MAY 2019 M.Sc. in Civil Engineering ADİL BOYBEYİ

HASAN KALYONCU UNIVERSITY

GRADUATE SCHOOL OF

NATURAL & APPLIED SCIENCES

A PARAMETRIC STUDY ON THE EFFECT OF POST-TENSIONING IN BRIDGE PIER FOUNDATION

M. Sc. THESIS

IN

CIVIL ENGINEERING

BY

ADİL BOYBEYİ

MAY 2019

A Parametric Study on the Effect of Post-tensioning in Bridge Pier Foundation

M.Sc. Thesis

in

Civil Engineering

Hasan Kalyoncu University

Supervisor

Assist. Prof. Volkan KALPAKCI

by

ADIL BOYBEYI

MAY 2019

GRADUATE SCHOOL OF NATURAL & APPLIED SCIENCESINSTITUTE
M.Sc. ACCEPTANCE AND APPROVAL FORM

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I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all material and results that are not original to this work.

Adil BOYBEYİ

ABSTRACT

A PARAMETRIC STUDY ON THE EFFECT OF POST-TENSIONING IN BRIDGE PIER FOUNDATIONS

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Thesis Advisor: Asist. Prof. Dr. Volkan KALPAKCI
May, 2019; 73 pages

In this study, the effect of post-tensioning on a bridge pier foundation of a two span bridge was investigated by finite element analyses. For this purpose, a formerly constructed real bridge was selected and the pier foundation of this bridge was analysed for three different post-tensioning tendons and for three different tendon spacings making a total of nine different cases. The results were compared in the meaning of maximum and minimum base pressures, base pressure distributions; bending moments and their distributions at six different cross-sections in the longitudinal and transverse directions. The results have revealed that; without post-tensioning a significant pressure concentration was observed beneath the columns. While much more uniform base pressure distributions were obtained for most of the post-tensioned foundations at the expense of increased base pressures at the edges of the foundation. This fact was expected since the total vertical force was not changed but the deformation profile of the foundation was improved with the help of post-tensioning. Similar to the case in base pressures, moment distributions were also more uniform for most of the cases with post-tensioning. But after an optimum point for tendon type and spacing, the moment directions were altered which may have a diverse affect in the design. Another advantage of the application of post-tensioning was that the whole foundation was under compression which had a positive effect on the long term behaviour of bridge foundations against cracking and corrosion.

Key Words: Post-tension, foundation, bridge, bearing and moment

ÖZET

KÖPRÜ TEMELLERİNDE ARDGERME ETKİSİ ÜZERİNE PARAMETRİK BİR ÇALIŞMA

BOYBEYİ, Adil
Yüksek Lisans Tezi, İnşaat Mühendisliği Bölümü
Tez Yöneticisi: Dr. Ögretim Üyesi Volkan KALPAKCI
Mayıs, 2019; 73 sayfa

Bu çalışmada, iki açıklıklı bir köprünün orta ayak temelinde ardgerme etkisi sonlu elemanlar analizleri ile araştırılmıştır. Bu amaçla, daha once inşa edilmiş olan gerçek bir köprünün temeli seçilmiş ve bu temel 3 farklı halat tipi ve 3 farklı halat aralığı için olmak üzere toplam 9 farklı ardgerme opsiyonu ile analiz edilmiştir. Sonuçlar enine ve boyuna toplam 6 kesitte; maksimumve minimum temel basınçları, temel basınç dağılımları; temele gelen eğilme momentleri ve moment dağılımları açısından incelenmiştir. Analiz sonuçlarına gore ardgerme uygulanmadığı durumlarda temellerde kolon altına denk gelen bölgede temel basınçları ciddi şekilde yoğunlaşmaktadır. Ardgerme uygulanan analizlerin çoğunda ardgermesiz duruma gore çok daha düzgün yayılı bir basınç dağılımı elde edilirken, temel uç basınçlarında bir miktar artış görülmüştür. Bu etkinin, toplam düşey yükün değişmemesi ancak temelin deformasyon profilinin iyileştirilmesi sebebiyle oluştuğu sonucuna varılmıştır. Temel basınçlarına benzer şekilde moment dağılımlarının da, analiz edilen bir çok durum için ardgerme uygulanması halinde ardgermesiz duruma gore çok daha düzenli olduğu görülmüştür. Ancak halat sayısı ve halat aralıklarında belirli bir optimum noktadan sonra moment yönlerinin değiştiği görülmüştür. Bu durum dikkatli olunmazsa tasarımda olumsuz etkiler yaratabilecektir. Temelde ardgerme uygulanması ile görülen bir diğer avantaj da, tüm temelin basınç altında çalışmasıdır. Bu durumun temelin uzun dönemdeki çatlama ve korozyon problemlerine olumlu yönde katkı yapacağı değerlendirilmiştir.

Anahtar Kelimeler: Ardgerme, temel, köprü, dayanım ve moment

To My Parents

ACKNOWLEDGEMENTS

To begin with I would like to thank the ‘ALLAH’ for the virtues of this blessing for implanting the soul of endurance and faith in myself to complete this study.

I sincerely acknowledge Prof. Dr. M. Yaşar KALTAKCI who has given me the foundations of engineering during my undergraduate years and contributed greatly to the advancement of my thesis with his valuable ideas and comments.

I sincerely acknowledge my advisor; Assist. Prof. Dr. Volkan KALPAKCI, for his valuable explanations, patience and intense interest in detail. Without my advisor, this thesis could not have been completed successfully.

Also, I would like to thank Kınacı Engineering and Con-AK Construction Companies for their valuable help.

Finally, I would like to thank my parents and my fiancé Architect Özge FALAY for supporting me in this process.

TABLE OF CONTENTS

Page

LIST OF FIGURES

Page

LIST OF TABLES

Page

LIST OF SYMBOLS / ABBREVIATIONS

As	Quantity of equipment
α	Equal pressure block depth
B	Basic transverse length
d	Section useful height
d'	Concrete cover
E	Elasticity module
f_{ck}	Concrete characteristic strength
f_c	Characteristic strength
f_{pu}	Rupture strength
f_r	Breaking module
H_x	Vertical load on foundation from direction x
H_y	Vertical load on foundation from direction y
I_g	Moment of inertia according to the gravity center of the section
L	Basic longitudinal length
M_{max}	Maximum moment value
M_{cr}	Cracking moment
M_x	Moment on foundation from direction x
M_y	Moment on foundation from direction y

M_u	Design moment
n	Amount of Tendon
P	Amount of load used
s	Tendon range
q_{max}	Maximum stress value
yt	Lower base distance of the gravity center of the section
ρ_b	Balanced equipment ratio
ρ	Equipment ratio

CHAPTER 1

INTRODUCTION

1.1 Introduction

Post-tensioning is a way of counteracting the effect of external loads on a structure by applying a stress condition that is contrary to the load effect in a reinforced concrete system. This is mostly achieved through the stretched tendons before the final loading of the structure. During the application of these tendons, a certain part of the reinforcing steel reinforcement detail is applied and after the placement of the post-tensioner tendons according to the profile determined by the projector, in a certain form and the first stage is ended with the installation of the jack in the anchorage system and the concrete pouring process. In the second stage, after taking the 75% strength of the concrete, the tendons are stretched with a certain force only on the basis of their self-weight and the jancks are locked and the external loads are successfully terminated. In this way, the concrete section is completely free of tensile forces and works to pressure. In this way, no cracks occur in the concrete section and the reinforcement inside it is protected against corrosion. This process is known as 'posttensioning method' in world literature. The fact that the post-tensioning system is included in the engineering technologies in this way has brought with it many computer programs, researches and applications. The post-tensioning system has been mostly used in bridge superstructures in our country and has found a wide place for itself in housing projects. Its usage has become frequent in many special superstructure projects, the use of large conference rooms, parking lot, raft foundation, bridge and wide-span console balconies.

Post-tensioning concrete structures are generally designed and analyzed with SAFE and SAP 2000 analysis programs. The primary specification for this analysis is TS. 3233 which is a specification of the Calculation and Construction Rules for Post-tensioning Concrete Structures. This specification entered into force in February 1979 and is still valid.

In this study, the middle foot foundation of the Hal Junction Bridge, which is a two span (1 middle, 2 edged) bridge in Denizli Hal Junction, was analyzed for the cases with and without post-tensioning. 3 different types of post tension rope and 3 different spacings were applied in the analyses.

As a result, the change in the pressure and moment distributions of the foundation was examined according to the conditions without post-tensioning for 9 different post-tensioning combinations.

Figure 1.1 Turkey Political Map

Figure 1.2 Denizli Hal Junction Bridge

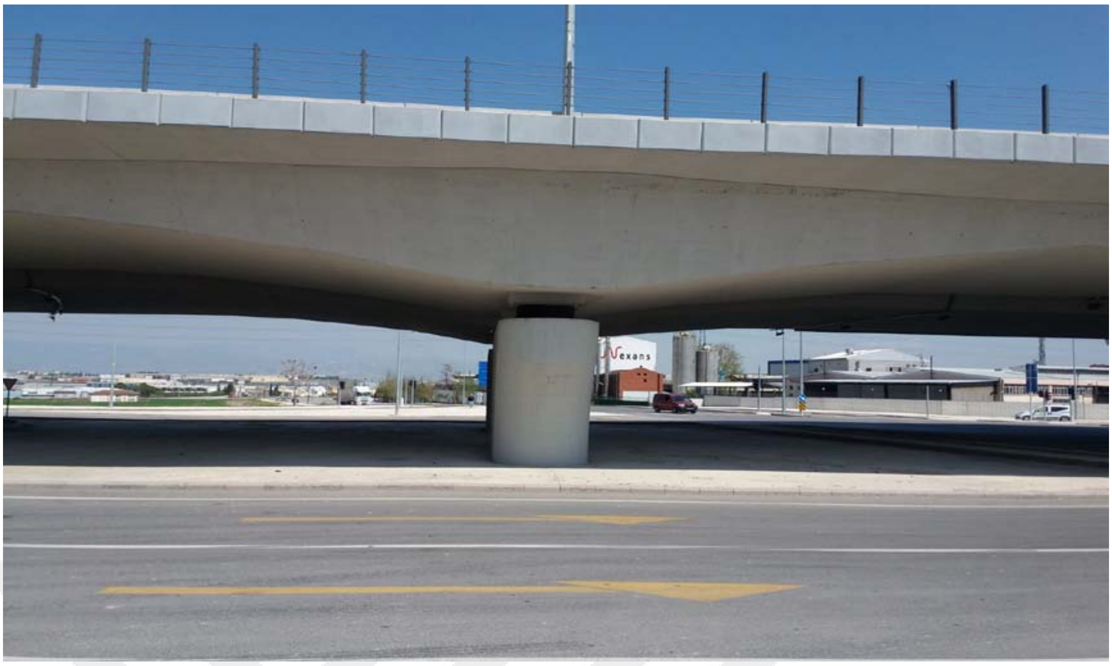

Figure 1.3 Denizli Hal Junction Bridge Middle Foot

Figure 1.4 Denizli Market Hall Junction Bridge Middle Support

Within the scope of the thesis, after a brief introduction in Chapter 1, a detailed literature review was given in Chapter 2. The analyses and their results were explained in chapter 3 and 4 respectively while the conclusions were given in Chapter 5.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

The post tensioning concrete was first developed by French Eugene Freyssinet in the 1930s, who realized that the strength of the compacted concrete greatly increased its strength. Post tensioning is the stretching and picketing of cables after the concrete gets its strength to a certain extent, following the laying of sheathed cables, pouring concrete around them and applying them in accordance with the project.

After World War II, due to the shortage of steel and the need to replace damaged bombed bridges, post-tensioning concrete became a popular structure. While the design and use of this method continued until the mid-1960s, it was observed to be largely used as flooring for warehouse, apartment and residential floors and supported tiling. In 1976, the Post-tensioning Institute was established because of the increasing interest due to the expanding applications in the field. Besides, the use of stretched concrete slabs for tennis courts became widespread. The United States Tennis Court and Athletics Association published a general standard valid until 1982 for after post-tensioning construction.

The post-tensioning method has become a technique used in the construction of bridges, long-span large structures, footings , nuclear reactors, large diameter pipes, silos, piles, sheet-pilings, marine structures, offshore platforms, ships, sleepers, silos and large-section bridge foundations.

The post-tensioning system makes the section smaller in concrete structures by being used in areas of high pressure and high stress. Its difference from pre-tensioning is its application phase. Very high-strength concrete is used in post-tensioning application.

The reason for this is that as it is in (T.S.3233), the comparisons vary according to the parameters of the concrete. In addition, a cross-section designed with post tensioning, shows a number of differences when compared with the section without post tensioning. The concrete material without post-tensioning is complex, composite, non-homogeneous and also inelastic. Therefore, the inelastic reinforced concrete uncontrollably cracks and in the case of cracking, the rules of resistance do not apply. In the case of post-tensioning, the cracking of the reinforced concrete is prevented and kept within certain limits. Thus, a gross section is obtained. The strength rules apply.

2.2 Concrete

Since humanity did not have a good binding agent when it started to use crushed stone for construction material many years ago, the practice was limited. Certain beam openings were created using one-piece crushed stones. Since the stone used had a low tensile strength, the stone was heavy, and the cross sections were too large, the constructions remained limited. For years, however, the love of human to construct large structures and to exceed large spans, have triggered the search for other systems. Durable and aesthetic structures were started to be built, using lime and natural cement with a binding feature.

The origin of concrete, which is now used in many places and is the most important material of the construction systems of humanity, belongs to the town of Pozzuloi of the city of Rome. It was observed that "pozzolan," found around the town, rose to the surface as a result of volcanic formations and that it became a strong and technical material when mixed with water. Throughout the period, aesthetic and very complex structures belonging to the Roman Empire were built in addition to little structures by the usage of this strong mixture. As the Roman Empire lost its power, the widely used material lost its area of utilization over time and in the middle of 19th century, concrete material with more contemporary, technical and mastered features was established.

As is known, "concrete" has the property of being a plastic, soft and liquid material consisting of gravel, sand, cement and water. The most superior side of this material, when compared with other materials is that it has a plastic consistency which may be shaped arbitrarily. Concrete is poured into the molds and after some waiting period, gains a certain strength according to the material and additive it involves. When concrete has the intended solid consistency, it does not have the intended level in tensile but it also shows tensile stress strength.

2.2.1 Cement

Cement is a material with high binding properties, which is obtained by baking the mixture of limestone together with clay stones at high temperature and then grinding. It is mixed with water and becomes a fluid paste and afterwards starts to petrify. This solidification, which we call the wall plug, varies according to the ambient conditions. After it is placed in the concrete in a certain ratio, as the temperature increases the solidification is replaced with hardening together with the concrete and so the concrete gains strength. Durability is an increasing event because it is a long process for cement paste to show its strength exactly.

2.2.2 Water

There is no harm in using drinking water in concrete mixture.

However, there should never be acid in water. In addition, high amounts of salt, sulfate, clay, mineral oil, organic matter and industrial wastes should not be in the mixture.

2.2.3 Aggregate

It is defined as the mixing of gravel and sand in a certain ratio. It is generally defined as sand between 0-7 mm and gravel between 7-70 mm. The human being actually used the aggregate as a filling material. However, it was preferred to use water together with cement, since it decreased the volume change in the concrete. Because concrete is a material that is both expensive and shows too much change in volume. Aggregate granulometry is very important for the concrete to be processed and replaced well. The granulometer significantly affects the impermeability, straining and shrinkage. Aggregate is a special material that is irrevocable for concrete, which increases strength and can be obtained in natural and artificial forms. The strength of the cement must never be higher than the strength of the gravel and sand in the resistance sequence. If the concrete strength is more than the strength of aggregate, the strength of the concrete will not reach the cement strength that will end with aggregate breakage. In this case, it will be impossible to increase the strength by increasing the amount of cement.

In order to obtain a good mixture, aggregate, water and cement must be in a desired and certain proportion in the concrete. Aggregate diameters must also be present in a certain proportion in the mixture.

2.3 Reinforced Concrete

Due to the low tensile strength of concrete, steel bars were placed into the concrete, starting from the beginning of 19th century. Within this context, the boat that JL Lambot obtained with this cement mortar mixture with iron reinforcement in 1855, and the pot made by J. Monier placing wire in concrete in 1867 were considered as the beginning of reinforced concrete.

Together with steel concrete, it exposed reinforced concrete, which is a composite building material. Using the steel in the concrete, the material has a higher ductility and more resistance to tensile stresses. This made history in engineering as a very appropriate form of solution.

For a structure to be reinforced concrete, steel and concrete must be well mixed. In other words, the steel bar around the concrete mass must be so clamped that there should be no deformation difference in the concrete around the steel. This situation, i.e. concrete and steel working together is defined as adherence or clamping.

The cracks in the concrete are in the direction of the principal tensile stresses. Steel does not prevent the cracking of concrete, but prevents the cracks to be deeper and longer. Cracking is related to concrete. Therefore, when a reinforcement is placed in a concrete structure, it should be used in the direction of principle tensile regression But this is unlikely. When the angle with principal tensile regression in placing the steel in reinforced concrete increases, it loses its benefit.

Since the beginning of the twentieth century, the reinforced concrete system, which has been widely used in buildings, has brought many advances in the field of Civil Engineering however, from time to time it did not meet the needs of structures with wide opening. Post-tensioning and pre-tensioning methods have been developed over time in such cases.

2.4 Pre-tensioning Concrete

Towards the end of the 19th century, some weaknesses in concrete have started to emerge after intensive use of reinforced concrete. The main weakness of concrete is its low tensile strength. Several studies have been carried out to reduce the disadvantages of this situation. One of the methods developed as a result of these studies is the pre-tensioning method. In this way, tensile stress especially in wide span floors and columns have been met by pretension ropes. This method, which is used to control cracks and reduce tensile stresses in concrete, has become one of the most technological methods of building systems.

With this method, high strength steel wires in certain forms are placed into the mold. These wires are stretched from two ends to a certain level by stretching and then concrete is poured into the mold. After the concrete gains 75% of the compressive strength of 28 days, the ropes stretched at both ends are released by cutting the ends and the tensile stresses in the reinforced concrete section are met by the pre-stressing loads applied to the ropes. Since the forces applied in pretensioning application are high, high strength concrete and multi-tendon ropes are used.

This method is an ideal method for prefabricated production since large anchors are required to hold all tendons and a large number of elements can be poured along the same series of tendons. When the anchors are released in advance, it is important to ensure that the sections move freely along the pre-tensioning bed, otherwise undesired stresses may generate when the end anchors are released.

2.5 Post-tensioned Concrete

Post-tensioned concrete is one of the most important construction technologies that will be the subject of structural engineering as pre-tensioned concrete. Although several patents have been issued for various similar methods in the last century, unsuccessful results was encountered due to the use of low strength steel. However, as a result of the developments, the long-term effects of the creep and shrinkage of the concrete decreased the post-tensioning force significantly. At the beginning of the twentieth century, the French engineer Eugène Freyssinet successfully dealt with the problem in a systematic way, using high-strength steel, firstly applying the pre-stress technique in concrete. Eugène Freyssinet showed the main reason for some of the failure in this issue by many experimental and mathematical concepts that it was due to the shortening of concrete shrinkage and stretching of steel over time, caused. Concrete is a common material used in various engineering methods in structural engineering. It has high compressive strength, but its tensile strength is considerably lower than its compressive strength. The tensile strength of the concrete varies from 8 to 14 percent of the compressive strength. After the loads are applied to the concrete element, cracks are seen in the pulling areas due to the low strength of concrete. Some techniques have been discovered to avoid cracks in the drawing region, one of which is the method of applying a high compression force longitudinally or parallel to the axis of the structural members. With this method, the force can reduce the stress caused by the tensile stress and can significantly reduce the tensile stress. Thus, this applied force may affect increasing the cutting, torsion and bending moment capacity of the sections of the element, the section being elastic. This method is called as 'post-tensioning method' in the literature.

The post-tensioning method is widely used in slabs and beams, especially in large span structures. However, its use in foundations has recently started to be seen. With the help of post-tensioning ropes, the distribution of the basic pressure and the distribution of the momentum in the foundation can be changed and more positive results are obtained in terms of design.

The idea of pre-tensioning and post-tensioning of a structure is not new. The barrels were made of separate wooden slats held in place by metal hoops. They are slightly smaller in diameter than the diameter of the barrel and are forced to be placed on the pits, so they squeeze them and form a waterproof barrel. The wheels stretch the heated iron tire steering it over the wooden edge of wheel similarly. This is posttensioning. In cooling, the tire is held firmly on the rim so that the joints between the rims and the rim are compressed.

2.5.1 Advantages of Post-tensioned Concrete;

Very large openings can be applied by using the post-tensioning method. This situation facilitates the work of architects and engineers and enables the realization of projects with different architectural and aesthetic geometry that seem very difficult to do.

- The number of columns and beams in the project is reduced with the post-tensioning system.
- Since the amount of concrete and iron required in the project decrease, the floor and the total building weight decrease as well.
- It provides a significant economic advantage.
- As the total building weight decreases, the seismic forces acting on the structure decrease in case of earthquake. In this way, the structure performs better against earthquakes.
- Since steel post-tensioning ropes are covered with grout in the sheaths, no corrosion problems occur.
- In post-tensioning systems, deflections are controlled.
- Crack formation in concrete decreases.
- The stiffness of the structure increases.
- Smaller sections can be obtained under higher loads.

2.6 Materials and Equipment

The material to be used in the post-tensioning concrete is pre-tensioning steel together with the concrete. The preliminary equipment are the post-tensioning bed, locking mechanisms (jack), hydraulic jacks, post-tensioning sheath and anchor plates.

2.6.1 Concrete

The concrete type to be used in the post-tensioned concrete is at least BS 30 (Concrete Class 35). Accordingly, the concrete cylindrical compression strength should be at least 35 MPa at the end of day 28. In most manufacturing, BS 40, 45, 50, 55, 60 and even BS80 are used.

Cement: Portland cement used is mostly used in a post-tensioned concrete structure.

In Turkey following Portland cement types are used:

1.PÇ 32.5

2.PÇ 42.5

3.PÇ 52.5.

Sand: Only well-washed river sand is used as sand**.**

Aggregate: Aggregate gravel or crushed stone are generally used. The grain diameter should not exceed 20-25 mm.

Water: The water that can be used in the concrete mixture should be in drinking water quality. The water / cement ratio should be 0.40-0.45.

Collapse: The collapse of the concrete mortar should be at most 50-100 mm.

Additives: Additives containing calcium chloride should not be used.

Cure: Concrete surfaces must be kept moist. If the steam cure is applied, the maximum temperature should not exceed 60-80 degrees Celsius and the curing should be applied in 20 degree steps.

2.6.2 Steel

Post-tensioning reinforcement steel can be wire, toron or rod shape. The 1st class bundles were observed to have higher strength in the typical stress-strain deformation curve of the reinforcing steel reinforcement and largely deformed under high stresses.

Steel used for pres-tensioning has a nominal flow strength from 1550 to 1800 N/mm^2

Different forms of steel;

Wires: 7 mm diameter separately drawn wires

Wires-1 It is a collection of wire (usually 7) that are assembled and therefore have a different diameter according to their field.

Tendon: A series of boxes arranged in a channel which is used only in the post-tensioning system.

The bridges in the post-tensioned concrete systems typically use super wire with 7-wires in 15.7 mm in diameter (with 150 mm² field) with a breaking load of 267 kN in reinforced concrete beams, pavements and planted foundations.

2.7 Anchor System

The most important element of the post-tensioning system. The clamping element, called the anchor head, is one of the most critical equipment of the post-tensioning technology. Although the anchoring system does not differ in the application of post-tensioning application, it differs in terms of the production of the firms. This anchorage system is made by clamping the ropes or rope groups at both ends of the anchoring system while it can also be made by one-side clamping. In this system, the other side of the rope stretching process is active and the other side is passive which is blind anchore.

According to patents, the most used anchorage system in which many properties change is 'active anchor' type.

This active anchoring can be defined as follows.

When the ropes that make up a cable approach the free end (or clamping zone), they are separated from each other and inserted through each hole separately into a conical cavity. Torons (Ropes) are wire sets formed by the bending of 3,5, or 7 wires. These ropes are passed through the anchoring system and are stretched according to the appropriate pulling force in the project. The rope on the side of the tensioning process is squeezed into the rope anchoring mechanism after the initial discharge slip by 4 to 8 mm with the first move and can no longer go back.

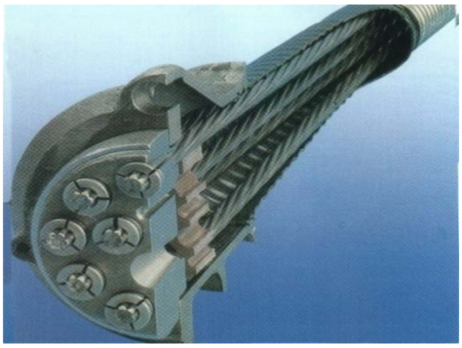

Figure 2.1 Conical anchor

Figure 2.2 Active anchor

Since the anchorage system is a very effective and important factor in the operation of post-tensioning system, it is an important element to be considered during the installation. Because the anchors used in the bridges and bridge foundations are more 12,19 and 31 rope anchorage system, it is necessary to put the project in the place determined by the projector and to apply the desired force during the stretching process.

2.8 General Features of the Material to be Used in Post-tensioning System

The properties of material to be used during this manufacture are listed below.

Post-tensioning ropes to be used:

- Nominal diameter of ropes to be used = 0.62 " (16mm)
- cross section of 1 rope = $150mm^2$
- Tensile strength = $1860N / mm^2$
- Flow strength (0.2% plastic deformation) = $1660 N / mm^2$
- Tension Limit (0.75 x 1860) = $1395 N / mm^2$
- Tensile force for 1 rope = 209 KN

Post-tensioning rods to be used:

- Breaking strength = $1050N / mm^2$
- Flow strength (0.2% plastic deformation) = $950 N / mm^2$
- Tension Limit (0.7 x 1050) = $735 N / mm^2$
- **Retaining sheath**; It is usually aluminum material which has no strength and is used to prevent the contact of ropes with concrete.
- **Blasting equipment**: It is a spiral iron reinforcement which is used for spreading the force coming into the area where the strapping ropes come out of the sheath after the finishing and to prevent any cracks in the concrete due to high tension during the stretching process.

2.9 Safety Stress in Post-tensioning Flexural Member

2.9.1 Concrete Safety Stress During Load Transfer

Some safety stresses occur during the transfer of load in the bending tool. These safety tensions are the safety tensions that the post-tensioning strength transfer to concrete. At this point, time dependent post-tensioning losses have not occurred yet.

2.9.1.1 Pressure

These values are:

- Elements made in factory: 0.60 f_{ck}
- Elements made in building site: 0.55 f_{ck}

2.9.1.2 Tension

It is tension safety stresses in reinforced concrete structure during load transfer.

These values are:

- For all construction elements: 1.6 $\sqrt{f_{ck}}\ \ kg/cm^2$

The calculated tensile stress must not exceed these values, if exceeds, all tensile strength should be exceeded with additional adherent reinforcement. The unbroken section should be considered when making these calculations.

2.9.2 Concrete Safety Stresses for Use Loads (Long Term)

Use loads are the loads that all load on the section affect the section. The cross-sectional area analyzed under these usage loads is designed to have all deformations and exposure effects. In this case, time-dependent post-tensioning losses are considered. The stressed cross-section under these loads shall not exceed the following values as tension.

2.9.2.1 Pressure

Pressure safety stresses in concrete under use loads (long term)

- Bridge elements: 0.40 f_{ck}
- Other structure elements: 0.45 f_{ck}

2.9.2.2 Tension

Tensile stresses in section under use loads (long term)

- For all structure elements: 1.6 $\sqrt{f_{ck}}\ \ kg/cm^2$

2.10 Literature Review

The sub-grade reaction module calculated by Shukla (1984) has been a simplified solution for the design of the post-tensioning on elastic foundations. Basically, the reaction of bending a plate onto an elastic base was evaluated using differential equations. Thus, this study found that the basic reaction of the concentrated load was rapidly absorbed by post-tensioning.

Another article about the analysis and design of great raft foundation was presented by Ball and Notch, J.S. (1984). Guidelines are provided to assist the designer in selecting the finite element type, which provides information on the modeling of the superstructure coefficient, calculation of the coefficient of ground coefficient and interpretation of inter-element forces.

A study by Alver et. al. (2012) aimed to determine the effect of short beam formation under the beam element on the pre-tensioning connections to the structural capacity of the system. To this end, four sample tests were designed. One of them (SCP0) is short cantilevered beams and three of them (SCP1, SCP2 and SCP3) are formed with short cantilever beams of different sizes. The research was carried out both experimentally and numerically. In the experimental part of this study, displacement controlled analysis was performed and the displacement values between 0% and 4% of the hysteresis losses of the samples were obtained. In the numeric section, these four examples SAP2000Vare modeled at 14. Structural capacities were observed in terms of connection stiffness and ductility. The results obtained from numerical studies were compared with experimental study results. Closer results were obtained in both frames. As a result, the contribution of the short cantilever beam to the structural capacity of the system is clearly seen and it is observed that the ductility is increased at the joining points. The effects of the tensile strength of pre-tensioned columns and beams on the prefabricated torons and the tensile rates applied to these torons were investigated. The study concluded that the diameter of the post-tensioning toron and the stretching rates applied to the torons are the two basic variables.

A special study including experiments on prefabricated column-beam joins with post-tension ropes and analytical modeling of these elements is presented (Kaya, 2007). At the end of the experimental studies, it was seen that these properties of prefabricated elements were better compared to monolithic elements in terms of moment capacity, rigidity and energy consumption of prefabricated elements. In particular, the tensile strength of the torons with a diameter of 15 and 24 mm was used with a 60% strain, and the moment capacity with the monolithic elements was found to be approximately equal. In the analytical modeling of the experimental elements, although the initial load capacity and rigidity of the analytical models were lower than the experimental elements, the load capacity and rigidity of the analytical models were compatible with the experimental elements.

A study conducted by Sümerkand (2015) aimed to develop simplified natural frequency formula based on Operational Modal Analysis and Finite Element Method for balanced stabilized post-tensioned bridges. For this purpose, experimental and analytical studies were carried out on five bridges which were constructed with balanced cantilever method. In experimental studies, Operational Modal Analysis Method based on environmental vibrations and Finite Element Method in three dimensional analytical studies were used. Then, the most effective parameters were determined for the dynamic behavior of the bridge which gave the closest result to the experimental and analytical analyzes, and analytical modal analysis of many bridge models with different span and height were conducted based on this bridge. A simplified natural frequency formula has been developed for balanced cantilever reinforced bridges with the Least Squares Method using the frequency values calculated from the analyzes. The developed formula was compared with the experimental results of the existing bridges and the results were close to each other.

In the thesis study prepared by Çakır (2015) investigated segmental colon safety. In this examination, bridge systems with a designed main span of 90 m designed according to AASHTO LRFD were used. In this study, 120, 150, 180 m spans were studied. Four types of trucks were used. H30-S24T and L (which is still being used in Turkey), HL-93 (AASHTO, 2012), KGM-45 (a new load type including both the axle load and the lane load). Statistical parameters are determined according to the data collected from truck loads. (Turkey General Directorate of Highways and Transportation Cost Studies). The target reliability index for the final case was chosen as 4 and 5, then as Service 3 stresses. The corresponding reliability indexes are calculated for the final capacities. In addition, AASHTO LRFD showed whether the tension limit was suitable to stress or not. Service 3 Load Combination. Extrapolation factors were calculated for 75 years and the results were compared to calculate the uncertainty of live load models.

In the thesis study prepared by Aktaş (2006), column joist connections obtained by stretching of a new system of post-tensioning steel after concrete casting were examined experimentally. In regions with high seismicity, pre-fabricated beam and post-tensioning column joints are widely used. This system was observed to perform well against the earthquake loads within the acceptable displacement limits. The post-tensioning system provided a higher initial stiffness to the carrier frame system than with the conventional casting system. As a result of the studies conducted, this new connection type can be used safely in regions with high seismic activity.

In PhD thesis of Dönmez (2015), post-tensioning values of post-tensioning concrete elements production generally used in construction industry. To this end, detailed information about the distribution of the stresses to be formed in order to make the design of these elements successful is required. The exact values of the stress state can be measured at each point of the examined element by the photo elastic method. In this respect, it is concluded that the application of photo elastic modeling method is suitable for the evaluation of the stresses occurring during the production phase in the pre-tensioned concrete elements, based on the similarity between the related stresses and the thermal stresses. In this study, the pres-tressing of the construction elements were examined using the method of freezing photothermoelasticity

A study by Korkmaz et. al (2005) introduced the experiment on strengthening rural masonry construction with post-tensioning method using elastic webbing and discussed the results of the experiment conducted with of 1:10 scale vibratory unit The study aims to investigate the effect and validity of post-tensioning method applied with elastic strips applied in various configurations. It is intended to use the car tire, which is used as an elastic strip in real scale (1: 1) application. Due to the fact that automobile tires are available free of charge, they are considered to be implemented by low-income homeowners. The validity of the proposed reinforcement method was investigated by testing the 1:10 scale masonry structures in a simplified shaking table under increasing acceleration. According to test results, the strength of the reinforcement with horizontal strips was found to be 70%, the strength of the reinforcement with vertical strips was 40%, and the strength of the reinforcement with both horizontal and vertical strips increased by 110%. The results of the test are preliminary in determining the parameters to be examined for the 1: 1 scale experiments performed in the civil engineering laboratory of METU.

According to a study by Arıöz et al. (2004), the increase in the quality of concrete was noteworthy as part of the modern technology used today. The quality of concrete can be increased without any additional effort and expenditure. The increase in the quality of concrete not only contributed to the overall quality of the construction, but also increased the structural resistance and the building safety level. In Turkish structure tradition, building bearing systems have been constructed in different designs and are becoming more and more custom designs. In this research, several types of building conveyor systems, which could be considered to represent the diversity in the building sector, were produced and the impact of the change of concrete quality on the building cost was analyzed extensively and the safe quality of this concrete quality contributed to the production of concrete that could meet the compressive force from the sequence.

Ali (2010) in his thesis study used two methods to find the maximum load of pre-tensioning concrete beam which has been widely used nowadays and the maximum displacement corresponding to this load. The first one is the experimental study in laboratory conditions and the second is analytical modeling. Test option is limited with size of the element, support conditions, loading condition, pre-stressing force, etc. In this study, nonlinear finite element modeling (FEM) and analysis were performed with ANSYS package program of 3 pre-stressed reinforced concrete simple girders with single span under four point loading with rectangular cross-sectional study. Thus, a computer model that is close to the actual behavior was tried to be created. The results were compared with the results obtained from the experiment and were observed to be close to each other.

In PhD thesis of Gündüz (2010), a pair of superimposed beams, fast-rail high-speed railway bridge, vertical loads, traffic loads and earthquake loads were solved and dimensioned and the necessary cross-sectional calculations were made. Bridge superstructure analyzes were modeled in the computer program, and structural checks, dynamic loads, critical wavelength and vertical acceleration, torsion, vertical displacement, vibration control were performed for traffic safety. He observed that the post-tensioning system provided positive results in wide openings under earthquake and vertical loads.

In a study by Cronin and Henry (1980), despite the use of post-tensioning tendons in the United States since the early 1960s, there has been a significant increase in applications over the last five years. Essentially, at the weak points, the successive foundation used in place of the deep foundations with little or no point minimized the typical amount of excavation and filling. As a result, the foundation has become resistant to water and cracking. In addition, this report described the different types of backgammon basic types, the advantages and limitations of their use, and the backgrounds of the Washington DC region and other projects in the US. The regulation on post-tensioning permits the reduction of costly construction materials and faster construction. With this approach, the basic design for the ongoing building increase yielded positive results.

A study by Berkey et al. (2003) described the post-tensioning system used in the construction of residential houses and other structures and the basic system after post-tensioning. Basically, the medium-level excavations were transferred to the foundation on a flooring which was formed at the bottom of the floor. Concrete foundation walls were formed on the circumference of the stretched floor plate. The weight of the upper structure of the dwelling and the weight of the foundation itself were distributed to the flooring of the backed foundation so that the foundation was protected against the harmful effects of the large soils on the foundation.

Thayapraba (2014) has done its work in the developing Indian region, which is only recently introduced with the post-tensioning system. Natural forces, of course, are a big problem for the building. The important benefits of this problem are considered in the solution. The systems related to the concrete slab system were analyzed using SAP and developed based on the MS Excel program design methodology to compare the cost-effectiveness of the post-tension flat plate. The results indicated that the flat plates were less expensive than the RCC slabs.

In an article by Ganz (2008), some new developments and trends in the materials used for the stratified concrete were revealed. In particular, Ganz introduced ultra-high performance concrete and described the advantages of ultra-high performance concrete. After reviewing the developments in post-tensioning systems, some new damping devices for bridge and building structures were presented. Finally, a number of specific applications with significant potential for the coming years were provided. These included stretched buildings, floor coverings, pretensioning and post-tensioning bridges, drainage bridges, reinforced concrete and storage structures.

Şahin Aydın (2001) designed and presented a computer program which is the solution of post-tensioned spread foundations. By determining the amount of back-stretch cable in the program, the basic thickness for a short period of time, how fast, economic design can be predicted. In addition, cost comparison was made in the study. As a result of this study, the design of the post-tensioning cables were found to be a cheaper and more economical system according to the comparison of the cost of accelerated system by designing the post-tensioning cables in a faster manner.

CHAPTER 3

ANALYSES

3.1 Introduction

The design of the post-tensioning concrete systems varies according to the section to be used. Popular structures of these different designs are generally single-beam beams, multi-span beams, curtains, beams without floors, brackets, domes, wide span bridges and raft foundations. In this section, the basic application of raft foundation was briefly explained. These differences are usually caused by the use of the differentiated rope in different forms. Also during the design of this method TSE, TS. 3233, (1979) and AASHTO (2012) regulations were used.

3.2 Basic Analysis Methods

3.2.1 Flat Plate Analysis

Considering that the foundation is an inverted two-way plate, the basic bending moments can be calculated with this method, provided that the base full fills certain requirements specified in TS-500.

3.2.2 Elastic Basic Analysis

Basic settlements can be calculated according to the calculated stress distribution under the foundation. The element is divided into elements with the grid and modeled on the linear springs representing the reaction module. Computer analysis is required for a large amount of calculations in this method. Some of the structures are constructed on the so-called poor in terms of engineering. Due to the low bearing capacity of the ground in such structures, continuous or raft basic types are frequently used as the basic system. Although the raft foundations are rigid, they can make rigid displacements due to the flexibility of the floor.

3.2.3 Equivalent Framework Method

This method is an analysis method to be applied to flat plates and bi-directional plates. A foundation can be considered as an inverted two-pronged floor, which transfers the charge to the soil and acts as a point on which the column loads. This method of analysis is also the division of the base into strips which are loaded by a column line and are resistant to earth pressure and analyzed as a composite basis of the strip. Soil stresses may resemble an inverted beam when considered as loads between columns in the system, but an analysis that solves such a system will not result in a real solution. Loads of moving earth stresses and calculated support reactions shall not be equal to the axial load in the columns. In this method, the assumption is made that the superstructure load is transferred to the rigid foundation.

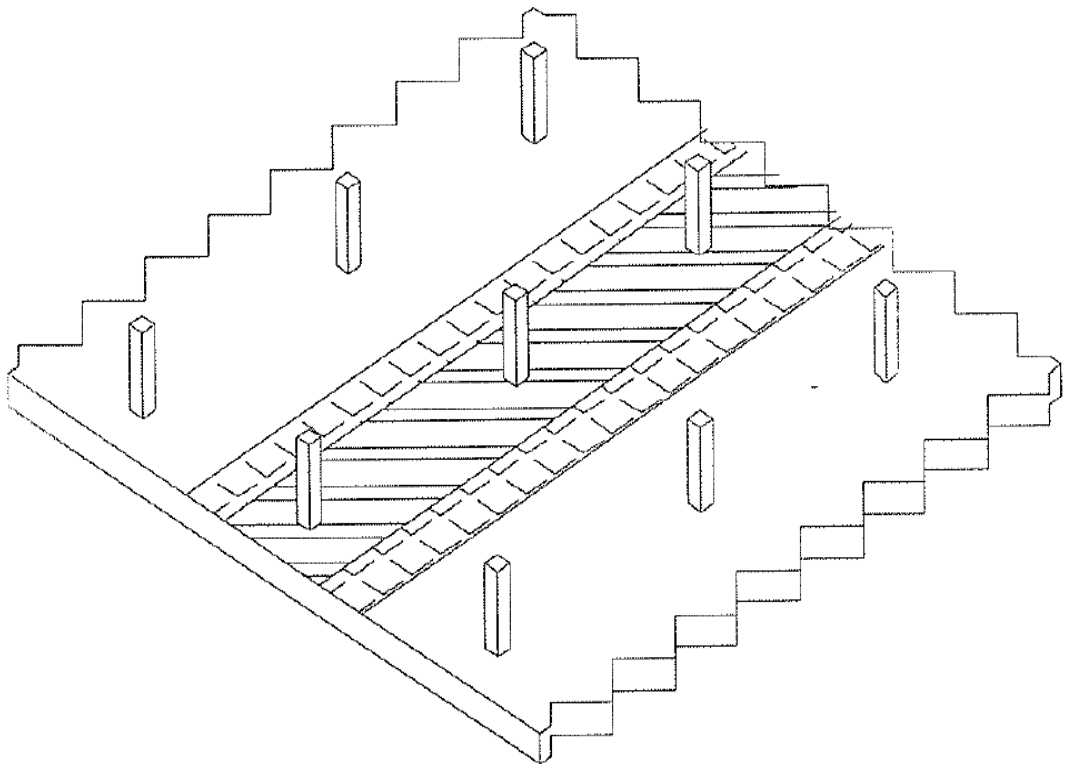

Figure 3.1 Design strips on the base

On the other hand, strips in a raft foundation can be defined as recommended in this method because the moment distribution in the direction perpendicular to the designed direction will not be different. In a flat plate supported directly on the columns with any beam, the hardest parts are the parts of a base panel extending to the column along the sides so that the greatest stresses occur in these regions. To check the stresses under specified loads, the stress distribution across the width of the design strip should be considered. For flat plates, 75 percent of the negative moment is allocated to the column strip while 25 percent is allocated to the middle bands. These ratios are 60 percent and 40 percent respectively for the positive moment.

3.2.4 Rigid Base Method

In the rigid foundation method, the analyzed base will only perform rigid rotation. This method is an approximate method whereby the base is divided into strips which are loaded with column lines and resistant to earth pressure. The strips are then individually analyzed as a combined basis.

According to its rigid base assumption, the ground stresses can be equally separated in the edge region of each column as long as the geometric center of the foundation and the results of the column axial forces are at the same point. The cutting force and bending moment diagrams can be drawn using this voltage distribution. (Figure 3.2).

The stresses on the base of the foundation are dependent on the superstructure loads and their basic dimensions. The basic dimensions must be chosen in a way that the floor does not exceed the strength of the safety stone. In addition, the ground seating can be calculated under the calculated basic loads.

This approach ignores the deformation of the foundation as this deformation is very small compared to the ground deformation. The use of this method has decreased since computer analysis provides more detailed and faster solutions. Moreover, this method is less preferred since it gives conservative results.

For all these reasons, in this study, finite element models were used in which the ground was simulated with springs.

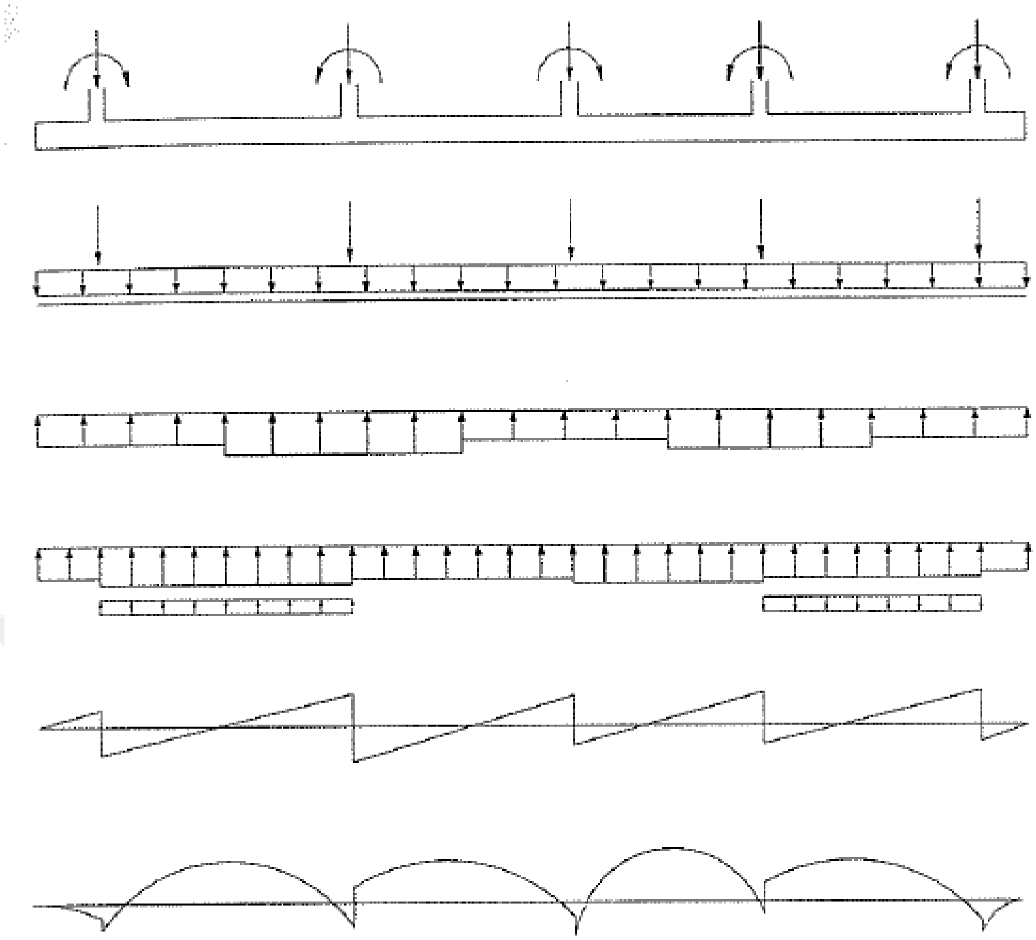

Figure 3.2 Rigid baseline analysis

3.3 The Design Concepts of Post-tensioned Concrete

3.3.1 Rope Distribution

The application of the straddling ropes in different ways according to the structures to be used. These differences vary according to the tensile forces that the structure must resist.

An equal tendon distribution must be provided in both directions to ensure load balancing in a two-way floor panel supported by the wall or beams around it. However, a uniform distribution of the tendons on the columns in two-way panels does not result in a load-balancing effect equal to all regions of the panel. The rope distribution can be applied to the areas where the stress is formed intensively on the section so that it is counteracted with the rope against moment stresses in that region. Especially, although the moment of span is dense in places where span is large, more ropes are considered to be applied in places close to structural bearing to meet the tensiles in support moment.

An alternative solution is to tap the tendons in only one direction and distribute them in the other direction homogeneously. This results in the panel behaving like a unidirectional plate.

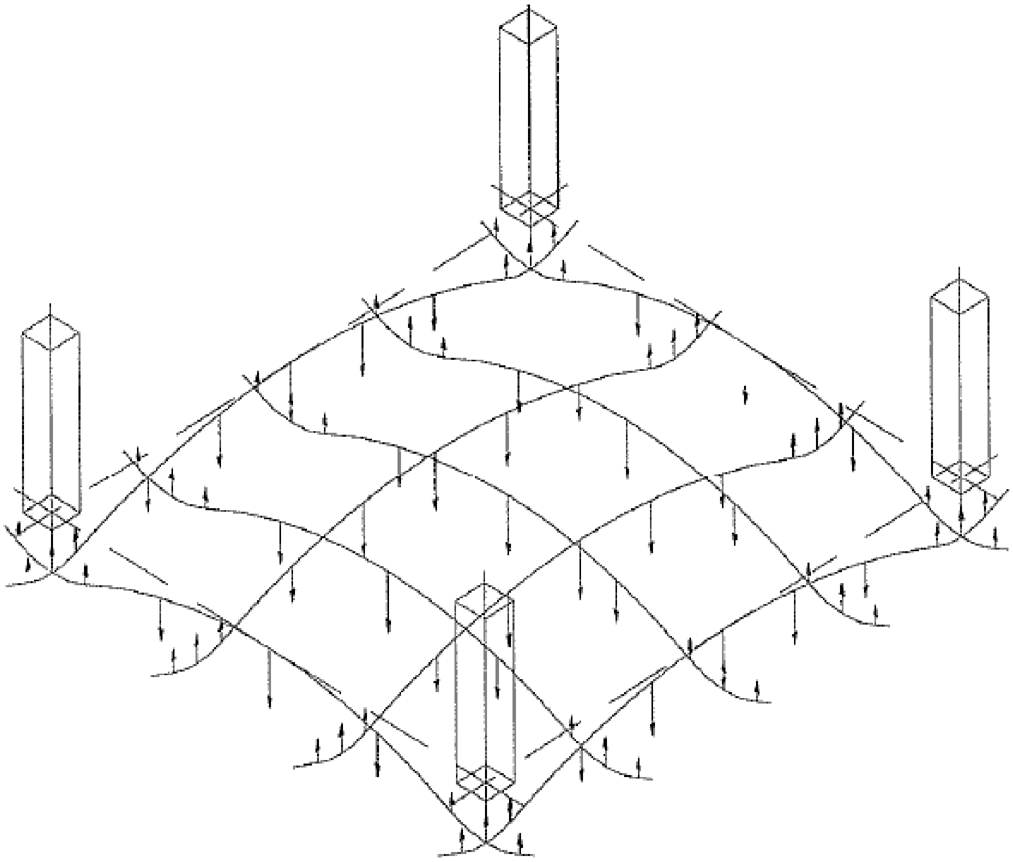

Figure 3.3 Equally spaced tendon strengths supported by four corner columns

Another tendon placement can be obtained by closely matching the tendons in both directions with the elastic distribution (Figure 3.4). This proposed distribution is planned to be 65 to 75 percent of the column stripe tendons and 35 to 25 percent in the middle strips.

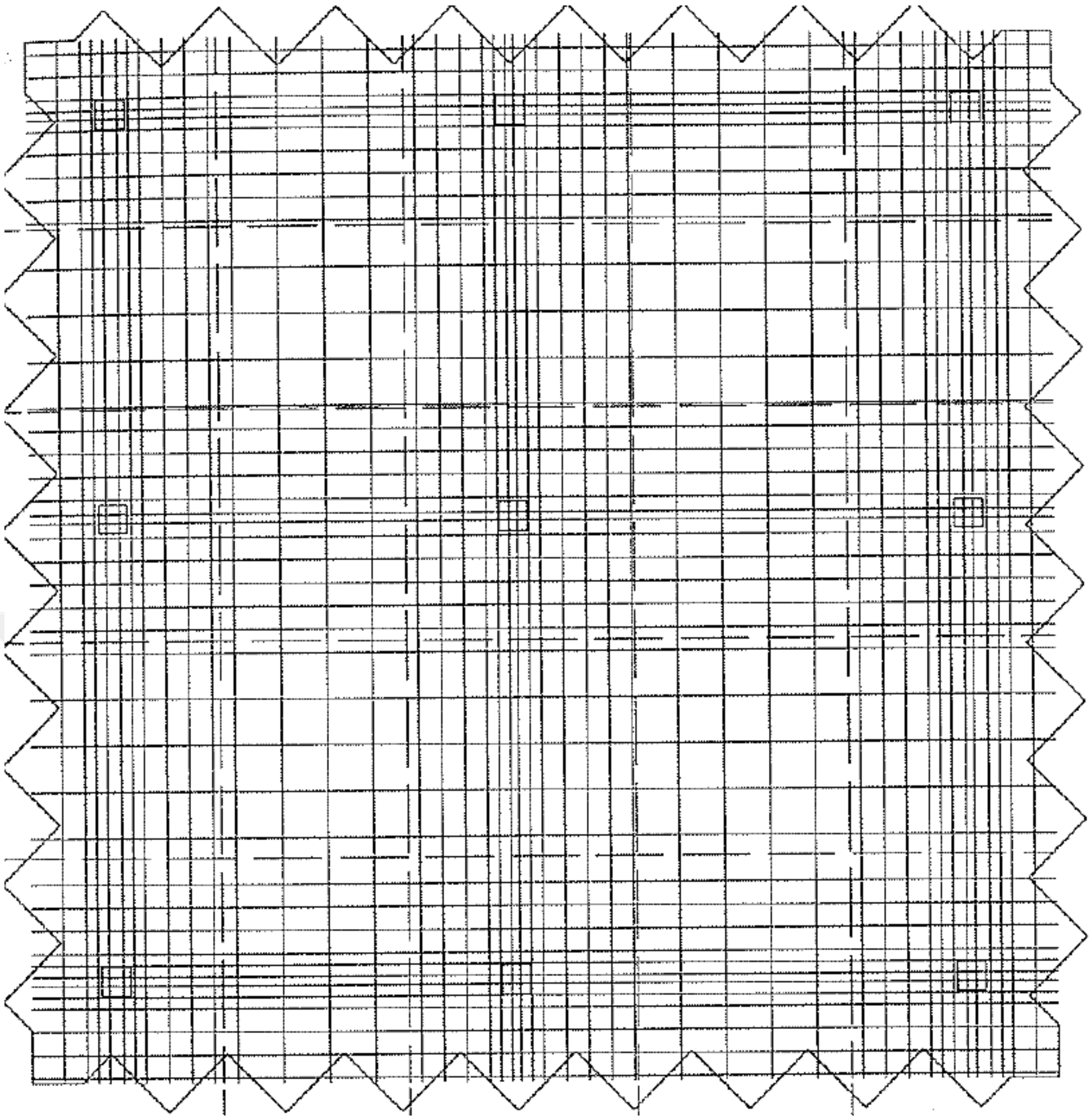

Figure 3.4 Ropes placed on column strips

There are several advantages of placing ropes on column strips. Smaller sections can be obtained under higher loads. It provides a smoother load distribution and balance. It minimizes the stresses in the section and increases the carrying power of the section. It provides more accurate operation of the system. It easier to place the ropes. The tendons that concentrate around the columns increase the tensile and shearing resistance. For all these reasons, the ropes are placed in two directions and the bottom of the column in this study.

3.3.2 Tendon Profile

Tendon profile changes according to the moment and shearing stress areas to be met in the section. These profiles are applied in the reception direction of span moment in spacious beams, at the end regions where the moment is dense in partition, while they are applied over top of the structural bearing to meet the support moment in beams or flooring and It is applied in bottom area of the column where the tension is dense in foundation and momentum is high. Therefore, the tendons were applied intensively in the lower areas of the colon in this study.

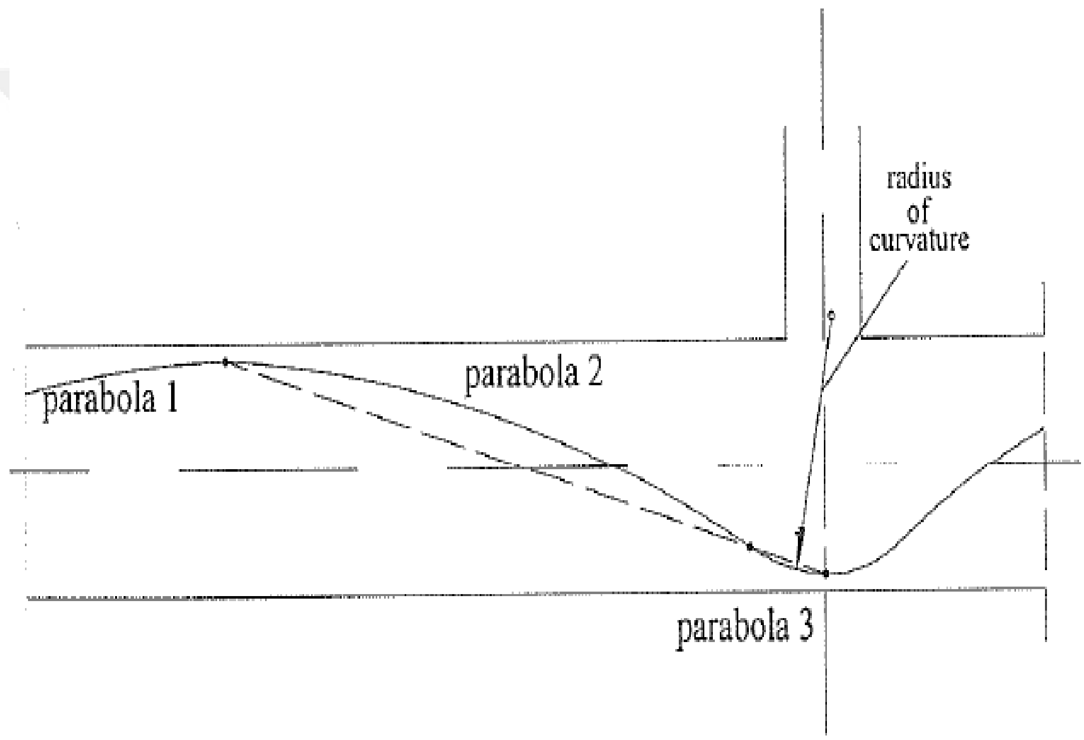

Figure 3.5 Tendon profile applied in basic design

3.4 Post-tensioning Losses

The posttensioning rope may begin to lose strength during or after the transfer of the load to the concrete. The reasons for this lose in force are posttensioning losses. These losses are divided into short-term and long-term losses.

3.4.1 Short Term Losses

3.4.1.1 Anchoring Losses

This loss is a loss caused by aging in the anchorage block or lock mechanism during the post-tensioning application. For example, the chuck used as a lock causes the pulling force to decrease due to damage to the teeth. The chuck end section must be constantly renewed and cleaned to reduce anchor losses.

3.4.1.2 Elastic Shortening

During the load transfer, the concrete around the strut rope is shortened due to high pressure. The pre-tensioning force decreases according to the shortening amount. Since the rope remains within elastic limits when stretched, the unit deformation of the concrete in the adherence steels is lost with the same value as the post-tensioning steel. Elastic shortening is not mentioned when the cables are pulled together in the traversing elements.

3.4.1.3 Friction Loss

In post-tensioning systems, the friction of the ropes during the stretching of the ropes results a reduction in the force of certain proportions, which is called friction loss. In addition, if the profile of the ropes and the formation of unwanted curvatures, large friction losses occur in the rope. In the systems where multi-rope system is applied in the post-tensioning elements, friction losses of the ropes to the sheath and the friction cause. The curvature should be created as much as possible at the desired level to reduce the friction loss and provide more force from the rope.

3.4.2 Long Term Losses

3.4.2.1 Straining Loss

Straining is a major problem for reinforced concrete structures. Deformation under high loads can be quadrupled over time. However, the deformation of the stratified concrete is much less than that of the reinforced concrete system. The most important reason for this is that the post-tensioning force has an opposite effect to the permanent loads. Therefore, the straining deformation can be reduced or deformed. Therefore, approximate methods are used for straining losses.

3.4.2.2 Retreat (Shrinkage) Loss

It is also advantageous in retreat as it is in straining in posttensioned concrete. Retreat of the concrete, mixing ratios, post-tensioning type, type of cement, curing time, time between curing of external curing and post-tensioning application, size of section and environmental conditions are affected by various factors. For elements after post-tensioning, the loss in the sequence prior to the drawing process was performed before the process was stretched.

3.4.2.3 Steel Straining Loss

The post-tensioning rope also makes a small amount of loosening and straining under the post-tensioning force applied at first. The loosening value is determined by the company manufacturing the steel. Recently, the loss of straining in the loose toron and wires has decreased to 1/4 of the former loss. The loss in steel can be taken as 3%. The magnitude of the reduction in the pre-tensioning not only depends on the strength of the continuous biasing force, but also on the ratio of the initial voltage to the yield stress in the tendon.

Table 3.1 Average Total Lost Values

	Pretensioning Losses (%)	Posttensioning Losses (%)
Elastic Shortening in Concrete	4	1
Straining in Concrete	6	5
Retreat in Concrete	7	6
Straining in Steel	3	3
Total Loss	**20**	**15**

As seen in Table 3.1, the total losses are at 15% range in the posttensioning method. Therefore, 15% strength loss was taken into account when calculating the rope capacity.

CHAPTER 4

POSTTENSIONING BRIDGE FOUNDATION ANALYSIS RESULTS

In this section, the raft foundation in the middle support of the Hal Junction Bridge, which is solved by post-tensioning technology, has been solved by using post-tensioning technology. In this solution stage, changes in stress analyzes and moment amounts are designed to remain within the limits of the regulations and solutions are produced according to these designs. In addition, it was observed that the pressure distribution at the bottom was reduced by post-tensioning.

The cross-junction bridge is a two-span bridge with account openings L1 = 35.00 m, L2 = 60.00 m and L3 = 35.00 m. The width of the bridge from outside the border to outside the border is 14.49 m. The height of the superstructure for the casted post-tensioning system is 140 cm to 280 cm. There are 6 cm thick coating on the support system including 1 cm water insulation and 5 cm asphalt. The bridge was built in one stage. The ropes were drawn in one step. During the design, sheaths consisting of 12,19 and 31 ropes used in post-tensioning systems were preferred. Sheath types are preferred in analysis because they are the most used sheath types. In the solution of the system, the section was completely pressurized and the drawing remained within the limits of the regulation. In the bridge with 2 m foundation thickness, post-tensioning was applied without making a change in thickness. Its basic length and width remained 14.40m -13.20m as it did not provide any economic and design benefits. That is, the B / L ratio is kept constant. In addition, the concrete class whose compressive strength is 30 MPa (C30) was used. During the analysis, the combination of COMB1 specified in AASHTO (2012) standards was applied. In the post-tension analysis, PT-FINAL + COMB1 combination was used because posttension designs are made according to the coefficientless load combinations and comparisons are made with the lower limit values that the regulation deems appropriate. During the solution, the baseline loads were compared to the baseline behavior and the non-sequential analysis were compared.

4.1 Vertical Loads On the Foundation

Dead Load on the Basis: P = 25113 / 9.58 = 2620.3 kN / m^2

Basic Weight: P = 2.00 x 25 = 50 kN / m^2

Moving Load: P = 1496.00 / 9.58 = 156.09 kN / m^2

Earth Load over Foundation: P = 2.00 x 19.20 = 38.04 kN / m^2

Table 4.1 AASHTO (2012) Load Combinations

Loading Effects	Comb1	Comb2	Comb3	Comb4
DL	**1.0**	1.30	1.0	1.0
FOUNDATION	**1.0**	1.30	1.0	1.0
MOVEMENT	**1.0**	2.17		
EARTH	**1.0**	1.30	1.0	1.0
EQX+0.3EQY+DL			1.0	
0.3EQX EQY + + DL				1.0

4.2 Post-tensioning Loads Affecting the Foundation

The post-tensioning loads affecting the foundation are calculated by considering the rupture strength, rope diameter and losses. This value is reduced by 25%as the tensile strength of the post-tensioning steel at TS. 3233 is expected to be stretched to 75%. In addition, post-tensioning losses were taken as 15%. In addition, COMB1 + PT-FINAL combination was used during the post-tensioning solution.

For 12 rope;

P1 = 186.0 x 0.75 x 12 x 1.54 x 0.85 = 2190.12 kN

For rope 19;

P1 = 186.0 x 0.75 x 19 x 1.54 x 0.85 = 3469.50 kN

For 31 ropes;

P1 = 186.0 x 0.75 x 31 x 1.54 x 0.85 = 5660.77 kN

4.3 Sap2000 Analysis Results

4.3.1 Moment values of Post-tensioning Foundation

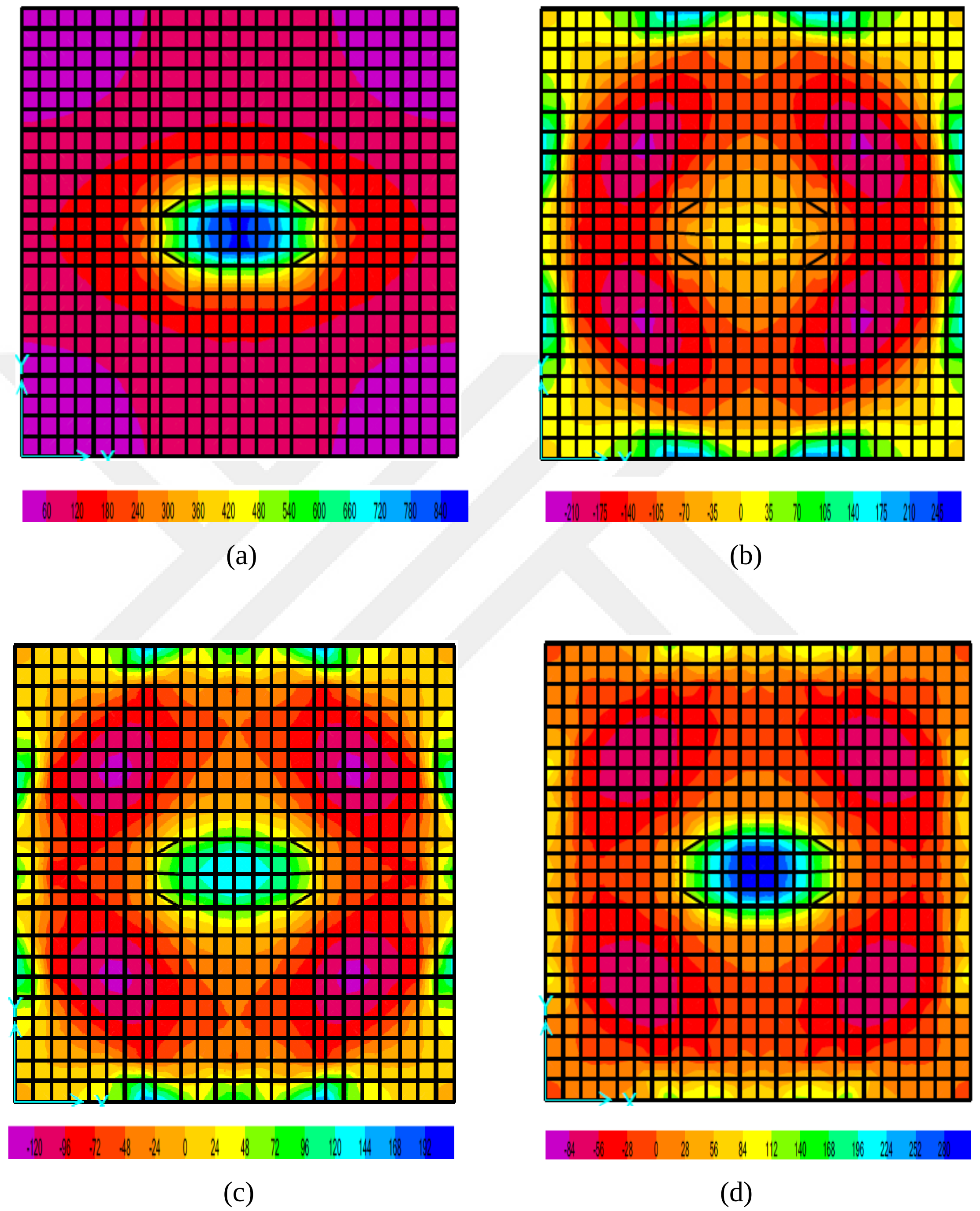

Figure 4.1 Foundation max moment distribution (a: without the post-tensioning, b: with post-tensioning n = 12 s = 0.5 m, c: with post-tensioning n = 12 s = 0.75 m, d: with post-tensioning n = 12 s = 1.00 m)

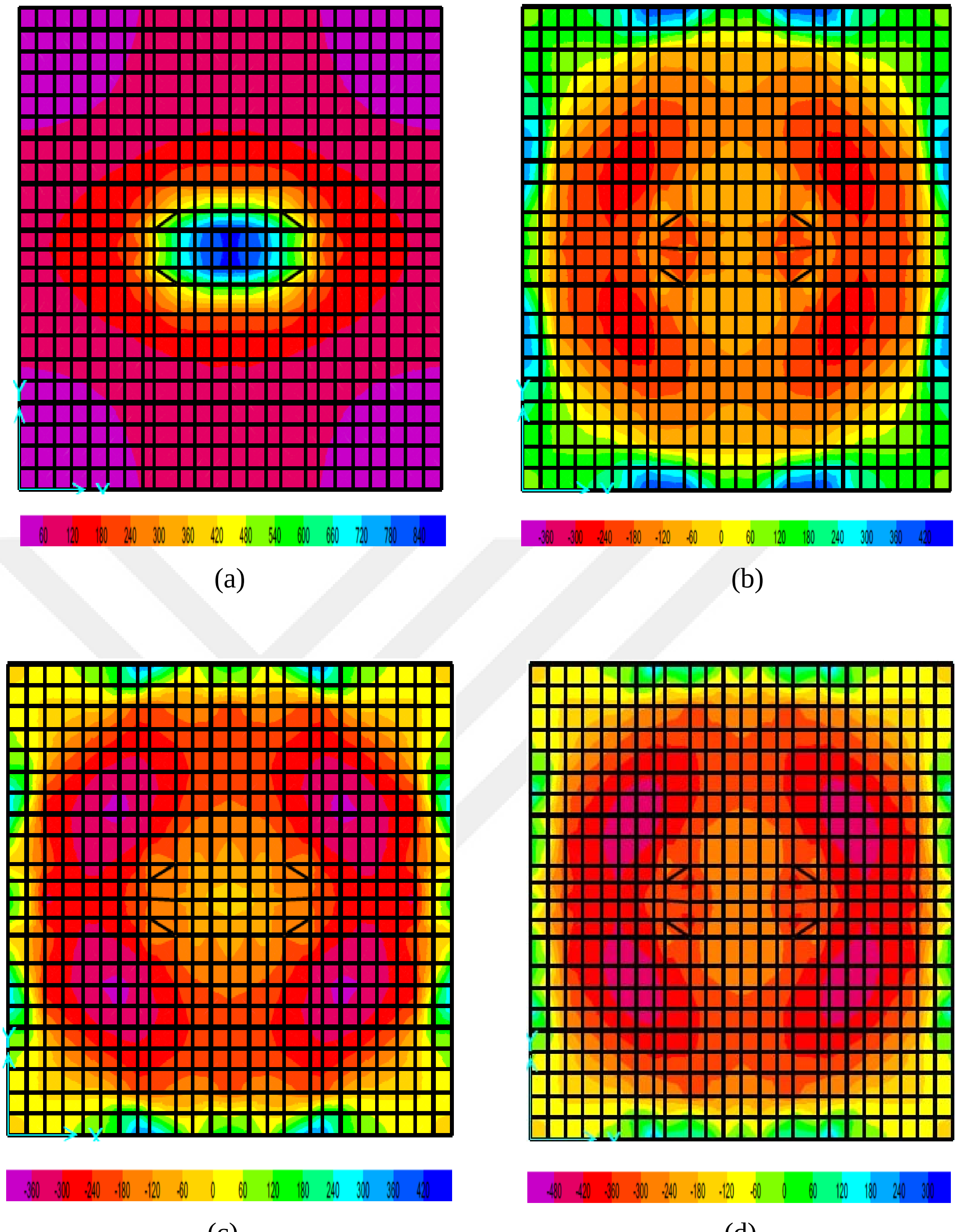

Figure 4.2 Foundation max moment distribution (a: without post-tensioning,b: with post-tensioning n = 19 s = 0.5 m, c: with post-tensioning n = 19 s = 0.75 m, d: with post-tensioning n = 19 s = 1.00 m)

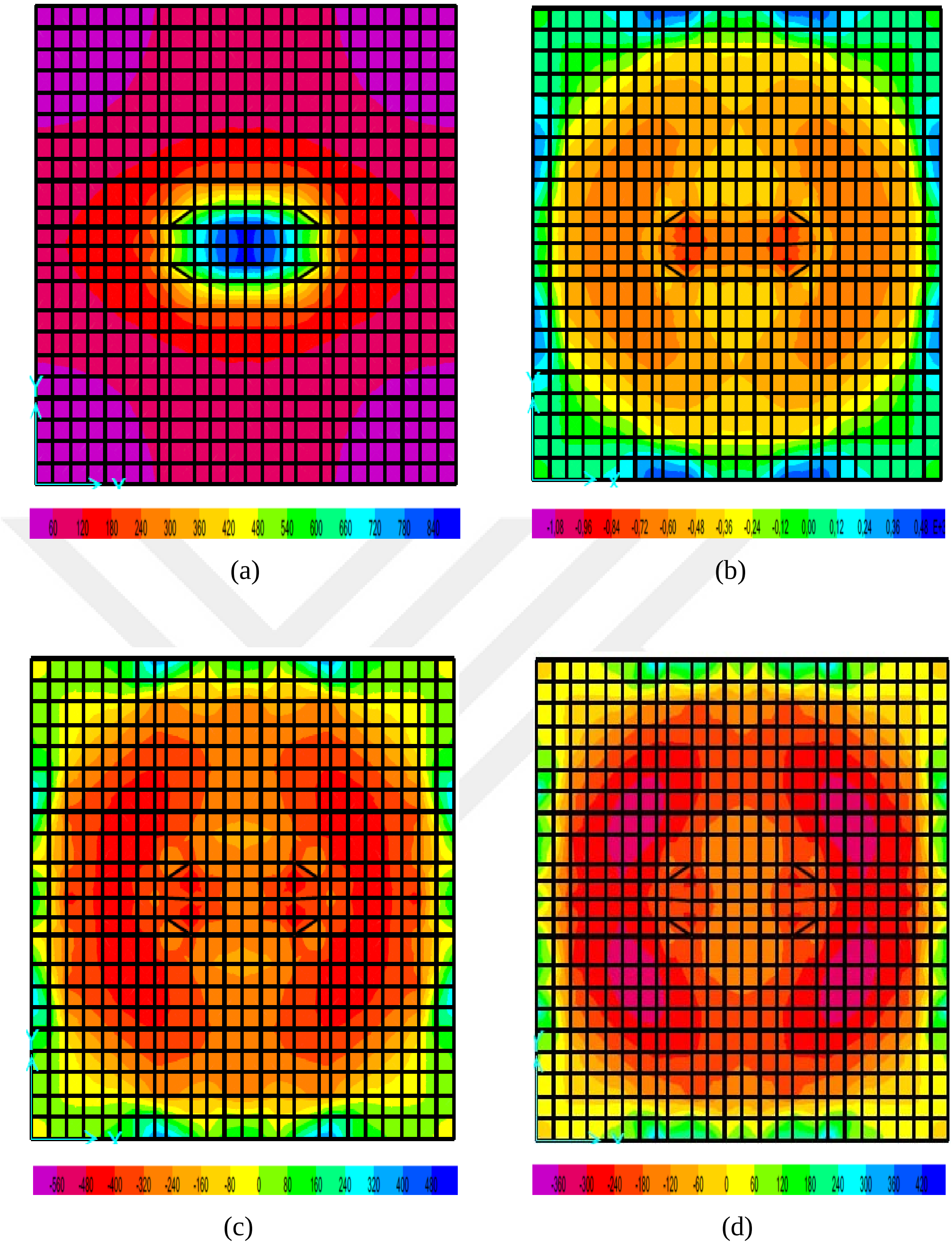

Figure 4.3 Foundation max moment distribution (a: Without post-tensioning, b: with post-tensioning n = 31 s = 0.5 m, c: with post-tensioning n = 31 s = 0.75 m, d: with post-tensioning n = 31 s = 1.00 m)

4.3.2 Foundation drawings with and without post-tensioning

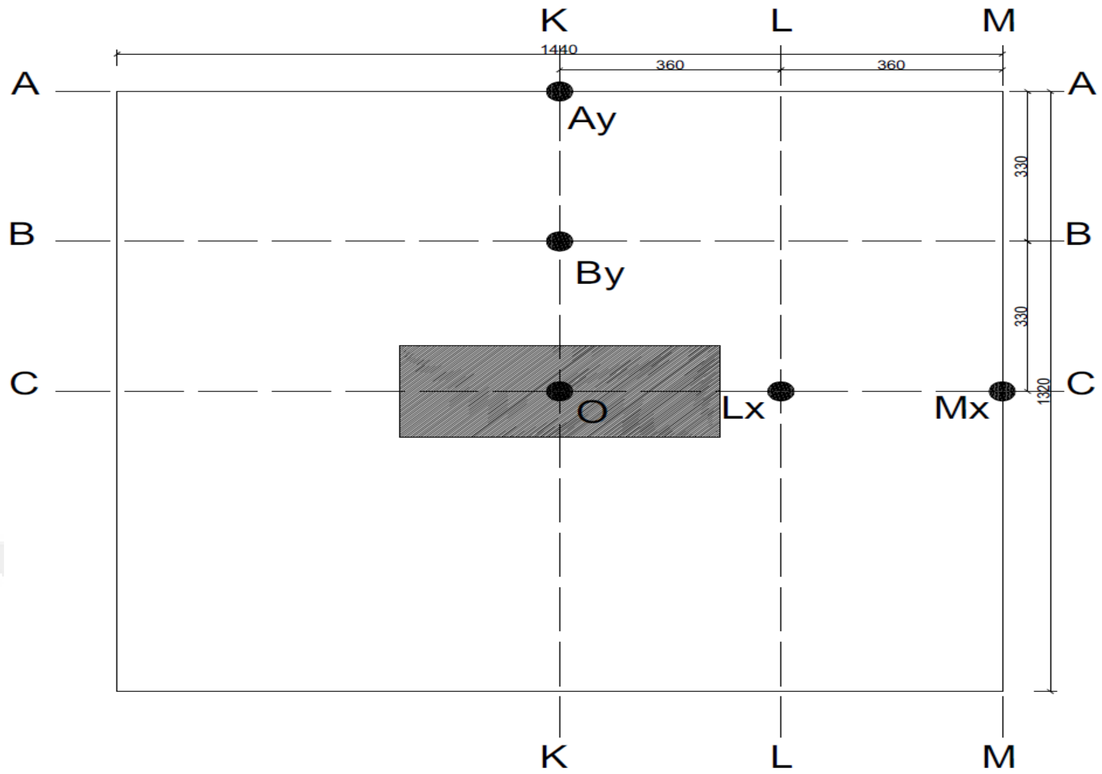

Figure 4.4 Foundation drawing without post-tensioning

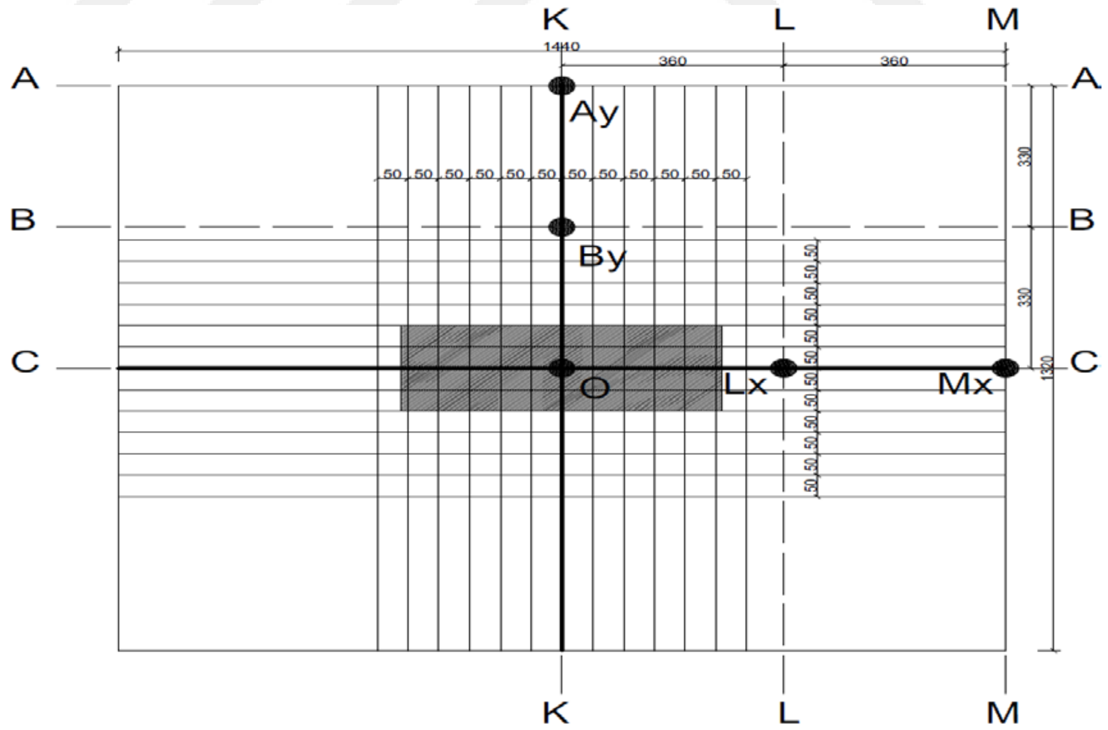

Figure 4.5 The foundation plan for the post-tensioning s = 0.5 m tendon spacing

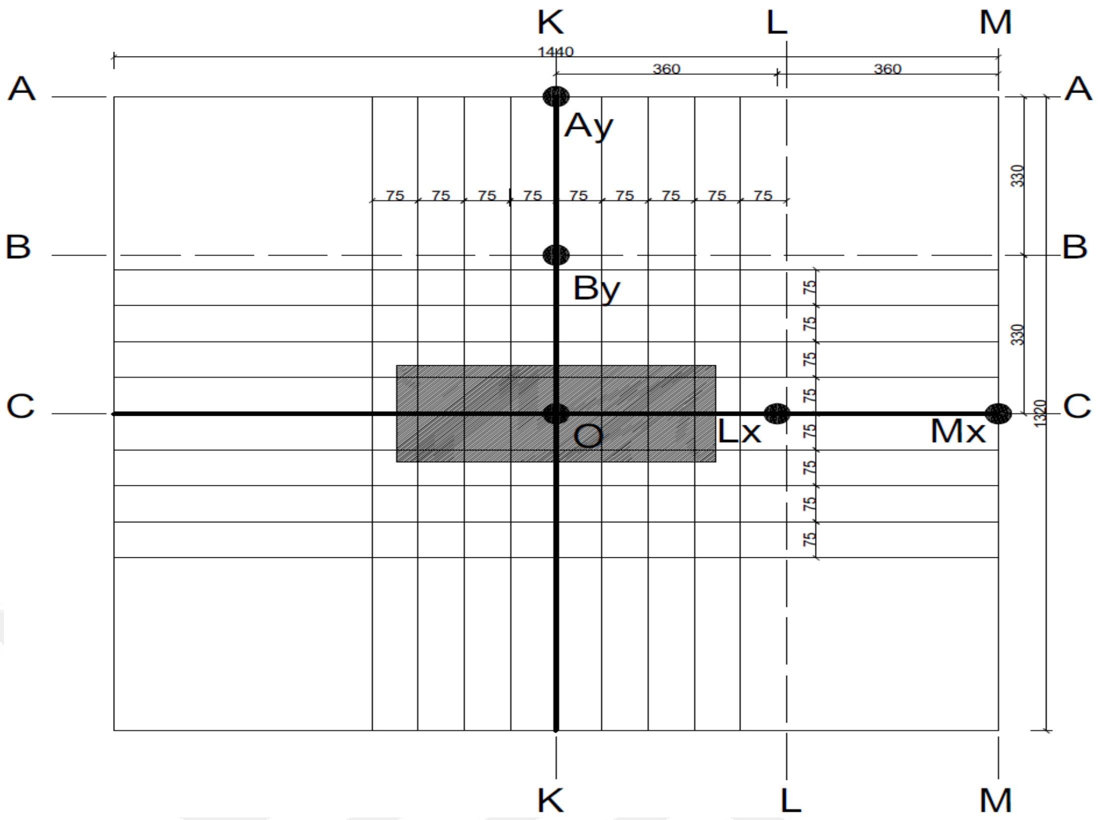

Figure 4.6 The foundation plan for the post-tensioning s = 0.75 m tendon spacing

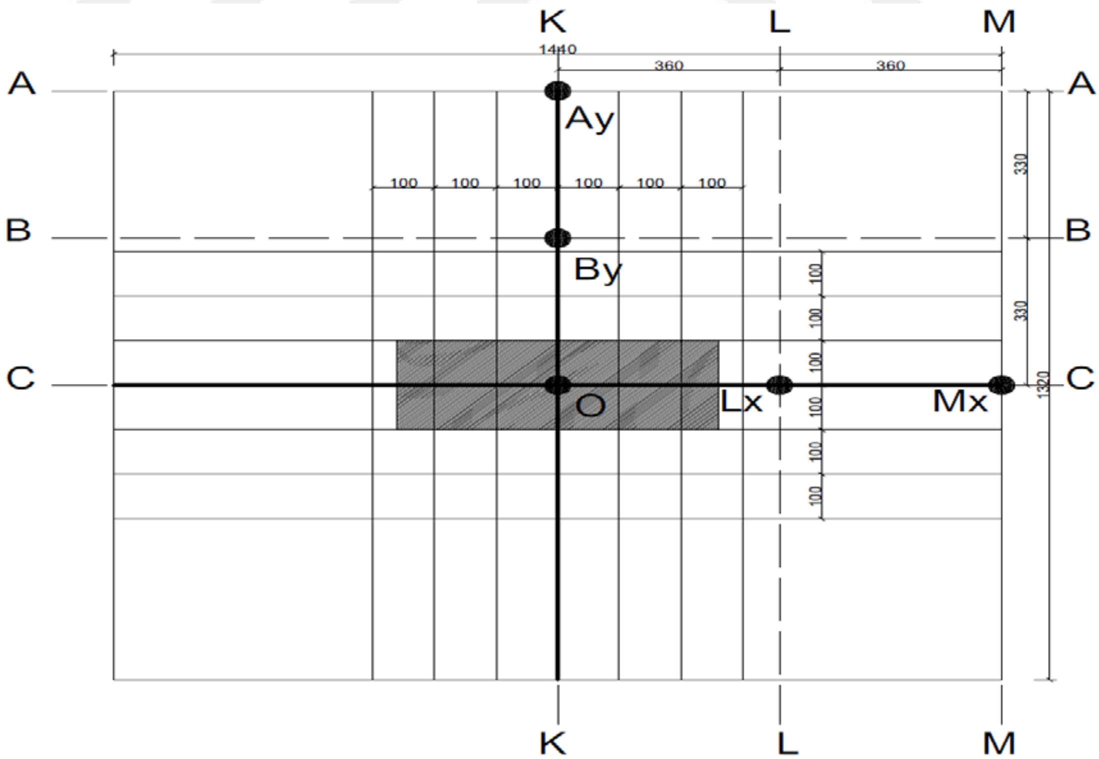

Figure 4.7 The foundation plan for the post-tensioning s = 1.00 m tendon spacing

4.3.3 Moment diagrams of the post-tensioning foundation

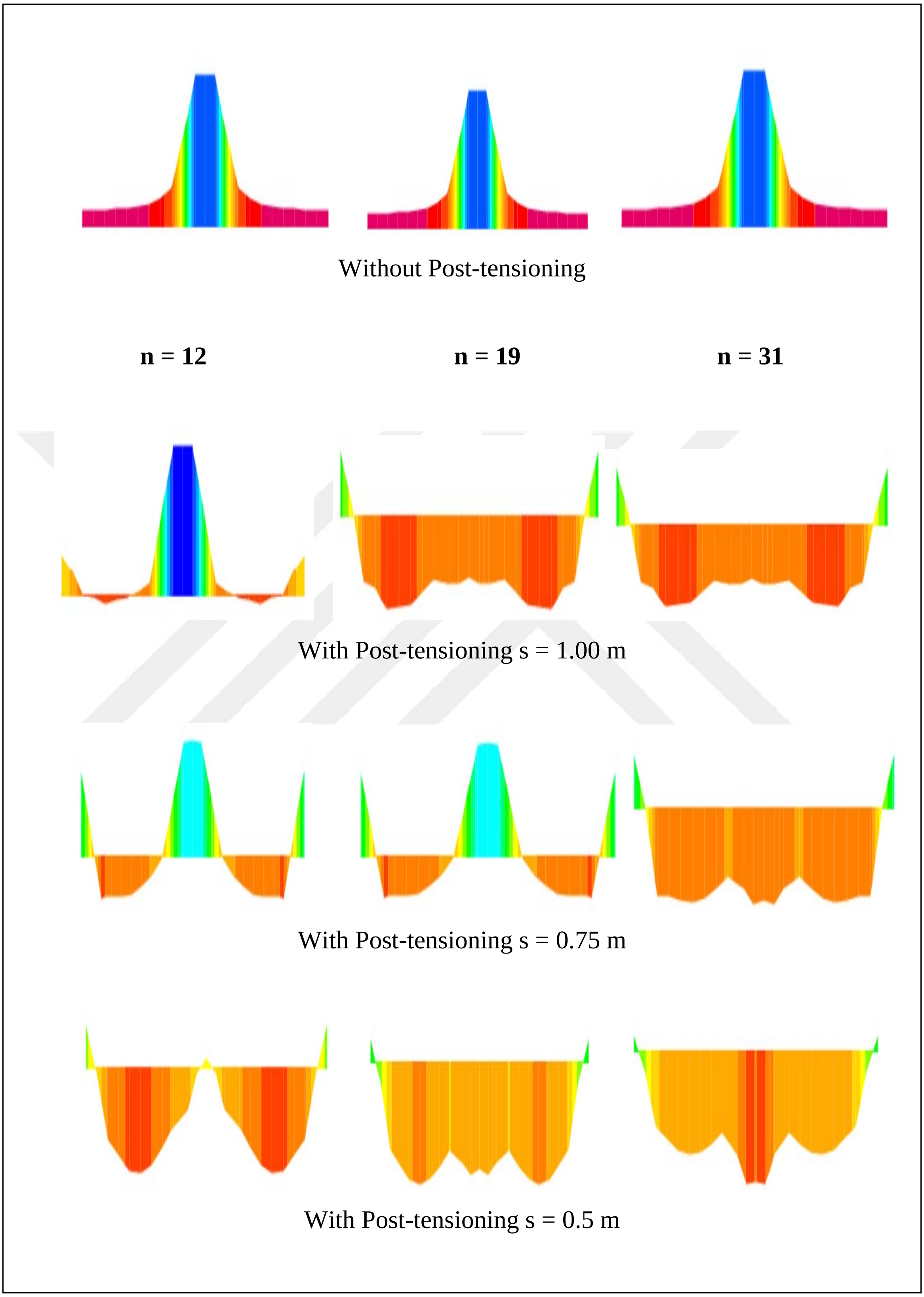

Figure 4.8 Moment diagrams of K-K cross section with post-tensioning foundation

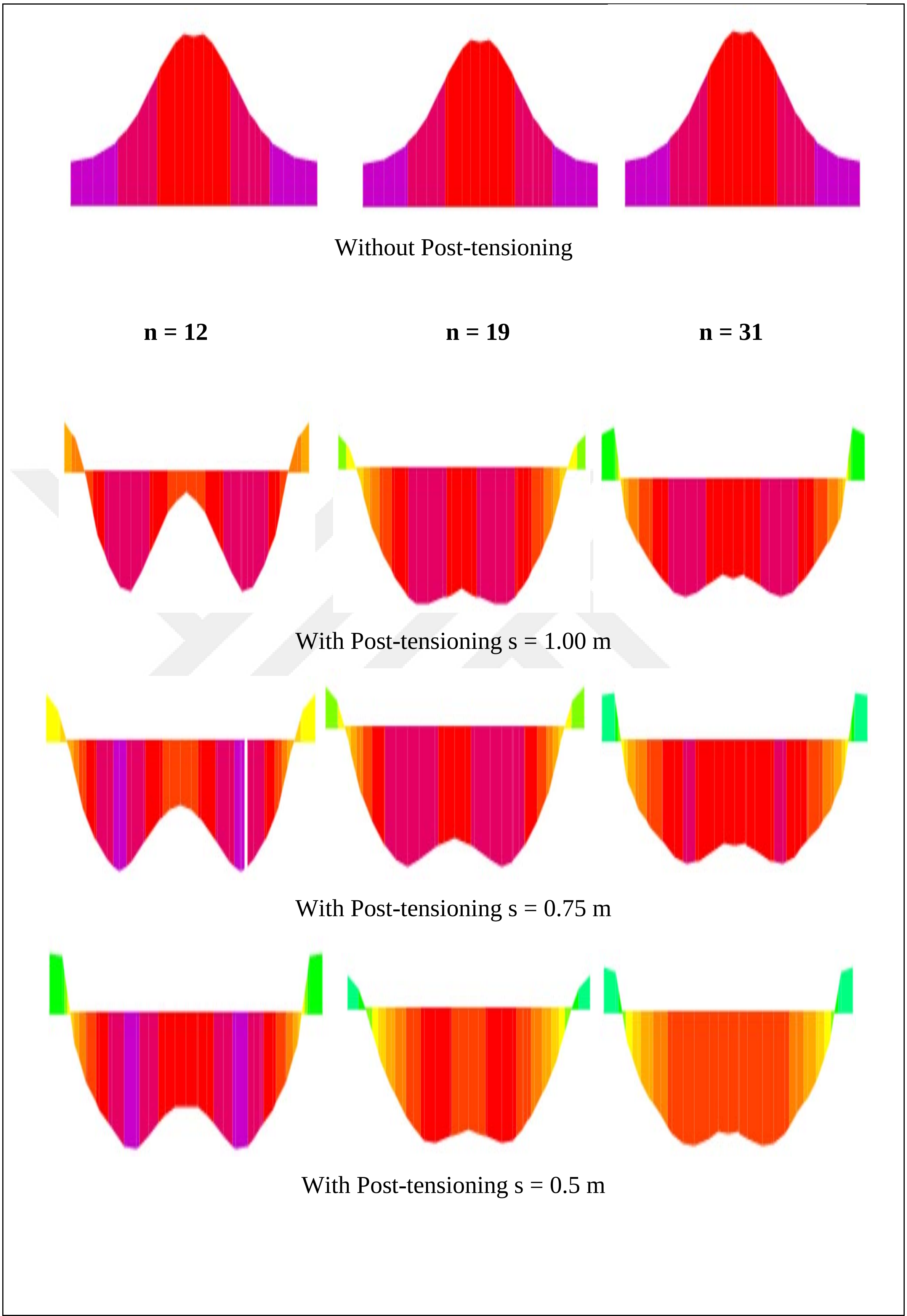

Figure 4.9 Moment diagrams of L-L cross section with post-tensioning foundation

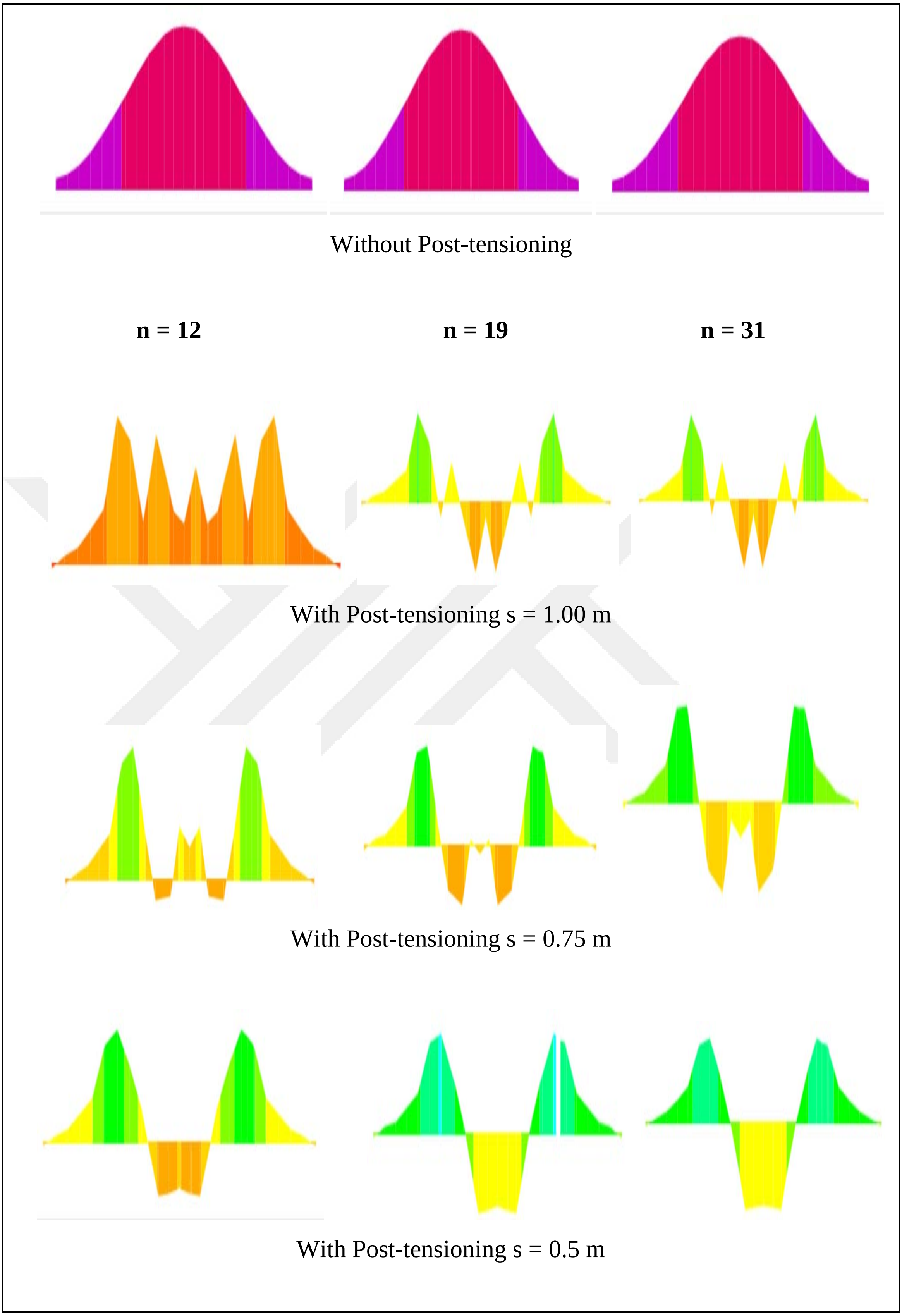

Figure 4.10 Moment diagrams of M-M cross section with post-tensioning foundation

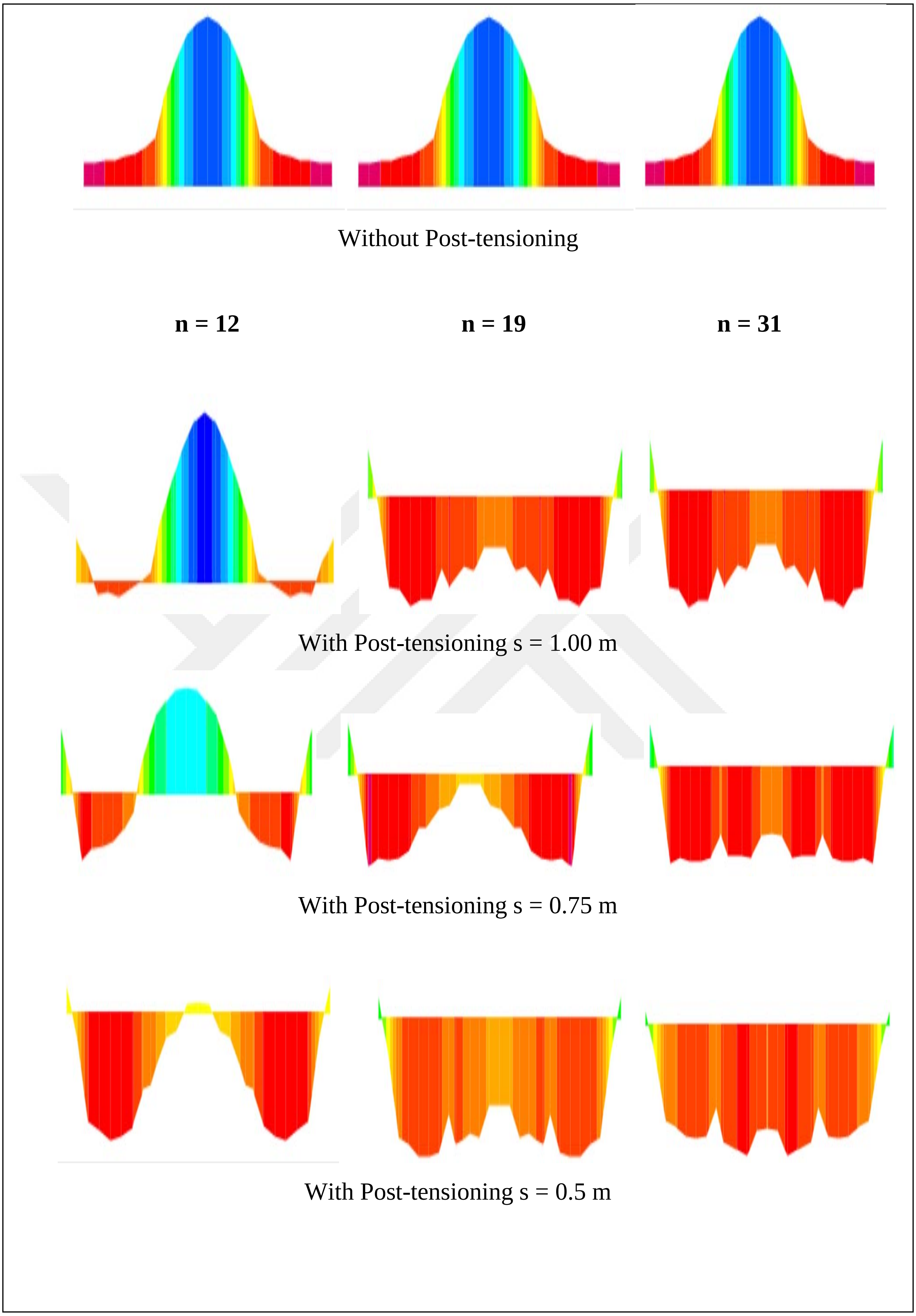

Figure 4.11 Moment diagrams of C-C cross section with post-tensioning foundation

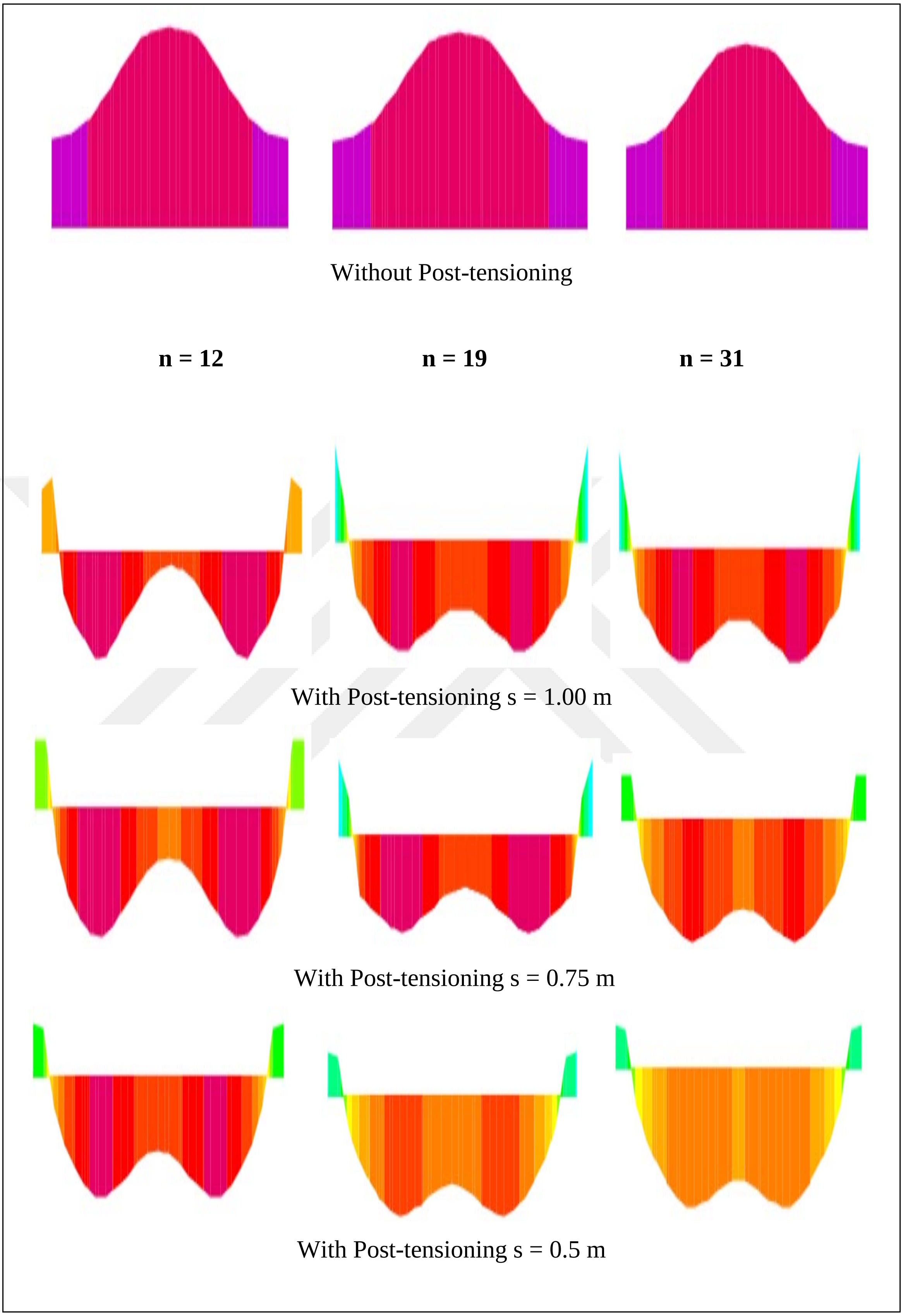

Figure 4.12 Moment diagrams of B-B cross section with post-tensioning foundation

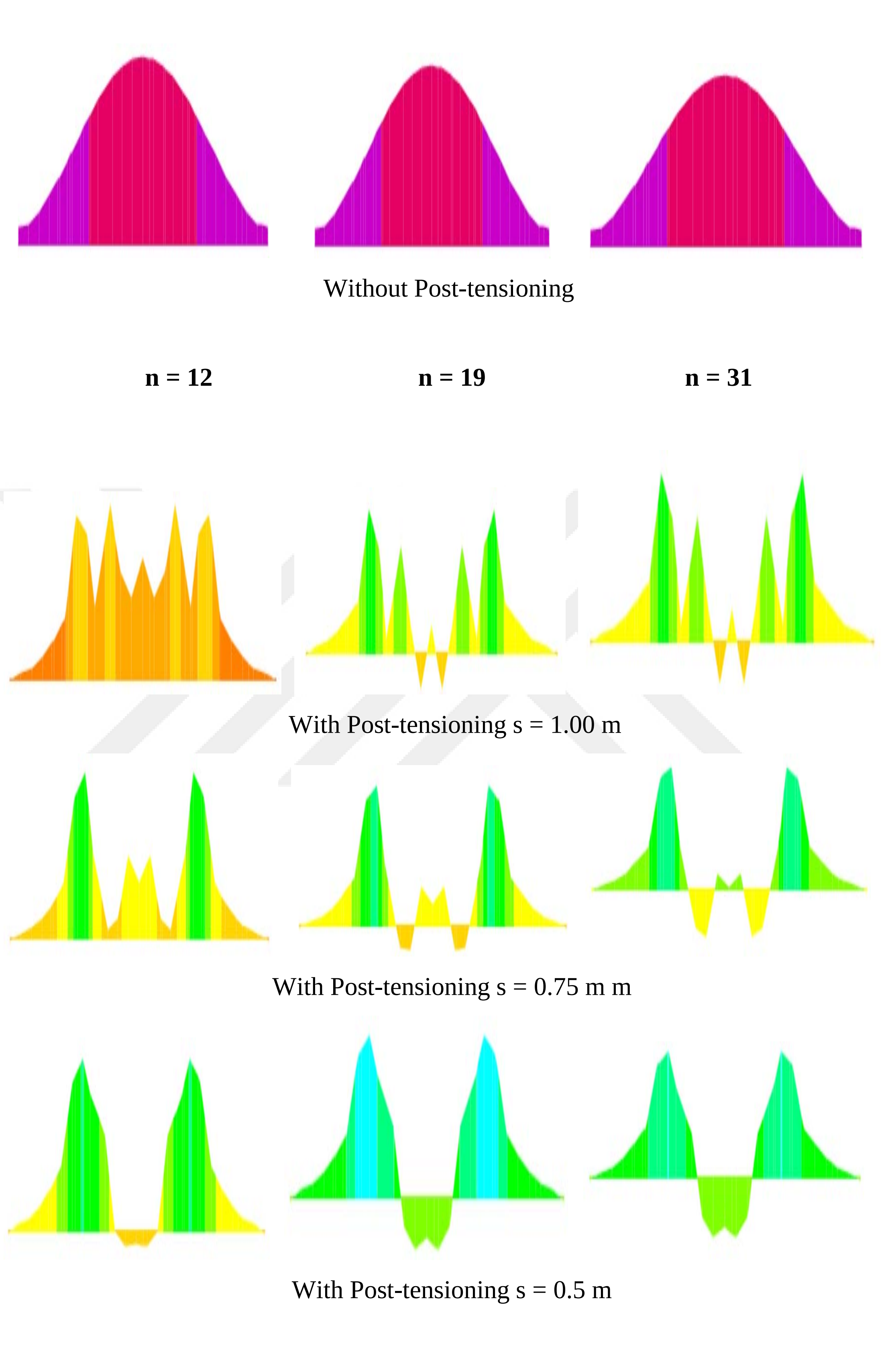

Figure 4.13 Moment diagrams of A-A cross section with post-tensioning foundation

4.3.4 Pressure values of the post-tensioning foundation

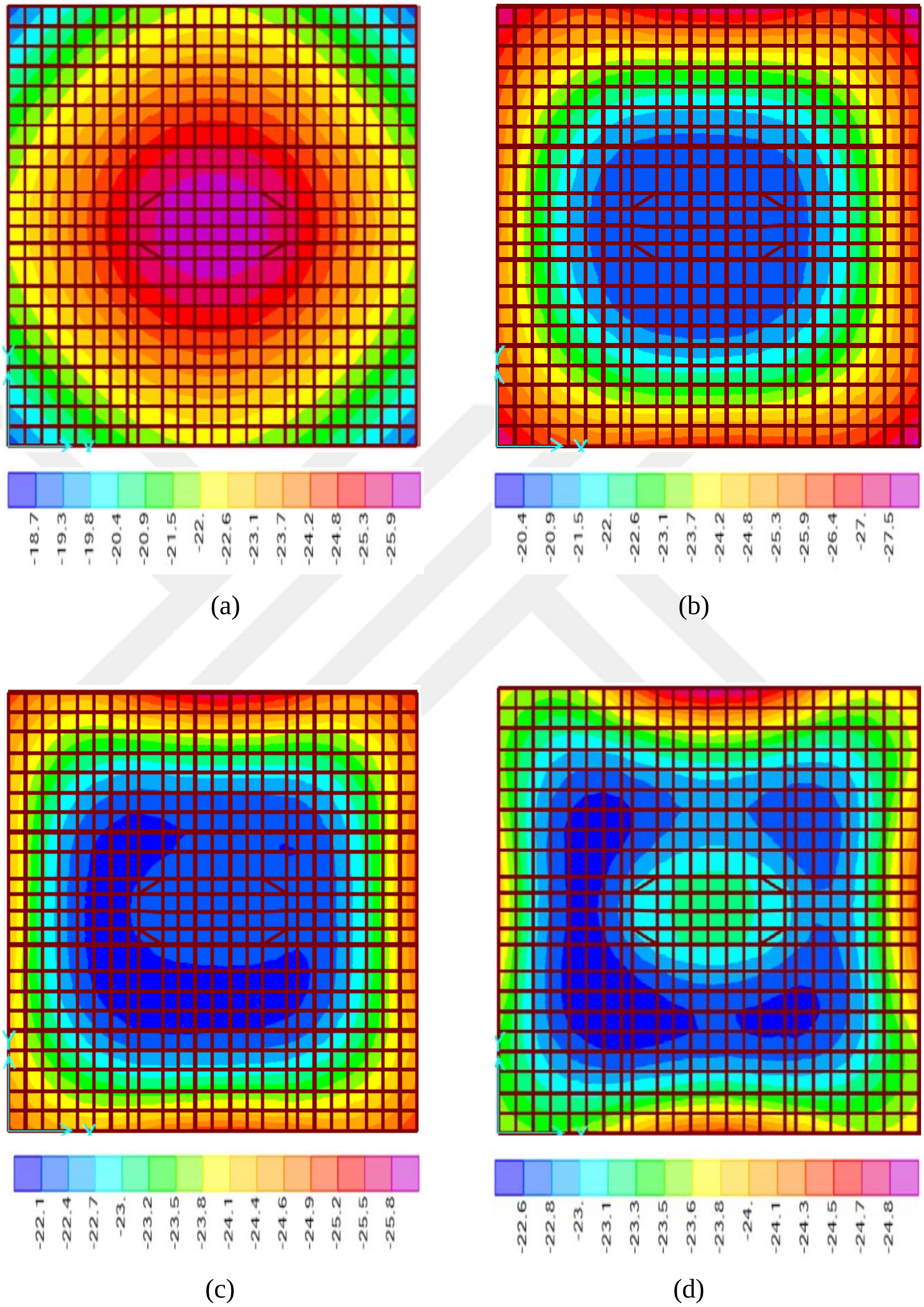

Figure 4.14 Foundation max momentum distribution (a: without post-tensioning, b: with post-tensioning n = 12 s = 0.5 m, c: with post-tensioning n = 12 s = 0.75 m, d: with post-tensioning n = 12 s = 1.00 m)

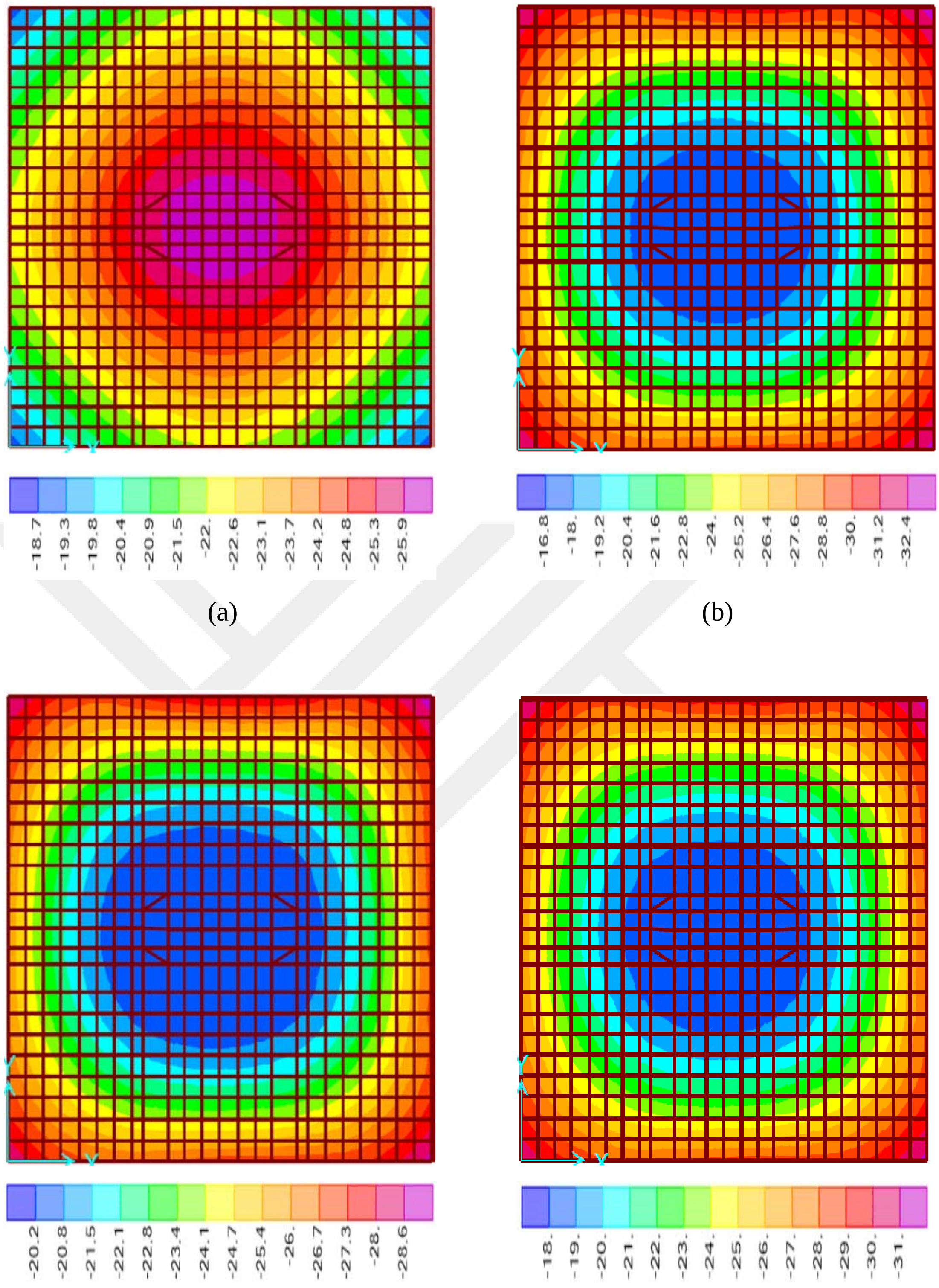

Figure 4.15 Foundation max momentum distribution (a: without post-tensioning, b: with post-tensioning n = 19 s = 0.5 m, c: with post-tensioning n = 19 s = 0.5 m, d: with post-tensioning n = 19 s = 1.00 m)

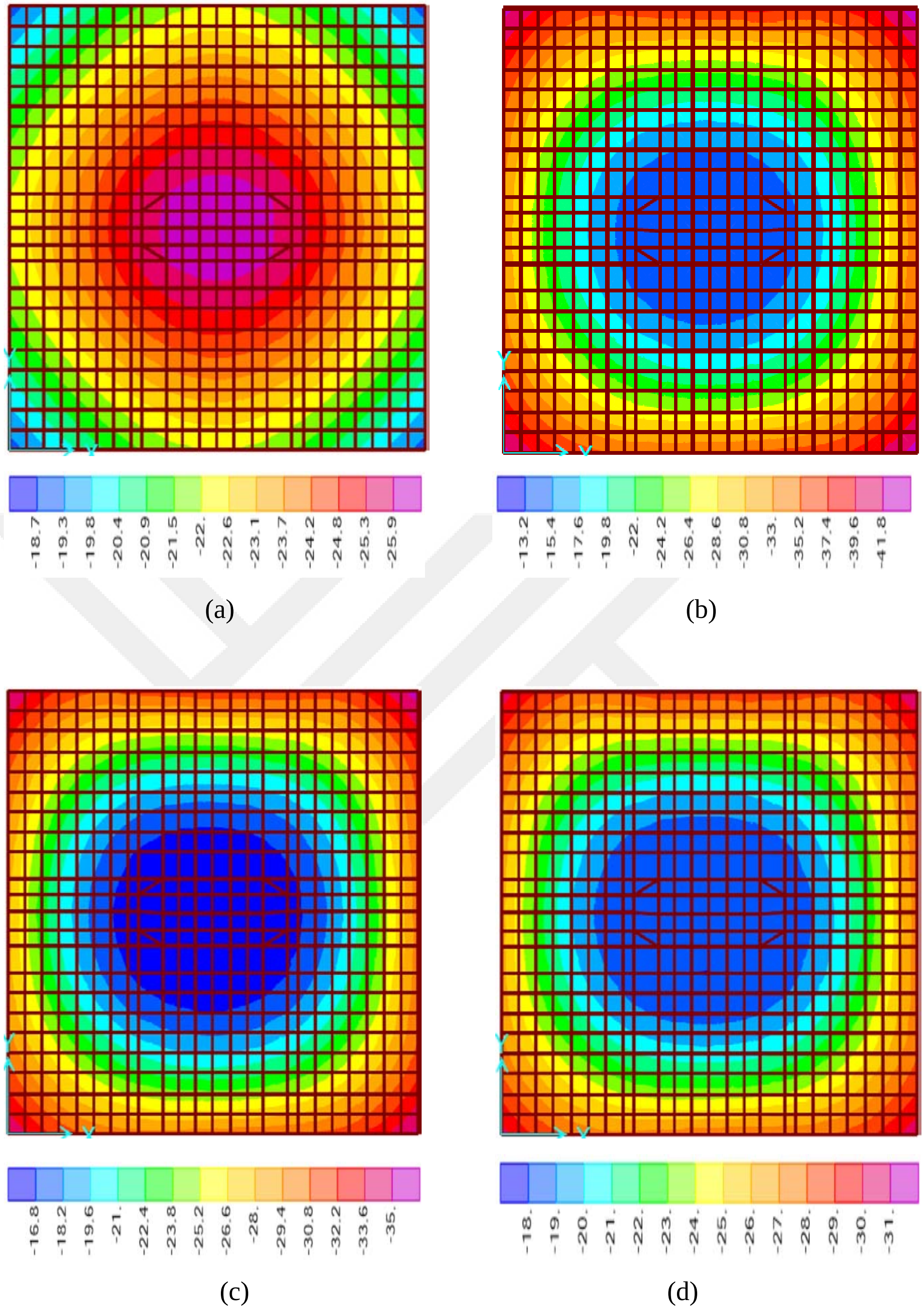

Figure 4.16 Foundation max momentum distribution (a: without post-tensioning,b: with post-tensioning n = 31 s = 0.5 m, c: with post-tensioning n = 31 s = 0.75 m, d: with post-tensioning n = 31 s = 1.00 m)

Table 4.2 Values in Critical Points q_{max} , M_{min} , M_{max} without Post-tensioning

COMB1			
Without posttensioning	q_{max} (t/m^2)	M_{min}(tm)	M_{max}(tm)
0	-25.90	402.98	854.33
By	-24.20	111.349	118.72
Ay	22.00	-2.11	87.44
Lx	-24.20	97.98	159.18
Mx	-21.80	-1.78	110.96

Table 4.3 Values at the critical points with post-tensioning (n = 12, s = 0.5 - 0.75- 1.00 m) q_{max}

COMB1 + PT-FINAL			
q_{max} (t/m^2)	**n=12 - s=0.5**	**n=12 - s=0.75**	**n=12 - s=1**
0	-20.40	-22.70	-23.30
By	-21.50	-23.40	-22.80
Ay	-27.00	-25.50	-24.70
Lx	22.00	-22.50	-21.60
Mx	-25.80	-24.40	-24.10

Table 4.4 Values at the critical points with post-tensioning (n = 19, s = 0.5 - 0.75-1.00 m) q_{max}

COMB1 + PT-FINAL			
q_{max} (t/m^2)	**n=19 - s=0.5**	**n=19 - s=0.75**	**n=19 - s=1**
0	-16.80	-20.20	-21.64
By	-19.70	-21.05	-25.56
Ay	-28.80	-27.50	-26.25
Lx	-20.40	-21.80	-22.12
Mx	-27.60	-26.00	-25.82

Table 4.5 Values at the critical points with post-tensioning (n = 31, s = 0.5 - 0.75-1.00 m) q_{max}

COMB1 + PT-FINAL			
q_{max} (t/m^2)	**n=31 - s=0.5**	**n=31 - s=0.75**	**n=31 - s=1**
0	-13.25	-16.00	-18.50
By	-16.14	-17.25	-20.12
Ay	-34.50	-30.10	-28.86
Lx	-18.05	-19.45	-20.50
Mx	-31.05	-28.25	-27.46

Table 4.6 Values in Critical Points q_{max} , M_{min} , M_{max} without post-tensioning

COMB1			
Without posttensioning	q_{max} **(t/m^2)**	M_{min}**(tm)**	M_{max}**(tm)**
0	-25.90	402.98	854.33
By	-24.20	111.349	118.72
Ay	22.00	-2.11	87.44
Lx	-24.20	97.98	159.18
Mx	-21.80	-1.78	110.96

Table 4.7 Values at critical points (n=12, s=0.5 - 0.75 - 1.00 m), M_{max} with post-tensioning

COMB1 + PT-FINAL						
	n=12 - s=0.5		**n=12 - s=0.75**		**n=12 - s=1**	
M_{min}, M_{max}**(tm)**	M_{min}	M_{max}	M_{min}	M_{max}	M_{min}	M_{max}
0	-189.08	10.54	133.61	139.69	194.97	297.00
By	-313	-111.20	-167.69	-35.46	-118.30	-9.57
Ay	-7.74	47.20	54.44	101.39	-35.44	78.96
Lx	-284.82	-159.80	-175.11	-63.25	-132.94	-14.50
Mx	-3.18	34.50	56.04	85.31	-34.50	74.68

Table 4.8 Values at critical points (n=19, s=0.5 - 0.75 - 1.00 m), M_{max} , M_{max} with post-tensioning

COMB1 + PT-FINAL						
	n=19 - s=0.5		n=19 - s=0.75		n=19 - s=1	
M_{min}, M_{max}(tm)	M_{min}	M_{max}	M_{min}	M_{max}	M_{min}	M_{max}
0	-789.2	-219.64	-278.98	-23.9	-28.2	72.64
By	-532.45	-242.5	-353	-132.15	-232.4	-75.56
Ay	-58.75	44.66	-35.12	111,44	1.20	96.25
Lx	-562.3	-348.5	-340.11	-192.18	-239.40	-114.34
Mx	-192.4	-51.77	-24.64	112.62	-20.11	91.65

Table 4.9 Values at critical points (n=31, s=0.5 - 0.75 - 1.00 m), M_{max} , M_{max} with post-tensioning

COMB1 + PT-FINAL						
	n=31 - s=0.5		n=31 - s=0.75		n=31 - s=1	
M_{min}, M_{max}(tm)	M_{min}	M_{max}	M_{min}	M_{max}	M_{min}	M_{max}
0	-1842.34	-612.91	-992.25	-293.45	-584.99	-136.92
By	-990.63	-482.75	-640.85	-298.60	-497.45	-204.00
Ay	-44.65	72.25	-13.66	-155.63	-136.17	143.00
Lx	-971.26	-666.13	-640.90	-409.71	-492.31	-287.00
Mx	-176.16	82.97	-28.65	182.63	-89.13	108.00

CHAPTER 5

CONCLUSIONS

In this study, the pier foundation of Denizli Hal Junction Bridge was analysed for 9 different post-tensioning cases with SAP2000 software. The tendons were selected to have either n=12, 19 or 31 ropes and their spacings were either s = 0.5m, 0.75m or 1m. The foundation had a rectangular shape with dimensions of BXL = 13.40 m x14.20 m with a foundation thickness of 2 m.

The results were compared in the meaning of maximum and minimum base pressures, base pressure distributions; bending moments and their distributions at six different cross-sections in the longitudinal and transverse directions. Below, firstly the findings about base pressure are summarized which is followed by the findings for moments and moment distributions.

5.1 Base Pressures and Pressure Distributions

The results were impressing such that significant improvement was obtained in the meaning of base pressures and pressure distributions. Although the maximum pressures were almost constant or even higher than the original case; their point of applications were removed from the center of the foundation to the edges.

Also, the stress concentrations observed in the original case (without post-tensioning) under the column was mostly converted into a much more uniform distribution spreading over a wider area at the expense of increased base pressures at the edges. This was attributed to the fact that the stress distributions were altered without changing the total vertical load. So any decrease in the vertical stress distribution at a part of the foundation would be expected to result in an increase at another location.

The pressures under columns were decreased up to 50% (for the case with n = 31 and s = 0.5m) but this much reduction had resulted in a increase at approximately the same amount. A noteworthy point was that the pressures under columns were decreased by 13% and 22% for the cases with n =12, s =1m and n = 19, s = 0.75 cases respectively without any significant increase in the maximum base pressures. Also the base pressure distributions were highly uniform for these two cases. This finding had indicated that, with a suitable design the maximum base pressures beneath the bridge pier foundations may be significantly reduced and the base pressure distributions may become much more uniform with the application of post-tensioning.

5.2 Bending Moments and Moment Distributions

The analyses have revealed that the improvement obtained in the bending moments and moment distributions with the application of post-tensioning into the bridge pier foundations was even better than the improvement obtained for the base pressures.

The maximum bending moment was decreased by 43% - 77% depending on the tendon type and spacing. The maximum decrease in the bending moments were obtained for the case with n = 12 and s = 0.75m. The uniformization in the bending moment distributions were most clearly seen in the case with n = 19 and s = 0.5m. It was concluded from this finding that, there exists an optimum solution for the ideal post tensioning which may differ from case to case.

However, it should here be stated that the bending moment directions were altered in some parts of the foundations for all of the analysed cases. This fact was more visible for the cases with n = 31. The moments in the counteract direction were significantly increased for these cases.

Nevertheless, as a result of these analyses it was seen that the maximum bending moments could be significantly reduced and the moment distributions may become much more uniform with a proper post-tensioning design in bridge pier foundations. However, care should be taken to not to result in significantly increased bending moments in the reverse direction of the moments observed in the case without posttensioning.

In this study, it was aimed to investigate the effect of post-tensioning in the bridge pier foundations. The analyses have revealed that it was possible to improve the behaviour of the foundation both in base pressures and pressure and moment distributions. The results have also revealed that the maximum base pressures may be reduced with the help of post-tensioning which may open a way for future studies investigating the possible use of post-tensioning against bearing capacity problems.

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ANNEXES

CONCRETE DESIGN OF THE BRIDGE FOUNDATION

A.1 Introduction

In this section, what kind of changes in the cross-sectional design which is solved by applying reinforced concrete and posttensioning technology in the middle bearings of the Hal Junction Bridge which is solved by posttensioning technology were researched.

The cross-junction bridge is a two-span bridge with account openings L1 = 35.00 m, L2 = 60.00 m and L3 = 35.00 m. The width of the bridge from outside the border to outside the border is 14.49 m. The height of the superstructure for the casted posttensioning system is 140 cm to 280 cm. There are 6 cm thick coating on the support system including 1 cm water insulation and 5 cm asphalt. The bridge was built in single stage. The ropes were drawn in one step.

A.2 LOADS

A.2.1 Dead Loads

The basis weights for these loads are:

Concrete (prestressed and reinforced concrete) : 25.0 kN / m^3

Asphalt concrete, protective concrete and isolation : 23.0 kN / m^3

Railing : 0.5 kN / m

Barrier : 1.0 kN / m

A.2.2 Movement Loads

H30-S24 and KGM45 loads were considered to be suitable for the bridge to be designed.

A.3 Material Features

A.3.1 Concrete

C50 for the superstructure concrete and C30 for the foundation concrete.

Permissible pressure stress for the class of concrete σall, b = 0.45 fc = 22.5 MPa

Permissible tensile stress for the class of concrete σall, d = 0.5 gerfc √fc= 3.54 MPa

Elasticity module E = 4800 $\sqrt{f_c}$ = 33941 MPa

A.3.2 Posttensioning Rope

Breaking strength fpu = 1862 MPa

Design breaking strength 0.75 fpu = 1397 MPa

Approximately 20% of the losses after removal 0.8 x 1397 = 1117 MPa

A.4 Reinforced Concrete Raft Foundation Calculation (Middle Foot)

The basic calculation of the project has been done with two solutions as with and without posttensioning. The SAP2000 program was used for basic design and analysis. In the basic design, the values obtained from the superstructure were received from the projector and the analyzes of the load transferred from the columns were analyzed.

COMB1: STATIC COMBINATION WITHOUT FACTOR

COMB1 = 1.0 * (DL) + (1.0) * (E) + (1.0) * (LL)

COMB2: STATIC COMBINATION WITH FACTOR

COMB2 = 1.30 * (DL) + (1.30) * (E) + (2.17) * (LL)

COMB3: EARTHQUAKE COMBINATION IN THE NECK

COMB3 = 1.0 * (DL) + 1.0 * (E) + (1.0) * (EQ x)

COMB4: EARTHQUAKE COMBINATION IN HORIZONTAL DIRECTION

COMB4 = 1.0 * (DL) + 1.0 * (E) + (1.0) * (EQ Y)

A.5 Analysis Results

Aks	Element	EQX+0.3EQY				0.3EQX+EQY			
		Hx	Mx	Hy	My	Hx	Mx	Hy	My
P1	**50**	699	14340	2189	5076	2330	4301	657	16921
P2	**51**	697	13881	2186	4929	2324	4164	656	16431

Aks	Element	DC + DW			
		Hx	Mx	Hy	My
P1	**50**	1	1075	164	85
P2	**51**	1	1042	164	85

Combined Seismic Forces

Aks	Element	EQX+0.3EQY+DL				0.3EQX EQY + + DL			
		Hx	Mx	Hy	My	Hx	Mx	Hy	My
P1	**50**	700	15415	2353	5161	2331	5376	821	17006
P2	**51**	698	14923	2350	5014	2325	5206	820	16516

Counteracting behavior change factor (R) = 3 (Longitudinal direction)

Counteracting behavior change factor (R) = 3 (Horizontal direction)

Modified Seismic Forces

Aks	Element	Longitudinal Direction				Horizontal Direction			
		Hx	Mx	Hy	My	Hx	Mx	Hy	My
P1	**50**	700	5855	2353	1777	2331	2509	821	5725
P2	**51**	698	5669	2350	1728	2325	2430	820	5562

Counteracting behavior change factor (R) = 1 (Longitudinal direction)

Counteracting behavior change factor (R) = 1 (Horizontal direction)

Changed Forces For Foundation Calculation

Aks	Element	Longitudinal Direction				Horizontal Direction			
		Hx	Mx	Hy	My	Hx	Mx	Hy	My
P1	**50**	700	15415	2353	5161	2331	5376	821	17006
P2	**51**	698	14923	2350	5014	2325	5206	820	16516

Loads Affecting Foundation

Aks	Element	Longitudinal Direction				Horizontal Direction			
		Hx	Mx	Hy	My	Hx	Mx	Hy	My
P1	**50**	**700**	**15415**	**2353**	**5161**	**2331**	**5376**	**821**	**17006**
P2	**51**	698	14923	2350	5014	2325	5206	820	16516

Dead Load on the Basis: P = 25113 / 9.58 = 2620.3 kN / m^2

Basic Weight: P = 2.00 x 25 = 50 kN / m^2

Moving Load: P = 1496.00 / 9.58 = 156.09 kN / m^2

Earth Load over Foundation: P = 2.00 x 19.20 = 38.04 kN / m^2

(AASHTO, 2012) Load Combinations

Loading Effects	**Comb1**	**Comb2**	**Comb3**	**Comb4**
DL	**1.0**	1.30	1.0	1.0
FOUNDATION	**1.0**	1.30	1.0	1.0
MOVEMENT	**1.0**	2.17		
EARTH	**1.0**	1.30	1.0	1.0
EQX+0.3EQY+DL			1.0	
0.3EQX+ EQY + DL				1.0

A.6 Sap 2000 Analysis Results

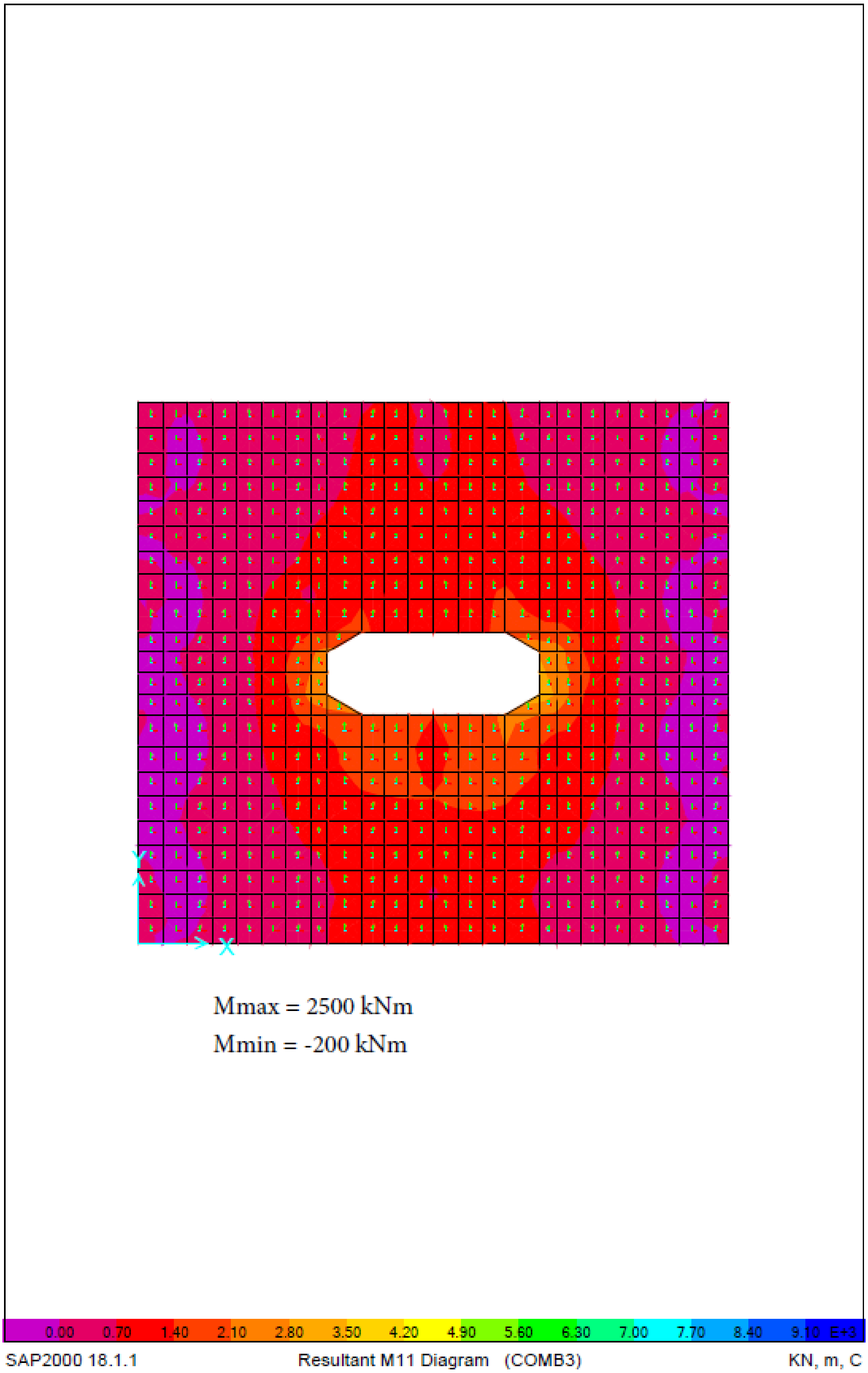

Figure A.1 Foundation max, min moment M11 (COMB3)

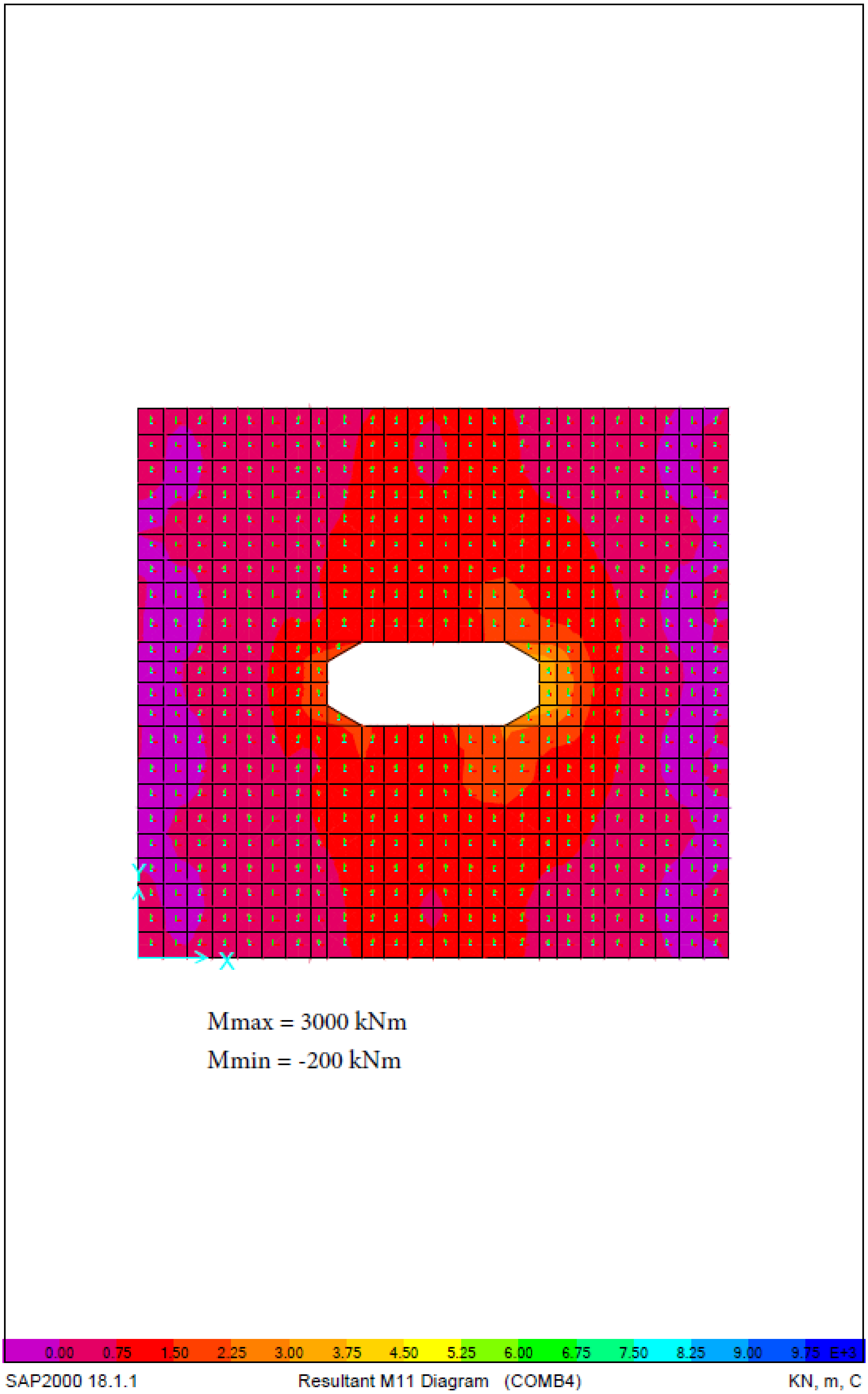

Figure A.2 Foundation max, min moment M11 (COMB4)

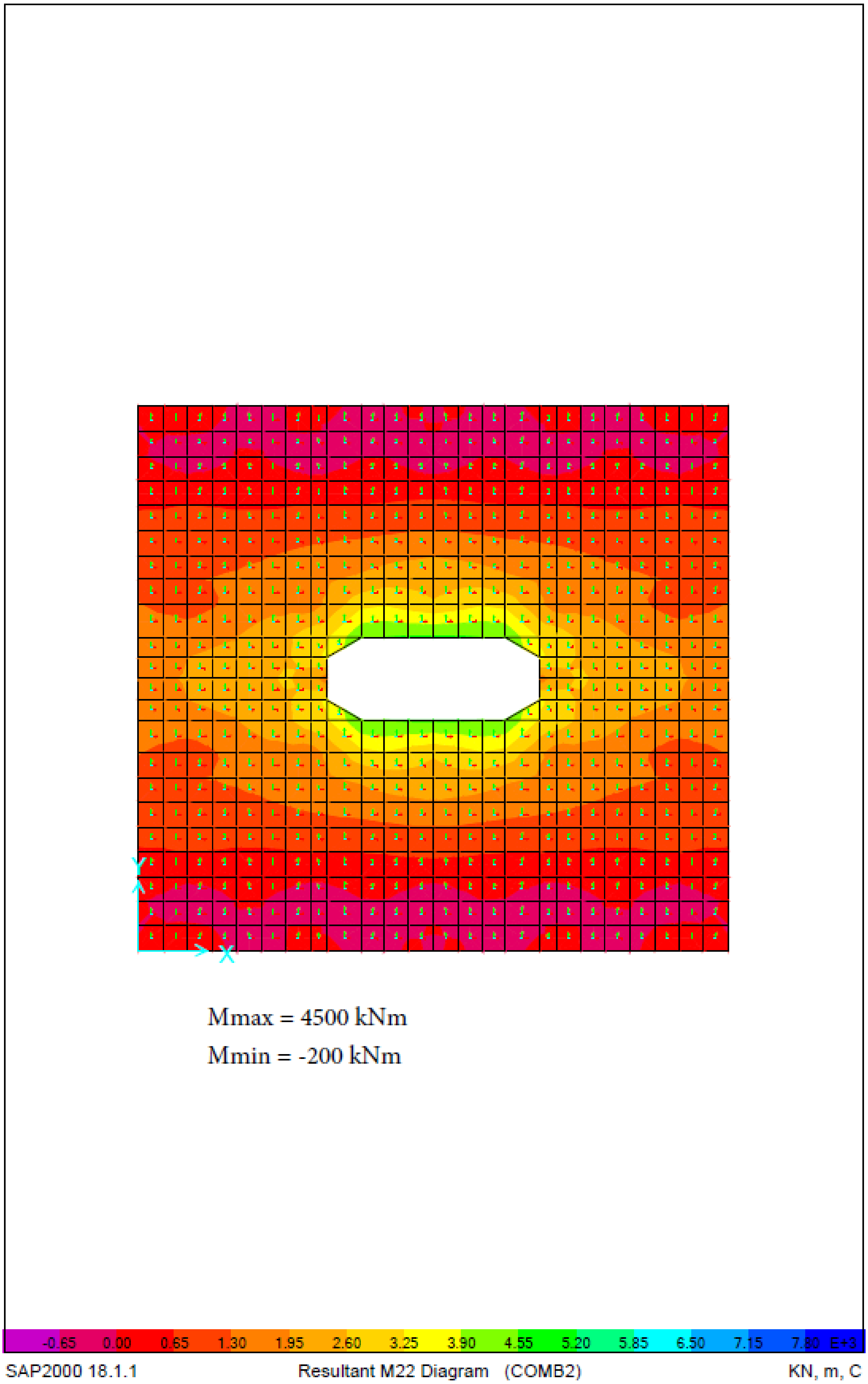

Figure A.3 Foundation max, min moment M22 (COMB2)

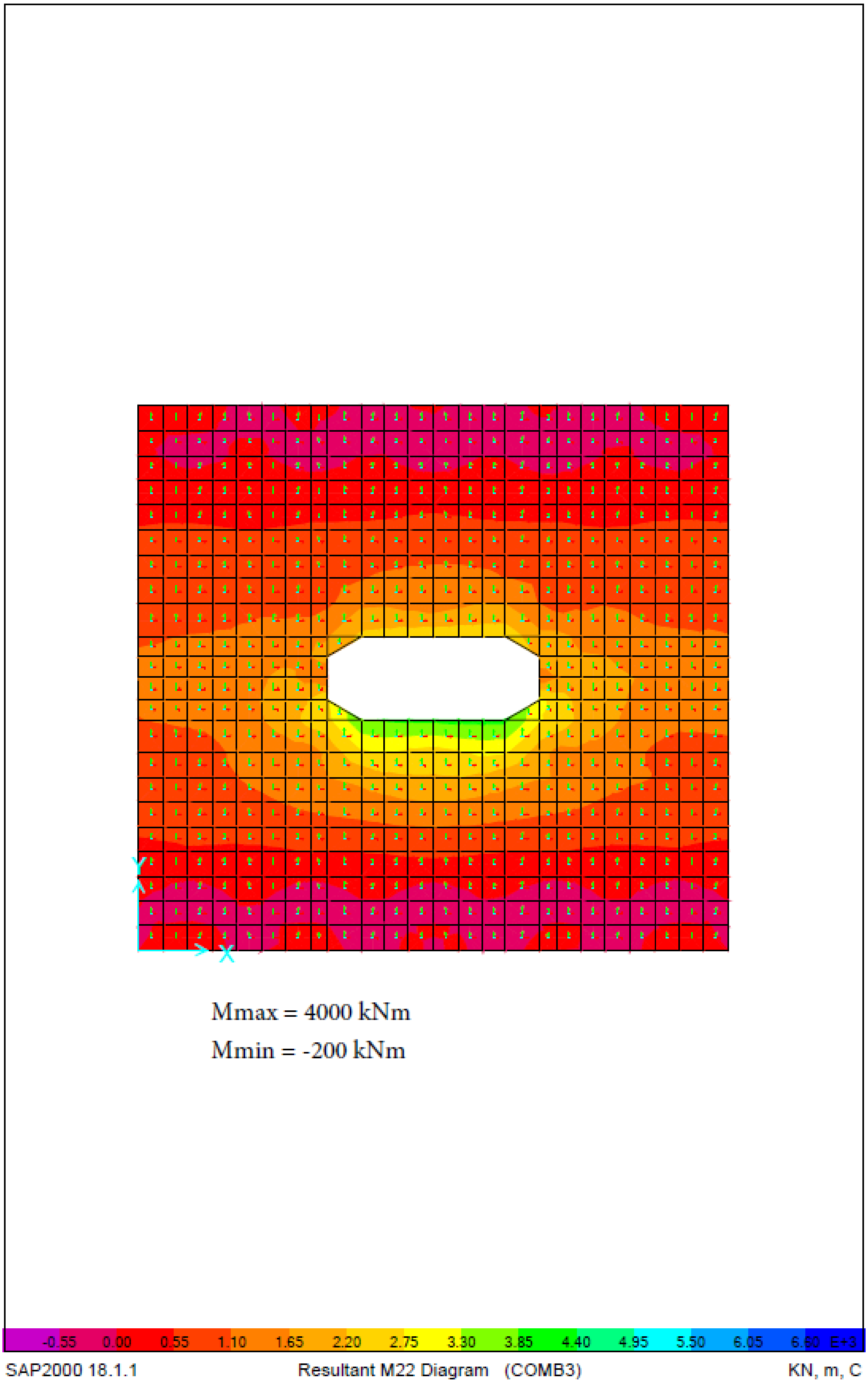

Figure A.4 Foundation max, min moment M22 (COMB3)

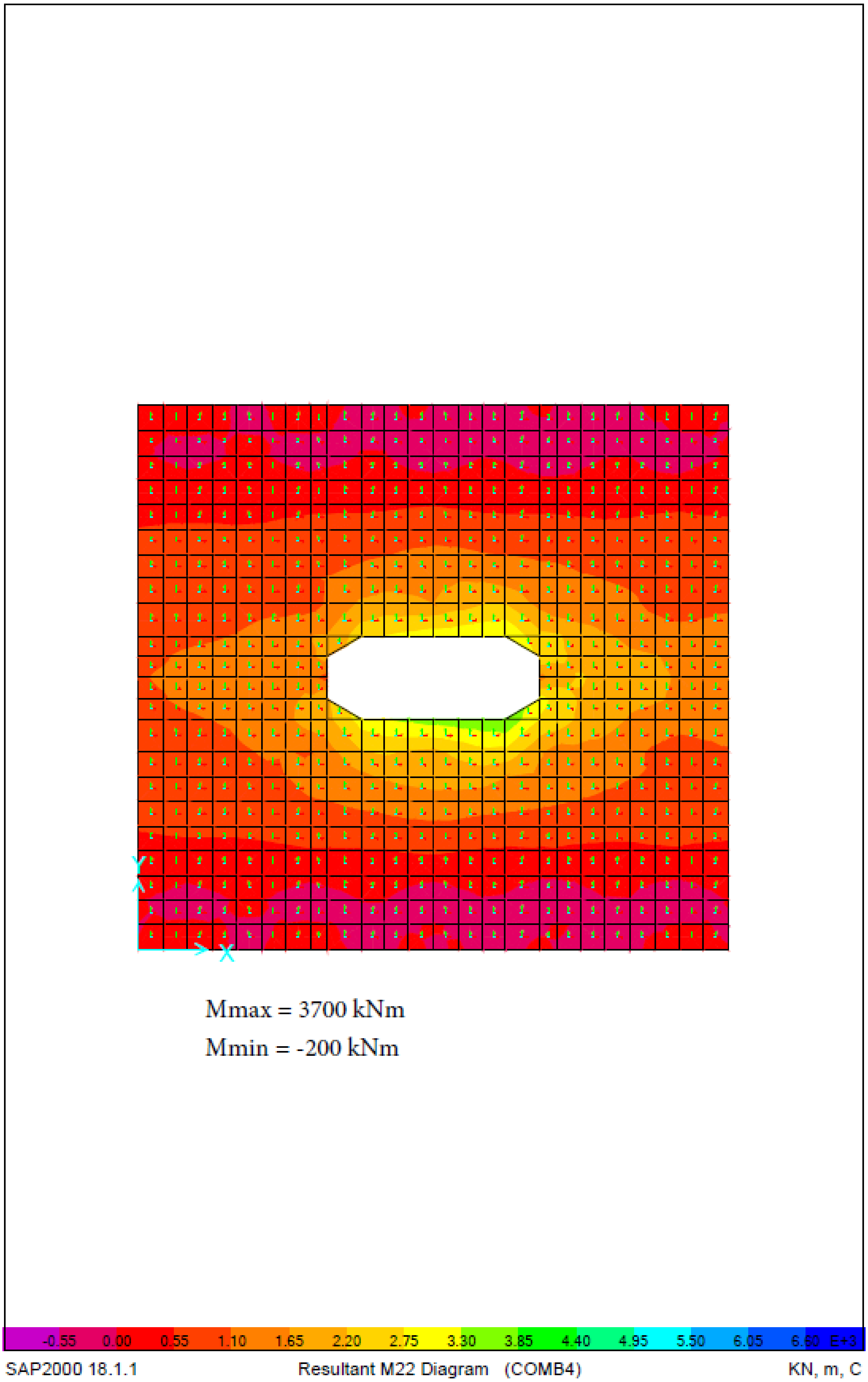

Figure A.5 Foundation max, min moment M22 (COMB4)

A.7 Foundation Bridge Longitudinal Reinforcement Calculation

Mu (kNm)	Φ (ASSHTO)	fc (MPa)	fy (MPa)	d (mm)	d' (mm)	bw (mm)	d' (mm)
4500.00	0.9	30	420	2000	1900	1000	100

$\emptyset M_n = \emptyset\left[A_s f_y\left(d' - \frac{a}{2}\right)\right]$ (AASHTO 8.16.3.2) Flexion bearing capacity

$\alpha = \frac{A_s f_y}{0.85 f_c' b_w}$ (AASHTO 8.16.3.2) Equivalent pressure block depth

$m = \frac{f_y}{0.85 f_c'}$ $R_n = \frac{M_u}{\phi b\, d^2}$ $\rho = \frac{1}{m}\left[1 - (1 - \frac{2 m R_n}{f_y})'\right]$

m = 16.47 R_n = 1:39 N/mm^2 ρ = 0.00339

Mu = 4500 KN required for the amount of reinforcement $A_{s,gerekli} = 6446\ mm^2$

Minimum reinforcement area

(AASHTO 8.17.1.1)

fr = 3.41 Mpa Breaking module

Ig = 6.67E +11 mm 4 Moment of inertia relative to the geometric center of the section

yt = 1000 mm Distance of the center of gravity of the section to the bottom

As = Quantity of equipment

Mcr = 2274.87 kNm Cracking moment $M_{cr} = f_r \frac{I_g}{y_t}$ (AASHTO 8.13.3)

1.2 Mcr = 2729.85 kNm $\longrightarrow$ As, min = 3866 mm^2

Minimum reinforcement requirement is not required if the reinforcement area at the section is 1.33 times greater than the required reinforcement area at the end of the analysis (AASHTO 8.17.1.2)

1.33 As, required = 8573mm^2 Ac, min = min (3866, 8573) = 3866 mm^2

Balanced equipment ratio

$\rho_b = \frac{0.85\beta_1 f_c'}{f_y}\left(\frac{599.843}{599.843 + f_y}\right)$ $fc' = 30$ MPa β_1 = 0.8324

$\rho_b = 0.029727$

Selected reinforcement = **Ø** 32/20 + **Ø** 26/20

Reinforcement Area As = 6676mm^2

As, required = 6446 mm^2

As = 6676 mm^2

As, min = 3866mm^2

ρ = 0.003514 <0.75 ρb = 0.022295 OK ρ <0.75 ρ_b (AASHTO 8.16.3.1.1)

A.8 Basic Bridge Transverse Bottom Reinforcement Account

Mu (kNm)	Φ (ASSHTO)	fc (MPa)	FY (MPa)	d (mm)	d' (mm)	bw (mm)	d' (mm)
3500.00	0.9	30	420	2000	1900	1000	100

$\emptyset M_n = \emptyset\left[A_s f_y \left(d' - \frac{a}{2}\right)\right]$ (AASHTO 8.16.3.2) Flexion bearing capacity

$\alpha = \frac{A_s f_y}{0.85 f_c' b_w}$ (AASHTO 8.16.3.2) Equivalent pressure block depth

$m = \frac{f_y}{0.85 f_c'}$ $R_n = \frac{M_u}{\phi b\, d^2}$ $\rho = \frac{1}{m}\left[1 - (1 - \frac{2mR_n}{f_y})'\right]$

m = 16.47 R_n = 1:08 N/mm^2 ρ = 0.00262

reinforcement amount required for Mu = 3500 KN $A_{s,gerekli}$ = 4981 mm^2

Minimum reinforcement area

(AASHTO 8.17.1.1)

fr = 3.41 Mpa Breaking module

Ig = 6.67E +11 mm 4 Moment of inertia relative to the geometric center of the section

yt = 1000 mm Distance of the center of gravity of the section to the bottom

As = Quantity of equipment

Mcr = 2274.87 kNm Cracking moment $M_{cr} = f_r \frac{I_g}{y_t}$ (AASHTO 8.13.3)

1.2 Mcr = 2729.85 kNm $\longrightarrow$ As, min = 3866 mm2

Minimum reinforcement requirement is not required if the reinforcement area at the section is 1.33 times greater than the required reinforcement area at the end of the analysis (AASHTO 8.17.1.2)

1.33 As, required = 6624 mm^2 Ac, min = min (3866, 6624) = 3866 mm^2

Balanced equipment ratio

$$\rho_b = \frac{0.85\beta_1 f_c'}{f_y}\left(\frac{599.843}{599.843 + f_y}\right)$$ $fc' = 30$ MPa β_1 = 0.8324

$\rho_b = 0.029727$

Selected reinforcement = **Ø** 32/20 + **Ø** 26/20

Reinforcement Area As = 6676mm^2

Ace, required = 4981 mm^2

As = 6676 mm^2

As, min = 3866mm^2

ρ = 0.003514 <0.75 ρb = 0.022295 OK ρ 0.75 ρ_b (AASHTO 8.16.3.1.1)

A.9 Foundation Bridge Longitudinal Reinforcement Calculation

Mu (kNm)	Φ (ASSHTO)	fc (MPa)	FY (MPa)	d (mm)	d' (mm)	bw (mm)	d' (mm)
200.00	0.9	30	420	2000	1950	1000	50

$\emptyset M_n = \emptyset\left[A_s f_y\left(d' - \frac{a}{2}\right)\right]$ (AASHTO 8.16.3.2) Flexion bearing capacity

$\alpha = \frac{A_s f_y}{0.85 f_c' b_w}$ (AASHTO 8.16.3.2) Equivalent pressure block depth

$m = \frac{f_y}{0.85 f_c'}$ $R_n = \frac{M_u}{\phi b\, d^2}$ $\rho = \frac{1}{m}\left[1 - (1 - \frac{2mR_n}{f_y})'\right]$

m = 16.47 R_n = 0.06 N/mm^2 ρ = 0.00014

reinforcement amount required for Mu = 200 KN $A_{s,gerekli} = 272\ mm^2$

As = Quantity of equipment

Minimum reinforcement area

(AASHTO 8.17.1.1)

fr = 3.41 Mpa Breaking module

Ig = 6.67E +11 mm 4 Moment of inertia relative to the geometric center of the section

yt = 1000 mm Distance of the center of gravity of the section to the bottom

Mcr = 2274.87 kNm Cracking moment $M_{cr} = f_r \frac{I_g}{y_t}$ (AASHTO 8.13.3)

1.2 Mcr = 2729.85 kNm $\longrightarrow$ As, min = 3763 mm2

Minimum reinforcement requirement is not required if the reinforcement area at the section is greater 1.33 times than the required reinforcement area at the end of the analysis (AASHTO 8.17.1.2) (AASHTO 8.17.1.2)

1.33 As, required = 361.6 mm^2 Ac, min = min (3761, 361) = 361 mm^2

Balanced equipment ratio

$\rho_b = \frac{0.85\beta_1 f_c'}{f_y}\left(\frac{599.843}{599.843 + f_y}\right)$ $fc' = 30$ MPa β_1 = 0.8324

$\rho_b = 0.029727$

Selected reinforcement = **Ø** 26/20 + **Ø** 0/20

Reinforcement Area As = 2655mm^2

Ace, required = 272 mm^2

As = 2655 mm^2

Ace, min = 361mm^2

ρ = 0.003514 <0.75 ρb = 0.022295 OK ρ 0.75 ρ_b (AASHTO 8.16.3.1.1)

A.10 Foundation Bridge Longitudinal Reinforcement Calculation

Mu (kNm)	**Φ (ASSHTO)**	**fc (MPa)**	**FY (MPa)**	**d (mm)**	**d' (mm)**	**bw (mm)**	**d' (mm)**
200.00	0.9	30	420	2000	1950	1000	50

$\emptyset M_n = \emptyset\left[A_s f_y\left(d' - \frac{a}{2}\right)\right]$ (AASHTO 8.16.3.2) Flexion bearing capacity

$\alpha = \frac{A_s f_y}{0.85 f_c' b_w}$ (AASHTO 8.16.3.2) Equivalent pressure block depth

$m = \frac{f_y}{0.85 f_c'}$ $R_n = \frac{M_u}{\phi b\, d^2}$ $\rho = \frac{1}{m}\left[1 - (1 - \frac{2mR_n}{f_y})'\right]$

m = 16.47 $R_n = 0.06\ N/mm^2$ ρ = 0.00014

reinforcement amount required for Mu = 200 KN $A_{s,gerekli} = 272\ mm^2$

Minimum reinforcement area

(AASHTO 8.17.1.1)

fr = 3.41 Mpa Breaking module

Ig = 6.67E +11 mm 4 Moment of inertia relative to the geometric center of the section

yt = 1000 mm Distance of the center of gravity of the section to the bottom

As = Quantity of equipment

Mcr = 2274.87 kNm Cracking moment $M_{cr} = f_r \frac{I_g}{y_t}$ (AASHTO 8.13.3)

1.2 Mcr = 2729.85 kNm $\longrightarrow$ As, min = 3763 mm^2

Minimum reinforcement requirement is not required if the reinforcement area at the section is greater 1.33 times than the required reinforcement area at the end of the analysis (AASHTO 8.17.1.2) (AASHTO 8.17.1.2)

1.33 As, required = 361.6 mm^2 Ac, min = min (3761, 361) = 361 mm^2

Balanced equipment ratio

$\rho_b = \frac{0.85\beta_1 f_c'}{f_y}\left(\frac{599.843}{599.843 + f_y}\right)$ $fc' = 30$ MPa β_1 = 0.8324

$\rho_b = 0.029727$

Selected reinforcement = **Ø** 26/20 + **Ø** 0/20

Reinforcement Area As = 2655mm^2

Ace, required = 272 mm^2

As = 2655 mm^2

Ace, min = 361mm^2

ρ = 0.003514 <0.75 ρb = 0.022295 OK ρ 0.75 ρ_b (AASHTO 8.16.3.1.1)

A.11 SAP 2000 Data Entry for the Posttensioning System

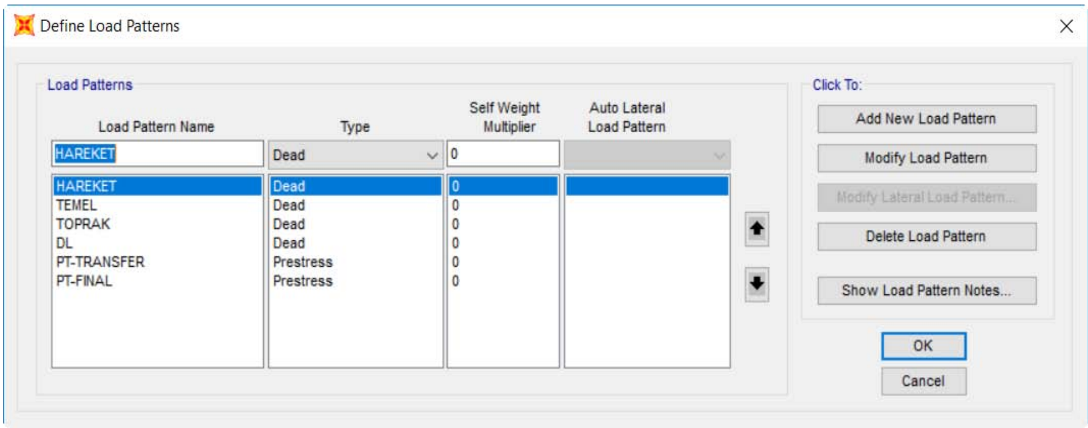

Figure A.6 Define load patterns

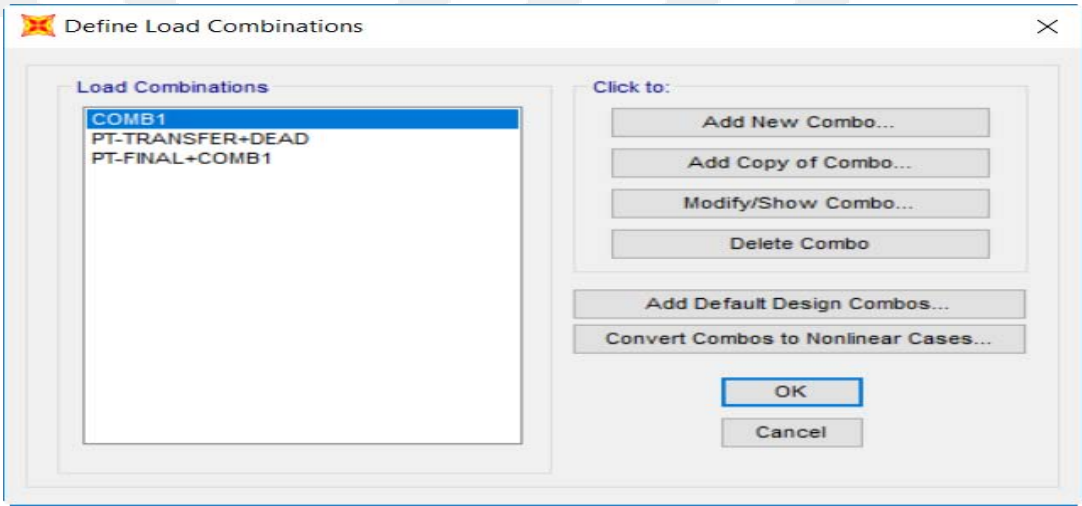

Figure A.7 Define load combinations

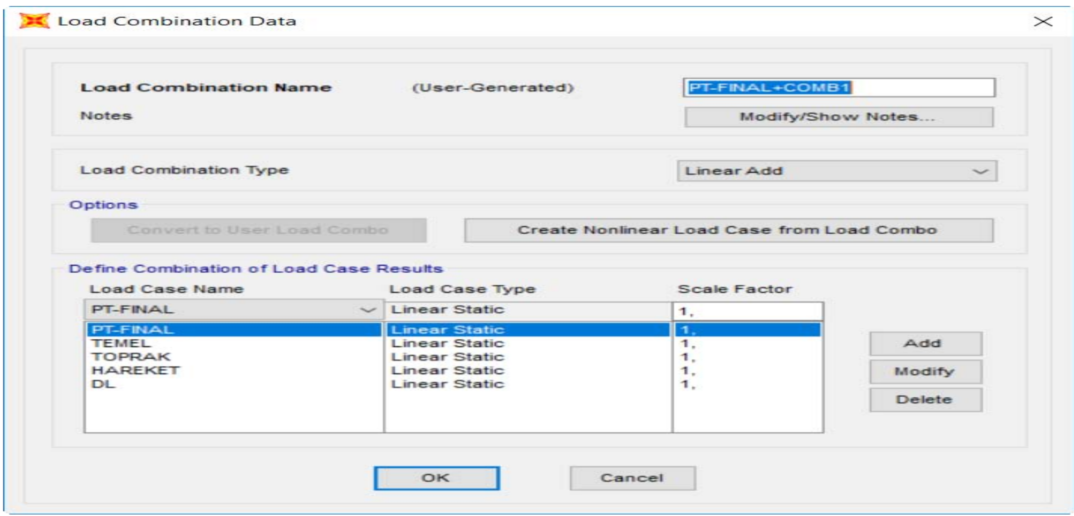

Figure A.8 Load combination data pt-final+comb1

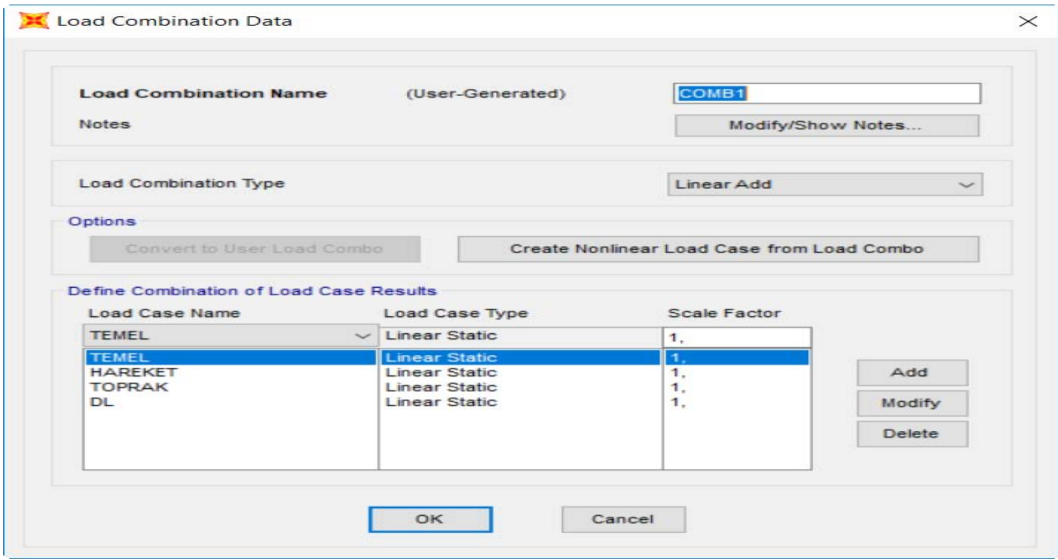

Figure A.9 Load combination data comb1

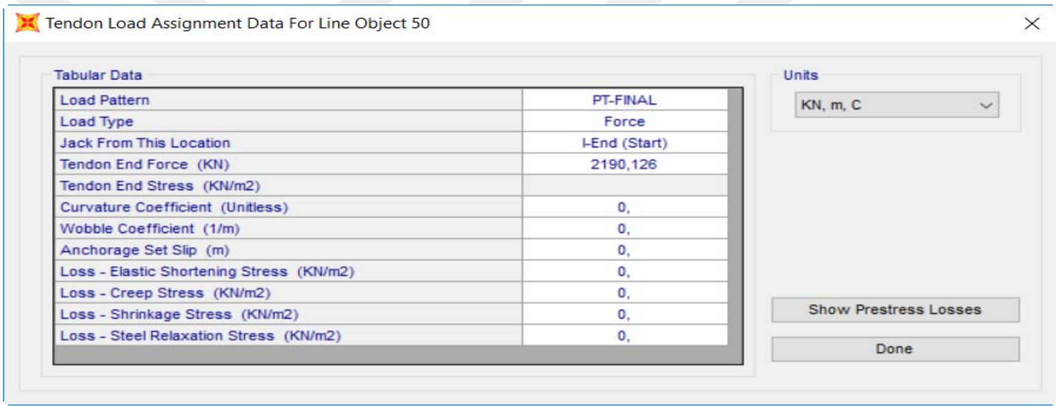

Figure A.10 Tendon load assignment data

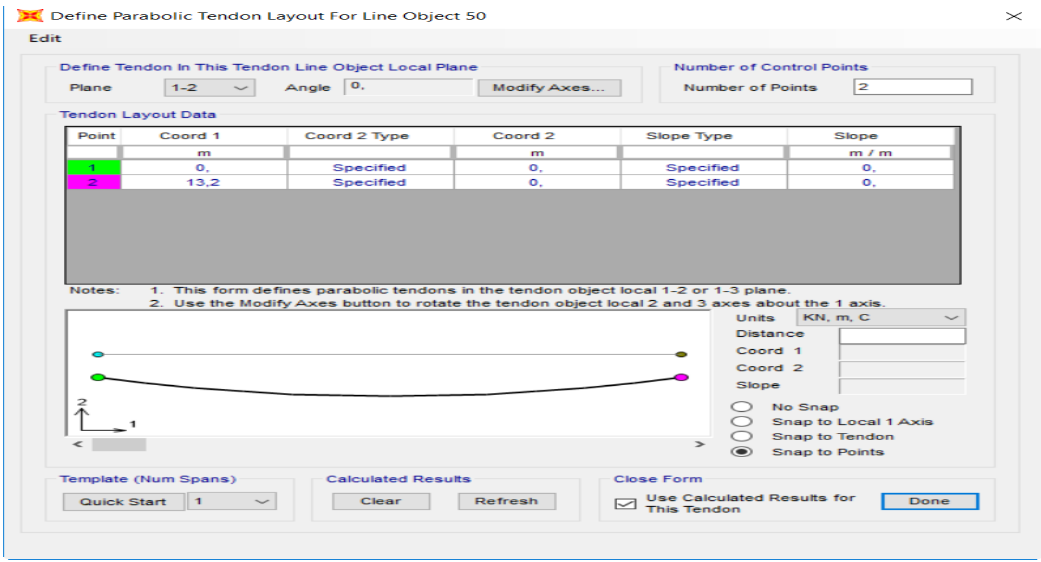

Figure A.11 Define parabolic tendon layout for line object