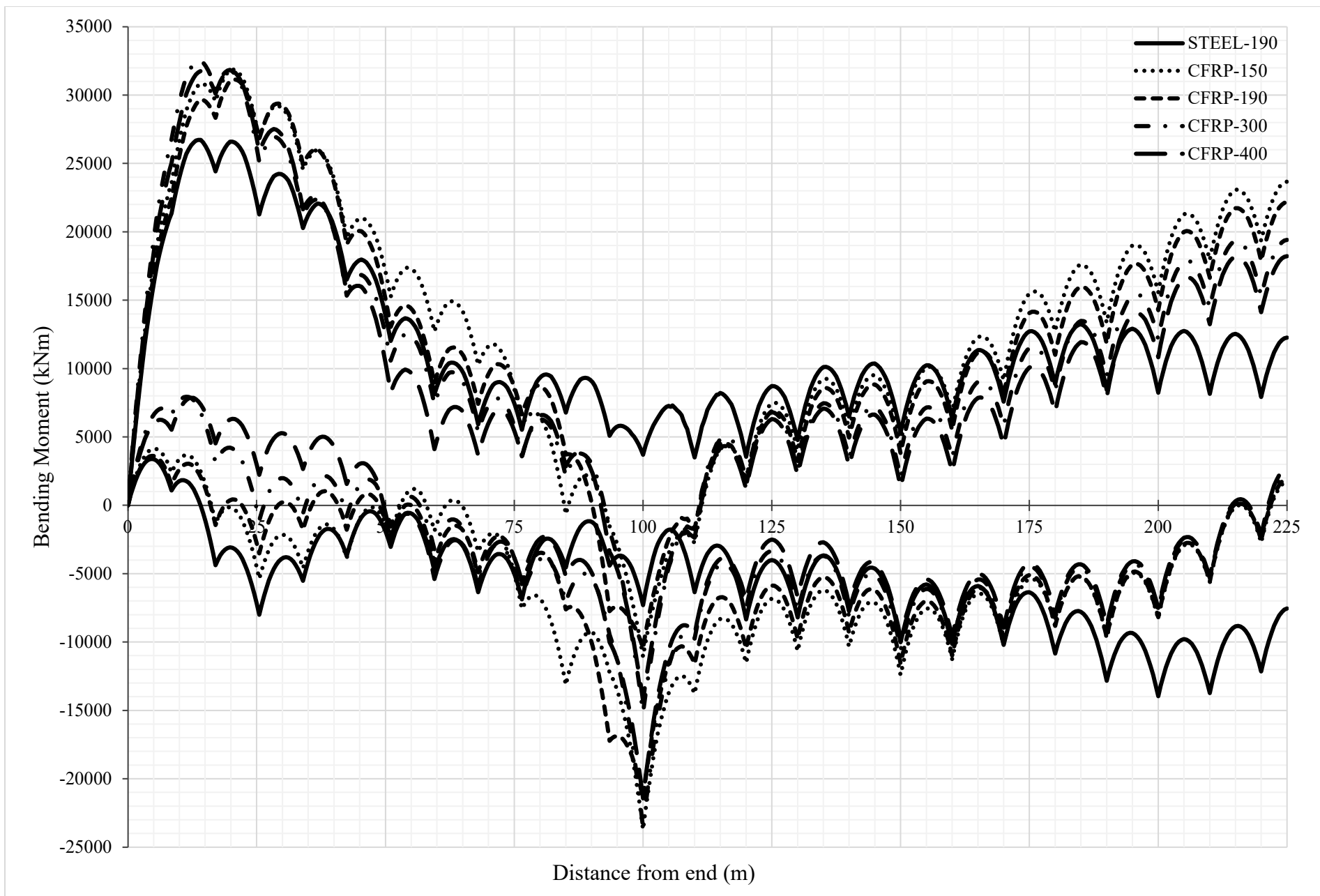


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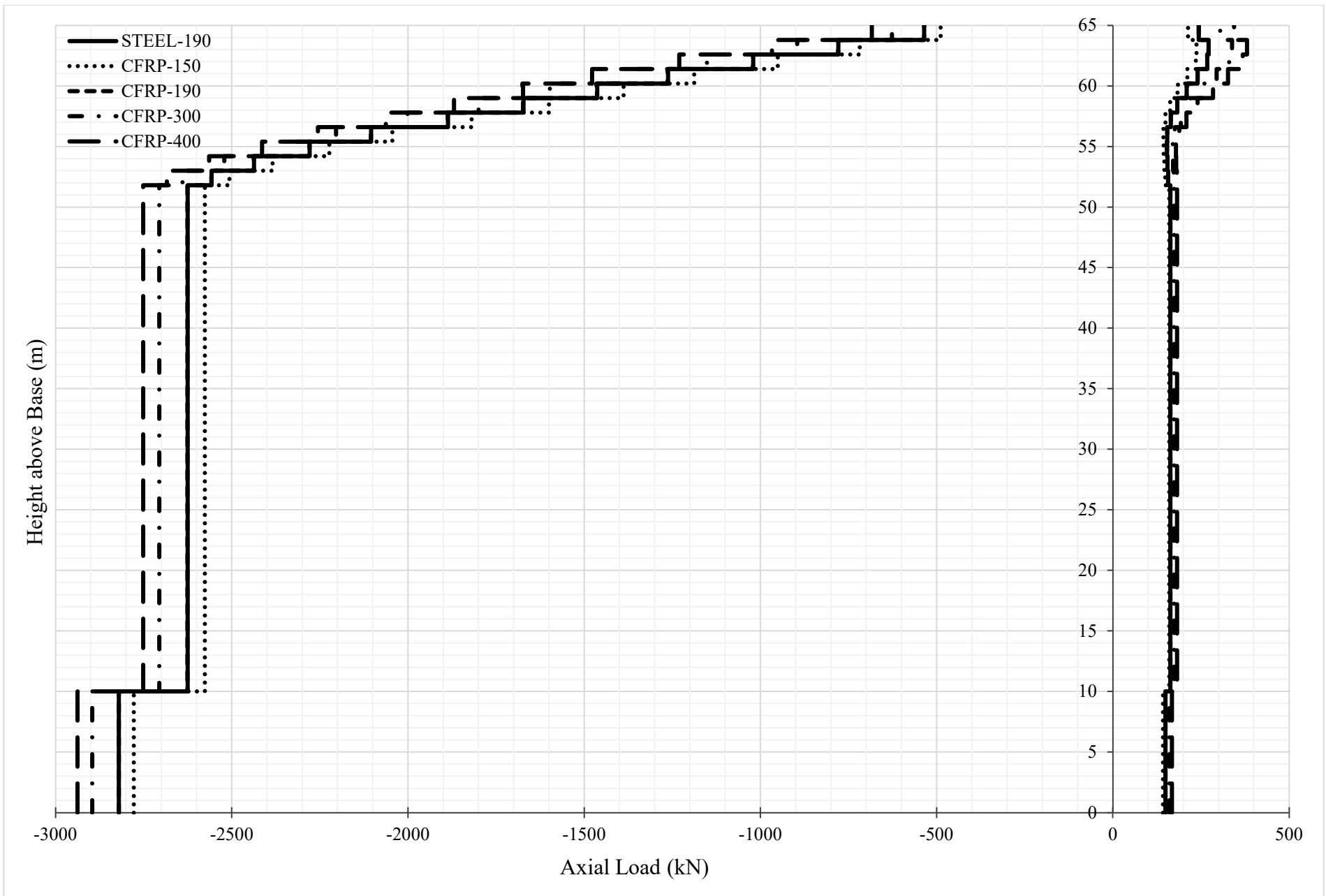


Figure A-30: Minimum and maximum pylon leg axial load under LL ENV

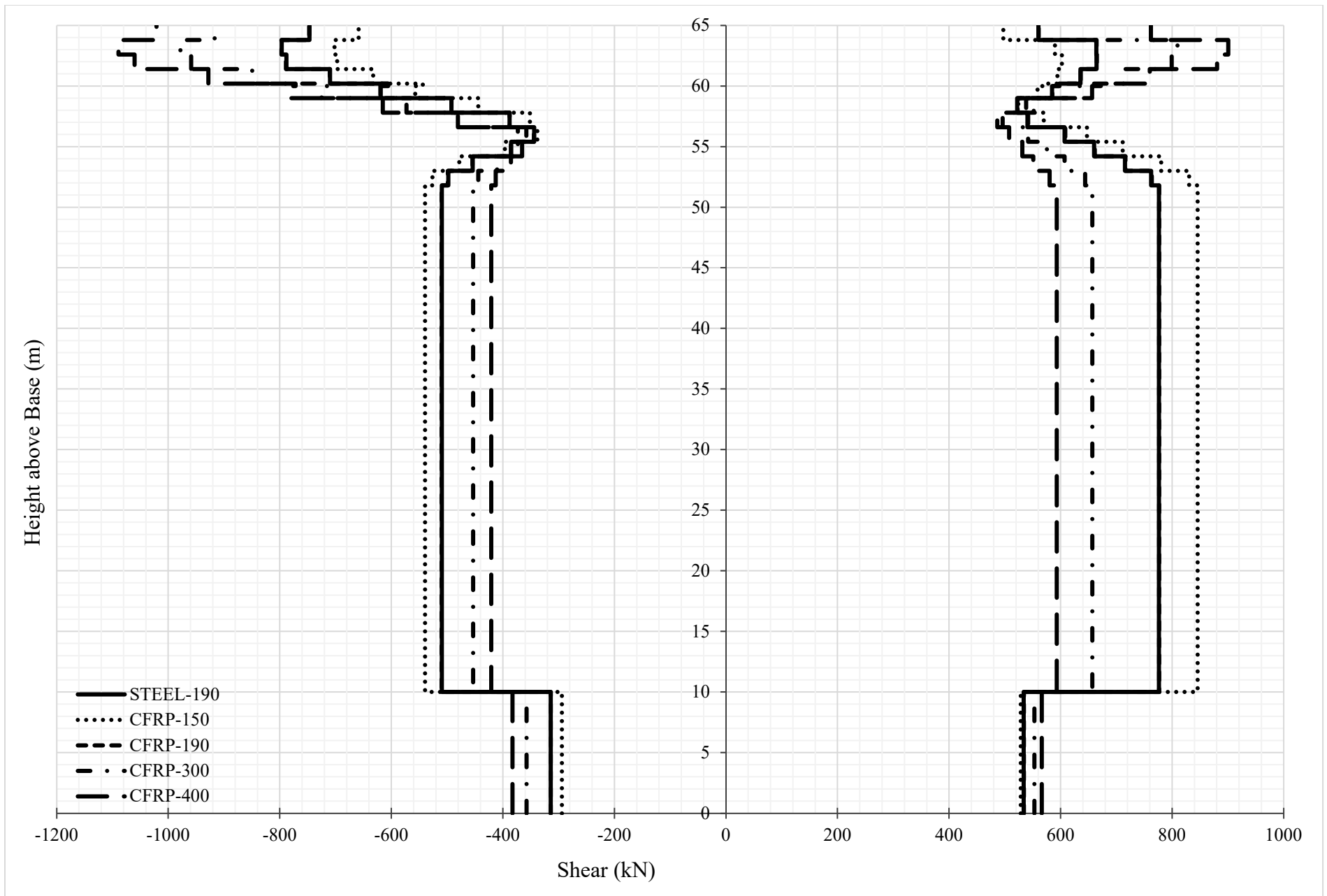


Figure A-31: Minimum and maximum pylon leg shear under LL ENV

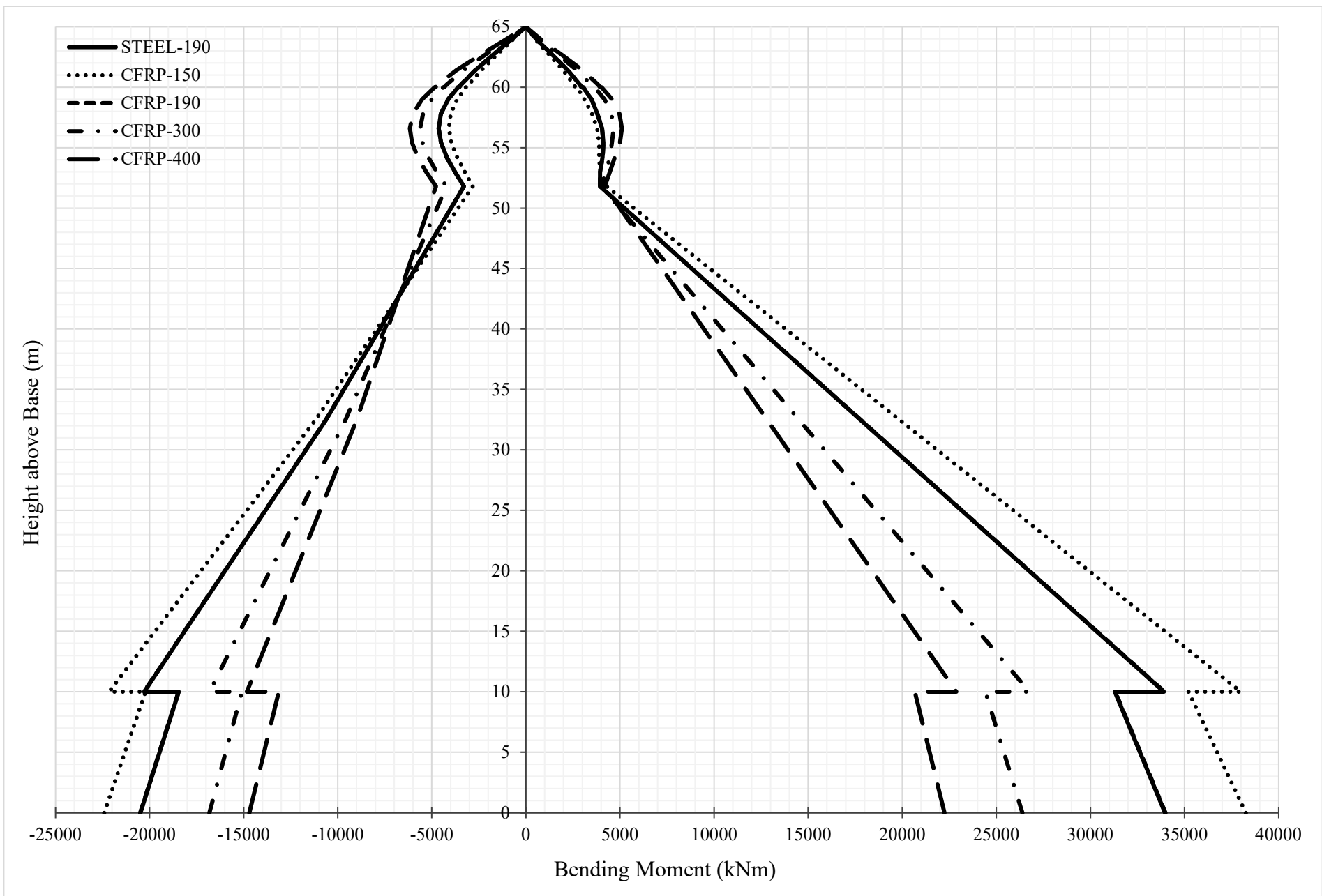


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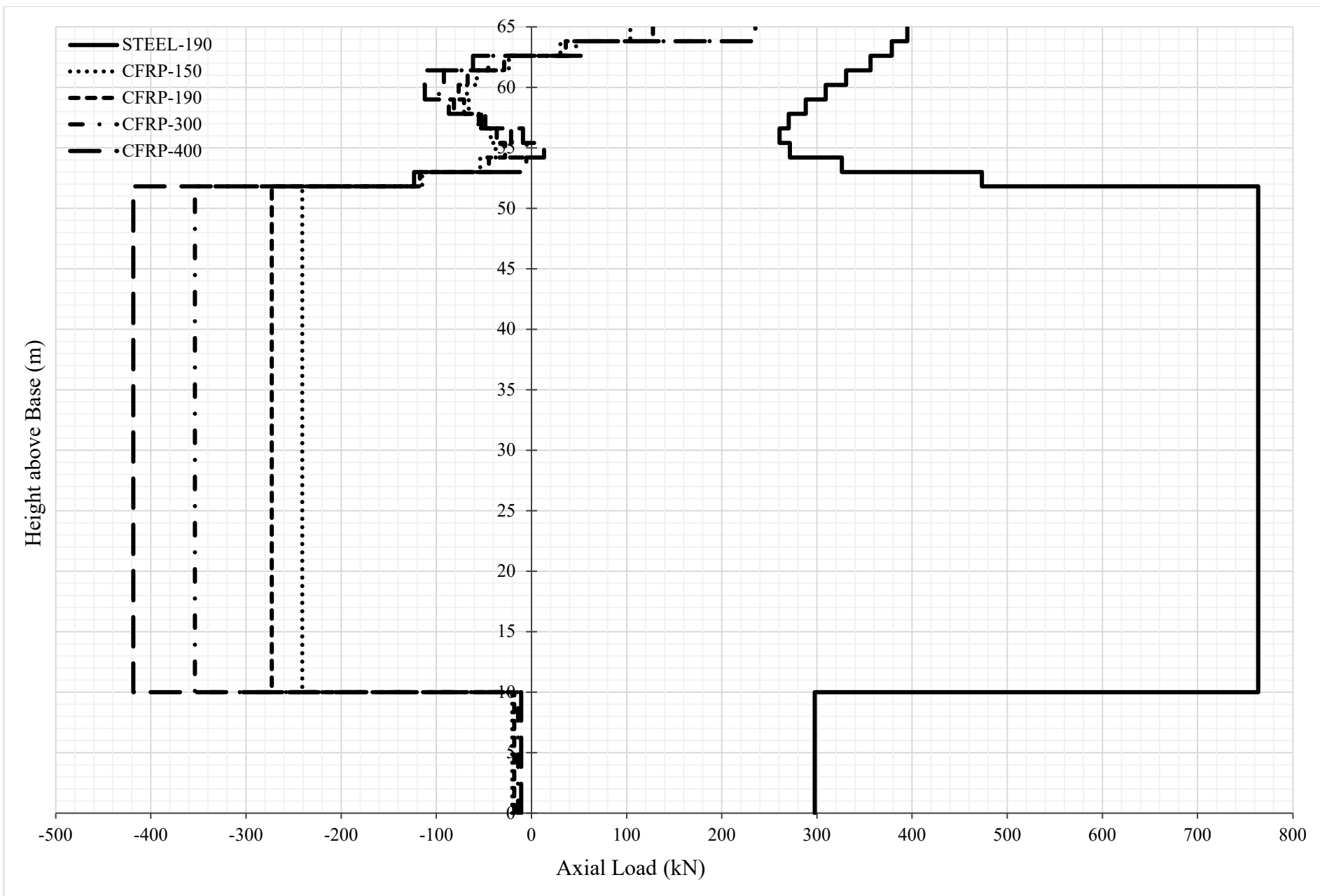


Figure A-33: Pylon leg axial load under T+

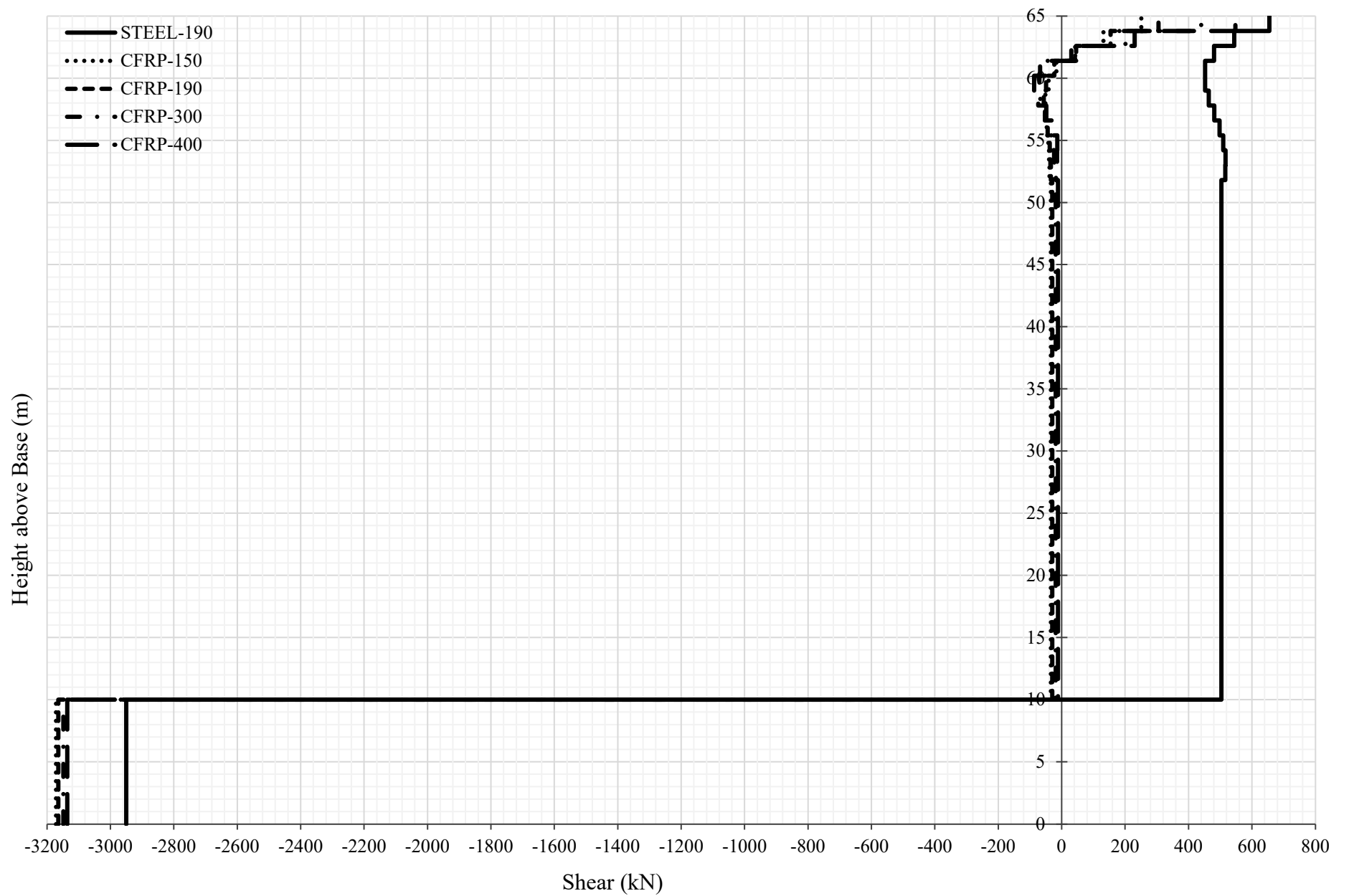


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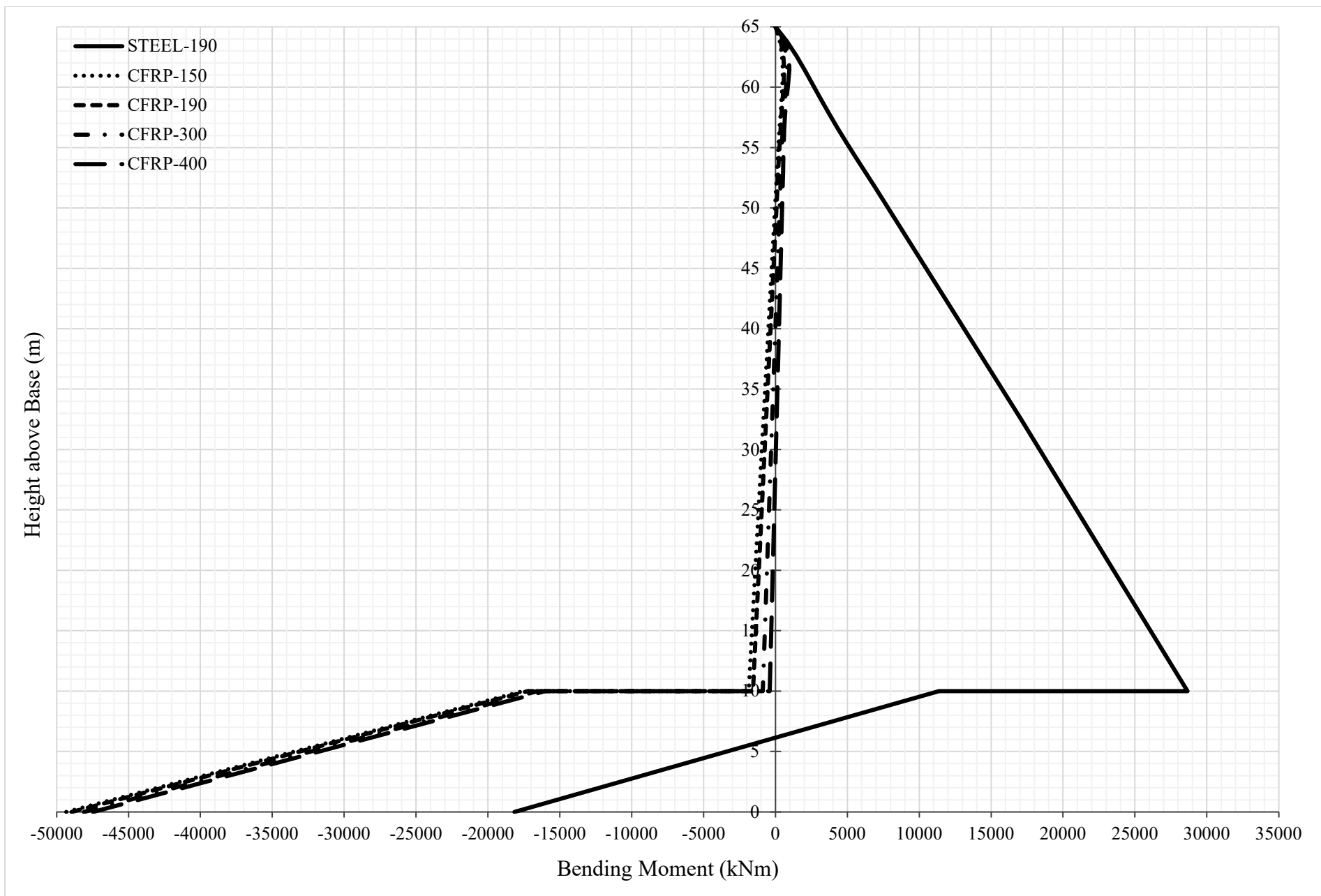


Figure A-35: Pylon leg bending moment under T+

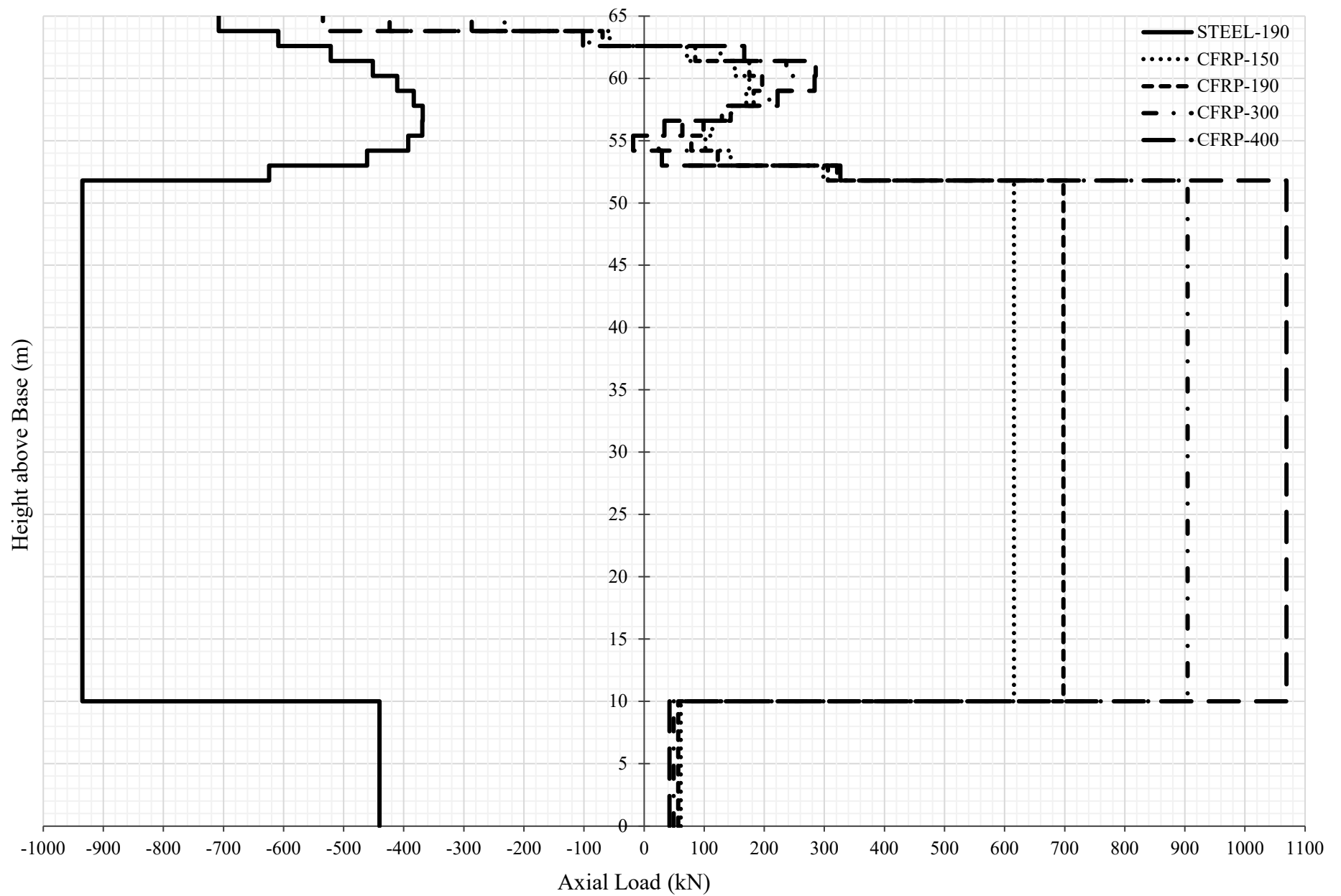


Figure A-36: Pylon leg axial load under T-

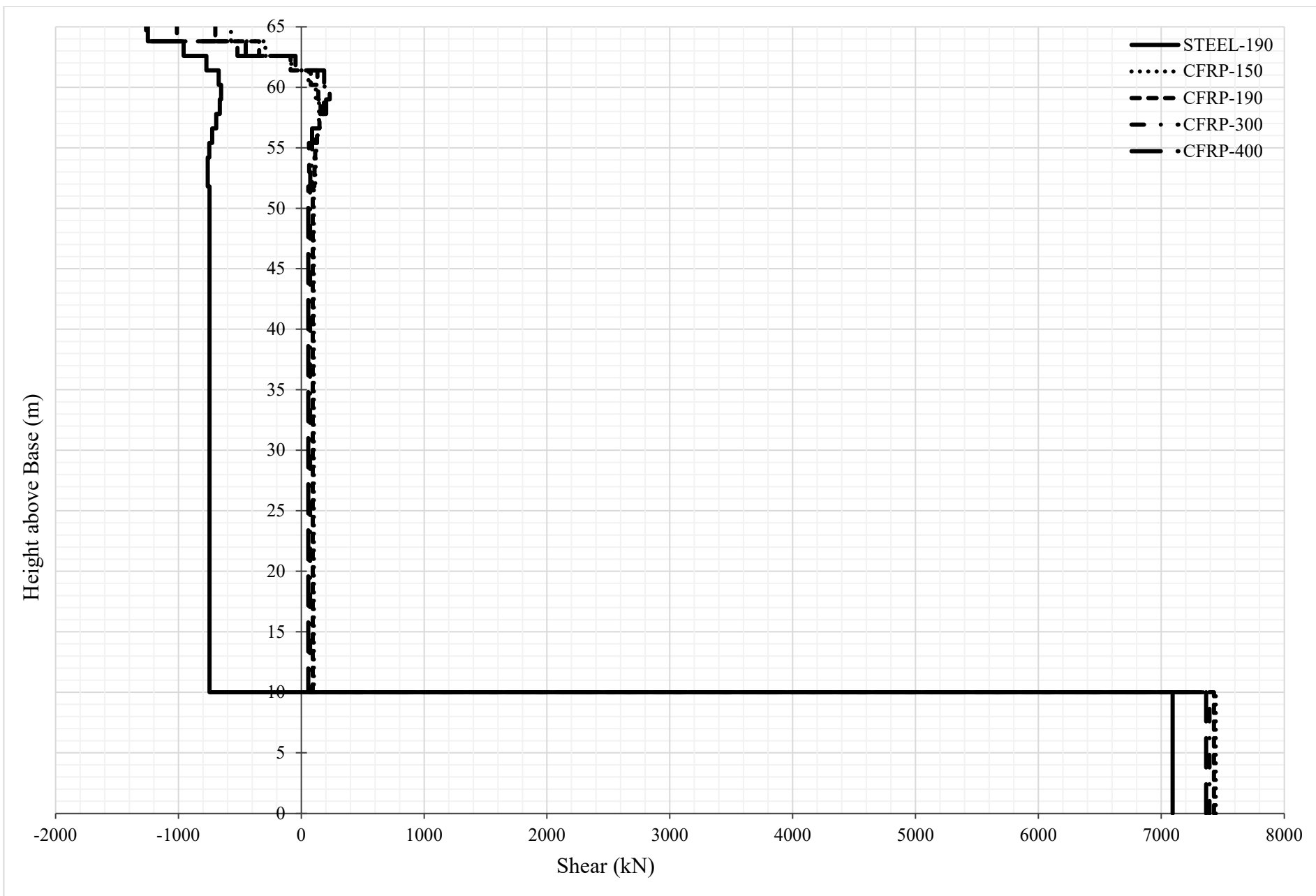


Figure A-37: Pylon leg shear under T-

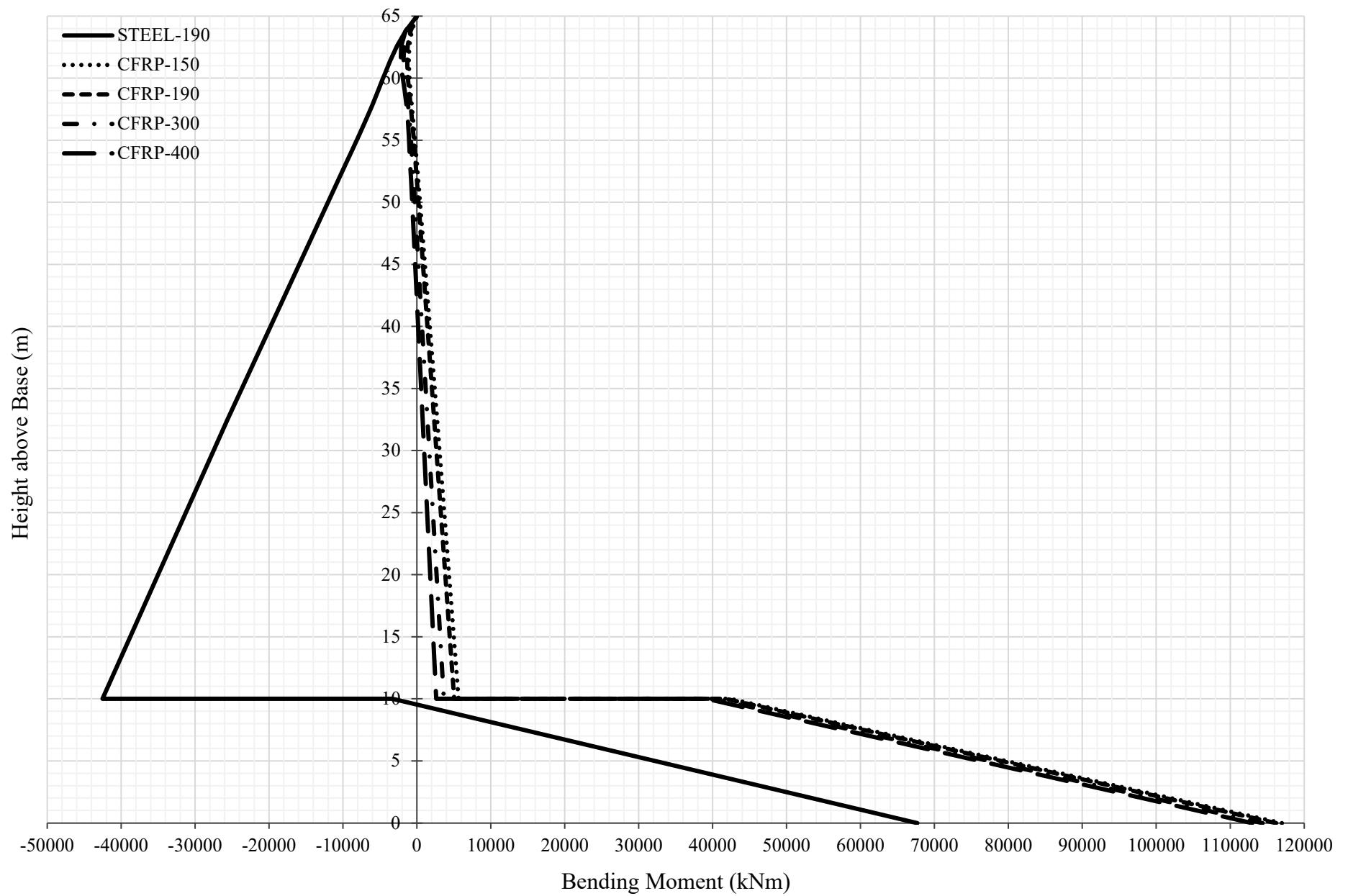


Figure A-38: Pylon leg bending moment under T-

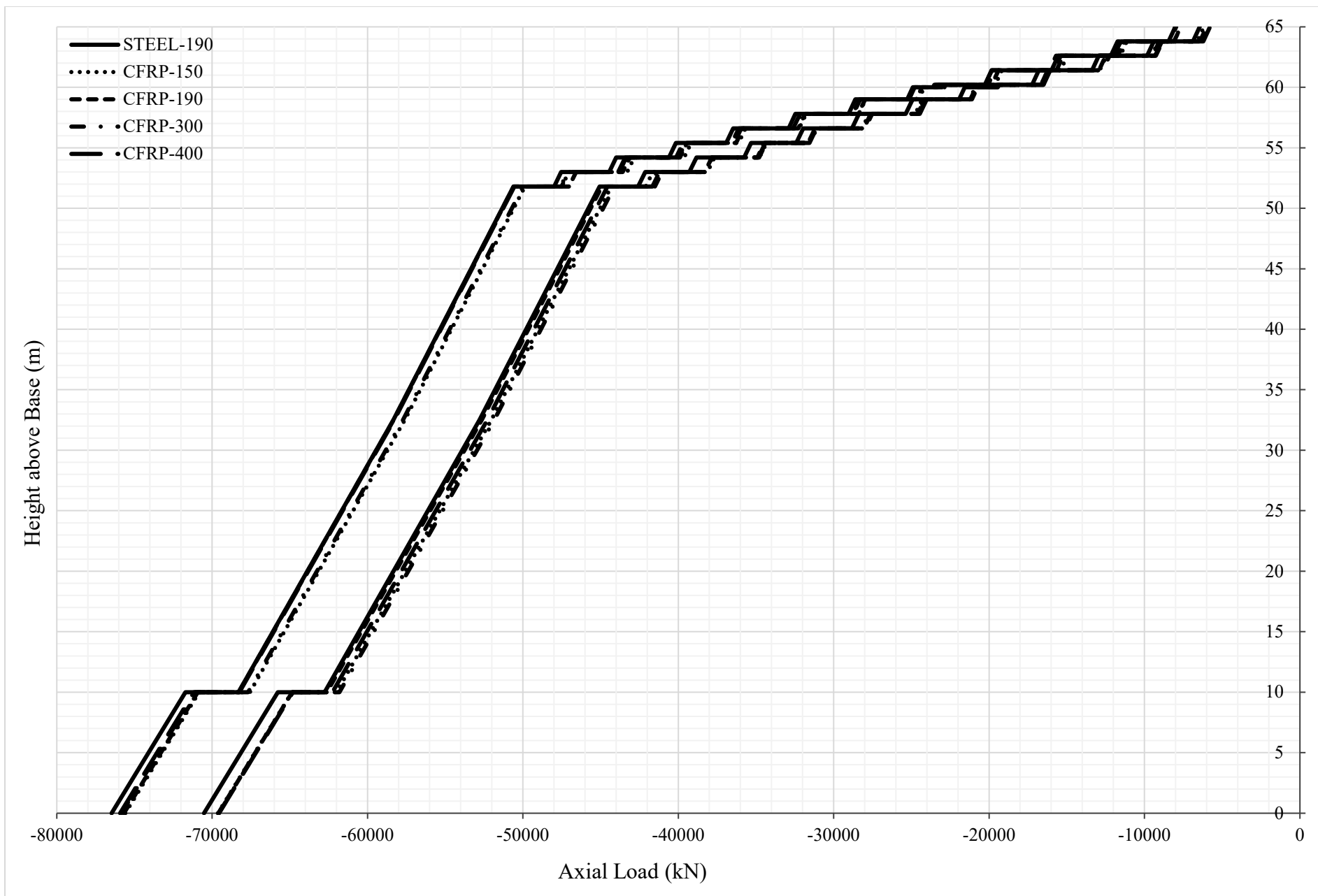


Figure A-39: Minimum and maximum pylon leg axial load under ULS-1

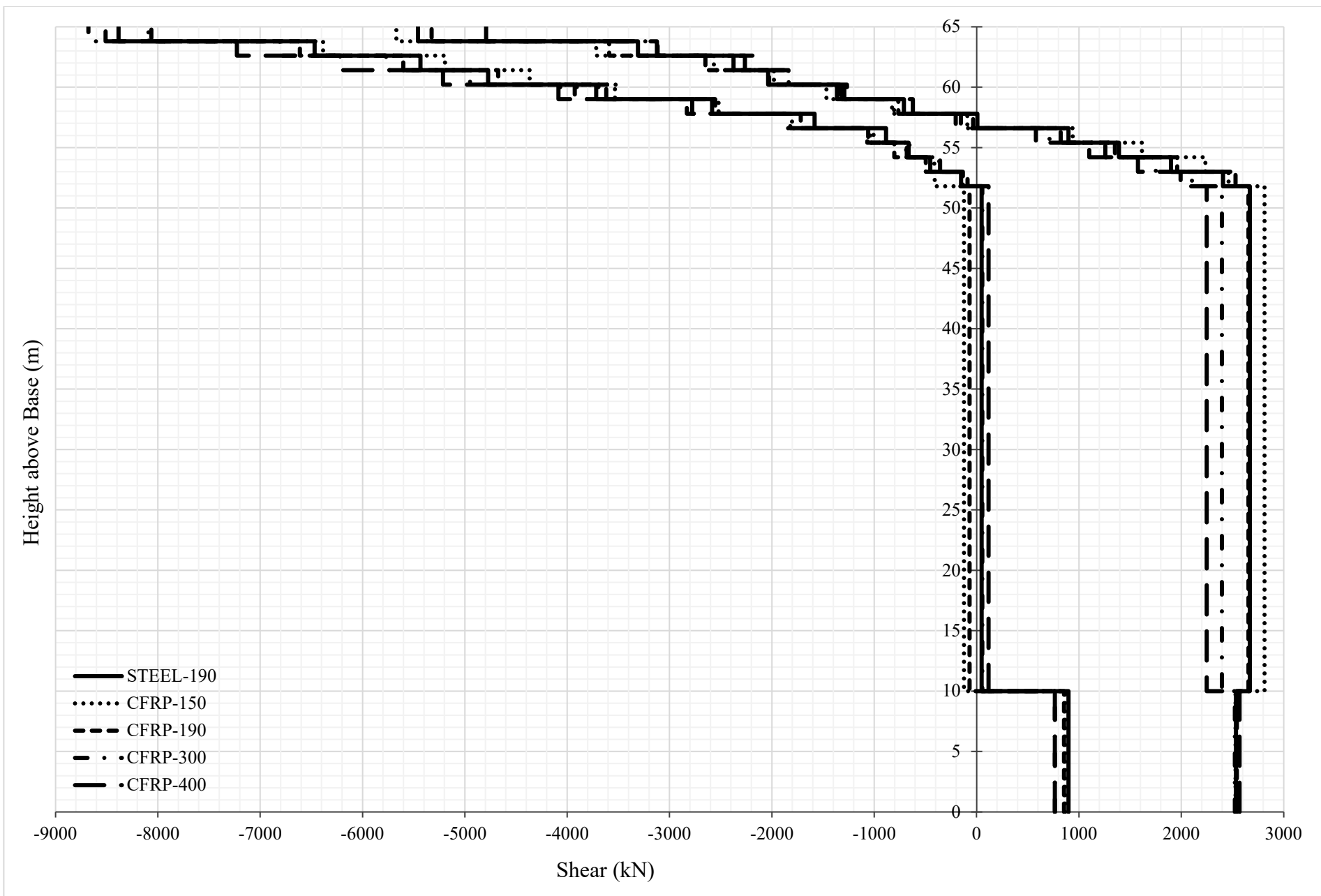


Figure A-40: Minimum and maximum pylon leg shear under ULS-1

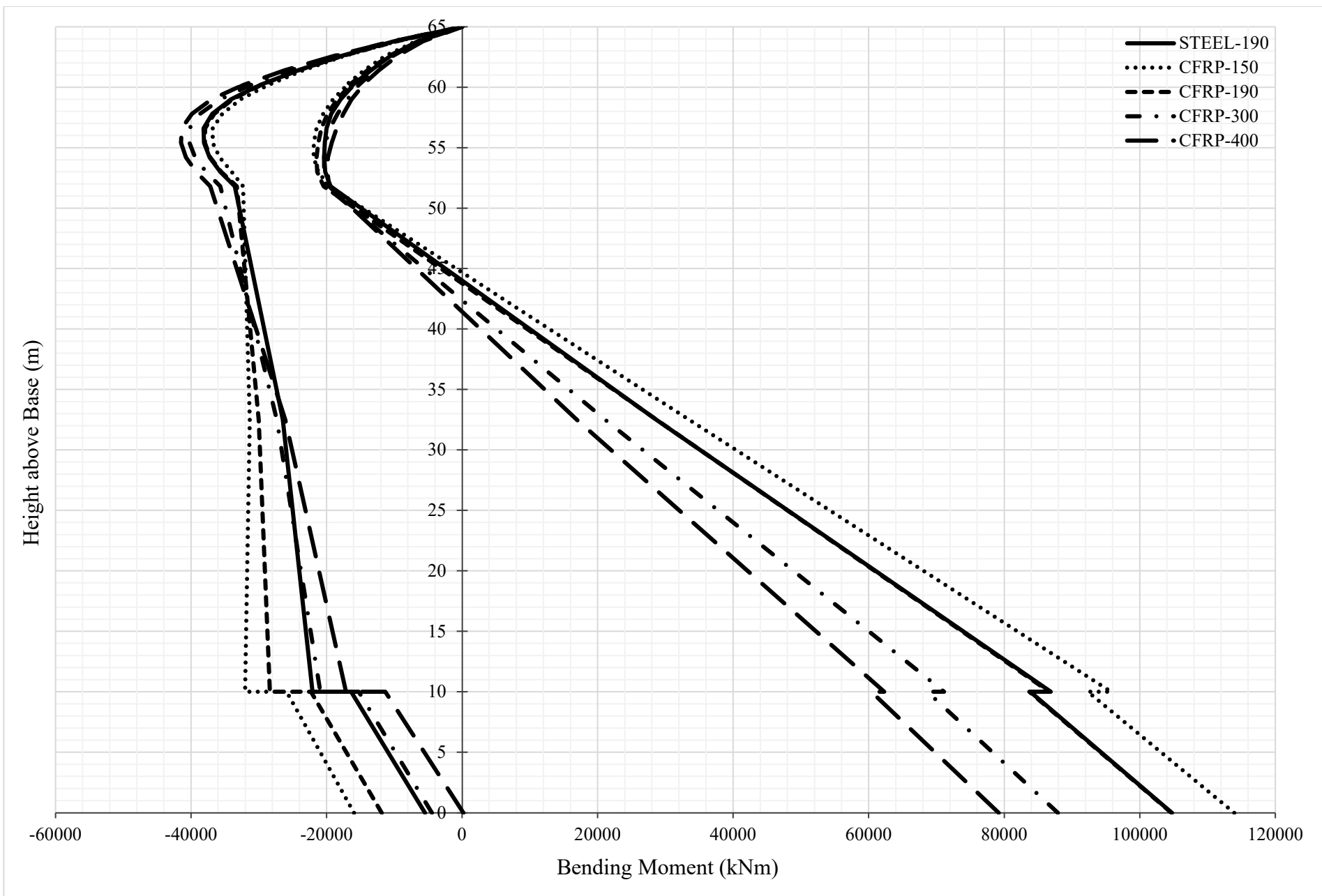


Figure A-41: Minimum and maximum pylon leg bending moment under ULS-1

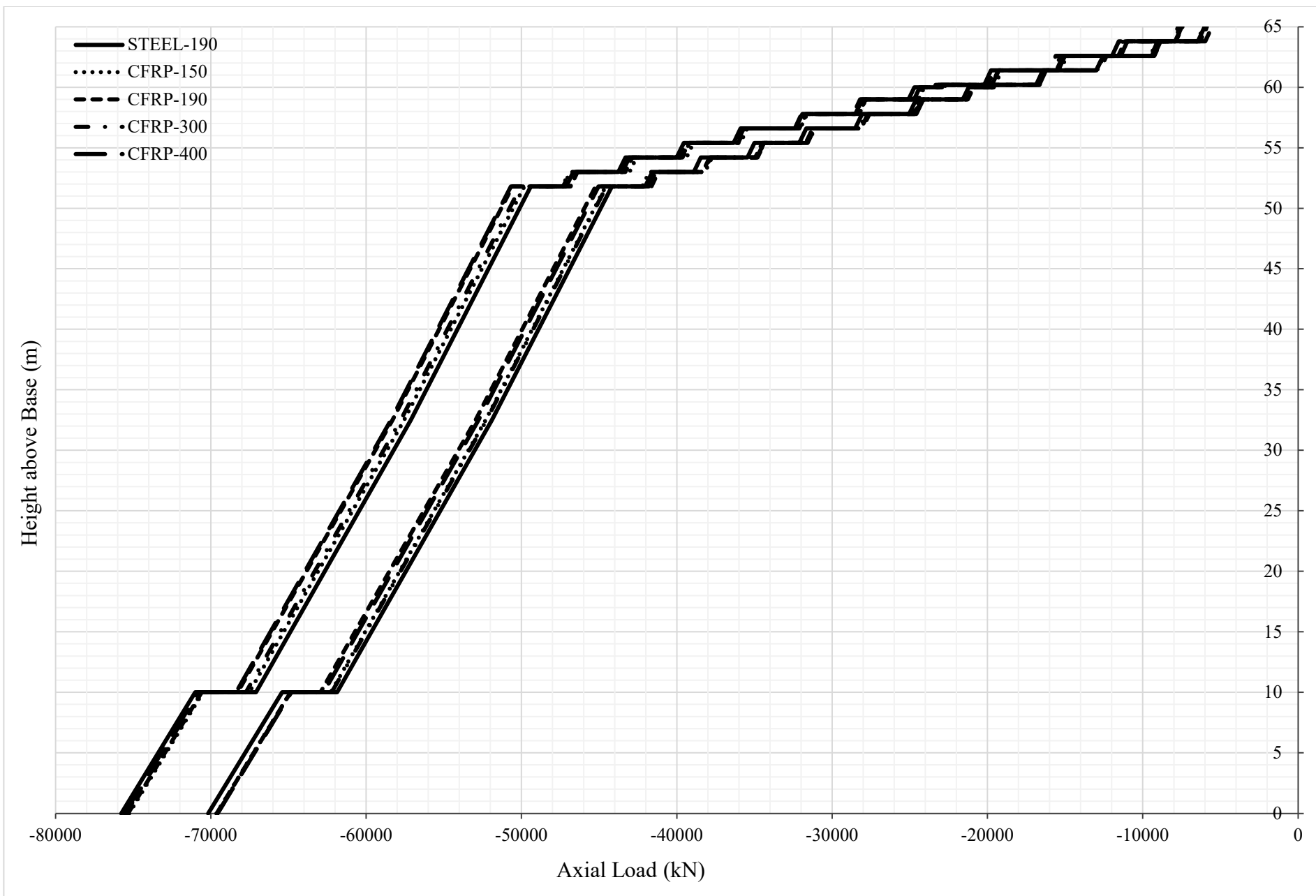


Figure A-42: Minimum and maximum pylon leg axial load under ULS-2-T+

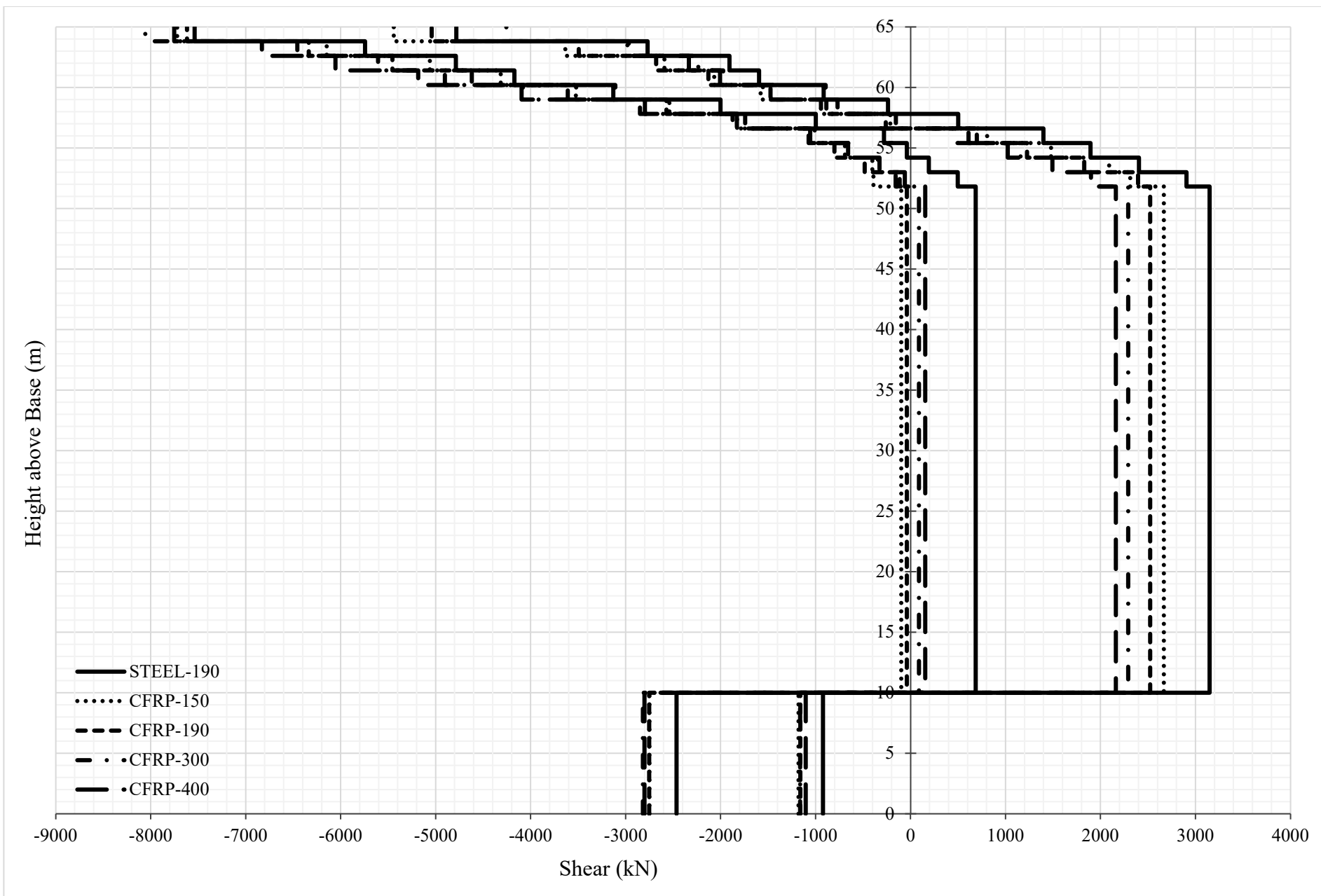


Figure A-43: Minimum and maximum pylon leg shear under ULS-2-T+

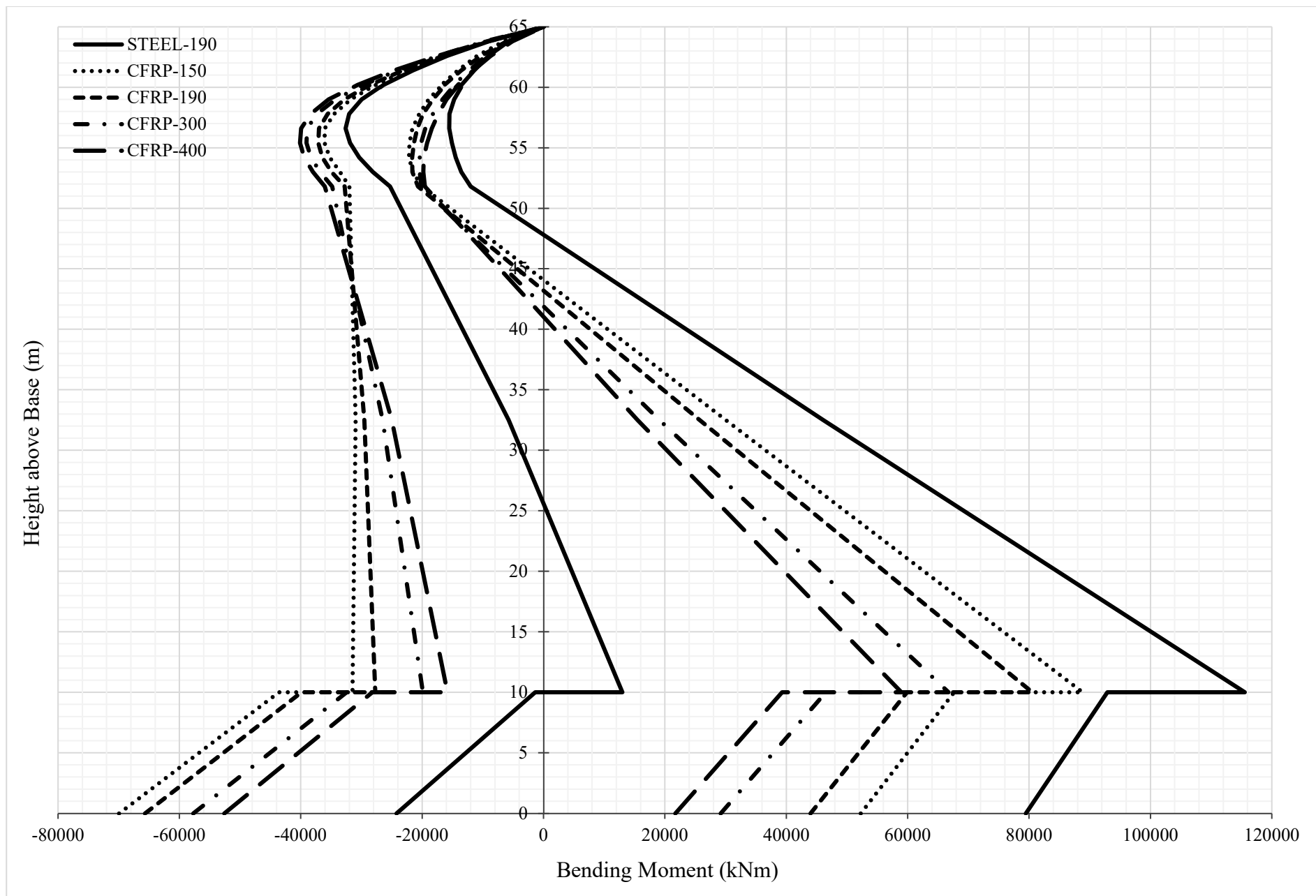


Figure A-44: Minimum and maximum pylon leg bending moment under ULS-2-T+

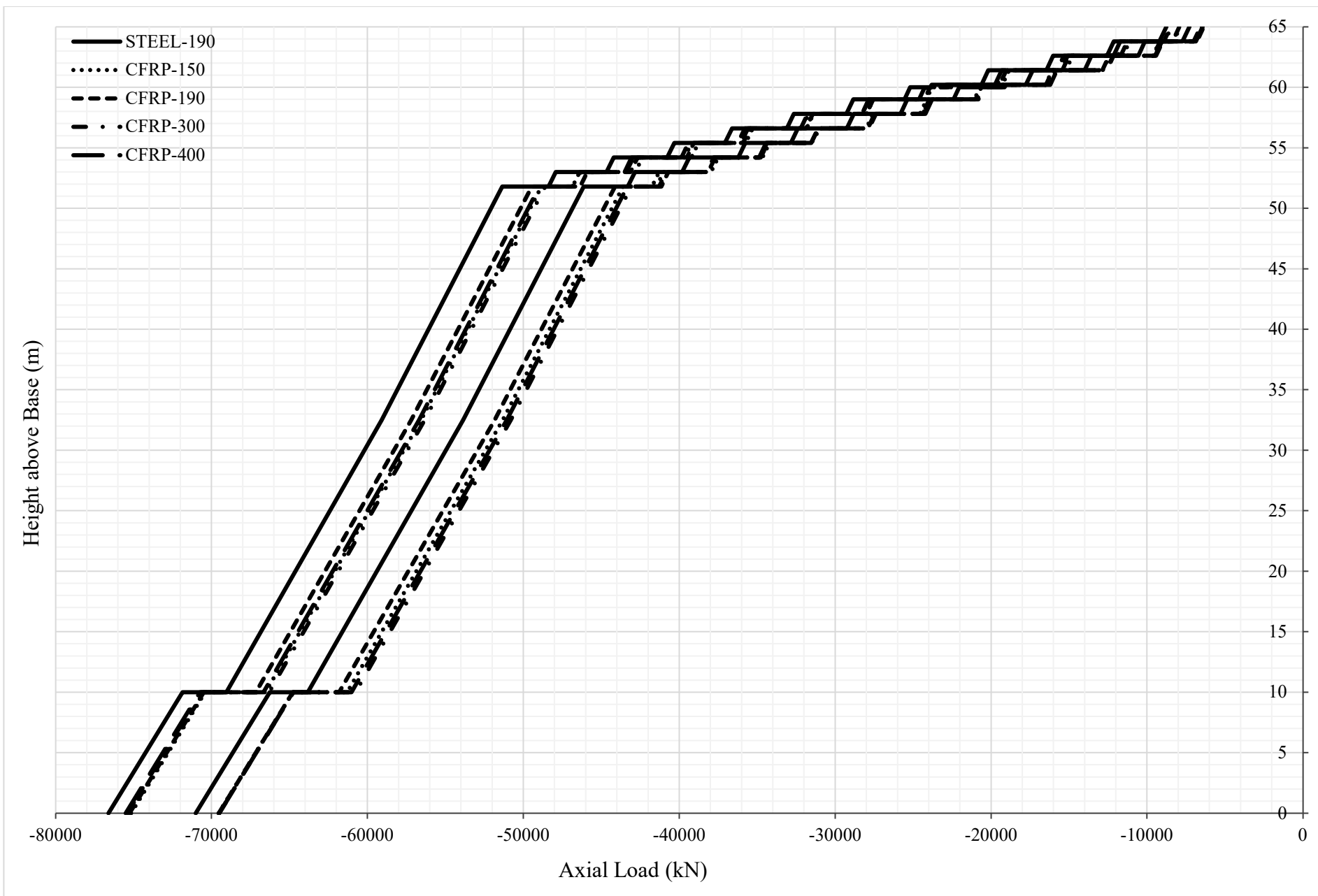


Figure A-45: Minimum and maximum pylon leg axial load under ULS-2-T-

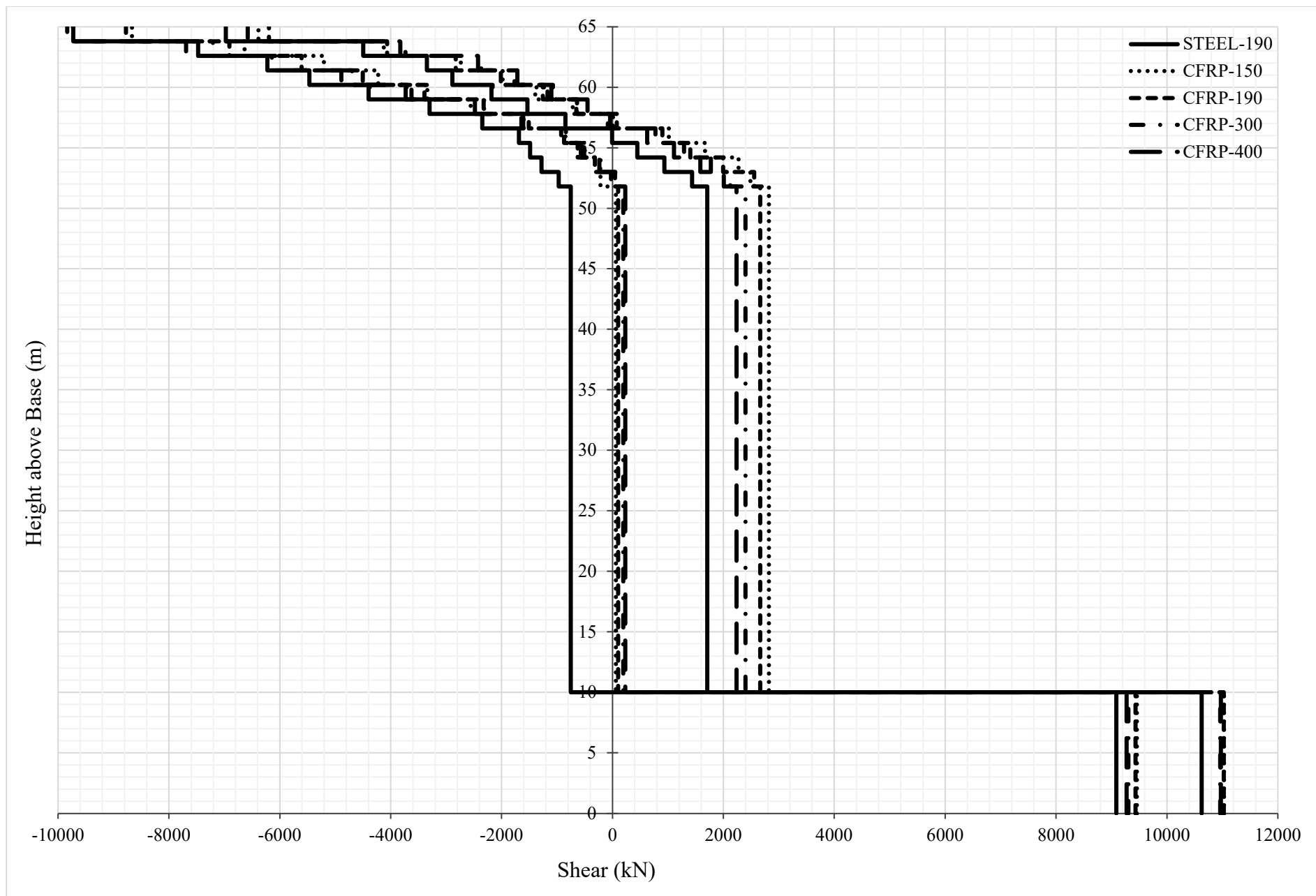


Figure A-46: Minimum and maximum pylon leg shear under ULS-2-T-

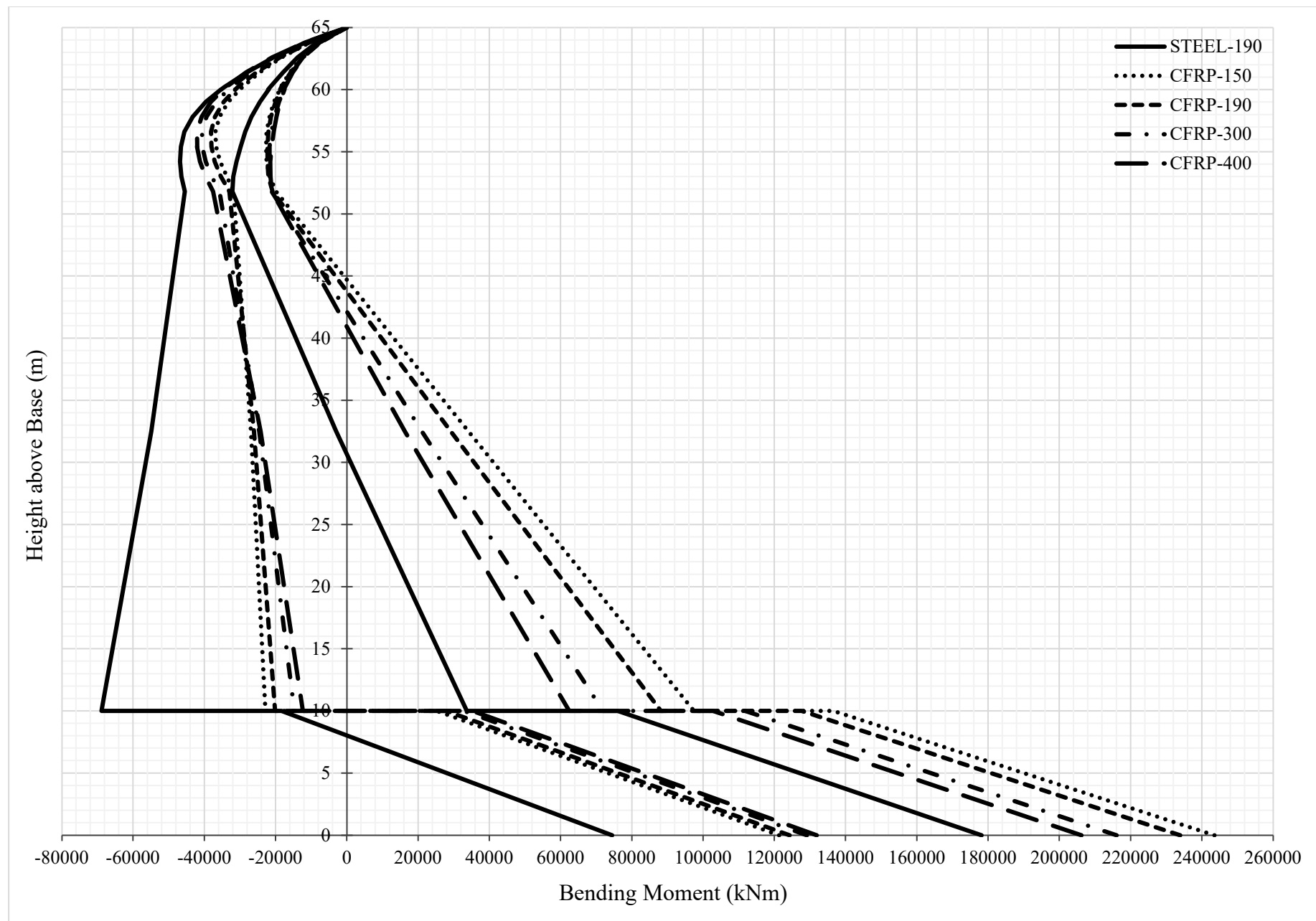


Figure A-47: Minimum and maximum pylon leg bending moment under ULS-2-T-

Appendix B. Phase II Results

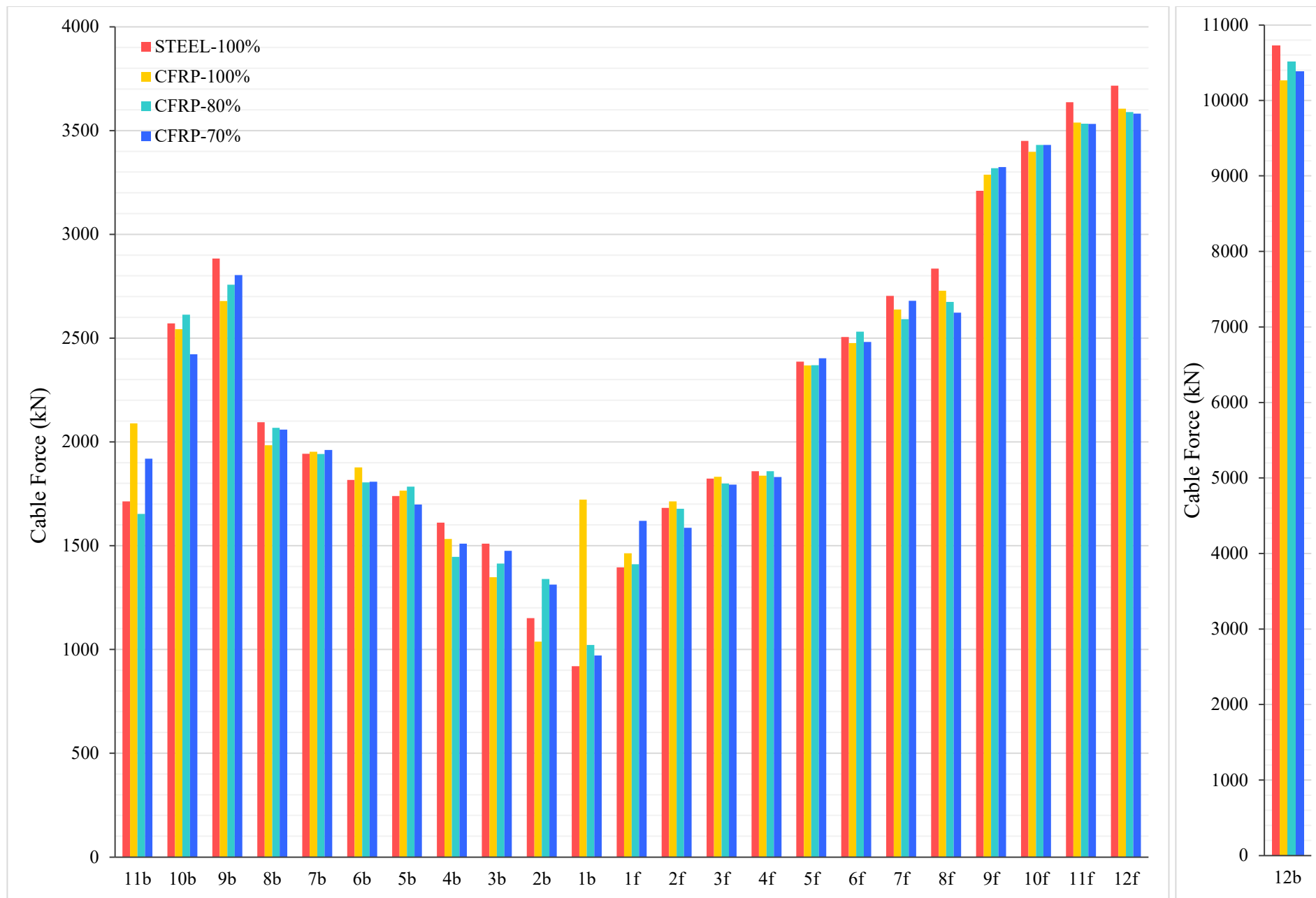


Figure B-1: Cable forces under SLS-1 TENSION BACKSTAYS

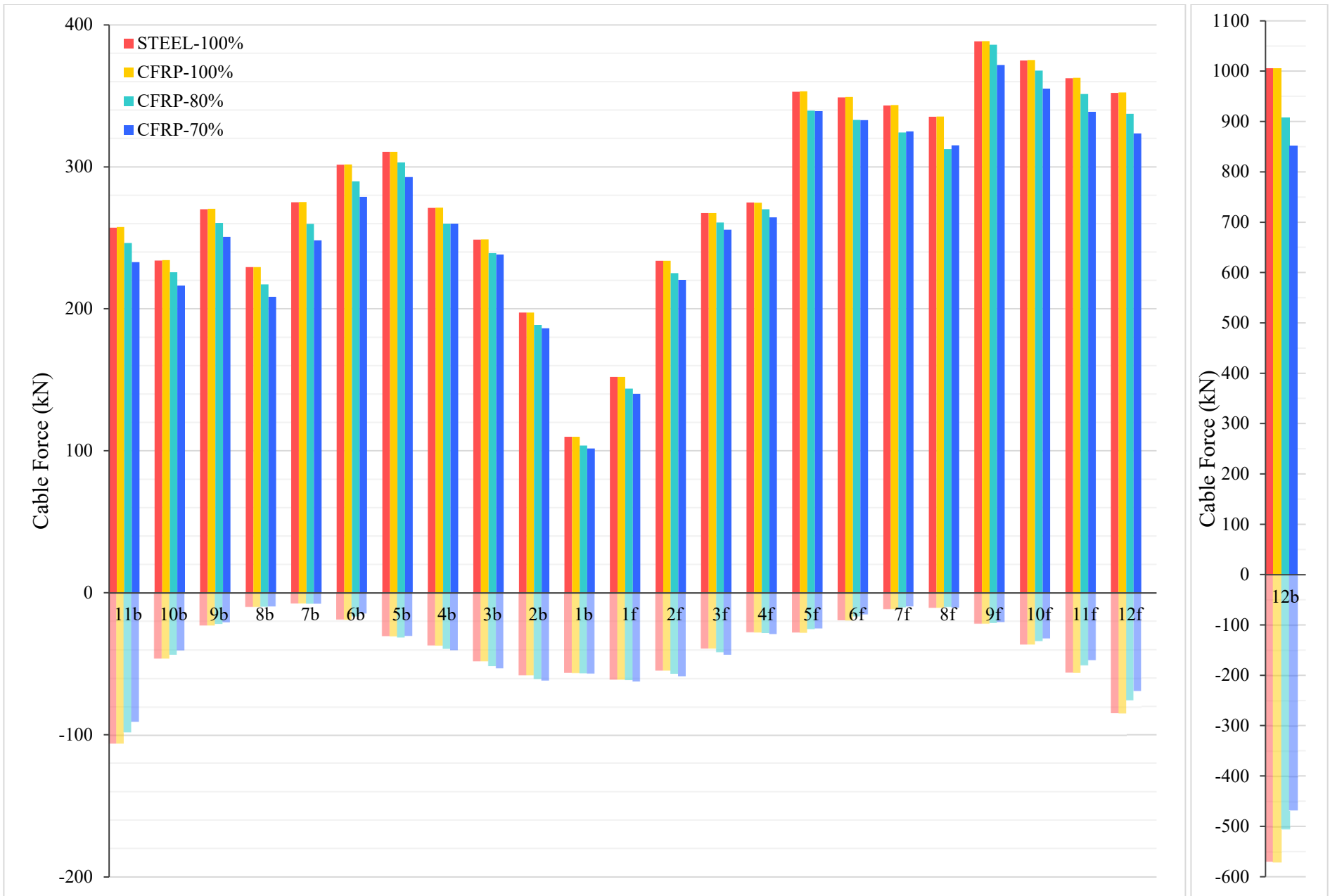


Figure B-2: Maximum increase and decrease in cable forces under live load envelope

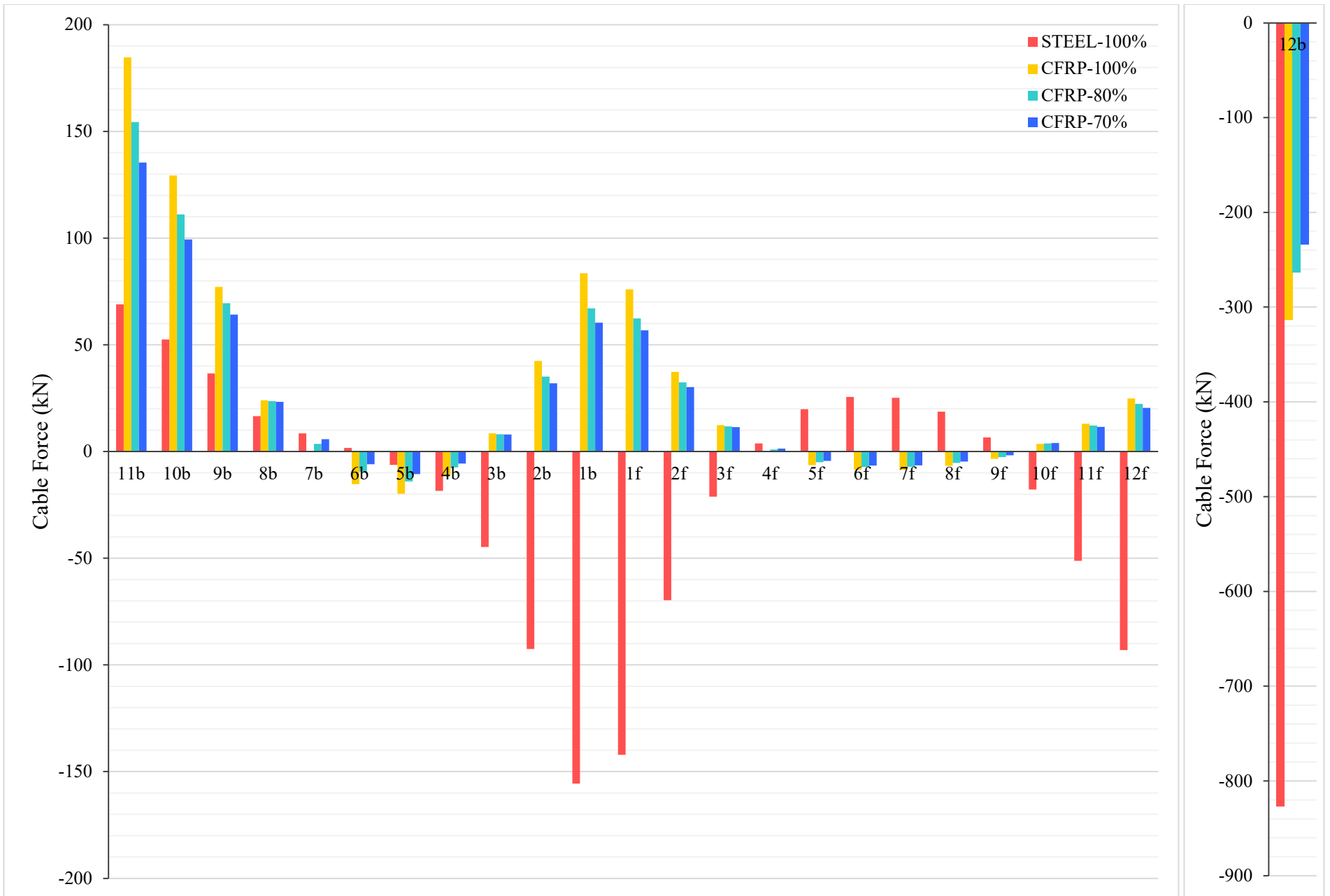


Figure B-3: Change in cable forces under T+

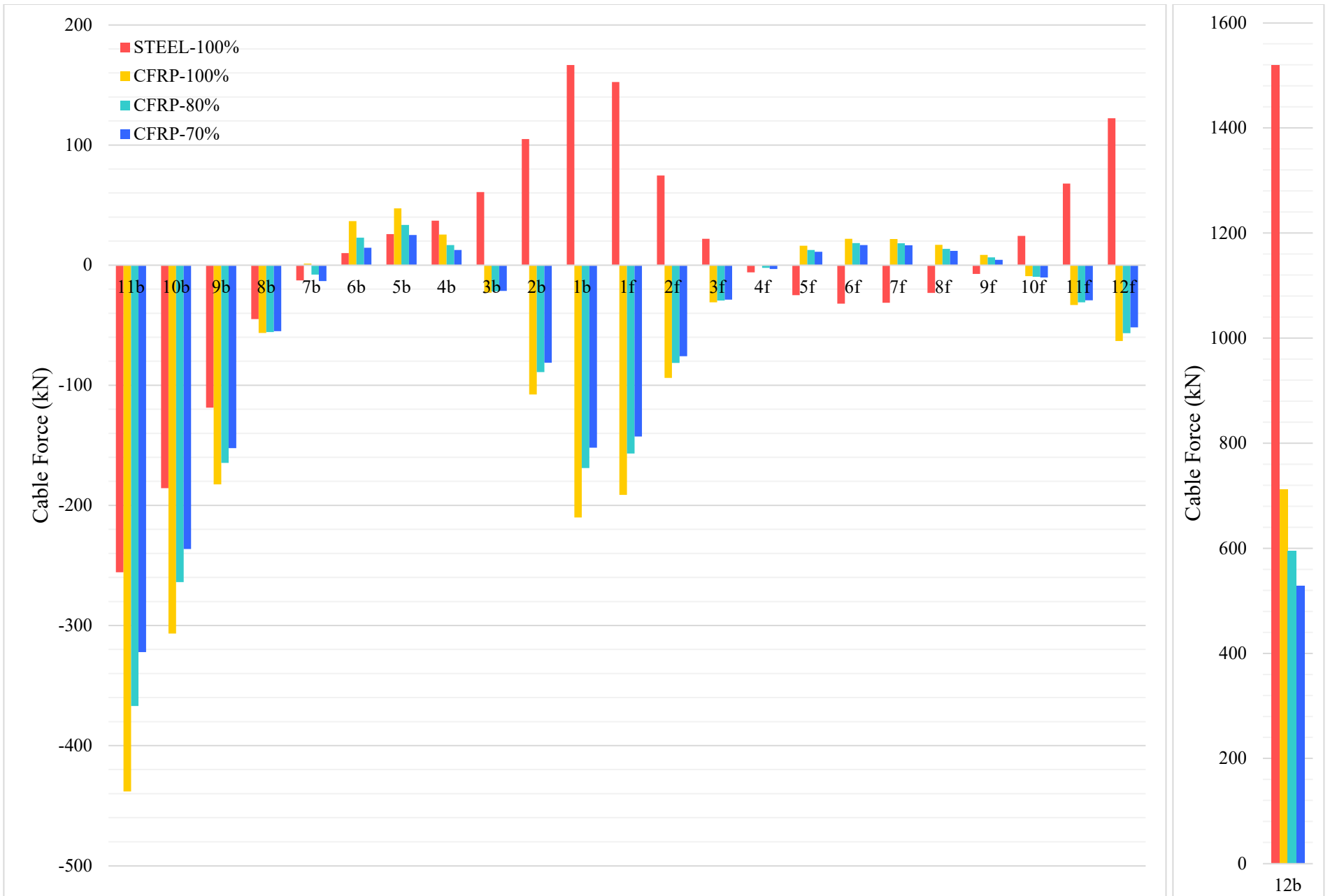


Figure B-4: Change in cable forces under T-

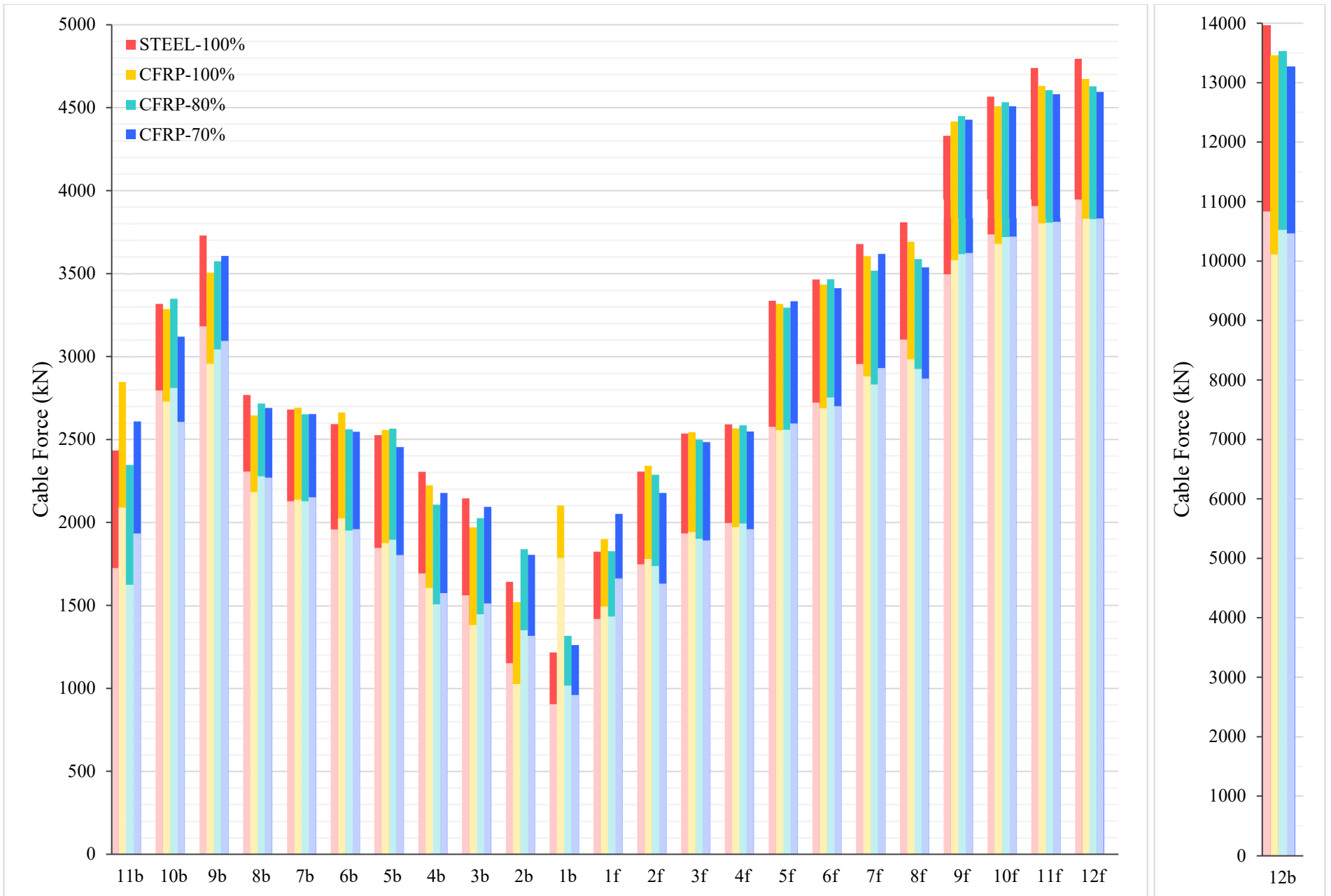


Figure B-5: Minimum and maximum cable forces under ULS-1 envelope

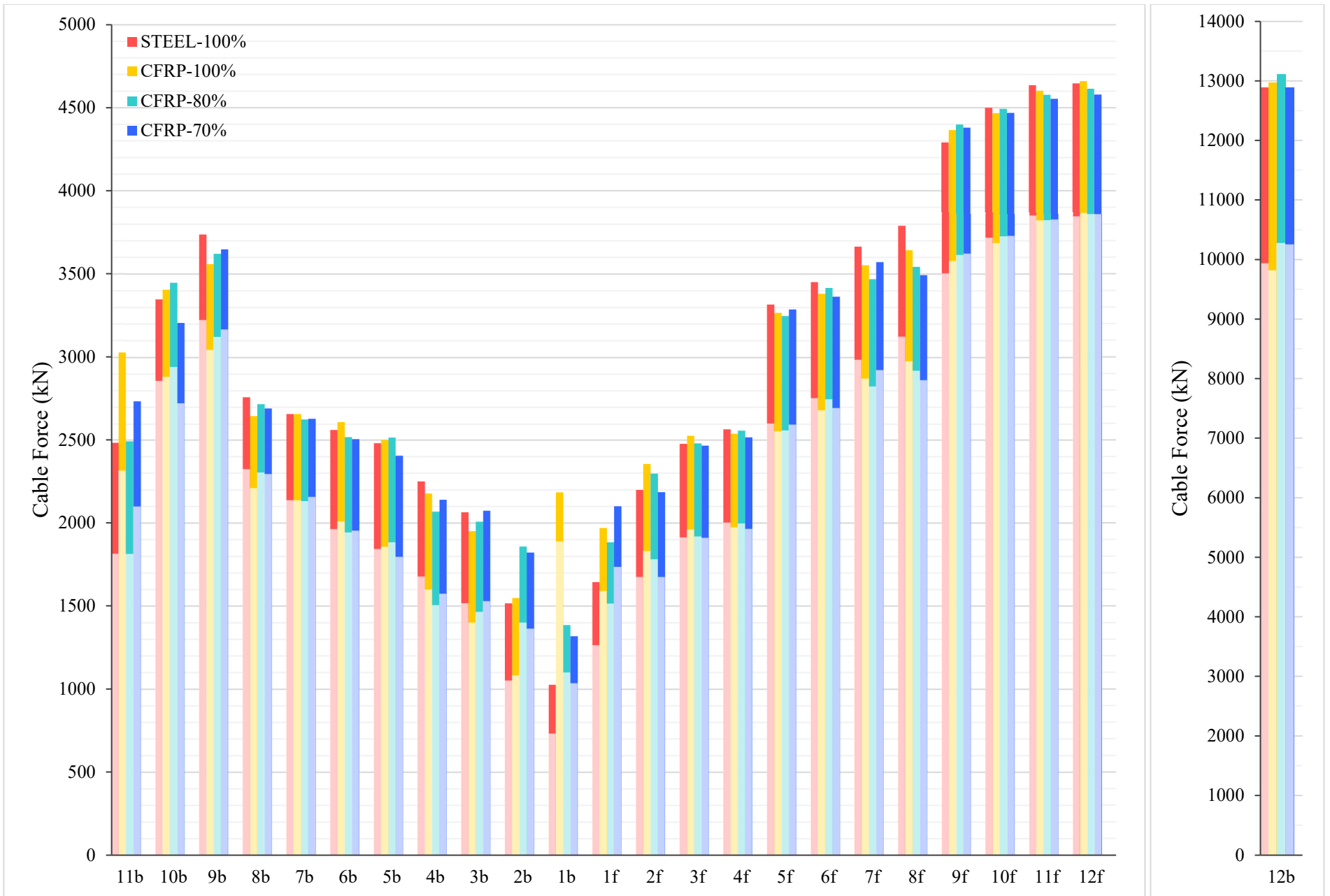


Figure B-6: Minimum and maximum cable forces under ULS-2-T+ envelope

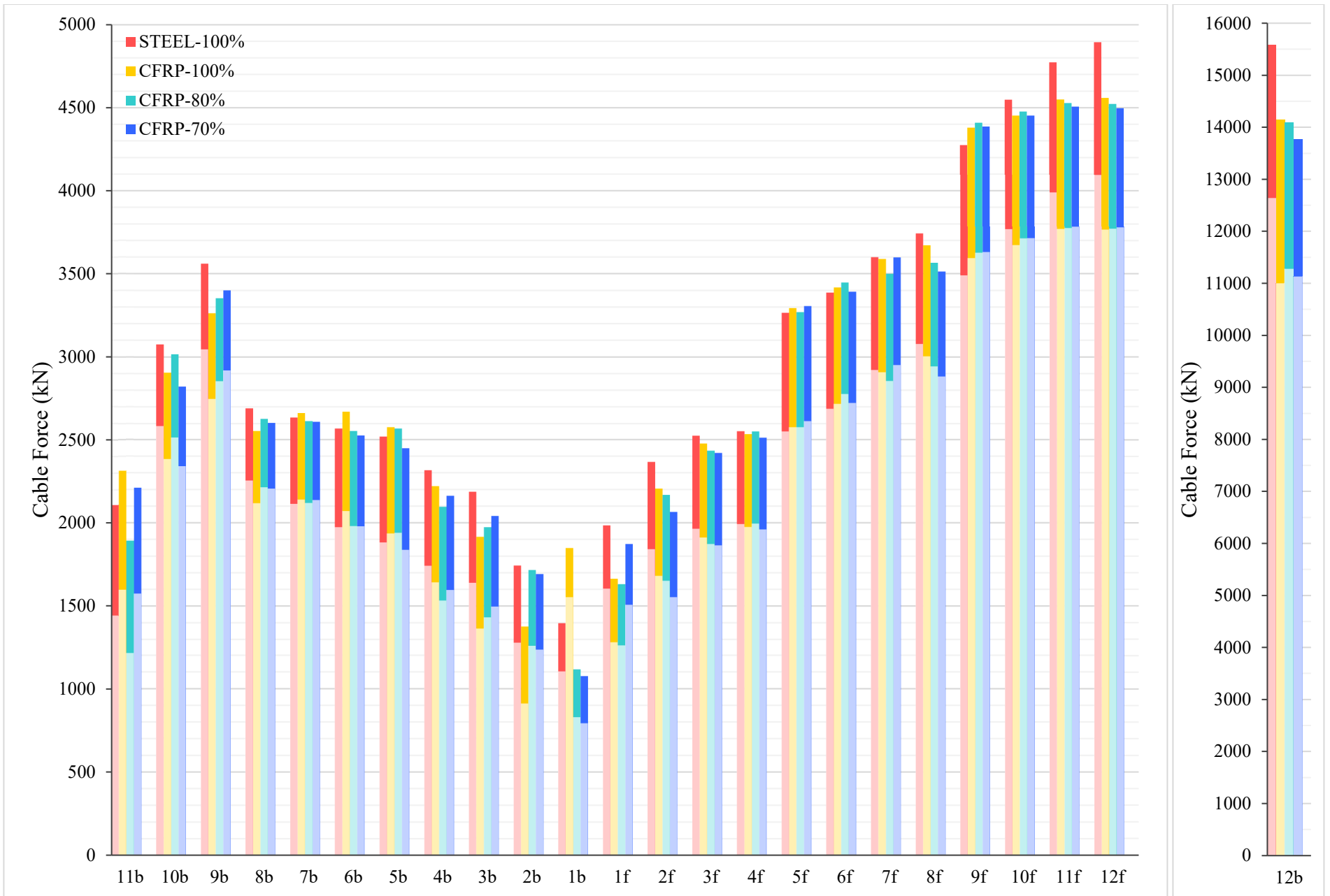


Figure B-7: Minimum and maximum cable forces under ULS-2-T- envelope

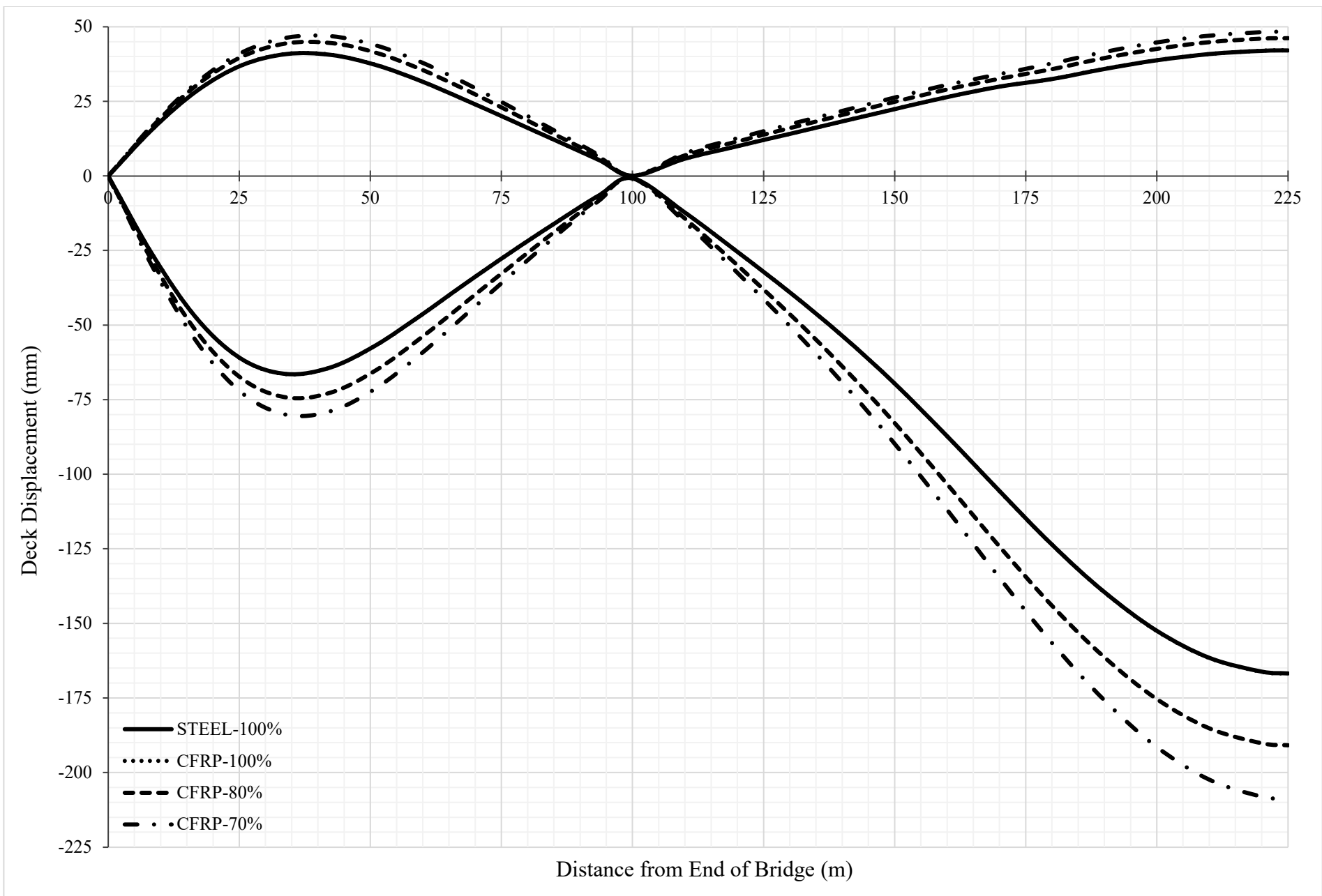


Figure B-8: Minimum and maximum vertical deck displacements under LL ENV

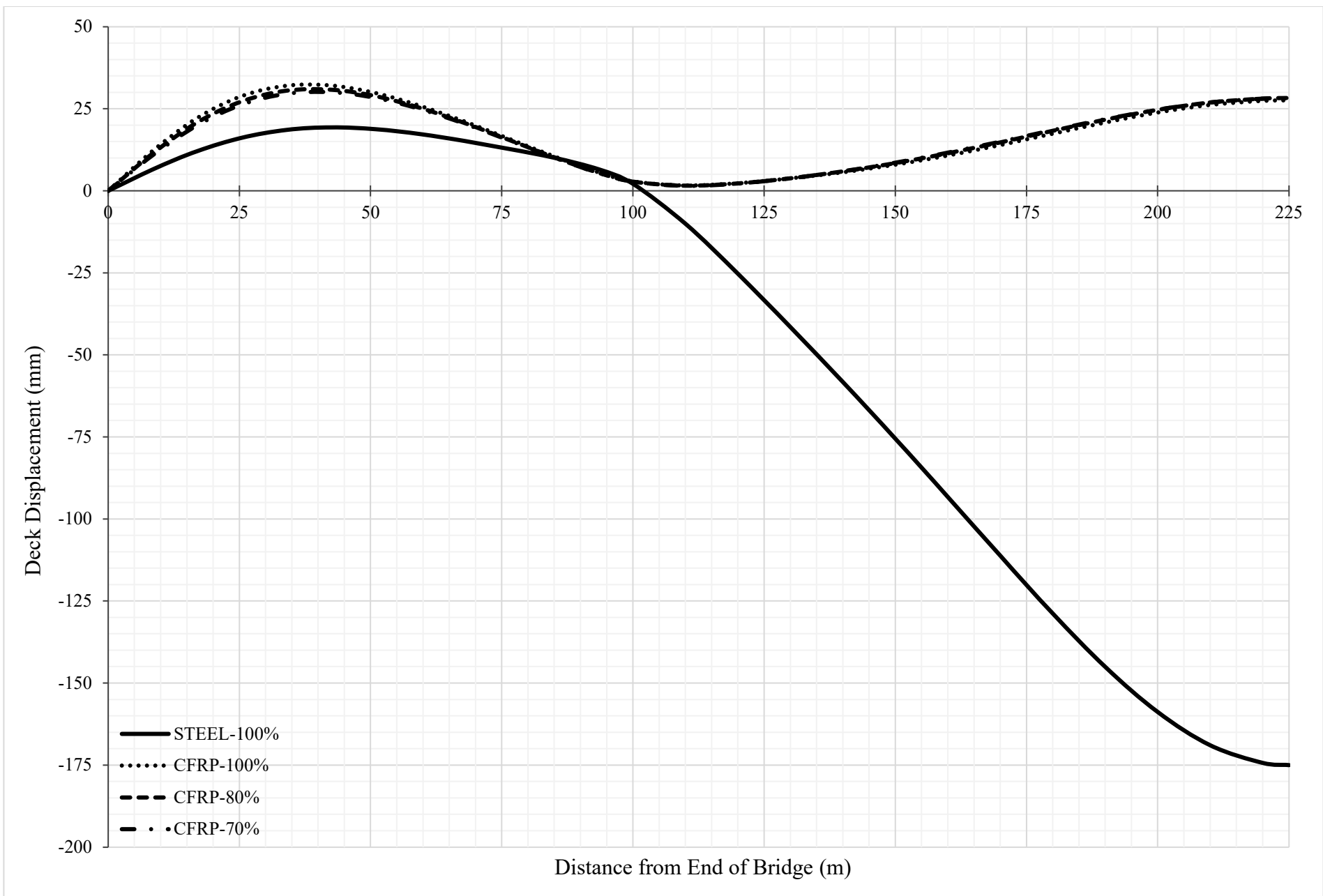


Figure B-9: Vertical deck displacements under T+

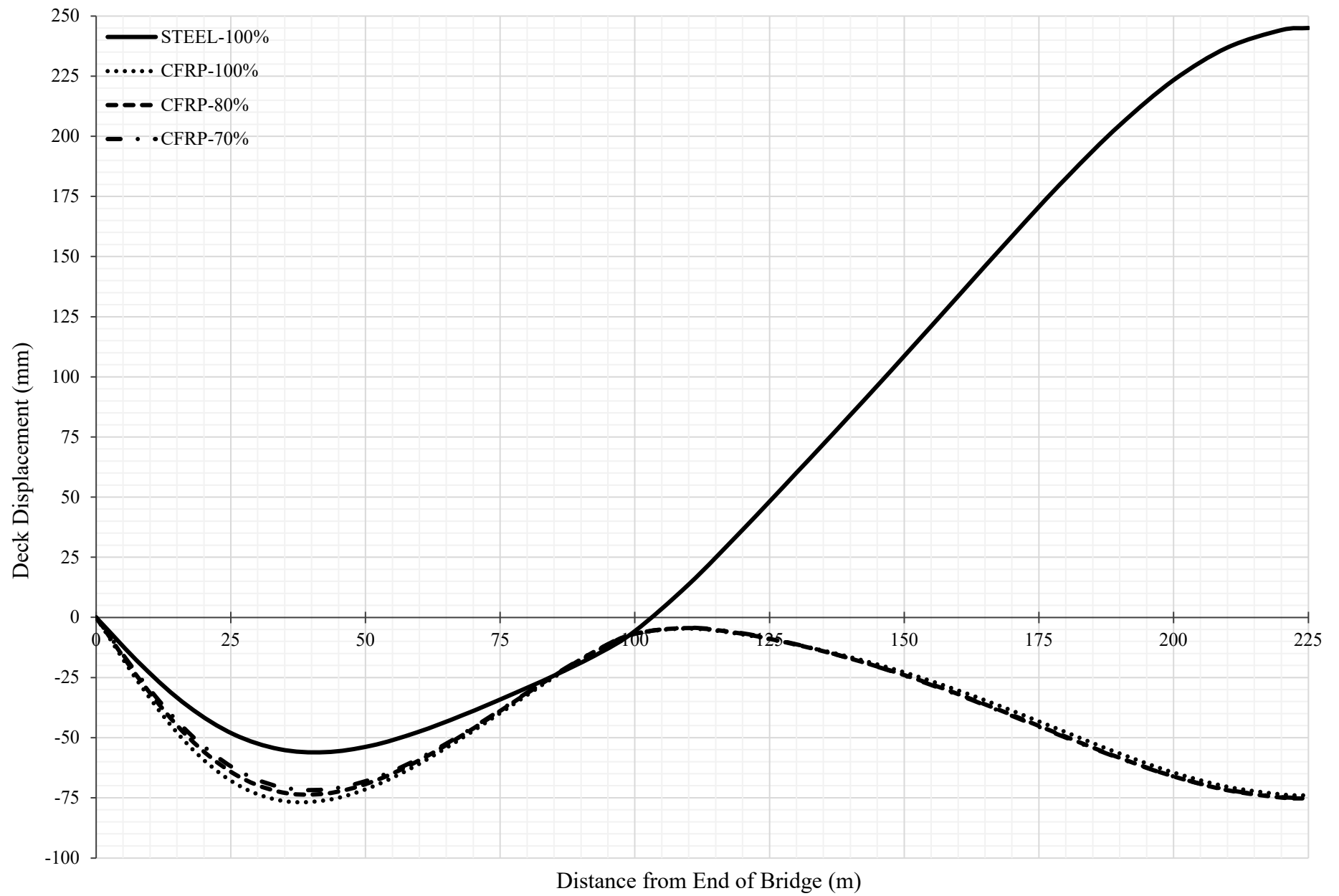


Figure B-10: Vertical deck displacements under T-

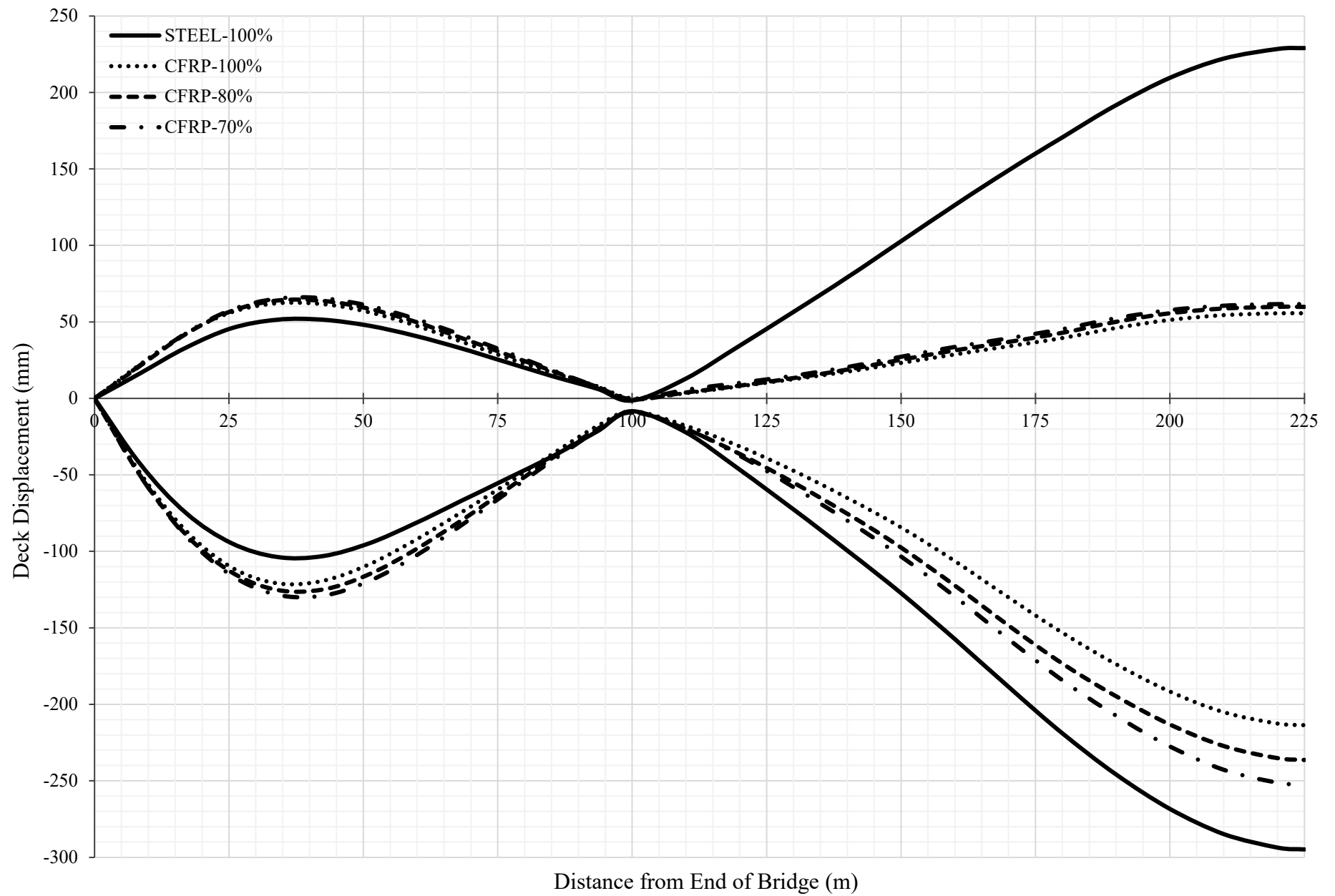


Figure B-11: Minimum and maximum vertical deck displacements under SLS-1 envelope

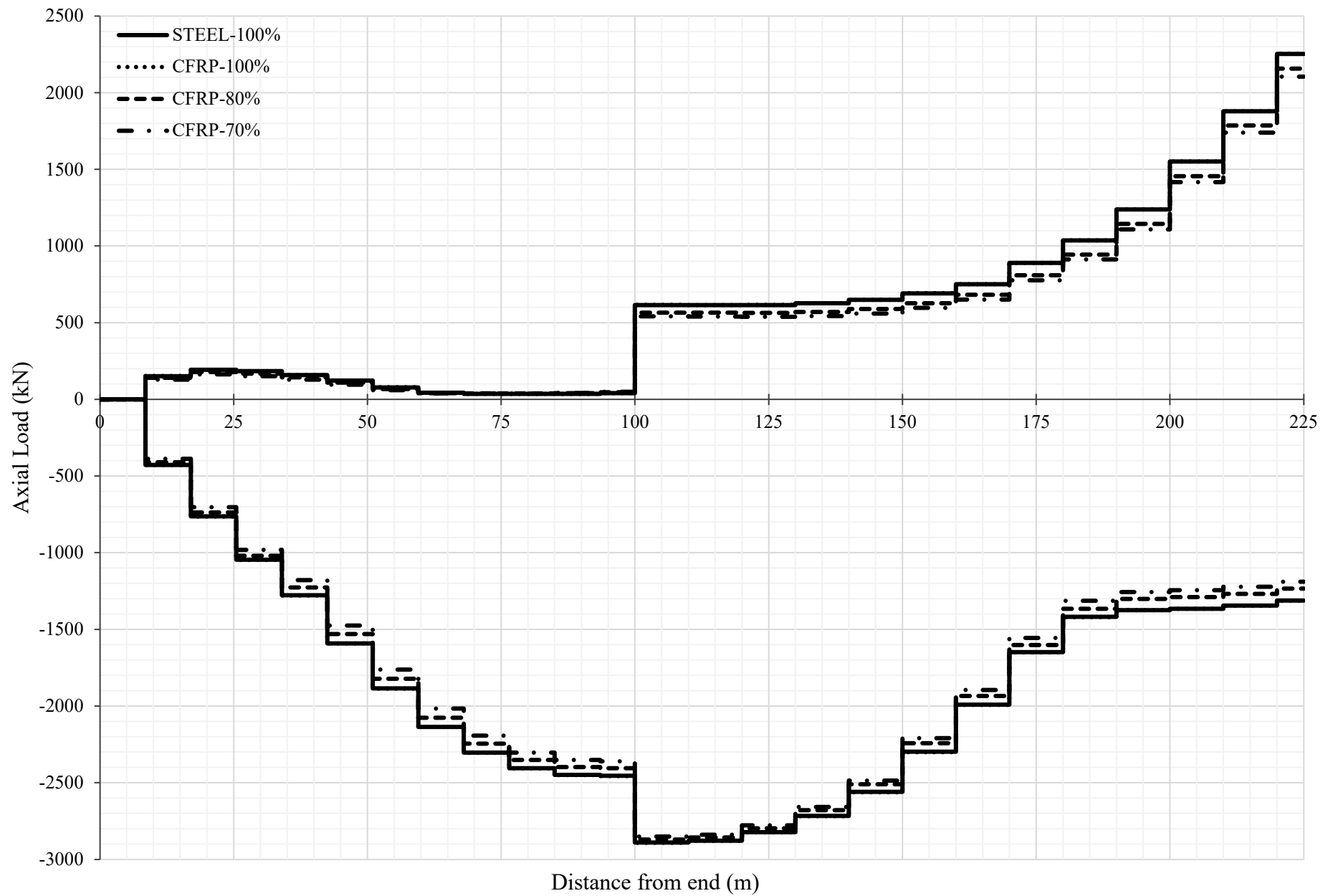


Figure B-12: Minimum and maximum deck axial loads under LL ENV

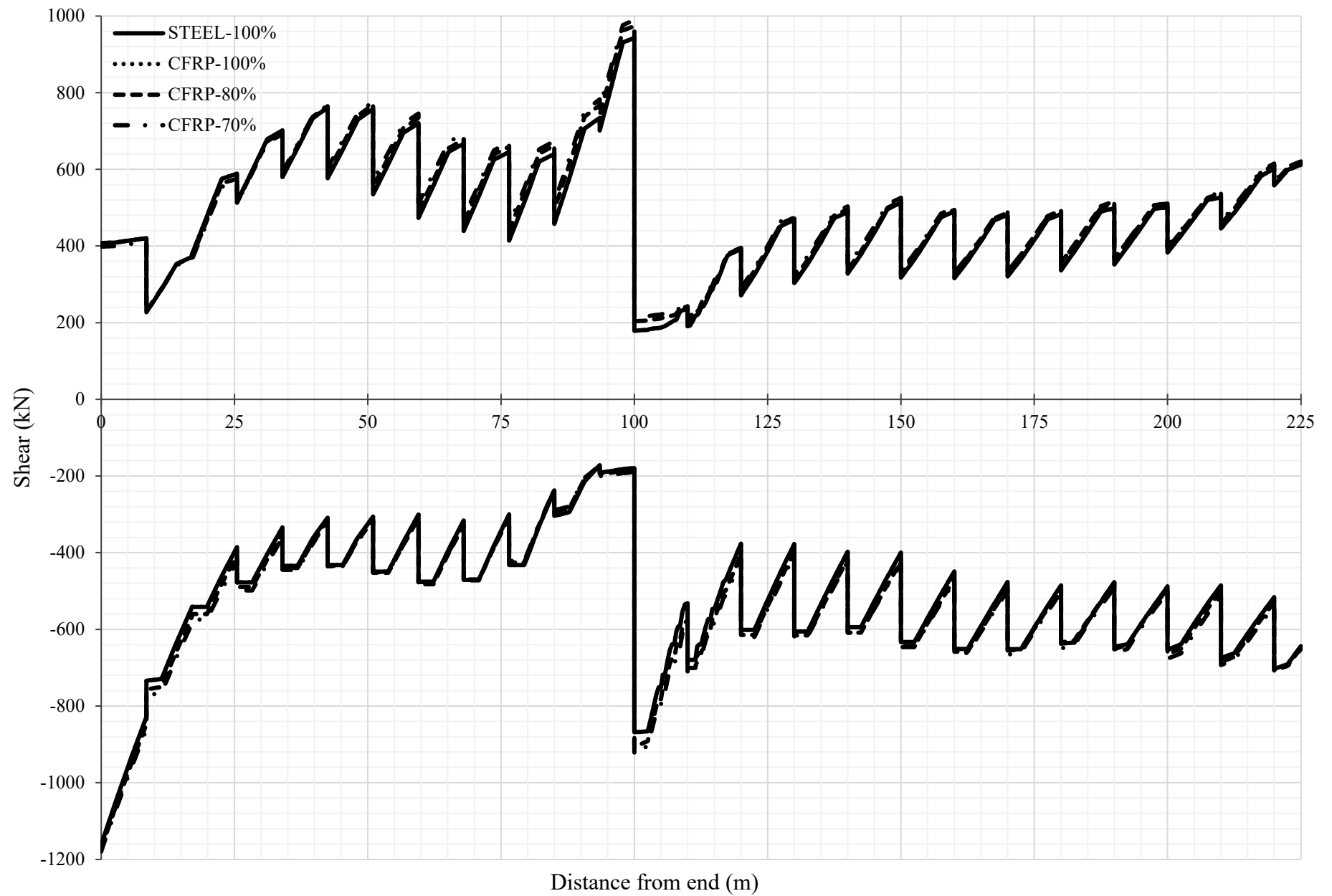


Figure B-13: Minimum and maximum deck shear under LL ENV

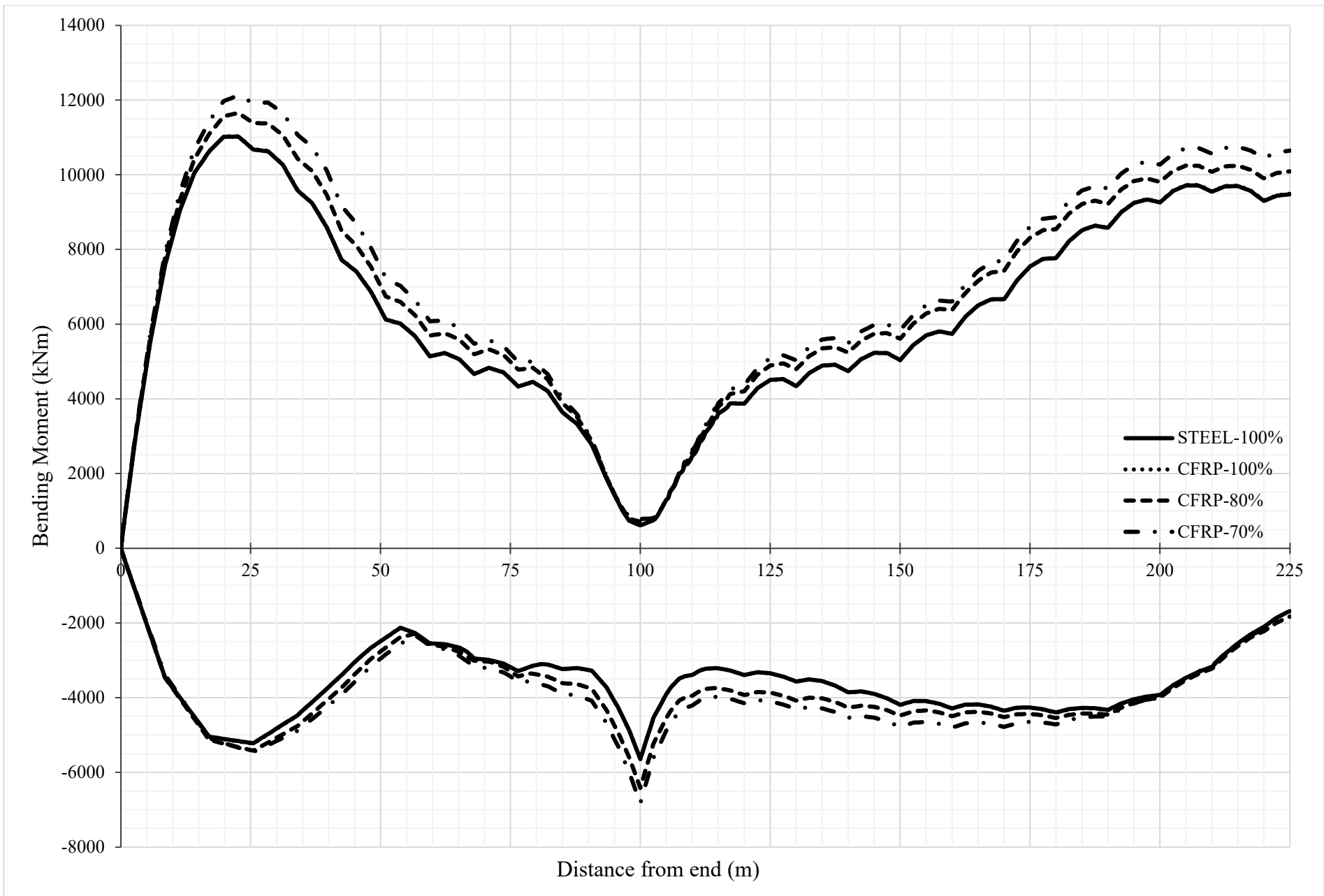


Figure B-14: Minimum and maximum deck bending moment under LL ENV

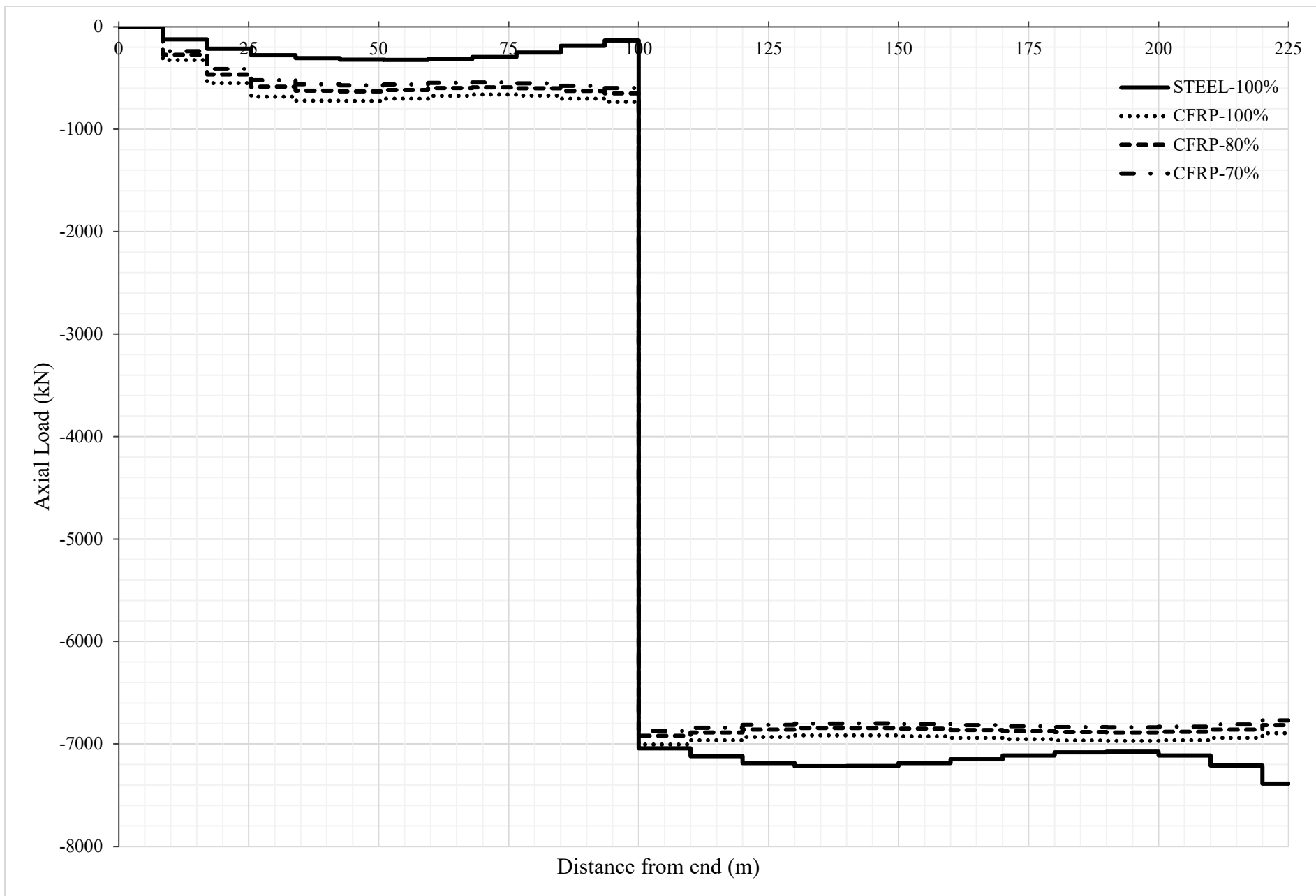


Figure B-15: Deck axial load under T+

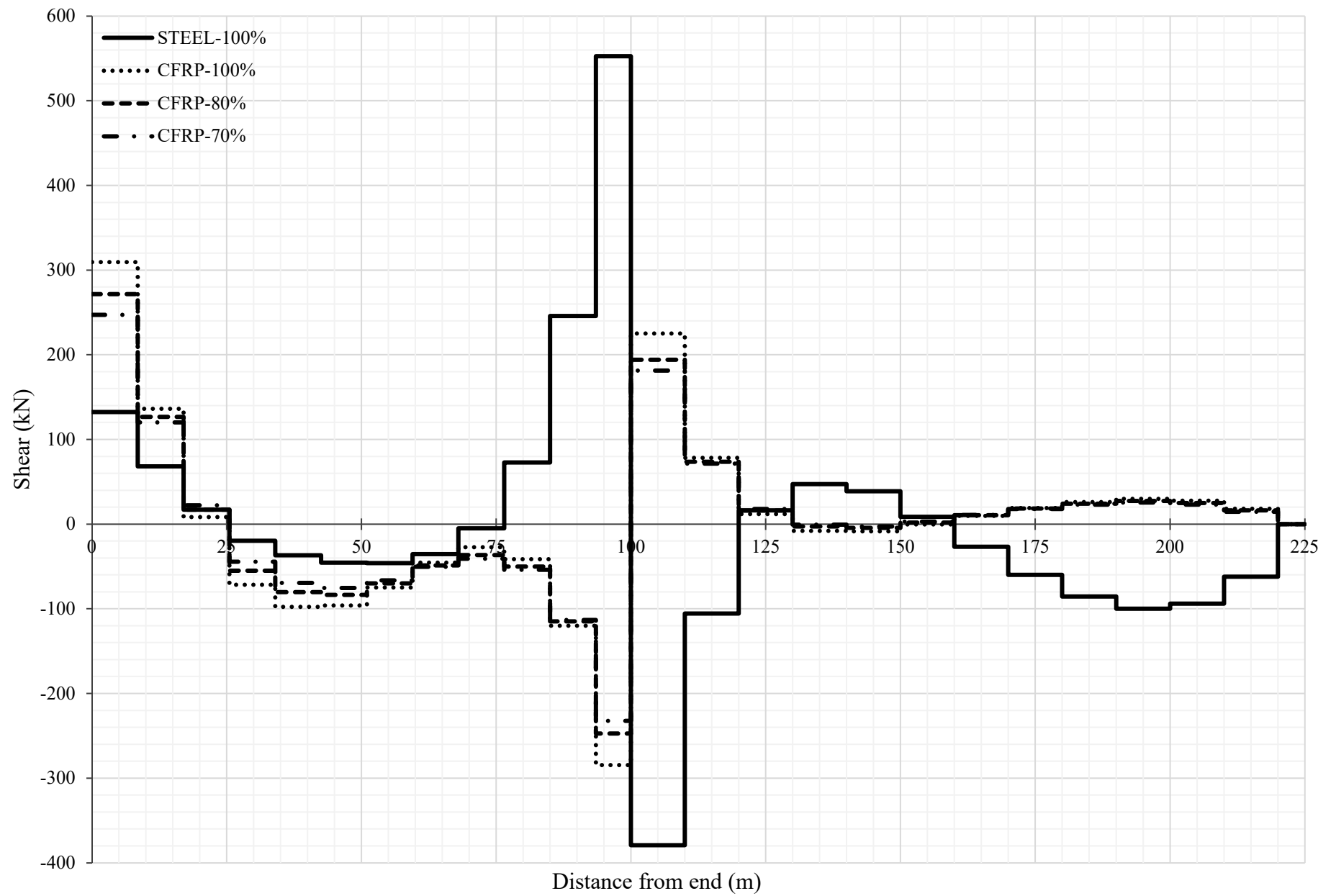


Figure B-16: Deck shear under T+

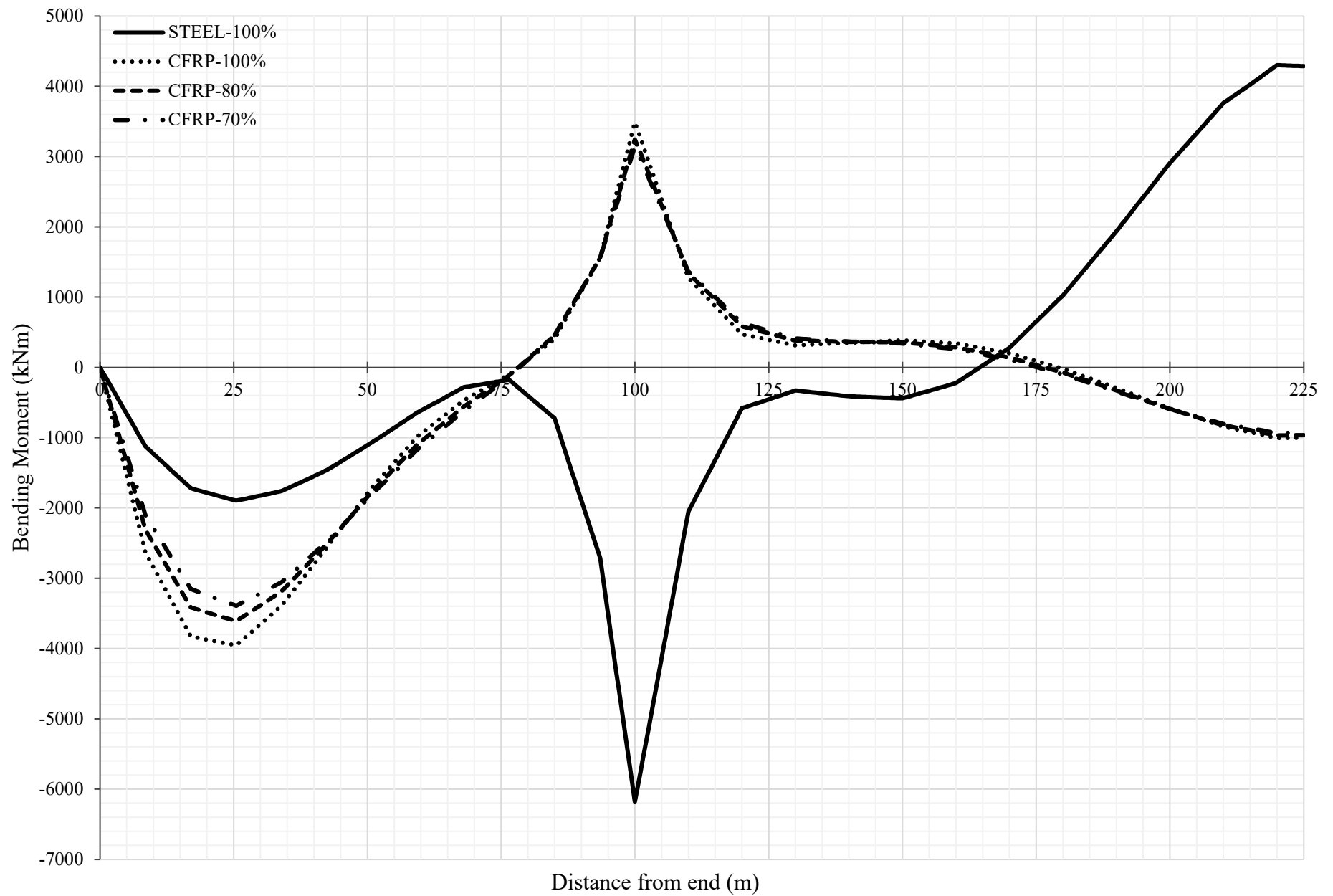


Figure B-17: Deck bending moment under T+

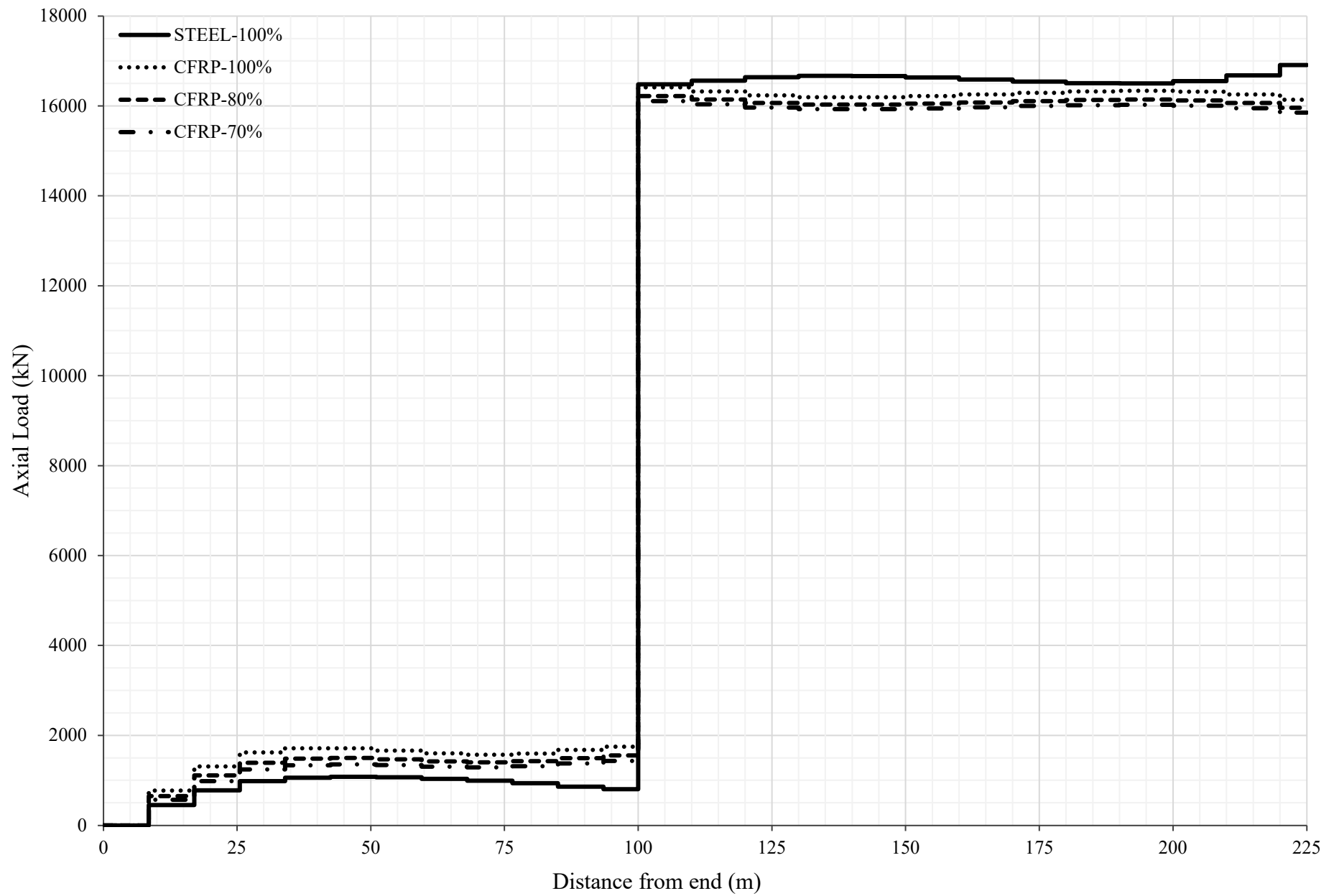


Figure B-18: Deck axial load under T-

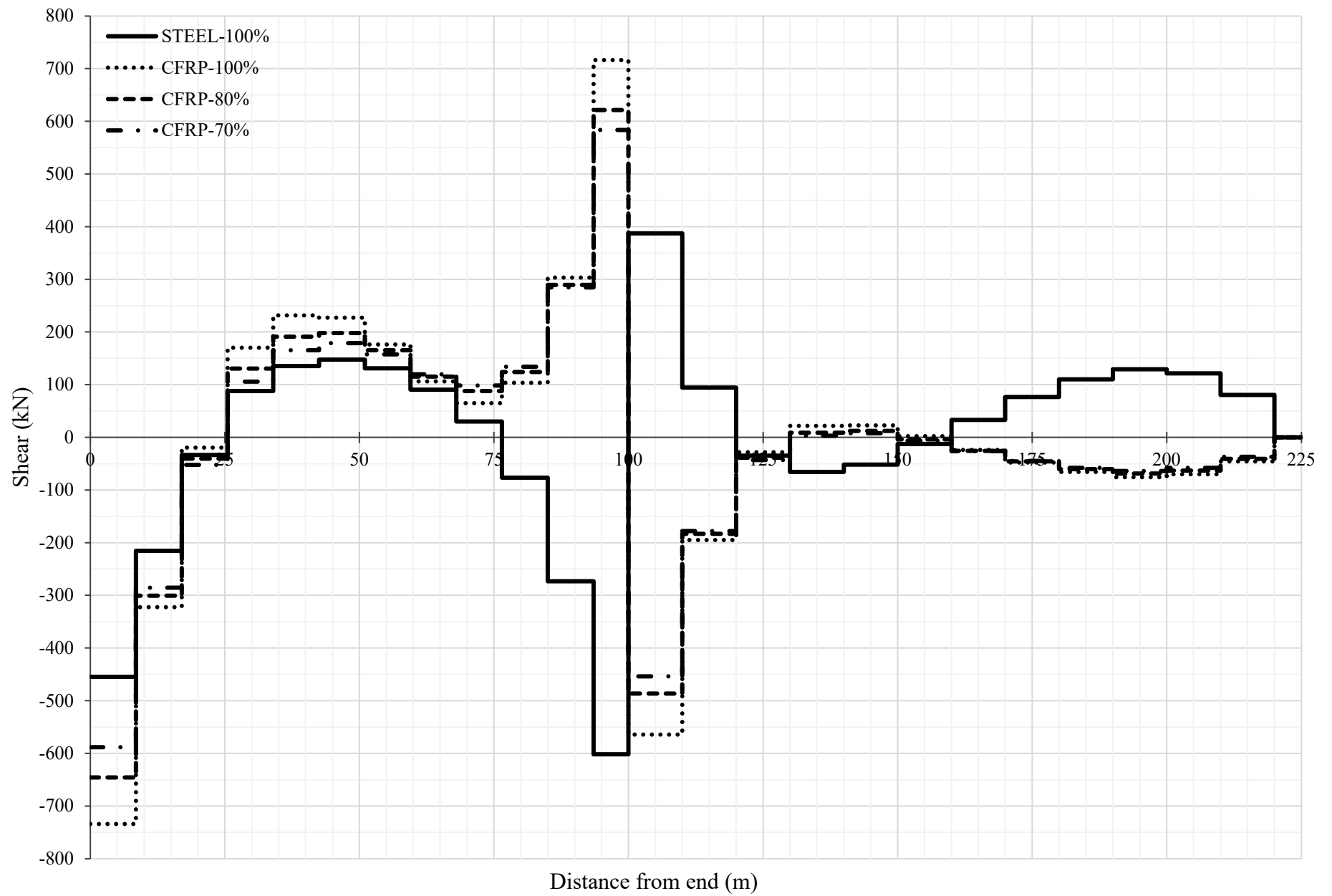


Figure B-19: Deck shear under T-

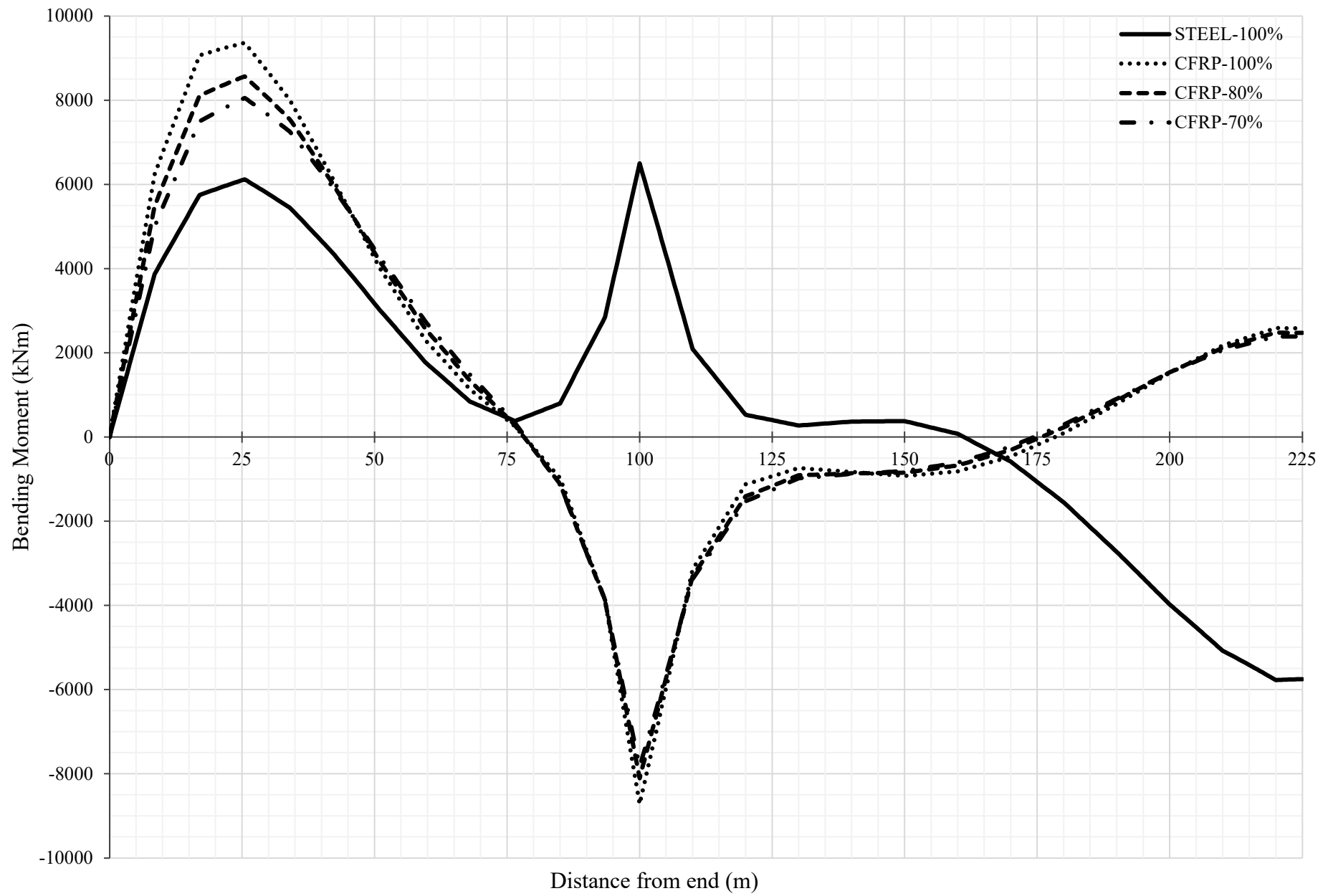


Figure B-20: Deck bending moment under T-

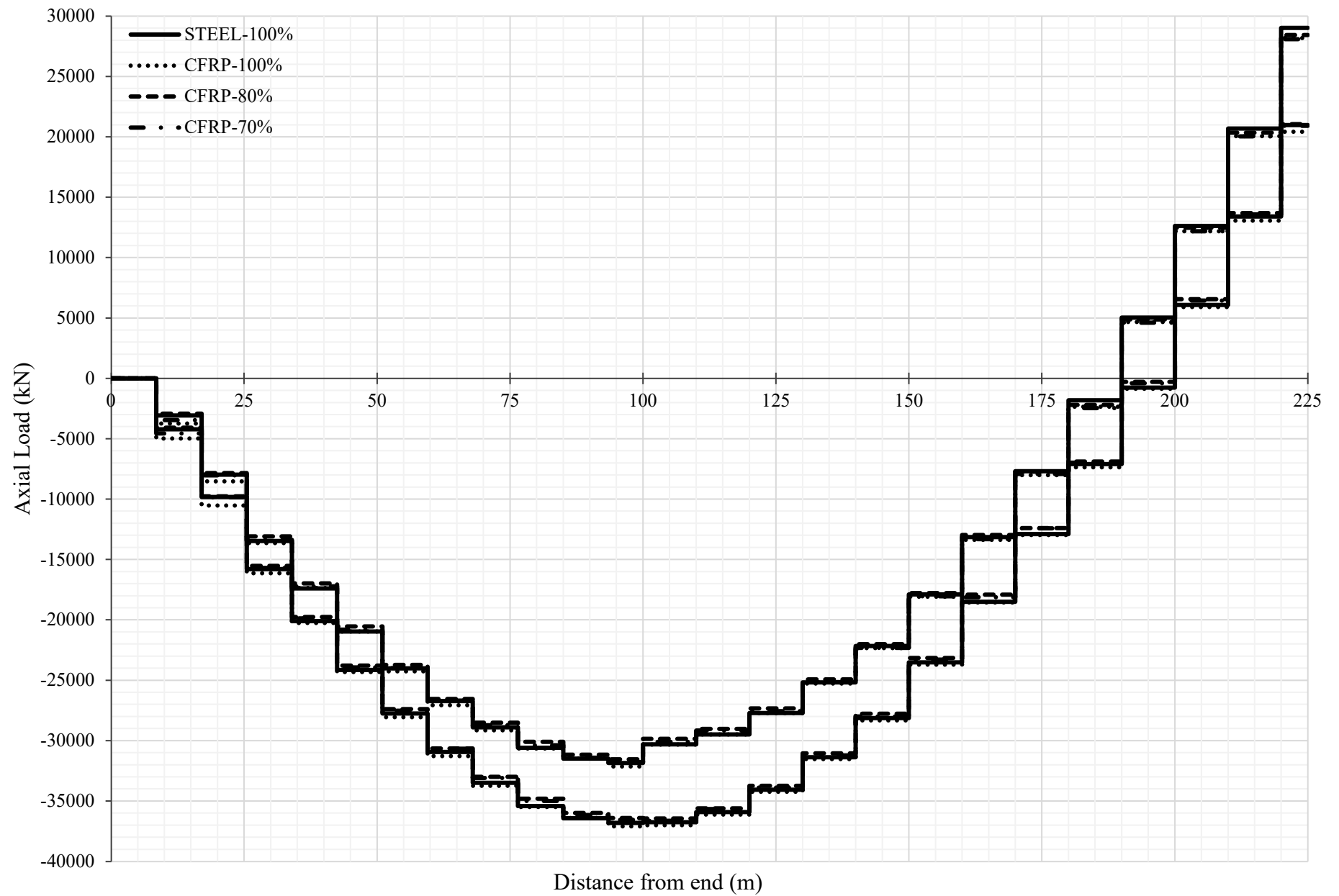


Figure B-21: Minimum and maximum deck axial load under ULS-1

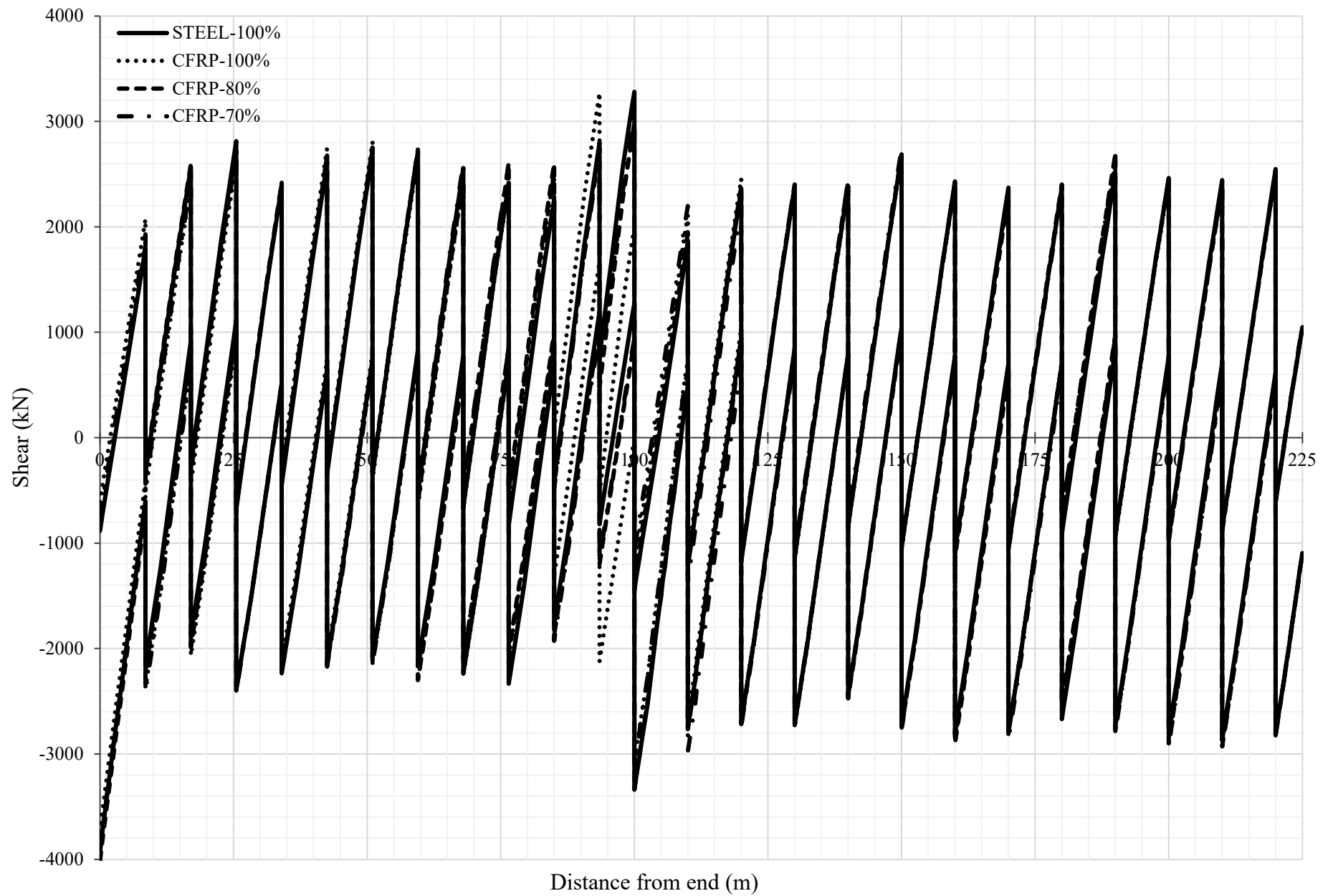


Figure B-22: Minimum and maximum deck shear under ULS-1

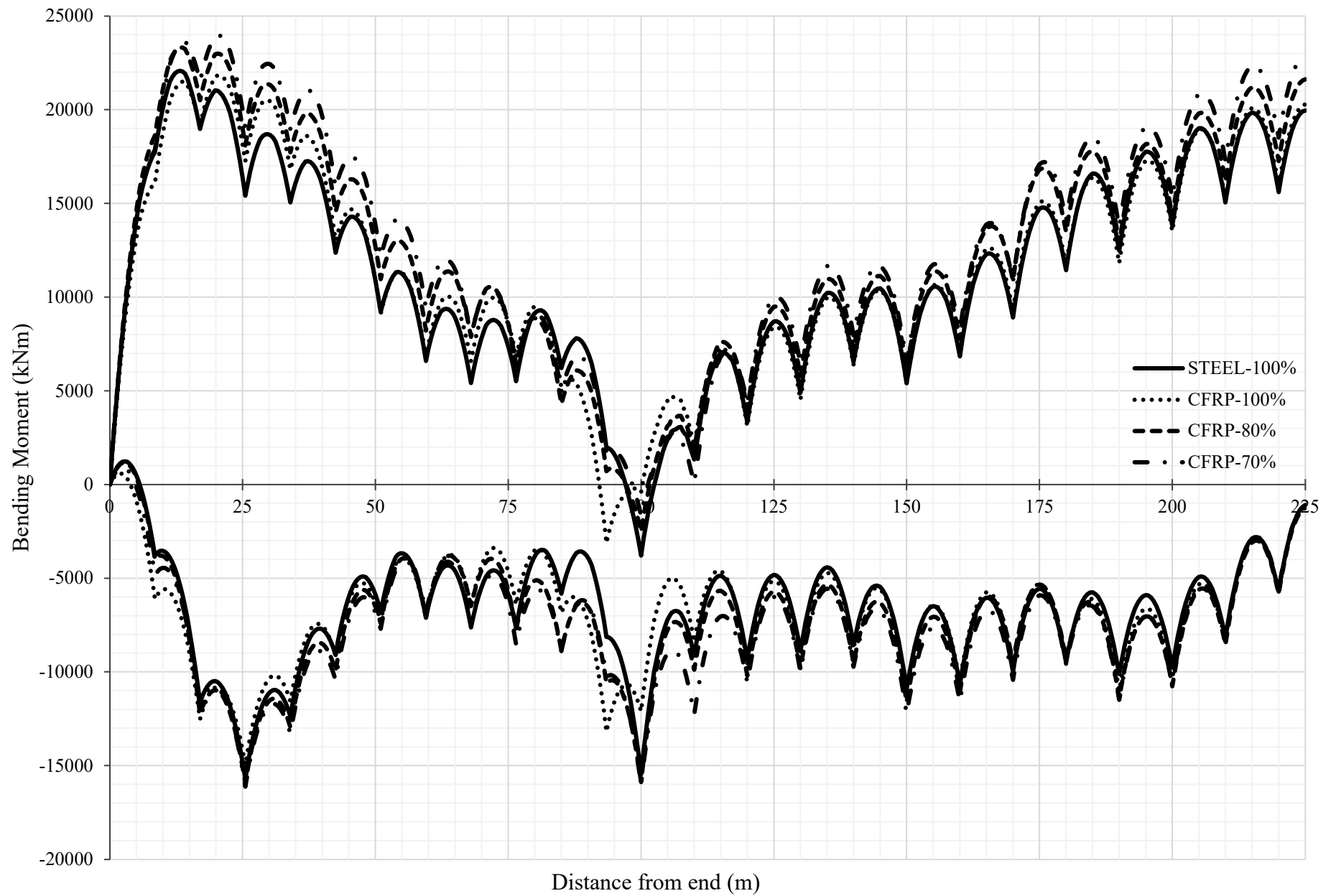


Figure B-23: Minimum and maximum deck bending moment under ULS-1

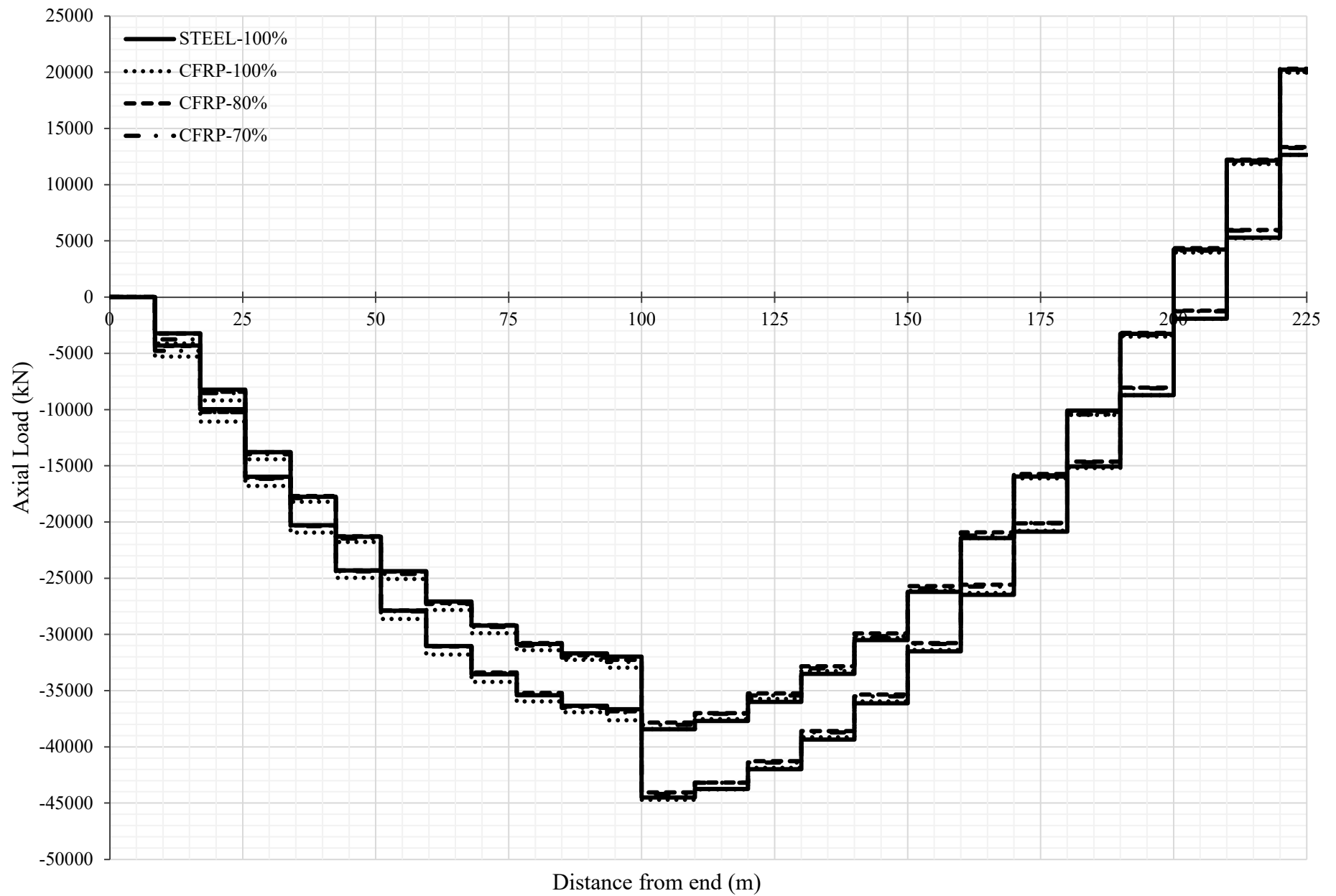


Figure B-24: Minimum and maximum deck axial load under ULS-2-T+

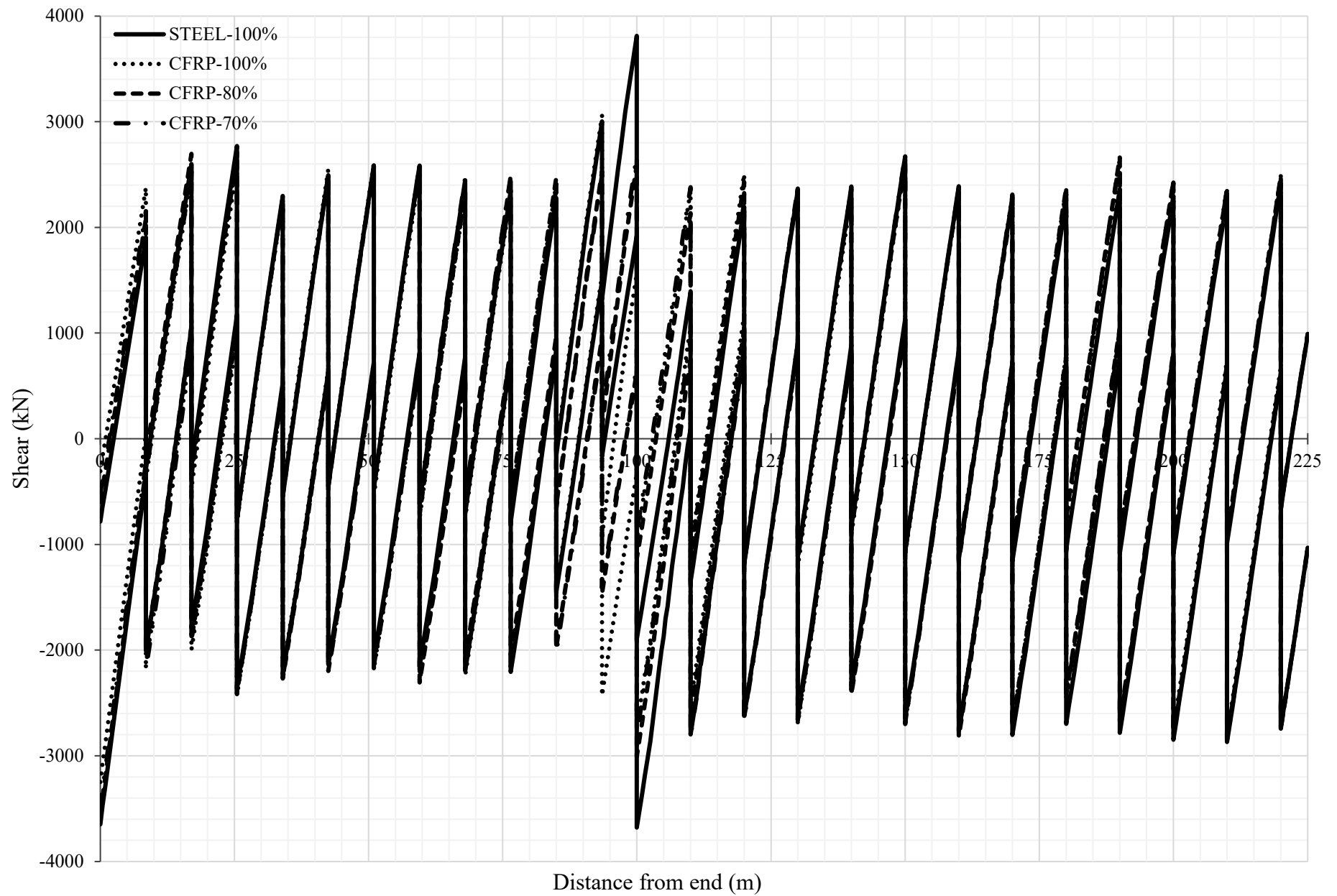


Figure B-25: Minimum and maximum deck shear under ULS-2-T+

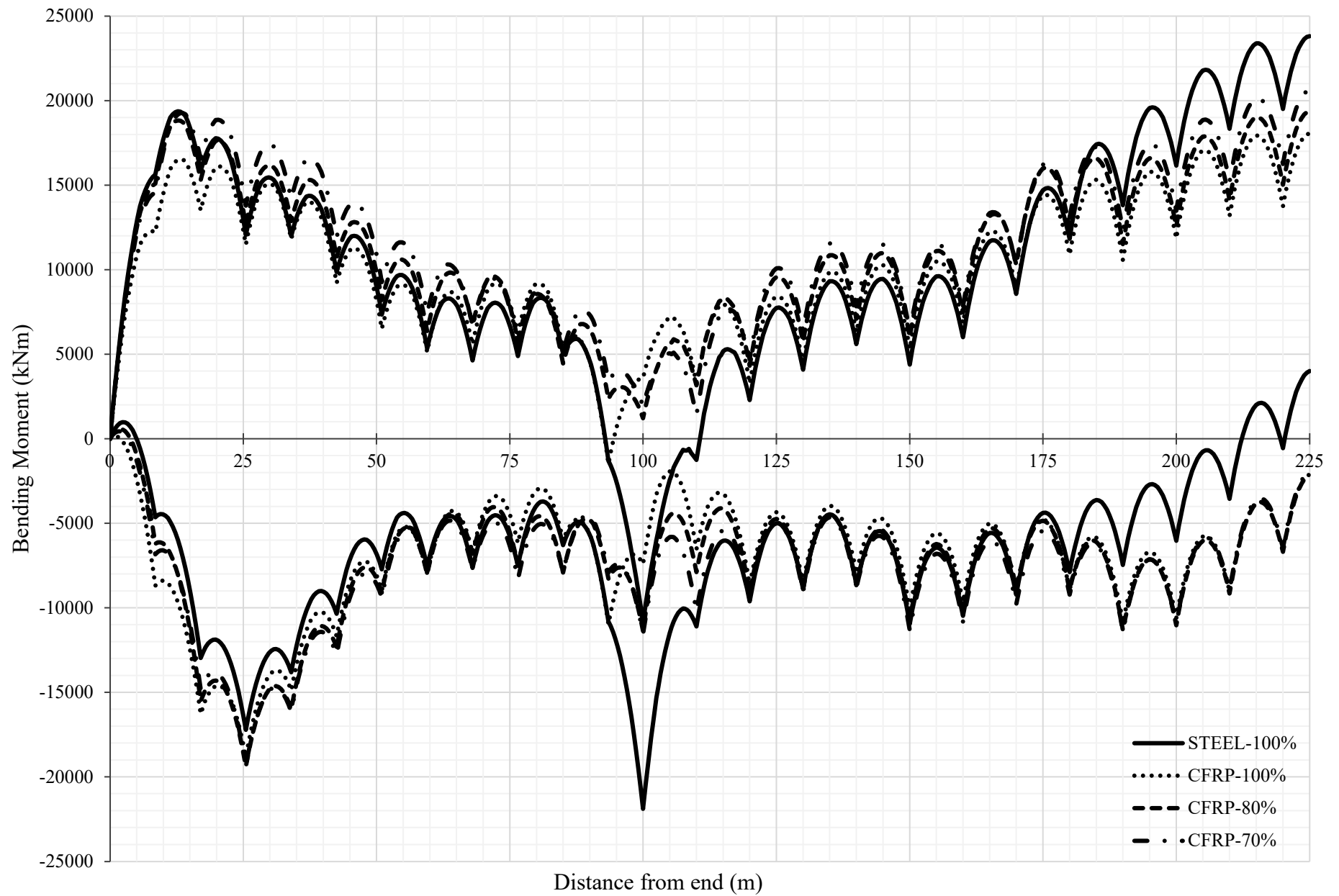


Figure B-26: Minimum and maximum deck bending moment under ULS-2-T+

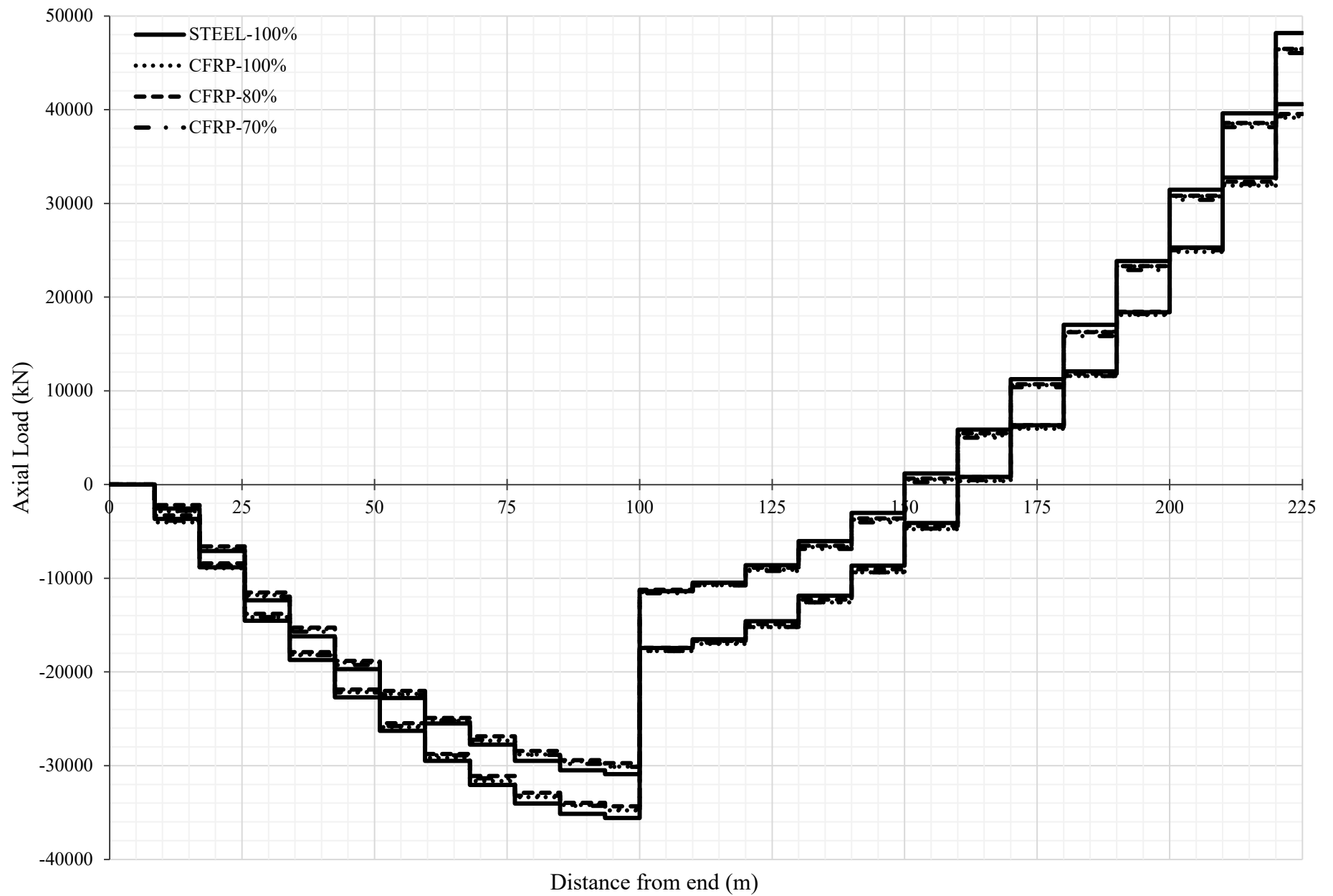


Figure B-27: Minimum and maximum deck axial load under ULS-2-T-

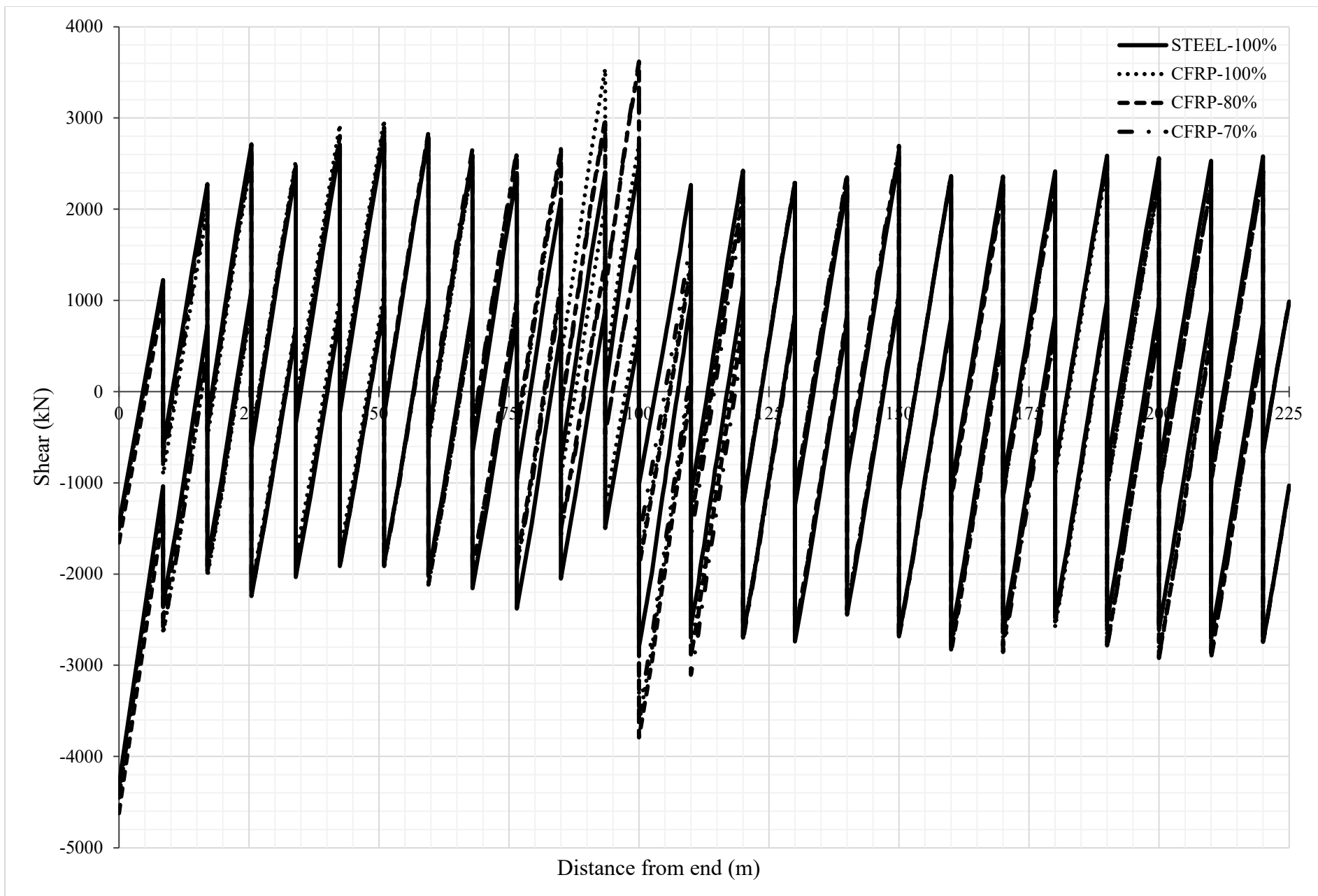


Figure B-28: Minimum and maximum deck shear under ULS-2-T-

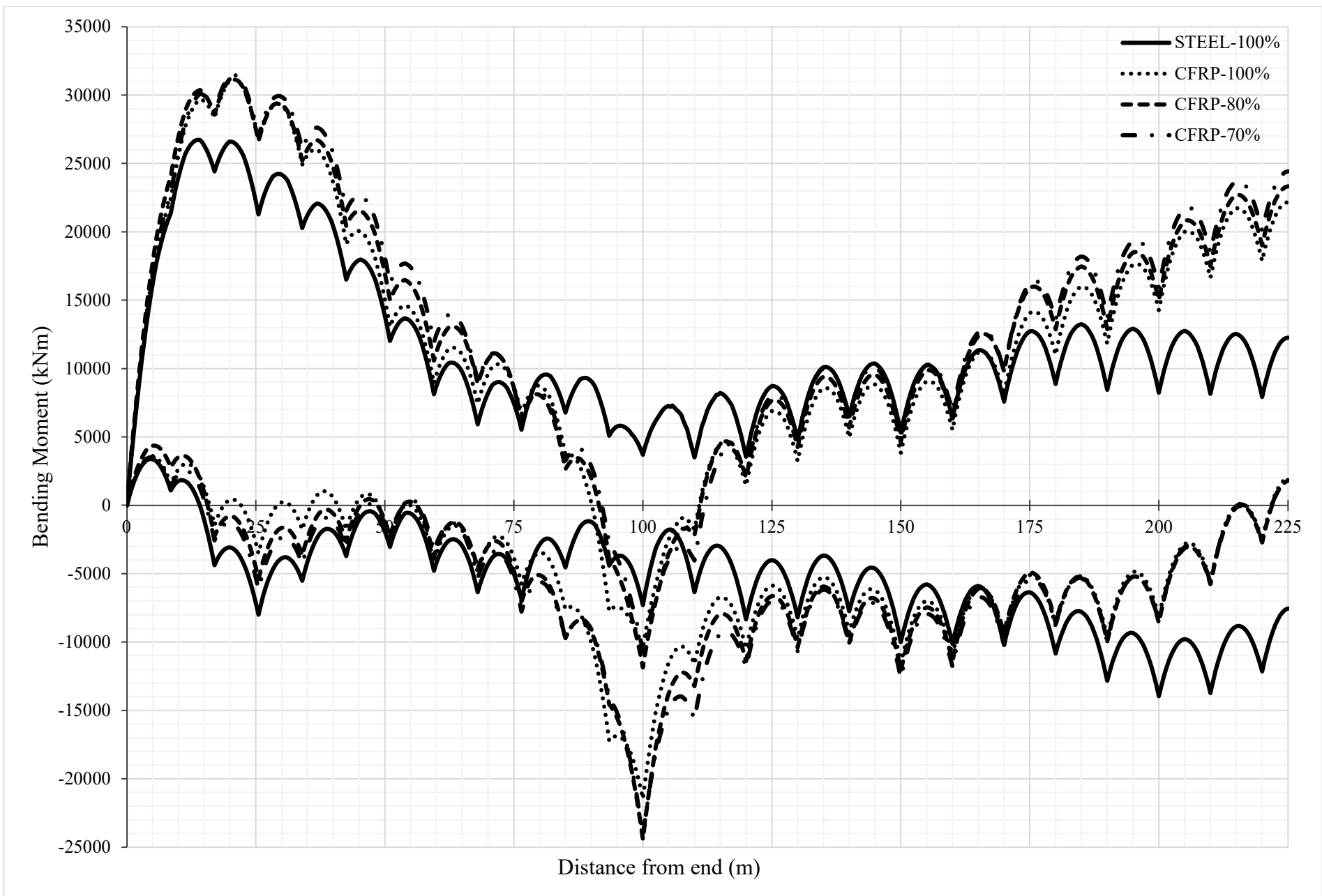


Figure B-29: Minimum and maximum deck bending moment under ULS-2-T-

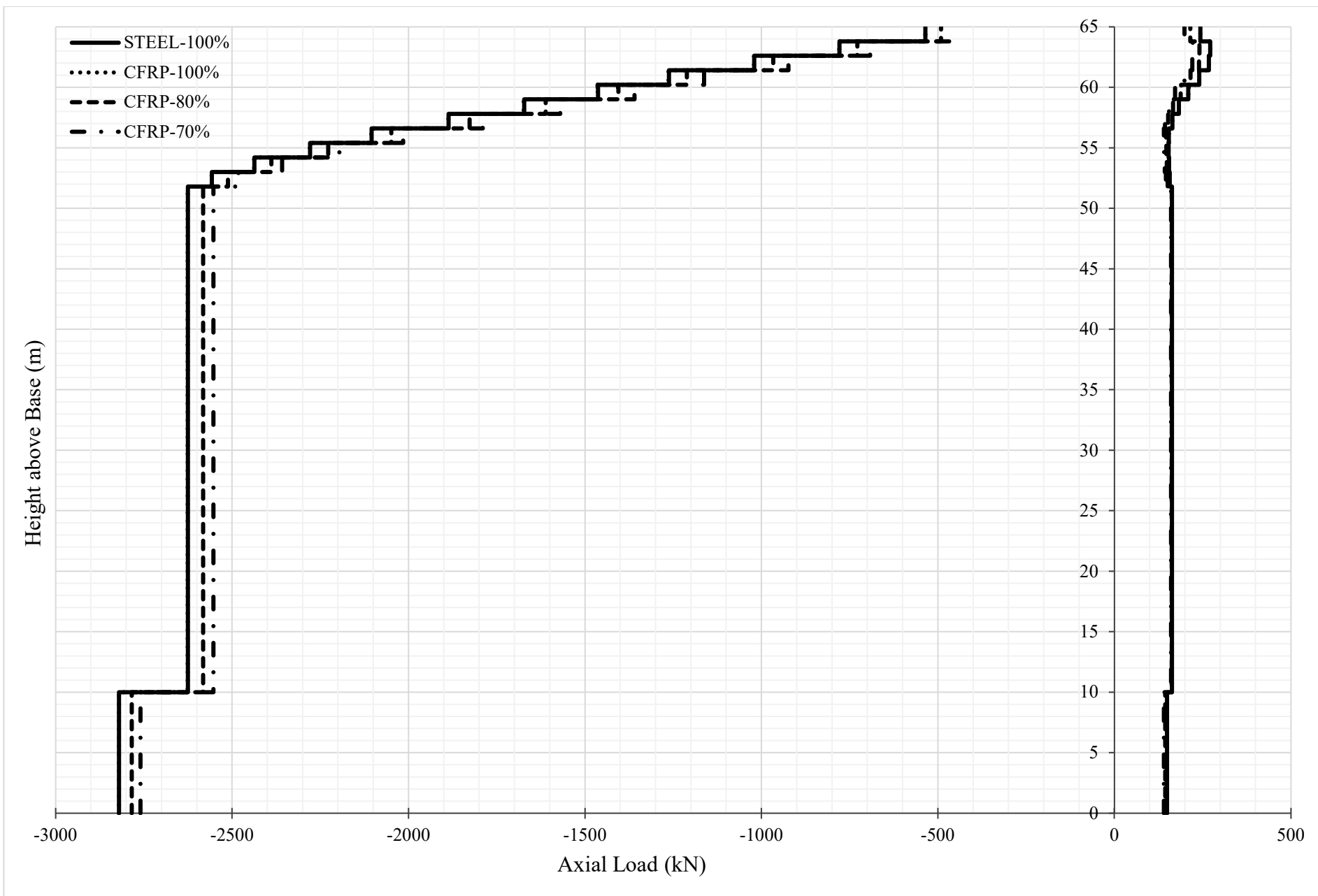


Figure B-30: Minimum and maximum pylon leg axial load under LL ENV

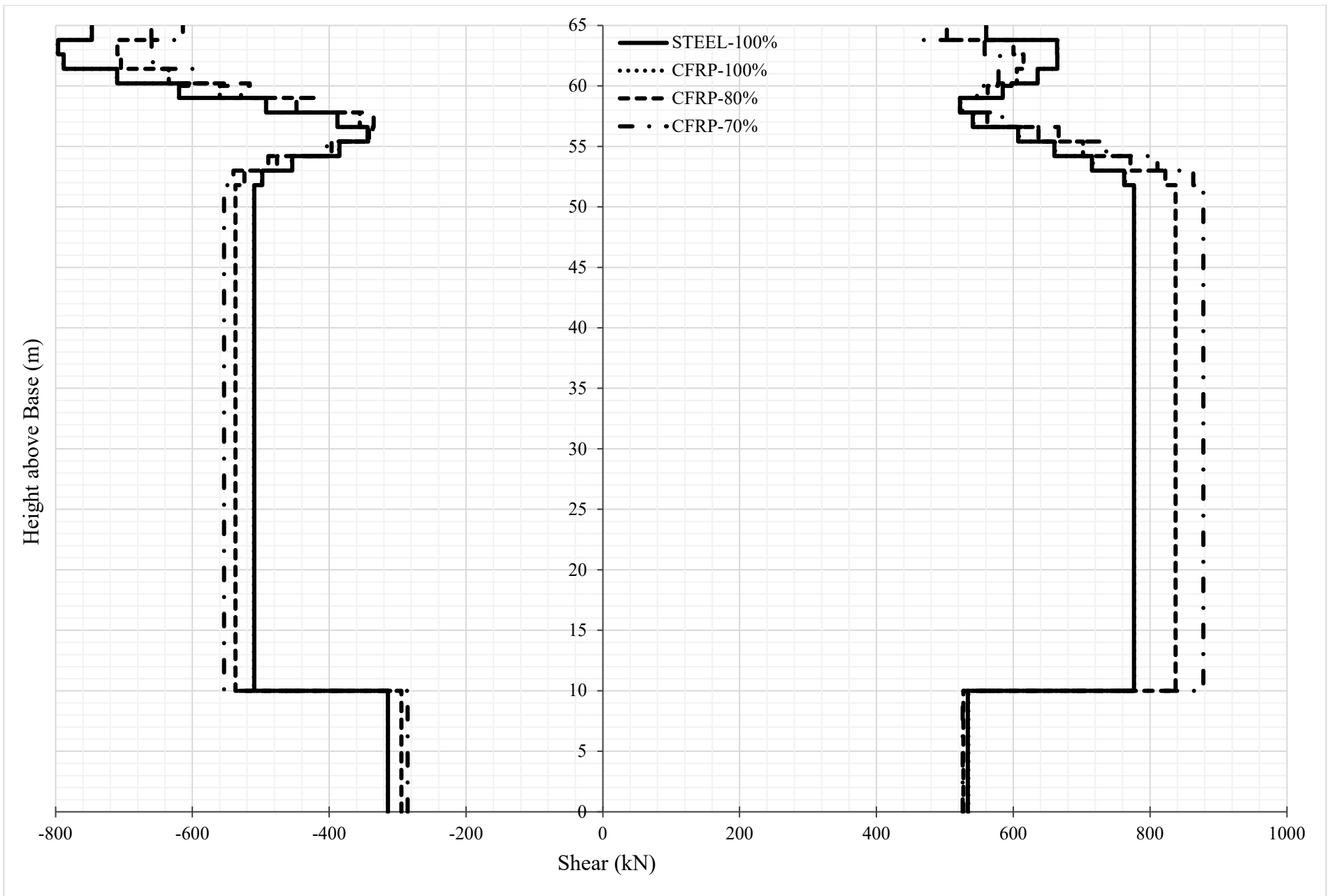


Figure B-31: Minimum and maximum pylon leg shear under LL ENV

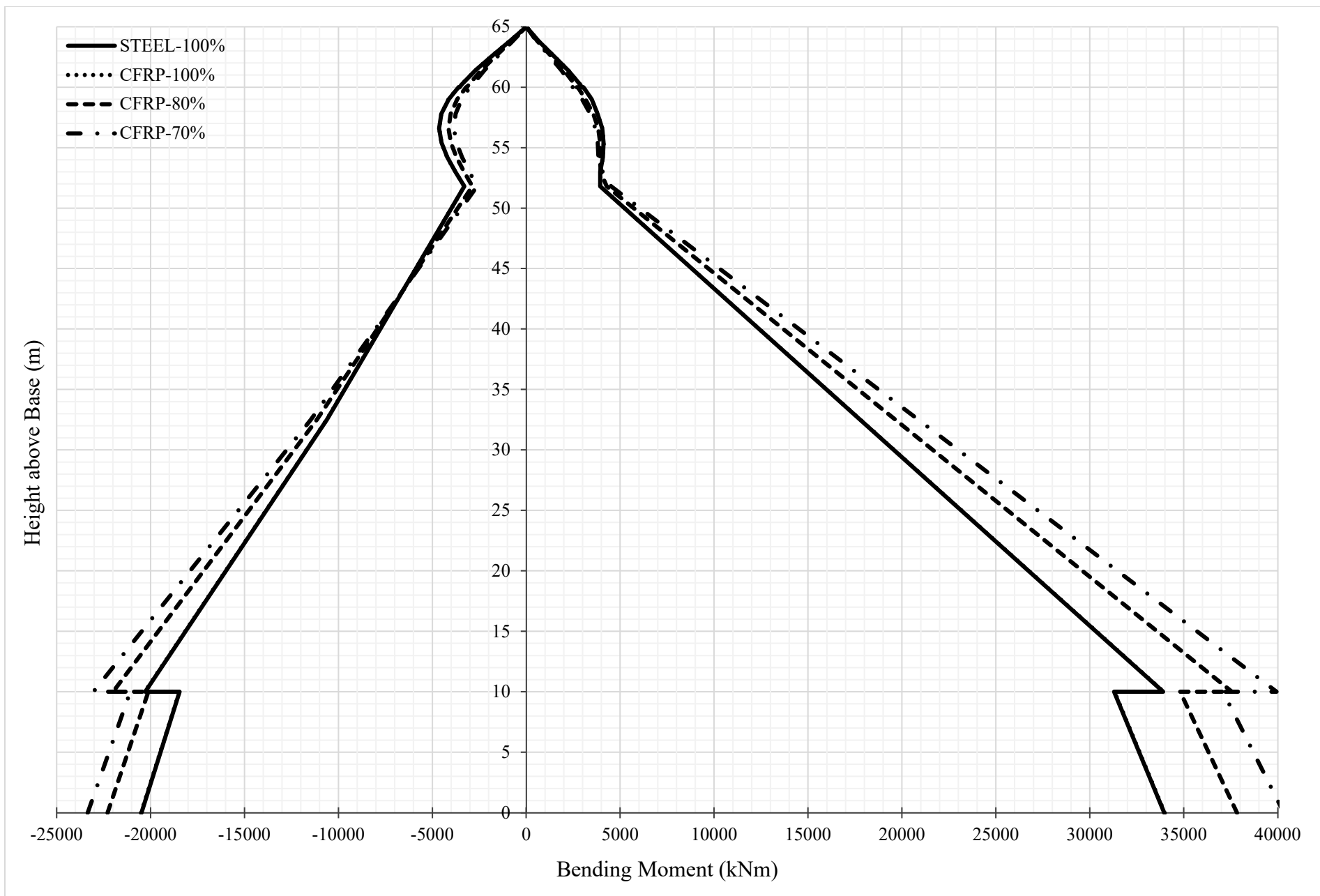


Figure B-32: Minimum and maximum pylon leg bending moment under LL ENV

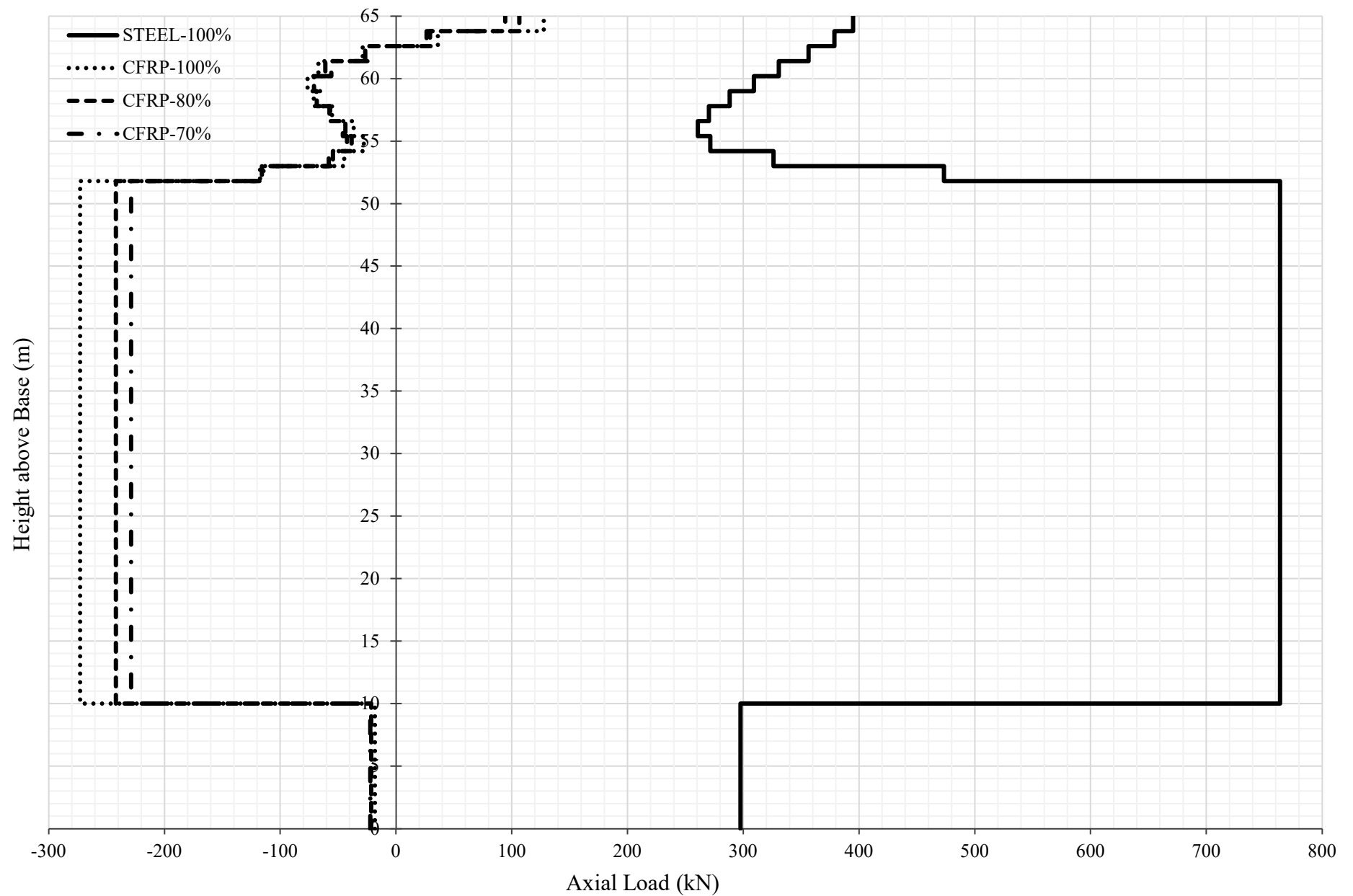


Figure B-33: Pylon leg axial load under T+

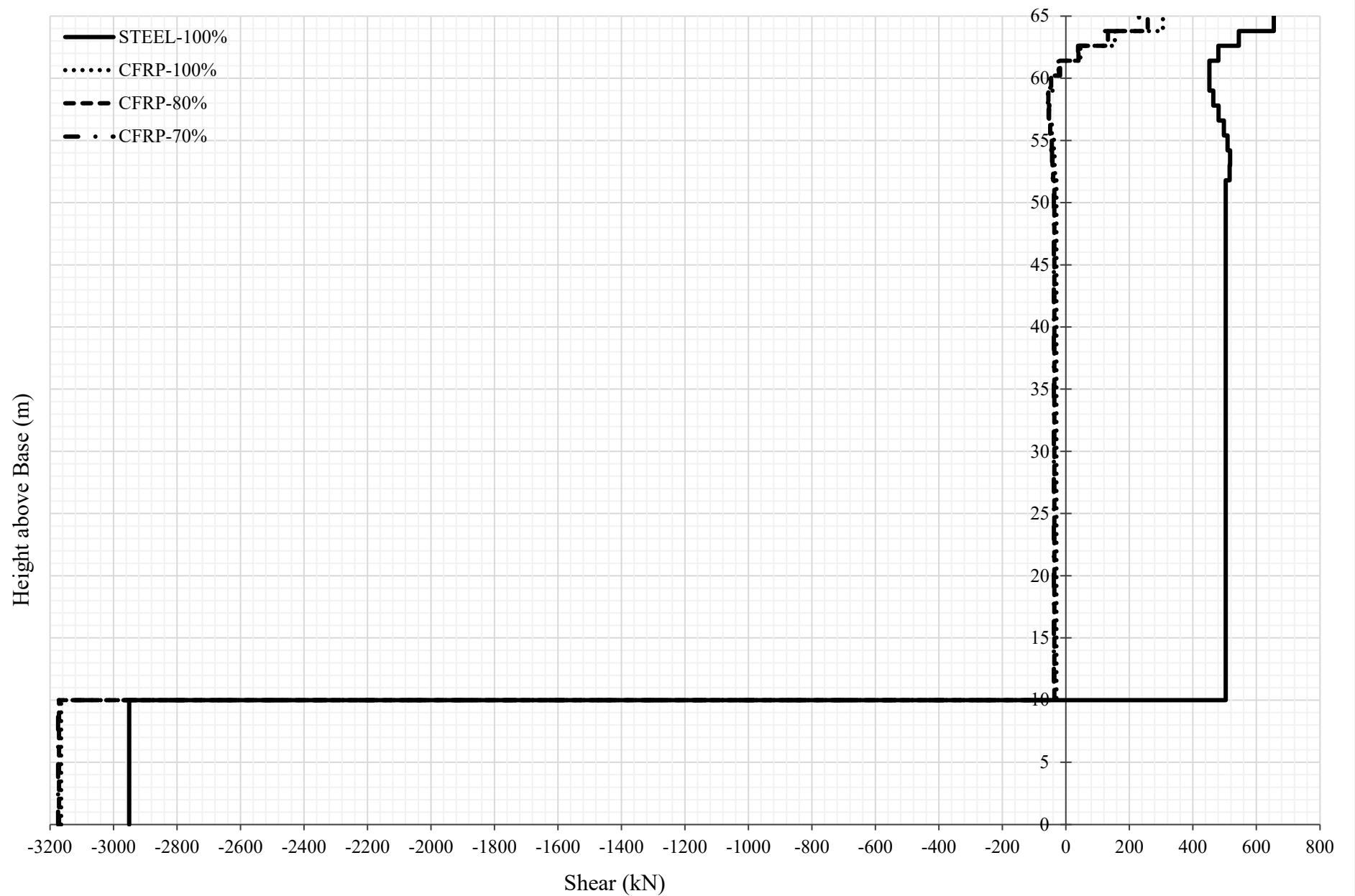


Figure B-34: Pylon leg shear under T+

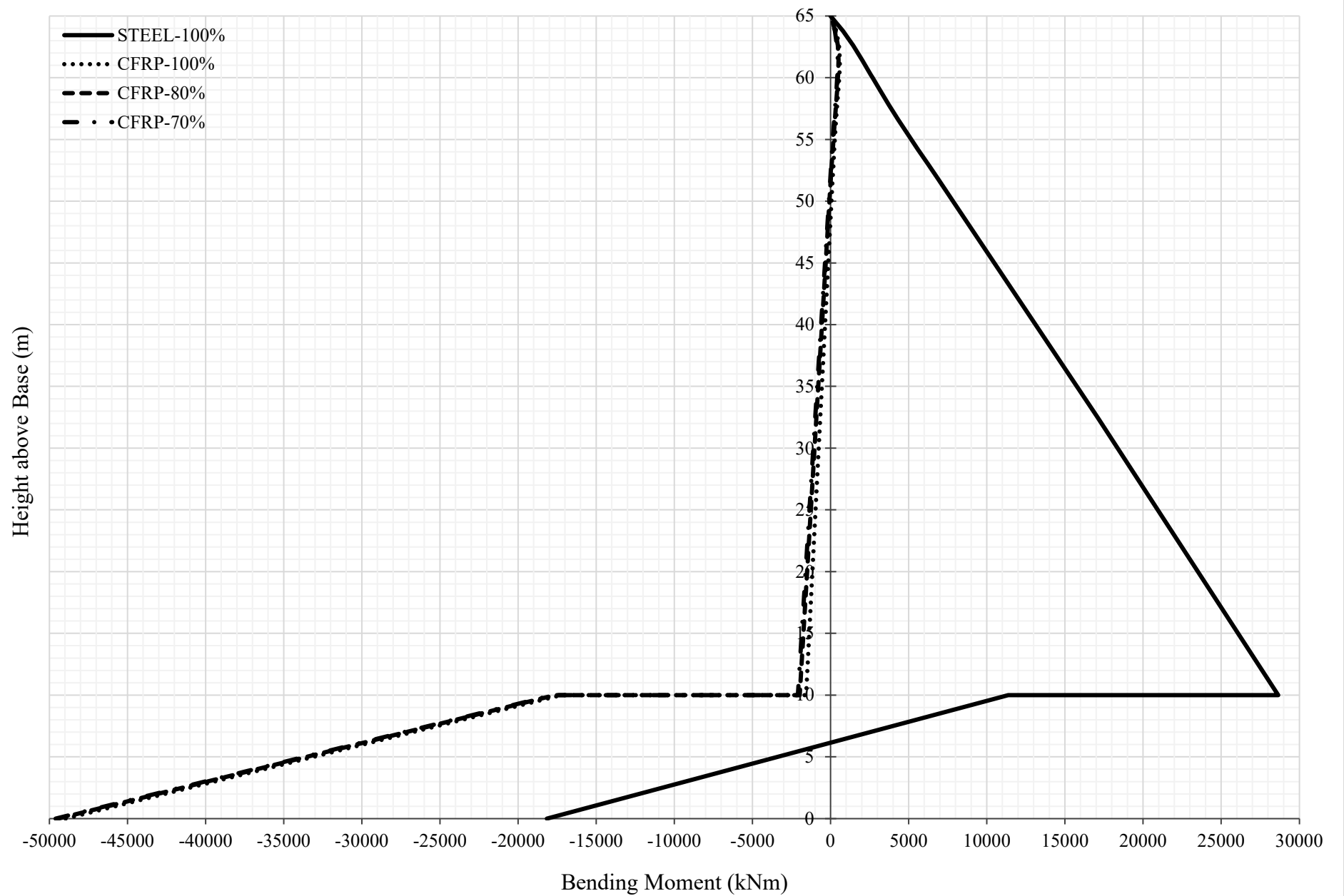


Figure B-35: Pylon leg bending moment under T+

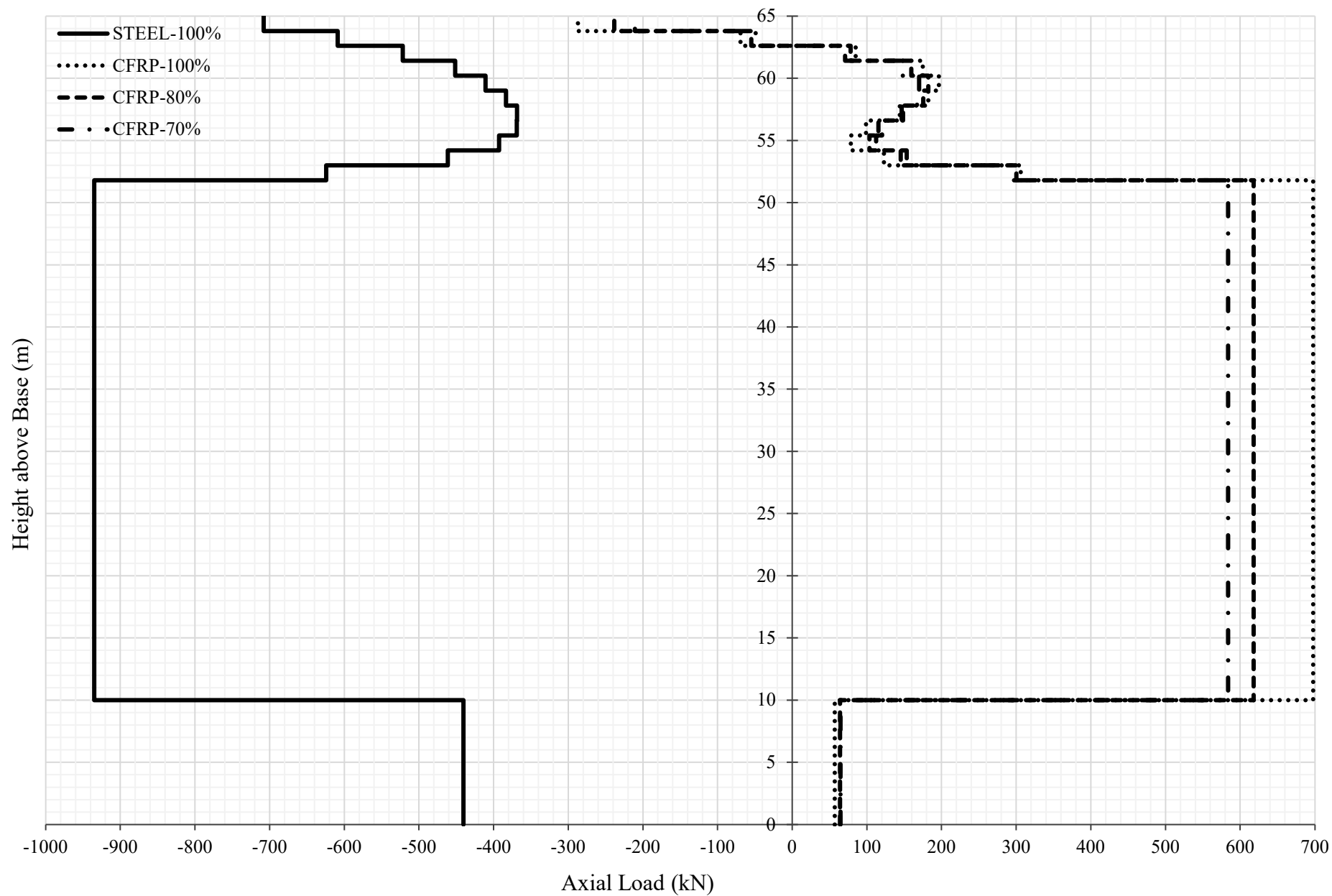


Figure B-36: Pylon leg axial load under T-

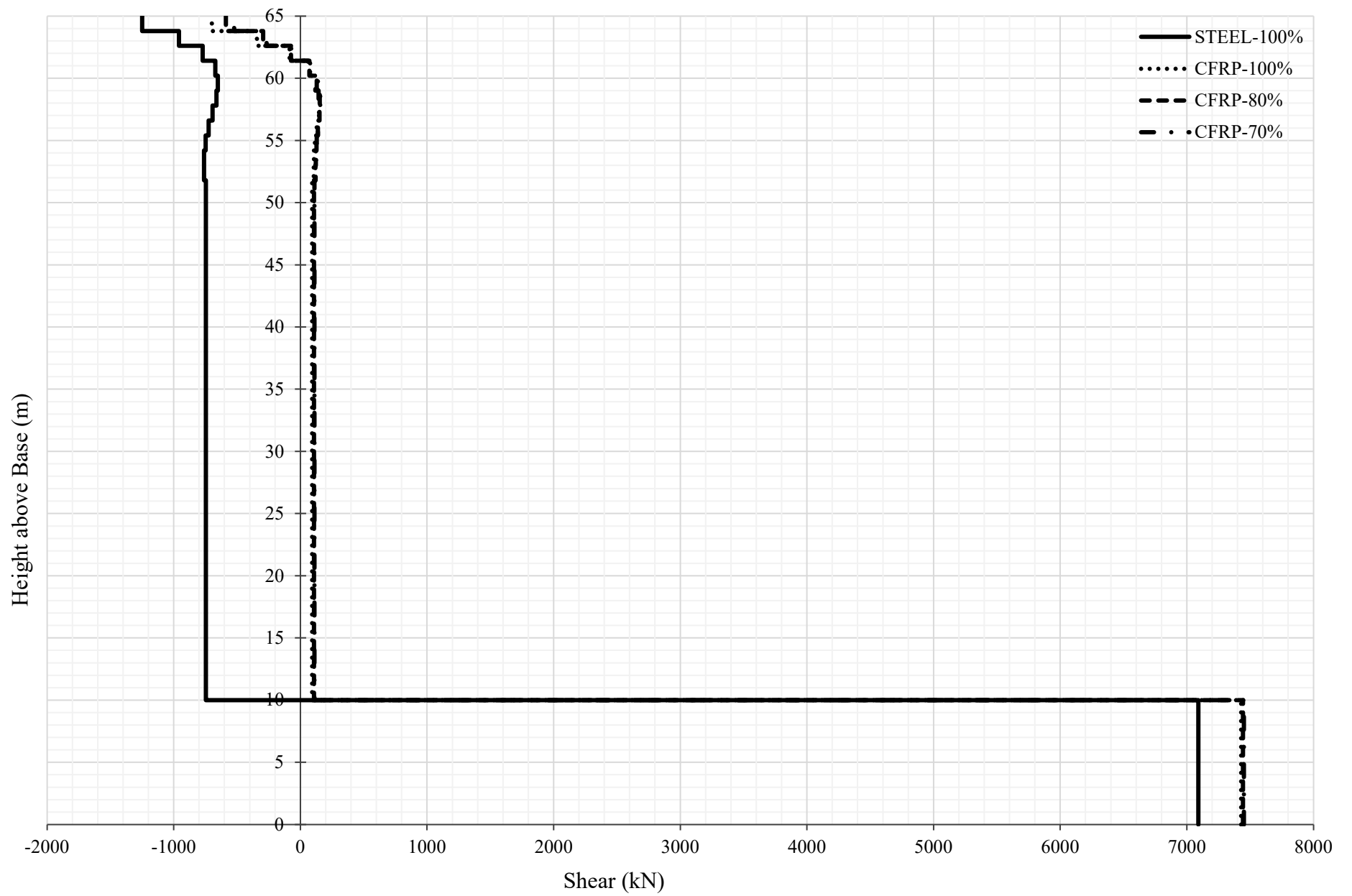


Figure B-37: Pylon leg shear under T-

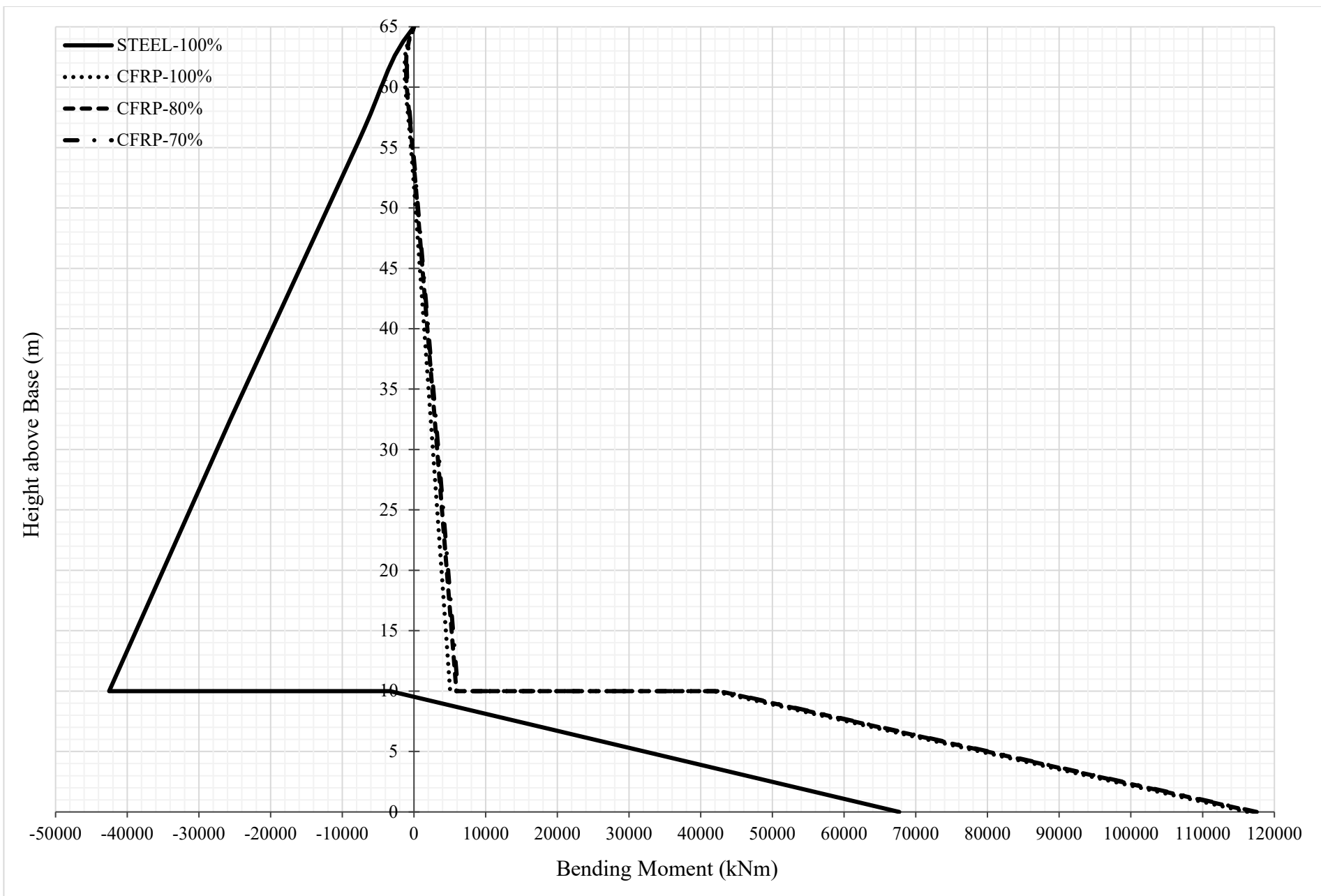


Figure B-38: Pylon leg bending moment under T-

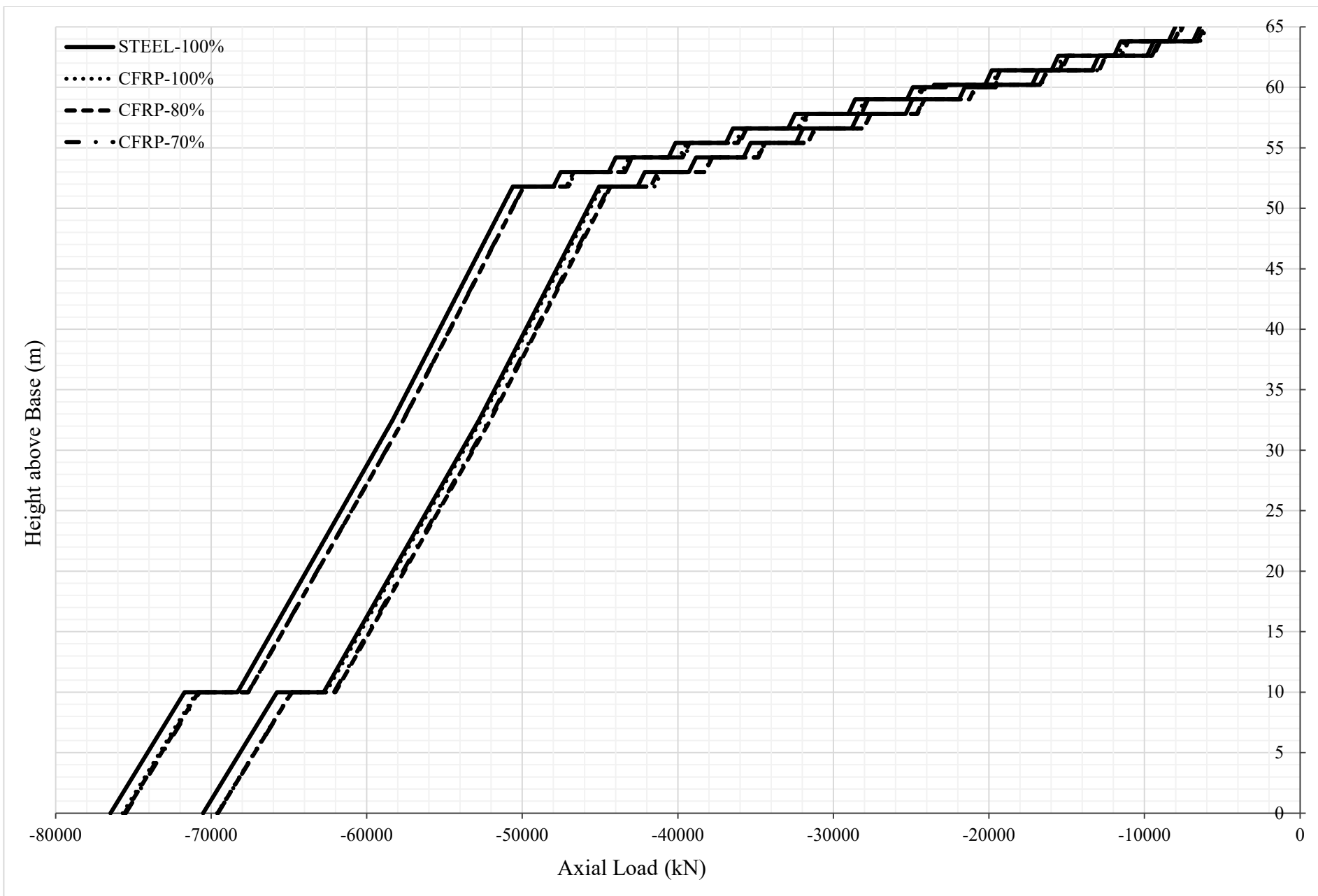


Figure B-39: Minimum and maximum pylon leg axial load under ULS-1

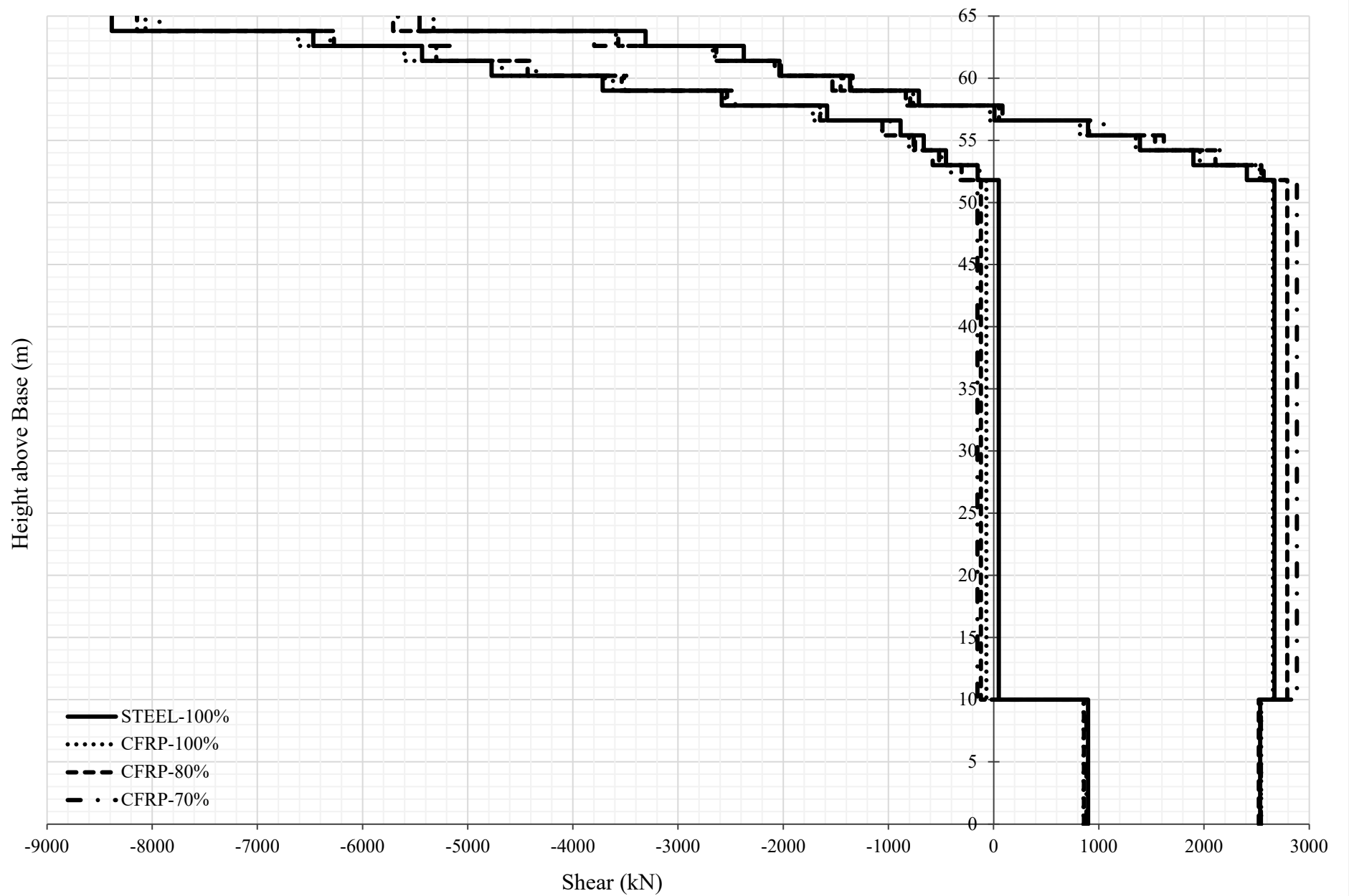


Figure B-40: Minimum and maximum pylon leg shear under ULS-1

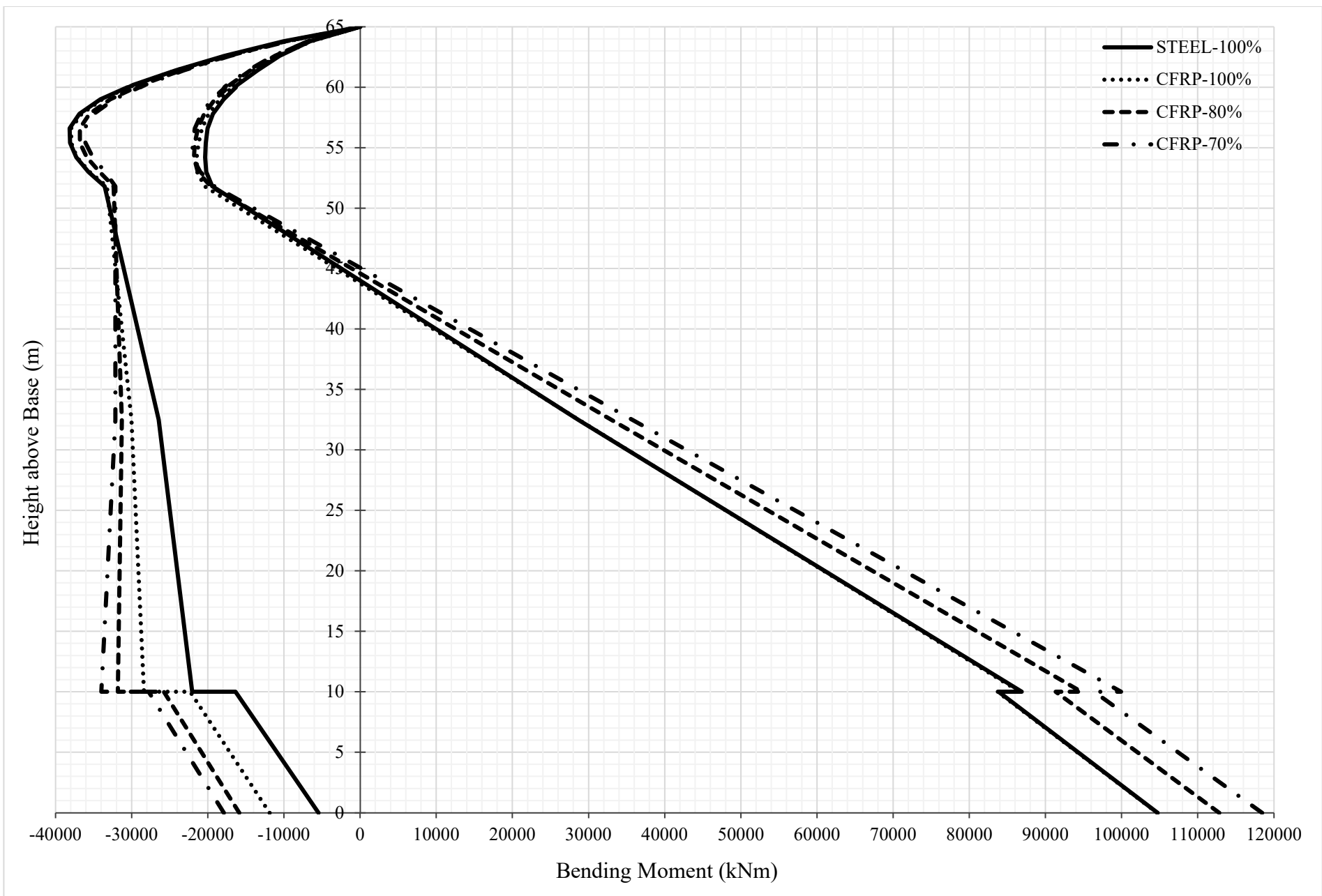


Figure B-41: Minimum and maximum pylon leg bending moment under ULS-1

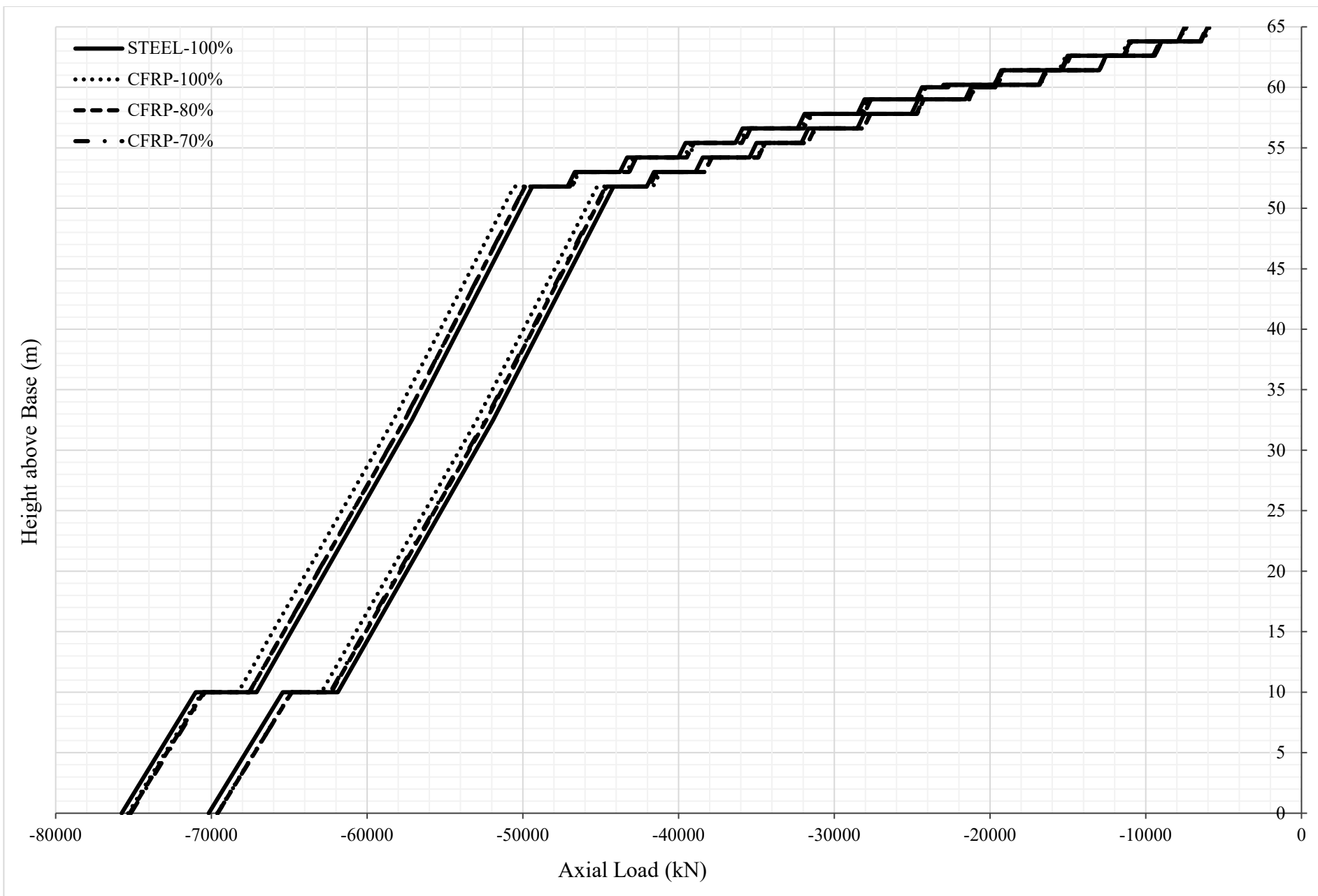


Figure B-42: Minimum and maximum pylon leg axial load under ULS-2-T+

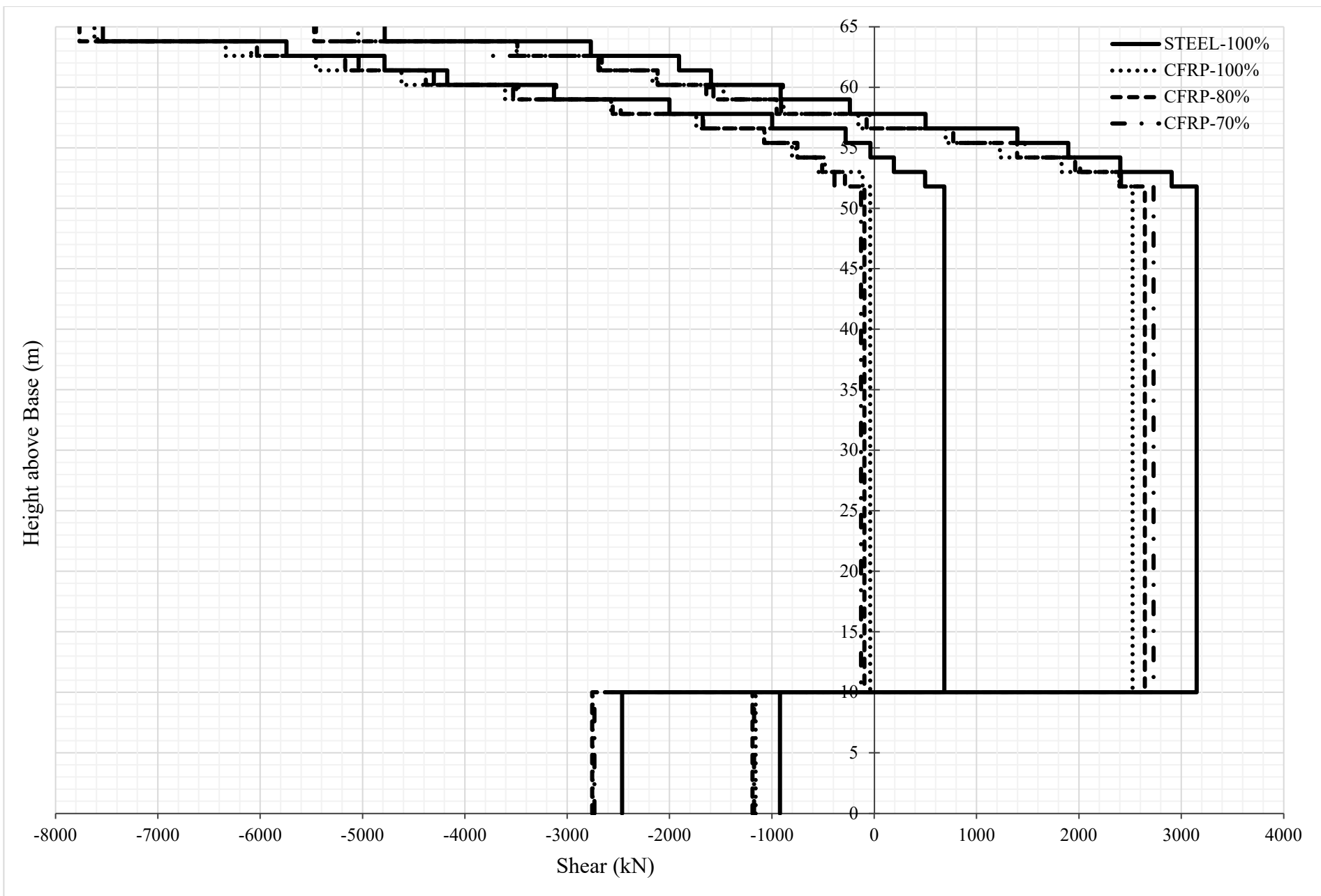


Figure B-43: Minimum and maximum pylon leg shear under ULS-2-T+

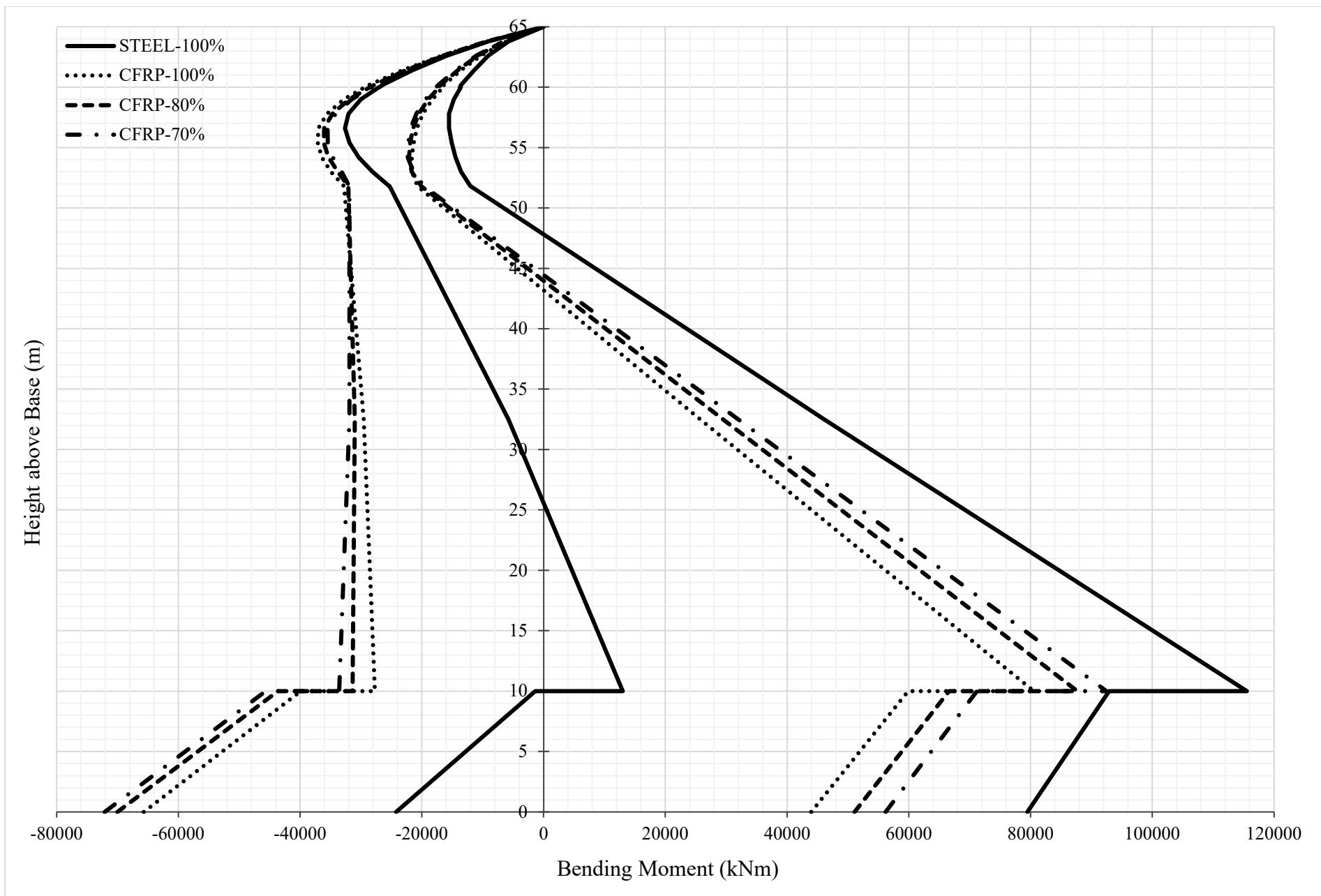


Figure B-44: Minimum and maximum pylon leg bending moment under ULS-2-T+

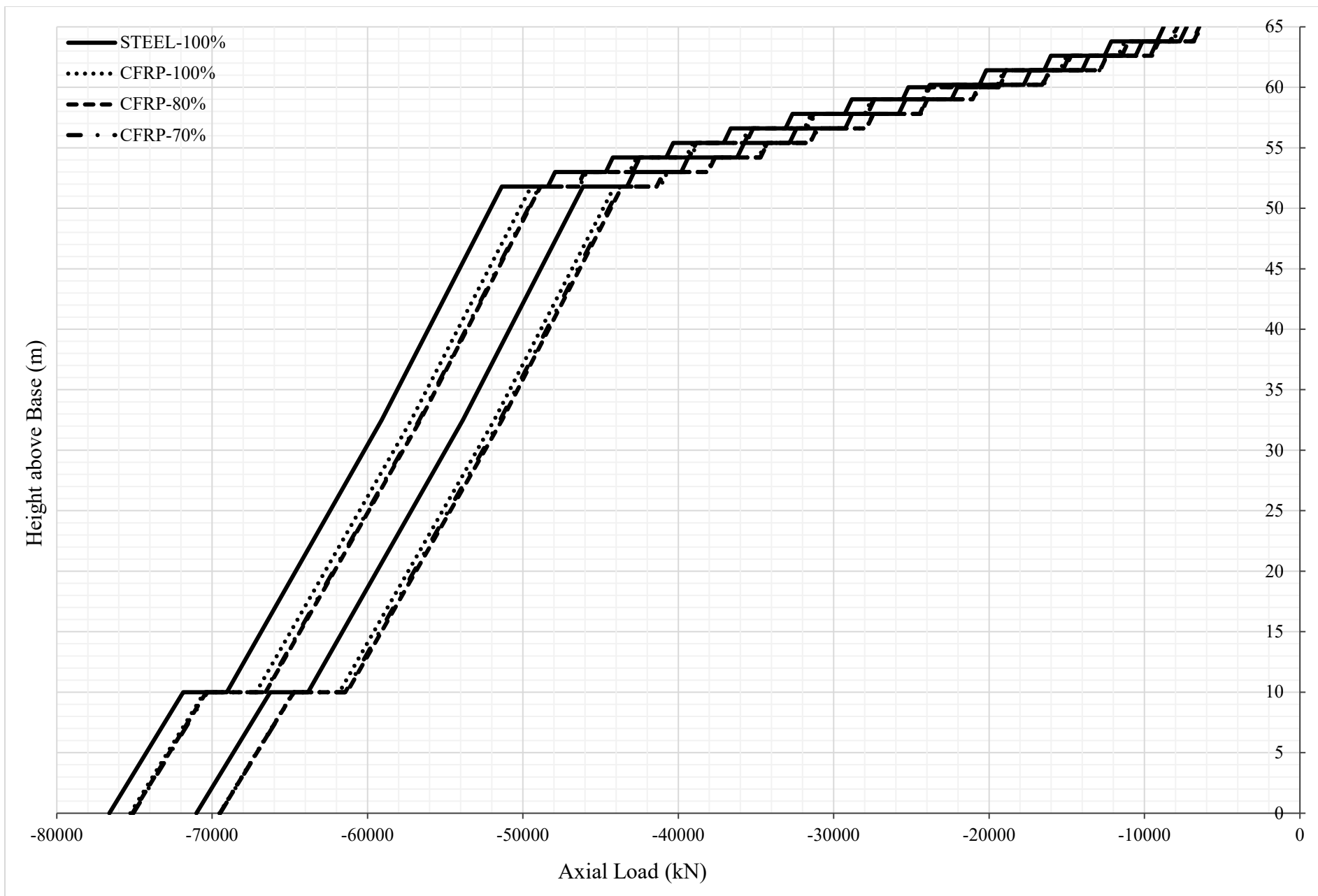


Figure B-45: Minimum and maximum pylon leg axial load under ULS-2-T-

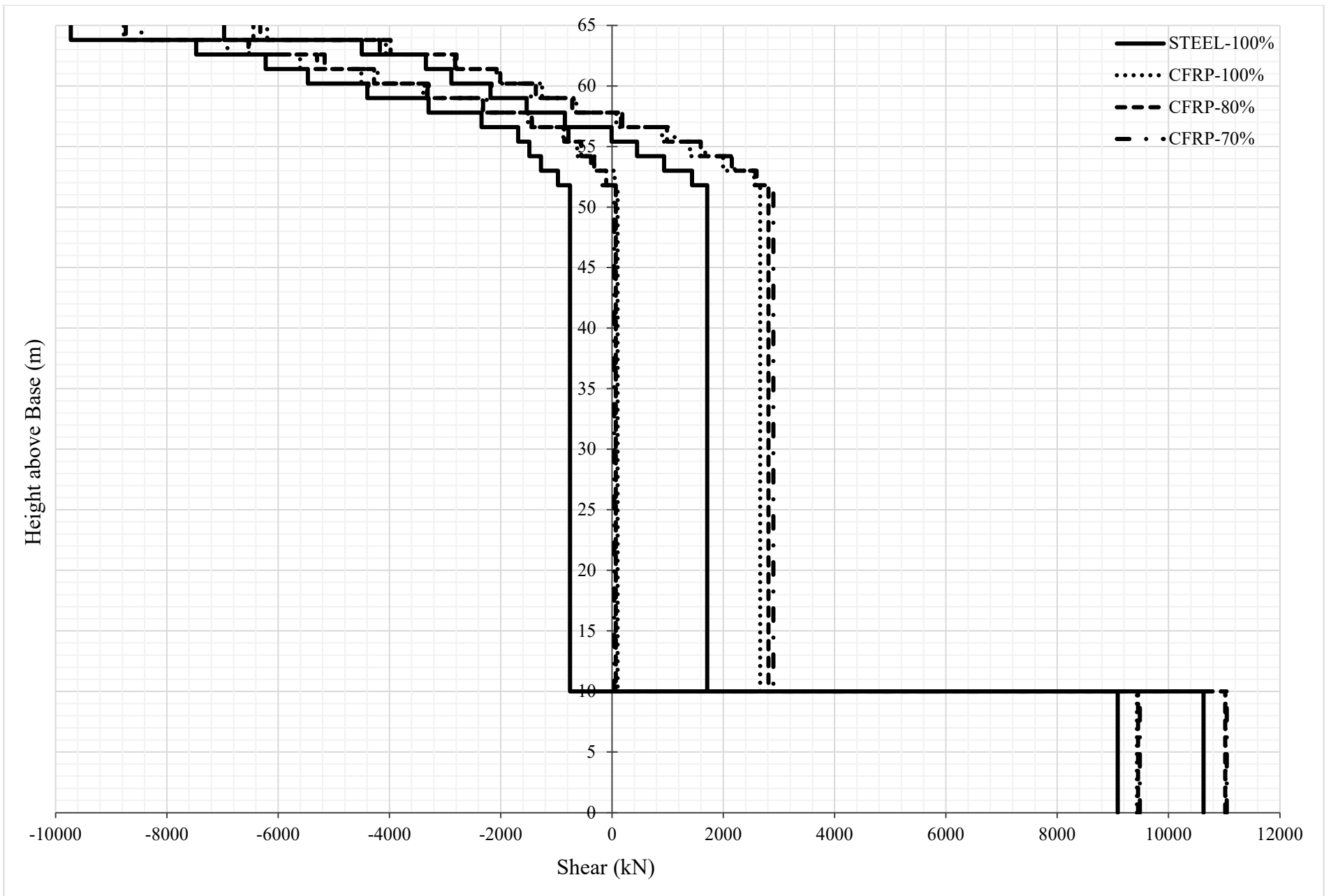


Figure B-46: Minimum and maximum pylon leg shear under ULS-2-T-

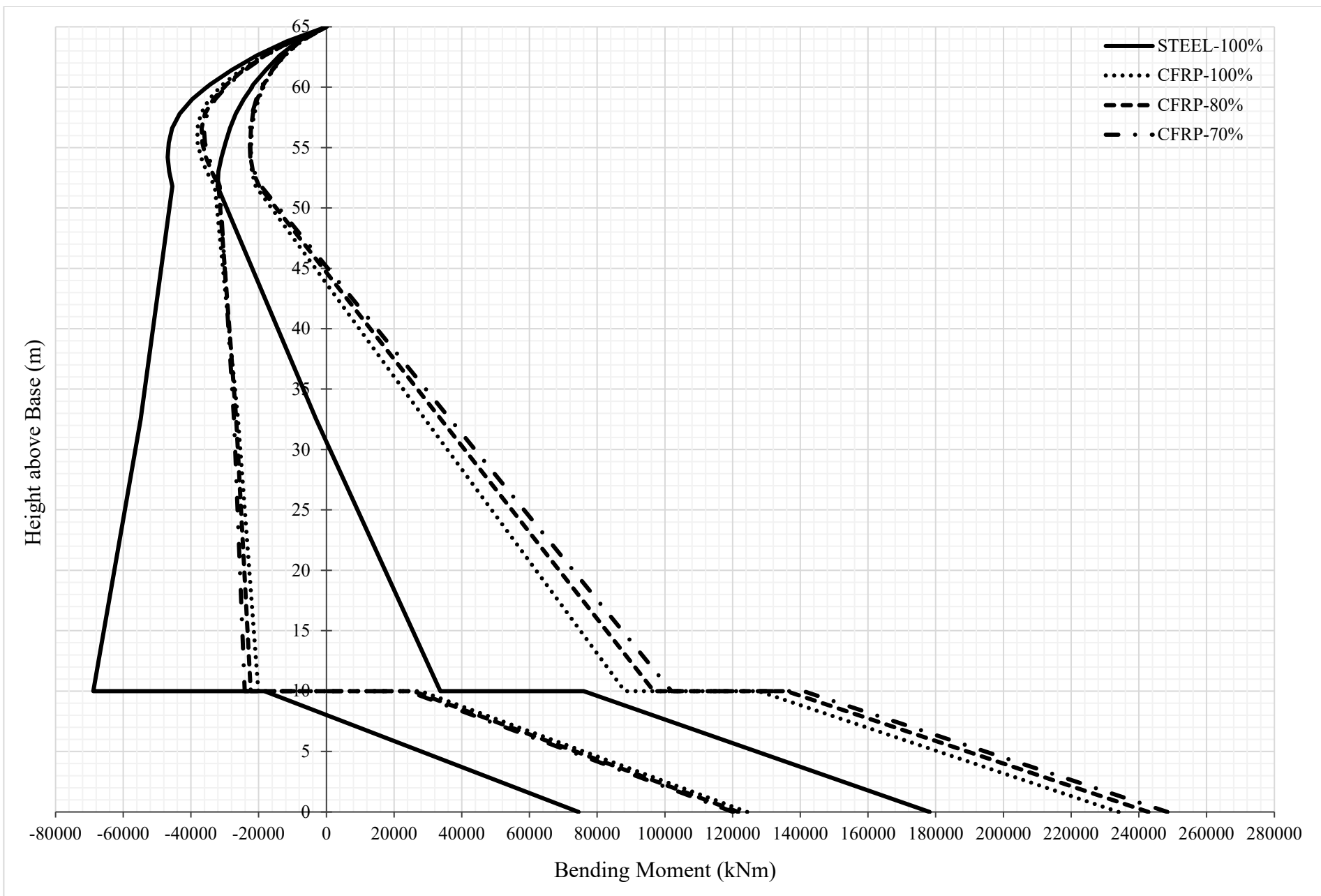


Figure B-47: Minimum and maximum pylon leg bending moment under ULS-2-T-

Appendix C. Phase I Bridge Frequencies

Mode #	Frequency (Cyc/sec)				
	STEEL-190	CFRP-150	CFRP-190	CFRP-300	CFRP-400
1	0.4317	0.4021	0.4357	0.5127	0.5703
2	0.5797	0.5497	0.5838	0.6503	0.6503
3	0.6397	0.6503	0.6504	0.6521	0.6521
4	0.6414	0.6521	0.6521	0.6567	0.7052
5	0.8772	0.8162	0.8839	0.9145	0.9146
6	0.9066	0.9140	0.9141	1.0353	1.1449
7	0.9945	0.9181	1.0014	1.1453	1.1860
8	1.0842	1.0044	1.0908	1.1885	1.3239
9	1.0871	1.0760	1.0967	1.2873	1.4309
10	1.3155	1.2374	1.3228	1.5223	1.6704
11	1.5925	1.5172	1.6016	1.8013	1.9515
12	1.7130	1.6830	1.7795	1.8833	1.9629
13	1.7696	1.7407	1.7831	2.0135	2.1526
14	1.8212	1.7420	1.8311	2.0525	2.1917
15	1.8653	1.8547	1.9156	2.0574	2.2370
16	2.0780	1.9679	2.0893	2.2616	2.4051
17	2.1294	2.0225	2.1579	2.4528	2.4563
18	2.4351	2.3419	2.4492	2.5397	2.7522
19	2.5407	2.4479	2.5682	2.7567	2.9379
20	2.5627	2.5156	2.5768	3.0079	3.2009
21	2.9570	2.9029	2.9735	3.2237	3.3701
22	3.0394	2.9948	3.0559	3.2400	3.4355
23	3.3117	3.2849	3.3299	3.4351	3.4356
24	3.4165	3.4339	3.4343	3.4352	3.5381
25	3.4165	3.4341	3.4344	3.4569	3.6188
26	3.9128	3.8987	3.9343	4.0343	4.1232
27	4.4158	4.4517	4.4520	4.4528	4.4532
28	4.5149	4.4930	4.5380	4.6747	4.8143
29	4.6375	4.6297	4.6640	4.7603	4.8452
30	4.8325	4.8205	4.8593	4.9667	5.0573

Note : first bending frequencies (mode number 1) and first torsional frequencies are highlighted for each bridge.

Appendix D. Phase II Bridge Frequencies

Mode #	Frequency (Cyc/sec)			
	STEEL-100%	CFRP-100%	CFRP-80%	CFRP-70%
1	0.4317	0.4357	0.4064	0.3894
2	0.5797	0.5838	0.5514	0.5353
3	0.6397	0.6504	0.6509	0.6512
4	0.6414	0.6521	0.6526	0.6529
5	0.8772	0.8839	0.8267	0.7905
6	0.9066	0.9141	0.9144	0.8844
7	0.9945	1.0014	0.9268	0.9145
8	1.0842	1.0908	1.0071	0.9689
9	1.0871	1.0967	1.0779	1.0671
10	1.3155	1.3228	1.2410	1.2038
11	1.5925	1.6016	1.5214	1.4843
12	1.7130	1.7795	1.6899	1.6461
13	1.7696	1.7831	1.7473	1.7077
14	1.8212	1.8311	1.7484	1.7283
15	1.8653	1.9156	1.8642	1.8341
16	2.0780	2.0893	1.9852	1.8912
17	2.1294	2.1579	2.0236	1.9957
18	2.4351	2.4492	2.3649	2.2552
19	2.5407	2.5682	2.4489	2.4485
20	2.5627	2.5768	2.5178	2.4934
21	2.9570	2.9735	2.9068	2.8782
22	3.0394	3.0559	2.9986	2.9725
23	3.3117	3.3299	3.2866	3.2689
24	3.4165	3.4343	3.4349	3.4351
25	3.4165	3.4344	3.4350	3.4353
26	3.9128	3.9343	3.9003	3.8864
27	4.4158	4.4520	4.4531	4.4537
28	4.5149	4.5380	4.4991	4.4793
29	4.6375	4.6640	4.6324	4.6182
30	4.8325	4.8593	4.8239	4.8071

Note : first bending frequencies (mode number 1) and first torsional frequencies are highlighted for each bridge.