

USE OF GLASS FIBER REINFORCED POLYMER (GFRP) FOR REINFORCING
IN-SITU SOILS

by

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“In Antoine de Saint Exupéry’s tale the Little Prince meets a businessman who owns stars with the sole purpose of being able to buy more stars. Yet he cannot remember what they are called and contributes nothing to them. The Little Prince is confused. He owns only a flower, which he waters every day and three volcanoes, which he cleans every week. “It is of some use to my volcanoes, and it is of some use to my flower, that I own them”, he says, “but you are of no use to the stars that you own””. It was indeed true; businessman got no use of them.

In a global world of businessmen, we -the ordinary people- should remember that the whole knowledge of mankind on anything is in existence, since the collective act of knowledge sharing existed once upon a time on Earth. We are not those businessmen who are producing knowledge to buy more. Knowledge which is produced on earth since the beginning of history, is the sole property of our world. And this property was produced by the people of the world for humanity. Therefore as a starting point;

I want to thank to all who produced and shared the knowledge in the past, are producing and sharing at the moment or will produce and share in the future.

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ABSTRACT

USE OF GLASS FIBER REINFORCED POLYMER (GFRP) FOR REINFORCING IN-SITU SOILS

Ground anchoring is a technique in which slender pre-stressed components are embedded into the in-situ soil or rock to counterbalance the uplift or tensile forces. It is a practical technique which have been used widely since the end of 18th century. However, corrosion issue is a great restriction for the use of ground anchors in corrosive environments for long term conditions. Nowadays, a newly emerged material called Glass fiber reinforced polymer is a promising alternative for this problem due to its high corrosion resistance, high specific strength and light weight. This present master's thesis aim is to compare the behaviour of GFRP materials over Steel ones in grouted ground anchors in several variable conditions to understand the feasibility of the use of Glass fiber reinforced polymer for reinforcing in-situ soils. In total of 18 pull-out numerical simulations are conducted by using Plaxis 2D v.2016.01. Pull out of a ground anchor with three distinctive fixed lengths (6m, 8m and 12m) are analyzed. In the FE models, ground anchors are embedded in three different geo-materials (over consolidates clay, dense sand and rock). As tendon material GFRP and Steel was used separately. All combinations of soil, tendon type and length have been analyzed and results are obtained for both traditional steel and GFRP tendon. Under high loads, Ground anchors with GFRP tendons embedded in highly stiff rock, presented increased deflections comparing with the steel. However, this situation becomes insignificant while using allowable load capacities (displacement control) with universally accepted factor of safety values.

ÖZET

CAM ELYAF TAKVİYELİ POLİMER (CETP) MALZEMELERİN YERİNDE ZEMİN TAKVİYESİNDE KULLANIMI

Zemin ankrajları, inşaat yapılarında oluşan çekme kuvvetlerini karşılamak amacıyla zeminde veya kayada açılan bir kuyu içerisine imal edilen ince/narin ön germeli elemanlardır. Ankrajlama tekniği, çok pratik olması dolayısıyla 18. yy'dan beri yaygın biçimde kullanılmaktadır. Fakat ankrajların uzun dönemli kullanımında korozyon sorunu büyük bir problem yaratmaktadır. Son zamanlarda, yeni ortaya çıkmış bir kompozit malzeme olan cam elyaf takviyeli plastik, yüksek korozyon direnci, yüksek mukavemeti ve düşük birim hacim ağırlığı sayesinde bu problemin çözümünde önemli bir alternatif oluşturmaya başlamıştır. CETP ve çelik malzemenin değişken koşullarda davranışlarını karşılaştırarak incelemek ve CETP malzemenin zemin güçlendirilmesinde kullanılabilirliğini araştırmaktır. Toplamda 18 sıyırılma deneyi, Plaxis 2D v.2016.01 programı kullanılarak modellenmiştir. Zemin ankrajının sıyırılma deneyi üç farklı kök boyunda (6m, 8m ve 12m) ve üç ayrı zemin birimine (aşırı konsolide kil, sıkı kum ve kaya) gömülü olarak analiz edilmiştir. Bütün modeller için cam elyafla takviyeli plastik ve çelik ayrı olarak kullanılmıştır. Bütün zemin, kök boyu ve tendon tipi kombinasyonları denenmiş ve sonuçlar çelik CETP için ayrı olarak elde edilmiştir. Sonuçlar gözönüne alınarak, kök boylarının kısa olduğu zemin birimlerde cam elyafla güçlendirilmiş plastik malzemenin kullanımı kolay ve uygun görünmektedir. Fakat yüksek yükler altında, yüksek rijitlikteki kaya malzemeye gömülü CETP tendonlu ankrajlar, çelik tendonlu ankrajlarla karşılaştırıldığında yüksek deformasyonlar göstermektedir. Fakat bu durum dünyaca kabul edilmiş güvenlik faktörleri kullanılarak elde edilen izin verilebilir taşıma gücü kapasiteleri kullanıldığında önemsiz hale gelmektedir. CETP malzemelerdeki ve ankraj kafası tasarımındaki sünme ve kopma etkisinin ileri tetkiklerde incelenmesidir.

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LIST OF SYMBOLS

c_u	Average undrained strength (kN)
d	Diameter of the borehole (grouted) (m)
E_f	Modulus of fibers (kN/m ²)
E_m	Longitudinal modulus of the composite (in the direction of fibers)
E_m	Modulus of the matrix (kN/m ²)
E_{50}^{ref}	Plastic straining due to primary deviatoric loading (kN/m ²)
E_{oed}^{ref}	Plastic straining due to primary compression (kN/m ²)
E_{ur}^{ref}	Elastic unloading and reloading (kN/m ²)
f_{eff}	Efficiency factor which is a function of the fixed length (dimensionless)
f_{pu}	the ultimate tensile strength (kN/m ³)
n	a capacity factor which is determined by taking into account of the grouting pressure (3-10 bar)
L	Fixed length of the anchor (m)
$[K]$	is the property matrix (dimensionless)
K_o	Coefficient of Earth Pressure at Rest (dimensionless)
R	is the vector of unknown variables (dimensionless)
r	is the nodal vector (dimensionless)
P_f	Density of fibre material (kN/m ³)
p_m	Density of matrix material (kN/m ³)
p_c	Density of composite (kN/m ³)
t_{ayg}	in the equation represents the average bond stress value along the fixed anchor length (kN/m ²)
T_G	is the tensile stress (kN/m ²)
TS_c	is the ultimate strength of the composite (kN)
TS_f	is the ultimate tensile strength of the fibers (kN)
TS_m	is the ultimate tensile strength of the resin (kN)
T_{ult}	Ultimate capacity of the ground anchor (kN)

t_{ult}	Ultimate bond stress/Factor of cu or SPT-N (kN/m ²)
V_f	is the volume percentage of fibers %
V_m	Volume fraction of matrix (%)
α	adhesion factor (dimensionless)
$\emptyset$	Peak friction angle of soil (°)

LIST OF ACRONYMS/ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
ACI	American Concrete Institute
ASTM	American Society of Testing Materials
BS	British Standards
FEM	Finite Element Method
FHWA	Federal Highway Administration
FRP	Fiber Reinforced Polymer
GRP	Glass Reinforced Polymer
GRRP	Glass Fiber Reinforced Polymer
ICE	Institution of Civil Engineers
ISO	International Organization of Standardization
SPT	Standard Penetration Test

1. INTRODUCTION

1.1. Background

Anchoring is a technique in which slender pre-stressed components are embedded into the in-situ soil or rock. It is quite antique technique because mankind have always used ground to support structures, or used some structures to support ground. In either of ways, it is practical to use ground anchors. The first possible use of this method was probably on stabilizing tents. Nowadays, they are crucial parts for many ordinary or conventional structures like deep excavations, cable stayed bridges, maritime structures, etc.

Ground anchorages are principally used to counterbalance the uplift or tensile force developed from generally the weight of the superstructure or the ground. They are also used to resist the “tension pressures” which is developed from the overturning moments, seismic activity and/or hydrostatic uplift pressures. For those applications generally grouted ground anchors with variable diameters are used. They resists the uplift forces by transferring those to the soil and/or rock by developing friction forces along the shear zone between grout/ground.

Grouted ground anchor’s tendon material is by a majority; conventional steel up till now. However those materials have their disadvantages in harmful environmental conditions such as waterfront areas. Traditional engineering materials like steel, concrete and timber have great disadvantages like very low resistance to corrosion, degradation and marine borer attacks which generates great reduction in service life and in section area for design purposes, and also creates further problems in the integration with the superstructure.

As grouted ground anchorages are being used in both harmful environments (corrosive areas) as permanent structure and in offshore engineering area to resist the uplift forces causing by the buoyancy effects, a need for an alternate material come forward

to resolve the high repair or replacement costs of those long established-traditional materials. Those newly emerging materials are called as engineered composites.

Engineered composites are the materials which are the combination of two or more materials by keeping their individual characteristics. The resulting properties of composite materials are normally the production of the combination of the properties of individual elements. Typical engineered composite materials are ceramic composites, reinforced plastics (fiber reinforced polymers, mortars and metal composites. Although, all of those engineered composites have their better or worse properties when comparing them with each other and traditional materials, the most popular and widely used one in civil engineering area is Fiber reinforced polymers. There are usually three types of FRPs; Glass fiber reinforced polymers, aramid fiber reinforced polymers and carbon fiber reinforced polymers, The focus of this present thesis will be on Glass Fiber Reinforced Polymers-Glass Reinforced Plastics also known as GFRPs or GRPs since they are by far most popular composite in civil engineering more particularly geotechnical study area.

GFRPs or GRPs are newly emerged materials with several important benefits versus traditional steel material such as; resistance to corrosion and acid, extended service life and no maintenance requirements for long term conditions. On the other hand, they have high cost when comparing with structural steel but this cost gap is getting narrower over time and will disappear eventually. Concerning those advantages, use of GFRP/GRPs are appears far more feasible in permanent geotechnical ground support structures, in maritime projects and/or in corrosive (harmful) environments even with the current cost gap.

The aim of this present master's thesis is to investigate the use of Glass fiber reinforced polymers to reinforce in situ soils by comparing the behaviour of GFRP bar over the steel bar in ground support area that is more particularly; in grouted ground anchorages. This comparison will carried on by performing number of numerical simulations by using finite element method.

Finite element method analyses are utilized by using Plaxis 2D v. 2016.01 with Axisymmetric Analysis Method. Analyses are conducted by modelling a single grouted ground anchor with 35cm in diameter and Ø40 (steel/GFRP) bar in center by modifying the fixed length as 6m, 8m, 12m in Over consolidated clay, dense sand and Rock conditions respectively. The pull-out effect is carried out by implementing several phases of 0.4cm prescribed displacements to the top of the tendon/bar of the modelled ground anchor to create uplift effect until the failure occurs.

Eighteen numerical simulation models are conducted and analyzed to observe the difference of the behaviour of GFRP and Steel grouted ground anchors and conclusions are derived mostly from those analyses and comparisons of them.

1.2. Scope of the Study

The scope of this present thesis is to compare the behaviour of GFRP materials over Steel ones in grouted ground anchors in several variable conditions and analyzing the results by taking account of the sole advantages of GFRP and conventional Steel. There several important targets of this study such as;

- (i) Investigate the effect of fixed length on ultimate load when comparing the GFRP reinforced ground anchors and steel reinforced ones,
- (ii) Study the effect of soil type on behaviour of grouted ground anchors when comparing the GFRP reinforced ground anchors and steel reinforced ones,
- (iii) Observe the effect of length and soil type on pull out failure behaviour in the shear zone of ground/grout interface of GFRP and Steel ground anchorages by comparing the failure mechanism of those.

1.3. Organization of the Thesis

The organization of this thesis advances in a style which will allow the reader to grasp the initial concepts; such as material types and benefits, later it will continue with the literature review about the grouted ground anchors and finite element modelling

later on calculation models and assumptions will be described before analysis results and discussions are presented. The contents of the thesis are divided as seven chapters, those chapters and their objectives can be seen as follows:

Chapter 1 briefly introduces the concept of ground anchorages and typical construction materials, also it presents the aim and the structure of this study.

Chapter 2 provides information about the tendon material FRP; especially GFRP, procedure of production, benefits and drawbacks and the reason that drive the researcher to introduce FRP as a feasible alternative for ground anchorage. This chapter also briefly presents the universal codes and standards of GFRP/GRP materials. Moreover, this section will cover the commercially available GFRP bars, anchors and rods.

Chapter 3 offers a review of ground anchorages as a ground support method. It will also present fields of applications, classifications, parts and assemblies and will finally provide informations about the design and safety factors.

Chapter 4 introduces Finite element modeling in geotechnics by providing information about finite element modelling software Plaxis 2D v 2016.01. This chapter also provides information about modelling phases in Plaxis 2Dv.2016.01 which is associated to simulate tension pile pull-out model.

Chapter 5 describes the physical conditions in ground anchorage pull-out model such as model geometries and simplifications, boundary conditions, and material parameters. Later, this section will provide the detailed information about modelling steps of anchorage pull-out model regardless of rebar material type (GFRP or Steel) in Plaxis 2D v.2016.01.

Chapter 6 presents the results of ground anchorage pull-out models including graphics to cover all the variables such as the effects of length, and surrounding soil type to the pull-out capacity of ground anchorages with GFRP bars and Steel bars.

Chapter 7 shows the comparison of the results in graphic format and presents the conclusion which have been drawn from the results of numerical models.

2. THE MATERIAL: FIBER REINFORCED POLYMER

2.1. Introduction

This chapter will provide the overall literature review which has been done for the understanding of the history and evolution, mechanical properties and area of utilization of GFRP bars.

2.1.1. What is FRP

Materials are selected for application by taking account of their strength properties, as most of the engineering structural systems need to carry heavy loads. However, by the advances in technology, unit weight of the material as a property becomes important for engineering materials. For instance, when we are taking account of a material which will be used to build a plane, it needs to have a good tensile strength but also needs to be light in weight for plane to fly and engineering composites are the most viable material by taking the unit weight to tensile strength ratio to account which can be seen in the Figure 2.1.

Fiber Reinforced Polymers are a subgroup of materials which are known as engineered composites (Bisby *et al.*, 2006). They are composed of fibers as a reinforcement polymer matrix or resin as a binding material. They are brand-new materials in which their area of use is expanding by the expanding developments in science and engineering. FRP is composed of two essential materials; Fiber-reinforced polymer (FRP) composite materials are comprised of fibers and resins, which are bonded through the connection zone to make sure that the composite system as a whole gives convenient performance (Bai, 2013). Fibers are tensile strong reinforcements and matrices are compression strong materials in this composition.

peak, they start to reach their theoretical value (Hollaway, 1993). Thus, the main role of fibers is providing strength and stiffness to the composites. According to Hollaway (1993) Useful properties for a fiber in fiber reinforced composites are;

- The fiber ought to have a high modulus of elasticity for an efficient utilization of reinforcement.
- The fiber ought to have a high ultimate strength.
- Strength difference between each other fibers better be low.
- The fibers ought to be stable and retain their strength during handling and fabrication.
- The diameter and surface of the fibers better be uniform.

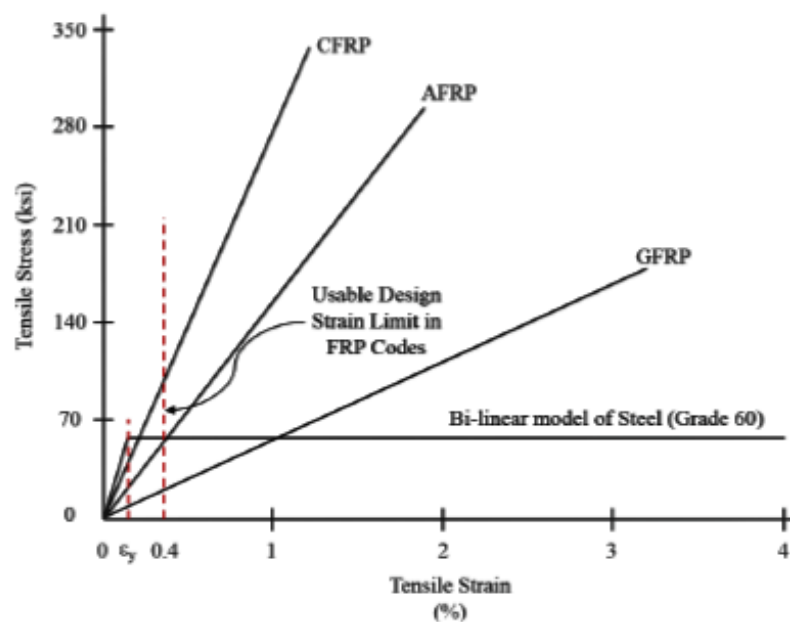


Figure 2.2. Properties of different fibers and typical reinforcing steel (ACI 440, 2006) (Connor, 2013).

2.1.3. Matrix

Matrix is a binding material, commonly in the form of liquid plastic, and its aim is to transfer forces between fibers and to protect them from the harmful environmental effects. Functions of matrices can be seen below (Hollaway, 1993);

- to bind together the fibers and protect their surfaces from damage during handling, fabrication and during the service life of the composite;
- to disperse the fibers and separate them in order to avoid any catastrophic propagation of crack and subsequent failure of the composite;
- to transfer the mobilized stresses to each fiber efficiently by adhesion and/or friction ;
- to be chemically compatible with fibers over a long period;
- to be thermally compatible with fibers.

There are various composite material possibilities for the use of both parts. The mechanical attributions of FRP composites may change according to the proportion of fiber and matrix material, the mechanical properties of the component materials, the fiber orientation in the matrix, and finally the procedures of the manufacturing process of the material. (Bai, 2013). An illustration for fiber and matrix composite may be seen in Figure 2.3. Next to that composites are also composed of fillers and additives and its main purpose is to provide a protective surface for the composite towards harmful environmental effects like corrosion, acid and hazardous agents.

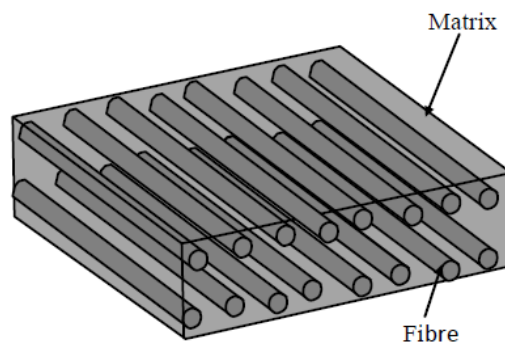


Figure 2.3. Properties of different fibers and typical reinforcing steel (Obaidat, 2010).

2.1.4. Manufacturing Process of FRP Materials

Polymer materials must be reinforced with fibers if they are to be used structurally and in doing so a wide range of composites with varying mechanical properties may be obtained by changing these:

- the relative proportions of matrix and fibers;
- the type of fiber and matrix;
- the orientation of the fiber within the resin matrix;
- the method of production of the composite.(Hollaway, 1993).

There are 5 types of frequently adopted manufacturing processes for FRP composite materials which can be seen below;

- Hand lay-up,
- Prepreg lay-up,
- Bag molding,
- Resin Transfer Molding,
- Pultrusion,

As those manufacturing process types are not within the scope of this thesis, they are not going to be reviewed in detail. However, to have a better understanding about FRP materials, one should conduct further investigations about those manufacturing processes.

2.2. History

There can be no doubt that the last two centuries, beginning with the industrial revolution, are abound with the most remarkable technological advances and scientific discoveries in human history. Humankind discovered the electricity, the evolution, the splitting of the uranium atoms, theory of relativity, quantum theory and DNA. Those discoveries are called as the greatest discoveries of humankind because they restructure the way of thing. Those revolutionary discoveries lead the world to other discoveries and developments in any other scientific study field. One of the most important of them is composite materials. Even though there are several types of composites in material sciences, the focus of this present thesis will be only on fiber reinforced polymers known as FRP's.

Discovery of FRPs has led to the great leap forward for material science and engineering. Over the last century, a huge industry has been created which symbolizes the 20th century just as steel characterized the 19th century (Hollaway, 1993). Bakelite was the first reinforced plastic material which has been used for its electrical insulation and heat resistant features. It was first produced in 1907.

“The earliest FRP materials used glass fibers embedded in polymeric resins that were made available by the burgeoning petrochemical industry following World War II. The combination of high-strength, high-stiffness structural fibers with low-cost, lightweight, environmentally resistant polymers resulted in composite materials with mechanical properties and durability better than either of the constituents alone” (Bakis *et al.*, 2002).

By growing technology and needs, FRPs started to be used respectively in Aerospace and Mechanical engineering areas for its light weight and high tension capacities. For civil engineering area to follow other engineering disciplines on FRP composites took a pretty much long time thus it relied on traditional materials like steel, concrete and timber until the beginning of eighties.

Nowadays, FRP composite materials are considered as a promising alternative material for the civil engineering industry, too. They are characterized by low specific weight, higher strength and electromagnetic transparency. “Engineered composites are becoming all-purpose and smart materials for structural and infrastructural-geotechnical engineering over several decades” (Bai, 2013).

2.3. Types of Fibers as a Reinforcement in FRP Composites

Fibers are the keystone of the composite materials. They produce the structural properties like stiffness and strength in composites. Therefore, the type of fibers which is used in the composites is the essential classification method for fiber reinforced composites. There are three dominating fiber types for civil engineering industrial area; Glass, Carbon and Aramid.

Table 2.1. Typical properties of widely used fibers (Hollaway, 1993).

Material	Fibre	Elastic modulus (Gpa)	Tensile strength (Mpa)	Ultimate strain(%)	Density (g/cm ²)	Max temp(°C)
Glass	E	72.4	2400	3.5	2.55	250
	A	72.4	3030	3.5	2.50	250
	S-2	88.0	4600	5.7	2.47	250
Aramid	49	125	2760	2.4	1.44	180
	29	83	2750	4.0	1.44	180
Tejin	Technora	70	3000	4.4	1.39	-
Carbon	XAS HS	235	3800	1.64	1.79	600
Grafil	HS	235	3450	1.64	1.79	600
	IM-S	290	3100	1.64	1.76	600
	HM-S6K	370	2750	1.64	1.86	600
Toray	T-300	230	3530	1.5	1.77	600
	M-46-J	450	4100	2.4	1.82	600

2.3.1. Carbon Fibers

Carbon fibers are long filaments mostly composed of carbon atoms. They are manufactured by pyrolysis and crystallization of organic precursors at temperatures above 2000°C. In this process, carbon crystallites are produced and orientated along the fiber length (Potyrala, 2011). They can be manufactured in variable forms like chopped fibers filament tows or mats.

Carbon fibers have high stiffness, high tensile strength, low unit weight, corrosion resistance, fatigue resistance and good friction. Since, these properties are the essential properties for civil engineering area, carbon fibers are expected to be most widely used composite among any other fiber types (being used) in civil engineering industry. However, they are not the most popular composite in civil engineering area because there is a need of high energy and ample time for their production thus they are very expensive.



Figure 2.4. Figure Carbon Fiber Fabric (Carbon, 2017).



Figure 2.5. Figure Carbon Fiber Tow (Carbon, 2017).

2.3.2. Aramid Fibers

Aramid fibers are heat-resistant, high performance- high strength fibers composed of aromatic polyamide fibers. Because of those properties, aramid fibers are mostly used in military industry like missiles. Composites which have reinforced with aramid fibers have a low density, high strength, relatively high cyclic durability properties and good fire resistance, however it also has a low compression strength and high sensitivity to environmental factors (Bai, 2013). Another drawback of aramid fibers is that it is difficult to cut them.



Figure 2.6. Aramid Fiber Cut.

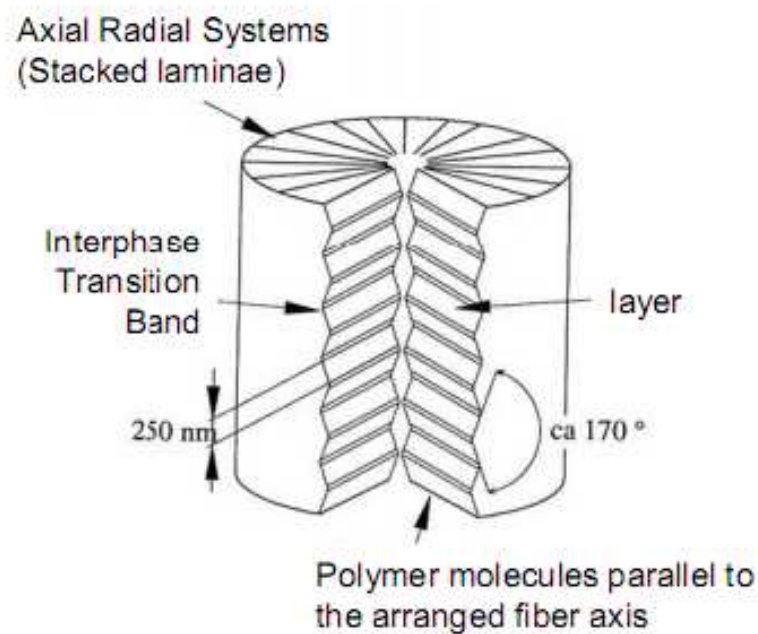


Figure 2.7. Single Aramid Fiber (Zhou and Lesko, 2006).

2.3.3. Glass Fibers

Glass fiber is a widely used inorganic fiber material which is composed of numerous number of oxides with other raw materials. The physical and chemical specialties of the glass fiber are influenced to a considerable extent by the composition of its components, the diameter of the elemental fiber, and the surface maintenance implemented (Bai, 2013). There are several types of glass fibers which are used as a reinforcement, such as; chopped fibers, chopped strands, chopped strand mats, woven fabrics, and surface tissue.

Glass fibers are the most popular fiber type as a reinforcement. More than 90% of all FRP composites which are used on earth are made of glass fiber, since glass fibers are the most low cost - high-performance fibers. Besides, they are to be used as electrical and thermal insulators. However, they have a low Young's Modulus and a relatively bad ductile performance, their behaviour is very brittle.

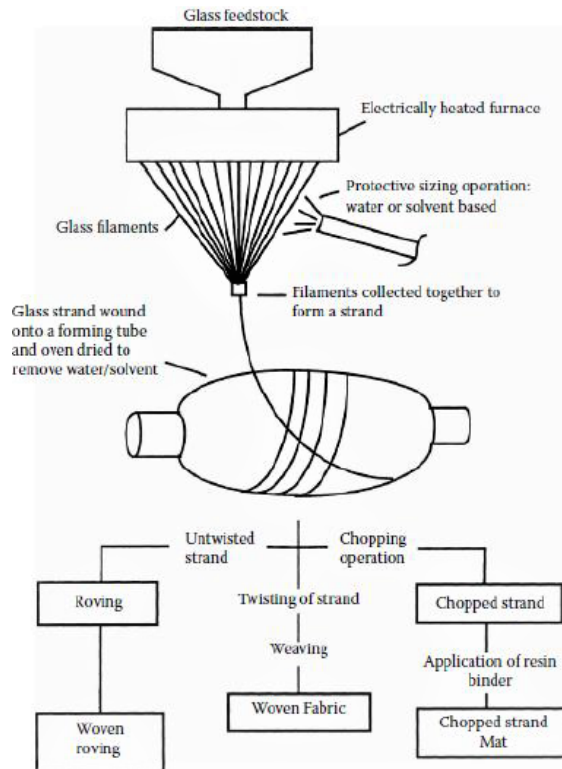


Figure 2.8. Fabrication process of Glass Fibers (Fireberg and Olsson, 2014).

Table 2.2. Approximate Properties of Common Glass Fibers Types (Bank, 2006).

Grade of Glass Fiber	Density [g/cm ³ (lb/in ³)	Tensile Modulus [Gpa(Msi)]	Tensile Strength [Mpa(ksi)]	Max. Elongation (%)
E	2.57 (0.093)	72.5 (10.5)	3400 (493)	2.5
A	2.46 (0.089)	73 (10.6)	2760 (400)	2.5
C	2.46 (0.089)	74 (10.7)	2350 (340)	2.5
S	2.47 (0.089)	88 (12.8)	4600 (667)	3.0



Figure 2.9. Chopped Glass Fiber (Johns, 2017).

2.4. FRP Bars

2.4.1. Introduction

FRP bars are reinforcing bars which is made from a combination of filaments or fibers and resin. “Current FRP reinforcing bars (referred to as FRP rebars in what follows) and grids are produced commercially using thermosetting polymer resins (commonly vinylester and epoxy) and glass, carbon, or aramid reinforcing fibers. The bars are primarily longitudinally reinforced with volume fractions of fibers in the range 50 to 65%” (Bank, 2006). The process can be seen below in Figure 2.10.

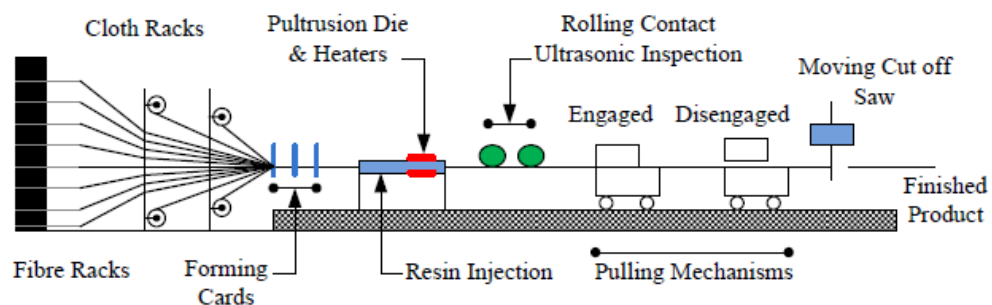


Figure 2.10. Pultrusion manufacturing process (Mahroug, 2013).

The FRP Bar/Rod/Tendon can be produced from various kinds of fibers such as; Glass (GFRP) or Carbon (CFRP) or Aramid (AFRP). In harmful environmental

circumstances steel has a limited resistance because of the influences of corrosion and they have been used in civil engineering area for more than 30 years. On the other hand, FRP Bars are recently intended to be used as a reinforcement instead of steel. (Bank, 2006). Some examples of FRP bars can be seen below in Figure 2.11.



Figure 2.11. Commercially available GFRP Bars (ACI 440, 2006).

2.4.2. Physical and Mechanical Properties

Fundamentally, the mechanical and physical properties of FRP reinforced composites depending on the properties, orientation and volume ratio of its individual constituents. Important mechanical properties of FRP composites are its ultimate tensile and compressive strengths, and its stiffness properties. Also, the quality of the bonding between the fiber and the matrix has direct effect on the mechanical properties of the FRP composites.

There are three groups of fiber orientation; unidirectional, bidirectional and randomly oriented (multidirectional). Mainly, direction of the fibers determines the strength and stiffness properties of the resulting composites. The FRP bars used in practice have commonly unidirectional fiber orientation. Therefore, fibers have a high strength and stiffness in longitudinal direction but very low strength and stiffness in transverse direction. As can be seen in the Table 2.3. The product specialties, such

as tensile strength and stiffness are affected by the orientation and the fiber volume fraction (Clarke, 1996).

Table 2.3. The influence of orientation and the volume fraction of the fiber on the product (Clarke, 1996).

Fibre type	Reinforcement directionality factor	Typical volume fraction V_f	Product
Unidirectional	1	65%	0.65
Bidirectional non-crimp	0.6	50%	0.3
Bidirectional woven	0.5	40%	0.2
Random in plane	0.375	20%	0.075

The general, strength of a composite is mainly influenced by the following factors:

- (i) type of fiber and matrix,
- (ii) orientation of fibers,
- (iii) volume fraction of fiber,
- (iv) curing rate and quality control (QC) during manufacture (O'Connor, 2009) (Friberg and Olsson, 2014).

Some of the basic mechanical properties of the FRP composites are briefly presented below.

2.4.2.1. Density. One of the most important properties of FRP composite is their density. The density of the FRP composite ranges from 0.9g/cm^3 to 2.3g/cm^3 , but it generally in the range of 1.2 g/cm^3 and 1.8 g/cm^3 (Potyrala 2011).

Table 2.4. Typical values of density for carbon, glass and aramid fibers (ACI 440, 2006).

Stell	GFRP	CFRP	AFRP
493.00	77.8 to 131.00	93.3 to 100.00	77.80 to 88.10
(7.90)	(1.25 to 2.10)	(1.50 to 1.60)	(1.25 to 1.40)

It may be calculated by the Equation 2.1 (Potyrala, 2011).

$$p_c = p_m x V_m + p_f x V_f \quad (2.1)$$

where is the p_c density of composite, is the p_m density of matrix material, is the V_m volume fraction of matrix, is the P_f density of fibre material, is the V_f volume fraction of fibers

The low density of FRP composites led to some other benefits such as the ease of transportation and handling, lowering the total weight of the structure and high specific modulus and strength.

2.4.2.2. Tensile Strength. FRP bars generally have a higher tensile strength than the steel reinforcing bars. However, they do not yield before rupture, so the ultimate strength and breaking strength are the same. Typical tensile strength and stiffness properties of FRP composite bars as compared to the conventional steel bars can be seen below in Table 2.5.

Table 2.5. Usual Tensile Properties of Reinforcing Bars (ACI 440, 2006).

	Steel	GFRP	CFRP	AFRP
Nominal yield stress, ksi (MPa)	40 to 75 (276 to 517)	N/A	N/A	N/A
Tensile strength, ksi (MPa)	70 to 100 (483 to 690)	70 to 230 (483 to 1600)	87 to 535 (600 to 3690)	250 to 368 (1720 to 2540)
Elastic modulus, x10³ ksi(GPa)	29.0 (200.0)	5.1 to 7.4 (35.0 to 51.0)	15.9 to 84.0 (120.0 to 580.0)	6.0 to 18.2 (41.0 to 125.0)
Yield strain, %	0.14 to 0.25	N/A	N/A	N/A
Rupture strain, %	6.0 to 12.0	1.2 to 3.1	0.5 to 1.7	1.9 to 4.4
Typical values for fiber volume fractions ranging from 0.5 to 0.7.				

Tensile strength and stiffness of the FRP bar are principally contingent upon tensile strength of the fibers because fibers are the base tensile load carrying constituent of the composite. Thus, the ratio of the fibers to the composite has the biggest impact on the behaviour of the composite under tensile stress. Furthermore, the mechanical

characteristics of the Bar are also affected by the rate of the curing, the fabrication process, and the manufacturing quality control (ACI 440, 2006).

The tensile strength in the longitudinal direction of a FRP composite can be calculated with the Equation 2.2 (Muniz and Bansal 2009, Freiberg and Olsson, 2014)

$$TS_c = TS_f x V_f + TS_m x V_m \quad (2.2)$$

where TS_c is the ultimate strength of the composite, TS_f is the ultimate tensile strength of the fibers, TS_m is the ultimate tensile strength of the resin, V_f is the volume percentage of fibers and, V_m is the volume percentage of the resin.

2.4.2.3. Young's Modulus. Stress-Strain behaviour of the FRP composites are most dominantly effected by the type and orientation of fibers which is the reinforcement of the composite. Table 2.6 lists the Young's modulus values for typical types of FRP composites.

Table 2.6. Typical values of modulus of elasticity for unidirectional FRP composites (Potyrala 2011; Clarke, 1996).

Composite	$E_{longitudinal}$	$E_{transverse}$	G	v
(fibres/resin)	GPa	GPa	GPa	-
carbon/epoxy	181	10.3	7.17	0.3
glass/polyester	54.1	14.05	5.44	0.25
aramid/epoxy	75.86	5.45	2.28	0.34

Carbon/epoxy composite shows the highest stiffness in longitudinal direction however, glass/polyester composite shows better performance in transverse direction. The fiber volume fraction of the fibers also has a great effect of the modulus of elasticity of the product which can be seen in Figure 2.12 (Muniz and Bansal 2009, Freiberg and Olsson, 2014)). For unidirectional reinforced polymers, the longitudinal and transverse

modules can be estimated with Equation 2.3 and Equation 2.4:

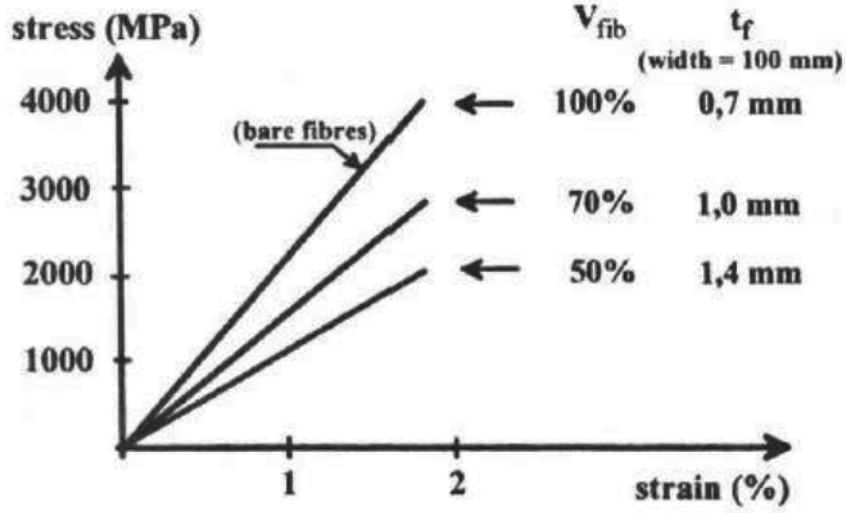


Figure 2.12. Stress-strain relation for different fiber volume fractions (Muniz and Bansal 2009).

$$E_L = E_f x V_f + E_m x V_m \quad (2.3)$$

$$E_L = \frac{E_f x V_m}{E_f x V_f + E_m x V_m} \quad (2.4)$$

where is the E_L Longitudinal modulus of the composite (in the direction of fibers), is the E_f modulus of fibers, V_f volume fraction of fibers, is the E_m modulus of the matrix, is the V_m volume fraction of matrix

2.4.2.4. Poisson's Ratio. The Poisson's ratio is primarily affected by the fiber orientation (Potyrala, 2011; Muniz and Bansal 2009). The relationship between the fiber orientation and the Poisson's ratio can be seen in Figure 2.13.

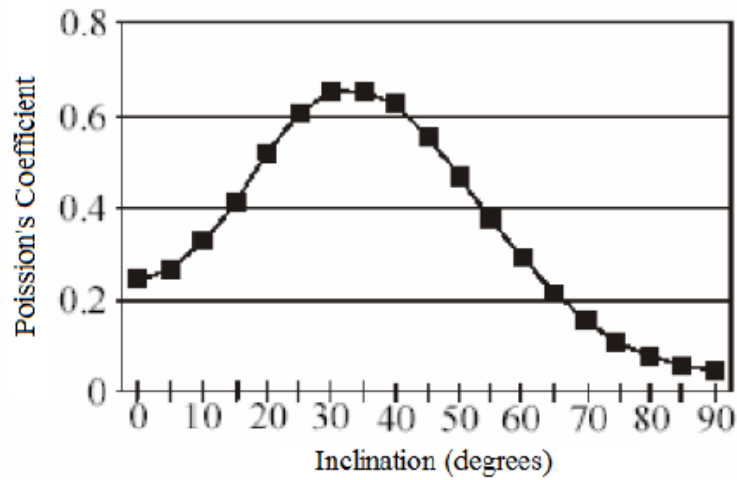


Figure 2.13. The relation between the fiber orientation and the Poisson's ratio for a glass/polyester composite (Muniz and Bansal 2009).

As can be seen from the graph above, by varying orientation in fibers, poison's ratio can fluctuate between nearly 0.7 and close to 0.02.

2.5. Benefits and Drawbacks (of GFRP's)

The advantages provided by fiber reinforced composites differs with the choice of resin, fiber and manufacturing process. Therefore, for the present thesis; the focus will be on GFRP because they are by far the most popular composite type, thus as a normal consequence of this popularity they are the most produced one. By way of summary, the main benefits and drawbacks of glass fiber-reinforced polymer composites are usually one or more of the following:

2.5.1. Benefits

2.5.1.1. High Specific Strength and Resistance. GFRPs have a lower unit weight to a higher tensile strength so the ratio of specific strength is higher, comparing with the traditional materials like structural reinforcing steel. This can be seen from the Table 2.7. The GFRP and A572 Gr. 50 Steel will be used as a GFRP and Steel rebar material in the tension pile pull-out modelling.

Table 2.7. Comparative list of properties of Composites and Traditional reinforcing steel (Maglev, 2017).

Property	GFRP	CFRP AS4/3501-6	A572 Gr. 50 Steel	Concrete
Density [g/cc]	2076	1.580	7.860	2.400
Longitudinal Modulus E_1 [GPa]	45	142	200	32
Transverse Modulus E_2 [GPa]	12	10.3	200	32
In-plane Shear Modulus G_{12} [GPa]	5.5	7.2	75	13
Poisson's Ratio ν_{12}	0.19	0.27	0.33	0.19
Longitudinal Tensile Strength F_{1t} [MPa]	1020	1830	345	5
Transverse Tensile Strength F_{2t} [MPa]	40	57	345	5
Longitudinal Compressive Strength F_{1c} [MPa]	620	1096	345	40
Transverse Compressive Strength F_{2c} [MPa]	140	228	345	40
Coefficient of Thermal Expansion	10	-0.9	12	10

The traditional reinforcing steel has a nearly three times lower longitudinal tensile strength with a density, four times higher than GFRP.

2.5.1.2. Corrosion Resistance. Products made from GFRPs have a high resistance to corrosion, outdoor weathering (alkali attacks) and chemical attacks comparing over the traditional reinforcing materials like steel. This benefits decreases the maintenance costs and extends the service life of structures. For example; when designing with steel in permanent structures, corrosion resistance correspondingly the maintenance costs has the utmost importance beginning with the design phase up to service time. There are several necessary course of actions which needs to be done such as section reductions and additional corrosion protections. However, corrosion is not an issue for GFRPs. Therefore there is no section reductions or additional corrosion protections needed when designing with GFRPs. This is solely great and sufficient benefit over conventional steel material in geotechnical area of use since ground water is the one of the primary geotechnical issues.

2.5.1.3. Light Weight. GFRP is approximately four times lighter than conventional steel. Lightweight is an important physical characteristic when considering the ease of transportation of the products from production areas to the construction sites, so there

will be no need to produce large sections in construction sites. It will also reduce;

- the assembly and installation time,
- weight of the total structure and,
- the section thicknesses of bearing parts.

2.5.1.4. Tailored Material Properties. Conventional construction materials like steel and concrete has specific stiffness properties in every direction. However, in many cases high performance in stiffness is only needed in one direction and for structural steel and concrete, there is no possibility to achieve that. On the contrary, FRP composites ensure the prospect to tailor material properties to be in compliance with only in the directions which are required, consequently enhancing capacity and economy (Potyrala, 2011).

2.5.1.5. Superior Durability. FRP materials have been used successfully over the half a century in a variety of applications in civil engineering industry in a wide range of area which includes bridge decks, slabs, column reinforcing wraps and reinforcing bars for concrete (Halliwell, 2002). They are resistant to atmospheric degradation, chemicals, etc., because of tailor made composition they have. Also they have increased lifespan.

2.5.2. Drawbacks

2.5.2.1. Low Young's Modulus. GFRPs have a high specific strength rate but, their drawback is a lower Young's modulus compared to conventional steel which limits the variability in the area of practice (Bai, 2013)⁴⁸. Because low modulus of elasticity leads to high deformations to mobilize high stresses and those high deformations may not be allowed for the sake of the stability of a particular structure. Thus, in that particular kind of structure, GFRP composite material will not be applicable.

2.5.2.2. Higher Initial Material Cost. The meaning of the cost term for civil engineering industry is material cost, fabrication cost and construction cost. GFRPs with their

lightweight, are reducing the transportation and construction costs. However, as their fabrication process is complicated and time consuming, material costs of GFRPs are still higher than reinforcing steel. The material cost comparison between reinforcing steel and composite materials can be seen in Table 2.8.

Table 2.8. Cost of materials per unit force 2004 (Burgoyne and Balafas, 2007).

Material	Strength (MPa)	Cost (£/Kn/m)	Cost ratio	Notes
Prestressing stell	1700	0.002	1	7-wire strand on coil
Reinforcing stell	460	0.006	3	Includes bending
GFRP	580	0.013	6.5	Excludes bending
Aramid fibre	2600	0.009	4.5	Fibre only
Aramid rope	2000	0.025	12.5	As a rope
AFRP	2000	0.025	12.5	As a pultrusion
CFRP	2000	0.025	12.5	As a pultrusion
(based on £1 = USD1.77= Euro 1.50,2004 prices)				

2.5.2.3. Lack of Ductility. Ductility is the ability of a material to deform under tensile stress or the ability of a material to endure plastic deformation without rupture. Ductile materials allow redistribution of internal forces, linked with increased structural safety, dissipation of energy from impact or seismic actions as well as warning of possible structural problems thanks to large plastic or inelastic deformations before failure (Keller *et al.*, 2001). However, GFRPs perform no ductility and shows no yield at all. This may be seen as a problem but, at the structural level, components fabricated from FRP composites can be designed to exhibit a sequence of damage mechanisms, which ensures a relatively slow failure with extensive deformation, leading to a progressive and safe mode of failure. One example of a structural system that can develop extensive deformation prior to failure is FRP bridge decks (Loper-Anido and Naik, 2000).

2.6. Codes, Standards and Test Methods

“One of the main factors delaying the widespread use of some types of FRP pultruded composites is the lack of technical specifications, design guidelines and codes” (Bai, 2013). Civil engineering design is directed largely by of the codes and standards which is mostly covering standardized traditional material types like conventional steel

(Lopez-Anido and Naik, 2000). Although new design standards and specification have been published by ASTM, ACI, ASCE, FIB, ISO which will be mentioned in this section, there is no standard, specification or research on FRP composites in Turkish competent authorities or organs.

Morrison Molded Fiber Glass Co. drew up the first design manual for pultruded structural profiles in 1971. The current manual was published in 1989 and is periodically updated (De Castro, 2005). Nowadays, there are several design codes, standards or test methods and researches to specify mechanical and physical properties of FRP composites all over the world. For almost thirty years, The American Society of Civil Engineers has founded a technical committee which is called as Structural composites and Plastics (SCAP) to address the design and implementation of composites (Megalooikonomou, 2007). This committee has published a design code and still working on to update this code.

2.6.1. Test Methods

There are several test methods which have been released and published by American Society of Testing and Materials, International Organization for Standardization and American Concrete Institute. Some of those test methods can be seen below;

- ASTM D7205 / D7205M: Standard Test Method for Tensile Properties of Fiber Reinforced Polymer Matrix Composite Bars,
- ASTM D7337 / D7337M: Standard Test Method for Tensile Creep Rupture of Fiber Reinforced Polymer Matrix Composite Bars,
- ASTM D7705 / D7705M - 12: Standard Test Method for Alkali Resistance of Fiber Reinforced Polymer (FRP) Matrix Composite Bars used in Concrete Construction,
- ASTM D7565 / D7565M - 10(2017): Standard Test Method for Determining Tensile Properties of Fiber Reinforced Polymer Matrix Composites Used for Strengthening of Civil Structures,
- ACI 440.3R-12 Guide Test Methods for Fiber-Reinforced Polymers (FRPs) for

Reinforcing or Strengthening Concrete Structures,

- ASTM D792. Standard Test Methods for Density and Specific Gravity (Relative Density) of Plastics by Displacement.
- ASTM D4476. Standard Test Method for Flexural Properties of Fiber Reinforced Pultruded Plastic Rods,
- ISO 10406-1:2015: Fiber-reinforced polymer (FRP) reinforcement of concrete – Test methods – Part 1: FRP bars and grids

2.6.2. Codes and Standards

Some of the Codes and standards which has been published by ACI, ASCE and AASHTO can be seen below;

- ACI 440.5-08 Specification for Construction with Fiber-Reinforced Polymer Reinforcing Bars,
- ACI 440.6-08 Specification for Carbon and Glass Fiber-Reinforced Polymer Bar Materials for Concrete Reinforcement,
- ACI 440.1R-15 Guide for the Design and Construction of Structural Concrete Reinforced with Fiber-Reinforced Polymer Bars,
- ACI 440.5-08 Specification for Construction with Fiber-Reinforced Polymer Reinforcing Bars,
- ASTM D7258 - 17: Standard Specification for Polymeric Piles,
- AASHTO Bridge subcommittee: T 21 FRP Composites.

3. GROUND SUPPORT SYSTEMS: GROUND ANCHORS

3.1. Introduction

3.1.1. History

Anchoring is a technique in which slender pre-stressed components are embedded into the in-situ soil or rock. It is a quite antique method which has been used for thousands of years. Tents are the oldest structures which were stabilized by using anchors or stakes. “Anchors were commonly used for stabilizing lightweight structures such as tents till nineteenth century” (Das, 1990). The first recorded use of conventional ground anchors were screw piles/anchors in 1836. This technology has been invented by Alexander Mitchell. Piles were screwed in to the ground by human and/or animal power using a large wood handle wheel called a capstan. “Screw piles on the order of 6 m long with 127 mm diameter shafts (Figure 3.1) required at least 30 men to work the capstan” (Perko, 2009).

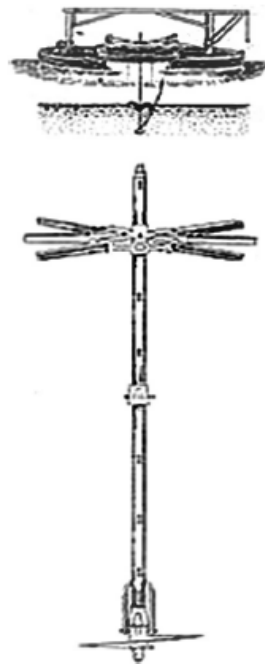


Figure 3.1. Early example of ground anchors (Perko, 2009).

the earliest and most notable implementation of anchors was in the strengthening of the Cheurfas dam project in Algeria, constructed by Coyne in 1934 (Xanthakos, 1991). This structure was destroyed twice because of foundation instability. That instability problem was solved by the use of several 1000 ton capacity anchors which can be seen in Figure 3.2.

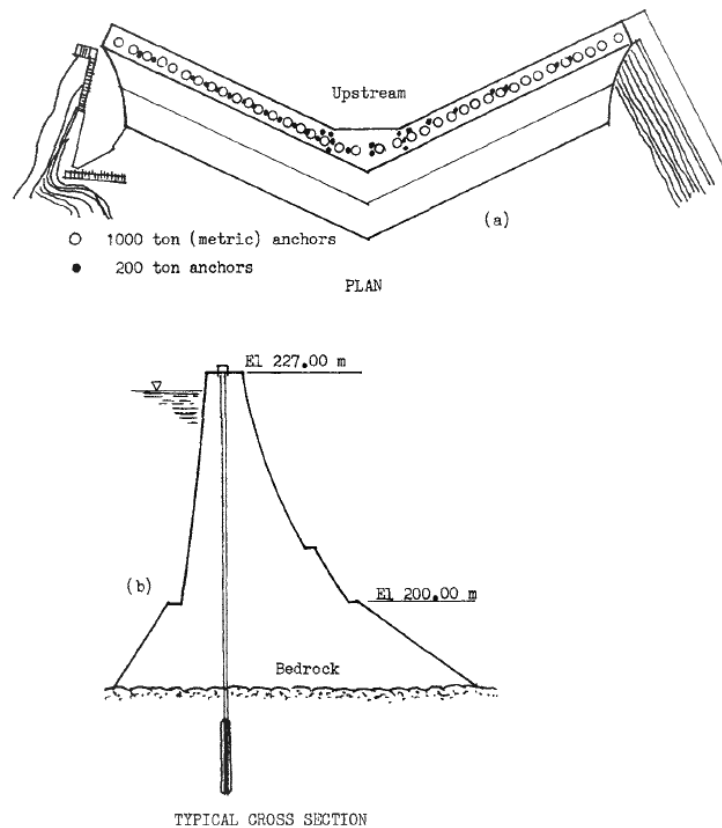


Figure 3.2. General Plan and Section of Cheurfas Dam in Algeria (Xanthakos, 1991).

Following these examples, the importance of tension bearing structural components like anchors have increased and this led to the development of those components in newly developed world.

3.1.2. Ground Anchor Types

Ground anchors are load bearing functional tools which is installed in ground to support structures or excavation by transmitting the applied stresses into the ground. Basically ground anchors can be divided into four categories, those are;

- Anchor piles,
- Plate Anchors,
- Helical Anchors,
- Grouted Ground Anchors,

3.1.2.1. Anchor Piles. Anchor piles are tension load bearing piles which have been used to resist uplift or inclined force coming through the structure or the ground itself. An illustration may be seen in Figure 3.3.

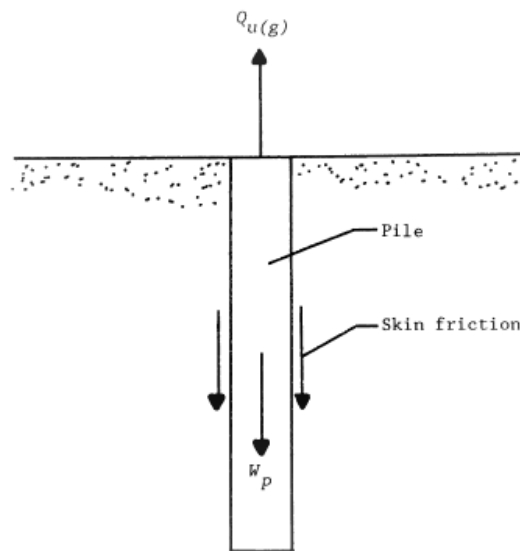


Figure 3.3. Figure Anchor Pile (Das, 1990).

3.1.2.2. Plate Anchors. Plate anchors may be made of steel plates, precast concrete slabs, poured concrete slabs, timber sheets, and so forth. They may be horizontal to resist vertically-directed uplifting load, inclined to resist axial pullout load, or vertical to resist horizontally-directed pullout load (Das,1990).

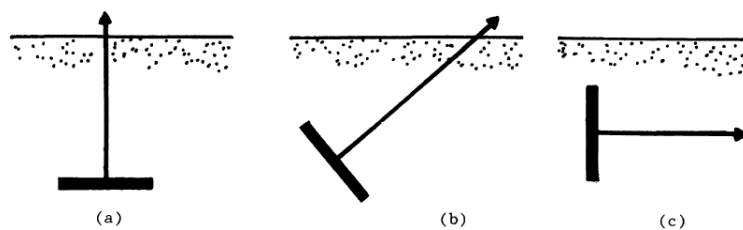


Figure 3.4. Anchor Pile (Das, 1990).

3.1.2.3. Helical Anchors. A special fabricated steel rod consisting of one or more helix-shaped plates affixed to a central shaft which is rotated into the ground to support structures (Perko, 2009). These anchors are driven into the ground in a rotating manner using truck- or trailer-mounted auguring equipment where the soil conditions permit. An axial load is adjusted to the shaft while rotating to move it into the earth (Das, 1990).

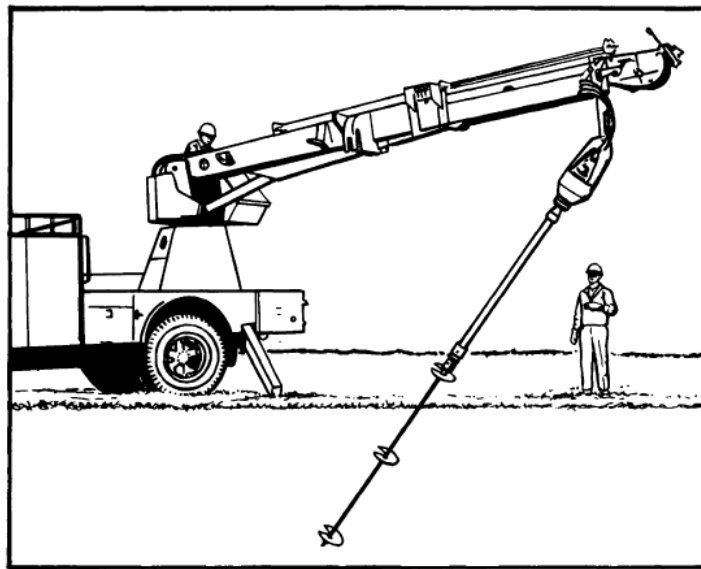


Figure 3.5. Figure Installation of Helical Piles (Das, 1990).

3.1.2.4. Grouted Ground Anchors. A grouted ground anchor consists of a structural component which is embedded (steel, composite bar or tendon) into the geo-material (soil or rock) shaft and a grout which is filling the shaft. The grouted ground anchor is used to transmit an applied tensile load from a main structure to the surrounding ground. The basic parts of a grouted ground anchor include the: (1) free (unbonded) length; and (2) bonded (fixed) length which may be seen in Figure 3.6. This research will cover the grouted ground anchorages thoroughly within this Chapter.

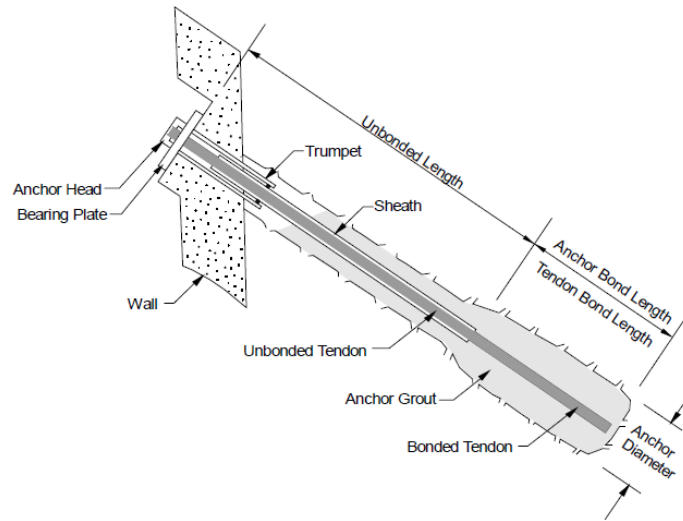


Figure 3.6. Figure of a Ground Anchor (FHWA, 1999).

3.2. Fields of Application of Ground Anchors

Grouted ground anchors has been successfully used for temporary and permanent design purposes in variety kind of structures and in-situ soils and/or embankments. The function of a grouted ground anchor is to mobilize the shear strength of a geo-material (soil or rock) by creating passive or active frictional forces. Therefore, An anchorage can bear earth loads which is acting on a structure, and transfer them back into the ground by frictional forces (Xanthakos, 1991). As, soil is very weak in the tensile strength, those tension forces must be transferred to the grout by different methods.

Grouted anchors produce minimal soil disturbance and they can be post-tensioned to create pre-determined reaction to provide a definite tensile bearing capacity, hence they propose a popular solution method for variety of problems which can be encountered in civil engineering industry. The most common application areas of grouted anchors in civil engineering industry can be seen in Chapter 3.2.1 respectively.

3.2.1. Soil Pre-consolidation

Pre-stressed ties can be utilized to apply a compressive force directly to soil making it consolidate and enhance its strength under new loads. (Xanthakos, 1991). Preloading is a popular technique which has been used to remove the settlements of very weak clayey soils. The methodology of the preloading technique is loading of the weak soil by the at least 1.2 to 1.3 times of the structures pressure. This method is normally implemented by using rock embankments or wastes as a load, however this is time-consuming and costly. Preloading may be done by utilizing a few number of ties which are anchored into bedrock and in this manner pre-stressed, producing the same impact as consolidation by pre-loading (Xanthakos, 1991) which may be seen in Figure 3.7.

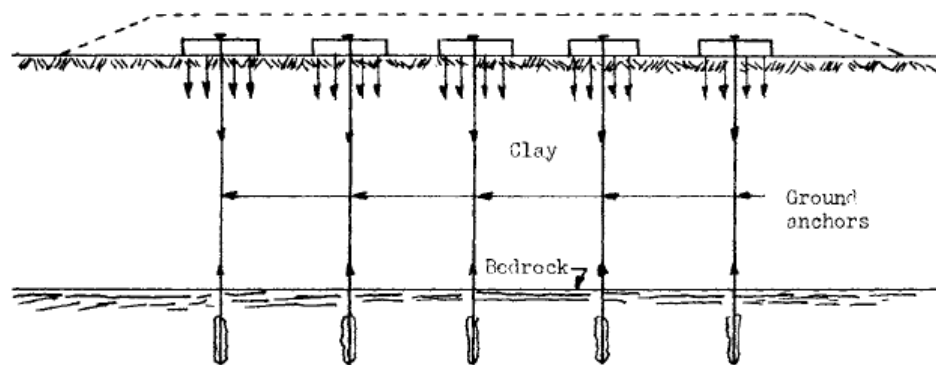


Figure 3.7. Pre-consolidation of a soft soil by the use of pre-stressed anchored ties (Xanthakos, 1991).

3.2.2. Uplift Forces

Engineering structures may experience uplift forces due to bottom heave, wind loads or hydrostatic pressure alike. Vertical anchors can be used to stabilize these forces by transferring the tensile forces originated from uplift forces to the bottom stable ground (soil or rock). Figure 3.8 and Figure 3.9 demonstrates a structure which is subjected to an uplift force due to groundwater pressure. The buoyancy force has been stabilized by the tension anchors.

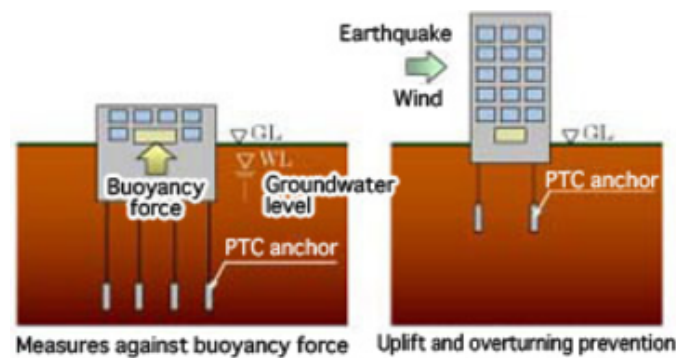


Figure 3.8. Use of Anchors to hold down a structure subjected to uplift forces (Solution, 2017).

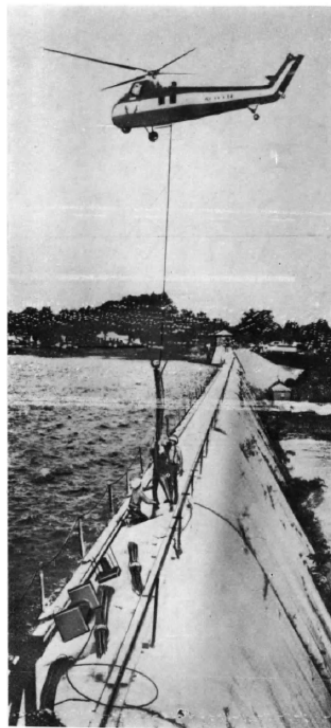


Figure 3.9. Stabilization of reconstructed Milton Lake Dam (USA, Ohio). The anchors are installed at the spillway crest (documentation Losinger International).

3.2.3. Overturning Forces

Many structures experience overturning moments due to lateral forces like wind and earthquake or eccentricity. In the Figure above, the overturning effect of the earthquake and wind forces on high rise building counter-balanced by the use of vertical grouted ground anchors.

The alternative categories of tie-down structures are concrete or masonry dams anchored to stabilize overturning arising from uplift or excessive hydrostatic pressure after foundation deterioration or in consequence of lifting the crest of the dam (Xanthakos, 1991).

Using ground anchors in structures which is subjected to overturning forces is advantageous because anchors can be placed at any distance (max.) from the point of overturning which leads to greater resisting moments by using smaller forces from ground anchors.

3.2.4. Deep Excavation

According to Peck *et al.*, excavations whose depth is more than 6m could be defined as deep excavations (Ou, 2006). One of the deep excavation methods is anchored excavation. In this method, grouted ground anchors are used to counter balance the lateral earth pressure which is acting on the deep excavation lining or wall. This can be seen in Figure 3.10. The method is most suitable for temporary excavations and should be used in weak soil layers.

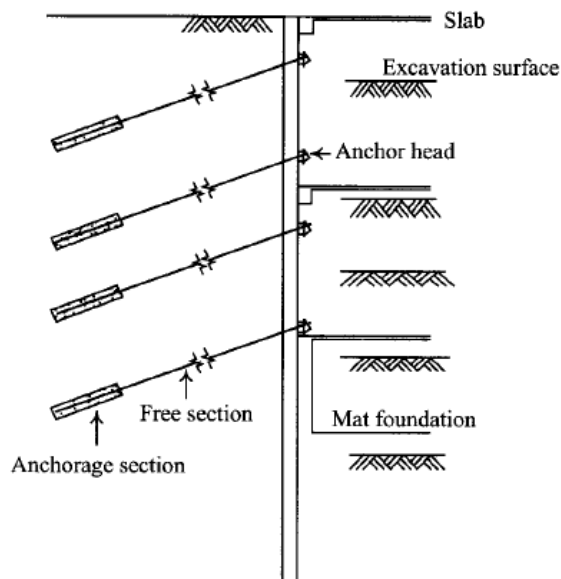


Figure 3.10. A Typical cross section of a anchored excavation (Ou, 2006).

3.2.5. Slope Protection

Slope is a surface of which one end or side is at a higher level than another. Slopes can be classified in two types (1) natural slopes (usually rock slopes), (2) man-made slopes (including slopes of cuts or slopes of embankments). Soil/rock nails and/or anchors may be used to stabilize a man-made slope like cuts or natural rock slope which is deteriorated due to natural hazardous effects. Figure 3.11 demonstrates an anchored slope which is supporting an abutment.

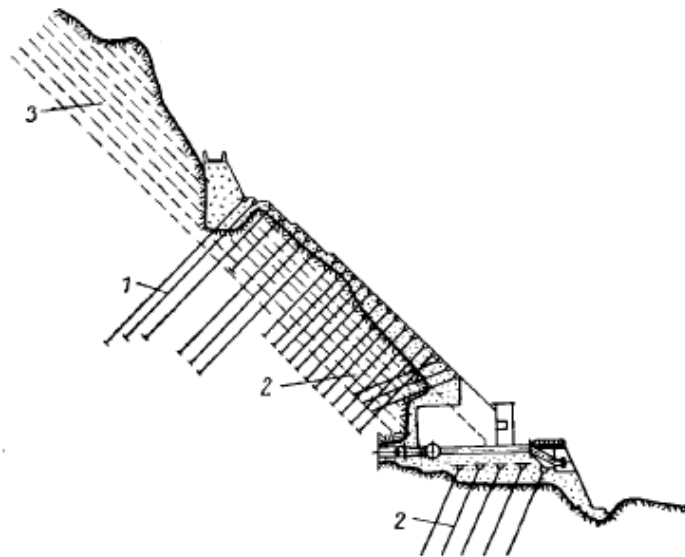


Figure 3.11. Stabilization of the unstable slope of a dam reservoir in South Africa using pre-stressed anchors (Hobst and Zajic, 1983) 1 - system of cable anchors (62 wires of 7 mm dia) 39.6 m long, 2 - cable anchors (10 wires 7 mm dia) 26 m long, 3 - phyllite beds.

3.3. Classification and Grouping of Grouted Ground Anchors

Grouted ground anchors may be classified by behaviour of anchors, by the method of grouting or by the conditions of engineering structure. These distinctions will be described below accordingly.

3.3.1. Permanent and Temporary Anchors

There is very arbitrary division between temporary (short-term) and permanent (long-term) (Xanthakos, 1991). Therefore this division is mostly in academic research interest. However, in practices of some countries temporary anchors service life is two years. If the grouted anchor needs to work more than 2 years, then it needs to be designed at permanent ground anchor. According to the British standard temporary anchorage being classified as two distinct types in itself;

- “Temporary anchorages where a service life is less than 1.40 for six months and failure would have no serious consequences and would not endanger public safety, e.g. short term pile test loading using anchorages as a reaction system”(BS).
- “Temporary anchorages with a service life of say up to two years where, although the consequences of failure are quite serious, there is no danger to public safety without adequate warning, e.g. retaining wall tie-back”(BS).

According to British standards, the definition of permanent anchors is;

- “Permanent anchorages and temporary anchorages where corrosion risk is high and/or the consequences of failure are serious, e.g. main cables of a suspension bridge or as a reaction for lifting heavy structural members”.

The distinctions in design of temporary and permanent anchors are in the safety factors for determining bearing capacity of the tendons and shear strength of grout-ground interface and the level of corrosion protection. Equally essential are the ground conditions and corrosive tendencies of the soil environment in which the anchors are embedded. Also, when estimating the allowable working loads, long-term stability and creep characteristics must be considered all together with possible reduction of frictional ability along shear zone (Xanthakos, 1991).

3.3.2. Anchor Types; Method of Grouting

Anchor geometry dictates the resulting anchor pull-out capacity for a given ground condition, however, the transfer of stresses from the fixed anchor to the enclosing soil is also affected by construction technique, especially the grouting method, and to a lesser extent drilling. (Littlejohn, 1980). According to Littlejohn, grouting pressure directly affects the pull-out capacity of grouted ground anchors (Figure 3.12). Mayer (1983) and Ostermayer (1993) also stated that the application of grouting pressure sufficiently increases anchor capacity up to a certain level which is the limit of the pull-out capacity. Although the effects of the grouting type and pressure are not emphasized by the researchers most of the time, it is generally accepted that grouting type and pressure has a great effect on pull-out capacity.

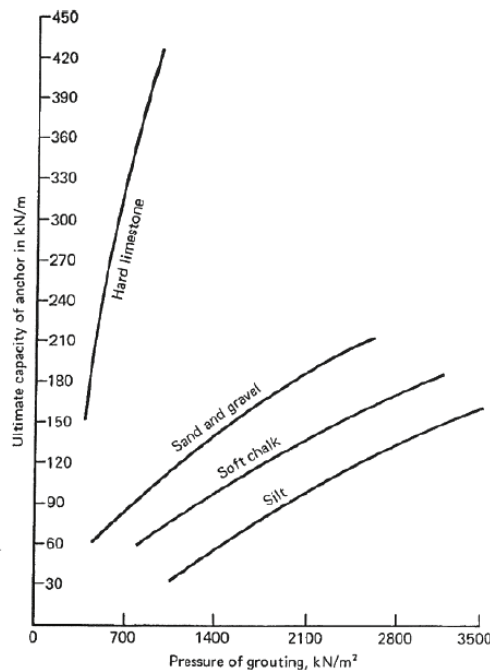


Figure 3.12. Influence of the grouting pressure on the pull-out capacity of anchors from Littlejohn (1980).

There are four main groups, characterized by the stress transfer mechanism from the fixed length to the surrounding soil (Xanthakos, 1991). These four main groups may be seen in Figure 3.13.

Type A; This type is characterized by gravity grouting of the drilled boreholes which may be lined or unlined according to the requirements on hole stability (Littlejohn, 1980). Transfer of load is provided through shear resistance which is resulted from the frictional surface between the grout and the ground. This grouting type is very suitable not only for rocks but also stiff cohesive soil medium.

Type B; This type is characterized by low pressure-grouting of the drilled boreholes by the use of a lining tube or in situ packer, where the effective diameter of the fixed anchor is increased with minimal disturbance as the grout permeates through the pores or natural fractures of the ground (Littlejohn, 1980). This kind of anchor is mostly preferred in fissured rocks, coarse alluvium and cohesion less granular soils because the grout cannot penetrate into soil in this pressure rate (nearly same as the overburden pressure) but it compacts the anchorage area locally.

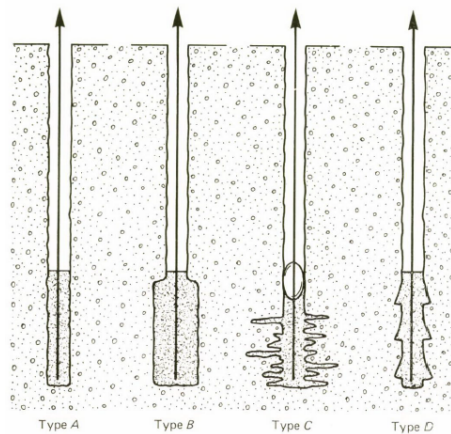


Figure 3.13. Anchor grouping according to the method of load transfer (Littlejohn, 1980).

Type C; High-pressure grouted borehole via a lining tube or in situ packer, where the grouted fixed anchor is enlarged via hydro fracturing of the ground mass to give a grout root or fissure system beyond the core diameter of the borehole (Littlejohn, 1980). This type of anchor is commonly used in cohesionless soils and cohesive deposits.

Type D; “Tremie-grouted borehole in which a series of enlargements (bells or under-reams) have previously been formed mechanically” (Littlejohn, 1980). The dis-

tion of this kind of anchor is mobilization of end bearing resistance by under reams. Pull out resistance force is the combination of end bearing resistance of under reams and side shear in this type of anchors. This type is commonly employed in stiff to hard cohesive soils.

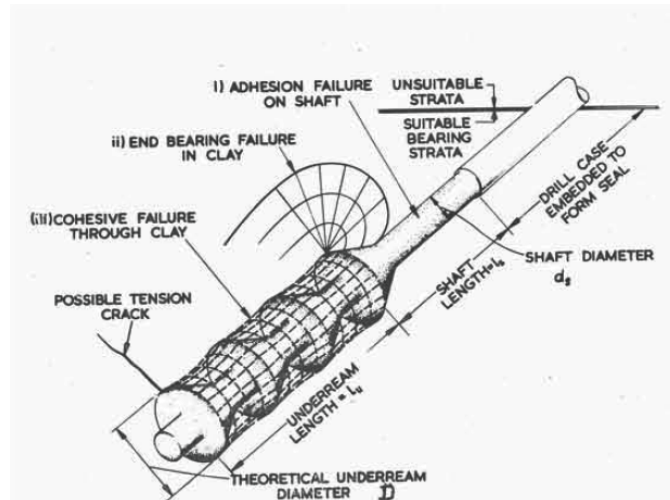


Figure 3.14. Diagram of an under-ream anchor (Basset, 1970).

3.3.3. Active and Passive Anchors

As an in situ reinforcement method, the load-transfer mechanism of ground anchors is caused by two distinct ways; anchor movement in soil or soil movement on anchor. Therefore, by taking account of the mechanical behaviour, ground anchors can be divided into two main types;

- Active Anchors
- Passive Anchors

3.3.3.1. Active Anchors. Active anchors are the pre-stressed anchors. The initial force is applied to the structure in active ground anchors. (Xanthakos, 1991). In this type of anchorage, the ground anchor is pulled by a jacking device to a certain degree of loading, thus the movement of anchors grouted length mobilizes a frictional force between the anchor-soil interfaces. Therefore, the interaction between anchor and soil is mobilized by the deflection and movement of the anchor (Xanthakos, 1991). This

type of anchor is commonly employed to support superstructure loads (such as in deep excavation design projects). This anchorage type is broadly reviewed in this present study.

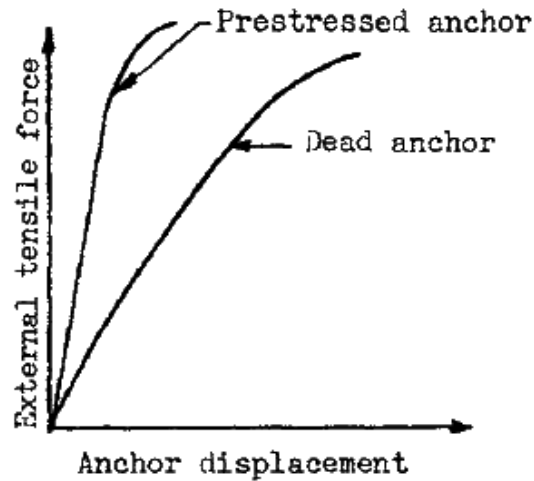


Figure 3.15. Active (prestressed) and passive (dead) anchors. Relative displacement as function of applied force.

3.3.3.2. Passive Anchors. Passive anchors, are not pre-stressed. They respond to loading only when the structure or soil around the anchor begins to mobilize deformations on one side (Xanthakos, 1991). Soil and rock nails/bolts can be regarded in this type of anchorages. Therefore, this type is most commonly suitable for slope stabilizations, tunneling support structures and retaining wall structures.

3.4. Parts and Assembly of Grouted Ground Anchor

Grouted ground anchorages has three main components; commonly known as “anchor head”, “fixed length-bond length” and “free length” which can be seen below in Figure 3.16. In this section, main assembly parts and materials will be discussed briefly.

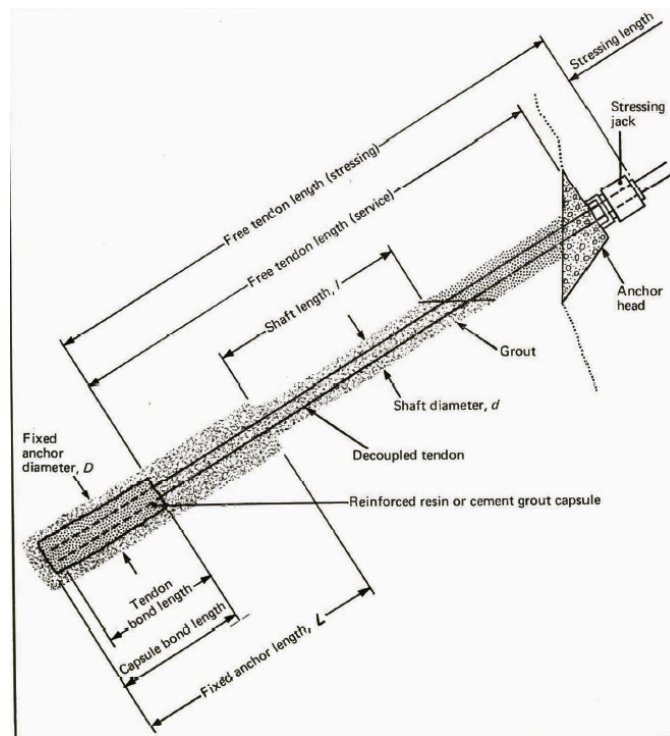


Figure 3.16. Schematic representation of ground anchors (Littlejohn, 1980).

3.4.1. Anchor Head

Anchor head is the connective part of the ground anchor with the main structure. Thus the anchor head is transferring the tensile force to the structure. By a simple mechanical interlock, the anchor head locks the anchor on to the structure, also it serves to apply the pre-stressing force to the anchor (Xanthakos, 1991). The anchor head body is composed of stressing head, wedges and bearing plate which can be seen in Figure 3.17. The bearing plate is employed to transfer the tensile load to the structure.

3.4.2. Free Length/Tendon

Free length-tendon length is the part of the anchor which connects the anchor head to the fixed length. There is no load transfer mechanism developed in this part of anchor. This part is free from the surrounding soil or the grout as the only function of this part is to elongate under pre-stressing without interferences.

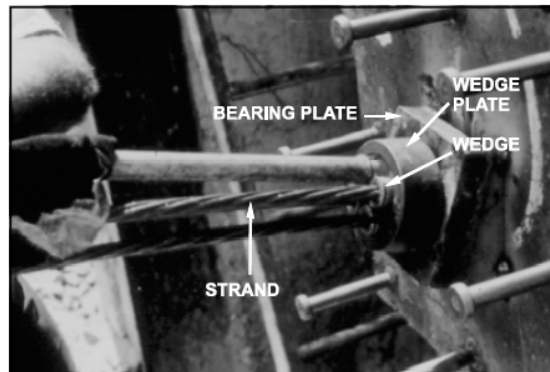


Figure 3.17. Schematic representation of typical ground anchor head (strand tendon)
(FHWA, 1999).

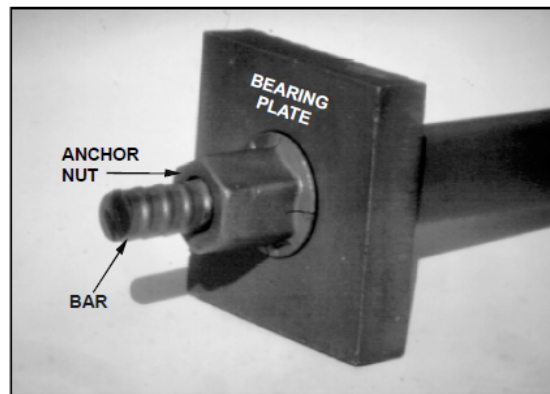


Figure 3.18. Schematic representation of typical ground anchor head (bar tendon)
(FHWA, 1999).

3.4.3. Fixed Length

This part is also called as a design length or bonded length/anchor body. It serves as the main part which dispatches the tensile force by the frictional bond to the surrounding rock or soil alike. This part is the grouted part of the tendon or bar. Cement grout used to establish the bond between tendon and ground. It normally consists of cement mortar, which in range of water/cement ratios from 0.38 to 0.50. (Smoltczyk, 2002). The grout provides the bonding in fixed length and also protects bar/tendon from the harmful (corrosive) effects. However, advanced corrosion protection is needed for steel bar and tendon for permanent applications.

“With the bond and tension type anchor, the load is initially transmitted to the front of the grout column. Due to the elasticity of the tendon the load is transmitted to the rear of the grout column, as load is increased” (Smolczyk, 2002).

3.4.4. Materials Used in the construction of Anchors

3.4.4.1. Tendon. Anchor tendons can most commonly consists of wires and strands but also it can consists of bars and pipes (but very rare). Variations in the size and cross-sectional area, ultimate strength, elastic limit and development of mechanical bond are generally common (Xanthakos, 1991).

Bar anchors which is subject to review in this present research, is plain or threaded, provide the simplest type of tendon. They can be more readily protected against corrosion, and for shallow or low-capacity installations they usually cost less. They have sufficiently high stiffness, which facilitates placing by allowing the tendon to be handled in appropriate lengths (in steel) and with minimum risk of mechanical damage, and in some cases the bar can be used as the drill rod even. (Xanthakos, 1991). The comparably large residual elongation of bars after stressing led significant ground movement to occur before anchor failure is induced. However, longer bars of conventional steel material are rather more difficult to handle, and for long anchors, tendons of wire or strands are preferable; the latter are flexible, and are thus more easily transported and inserted into boreholes of several tens of meters long, even from relatively small manipulating platforms and regardless of borehole direction. Pre-stressing wire is generally produced from cold-drawn plain carbon steel (Xanthakos, 1991). The basic heat treatment - the patenting - which gives the material its special structural properties and which provides the necessary strengthening during the cold-drawing, takes place in furnaces with automatic thermal control (i.e. control of the heating temperature and the temperature of the lead bath which constitutes the cooling medium). Heat treatment is followed by the pickling process, in which the material is de-scaled with chemicals and its surface is prepared for the subsequent shaping by cold-drawing (Hobst and Zajic, 1983). Tendon wire varies in diameter, the most common size ranging from 5 to 8 mm, and has a usual ultimate tensile strength 1670 N/mm² (Xanthakos,

1991).

This tendon type is composed of a group of wires generally ranges from 4 to 20. These wires are arranged in a stranded form by taking a common axis of a straight wire. They are generally in diameters of 12.7 and 15.2 mm, respectively (Xanthakos, 1991). This grouping leads to utilization of the full mechanical properties of wires and maintain their flexibility while creating larger pre stressing tendons (Hobst and Zajic, 1983).

The 7 wire strands are most common and have a simple structure. They are usable in sizes of 13 mm (0.5 in), 15 mm (0.6 in), and 18 mm (0.7 in); 19-wire strand is also common and available in sizes of 22.2, 25.4, 28.6, and 31.8 mm. The disadvantage of stranded cables compared with multi-wire cables is their larger tendency to creep and their lower tensile strength.

Corrosion: The risk of the corrosion for the anchor tendons depends on number of circumstances such as the aggressivity of water, soil or atmosphere, the ground water level, ground permeability and stress level in the tendon (Smoltczyk, 2002). The environment in which anchors are set is generally more aggressive with high degree of humidity or water seepage and the usual presence of salt solutions (Xanthakos, 1991). There are generally three types of corrosion attack;

Generalized attack; in this type of corrosion attack, the corrosion is uniform and covers the steel material which may be seen below in Figure 3.19 In basis, this may be termed as chemical corrosion since it corresponds to acid attack in a laboratory. The metal is gradually transformed into ferrous ions, uniformly from the outer surface inwards. The accompanying reduction in cross section is basically uniform, whereas the center of the metal remains intact and sound (Xanthakos, 1991).

Table 3.1. Technical data for pre stressing bars (Hobst and Zajic, 1983).

Technial data for prestressing bars				
Bar dia in mms	Sectional area in sq. Mms	Breaking load (kN)		
		Macalloy	Dywidag	Sumitomo
9.2	66.2			68.3 (3) 78.1 (4)
11.0	95.1			98.4 (3) 112.4 (4)
13.0	132.7			137.3 (3) 156.4 (4)
15.0	176.7	191	191	
16.0	201.1		296	
17.0	227.0			234.5 (3) 267.9 (4)
20.0	314.2	325		416.7 (3)
23.0	415.5			476.4 (4)
25.0	500.0	490.9		534.1 (3)
26.0	530.9			610.5 (4)
26.5	551.5		568 (1) 678 (2)	
32.0	804.2	800	828 (1) 989 (2)	812.7 (3) 928.8 (4)
35.0	862.1	950		
36.0			1.049 (1) 1.252 (2)	
40.0	1.256.6	1.250		
1) Grade ST 835/1030				
2) Grade St 1080/1230				
3) Class A2				
4)Class B2				

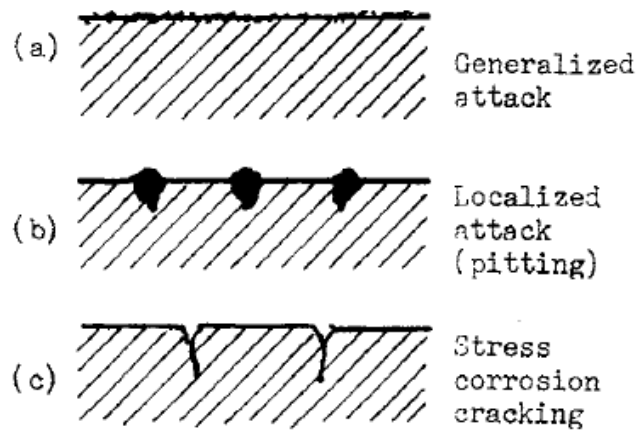


Figure 3.19. Main types of corrosion (Xanthakos, 1991).

Localized attack; This may be termed electrochemical corrosion, and is manifested as deep or shallow pitting as shown in Figure 3.19(b). The formation of holes

leads to local stress concentration and eventually leads to failure. Pitting is observed in the presence of salts used for thawing ice, or near seawater with traditional prestressing steel, (Xanthakos, 1991).

Other types are Stress Corrosion-Hydrogen Cracking, bacterial attacks, corrosion fatigue, and corrosion involving oxygen.

The corrosion attack on steel tendon has a significant effect both in design stage and the service life of the anchorage and as may be seen above there are many types of corrosion for a design engineer to take into account in his design. As explained before, for temporary purposes corrosion protection is not a significant problem but for permanent purposes, which the ground anchors are needed to work until the end of service life of the structure or to stabilize a slope, corrosion protection takes the first priority when designing with a bar/strand or wire which is produced from conventional steel material.

Recently, Fiber reinforced plastic (Glass, Aramid or Carbon) is increasingly being used for their enhanced durability which includes resistance to corrosion, alkalis and harmful solutions in soils, its light weight and high tensile capacity.

3.4.4.2. Fiber Reinforced Plastic Anchoring Rods. Traditional ground anchorage systems have not been regarded as suitable and durable stabilization method for slope stability/ landslide rehabilitation in highly corrosive and harmful environments because of corrosion (degradation) problem of steel material. There have been over 35 recognised cases historically of ground anchors where anchor failure has taken place in a direct consequence of tendon corrosion with many other cases in which influential improvement works have been needed to hinder failure. (Littlejohn, 1987, Zhang and Benmokrane, 2005). Therefore, other engineering methods rather than conventional ground anchorage or soil nailing method is being used for those areas. Recently, a new material namely FRP (fiber reinforced plastic) has been emerged as a noncorrosive and nonmetallic material which has a high tensile strength but low unit weight properties.

This material is now a good alternative for steel reinforcement.

The advantages of using FRP composites for ground anchors compared to the conventional pre-stressing steel tendon ground anchor include (Benmokrane et al., 2000, Zhang *et al.*, 2000): High axial tensile strength (generally equal to or greater than pre-stressing steel),

- High corrosion resistance,
- Lightweight; FRP is approximately 15% - 20% lighter than pre-stressing steel tendons,
- Optical fibers can be integrated into FRP strands allowing for real time monitoring of ground anchor performance.

Limitations of FRP ground anchorages may be considered as its relatively high cost and brittle nature.



Figure 3.20. Typical GFRP Ground Anchorages (Blugeo, 2017).

3.5. Ground Anchor Design

It is acknowledged by numerous researchers that the failure of ground anchor can occur in following modes;

- Bond failure between tendon and grout,
- Failure in ground which is supporting the anchorage,
- Failure in grout column around the tendon,

- Failure in anchor head (by excessive displacement, etc...),
- Failure in tendon (steel or composite)-rupture,
- Failure in the shear zone of grout/ground interface,
- Combination of those modes above.

Nevertheless, failure of ground anchors generally takes place in tendon or in shear zone of grout and ground interface. Thus, the following section and analyses in the present research will focus on these failures.

3.5.1. Tendon Design

By the time load is implemented, the steel tendon is stressed in tension. If the load surpasses the strength of material, it will be failed by excessive yielding followed by sectioning. (Xanthakos, 1991). Stress-Strain relationship for a steel tendon may be seen in Figure 3.21. In this Figure, f_{pu} is the ultimate tensile strength, T_G is the tensile stress which is corresponding to specific elongation, and E_s is the elastic limit.

The design load for tendon is always determined from ultimate tensile strength value for a specific safety factor. Certain codes defines different safety factors for conventional steel tendons. FRP strand anchors have to take account of the same design standard and methods as conventional steel tendon ground anchors.(Sentry M. *et al.*, 2009).

For conventional steel tendons; according to current recommendations, $0.625f_{pu}$ is the working stress for steel tendons in temporary installations, whereas for permanent anchors a working stress $0.50 f_{pu}$ is now accepted for design purposes. Thus, the factor of safety against tensile failure of the steel tendon is 1.6 for temporary and 2.0 for permanent anchors (petros).

For composite tendons; Maximum working load of $0.6f_{pu}$ is recommended when designing high capacity ground anchorage systems with FRP (Sentry M. *et al.*, 2009). Wide range of capacity values for working loads have been adopted. Those values may

be seen below in Table 3.2.

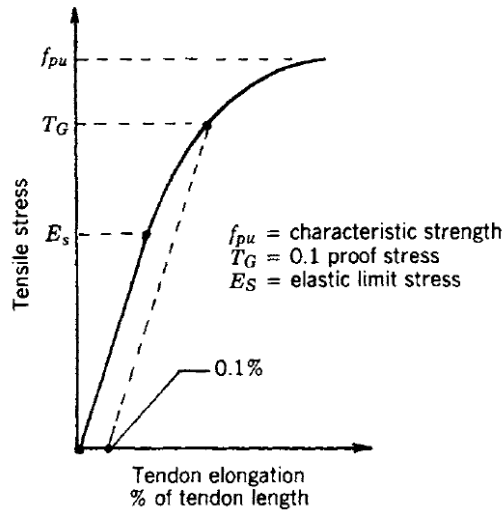


Figure 3.21. Stress-strain response for normal high-tension steel.

Table 3.2. CFRP ground anchors field application in Japan (Benmokrane *et al.*, 1997) (Sentry M. *et al.*, 2009)

Project	Year	Anchor number	Rock type	Tendon Size (mm)	Ultimate Load Pu (kN)	Design Load (% Pu)	Toatal Length (m)	Bond Length (m)
Slope stabilisation, Niigata	1993	42	Granite	6 ϕ 12.5	852	0.6	10.5	6.5
Slope stabilisation, Ito, Shizuoka	1995	20	Soft Rock	4 ϕ 12.5	568	0.63	9-11.5	
Slope stabilisation, Nazusa, Gifu	1995	30	Soft Rock	3 ϕ 12.5	426	0.43	7.6-10.6	
Slope stabilisation, Ishikawa	1995	40	Soft Rock	6 ϕ 12.5	852	0.57	10-11.5	
Slope stabilisation, Yamanashi	1996	10	Soft Rock	3 ϕ 12.5	426	0.53	7.3	
Slope stabilisation, Kyoto	1996	10	Soft Rock	2 ϕ 12.5	284	0.66	7.3-17.3	

3.5.2. Fixed Length Design -Grout/Ground Interface

It is generally known that the majority of failures in ground anchorage occurs in the shear zone between grout and ground (Xanthakos, 1991) and the shear stress distribution along this failure zone is not uniform. This derives from the overall discrepancy between the elastic modulus of the anchor tendon, of the anchor grout and of the surrounding ground. (Smolczyk, 2002).

Therefore, the analyses and investigations mainly focused on this failure mode with different material types and varying soil/rock types.

At the initial stage of anchor loading, the bond stress is first concentrated on proximal end of the anchor, at this stage there is no stress in distal end of the anchor. As the load increased, the shear stress along the proximal end increases until it reaches the peak value, after that the shear stress along the proximal end decreases until it reaches a residual value. As the load increases, so the concentration zone of peak shear stress moves towards to distal end of the ground anchor. After it reaches the distal end, more load increase, generates the failure. This shear failure mechanism may be seen below in Figure. Also, it is important to remember that this failure type is not applicable to the soft soil types like loose sand and deformable clay.

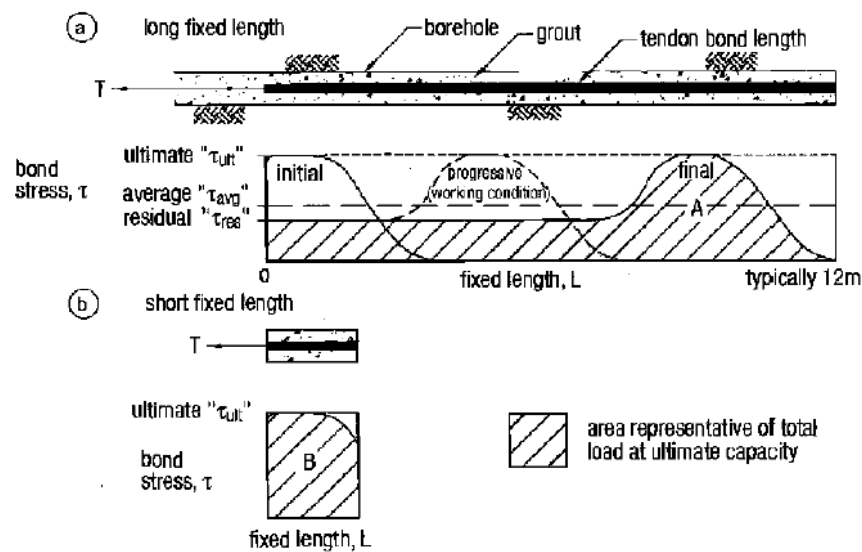


Figure 3.22. Development of bond stress distribution along a fully bonded fixed anchor, a) Long fully bonded fixed anchor with progressive de-bonding, b) Short fixed anchor with negligible effect of de bonding (Ostermayer, 1970) (Smoltczyk, 2002).

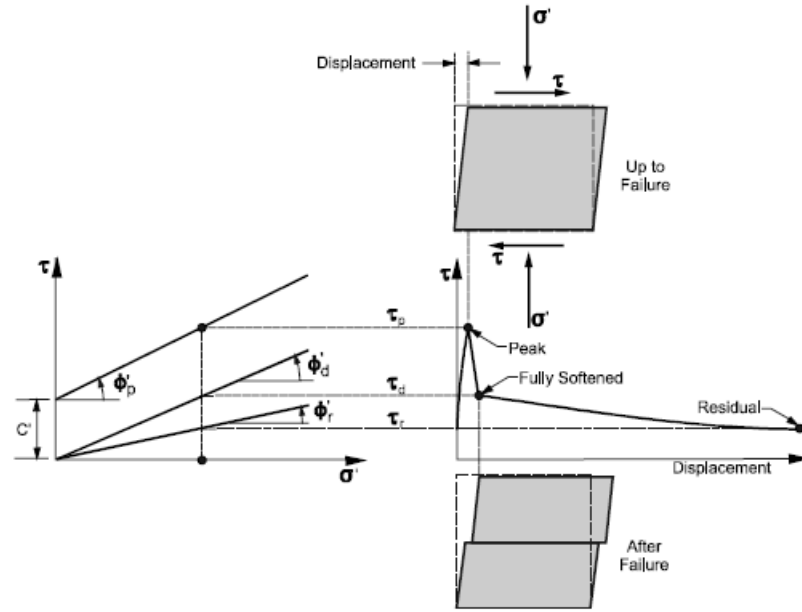


Figure 3.23. Simplified drained stress-displacement relationship for a stiff clay (FHWA, 1999).

3.5.3. Estimation of Bearing Capacity/Ultimate Anchor Load

3.5.3.1. General Design Approach. It is fully acknowledged that generally the bearing capacity of ground anchors are very dependent to soil/rock types, grout injection methods (which has been mentioned at Chapter 1.3.2) and length/diameter of the grout body. Most of the time bearing capacity of the ground anchor is the result of the combination of those factors. The general bearing capacity formulae for ground anchors may be seen in Equation 3.1;

$$T_{ult} = \pi \times d \times L \times t_{ult} \times f_{eff} \quad (3.1)$$

where is the $T_{ult}[kN]$ Ultimate capacity of the ground anchor, is the $d[m]$ diameter of the borehole (grouted), is the $L[m]$ Fixed length of the anchor, is the $t_{ult}[kN/m^2]$ Ultimate bond stress/Factor of cu or SPT-N, is the $f_{eff}[dimensionless]$ Efficiency factor which is a function of the fixed length and may be estimated in the formulae below

3.5.3.2. Bearing Capacity in Non-Cohesive Soils. It was Germany in 1958 that Bauer for the first time demonstrated that a bar could be anchored into gravels through a 150mm diameter borehole with the aid of cement grout injection under pressure. Since then the development of grouted anchors in frictional soils has steadily gained momentum, particularly in Europe, the Americas and South Africa (Littlejohn, 1980).

Following Bauer, two different approaches have been emerged to evaluate the bearing capacity of grouted ground anchors in non-cohesive soils after extensive surveys and researches.

3.5.3.3. Diagrammatic Interpretation. This approach estimates the bearing capacity of a ground anchor by using charts which represents the findings of the surveys and it is done with varying fixed ground lengths embedded in 3 different types of granular soils. The diagrams may be seen below in Figure 3.24.

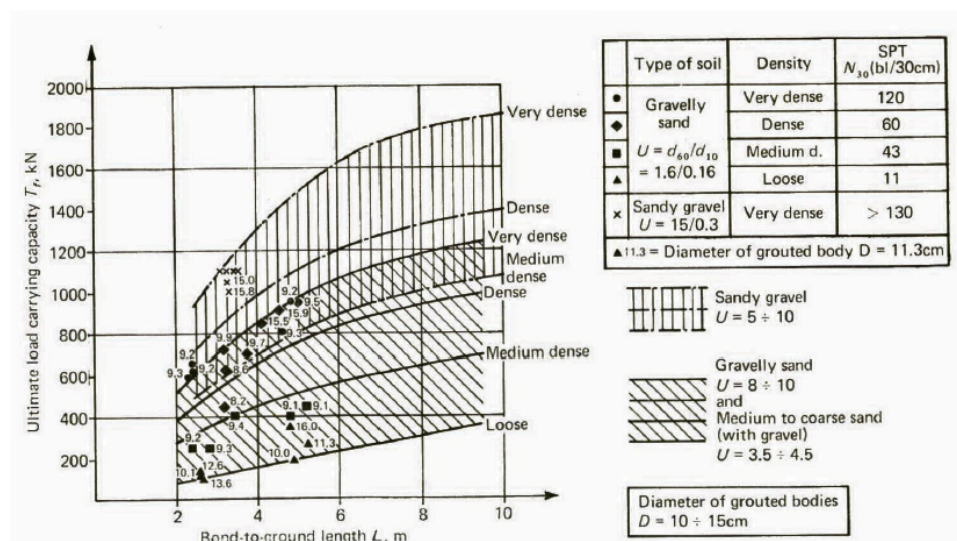


Figure 3.24. Ultimate load capacity of anchors in non-cohesive soils.

According to Ostermayer (1974) and Barley (1997) the results which can be interpreted from the diagram may be summarized as follows;

- (i) Ultimate load of the ground anchor increases rapidly with increasing density of the surrounding soil medium,

- (ii) With the same value of relative density of the surrounding soil medium, ultimate load increases with an increase of the coefficient of uniformity, U , and grain size,
- (iii) The increase of ultimate load tapers off gradually with increasing fixed length. This means that ultimate load is not commensurable to the fixed length. In dense grounds, the increment of ultimate load is particularly small for fixed lengths of more than 6 or 7 m. Therefore, the use of longer fixed lengths submits little efficacy.
- (iv) In non-cohesive conditions, where borehole diameters are in the 100 to 150 mm range, the ultimate load may not increase proportionally with borehole diameter. This is due to a decrease of radial normal stresses and the interlocking effects with an increasing diameter of the grout body.
- (v) There is little influence on ultimate load where overburden pressure from surcharge of the fixed anchor length is greater than 4m below ground level.
- (vi) The installation of a multiple of unit anchors in one borehole with short unit fixed lengths at staggered depths can allow the mobilization of very high capacities. The ultimate load of each of short unit (tult) can be evaluated from and summated to provide the ultimate load of the multiple of anchors (Barley, 1997).

3.5.3.4. Design Formulae Approach. For the B type grouted ground anchors (Chapter 1.3.2) the load bearing capacity may be estimated by Equation 3.1 for f_{eff} Equation 3.2 may be used;

$$f_{eff} = 1.6xL^{-0.57} \quad (3.2)$$

The graphical representation of the efficiency factor to increased fixed length may be seen in Figure 3.25 which was proposed by Ostermayer (1974) for the reduction in bonding between grout-ground interfaces zones with increase in fixed length

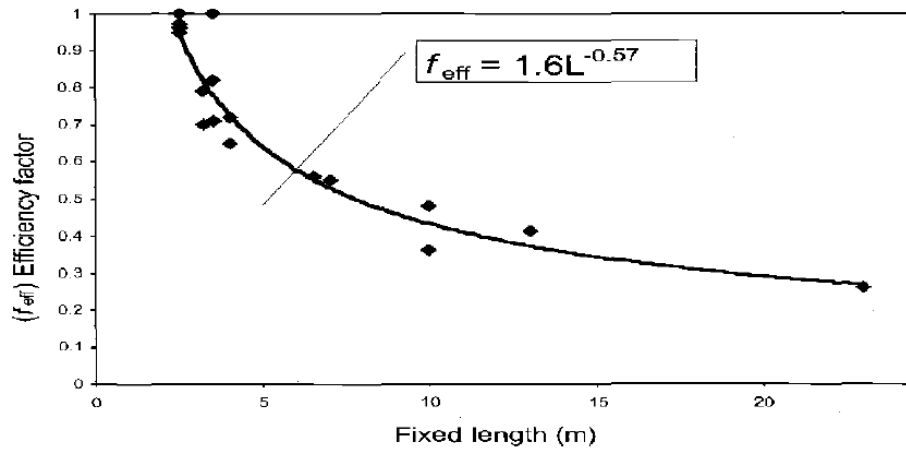


Figure 3.25. Reduction of “anchor efficiency” with increase in fixed length, L .

• “Efficiency factor” f_{eff} evaluated from best fit curve (Barley, 1997).

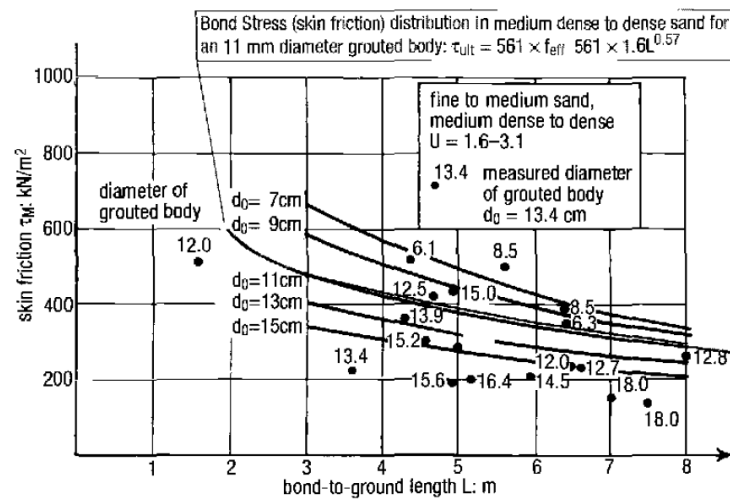


Figure 3.26. Comparison of skin friction to fixed anchor length with the values evaluated by using the efficiency factor (Ostermayer, 1974).

The alternative approach which was proposed by Littlejohn may be seen as follows;

$$T_{ult} = L \times n \times \tan \emptyset \quad (3.3)$$

In Equation 3.3 $\emptyset$ angle of internal friction, is the n a capacity factor which is determined by taking into account of the grouting pressure (3-10 bar), overburden stress

and the diameter of the borehole.

- $n=130-165$ kN/m in fine to medium sands
- $n=400-600$ kN/m coarse sands and gravels (Littlejohn, 1980)

For Ostermayer (1974) however the relationship between T_{ult} and L is not true generally for fixed lengths greater than 5m. The use of efficiency factor in Equation 3.4 may be appropriate;

$$T_{ult} = L n x \tan \phi x f_{eff} \quad (3.4)$$

3.5.3.5. Bearing Capacity in Cohesive Soils. Similar to the non-cohesive soils, load bearing capacity of grouted ground anchors in cohesive soil conditions are not only sensitive to the soil type and the fixed anchor length but also several other factors like grouting type and grouting pressure. Bearing capacity evaluations may be done by two different methods.

3.5.3.6. Diagrammatic Interpretation. As already explained above (Ostermayer, 1974) charts may be used to define the average bond stress value for cohesive soil conditions by using Equation 3.5 and the diagram in Figure 3.27;

$$T_{ult} = \pi \times d \times L \times t_{avg} \quad (3.5)$$

t_{avg} in the equation represents the average bond stress value along the fixed anchor length.

For type A anchors (Chapter 2.3.2), Littlejohn (1980) proposed Equation 3.6 to calculate the shear stress

$$t_{avg} = c_u x \alpha \quad (3.6)$$

where c_u average undrained strength, is the α adhesion factor,

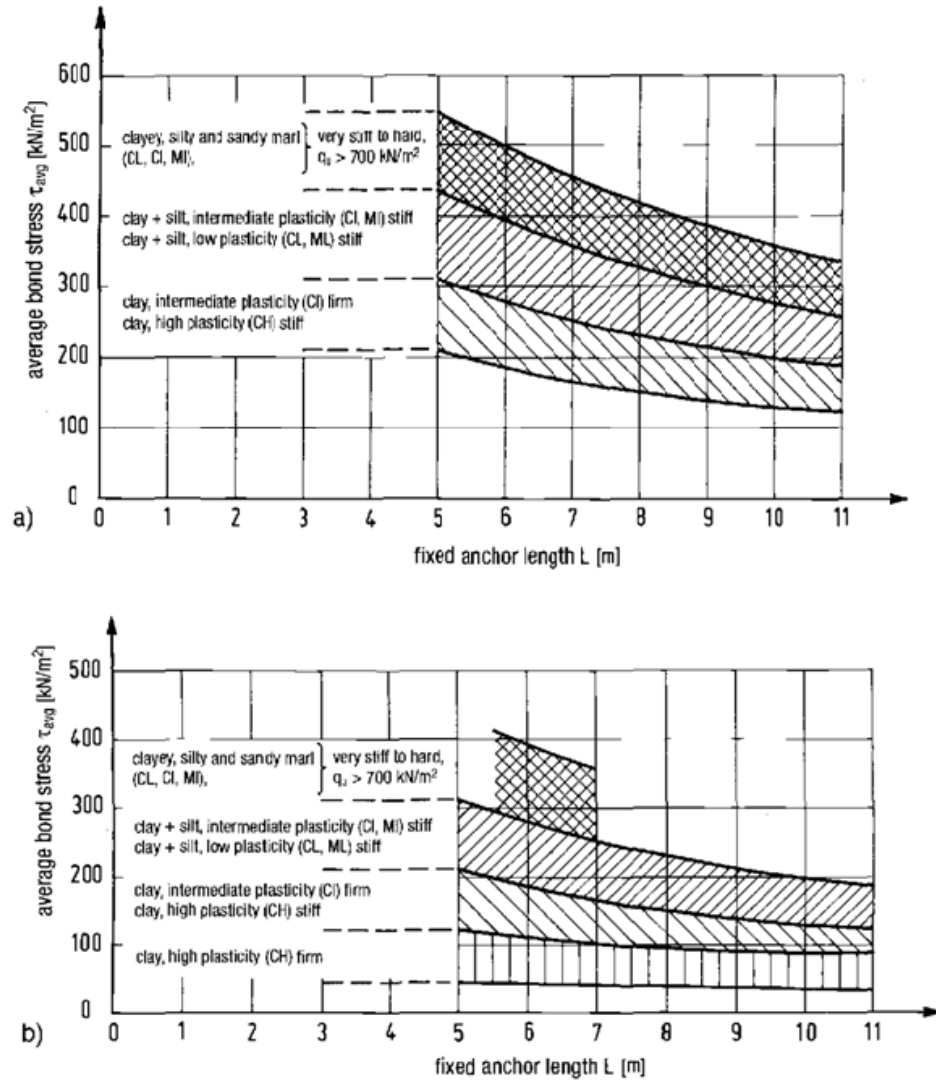


Figure 3.27. Figure Average bond stress at ultimate limit state of anchors in cohesive soils, a) with post-grouting, b) without post-grouting.

For London clay ($c_u > 90 \text{ kN/m}^2$), α value ranges between 0.3-0.35, for stiff to very stiff marls ($c_u = 287 \text{ kN/m}^2$), $\alpha = 0.48-0.6$.

After Littlejohn (1980) and Barley (1997), for firm cohesive soil to weathered rock the following ultimate bond stresses have been established

$$t_{avg} = 0.45 - 0.6xc_u [kN/m^2] \quad (3.7)$$

$$t_{avg} = (10 - 30) x N_{30} [kN/m^2] \quad (3.8)$$

3.5.3.7. Design Formulae Approach. Equation 3.1 is also appropriate for cohesive soils by using the diagrams in Figure 3.28. As it can be seen from the diagram, efficiency factor is identical considering both the cohesive and non-cohesive soil mediums. “The factor consistency is probably a consequence of the variation in elastic modulus of the tendon compared to the elastic modulus of the grouted ground being large, while the variation in the elastic modulus of the grouted ground (whether cohesive or non-cohesive) being small” (Smolczyk, 2002).

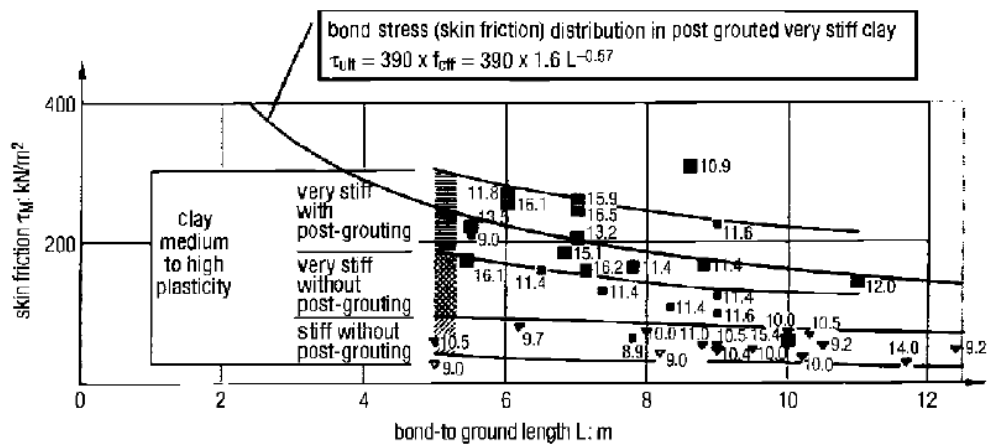


Figure 3.28. Relationship between the grouted anchor length and skin friction for cohesive soils (Ostermayer, 1974).

3.5.4. Factor of Safety

According to Littlejohn (1980), there are 4 types of failure modes which can occur in an anchored structure;

- Failure in the ground mass,
- Failure in the ground/grout bond,
- Failure in the grout/tendon bond,
- Failure in the tendon or anchor head.

Therefore, Littlejohn (1980) pointed out that factor of safety value for each potential failure mode needs to be chosen wisely by taking into account of the public safety consideration, cost of the structure, systems characteristics (temporary or permanent) and soil/rock conditions (harmful environments). Factor of safety values according to Littlejohn (1980) by considering those points mentioned above may be seen in Table 3.3;

Table 3.3. Suggested Safety Factors for anchor design.

Anchor category	Minimum safety factor	Proff load factor
Temporary anchors where the service life is less than 6 months and failure would have few serious consequences and would not endanger public safety e.g. Short term pile test.	1.4	1.1
Temporary anchors with a service life of up to 2 years, where although the consequences of failure are quite serious, there is no danger to public safety without adequate warning e.g. Retaining wall tie backs.	1.6	1.25
All permanent anchors. Temporary anchors in a highly aggressive environment, or where the consequences of failure are serious e.g. temporary anchors for main cables of a suspension bridge or as a reaction for lifting heavy structural members.	2.0	1.5

Factor of safety values for British design code BS 8081 may be seen below in Table 3.4.

Table 3.4. Suggested Safety Factors for anchor design according to BS 8081.

Table.2 Minimum safety factors recommended for design of individual anchorages				
Anchorage category	Minimum safety factor			Proof load factor
	Tendon	Ground/ grout interface	Grout/ tendon or grout/ encapsulation interface	
Temporary anchorages where a service life is less than six months and failure would have no serious consequences and would not endanger public safety, e.g. Short term pile test loading using anchorages as a reaction system.	1.40	2.0	2.0	1.10
Temporary anchorages with a service life of say up to two years where, although the consequences of failure are quite serious, there is no danger to public safety without adequate warning e.g. retaining wall tie-back.	1.60	2.5*	2.5*	1.25
Permanent anchorages and temporary anchorages where corrosion risk is high and/or the consequences of failure are serious, e.g. Main cables of a suspension bridge or as a reaction for lifting heavy structural members.	2.00	3.0	3.0*	1.50
*Minimum value of 2.0 may be used if full scale tests are available.				
May need to be raised to 4.0 to limit ground creep.				
NOTE 1. in current practice the safety factor of an anchorage is the ratio of the ultimate load to design load. Table 2 above defines minimum safety factors at all the major component interfaces of an anchorage system.				
NOTE 2. Minimum safety factors for the ground/grout interface generally lie between 2.5 and 4.0. However, it is permissible to vary these, should full scale field tests (trial anchorage tests) provide sufficient additional information to permit a reduction.				
NOTE 3. The safety factors applied to the ground/grout interface are invariably higher compared with the tendon values, the additional magnitude representing a margin of uncertainty.				

4. NUMERICAL MODELLING IN GEOTECHNICS

4.1. Introduction

4.1.1. History

Past decades have provided us a formidable growth and progress in numerical methods for modelling engineering problems. This progress has been achieved by the pace in the development of the availability of high speed processing computers. This development in mathematics and computer sciences have opened a vital corridor for other engineering disciplines like civil engineering.

The early spotlight of these numerical modelling software's was the structural engineering but geotechnics followed it by wasting no time in producing an industrious sub culture (Peck, 1985) (Woods, 2003). The early users of these numerical modelling softwares were academic researchers. Researchers used numerical modelling softwares to conduct tests by changing critical parameters and validated them by using physical modelling. This extensive works has improved the methods and the softwares which have been used and opened the safe gate of numerical modelling to the industrial users. Nowadays, many numerical modelling software programs have been emerged for various engineering practice and any ordinary designer uses one of those numerical modelling softwares which is flourishing in civil engineering industry. However, this is important for the user (engineer) to have a good insight of the methods and simplifications/assumptions which have been used to solve numerical problems.

4.1.1.1. Numerical Methods. Numerical methods is a mathematical device which uses algorithms and numerical approximations for seeking a solution for scientific and engineering problems by using simplifications of reality. There are various kind of mathematical methods which is implemented by engineers to solve engineering problems. Those methods may be seen in phone tree format in the Figure 4.1.

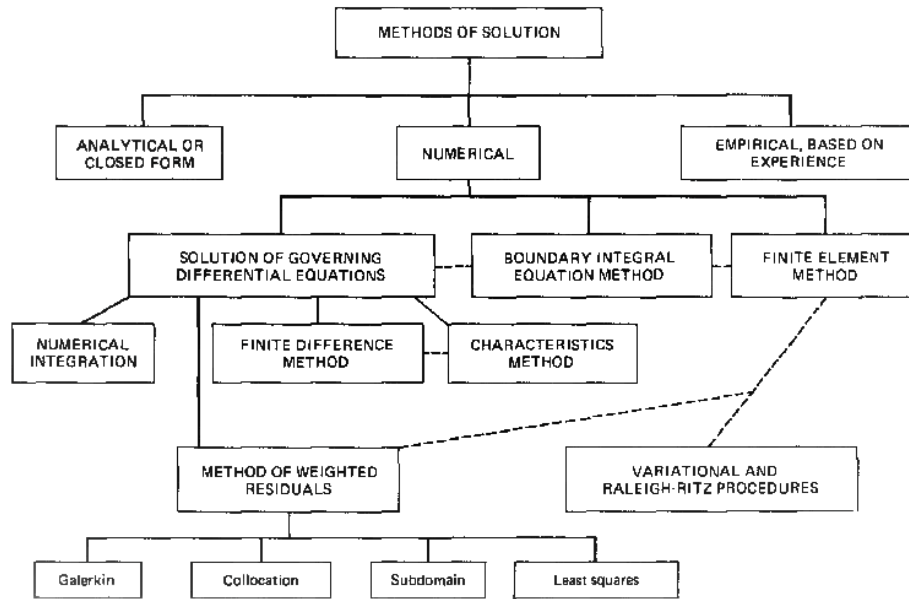


Figure 4.1. Numerical methods (Desai and Christian, 1977).

No aspects of other numerical simulation methods but finite element method will be covered in this thesis, since valuable information is available broadly from many other sources. This research will be focusing on finite element method.

4.2. Finite Element Method

Finite element method is the most universally and extensively used numerical simulation method although it is rather young tool comparing with its alternatives. FEM's root lie in 1950's. In those years, it was a little bit complicated and time consuming to use FEM with punching of cards to a calculating machine because the computer technology was newly emerging. Nowadays, finite element software packages available for every physical discipline in variety of different forms from the open source software's to the most expensive commercial ones. In respect of the area of application, FEM application does not have any limits. Consequently, finite element method may be regarded as an alternative tool to the experiments or tests. (Öchsner and Merkel, 2013). This feature is also a great benefit the other way round, experimental results and tests should be used to calibrate the finite element simulation to have better solutions.

According to Reddy, there are three distinct features which shows the excellence of the FEM to its alternatives such as (Reddy, 2006).

- Geometrically complex domains; a finite element may be seen as a domain and every engineering complex region is a collection and total of sub-domains (sub-regions) which may be seen in Figure 4.2. By using geometrically simple finite elements -subdomains-every complex geometric region turns into rather simple ones by this simplifying approach.
- Over each finite element, algebraic equations of unknown variables of different nodes are developed using the governing equations of the problem,
- Those sub-domains then bring together in acceptable global approximation by using specific internal functions.

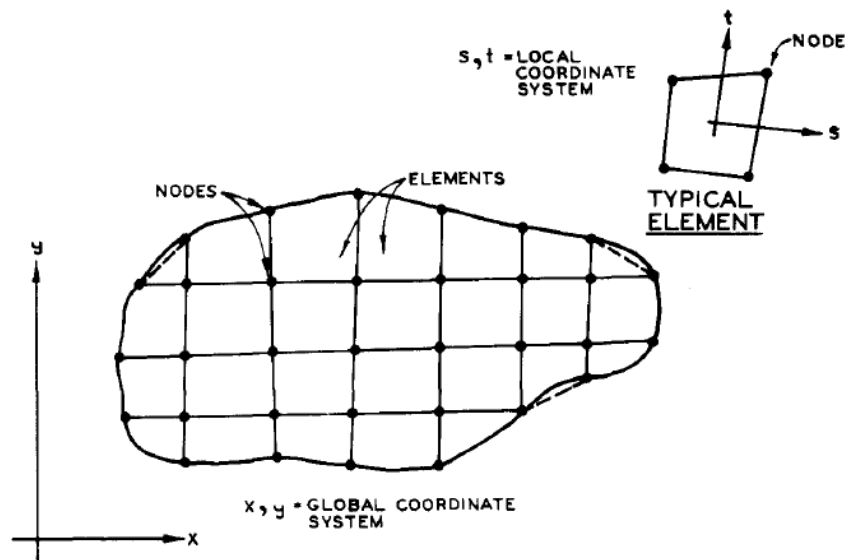


Figure 4.2. Subdivision into finite elements of arbitrary continuum (Desai and Christian, 1977).

The size, geometry, number and type of the nodes and elements may vary by the variation in structural model or the degree of approximation which the user seeks. According to Desai and Christian (1977) there are six important steps which is involved in any and every kind of finite element simulation.

- The discretization: The region-domain is discretized, meaning the domain is divided by subdomains and become an equivalent combination of sub-regions called finite elements. The intersection of the lines which is dividing the subdomains called as nodal points.
- Selection of approximation functions: modeling a pattern of solution for the unknown variables at different nodes over each element by the use of interpolation polynomials generally known as shape functions. By adoption polynomials of higher orders, one can get closer the exact solution.
- Evaluation of element equations; in every kind of aspects when using any type of formulation procedures leads to a matrix formulation which may be expressed as;

$$[K] \{r\} = \{R\} \quad (4.1)$$

where $[K]$ is the property matrix (stiffness matrix, permeability matrix, etc.,) and R is the vector of unknown variables (nodal forces, applied fluid fluxes, etc.,) and r is the nodal vector (for example displacement vector) in Equation 4.1.

- Assembly of elements properties to form global equations; after all equations has been derived, it is time to combine them and form a stiffness relation for the continuum. In this step, if the elements have been formed in local coordinates, it is important to transform them into global coordinate of the system. Also, the boundary conditions for the engineering problem needs to be satisfied in this step too.
- Solution to nodal unknowns; in this step for displacement approach, by using property equations, displacement vector may be calculated.
- Computation of primary and secondary quantities; Primary and secondary quantities for some of the engineering problems may be seen in Table 4.1. This step is the most important one which includes the analysis of the results of the computation.

Table 4.1. Primary unknowns and secondary quantities for various problems (Desai and Christian, 1977).

Problem	Primary unknown	Secondary quantities
Stress deformation, static and dynamic foundations, dams and embankments, slopes, pavements, underground structures	Displacements	Strains, stresses, accelerations, velocities
Seepage and flow	Fluid potentials	Velocities, quantity of flow
Coupled consolidation, liquefaction	Displacements and pore water pressure	Strains, stresses, quantity of flow

4.3. Plaxis 2D

Plaxis 2D is a commercial software which uses the finite element method to model geotechnical engineering projects. “Geotechnical applications require advanced constitutive models for the simulation of the non-linear and time-dependent behaviour of soils. In addition, since soil is multi-phase material, special procedures are required to deal with hydrostatic and non-hydrostatic pore pressures in the soil. Although the modelling of the soil itself is an important issue, many geotechnical engineering projects involve the modelling of structures and the interaction between the structures and the soil. PLAXIS software is equipped with special features to deal with the numerous aspects of complex geotechnical structures” (Brinkgrieve, 2002).

4.3.1. Model

There are two options to model the geometry of a geo-engineering structure in Plaxis 2D; plane strain or Axisymmetric which may be seen below. Each of those models has its own unique cases of implementation

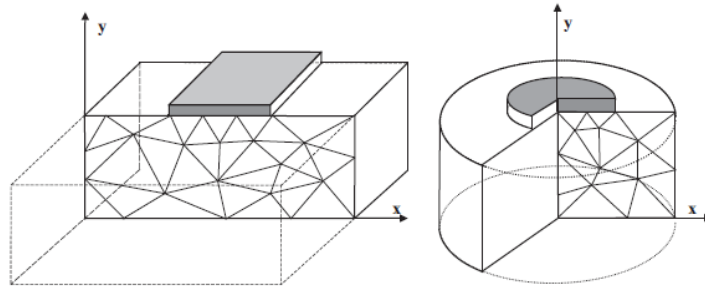


Figure 4.3. Example of Plane Strain (1) and Axisymmetric (2) in PLAXIS 2D
(modified from PLAXIS, 2016).

4.3.1.1. Plane Strain Model. Plane strain model which may be seen in Figure 4.3 is useful for problems when one dimension of the structure is very large comparing with the other two. It is assumed that the displacements and strains in the z-direction are zero, but the normal stresses are fully accounted for (Plaxis 2D, 2016a). Examples of some of the practical applications are tunnels, retaining walls, soil nailed and/or anchored walls, etc...

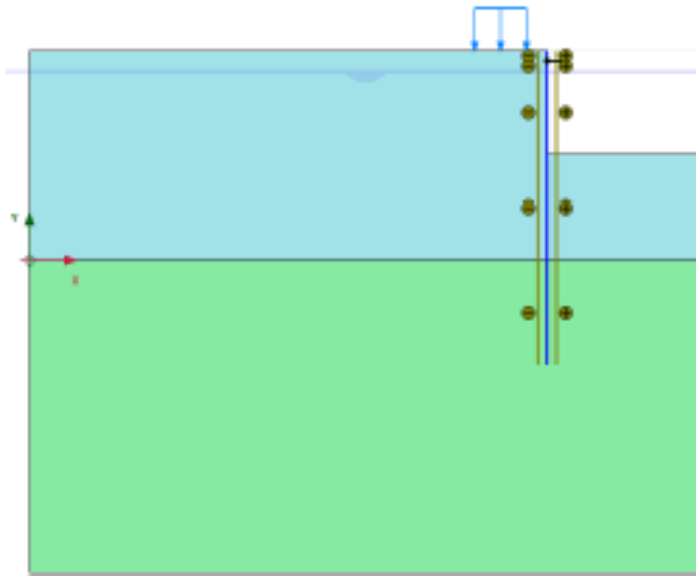


Figure 4.4. Representation of a plane strain model (Plaxis 2D, 2016).

4.3.1.2. Axisymmetric Model. Axisymmetric model allows the user to simulate circular structures with uniform cross-sections where deformation and stress state is ac-

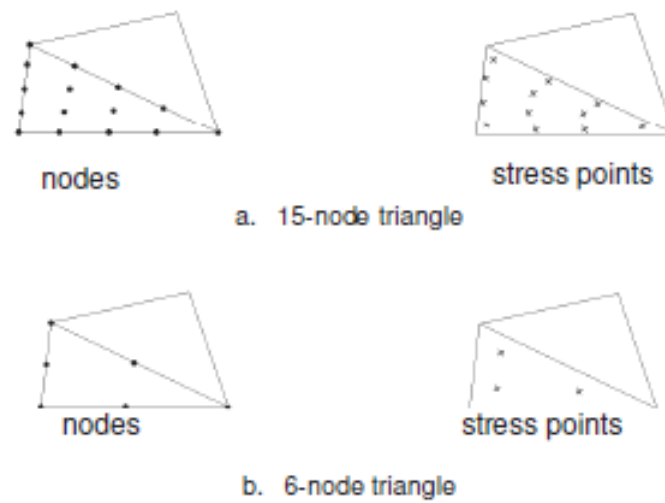


Figure 4.7. Position of nodes and stress point in elements (Plaxis 2D, 2016a).

4.3.2.1. 15 node elements. 15 node elements involves 12 stress points and it provides fourth order interpolation for displacements (Plaxis 2D, 2016a). This leads to very high quality results, however; the calculation process is very time consuming and uses great amount of computer memory. Therefore, there is another basic element type.

4.3.2.2. 6 node elements. 6 node elements have 3 stress points and it provides second order interpolation for displacements (Plaxis 2D, 2016a). This element type is less time consuming and leads to fairly good results for standard deformation analysis. Because of the time constraints, 6 node elements are chosen for the implementation of the numerical models in this present thesis.

4.3.3. Geometry Modes

The geometry of the project is defined in geometry modes namely, soil mode and structural mode. The production of the model is possible only in geometry modes.

4.3.3.1. Soil. Soil is the mode in which user defines the soil layers and stratigraphy by using boreholes.

4.3.3.2. Structure. The geometric entities and structural elements, forces and prescribed displacements can only be defined in structure mode. Geometry of the project consists of the following (Plaxis 2D, 2016a);

- Points,
- Lines,
- Polygons,
- Soil Polygons.

The Structural elements which can be defined in geometry mode of Plaxis 2D is listed below (Plaxis 2D, 2016a);

- Soil,
- Borehole,
- Embedded beam rows,
- Plates,
- Geogrids,
- Tunnels,
- Loads,
- Prescribed Displacements,
- Interfaces,
- Node to node anchors,
- Fixed-end Anchors,
- Groundwater Flow BC's,
- Thermal Flow BC's,
- Wells,
- Drains,
- Connections.

4.3.4. Boundary Conditions

The boundary conditions at the extremities of the model can be conveniently specified in the model conditions sub-tree in the model explorer (Plaxis 2D, 2016a). It is important to place the boundary of the model to a sufficient distance for not to affect outputs.

4.3.5. Fixities

Fixities are the zero deformation locations. There are 4 types of fixities in Plaxis 2D;

- Vertically fixed; Displacements in y direction is zero,
- Horizontally fixed; displacements in x direction is zero,
- Fully Fixed; Displacements in x and y direction is zero,
- Free; No fixities in any direction.

By default, the bottom boundary of the model is fully fixed, and x-min and x-max of the boundary is normally (horizontally) fixed and the upper boundary is free in Plaxis 2D (Plaxis 2D, 2016a).

4.3.6. Prescribed Displacements

Prescribed displacements are special conditions that can be imposed to the model to control the displacements at certain locations (Plaxis 2D, 2016a). Prescribed displacements creates reaction forces against the pres. displacements in x or y direction. Prescribed displacements can be assigned as point prescribed displacements and distributed prescribed displacements.

Prescribed displacements should be interpreted as specified total displacements at the end of the phase instead of additional phase displacements (Plaxis 2D, 2016a).

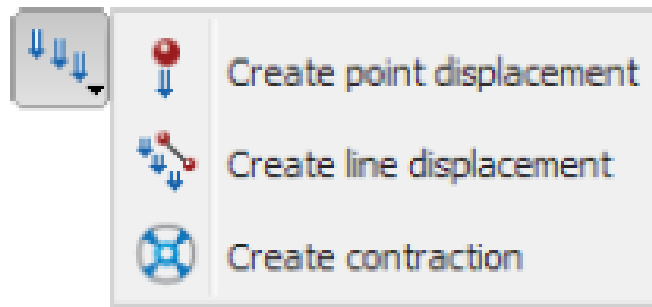


Figure 4.8. Create prescribed displacement menu.

4.3.7. Mesh Generation

A composition of finite elements (sub-regions) called as mesh (Plaxis 2D, 2016a). It is a network topology. Generation of the meshes is the total triangulation of the project. Plaxis 2D allows the user to generate fully automated mesh such as very fine, fine, medium, coarse and very coarse. However, these automatically generated meshes may not lead very accurate and sufficient results, therefore Plaxis recommends that the mesh should be refined in parts that the user expects the deformations to occur (Plaxis 2D, 2016a).

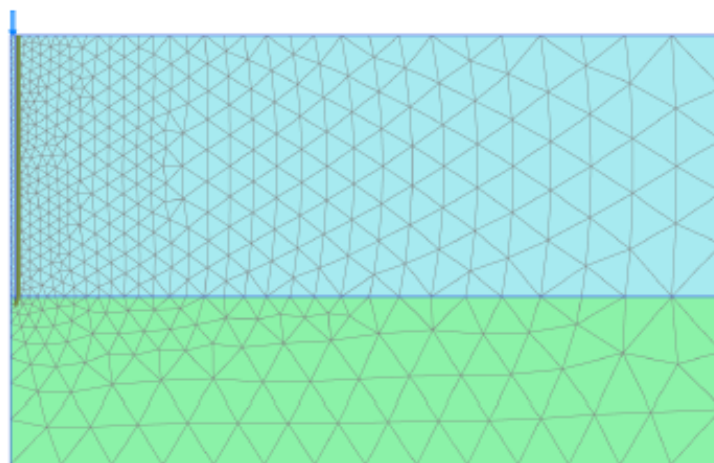


Figure 4.9. Generated mesh (Plaxis 2D, 2016b).

4.3.8. Material Models

Plaxis 2D instores variety kind of models which represent different behaviours with varied level of sophistication in its library. In this present thesis only Mohr Coulomb (MC), Linear Elastic (LE) and Hardening Soil (HS) material models have been used. Therefore those material models will be explained briefly below. However, other material models may be reviewed by the reader from the Plaxis Material Models Manual (2016).

4.3.8.1. Linear Elastic (LE) Material Model. The Linear Elastic model is described by Hooke's law of isotropic linear elasticity. It is important for the user to remember to use LE model to simulate stiff-strong structures such as concrete in walls or grout body etc... because it is too limited to simulate essential soil behaviour (Plaxis 2D, 2016c). There is no limit in stress-strain behaviour in LE model means it shows infinite strength (Plaxis 2D, 2016c). This material model is implemented to simulate GFRP and Steel Tendon behaviour in this present thesis.

Property	Unit	Value
Stiffness		
E'	kN/m ²	0.000
ν' (ν)		0.000
Alternatives		
G	kN/m ²	0.000
E_{oe}	kN/m ²	0.000
Velocities		
V_s	m/s	0.000
V_p	m/s	0.000
Advanced		

Figure 4.10. Material Tab-sheet for LE Model (Plaxis 2D, 2016c).

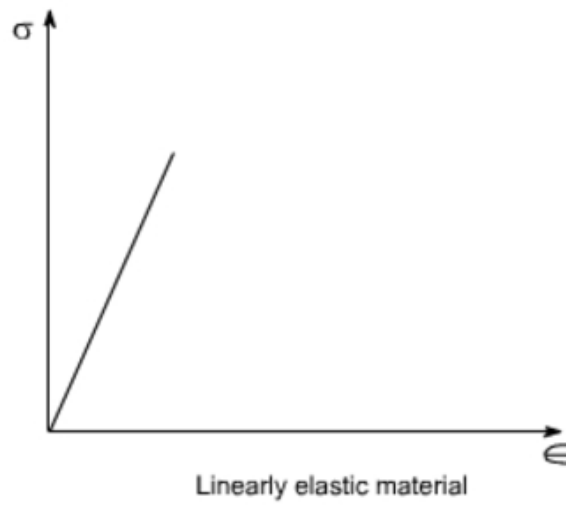


Figure 4.11. Stress-strain behaviour of LE Model.

4.3.8.2. Mohr-Coulomb (MC) Material Model. It is widely known and popular model which follows the linear elastic-perfectly plastic material behaviour. The linear elastic part of the behaviour follows Hooke's law like linear elastic model and perfectly plastic part of the MC model follows the Mohr-Coulomb failure criterion (Plaxis 2D, 2016c).

In MC model, yield surface does not affected by plastic strains means fully characterized by model parameters (Plaxis 2D, 2016c). The linear elastic-perfectly plastic material behaviour can be seen in Figure 4.12. This material model is implemented to simulate grout-concrete behaviour in this present thesis.

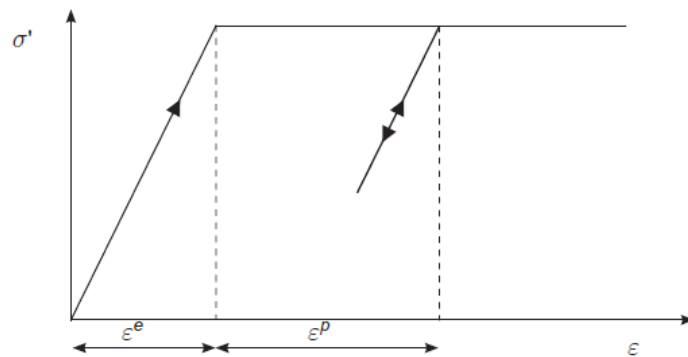


Figure 4.12. Stress-Strain behaviour of MC Model (Plaxis 2D, 2016c).

Property	Unit	Value
Stiffness		
E'	kg/m ²	0.000
ν' (nu)		0.000
Alternatives		
G	kg/m ²	0.000
E_{oed}	kg/m ²	0.000
Strength		
c_{ref}	kg/m ²	0.000
ϕ' (phi)	°	0.000
ψ (psi)	°	0.000
Velocities		
v_s	m/s	0.000
v_p	m/s	0.000

Figure 4.13. Material Tab-sheet for MC Model (Plaxis 2D, 2016c).

4.3.8.3. Hardening Soil (HS) Material Model. Hardening soil material model shows a sophisticated-complex material behaviour. Unlike to MC model, the yield surface is not fixed but it can accumulate by the increase in stiffness (Plaxis 2D, 2016c). The basic notion behind the Hardening soil model may be seen below in Figure. HS model differs from other two models by including these features;

- Using Theory of Plasticity rather than Elasticity,
- Consists of Soil dilatancy,
- Introducing a yield cap (Plaxis 2D, 2016c).

The basic feature of the HS model is the stress dependency of soil stiffness (Plaxis 2D, 2016c). Hardening soil model contains following material properties

- Stress dependent stiffness according to the power law (m),
- Plastic straining due to primary deviatoric loading (E_{50}^{ref}),
- Plastic straining due to primary compression (E_{oed}^{ref}),
- Elastic unloading and reloading (E_{ur}^{ref})

This material model is implemented to simulate soil behaviour in this present thesis.

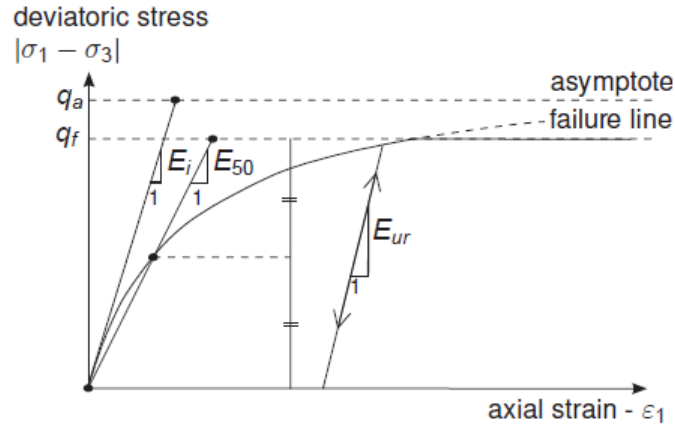


Figure 4.14. Stress-Strain behaviour of HS Model (Plaxis 2D, 2016c).

4.3.9. Drainage Types

Plaxis 2D allows the user to choose 3 different drainage types namely undrained A, undrained B and drained. Since there is no need for excess pore pressure in analysis and considering the similar simulations (Ann *et al.*, 2004b, Watzlik, 2015). Drainage type in this present simulations are taken as drained.

4.3.10. Types of Analysis

Plaxis 2D supplies users with the variety kind of analysis types which may be seen below;

- The Initial Step which includes; K0 procedure, Field Stress and Gravity loading,
- Plastic calculation step,
- Consolidation calculation step,
- Safety Calculation step and,
- Dynamic calculation step.

4.3.10.1. Plastic Analysis. Plastic calculation in Plaxis 2D is utilized to implement an elastic-plastic deformation analysis in which it is not considered necessary to allow for the modification of pore pressure with time (Plaxis 2D, 2016c). Plastic analysis option was selected for this present research.

5. CALCULATION MODEL AND ANALYSIS PROCEDURE

This Chapter illustrates the assumptions, geometries and material parameters in detail for ground anchorage pull-out models.

5.1. Introduction

Accurate results, comparisons and measurements may be obtained by conducting physical tests on any area of research. However, those physical tests implemented by changing parameters which influences the outputs that a researcher tries to investigate are time consuming and costly in any other way. Numerical simulations on the other hand are easy to conduct and fast and handy to have the results if they are conducted accurately. As explained in Chapter 4, by the great developments in computer sciences, numerical modelling, particularly FEM have been broadly used to model any physical test nowadays. Therefore for novice and expert researchers, it is easier to conduct numerical analysis first, reach some results and conduct physical tests later to validate those results for a long time.

The Finite element method has been well conducted in geotechnical area of research also for decades as it has the advantages of modelling the stress-strain behaviour and distribution and failure characteristics of any civil engineering, especially geotechnical structure (soil-structure interaction). Since, there are specifically substantial researches which have been conducted by numerical simulations and some physical tests (for validation) in the subject of the pull out capacity of soil nails and ground anchors (Ann at all, 2004b, Watzlik 2015), a numerical simulations for pull-out capacity of ground anchorages, based upon validated and well conducted FEM models are carried out and will be explained in this Chapter. In this present research, Numerical simulations, particularly finite element modelling of pull-out behaviour of GFRP and steel ground anchors are simulated by using Plaxis 2D 2016.01, highly popular and powerful

FE software. 2D and axisymmetric finite element model is set for pull-out test and it will be recommended in further studies Chapter to perform physical tests to validate simulations onwards.

5.2. Simplifications and Material Parameters

In order to implement FE model, some basic assumptions are made which will be explained in details in this section. Those assumptions are needed to be made to simplify the simulation models since eighteen different models are conducted to assess the effects of the several essential factors on the pull-out behaviour of the ground anchors; they are (1) length of grouted zone, (2) type of tendon (GFRP and Steel) and (3) type of surrounding soil. A researcher who surveys this present thesis, should acknowledge that it would have been better if those analyses are done by implementing 3D FE models. Since time limitation and multiplicity of FE models (eighteen models), 2D axisymmetric FE-models are conducted by simplifications instead of 3D FE models.

5.2.1. Geometries and Simplifications

As mentioned previously, there are a number of FE models which are conducted by several other researchers (Ann *et al.*, 2004b, Watzlik 2015) to simulate the pull-out behaviour of soil nails, piles and ground anchors. It is admissible that 2D axisymmetric model has been used in the majority of those models. In broad range of FE modelling softwares, axis of symmetry is the vertical axis however, the simulation of ground anchorage pull-out test model which can be seen below is 15o inclined with horizontal (y) axis. Therefore, the basic simplifications have been made. Those simplifications which have been made in this present models are very similar with the study conducted by Watzlik (2015) and Ann *et al.*, (2004b).

- Tendon of the Ground anchor is modelled by LE material (both for conventional steel and GFRP),
- Grout is modelled by Mohr-Coulomb material (Watzlik, 2015). The parameters and their determination procedure will be discussed ahead.

- Soil bodies are modelled by HS material to see progressive failure mechanism at the grout-ground shear zone.
- Grout-ground shear zone is conducted by adding manually 2cm wide soil cluster with very fine mesh dividing the borders of grout and ground to the model.
- Pull-out mechanism is modelled by prescribed displacements mechanism (Watzlik, 2015) in Plaxis and will be discussed later on.
- The material in front of the grout body is removed to simulate unbounded length (Ann et al, 2004b; Watzlik, 2015).
- The Horizontal fixity is applied to the area of removed soil in unbounded length (Watzlik, 2015).

5.2.1.1. Model Simulation. The FE-model simulations in Plaxis is modelled by taking the advantage of axisymmetric modelling for doing that horizontal inclined grouted ground anchor model is converted to vertical axis. To fully represent the stress state in initial conditions, ground anchor with no overburden in anchor head is chosen.

The procedure is converting radial stresses in horizontal direction to vertical axis so the unit weight and K_o values are modified for all soil mediums respectively. In Plaxis, K_o value is calculated by the formula which is

$$K_o = 1 - \sin(\phi) \text{ for normally consolidated clays} \quad (5.1)$$

$$K_o = 1 - \sin(\phi) OCR^{-1/2} \text{ for over consolidated clays} \quad (5.2)$$

and in default Plaxis uses the Equation 5.1. However, the K_o values for the geo mediums are taken as 1 to produce no stress difference between horizontal and vertical axis. The conversion of the geometry may be seen below in Figure 5.1 and Figure 5.2.

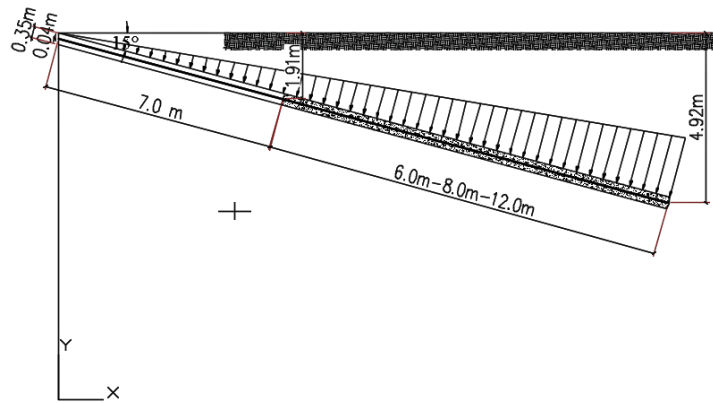


Figure 5.1. Actual Grouted ground anchor model (inclined to the horizontal axis).

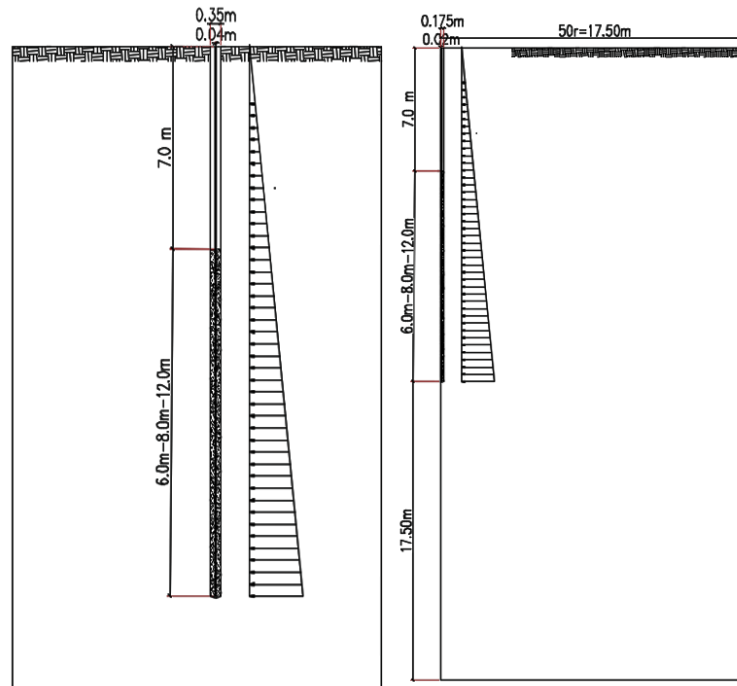


Figure 5.2. Vertical model (on the left) converted to axisymmetric model (on the right) which is implemented in Plaxis for grouted ground anchor with 6m, 8m, 12m fixed length (Ann *et al.*, 2004b; Watzlik, 2015).

As explained above, the conversion of the geometry and the conversion of the horizontal stress state to vertical stress state are done by the conversion of the unit weight from horizontal to the vertical stress state (Watzlik, 2015). The results of the conversion are listed in Table 5.1 and Table 5.2.

Table 5.1. Parameters and Results of the Initial Stress State of Stiff Clay and Sand (Watzlik, 2015).

Free Length (Tendon Length)	:	7.00	m
Fixed Length (Grouted Length)	:	12.00	m
Actual Unit Weight	:	18.50	kN/m ²
Inclination	:	15.0	°
The overburden at the anchor head	:	0.00	m
The overburden at the distal end of the grout body	:	4.92	m
The overburden at the end of the free length	:	1.81	m
Input Unit Weight	:	4.79	kN/m ²

Table 5.2. Parameters and Results of the Initial Stress State of Rock (Watzlik, 2015).

Free Length (Tendon Length)	:	7.00	m
Fixed Length (Grouted Length)	:	12.00	m
Actual Unit Weight	:	24.00	Kn/M ²
Inclination	:	15.0	°
The overburden at the anchor head	:	0.00	m
The overburden at the distal end of the grout body	:	4.92	m
The overburden at the end of the free length	:	1.81	m
Input Unit Weight	:	6.21	Kn/M ²

5.2.2. Material Parameters

As mentioned in Chapter four, there are three different types of soil models that are used on the simulation of the ground anchor pull-out behaviour models which are;

- LE (Linear Elastic) Model which is used to model the tendon (GFRP and Steel) behaviour of the ground anchor,
- MC (Mohr-Coulomb) Model which is used to model the grout behaviour in the fixed length of the ground anchor,
- HS (Hardening Soil) Model which is used to model the behaviour of the soil bodies (Over consolidated Clay, Dense Sand and Rock) around the ground anchor.

5.2.2.1. Tendon. The primary concern of this thesis is the use of GFRP bars (Geobars) as a tendon part of the ground anchor which may create great benefit in permanent anchor types and/or in very corrosive and harmful environmental conditions or in maritime projects. In order to distinguish the behavioural differences those can create in grout-ground shear band behaviour and load displacement behaviour of ground anchors; GFRP bar and Conventional Steel bar parameters are chosen from a research and may be seen in Table 5.3, Table 5.4 and Table 5.5 respectively.

Table 5.3. Comparative list of properties of fiberglass composites, carbon fiber composites, steel A572, and concrete (Maglev, 2017).

Property	GFRP	CFRP AS4/3501-6	A572 Gr. 50 Steel	Concrete
Density [g/cc]	2.076	1.580	7.860	2.400
Longitudinal Modulus E_1 [GPa]	45	142	200	32
Transverse Modulus E_2 [GPa]	12	10.3	200	32
In-plane Shear Modulus G_{12} [GPa]	5.5	7.2	75	13
Poisson's Ratio ν_2	0.19	0.27	0.33	0.19
Longitudinal Tensile Strength F_{1t} [MPa]	1020	1830	345	5
Transverse Tensile Strength F_{2t} [MPa]	40	57	345	5
Longitudinal Compressive Strength F_{1c} [MPa]	620	1096	345	40
Transverse Compressive Strength F_{2c} [MPa]	140	228	345	40
Coefficient of Thermal Expansion	10	-0.9	12	10

Table 5.4. Material parameters of the Steel Bar Tendon.

Metarial Model	:	Lineer Elastic
Metarial Name and Type	:	A 572 Gr. 50 Steel
Youngs Modulus	:	200 Gpa
Poissons Ratio	:	0.33
Density	:	78.6 kPa
Tensile Strength	:	345 MPa

Table 5.5. Material parameters of the GFRP Bar Tendon.

Metarial Model	:	Lineer Elastic
Metarial Name and Type	:	GFRP
Youngs Modulus	:	45 Gpa
Poissons Ratio	:	0.19
Density	:	20 kPa
Tensile Strength	:	1020 MPa

5.2.2.2. Grout. The Grout is modelled by using Mohr Coulomb material model. The material parameters which is chosen for grout body may be seen in Table 5.6. The cohesion is determined using the friction angle ($\emptyset$) and the uniaxial compressive strength UCS (Watzlik, 2015).

Table 5.6. Material parameters of the Grout Body (Watzlik, 2015).

Metarial Model	:	Mohr Coulomb
Metarial Name and Type	:	C25 (Grout Body)
Youngs Modulus	:	30 Gpa
Poissons Ratio	:	0.2
Density	:	25 kPa
Uniaxial Compressive Strength (UCS)	:	25 MPa
Cohesion, c'_{ref}	:	5829 Kn/m ²
Friction Angle $\emptyset$	:	40
Dilatancy Angle $\hat{\omega}$	:	0

5.2.2.3. Soil Body. In order to investigate the various conditions and their effects on the behaviour of ground anchors with GFRP and Steel bars and then to compare them, three different soil types are used in pull-out simulation of ground anchors, which of these are;

- Over consolidated Clay; this soil type represents the stiff clay behaviour with a lower young's modulus and stiffness parameters. Hardening soil material parameters of this soil type is taken from the research of Watzlik (2015) and may be seen in Table 5.7;

Table 5.7. Hardening Soil Material Parameters of Over Consolidated Clay (Watzlik, 2015).

Metarial Model	:	Hardening Soil
Behavior Type	:	Drained
Metarial Name and Type	:	Over Consolidated Clay
Secant Modulus at reference pressure E_{50}^{ref}	:	11860 kPa
Reference Youngs Modulus for Unloading Reloading E_{50}^{ref}	:	36170 kPa
Tangential Modulus from Oedometer test at ref. pressure E_{50}^{ref}	:	11860 kPa
Poissons Ratio	:	0.2
Unit Weight, kN/m ³	:	4.79 kPa
ref pressure	:	100 kN/m
Cohesion, c'_{ref}	:	6 kN/m
Friction Angle, $\emptyset$	:	20
Dilatancy Angle, $\hat{\omega}$	:	-
Earth Pressure at Rest, K_o	:	1

- Dense Sand: This soil type represents the medium stiff soil behaviour with average level stiffness parameters. Generally accepted soil parameters are used as the Hardening Soil material parameters for this soil type and may be seen in Table 5.8;
- Rock: This soil type represents the hardly brittle behaviour of rock material with high level stiffness parameters. Generally accepted soil parameters are used as the Hardening Soil material parameters for this soil type and may be seen in Table 5.9;

Those soil parameters are used for each particular anchor with differing fixed length and tendon type (GFRP and Steel) and also with combination of each other.

Table 5.8. Hardening Soil Material Parameters of Dense Sand.

Metarial Model	:	Hardening Soil
Behavior Type	:	Drained
Metarial Name and Type	:	Dense Sand
Secant Modulus at reference pressure E_{50}^{ref}	:	50000 kPa
Reference Youngs Modulus for Unloading Reloading E_{50}^{ref}	:	50000 kPa
Tangential Modulus from Oedometer test at ref. pressure E_{50}^{ref}	:	150000 kPa
Poissons Ratio	:	0.3
Unit Weight, kN/m^3	:	4.79 kPa
ref pressure	:	100 kN/m
Cohesion, c'_{ref}	:	1 kN/m
Friction Angle, ϕ	:	35
Dilatancy Angle, $\bar{\omega}$	:	
Earth Pressure at Rest, K_o	:	1

Table 5.9. Hardening Soil Material Parameters of Rock.

Metarial Model	:	Hardening Soil
Behavior Type	:	Drained
Metarial Name and Type	:	Rock
Secant Modulus at reference pressure E_{50}^{ref}	:	300000 kPa
Reference Youngs Modulus for Unloading Reloading E_{50}^{ref}	:	300000 kPa
Tangential Modulus from Oedometer test at ref. pressure E_{50}^{ref}	:	900000 kPa
Poissons Ratio	:	0.3
Unit Weight. kN/m^3	:	6.21 kPa
ref pressure	:	100 kN/m
Cohesion, c'_{ref}	:	30 kN/m
Friction Angle, ϕ	:	40
Dilatancy Angle, $\bar{\omega}$	:	
Earth Pressure at Rest, K_o	:	1

5.2.3. Simulated Models

In order to distinguish the effects of several factors in the pull-out behaviour of ground anchors, three different geometries, are modelled by changing geo-medium conditions and tendon (bar) types. By concerning these, six models derived from each of these three geometries. Simulated models may be seen below in Chapter 5.2.3.1, 5.2.3.2 and 5.2.3.3 respectively.

5.2.3.1. Ground Anchor with 6m Fixed Length. In order not to effect stress distribution and deformations, the horizontal boundary is placed at $50r$ distance (Randolph and Wroth, 1978) also upper vertical boundary is $20r$ away from the head of the fixed length, although the function of this region is to simulate free length, it is also worth to mention that according to Randolph and Wroth; this zone eliminates the boundary confinement effect. Also, a region with a 2cm width with very fine mesh is added to the model to simulate the grout-ground shear zone. The simulated model may be seen in Figure 5.3.

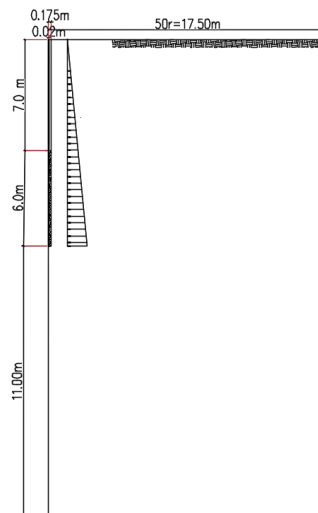


Figure 5.3. The model geometries of ground anchor with 6m fixed length.

5.2.3.2. Ground Anchor with 8m Fixed Length. The same conditions with the model above is ensured in this model too. The simulated model may be seen in Figure 5.4.

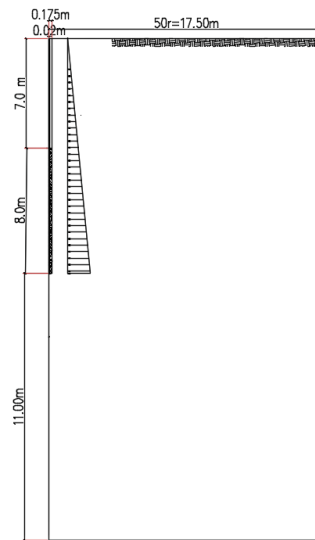


Figure 5.4. The model geometries of ground anchor with 8m fixed length.

5.2.3.3. Ground Anchor with 12m Fixed Length. The same conditions with the model above is ensured in this model too. The simulated model may be seen in Figure 5.5.

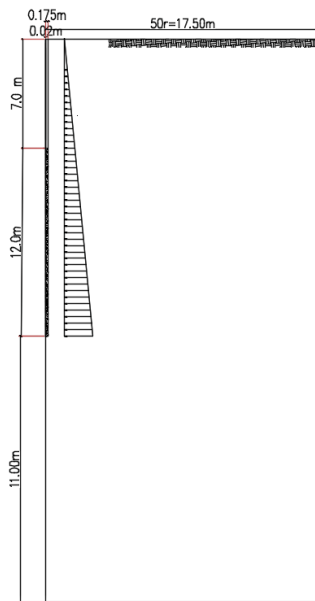


Figure 5.5. The model geometries of ground anchor with 8m fixed length.

5.2.4. Points and Cross-Sections

Several points and cross-sections are selected to draw graphs and monitor the difference in failure mechanisms of analyzed model. Those points and cross-sections

may be seen respectively;

5.2.4.1. Points. Three points are selected to observe failure mechanism related with the changing (tendon) bar material of the grouted ground anchor. Those points are coded as K, M and N and may be seen in Figure 5.6, Figure 5.7 and Figure 5.8;

- Point K presents the behaviour of the shear zone in the top of the grouted length.
- Point M presents the behaviour of the shear zone in the median of the grouted length.
- Point N presents the behaviour of the shear zone in the end of the grouted length.
- Point A is also selected to see the deformation on the top of the grouted zone.

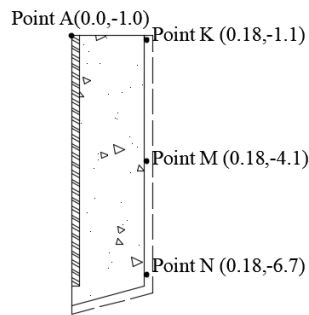


Figure 5.6. Selected points for 6m fixed length ground anchor model (10 times scaled in y direction).

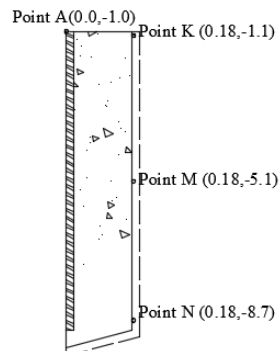


Figure 5.7. Selected points for 8m fixed length ground anchor model (10 times scaled in y direction).

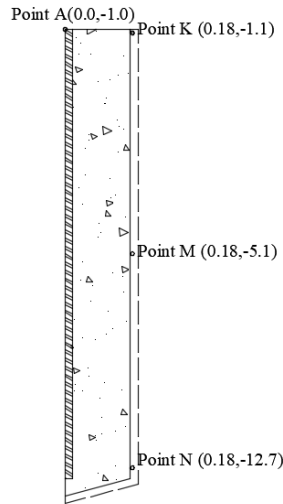


Figure 5.8. Selected points for 12m fixed length ground anchor model (10 times scaled in y direction).

5.2.4.2. Cross-Sections. Two different cross-sections are also selected to observe failure mechanism and to fully acknowledge the displacement behaviour of the surrounding ground by the change in the bar and soil type and length of the grouted zone of the ground anchor models. Those Cross-sections may be seen in Figure 5.9, Figure 5.10 and Figure 5.11;

- A-A'; This cross-section is selected to observe the failure behaviour in the shear zone,
- B-B'; This cross-section is selected to monitor the deformation characteristic of the surrounding soil type.

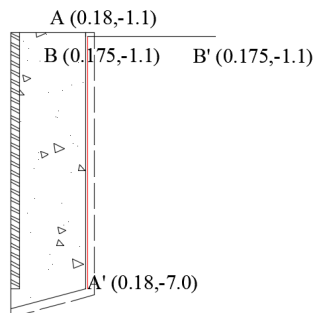


Figure 5.9. Selected cross-sections for 6m fixed length ground anchor model (10 times scaled in y direction).

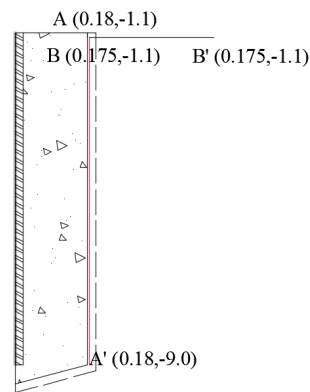


Figure 5.10. Selected cross-sections for 8m fixed length ground anchor model (10 times scaled in y direction).

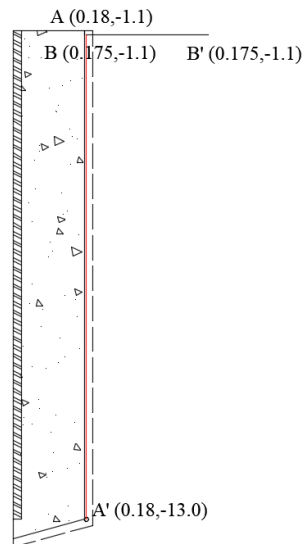


Figure 5.11. Selected cross-sections for 12m fixed length ground anchor model (10 times scaled in y direction).

5.3. Analysis Procedure

The procedure to model pull-out test in Plaxis 2D involves four main steps which can be seen below in order.

5.3.1. Step 1

This first step is involving to creating the geometry of the model and assigning the material properties to it.

- Dimension of the boundry were selected.
- Material parameters are defined.
- Ground anchor, fixities, and prescribed displacements are inserted on the simulated model,
- 2cm thick cluster which will be modeled as a material, is created to be used as grout-ground shear zone (Watzlik, 2015).

5.3.2. Step 2

This step includes mesh generation. In order to save calculation time, global coarseness set to fine but additional refinements are utilized on and around the ground anchor and 2cm thick zone which can be seen in Figure 5.12 as a sample.

5.3.3. Step 3

This step involves the generation of the calculation stages. The pull-out simulation model here includes four base stages.

Initial stage; Generation of the Initial stresses by using K_o procedure. $K_o = 1$ for all the cases as explained in Chapter 5.2.1.2.

Stage 1: Assignment of tendon and grout properties to their regions also deactivating the free length region and activation of the horizontal fixities on the border of the free length.

Stage 2: Nil step is conducted in this stage to reduce the unbalanced forces and deformations (Plaxis, 2016).

Stage 3: First step which represents the beginning of the pull-out test. A prescribed displacement which is 0.4 cm is activated in this first step and several more cumulative 0.4 cm prescribed displacements are activated in the subordinate steps. In each of these steps, 0.4 cm prescribed displacements are implemented on the top of the fixed length to model the pull-out effect on the ground anchor 0.4 cm. Several steps with 0.4 cm prescribed displacements are modelled until failure occurs.

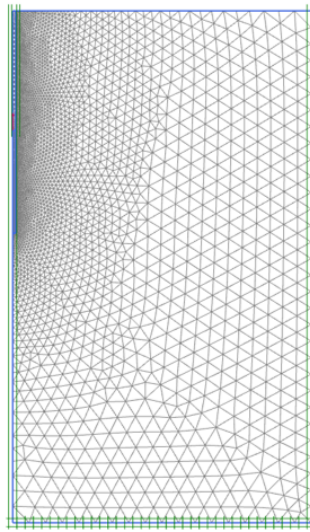


Figure 5.12. Sample of a generated mesh (Plaxis 2D 2016.01).

5.3.4. Step 4

Final step is the calculation step, plastic calculation type is chosen with and the numerical iteration control parameters (Watzlik, 2015) which can be seen in Table 5.10.

Table 5.10. Iteration Control Parameters (Watzlik, 2015).

Maximum Number of Iterations	:	100
Desired Min. Number of Iterations	:	5
Desired Max. Number of Iterations	:	55
Max. Load fraction per step	:	0.1
Tolerance	:	0.01
Arc-Length Control Type	:	Off
Max Steps	:	10000

6. RESULTS AND DISCUSSIONS

In this Chapter, the numerical results for the Plaxis 2D simulations of the pull-out model of ground anchors with varying fixed lengths and soil conditions for two different bar types (GFRP and Steel) were presented. Sub-Chapters will be briefly described respectively below;

- Over Consolidated Clay
 - 6m fixed length; the model will be solved by changing the tendon with GFRP bar and Steel bar,
 - 8m fixed length; the model will be solved by changing the tendon with GFRP bar and Steel bar,
 - 12m fixed length; the model will be solved by changing the tendon with GFRP bar and Steel bar.
- Dense Sand
 - 6m fixed length; the model will be solved by changing the tendon with GFRP bar and Steel bar,
 - 8m fixed length; the model will be solved by changing the tendon with GFRP bar and Steel bar,
 - 12m fixed length; the model will be solved by changing the tendon with GFRP bar and Steel bar.
- Rock
 - 6m fixed length; the model will be solved by changing the tendon with GFRP bar and Steel bar,
 - 8m fixed length; the model will be solved by changing the tendon with GFRP bar and Steel bar,

- 12m fixed length; the model will be solved by changing the tendon with GFRP bar and Steel bar.
- Results are presented with output plots from Plaxis;
- relative shear and
- plastic deformations output
- Besides, graphs will be presented which is related to
- pull-out load-displacements (at the top-point A),
- Mobilized shear stress- displacements (at the top) (for points K, M and N),
- Mobilized shear stress along A-A' cross-section for different displacement levels.

6.1. Over Consolidated Clay

This sub-Chapter will present the results of the analyses with the change in fixed lengths respectively (6m, 8m, and 12m). Analyses for each particular length is utilized by changing tendon material (steel and GFRP). First, Plaxis analysis outputs; deformation and relative shear plots at 0.4 cm, several prescribed pull out values will be presented. Later, graphs for the points and cross-Sections described in Section 5.2.4 will be presented respectively.

6.1.1. 6m Fixed Length

6.1.1.1. Ground Anchor with Steel Bar. Pull-out test of a ground anchor with steel bar embedded in over consolidated clay soil is simulated in this model. The fixed length of the ground anchor is 6m. In this analysis, the transition from elastic phase to plastic phase occurred prior to 0.4 cm pull out (at 0.2cm). Therefore only the deformation and relative shear plots for 0.4cm pull out will be presented in Figure 6.1 and Figure 6.2.

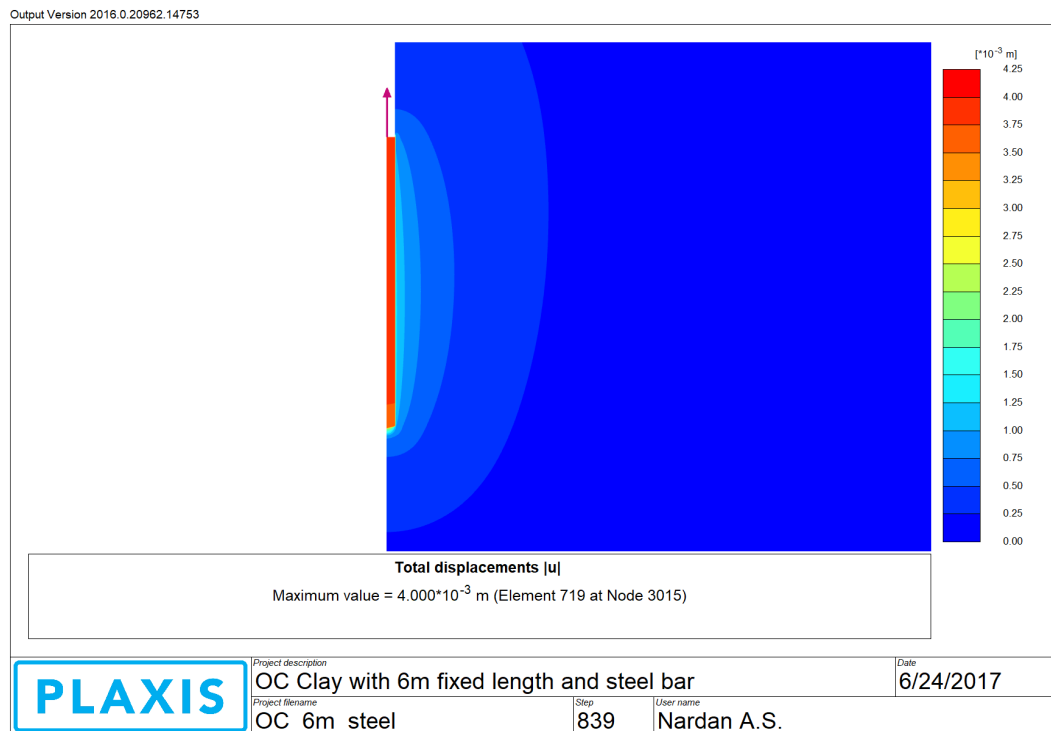


Figure 6.1. Deformation plot for 0.04 cm pulling.

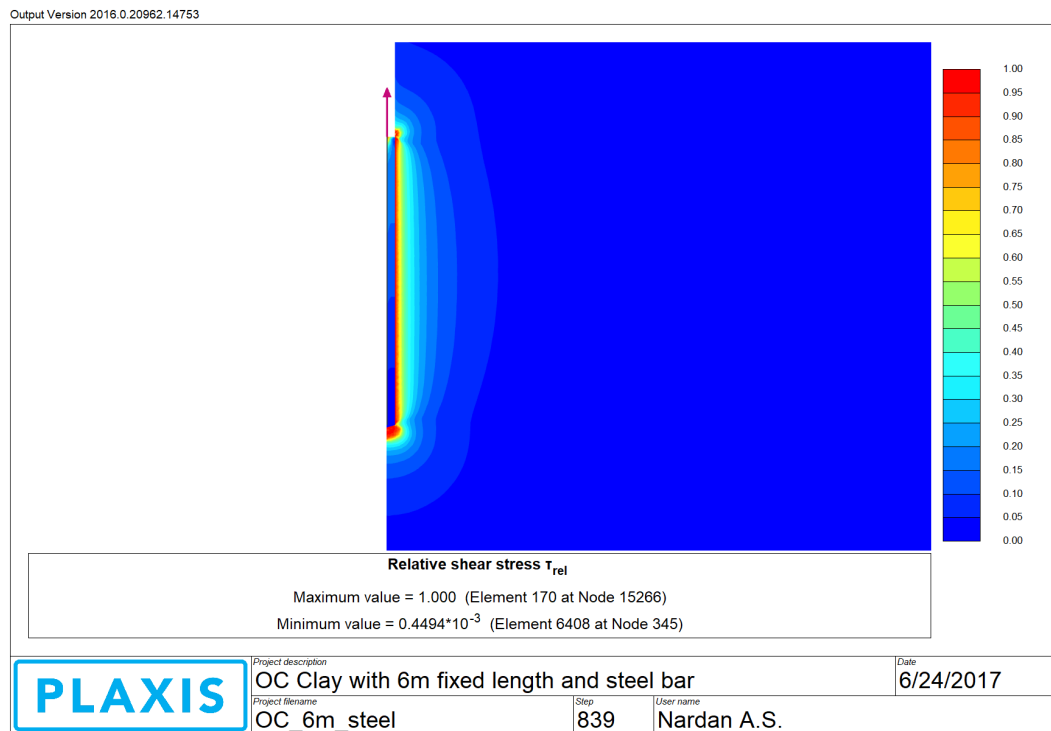


Figure 6.2. Relative Shear plot for 0.4 cm pull out.

Graphs for the load-displacement and mobilized shear stress to “deflection on the top of the fixed length” may be seen in Figure 6.3 and Figure 6.4. The mobilized shear stress on the point which is at the top of the fixed length K, has reached the peak shear stress almost instantly, as the displacement continues, it dropped to a residual value as expected. The medium point M, and the distal point N, has reached the peak stress value after the point K but still at a relatively small displacements and stayed at these reached values until failure. As can be seen from Figure 6.5, the shear stress doesn't change remarkably along the fixed length zone. The failure happened at 0.2 cm pull-out, no difference is apparent but the change in the stress at the top of the fixed length slightly decreased to residual level.

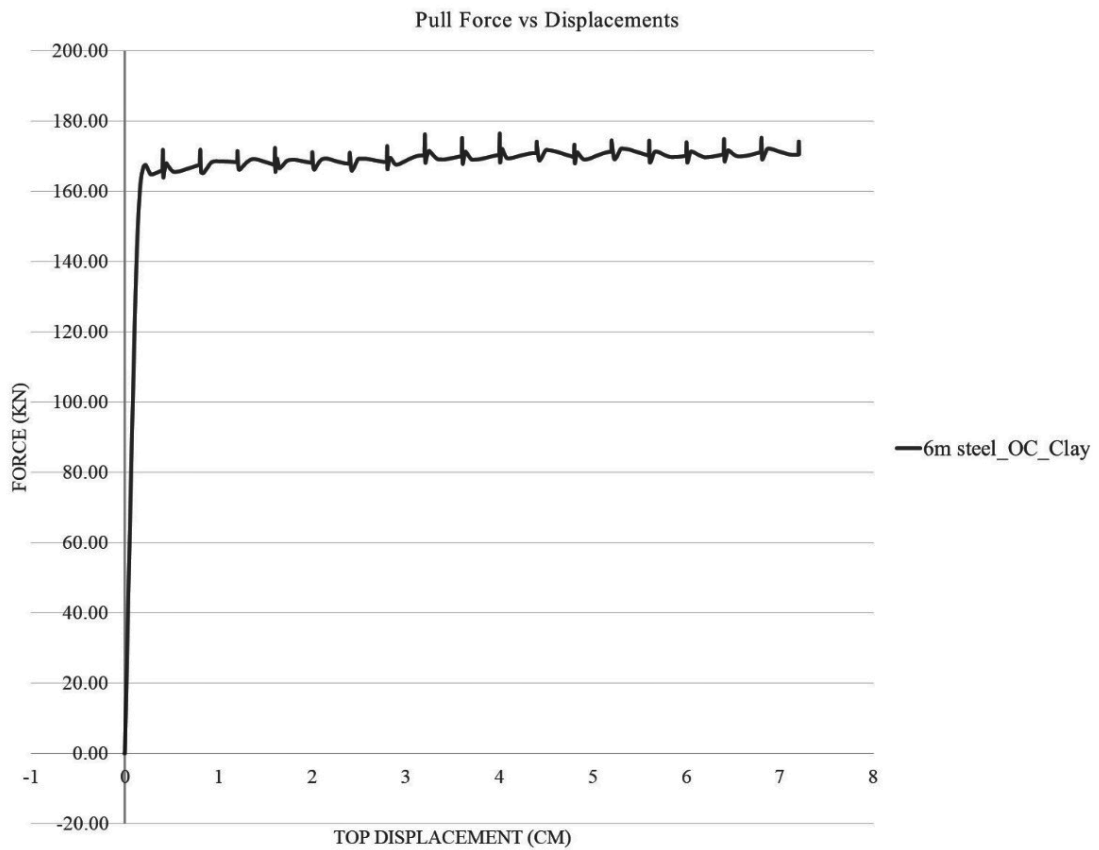


Figure 6.3. Load vs Displacement plot.

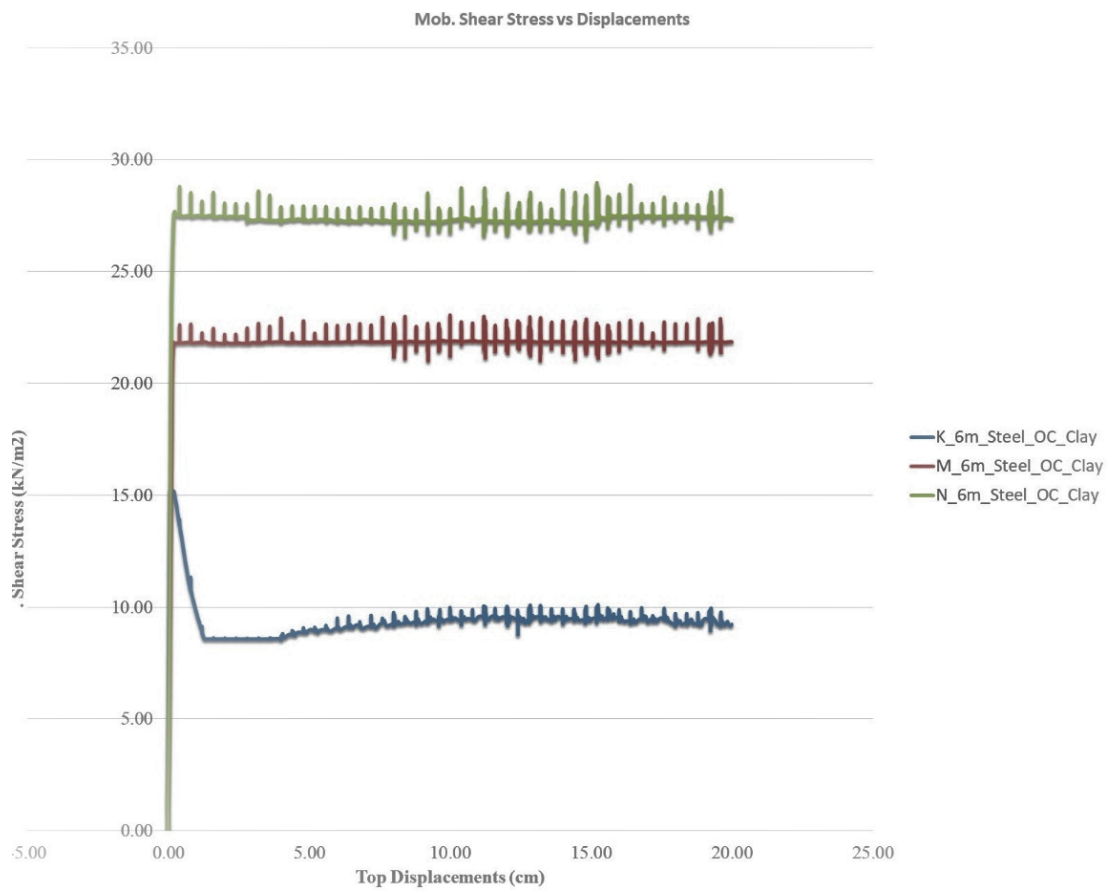


Figure 6.4. Mobilized Shear Stress vs Displacement plot for points K, M and N in the FE mesh.

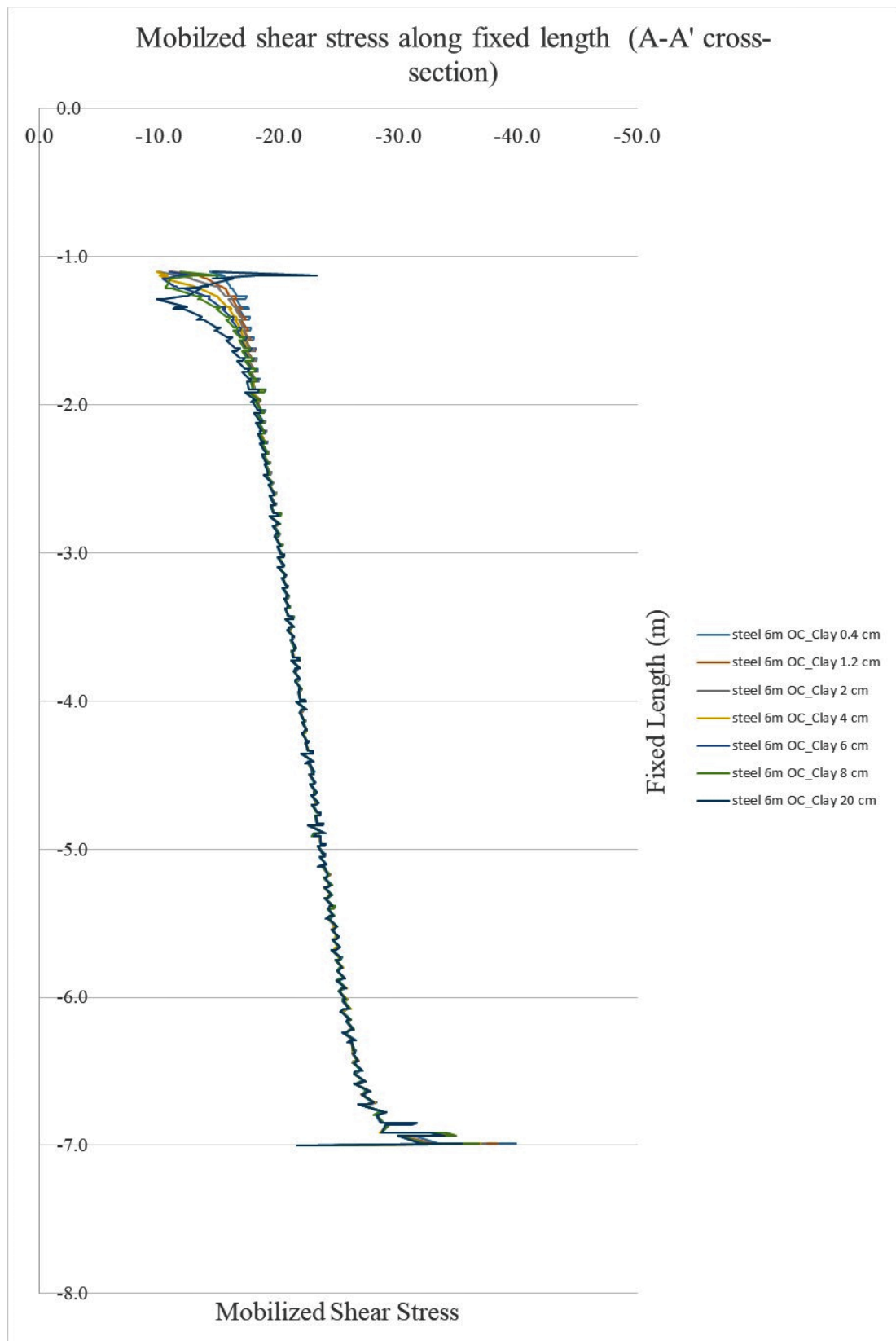


Figure 6.5. Mobilized shear stress for different load levels along A-A' cross-Section.

6.1.1.2. Ground Anchor with GFRP Bar. Pull-out test of a ground anchor with GFRP bar is simulated for the same conditions with the previous model. In this analysis, the transition from elastic phase to plastic phase occurred prior to 0.4 cm pull out (at 0.2cm) likewise the previous model. Therefore only the deformation and relative shear plots for 0.4cm pull out will be presented in Figure 6.6 and Figure 6.7.

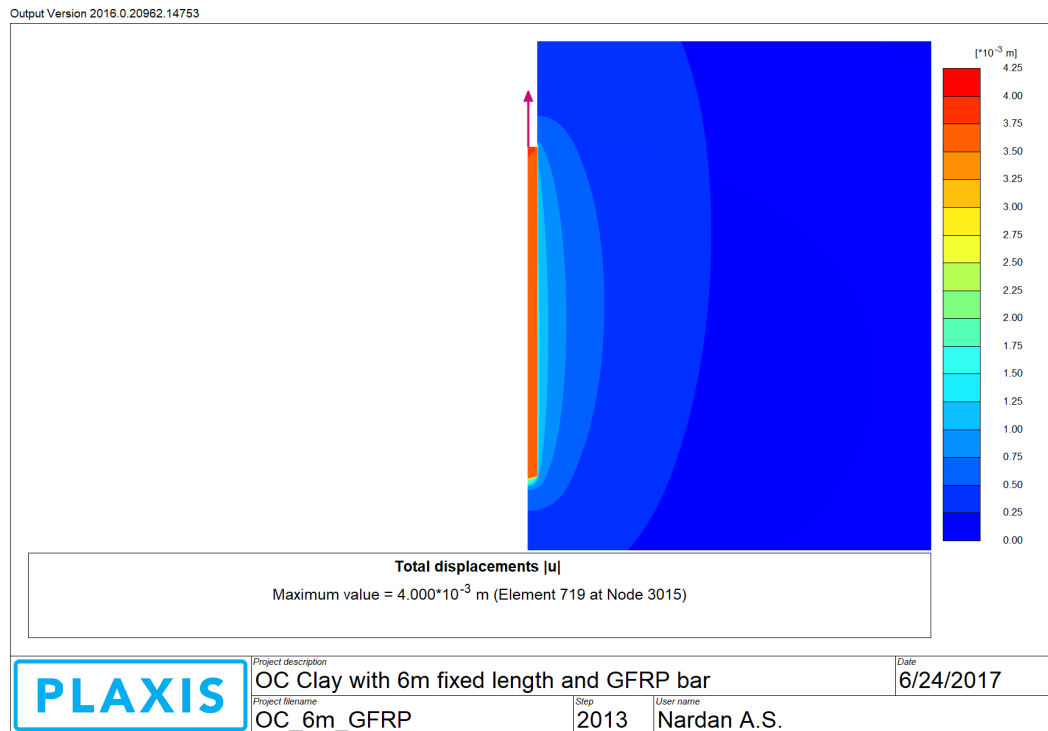


Figure 6.6. Deformation plot for 0.4 cm pull out.

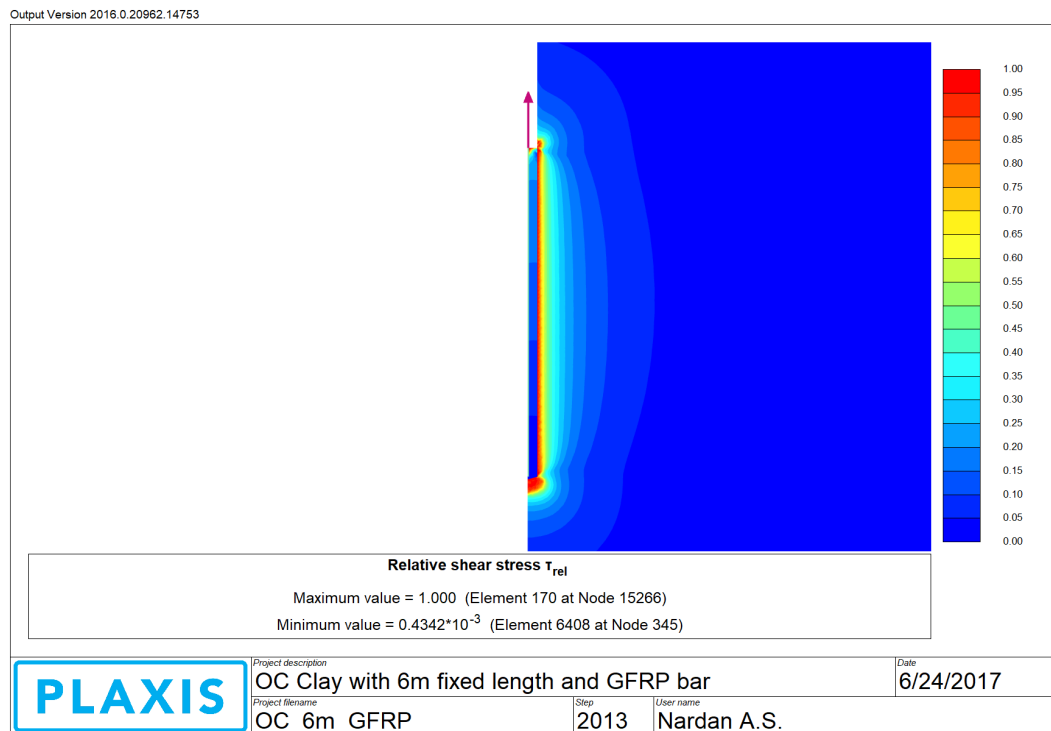


Figure 6.7. Relative Shear plot for 0.4 cm pull out.

Graphs for the load-displacement and mobilized shear stress to “deflection on the top of the fixed length” may be seen in Figure 6.8 and Figure 6.9.

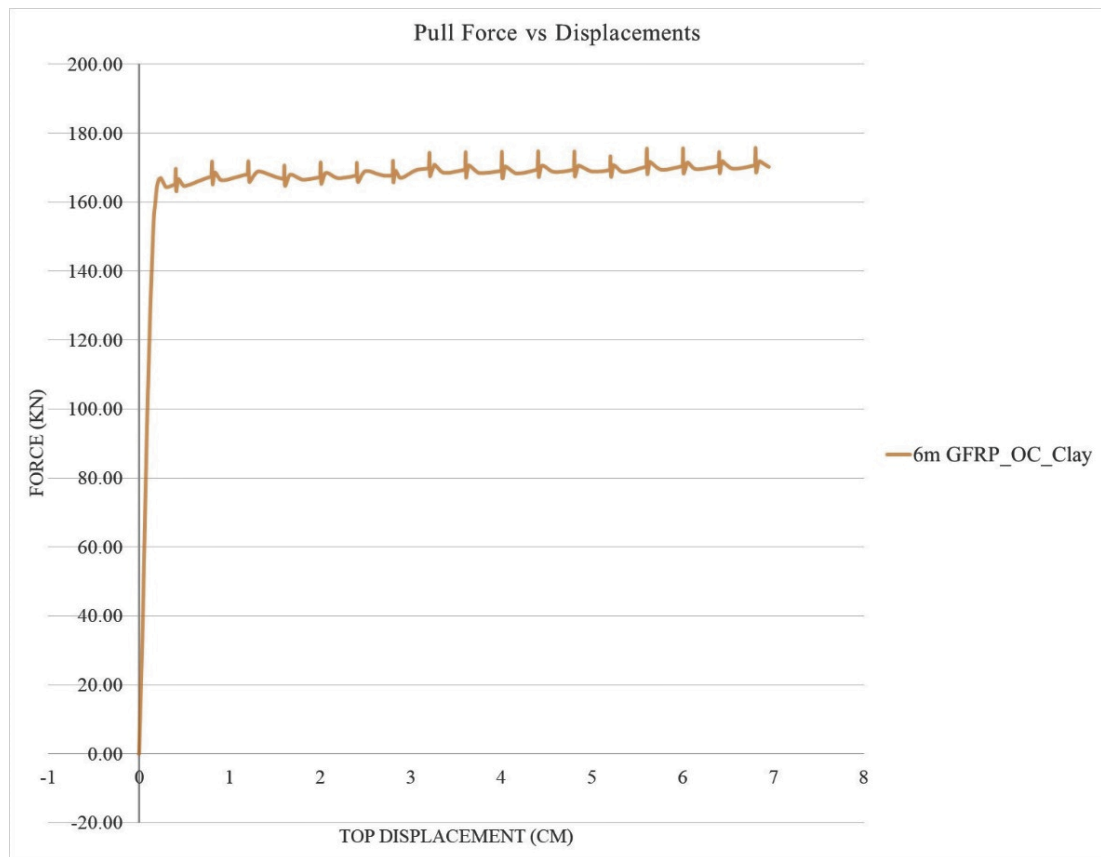


Figure 6.8. Load vs Displacement plot.

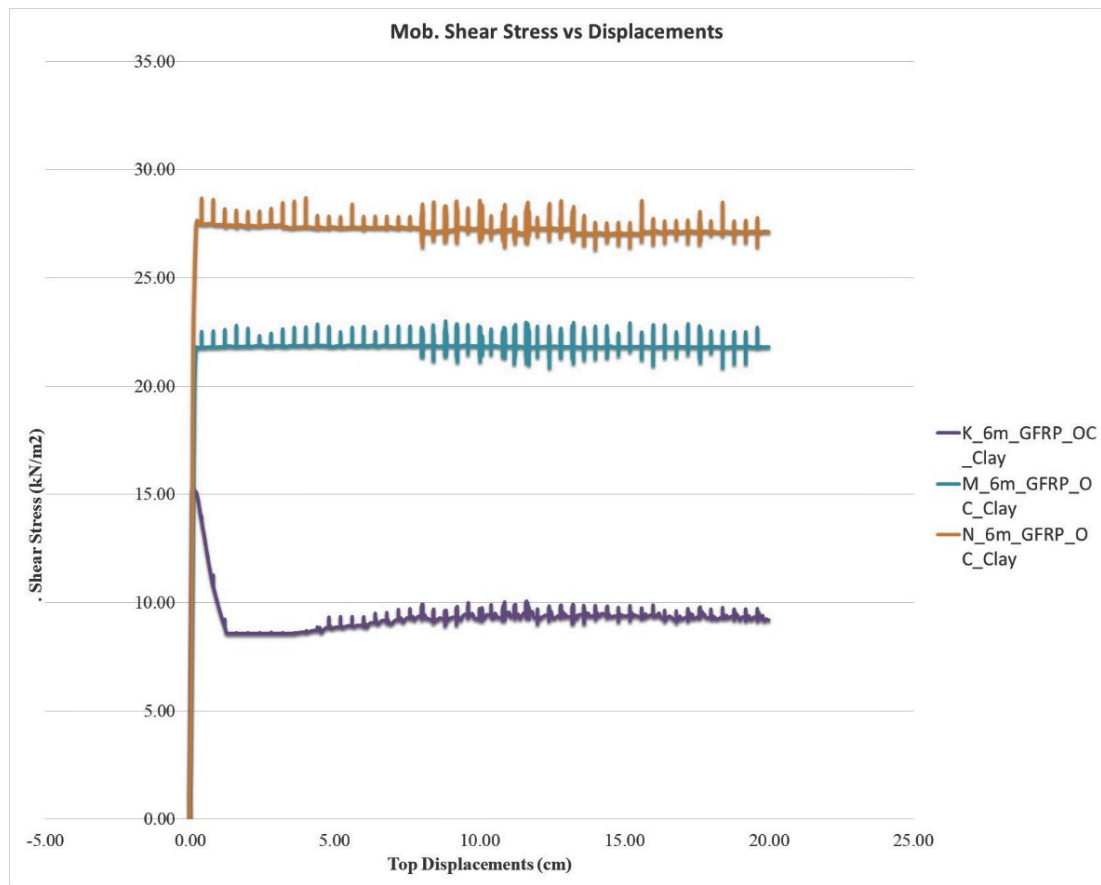


Figure 6.9. Mobilized Shear Stress vs Displacement plot.

As can be anticipated, the behaviours of points; K, M, N for the ground anchor with GFRP bar and the steel bar are identical; The mobilized shear stress on the point which is at the top of the fixed length K, has reached the peak shear stress almost instantly as the displacement continues, it dropped to a residual value as expected. The medium point M, and the distal point N, has reached the peak stress value after the point K but still at a relatively small displacements and stayed at these reached values until failure.

As can be seen from Figure 6.10, the shear stress doesn't change remarkably along fixed length zone. The failure happened at 0.2 cm, no difference is apparent but the change in the stress at the top of the fixed length slightly decreased to residual level.

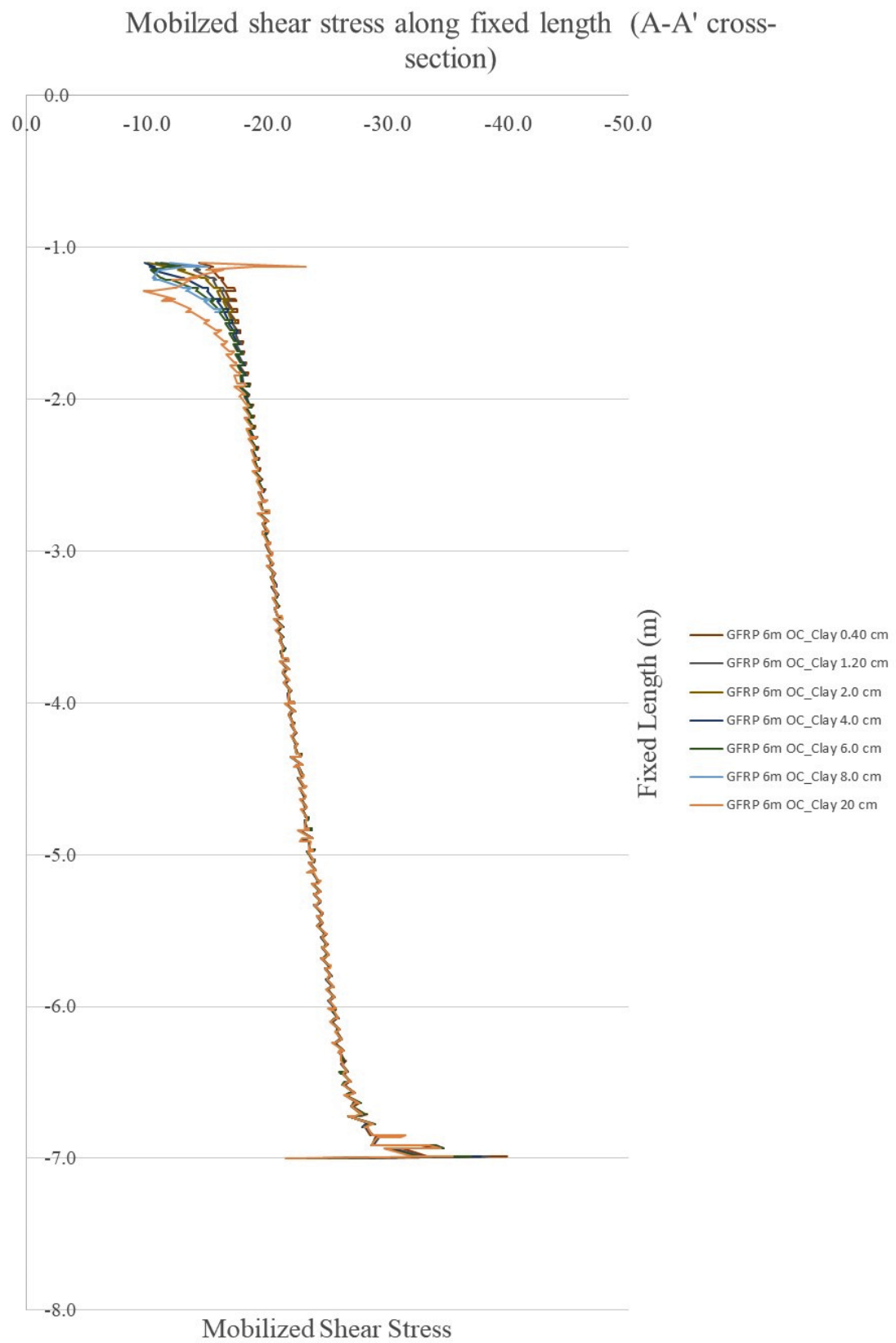


Figure 6.10. Mobilized shear stress for different load levels along A-A' cross-Section.

6.1.1.3. Comparison of the Results. As can be anticipated, at this stress level with the over consolidated clay material with 6m fixed length, there is no significant difference between ground anchors with GFRP bar and conventional steel bar. The pull-out force-displacement, shear stress to top displacements and Mobilized shear stress distribution along the fixed length (A-A') for two anchor bar types can be seen in Figure 6.11, Figure 6.12 and Figure 6.13.

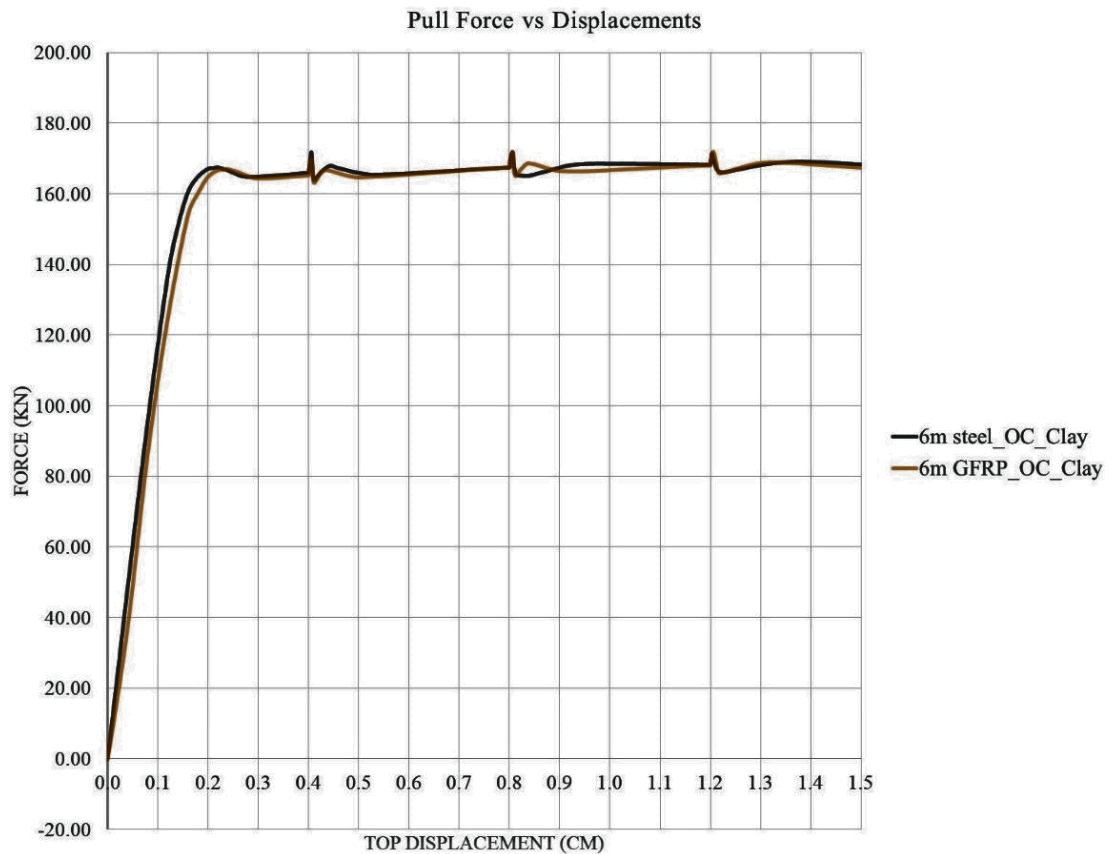


Figure 6.11. Load vs Displacement plot for the comparison of GFRP and Steel Bar.

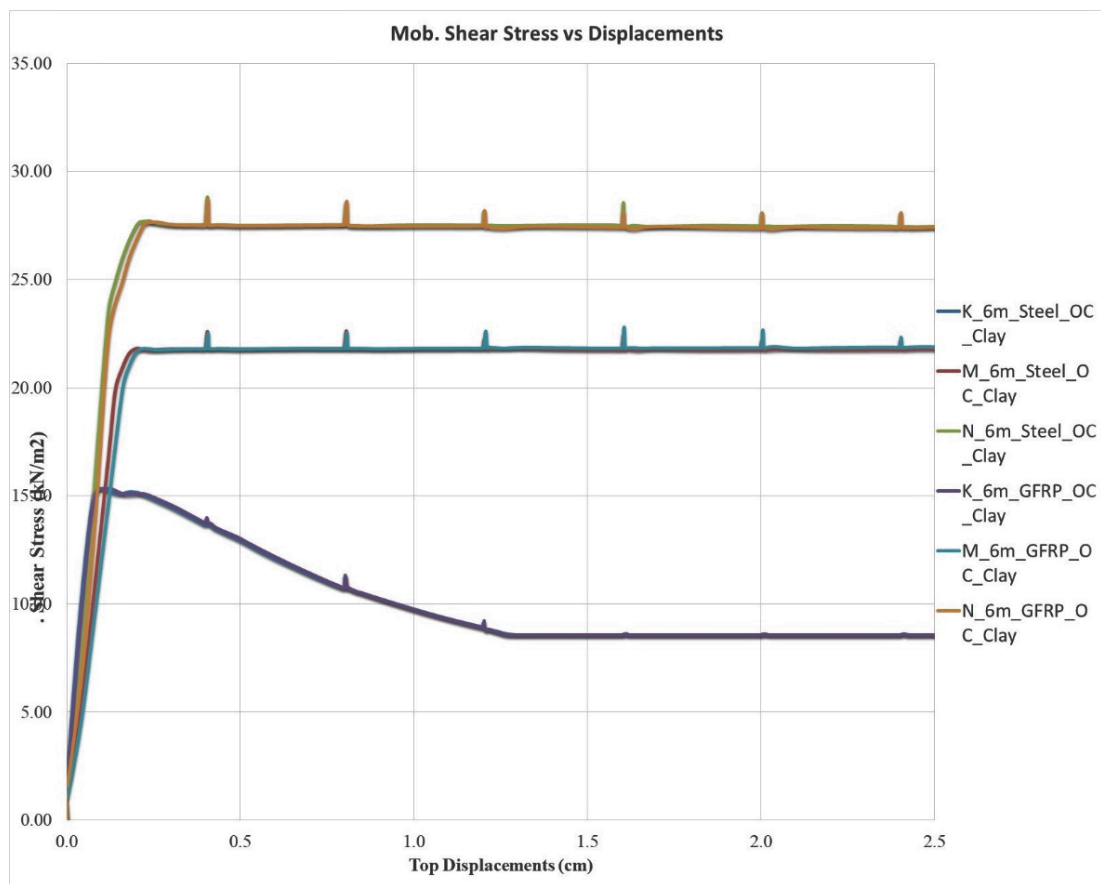


Figure 6.12. Mobilized Shear Stress vs Displacement plot for the comparison of GFRP and Steel Bar.

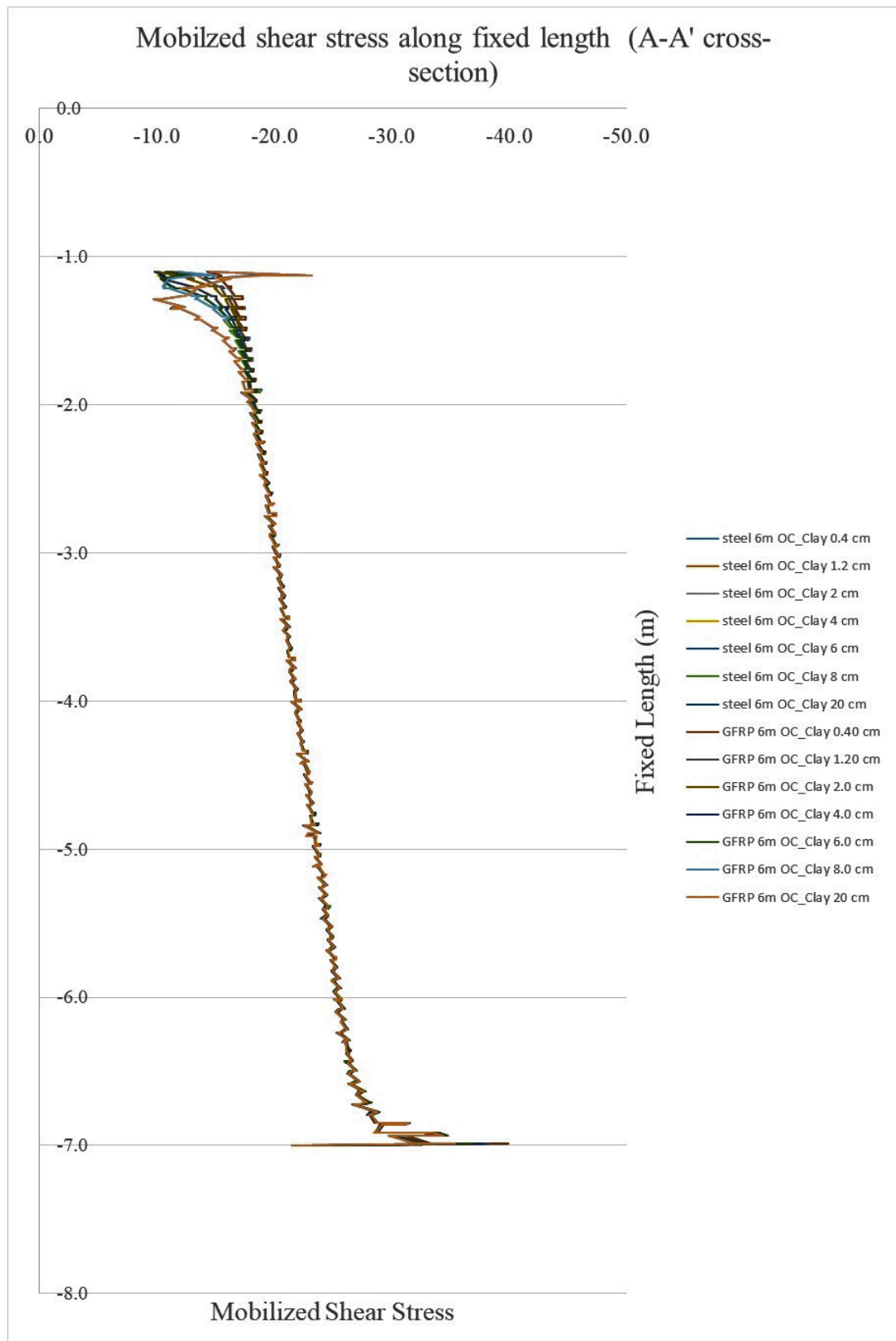


Figure 6.13. Mobilized shear stress for different load levels along A-A' cross-Section for the comparison of GFRP and Steel Bar.

6.1.2. 8m Fixed Length

6.1.2.1. Ground Anchor with Steel Bar. Pull-out test of a ground anchor with steel bar embedded in over consolidated clay soil is simulated in this model. The fixed length of the ground anchor is 8m. In this analysis, the transition from elastic phase to plastic phase occurred prior to 0.4 cm pull out (at 0.3cm). Therefore only the deformation and relative shear plots for 0.4cm pull-out will be presented in Figure 6.14 and Figure 6.15.

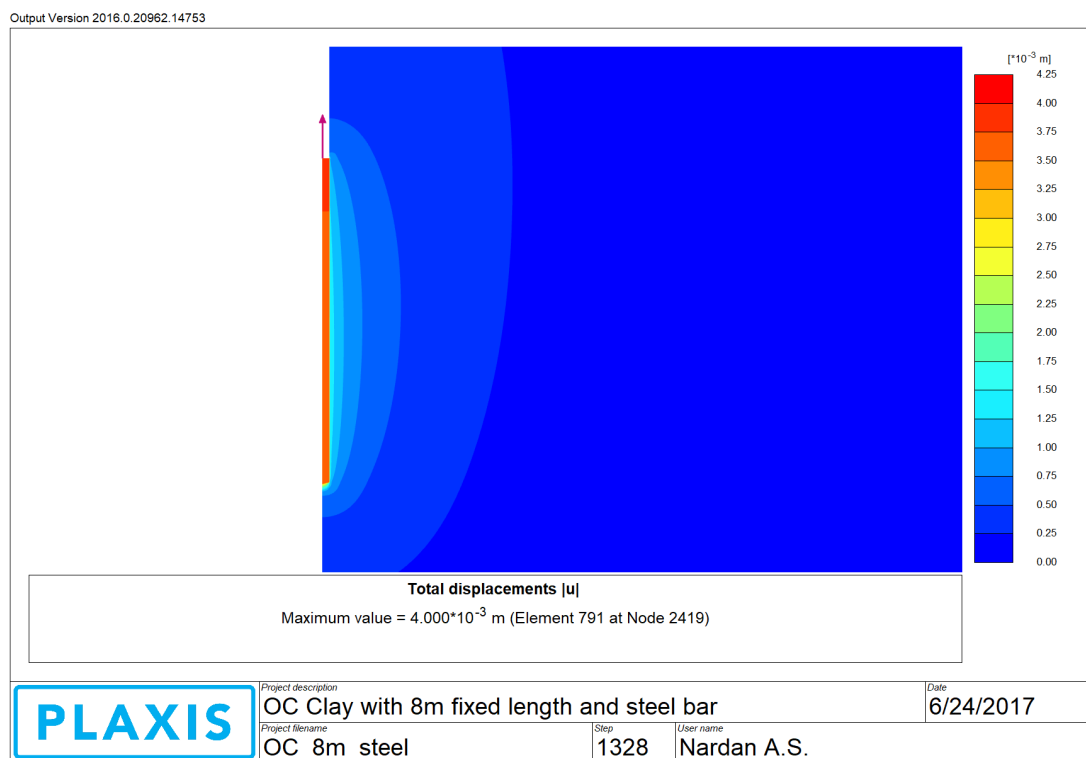


Figure 6.14. Deformation plot for 0.4 cm pull out.

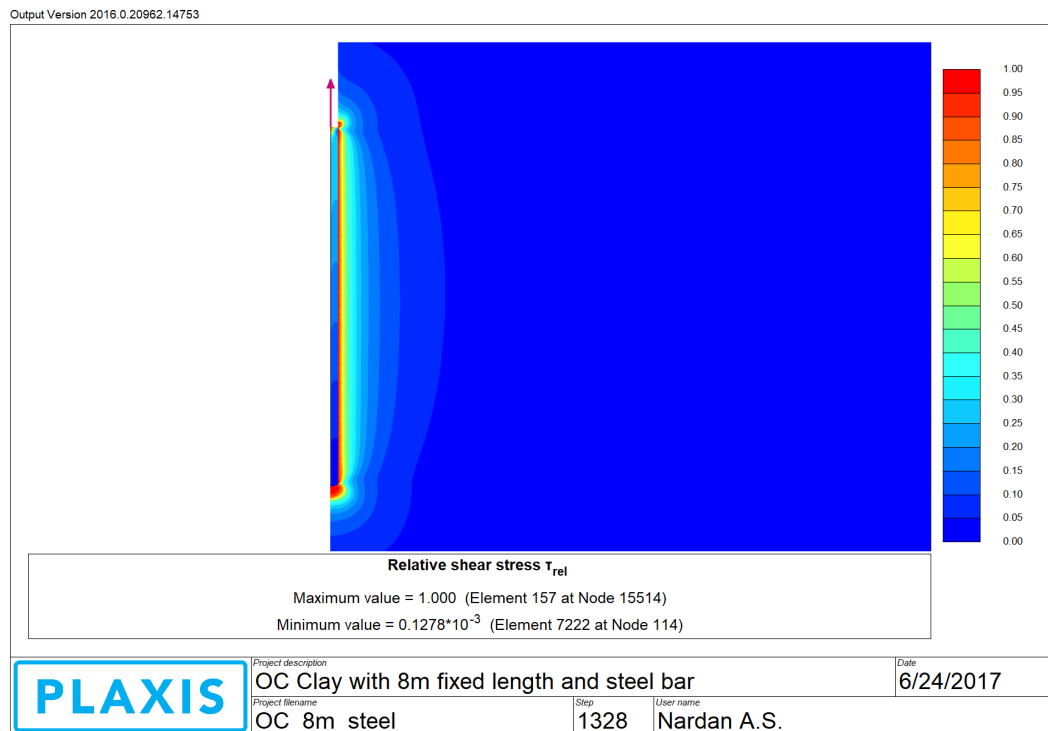


Figure 6.15. Relative Shear plot for 0.4 cm pull out.

Pull-out resistance is around 1.5 times larger than ground anchor with 6m fixed length. Also, it can be seen from the contour diagram the transmission of the deformations from the proximal to the distal part of the fixed length slightly slower than ground anchor with 6m fixed length. Graphs for the load-displacement and mobilized shear stress to “deflection on the top of the fixed length” may be seen in Figure 6.16 and Figure 6.17.

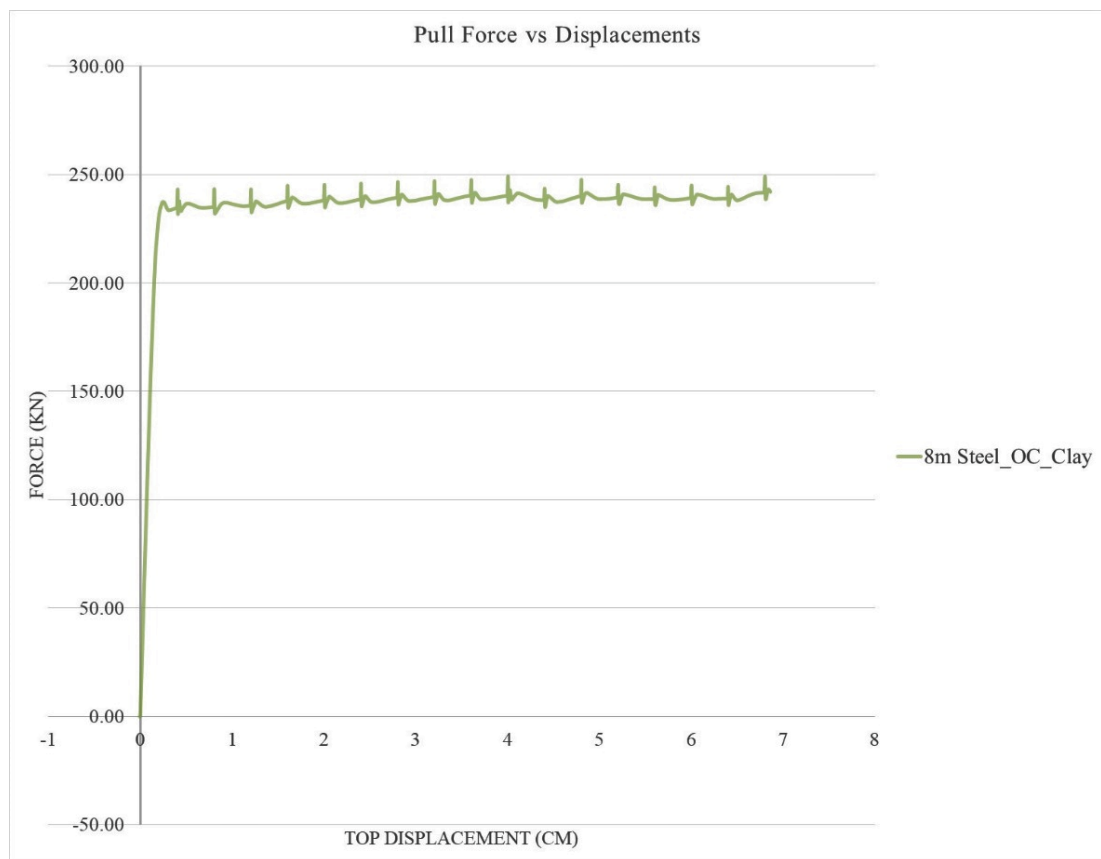


Figure 6.16. Load vs Displacement plot.

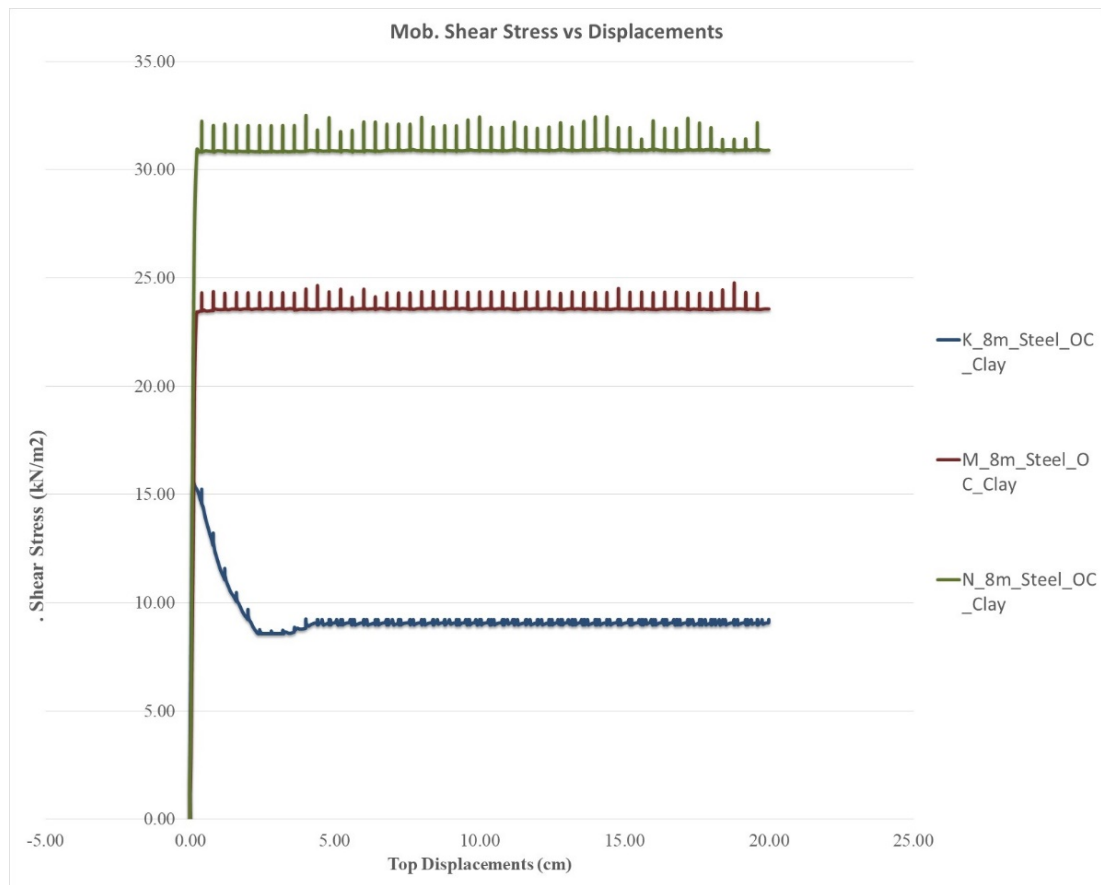


Figure 6.17. Mobilized Shear Stress vs Displacement plot.

The mobilized shear stress on the point which is at the top of the fixed length K, has reached the peak shear stress almost instantly as the displacement continues, it dropped to a residual value likewise in the previous ones. The medium point M, and the distal point N, has reached the peak stress value after the point K at a relatively small displacements and stayed at these values until the failure.

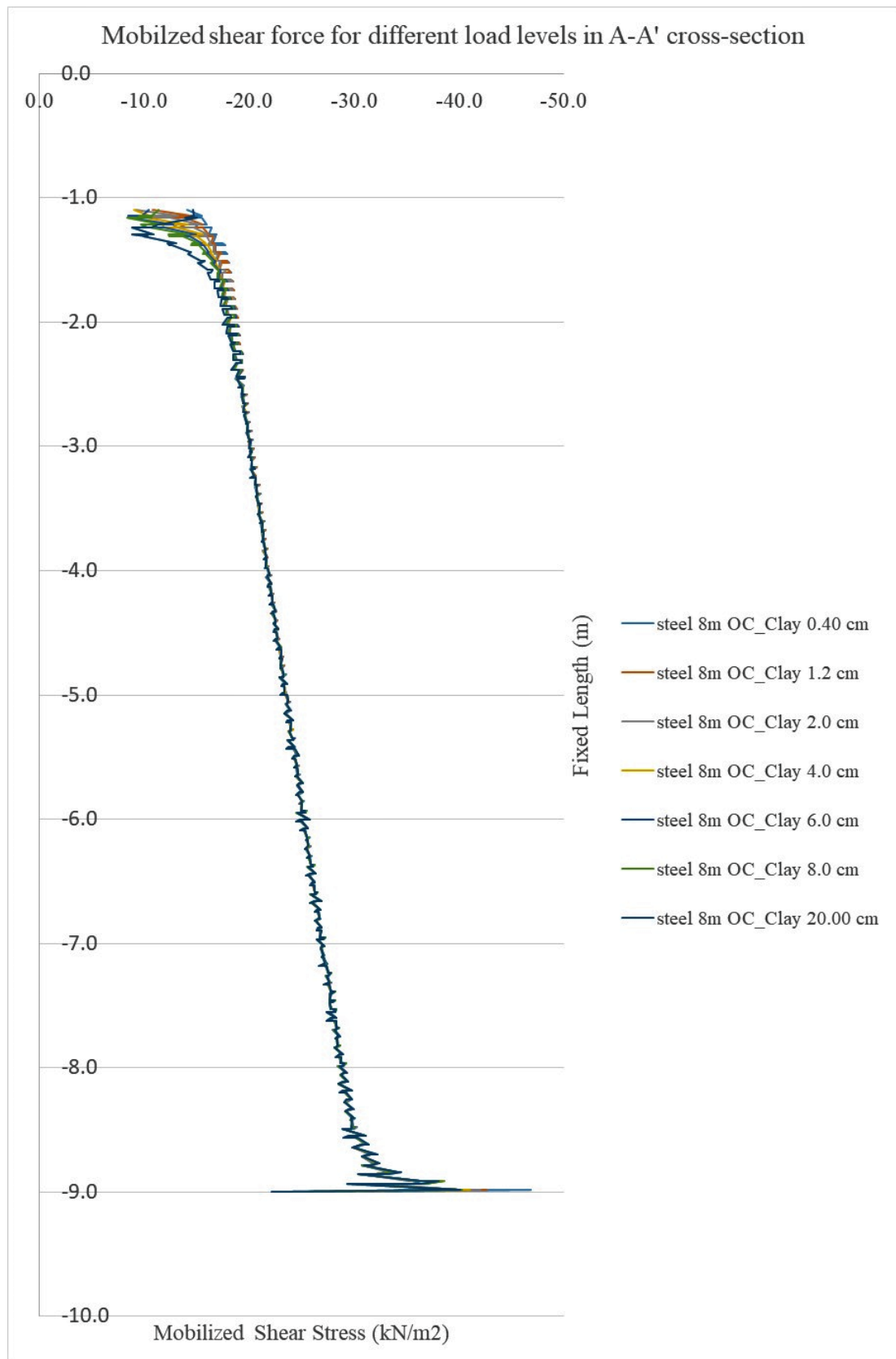


Figure 6.18. Mobilized shear stress for different load levels along A-A' cross-Section.

As it can be easily observed from Figure 6.18, the shear stress doesn't change remarkably along the fixed length zone. The failure happened at 0.3 cm almost instantly, no difference is apparent but the change in the stress at the top of the fixed length slightly decreased to residual level.

6.1.2.2. Ground Anchor with GFRP Bar. Pull-out test of a ground anchor with GFRP bar is simulated for the same conditions with the previous model. In this analysis, the transition from elastic phase to plastic phase occurred prior to 0.4 cm pull-out (at 0.36cm) likewise the previous model. Therefore only the deformation and relative shear plots for 0.4cm pull-out will be presented in Figure 6.19 and Figure 6.20.

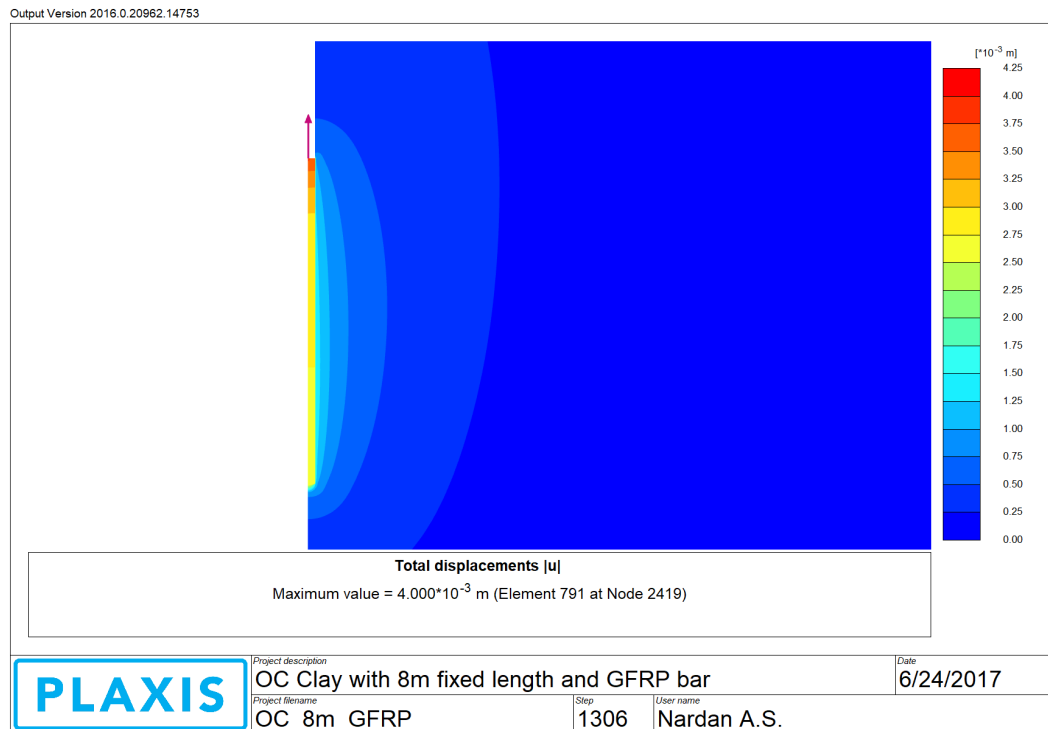


Figure 6.19. Deformation plot for 0.4 cm pull out.

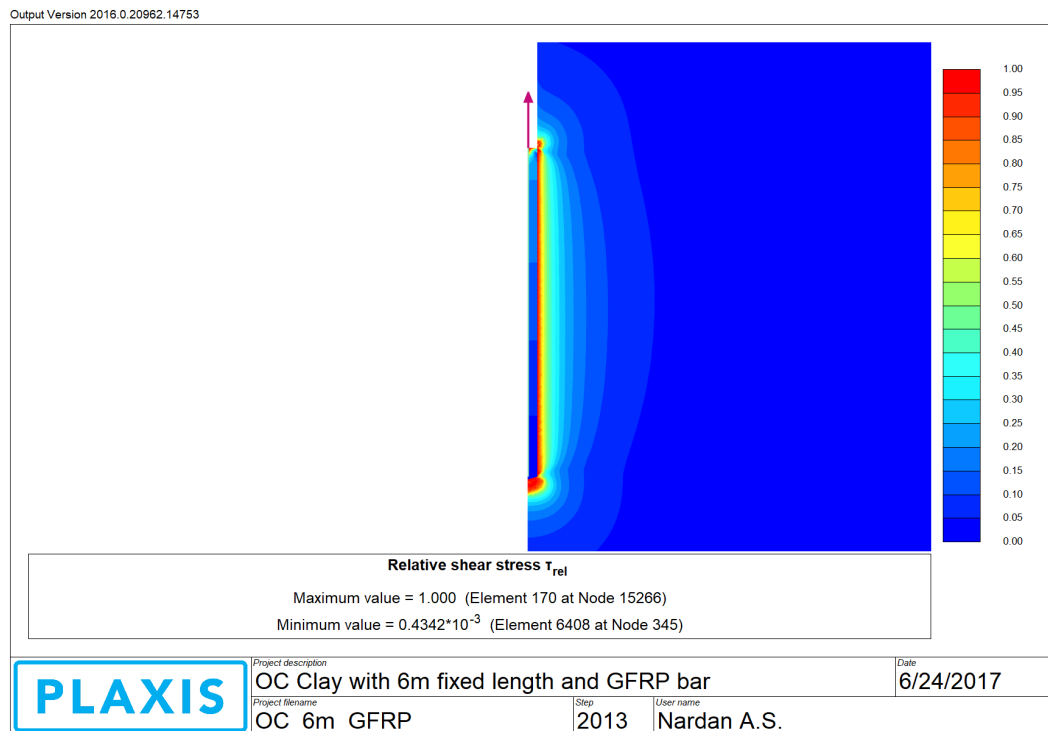


Figure 6.20. Relative Shear plot for 0.4 cm pull out.

Resulting pull-out force is approximately 1.5 times larger than ground anchor with 6m fixed length. Moreover, it is more distinguishable in this case from the contour diagram the transmission of the deformations from the proximal to the distal part of the fixed length is slower than ground anchor with 6m fixed length and 8m fixed length. This circumstance can be comprehended by following the yellow color in the deformation plot. Graphs for the load vs displacement and mobilized shear stress to “deflection on the top of the fixed length” may be seen in Figure 6.21 and Figure 6.22.

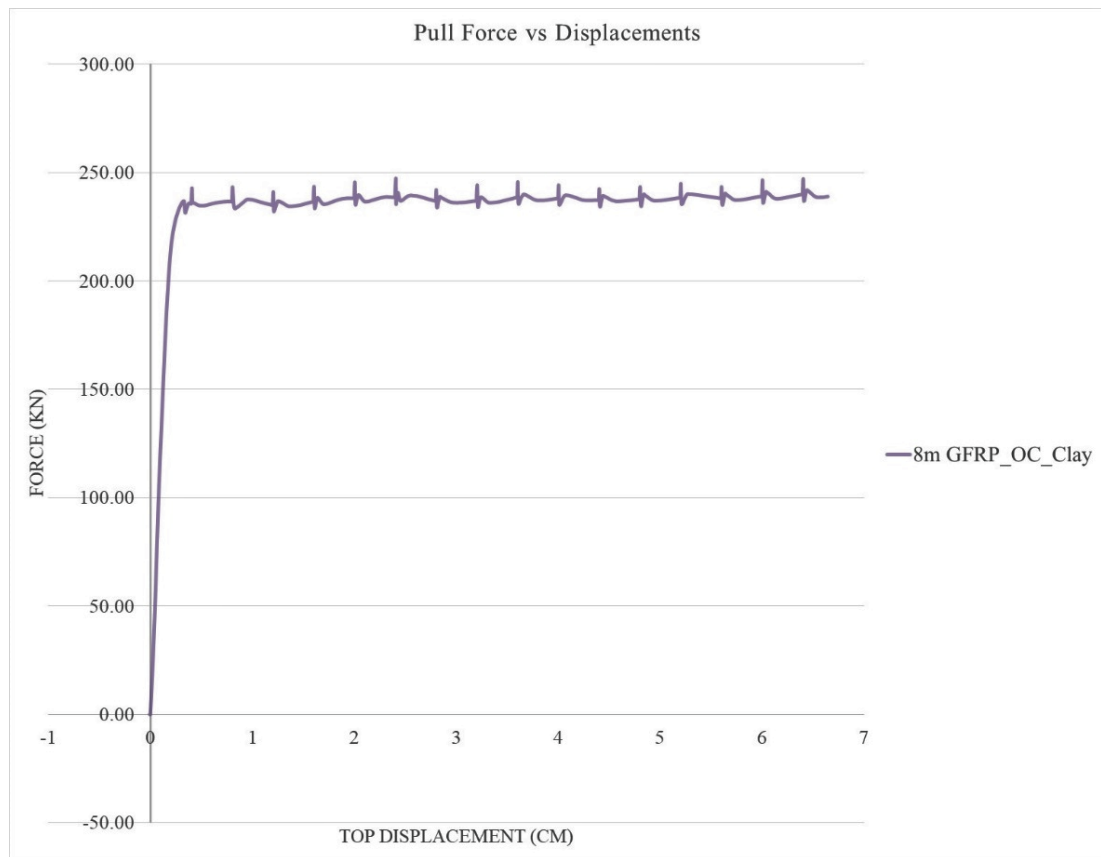


Figure 6.21. Load vs Displacement plot.

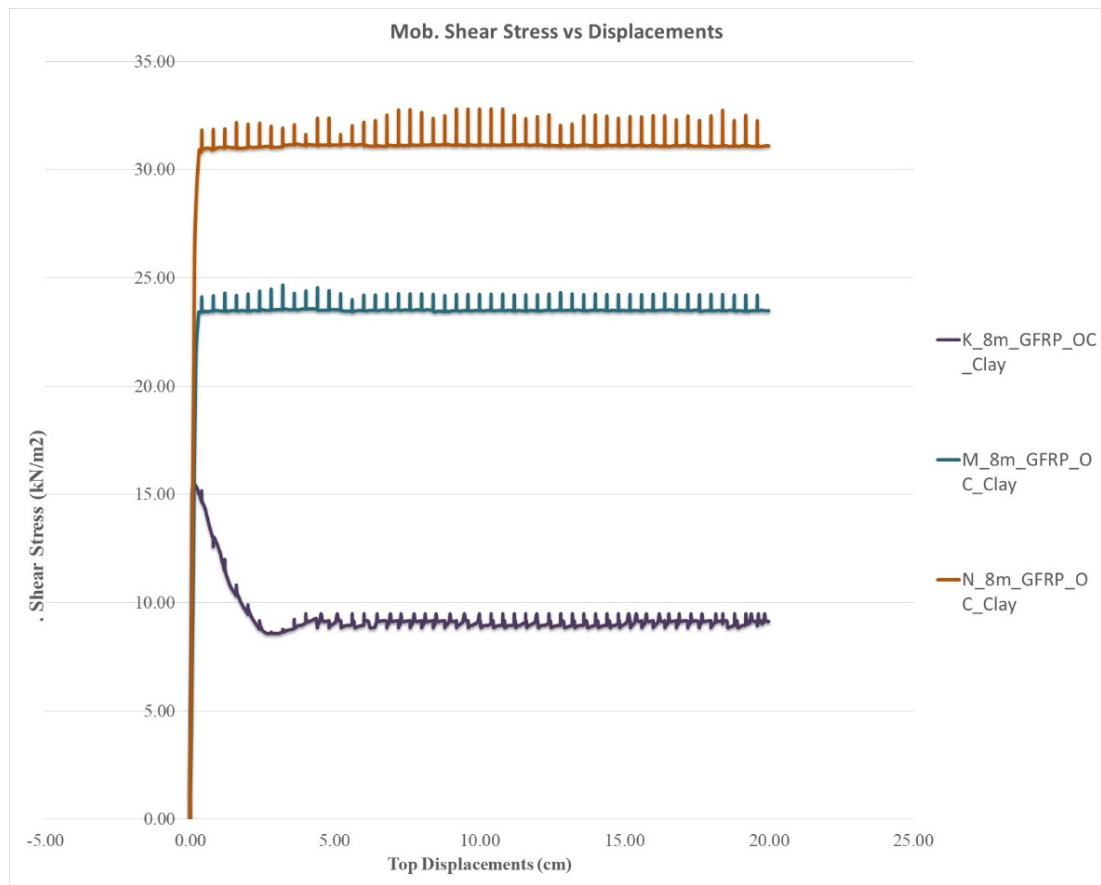


Figure 6.22. Mobilized Shear Stress vs Displacement plot.

As can be anticipated, behaviours of points; K, M, N for the ground anchor with GFRP bar and the steel bar are identical. The mobilized shear stress on the point which is at the top of the fixed length K, has reached the peak shear stress almost instantly as the displacement continues, it dropped to a residual value. The medium point M, and the distal point N, has reached the peak stress value after the point K, but still at a relatively small displacements and stayed at these values until failure. The identified behaviour is not significant in any model up to here.

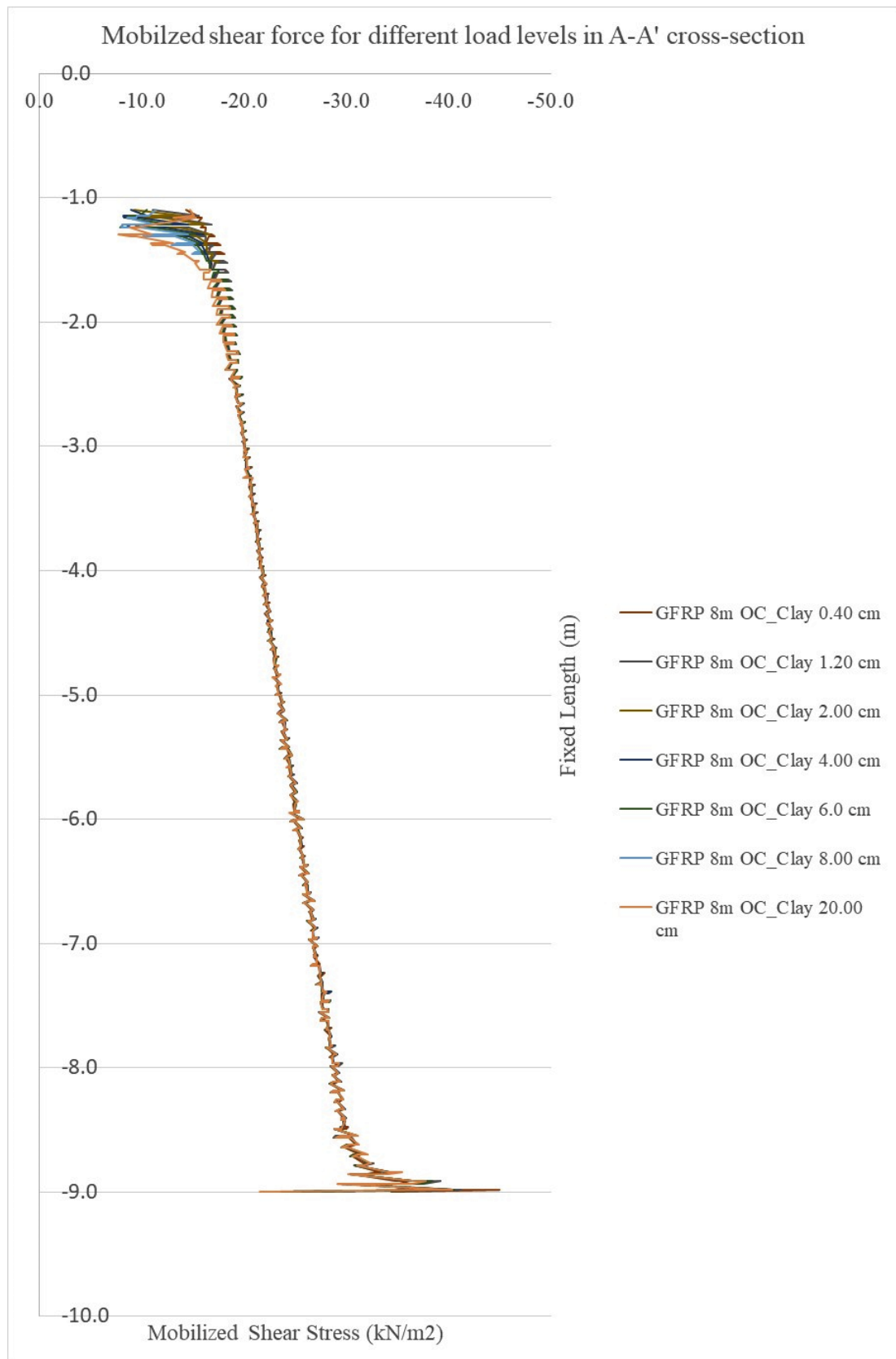


Figure 6.23. Mobilized shear stress for different load levels along A-A' cross-Section.

As can be seen from Figure 6.23, the shear stress doesn't change remarkably along fixed length zone. The failure happened at 0.3 cm deformation at the top with a relatively small displacements, no difference is apperant but the change in the stress at the top of the fixed length slightly decreased to residual level.

6.1.2.3. Comparison of the Results. As it can be anticipated, at this stress level with the over consolidated clay material with 8m fixed length, there is no significant difference between ground anchors with GFRP bar and conventional steel bar likewise in 6m fixed length. The pull-out force-displacement, shear stress to top displacements and Mobilized shear stress distribution along the fixed length (A-A') for two anchor bar types can be seen in Figure 6.24, Figure 6.25 and Figure 6.26.

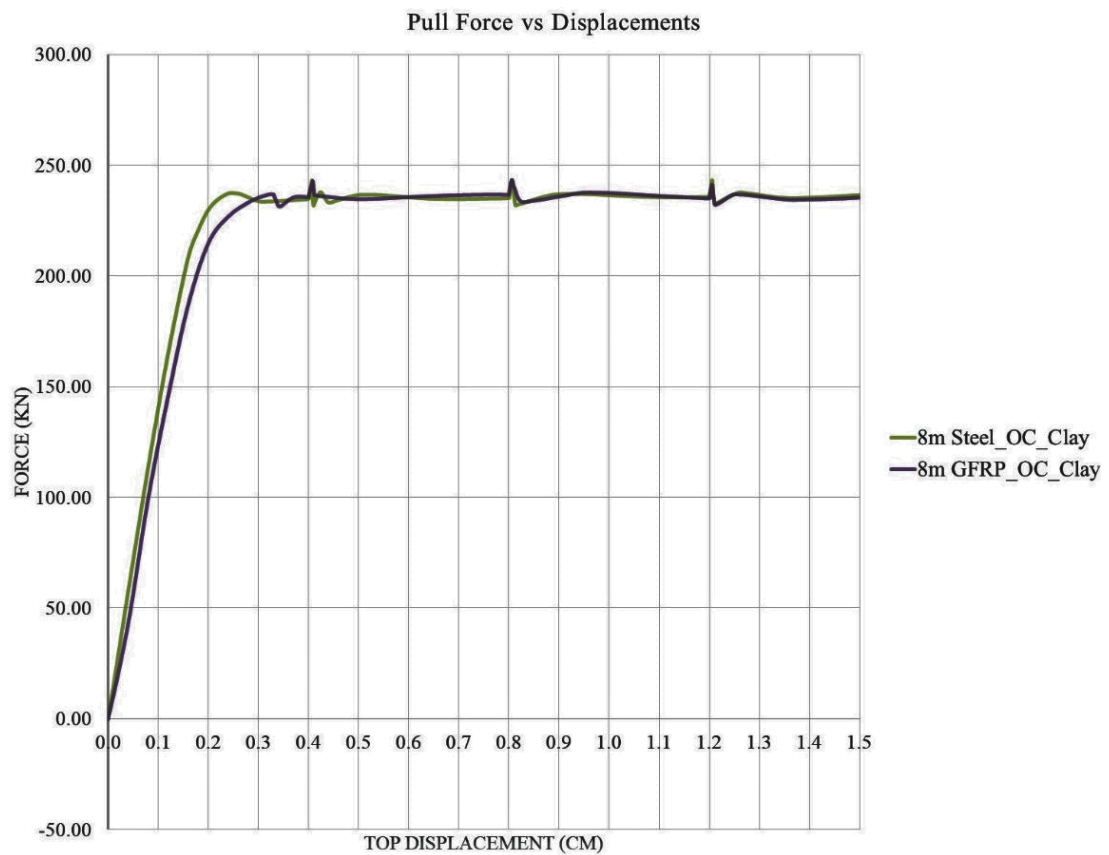


Figure 6.24. Load vs Displacement plot for the comparison of GFRP and Steel Bar.

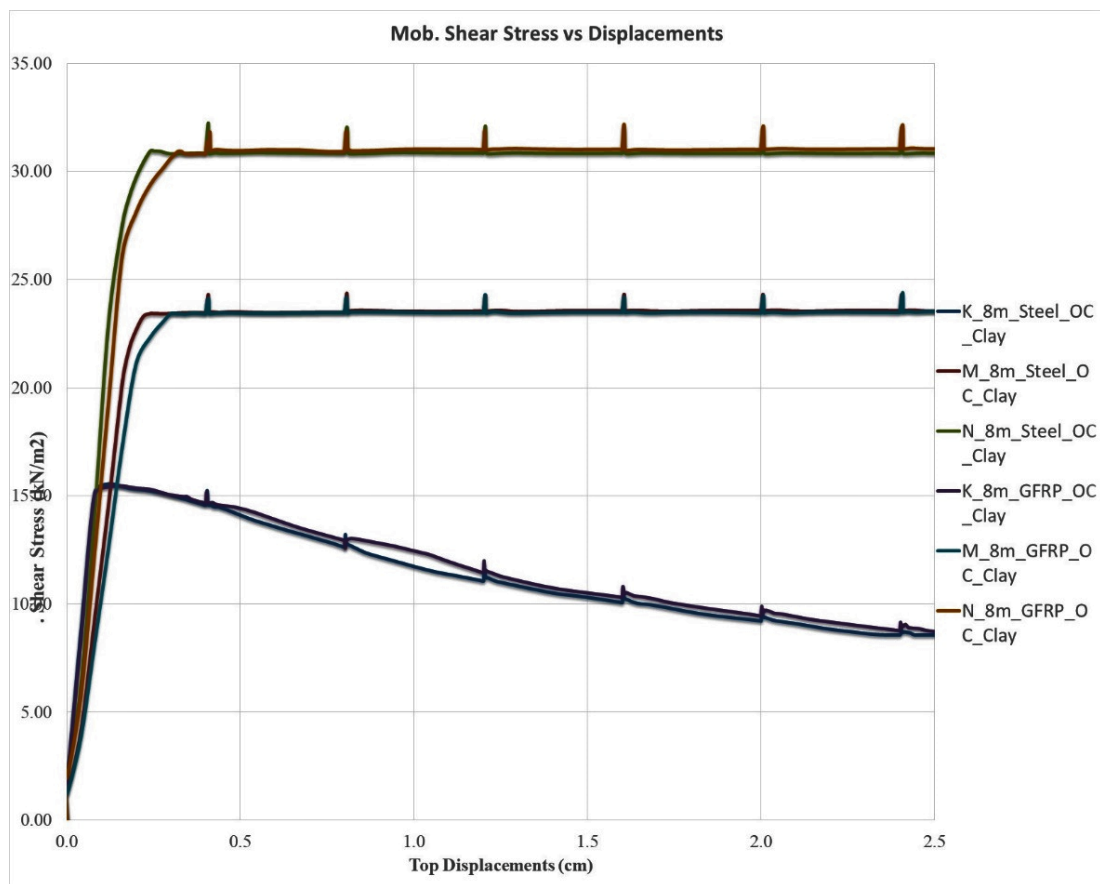


Figure 6.25. Mobilized Shear Stress vs Displacement plot for the comparison of GFRP and Steel Bar.

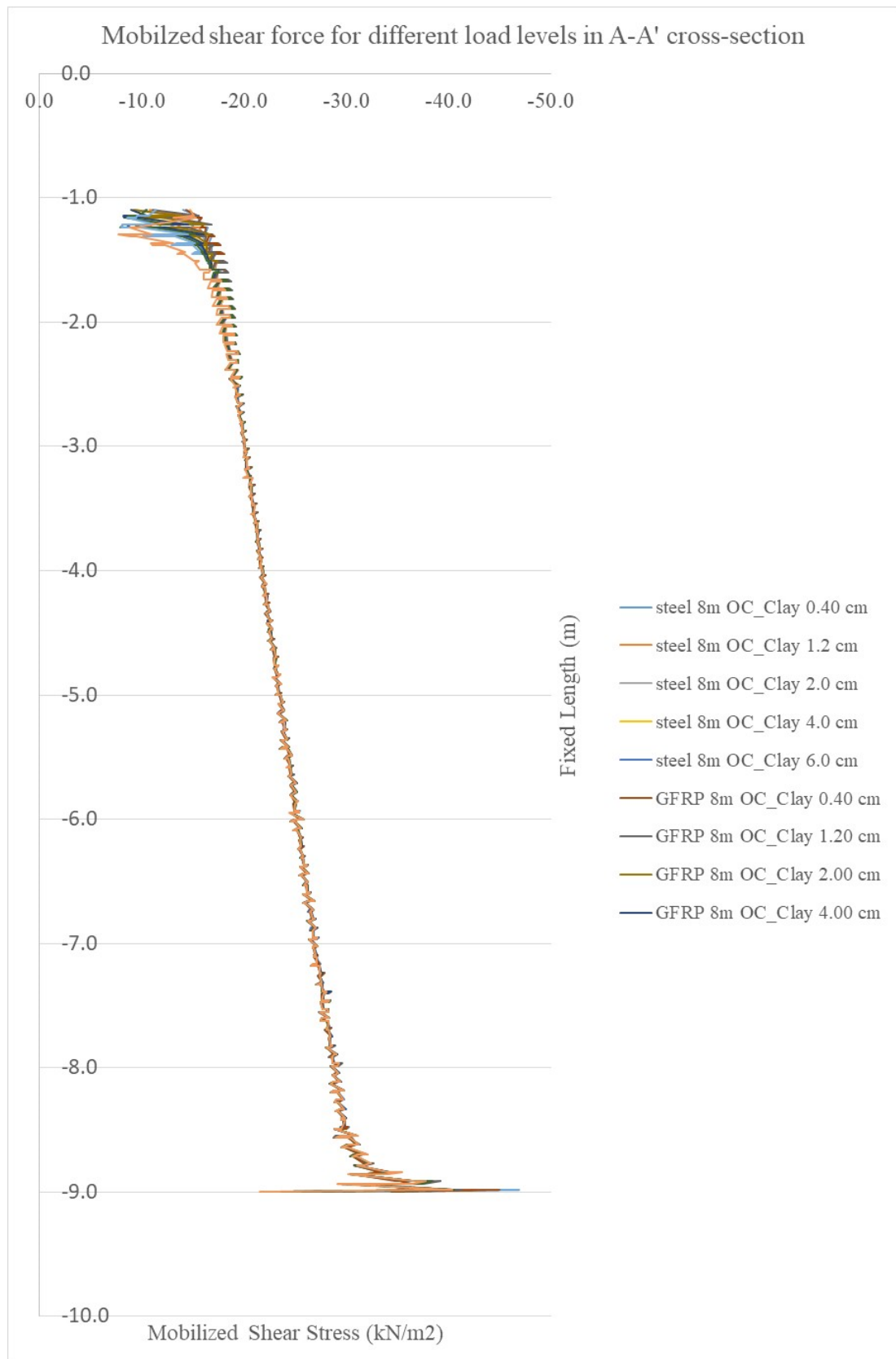


Figure 6.26. Mobilized shear stress for different load levels along A-A' cross-Section for the comparison of GFRP and Steel Bar.

6.1.3. 12m Fixed Length

6.1.3.1. Ground Anchor with Steel Bar. Pull-out test of a ground anchor with steel bar embedded in over consolidated clay soil is simulated in this model. The fixed length of the ground anchor is 12 m. First, Plaxis analyses outputs; deformation and relative shear plots for the first 0.4 cm pull-out and for the 8mm pull-out will be presented in Figures 6.27-30, as failure occurred at 0.5cm displacement.

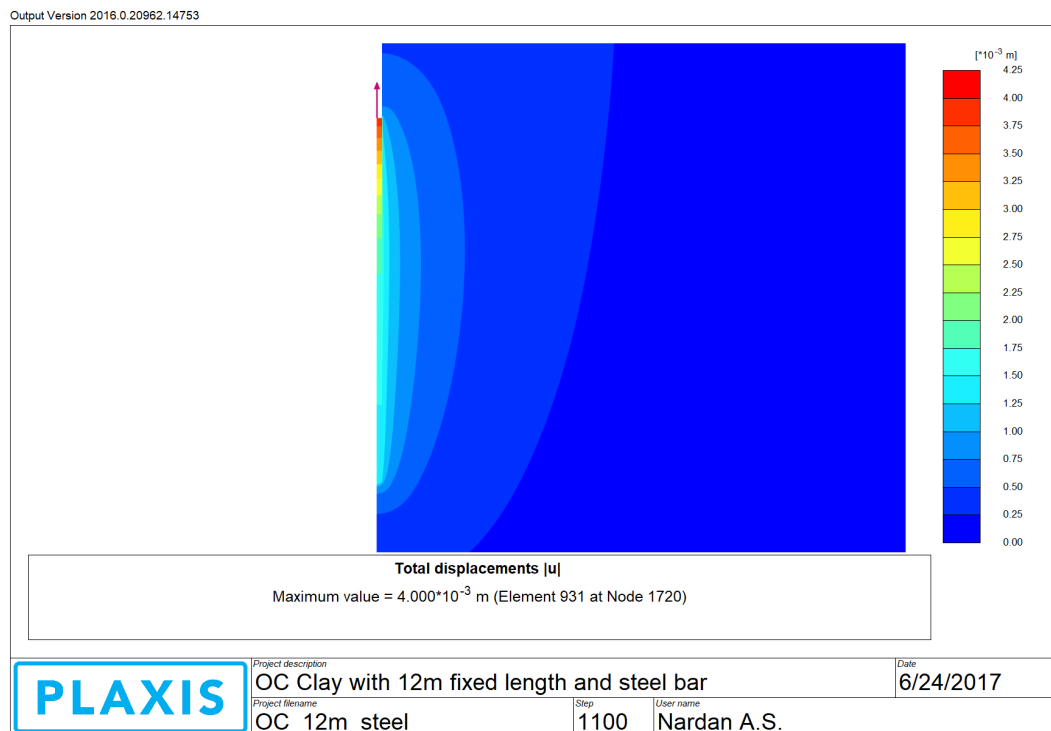


Figure 6.27. Deformation plot for 0.4 cm pull out.

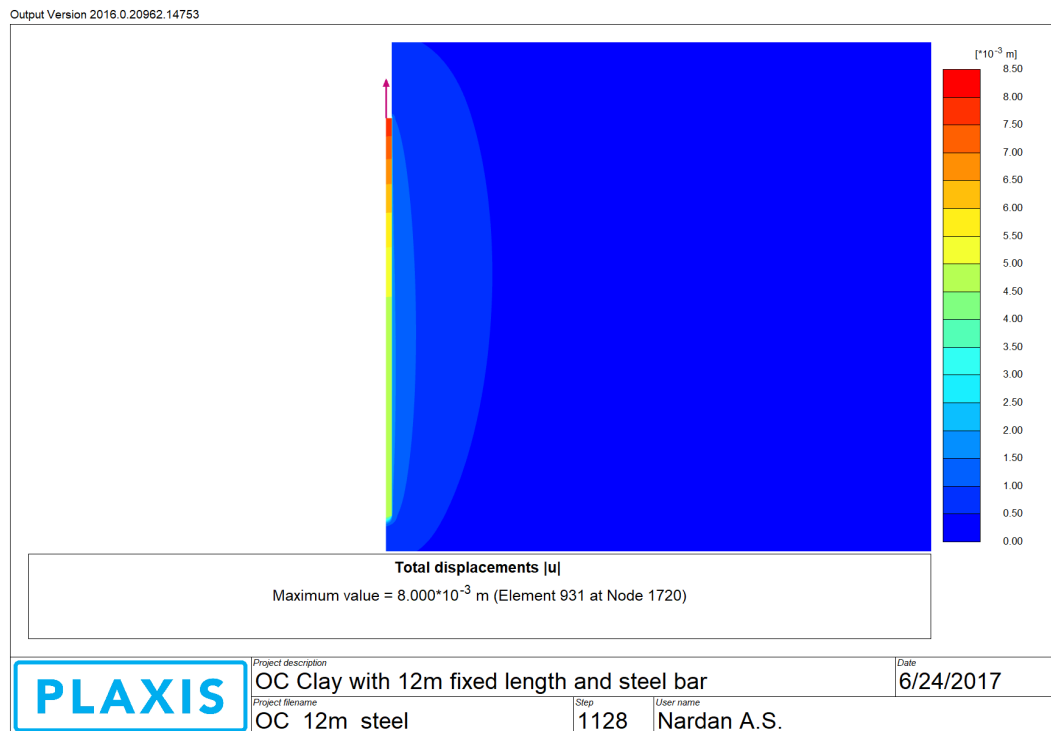


Figure 6.28. Deformation plot for 8mm pull out.

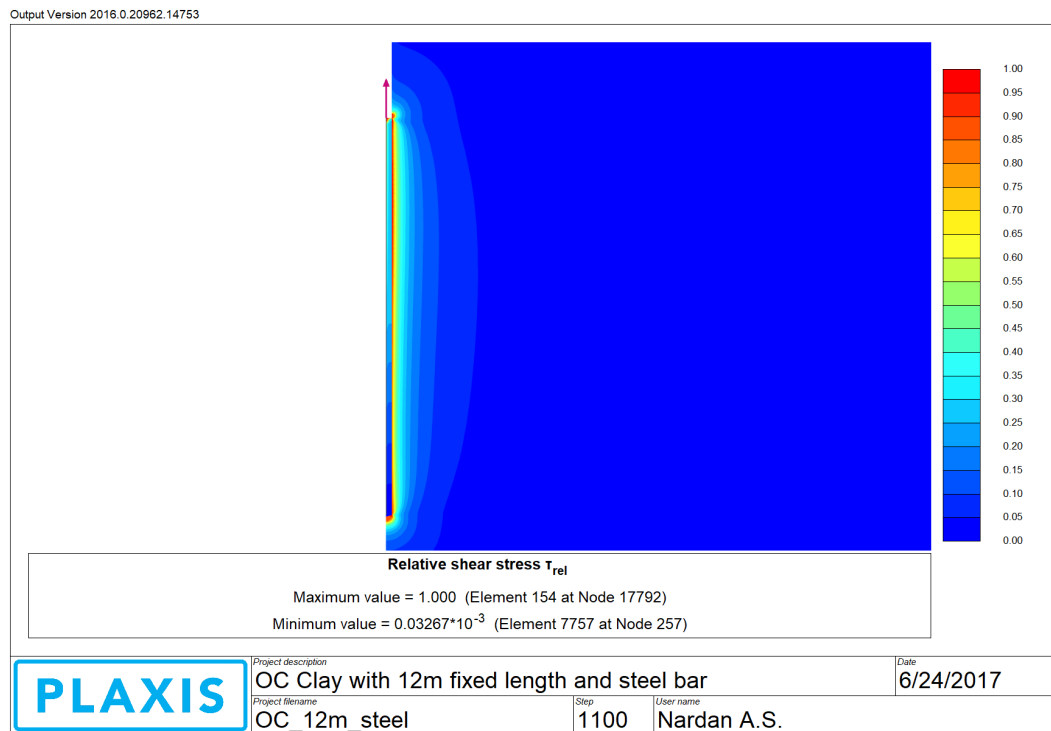


Figure 6.29. Relative Shear plot for 0.4 cm pull out.

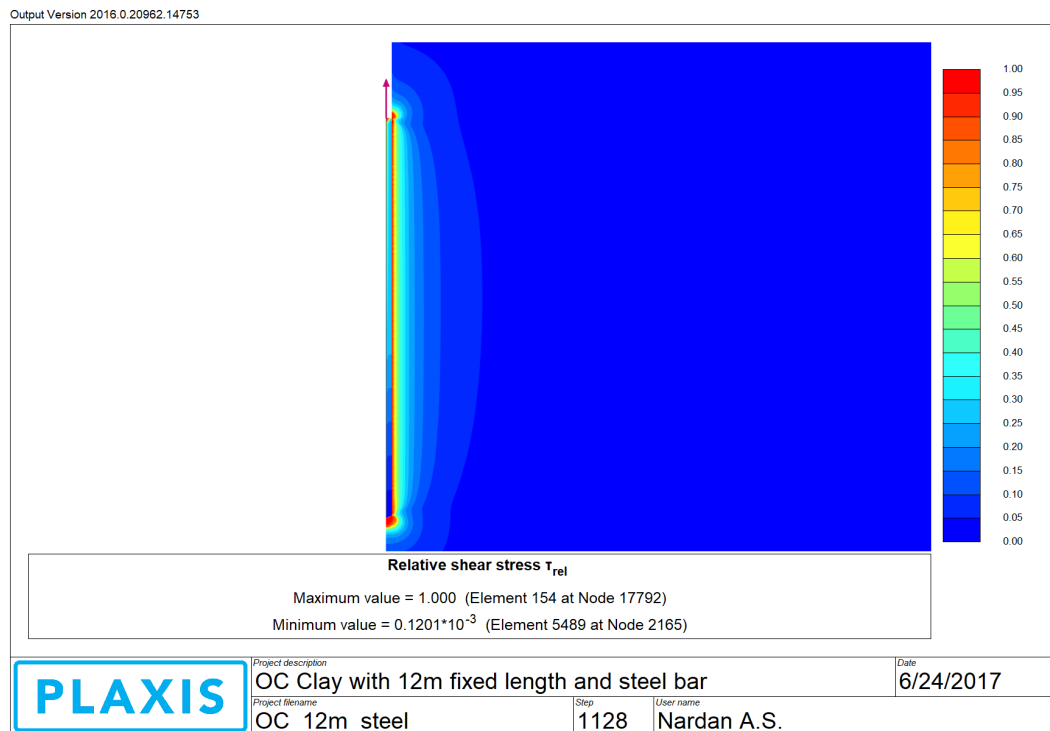


Figure 6.30. Relative Shear plot for 8mm pull out.

Apparently, transmission of the deformations from the proximal to the distal part of the fixed length is in very slow pace. This situation is prominent from the contour colors (mostly light blue) in Figure 6.27. However, in Figure 6.28 anchor capacity is reached before the deformation on the top haven't transferred to the end yet. The pull-out resistance is around 1.67 times larger than ground anchor with 8m fixed length and 2.5 times larger than the ground anchor with 6m fixed length. Graphs for the load-displacement and mobilized shear stress to “deflection on the top of the fixed length” may be seen in Figure 6.31 and Figure 6.32.

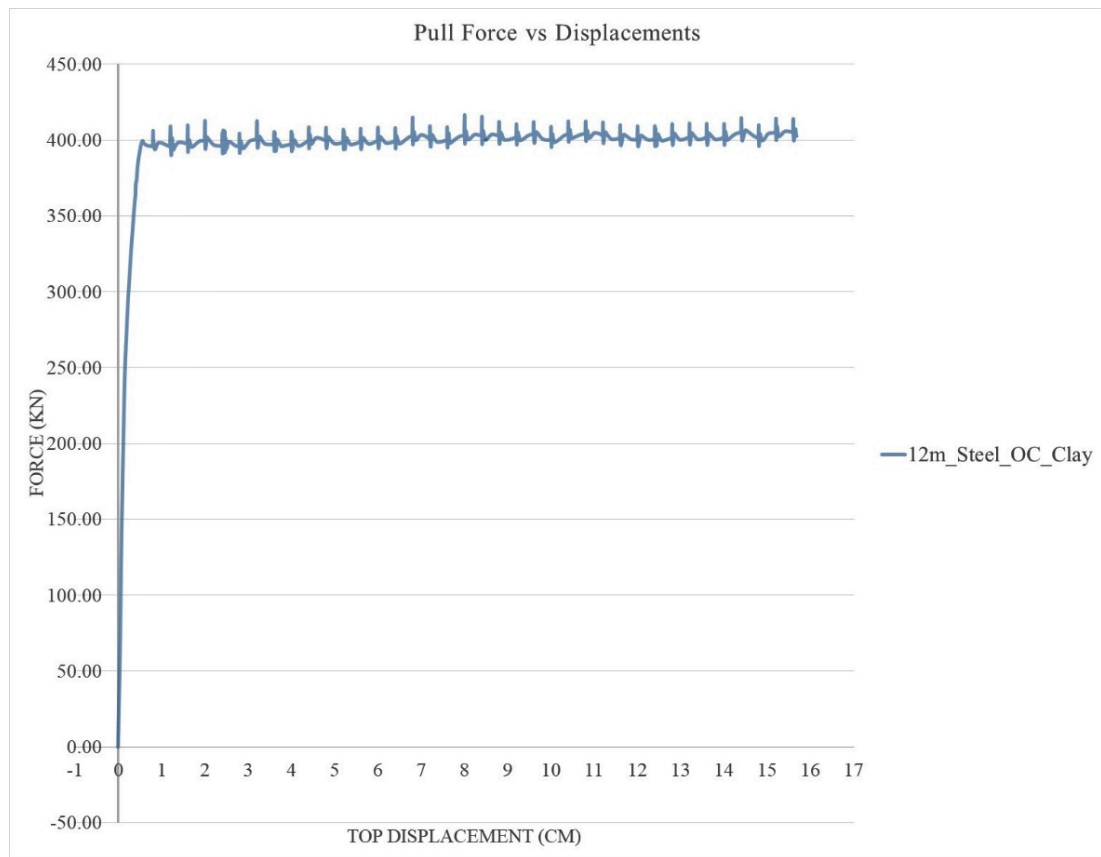


Figure 6.31. Load vs Displacement plot.

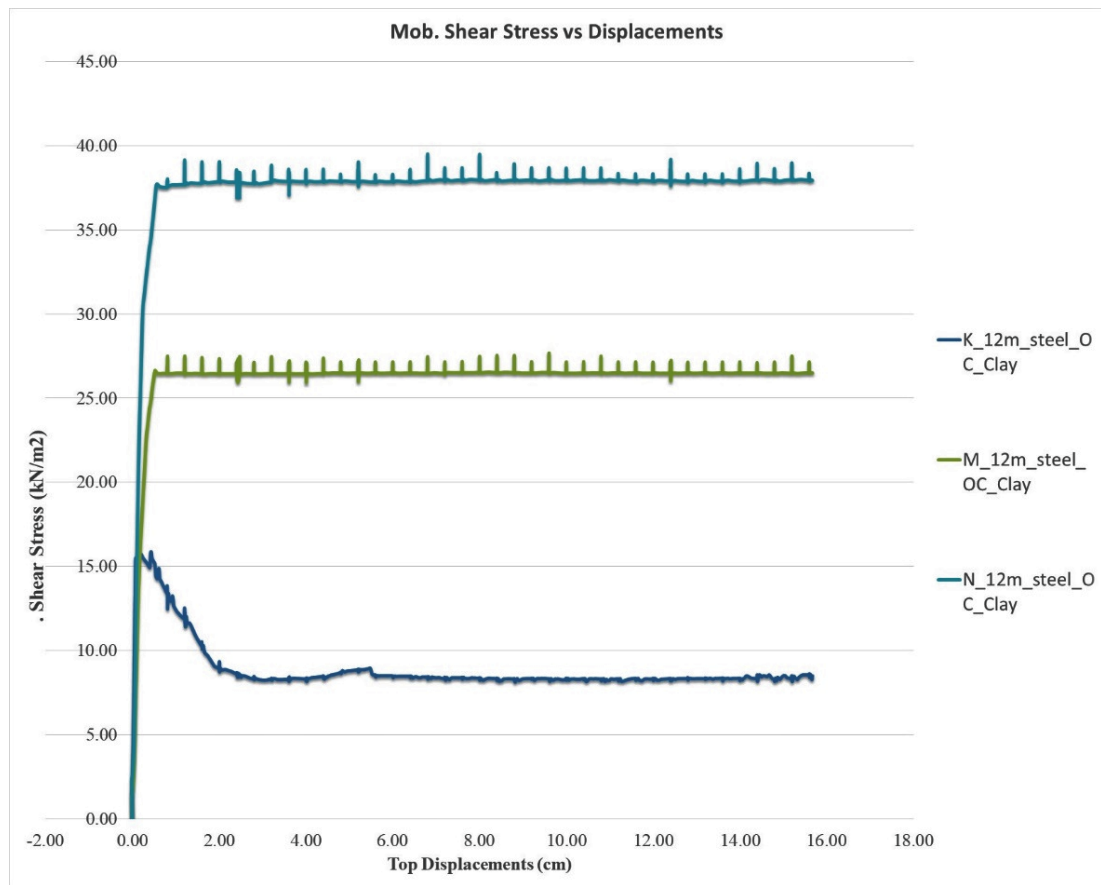


Figure 6.32. Mobilized Shear Stress vs Displacement plot.

The mobilized shear stress on the point which is at the top of the fixed length K, has reached the peak shear stress almost instantly however in this model, more deformation is needed for it to drop to a residual value. The medium point M, and the distal point N, has reached the peak stress value after the point K at a relatively small displacements and stayed at these values until the failure.

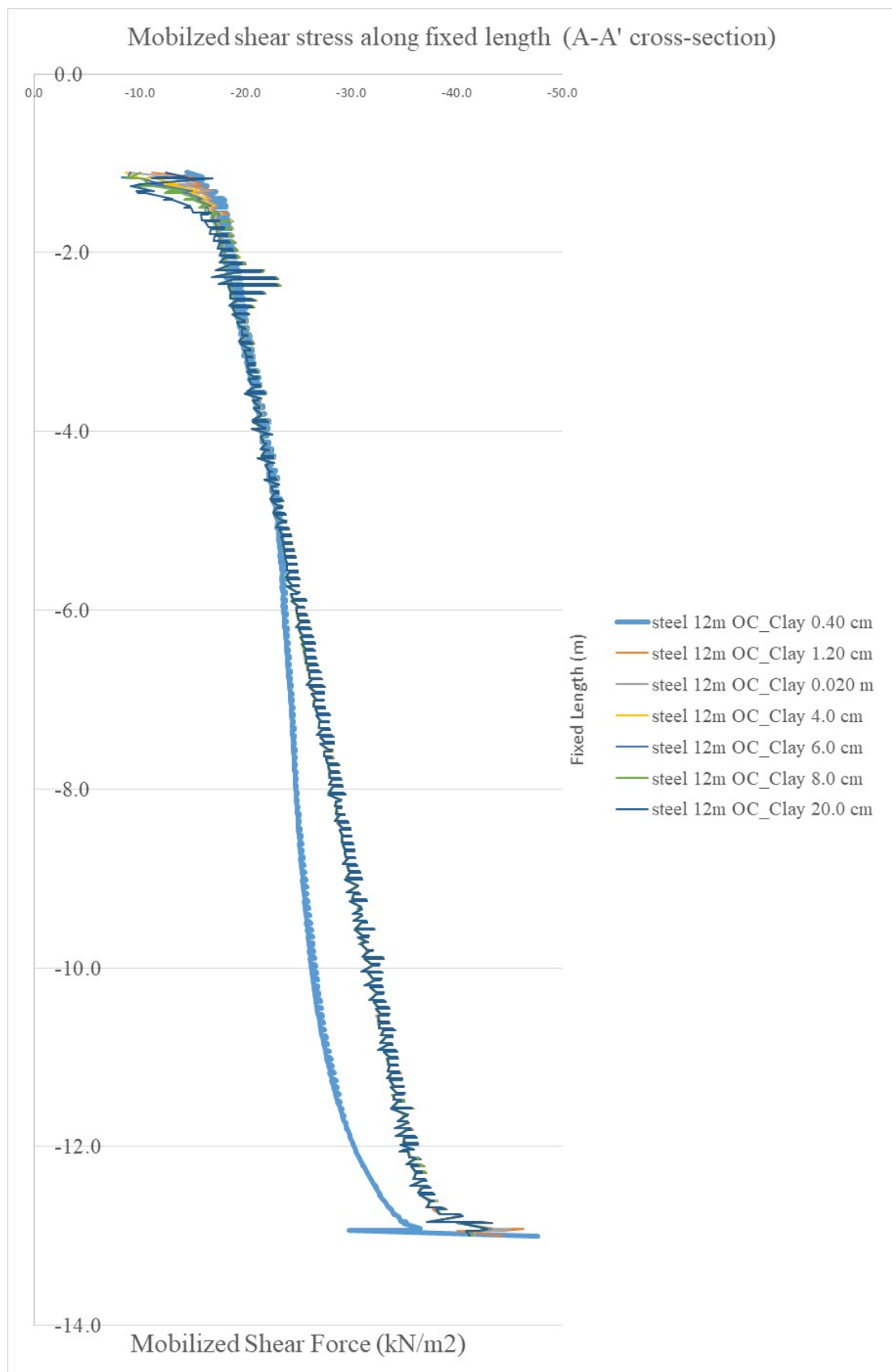


Figure 6.33. Mobilized shear stress for different load levels along A-A' cross-Section.

It can be easily observed from Figure 6.33, shear stress is fully mobilized after 0.5cm of deformation at the top of the ground anchor. This is a distinction in the behaviour of the pull-out resistance of ground anchor by comparing with 8m and 6m fixed length ground anchors. After 8mm deformation at the top there is still a little change in the stress at the top of the fixed length, it slightly decreased to residual level and it slightly increased at the end of the fixed length however, this doesn't change anything in total pull-out force.

6.1.3.2. Ground Anchor with GFRP Bar. Pull-out test of a ground anchor with GFRP bar is simulated for the same conditions with the previous model. In this analysis, the transition from elastic phase to plastic phase occurred after to 0.4 cm pull out (at 1.5cm). Failure in this model occurred at 15mm pull-out therefore the next level of output belonging 16mm (prescribed displacements are in the order of the multiplies of 0.4cm)pull-out has been presented.

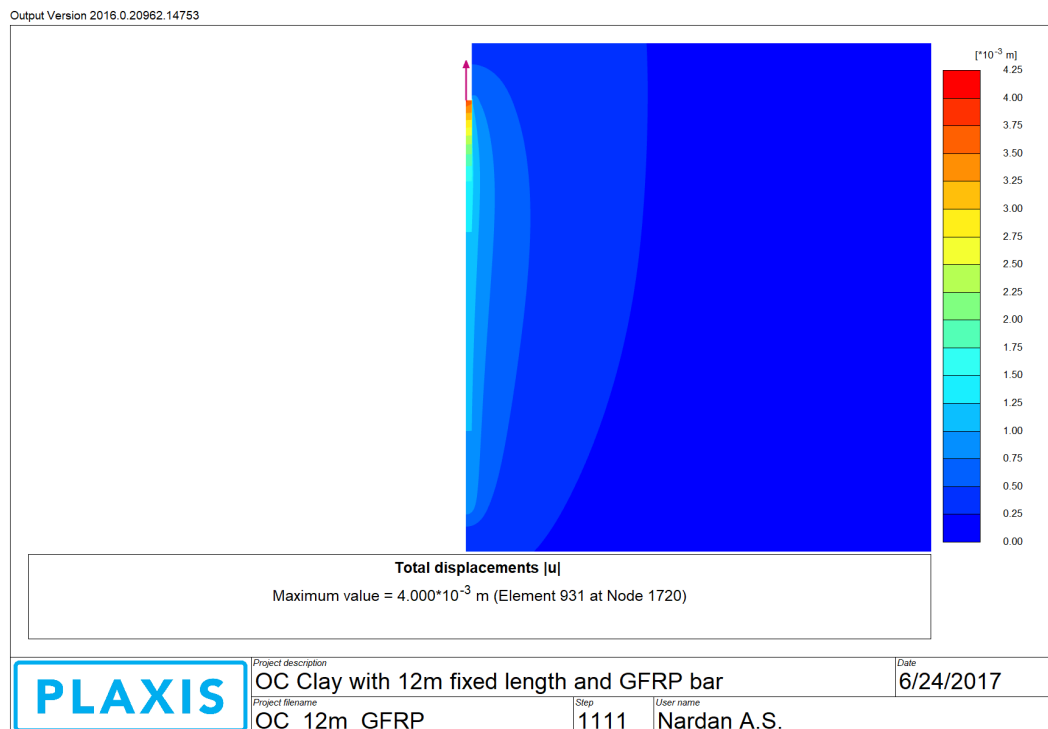


Figure 6.34. Deformation plot for 0.4 cm pull out.

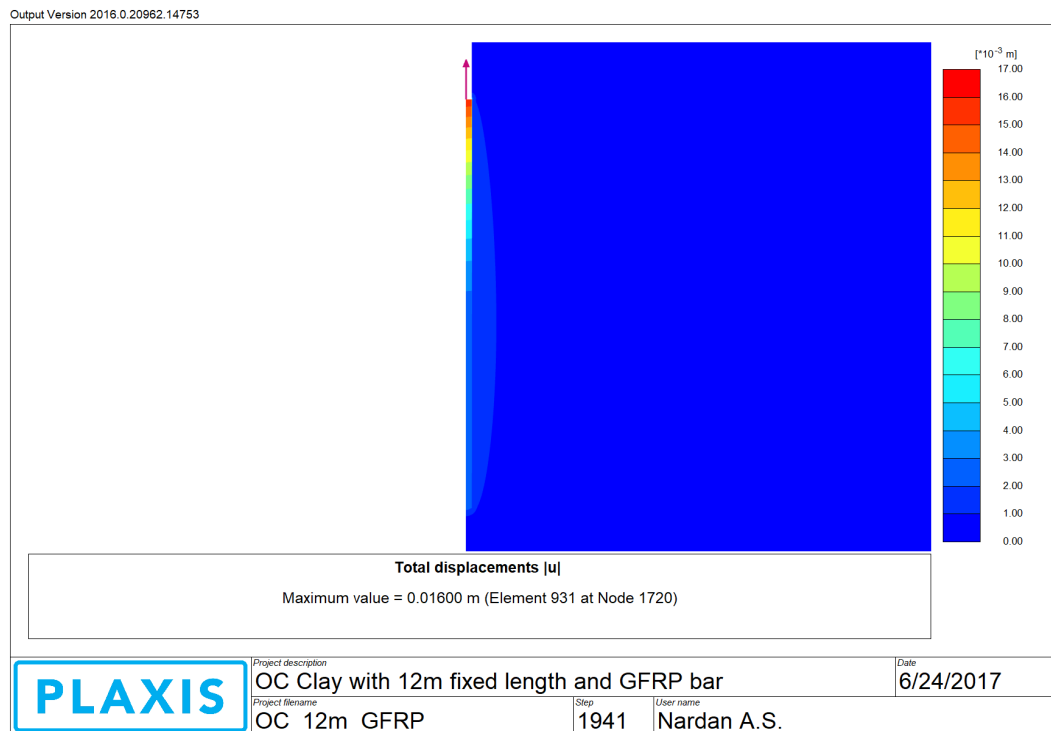


Figure 6.35. Deformation plot for 16mm pull out.

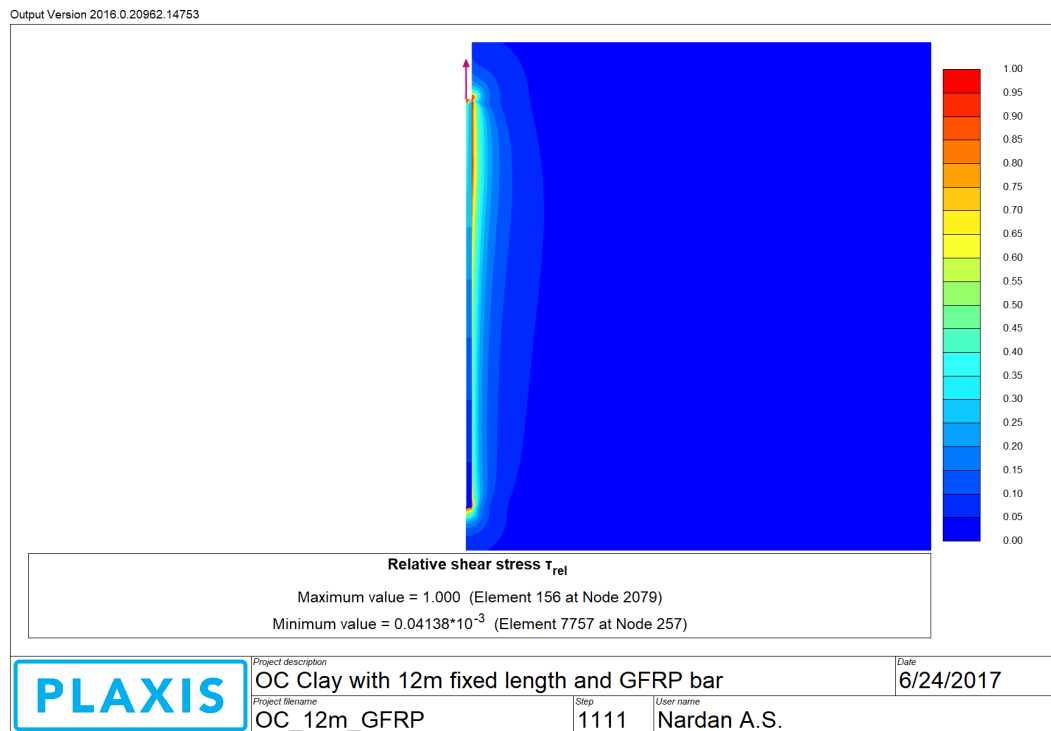


Figure 6.36. Relative Shear Stress plot for 0.4 cm pull out.

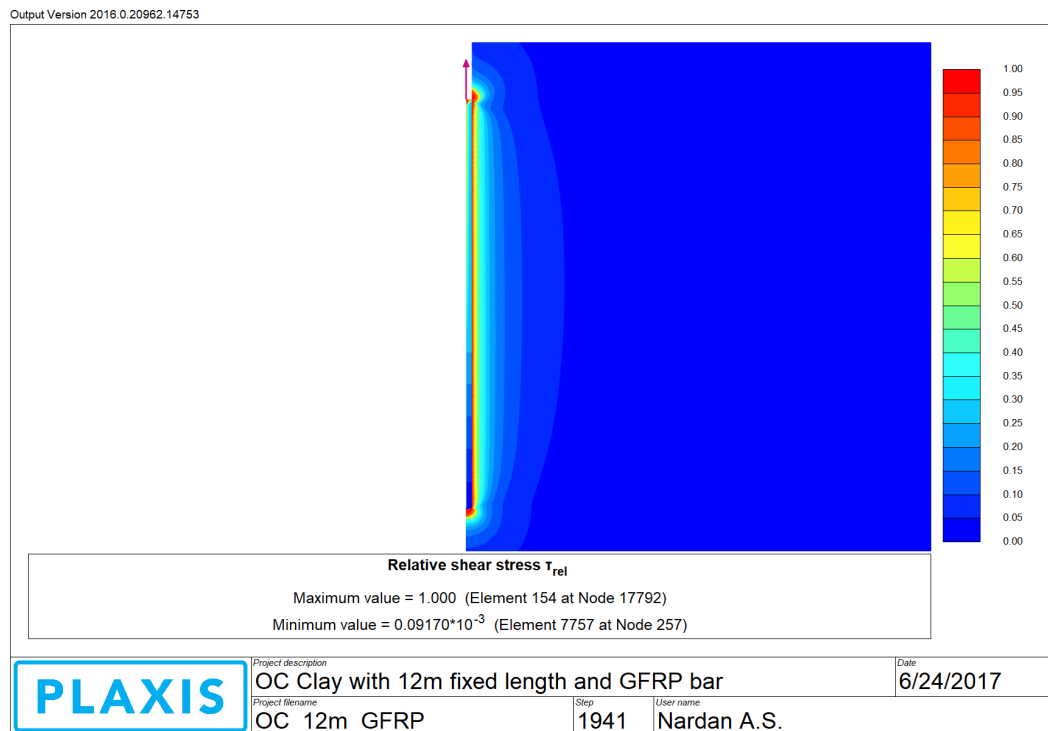


Figure 6.37. Relative Shear plot for 1.6cm pull out.

For this case, the transition from elastic phase to plastic one is occurred at 16mm pull out (first pull out stage). This circumstance is more distinct in this case with GFRP bar. It is very clear from the first and the second deformation plot, the deformations on the top is very slowly being transferred to the end of the fixed anchor. Although the maximum load is reached approximately at 1.6cm, the deformations at the top isn't yet transferred to the center even due to the lower young's modulus of GFRP bar. This situation can be comprehended by following the dark blue to light blue color in the deformation plot. The pull-out resistance is almost same with the one with the steel bar, although it is reached at a deformation of 10mm more than the steel bar. Graphs for the load vs displacement and mobilized shear to "deflection on the top of the fixed length" may be seen below.

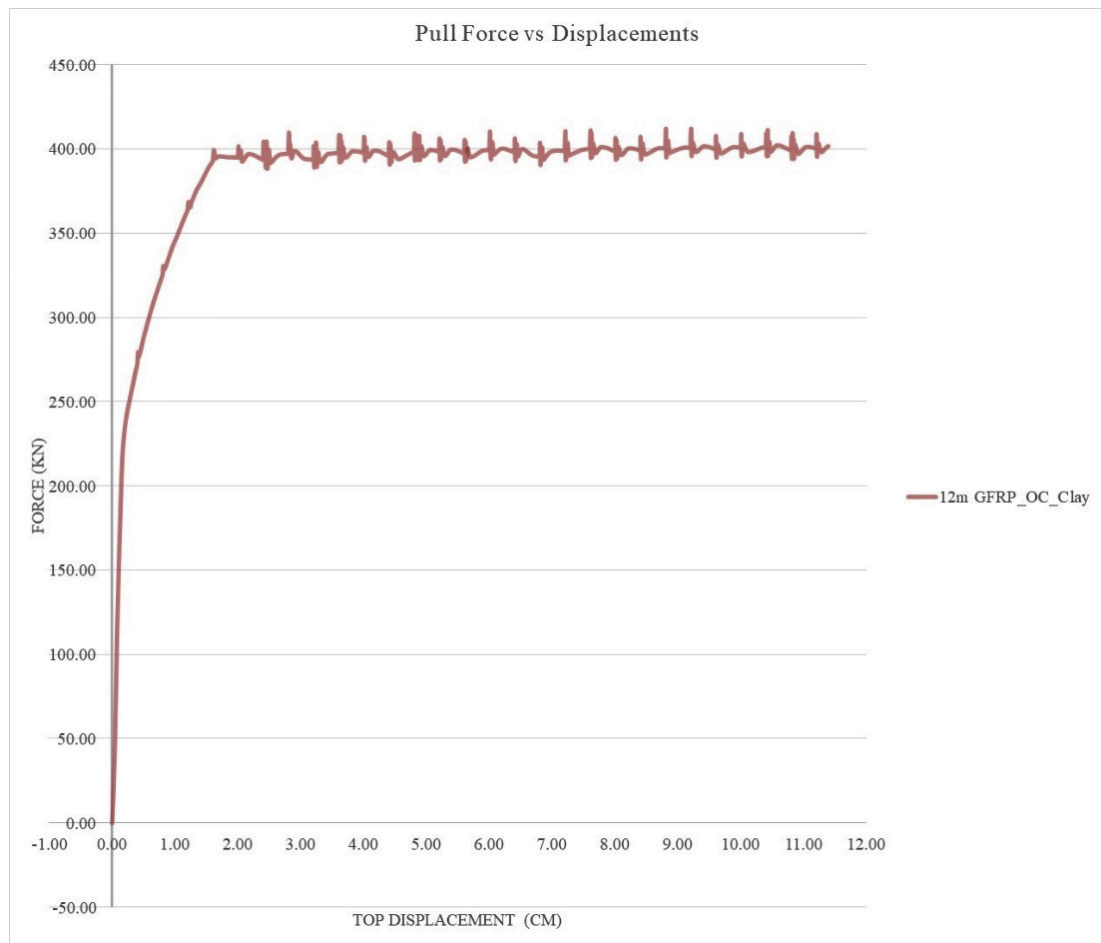


Figure 6.38. Load vs Displacement plot.

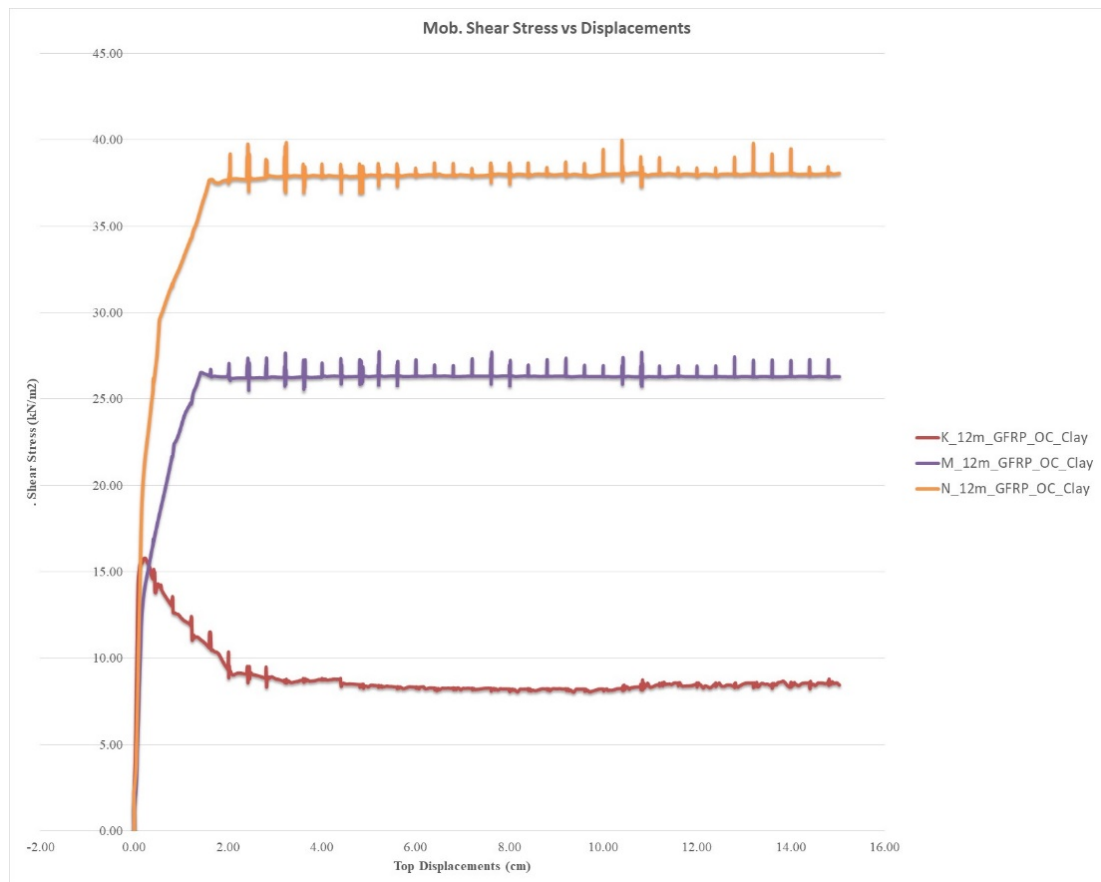


Figure 6.39. Mobilized Shear Stress vs Displacement plot.

The mobilized shear stress on the point which is at the top of the fixed length, K has reached the peak shear stress quickly but more deformations are needed for it to drop to residual value. The medium point, M and the distal point, N has reached the peak stress value after the point K with more deformation needed them to reach that value than 6m fixed length, 8m fixed length and 12m fixed length with steel bar cases.

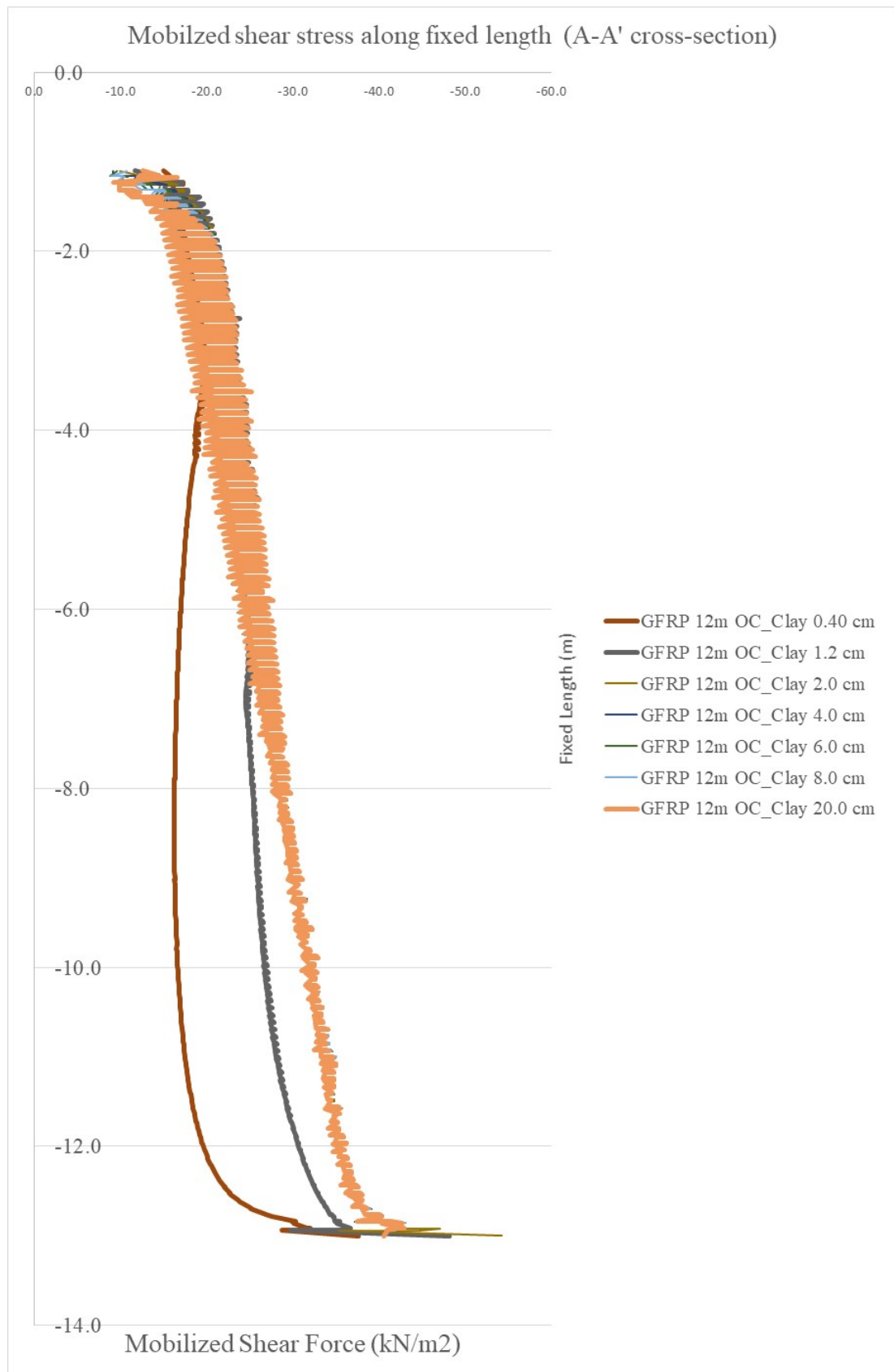


Figure 6.40. Mobilized shear stress for different load levels along A-A' cross-Section.

As can be seen from the Figure above, the shear stress fully mobilized after 2.0cm of deformation at the top of the ground anchor and it is more clearly progressive than the others until now. The shear stress doesn't change remarkably along the shear zone at fixed length after 20mm and the same up and down fluctuation is observable at the top and end of the ground anchor.

6.1.3.3. Comparison of the Results. As it can be observed from the graphs, for 12m fixed length ground anchor with GFRP bar, more deformations are needed to mobilize the full pull-out force. Even though the mobilization of the shear force at the top (point K) of the fixed length is identical in two distinct cases-the rate of deformation which is needed for the mobilization of the full shear stress at shear zone in distal end (of the fixed length), three times more for GFRP when comparing with Steel. The mobilized shear stress at 0.4cm, for steel is in the same order of magnitude with the mobilized shear stress at 1.2cm for GFRP. Even if this proportion seems extreme, the deformations are still comparatively low for practical purposes. Therefore the difference between the GFRP and Steel is not so significant in practice.

The pull-out force-displacement, shear stress to top displacements and Mobilized shear stress distribution along the fixed length (A-A') for two anchor bar types can be seen in Figure 6.41-Figure 6.43.

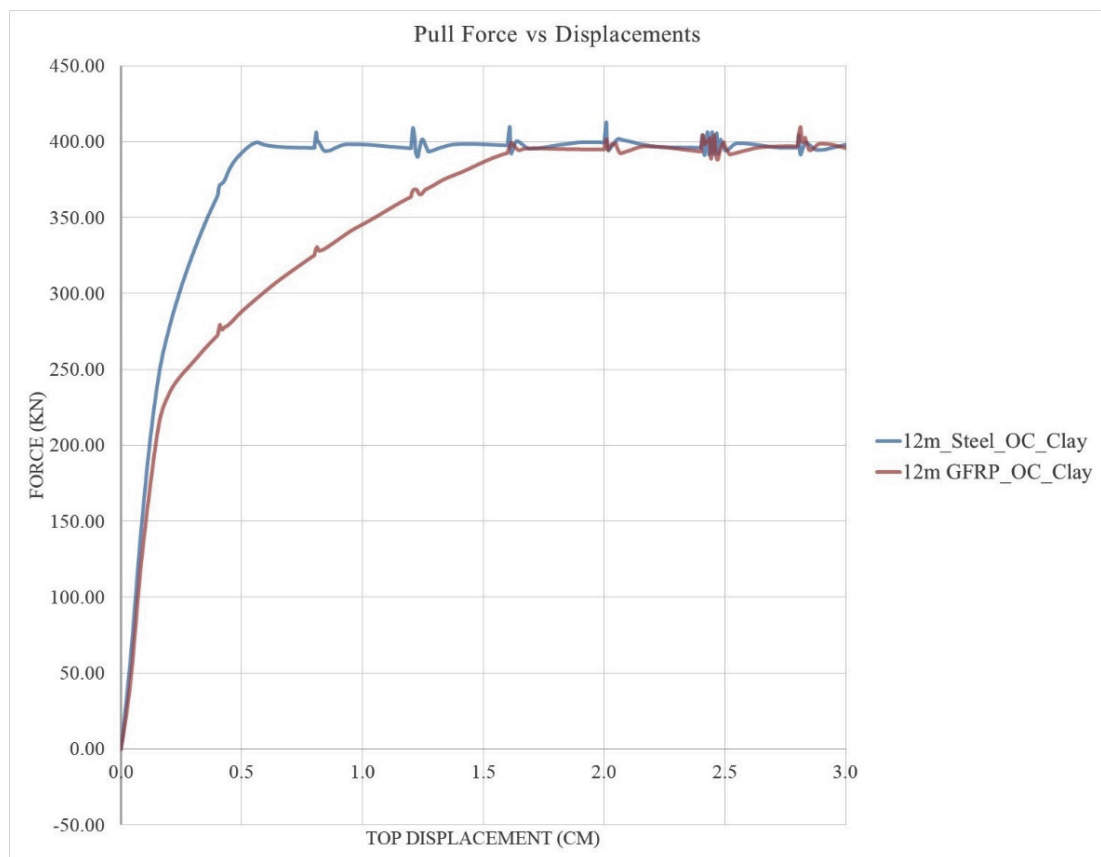


Figure 6.41. Load vs Displacement plot for the comparison of GFRP and Steel Bar.

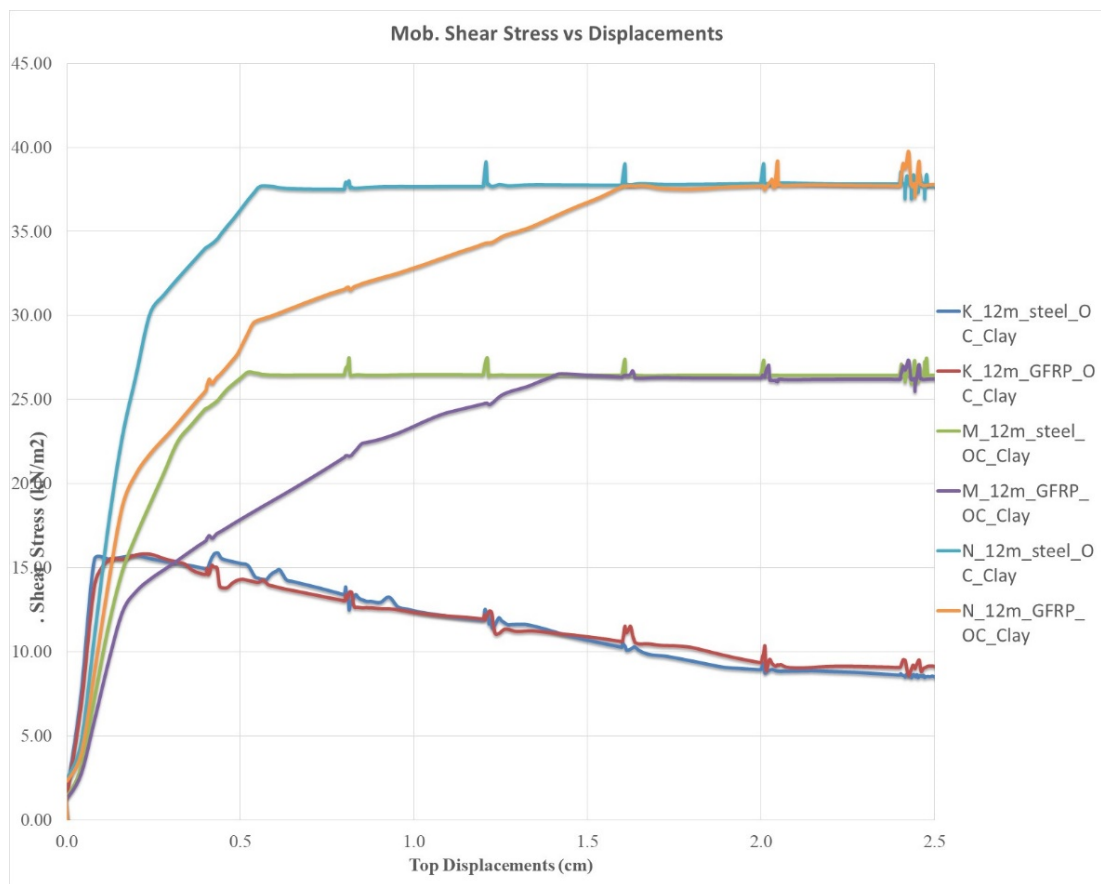


Figure 6.42. Mobilized Shear Stress vs Displacement plot for the comparison of GFRP and Steel Bar.

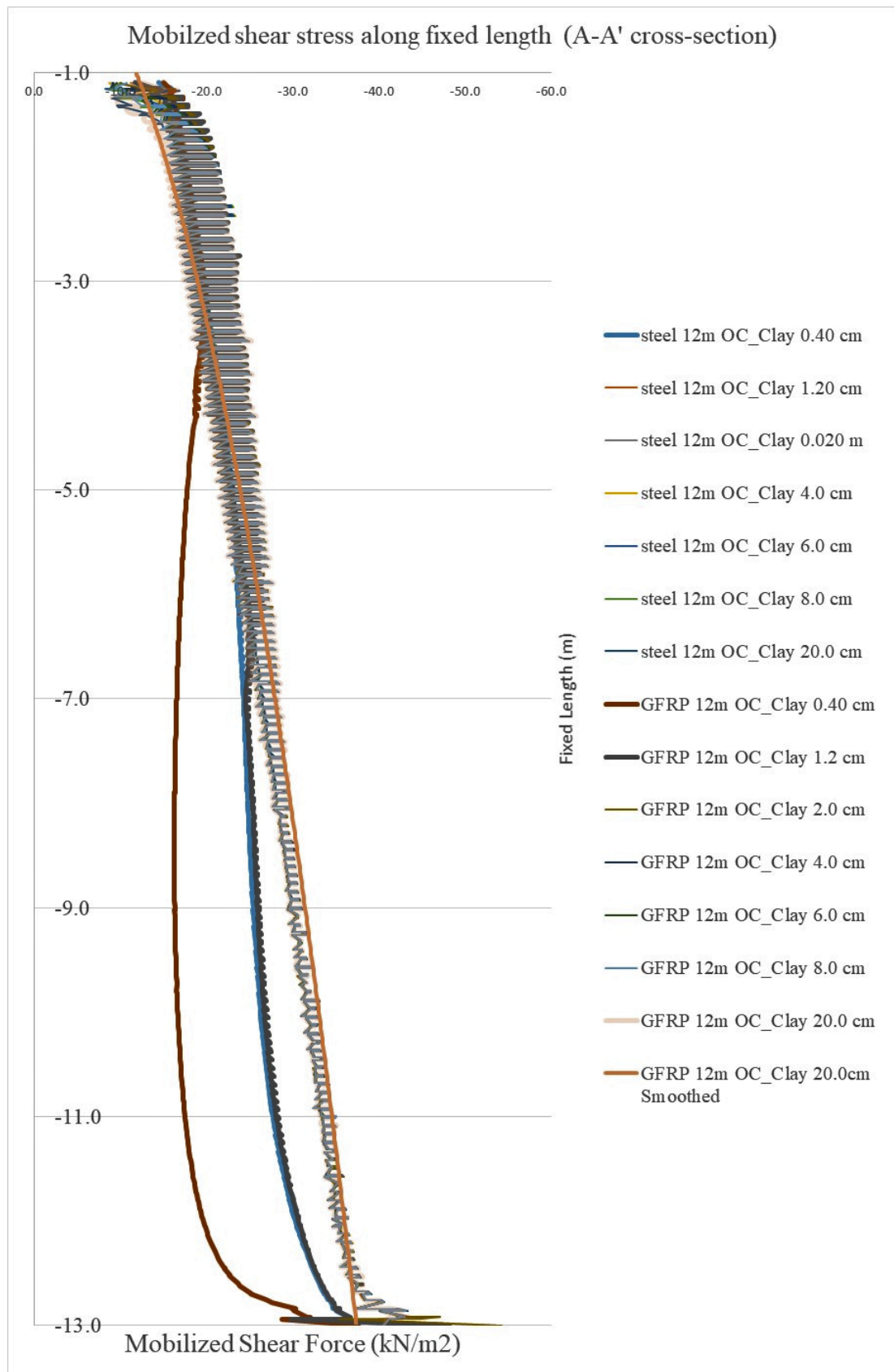


Figure 6.43. Mobilized shear stress for different load levels along A-A' cross-Section for the comparison of GFRP and Steel Bar.

6.2. Dense Sand

This sub-Chapter will present the results of the analyses with the change in fixed lengths respectively (6m, 8m, and 12m). Analyses for each particular length is conducted by changing tendon material (steel and GFRP). First, Plaxis analysis outputs; deformation and relative shear plots at 0.4 cm, several prescribed pull out values will be presented. Later, graphs for the points and cross-Sections described in Section 5.2.4 will be presented respectively.

6.2.1. 6m Fixed Length

6.2.1.1. Ground Anchor with Steel Bar. Pull-out test of a ground anchor with steel bar embedded in dense sand soil is simulated in this model. The fixed length of the ground anchor is 6m. In this analysis, the transition from elastic phase to plastic phase occurred prior to 0.4 cm pull out (at 0.2cm). Therefore only the deformation and relative shear plots for 0.4cm pull out will be presented in Figure 6.44 and Figure 6.45.

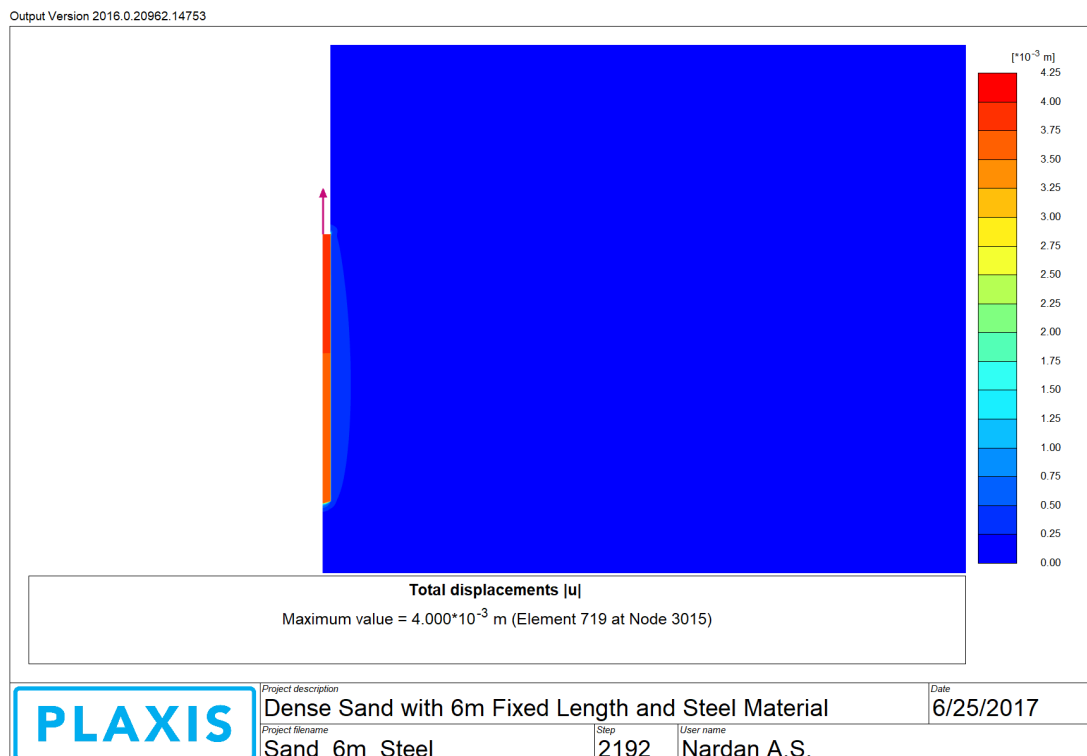


Figure 6.44. Deformation plot for 0.4 cm pull out.

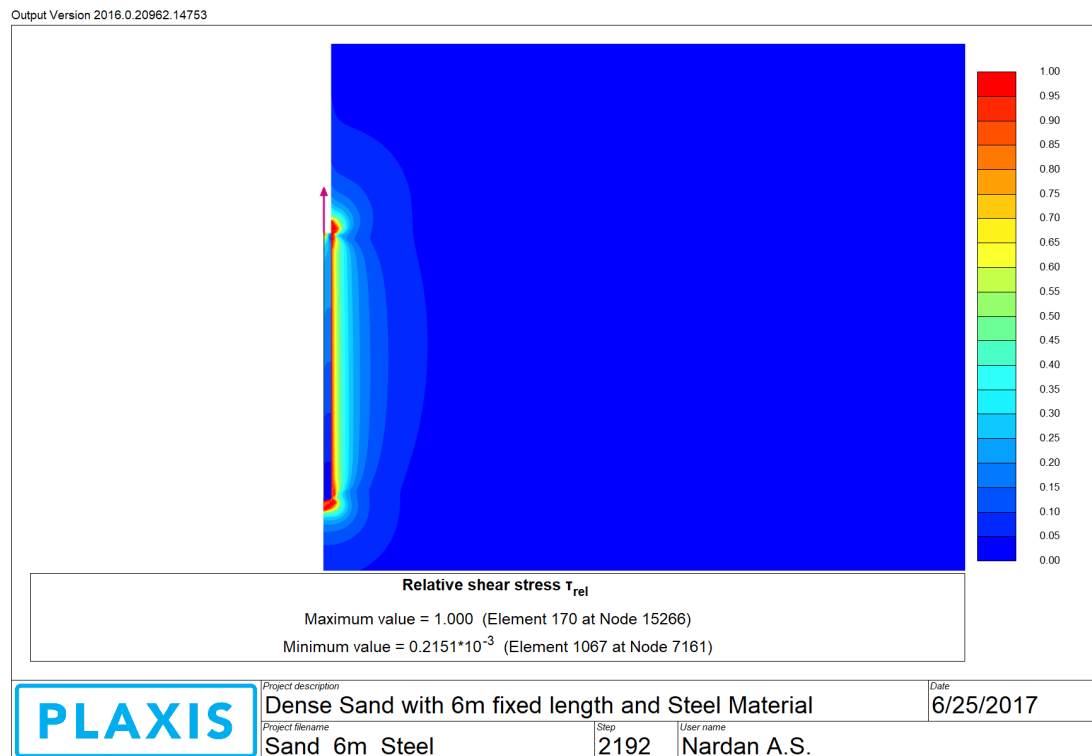


Figure 6.45. Relative Shear plot for 0.4 cm pull out.

Graphs for the load-displacement and mobilized shear stress versus “deflection on the top of the fixed length” may be seen in Figure 6.46 and Figure 6.47.

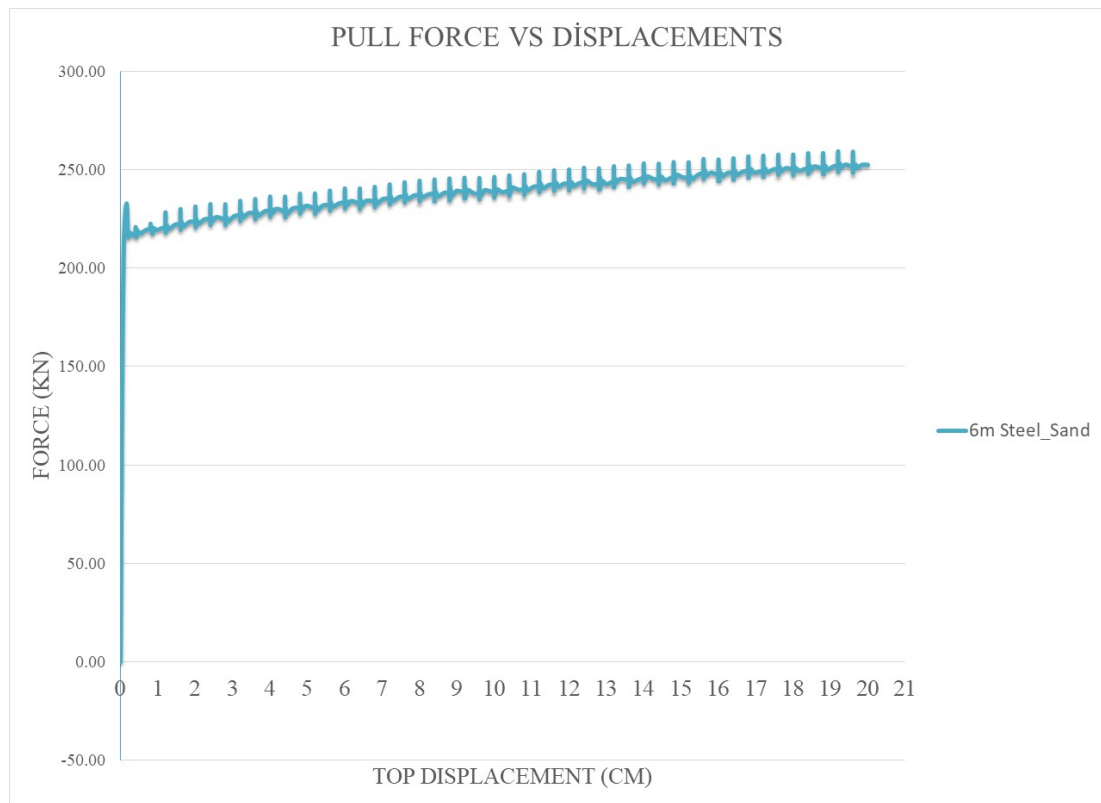


Figure 6.46. Load vs Displacement plot.

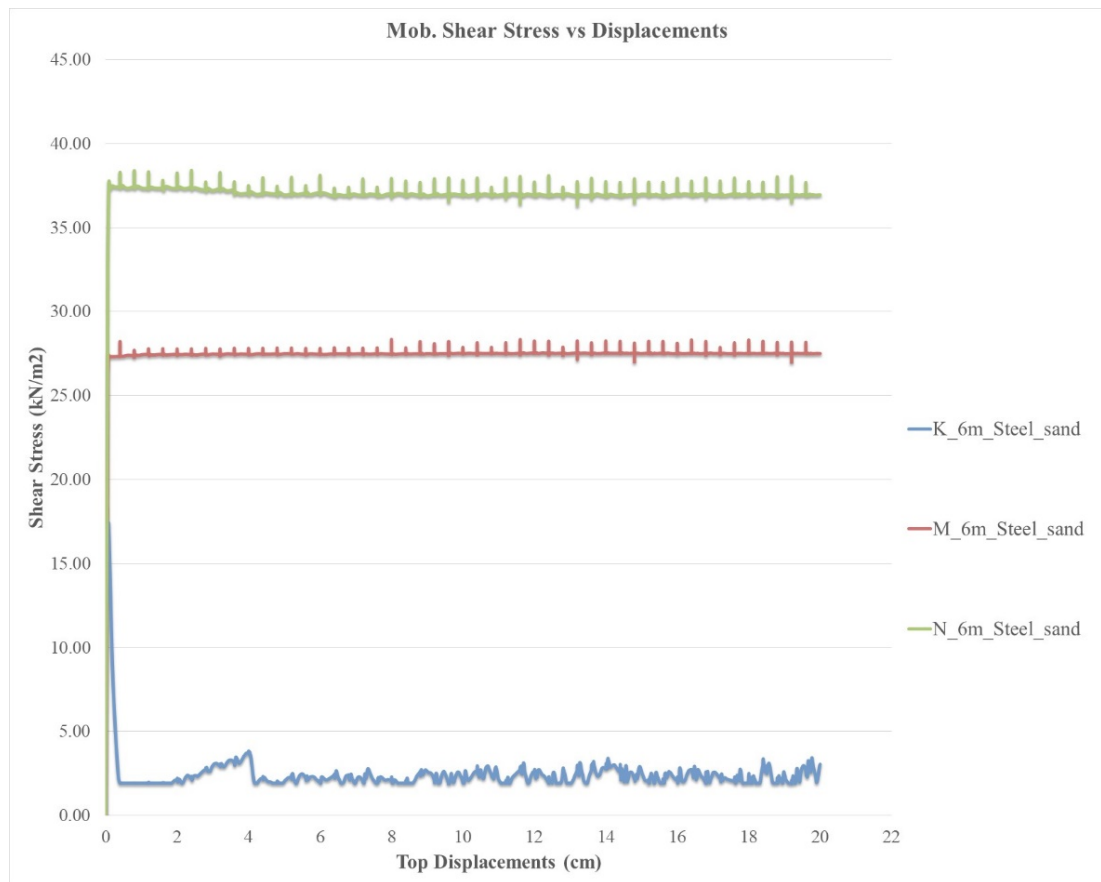


Figure 6.47. Mobilized Shear Stress vs Displacement plot for points K, M and N in the FE mesh.

The mobilized shear stress on the point which is at the top of the fixed length K, has reached the peak shear stress almost instantly, as the displacement continues, it dropped to a residual value as expected. The medium point M, and the distal point N, has reached the peak stress value almost at the same deformation value with the point K and stayed at these reached values until failure.

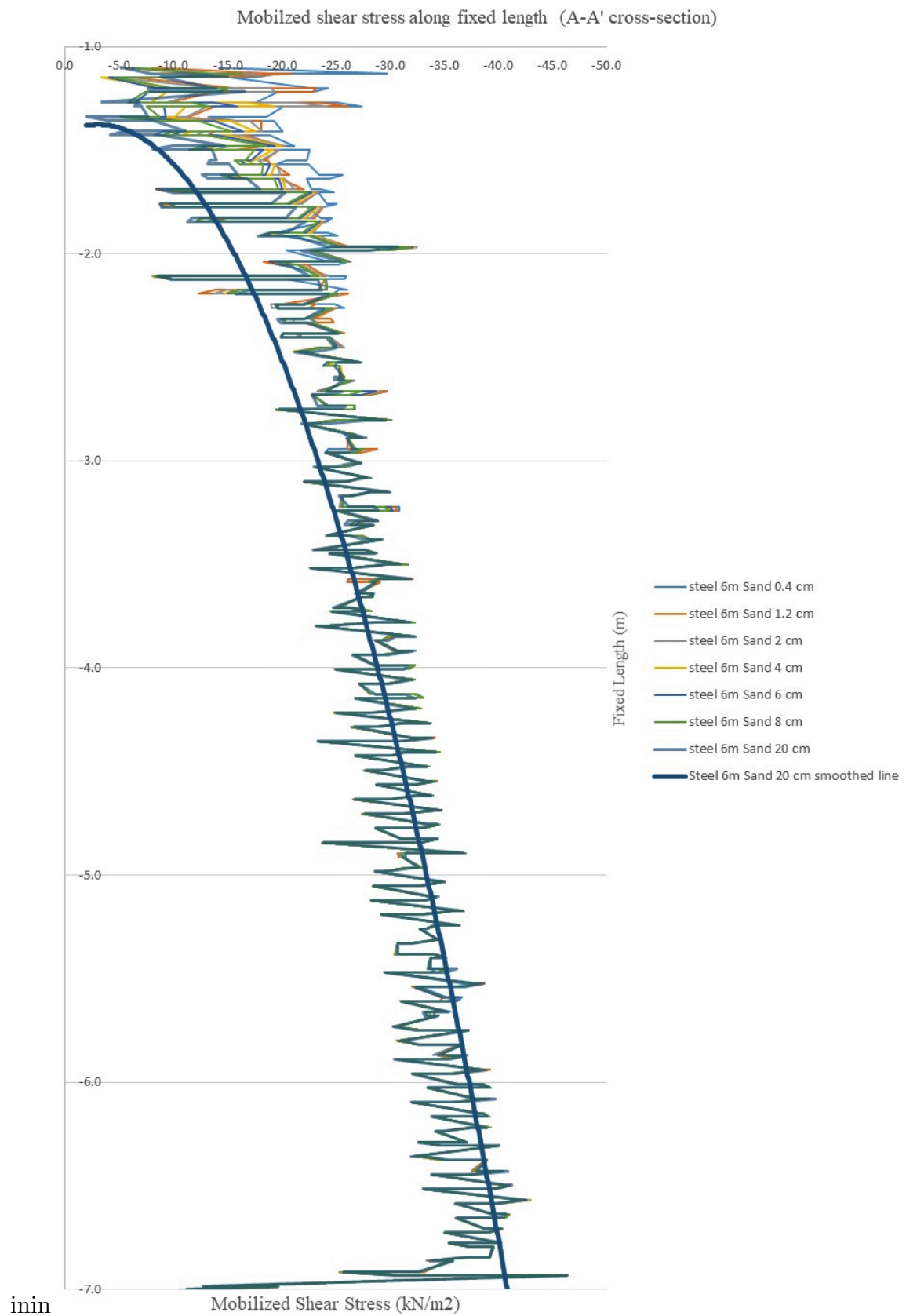


Figure 6.48. Mobilized shear stress for different load levels along A-A' cross-Section.

As can be seen from Figure 6.48, the shear stress doesn't change remarkably along the fixed length zone. The Maximum pull-out force mobilized at 0.18 cm deformation, no remarkable difference is apparent but the change in the stress at the top of the fixed length decreased to residual level again like in over consolidated clay.

6.2.1.2. Ground Anchor with GFRP Bar. Pull-out test of a ground anchor with GFRP bar is simulated for the same conditions with the previous model. In this analysis, the transition from elastic phase to plastic phase occurred prior to 0.4 cm pull out (at 0.18 cm) likewise the previous model. Therefore only the deformation and relative shear plots for 0.4cm pull out will be presented in Figure 6.49 and Figure 6.50.

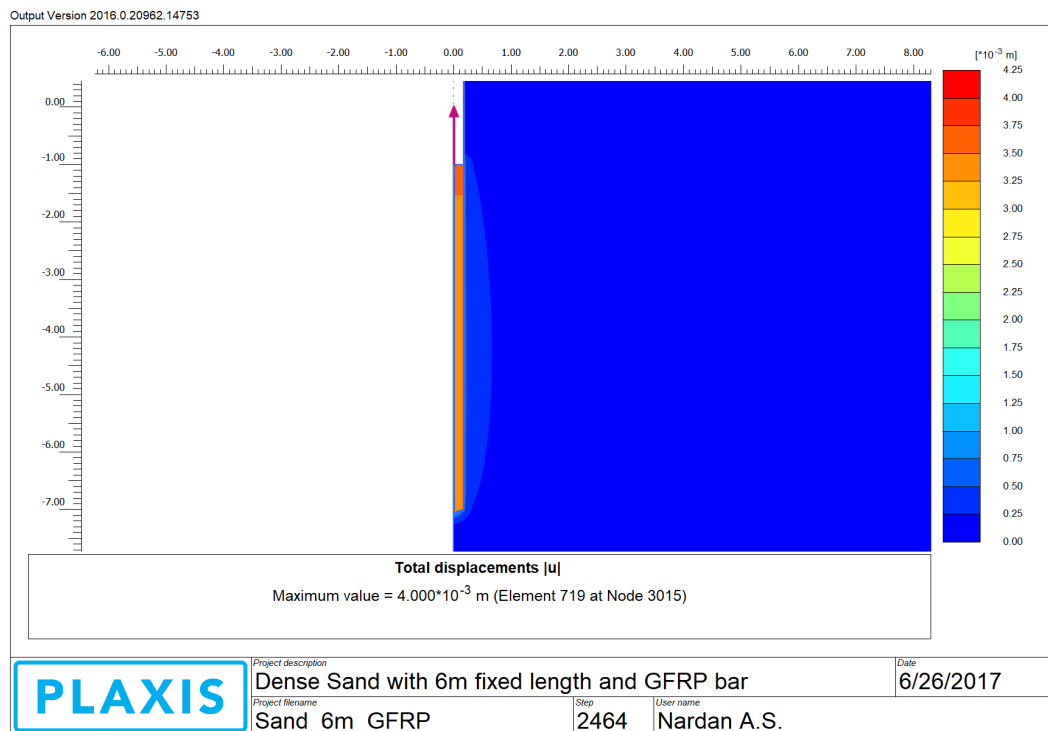


Figure 6.49. Deformation plot for 0.4 cm pull out.

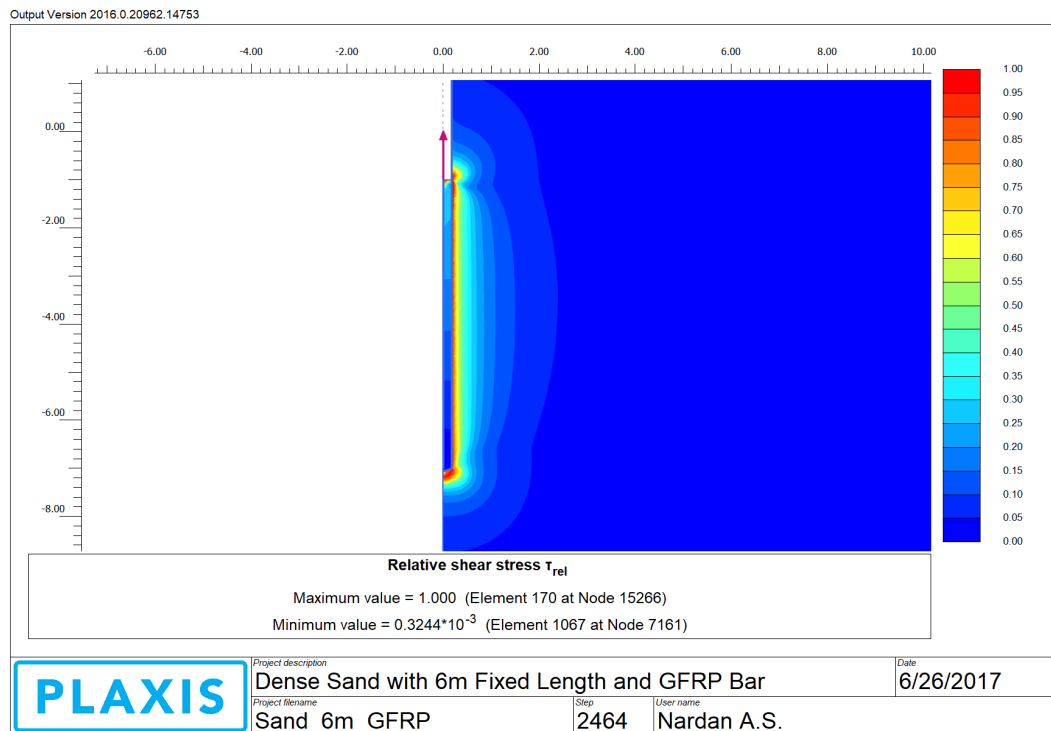


Figure 6.50. Relative Shear plot for 0.4 cm pull out.

Graphs for the load vs displacement and mobilized shear stress to “deflection on the top of the fixed length” may be seen in Figure 6.51 and Figure 6.52.

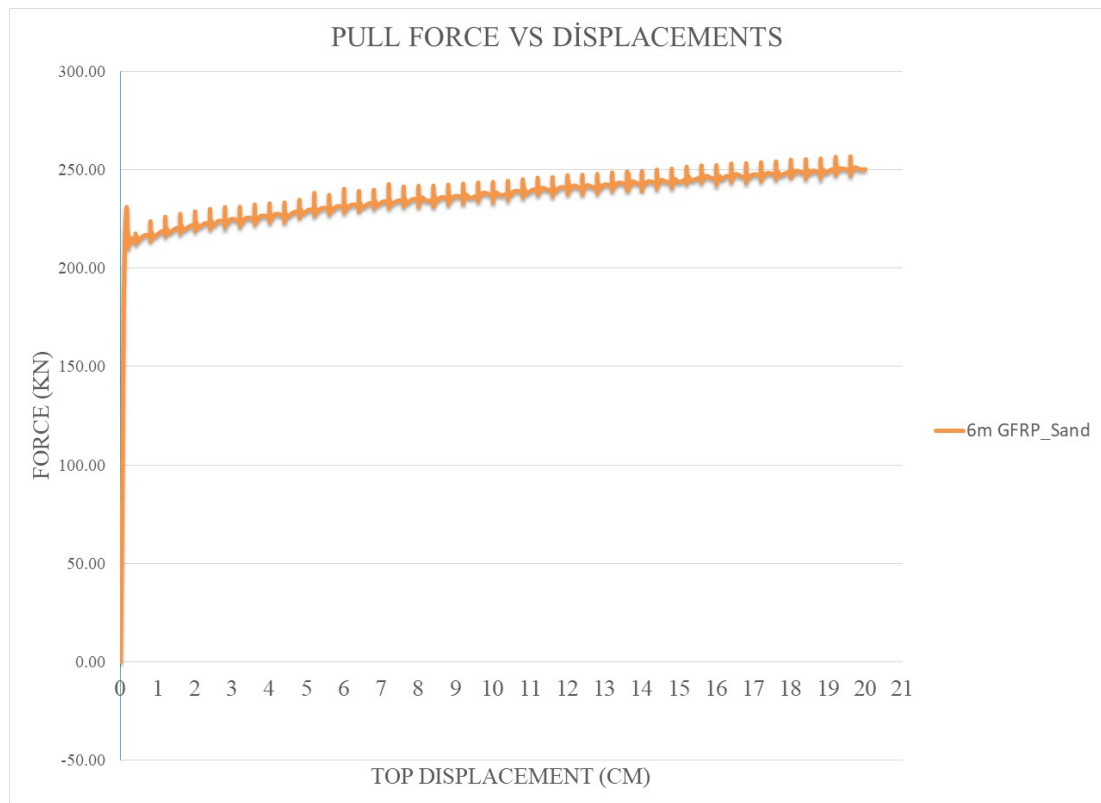


Figure 6.51. Load vs Displacement plot.

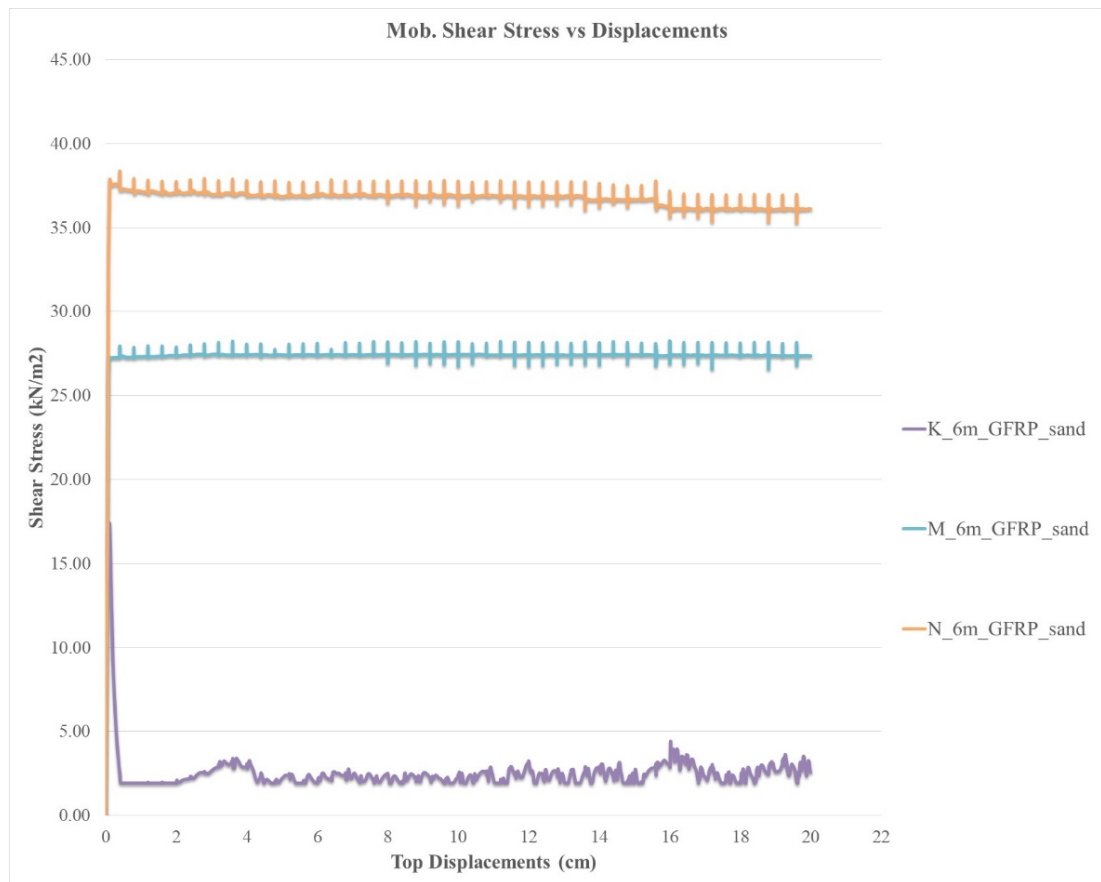


Figure 6.52. Mobilized Shear Stress vs Displacement plot for points K, M and N in the FE mesh.

As it can be expected, the behaviours of points; K, M, N for the ground anchor with GFRP bar and the steel bar are identical for this case, too. The mobilized shear stress on the point which is at the top of the fixed length K, has reached the peak shear stress almost instantly and as the displacement continues, it dropped to a residual value as expected. The medium point M, and the distal point N, has reached the peak stress value almost at the same deformation value with the point K and stayed at these values until the failure. As can be seen from Figure 6.53, the shear stress doesn't change remarkably along the fixed length. The failure happened at 0.2cm no difference is apparent but the change in the stress at the top of the fixed length decreased to residual level.

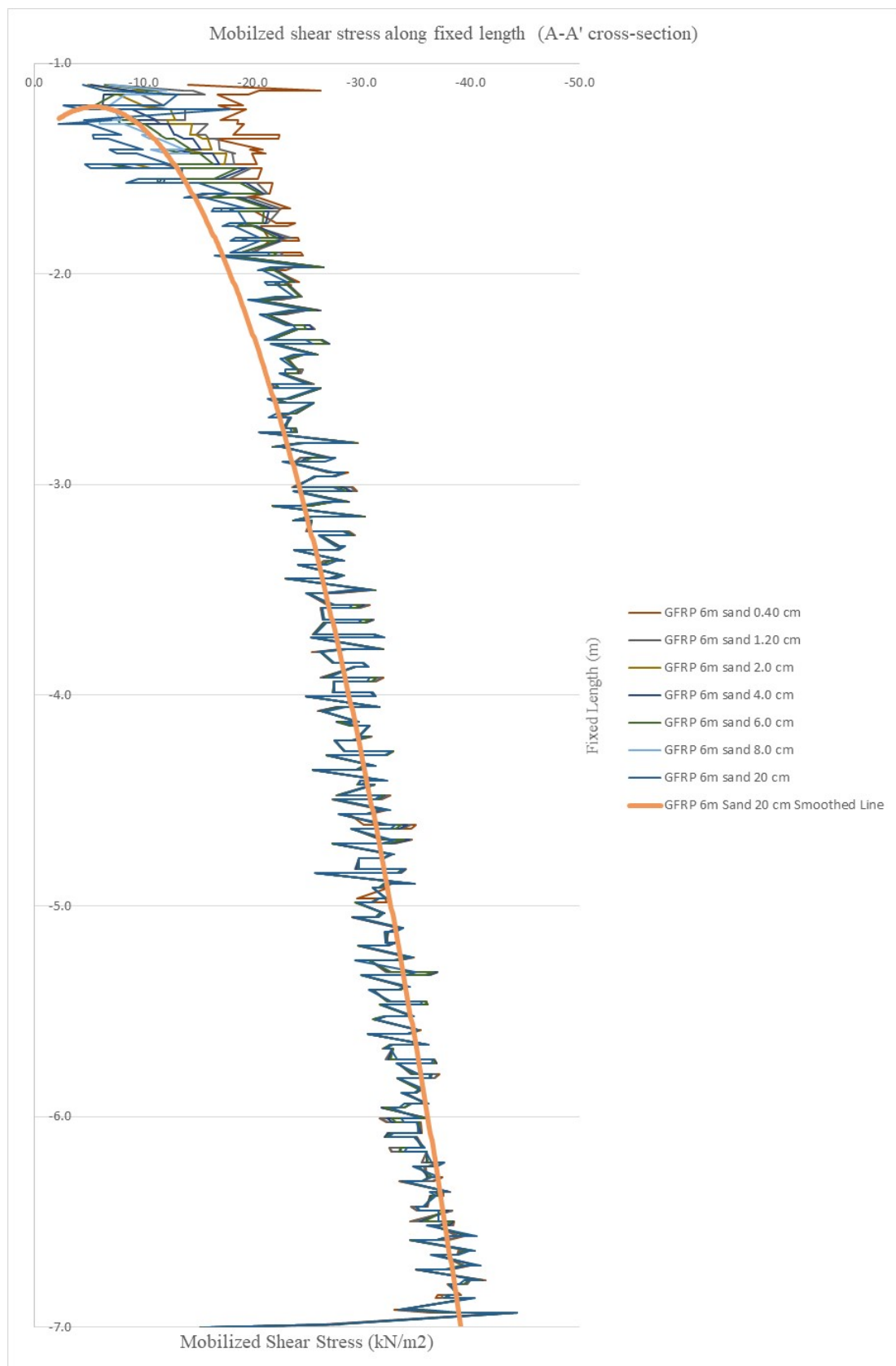


Figure 6.53. Mobilized shear stress for different load levels along A-A' cross-Section.

6.2.1.3. Comparison of the Results. As it can be anticipated, at this stress level in the dense sand material case with 6m fixed length, there is no significant difference between ground anchors with GFRP bar and conventional steel bar. However, in this case (dense sand) the load-displacement behaviour is not perfectly plastic but both ground anchor models Conventional steel and GFRP) experienced an increase in capacity with increased pull-out displacement. Because both of the material experienced the same behaviour and there is no change in resulting ultimate load reached in the analysis. This behaviour will be addressed in the next Chapter. The comparison of conventional Steel versus GFRP at the pull force-displacement graph may be seen in Figure 6.54.

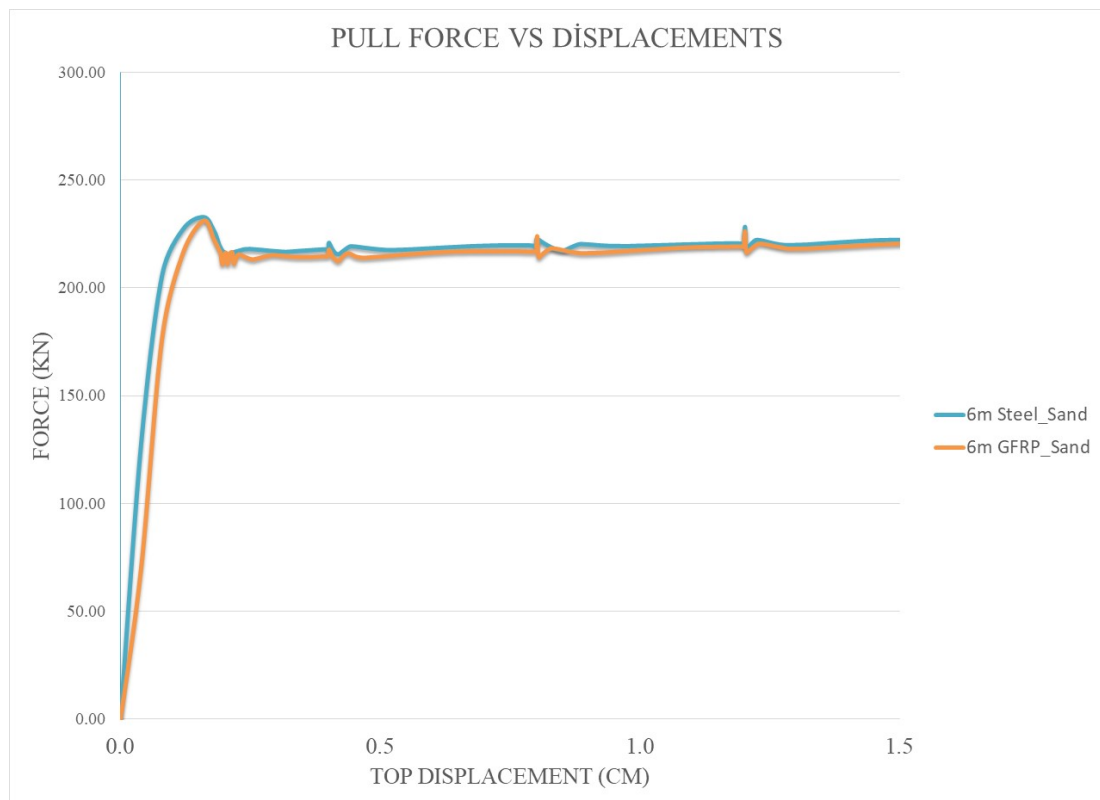


Figure 6.54. Load vs Displacement plot for the comparison of GFRP and Steel Bar.

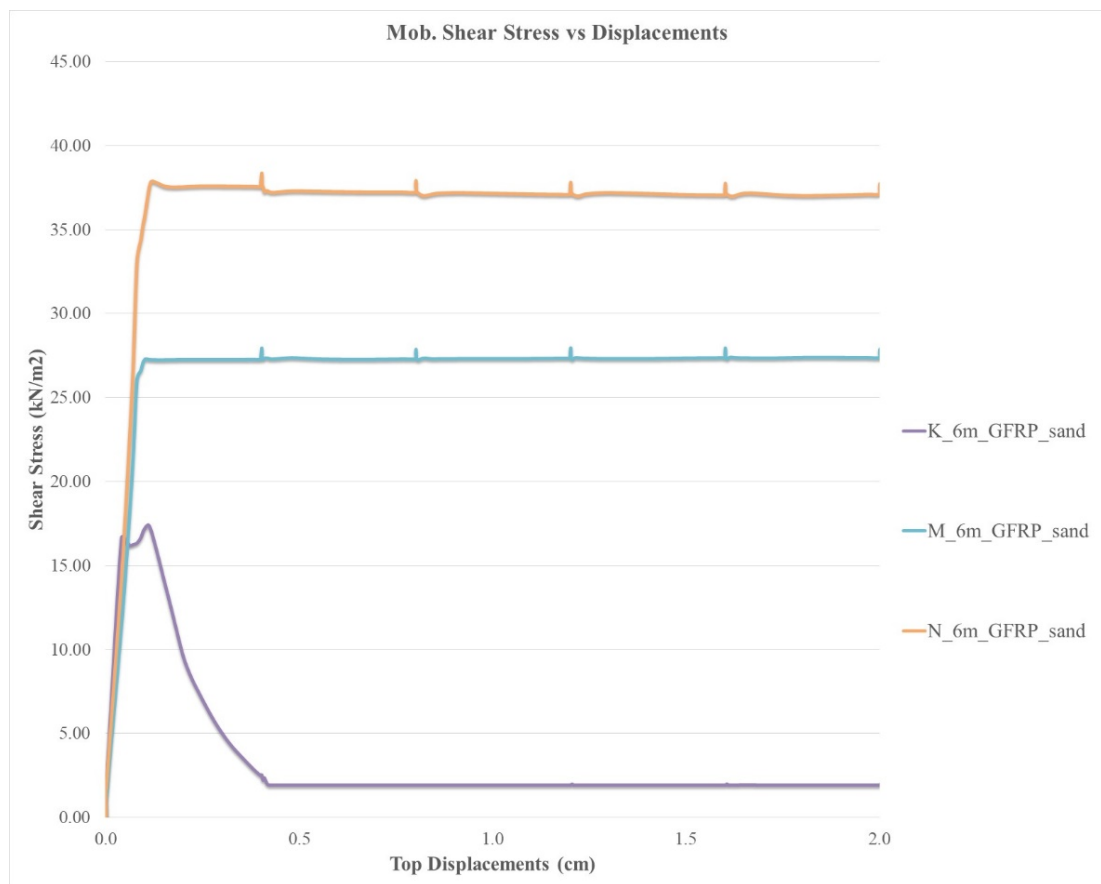


Figure 6.55. Mobilized Shear Stress vs Displacement plot for the comparison of GFRP and Steel Bar.

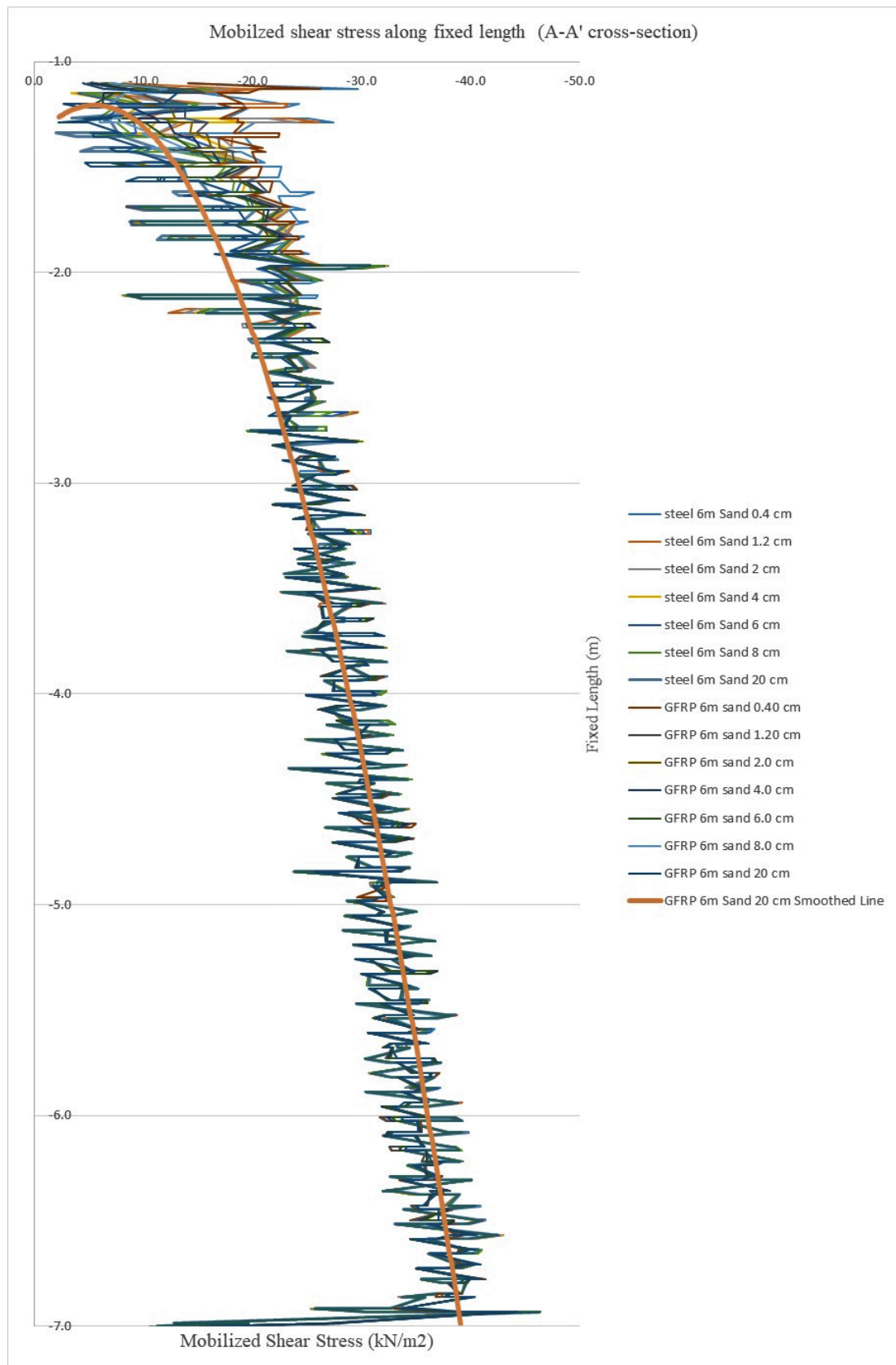


Figure 6.56. Mobilized shear stress for different load levels along A-A' cross-Section for the comparison of GFRP and Steel Bar.

6.2.2. 8m Fixed Length

6.2.2.1. Ground Anchor with Steel Bar. Pull-out test of a ground anchor with steel bar embedded in dense sand is simulated in this model. The fixed length of the ground anchor is 8m. In this analysis, the transition from elastic phase to plastic phase occurred prior to 0.4 cm pull out (at 0.2cm). Therefore only the deformation and relative shear plots for 0.4cm pull out will be presented in Figure 6.57 and Figure 6.58.

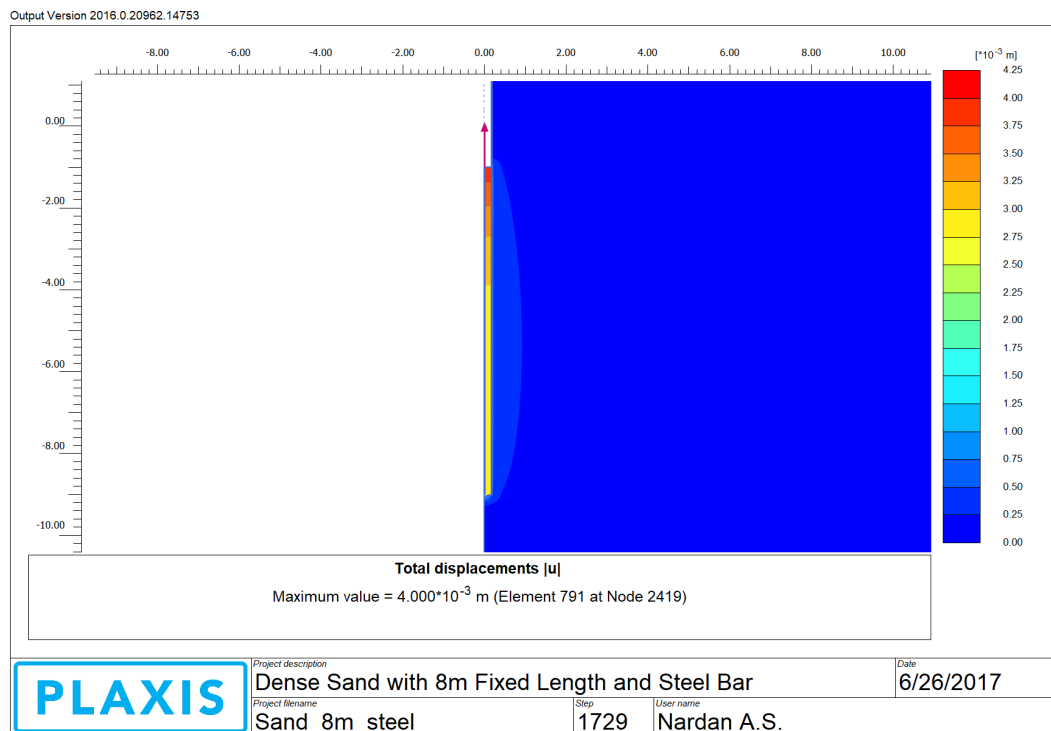


Figure 6.57. Deformation plot for 0.4 cm pull out.

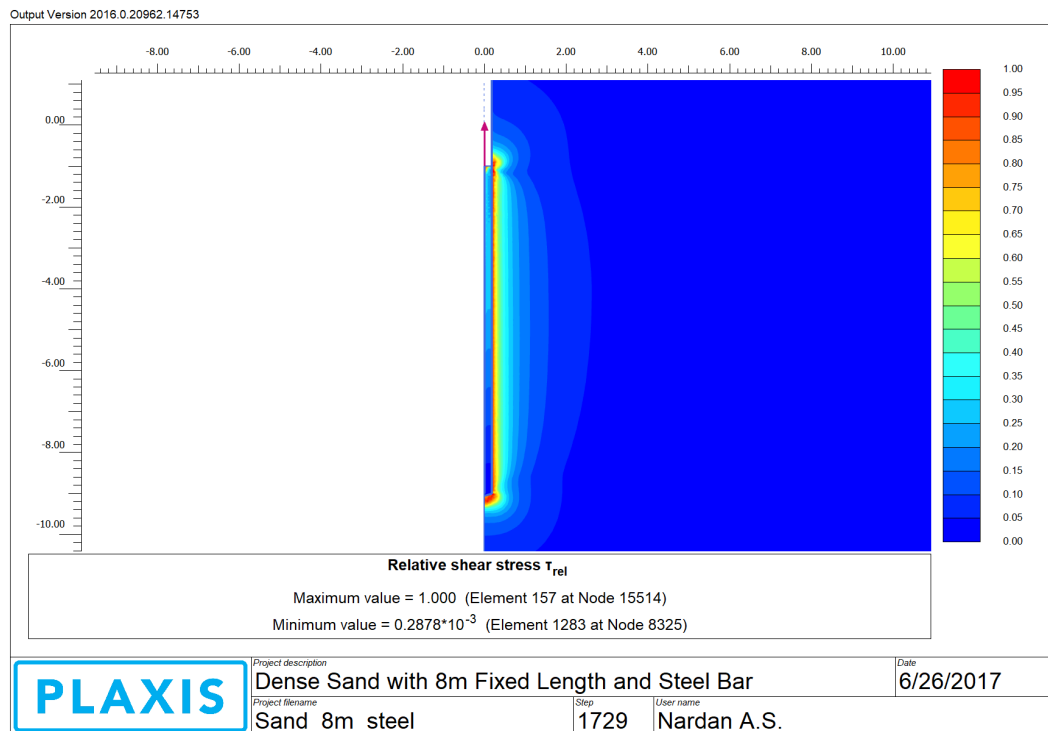


Figure 6.58. Relative Shear plot for 0.4 cm pull out.

Pull-out resistance is around 1.4 times larger than ground anchor with 6m fixed length. Also, it can be seen from the contour diagram the transmission of the deformations from the proximal to the distal part of the fixed length slightly slower than ground anchor with 6m fixed length. Graphs for the load-displacement and mobilized shear stress versus “deflection on the top of the fixed length” may be seen in Figure 6.59 and Figure 6.60.

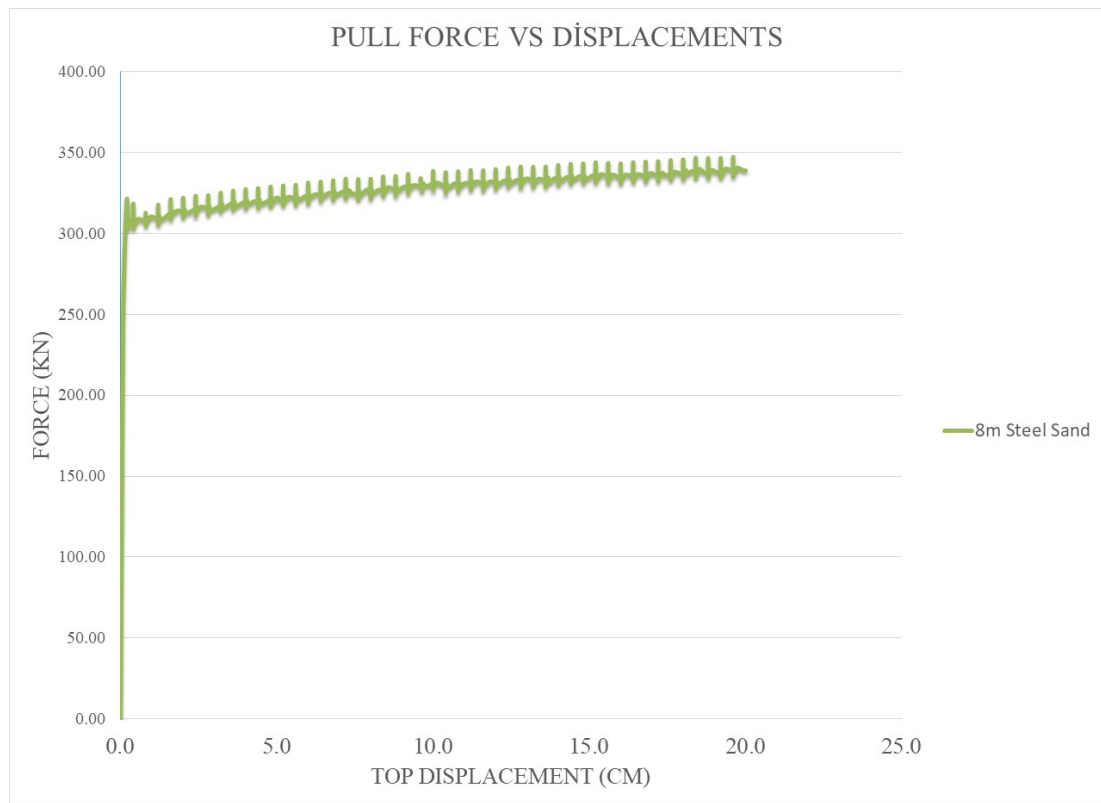


Figure 6.59. Load vs Displacement plot.

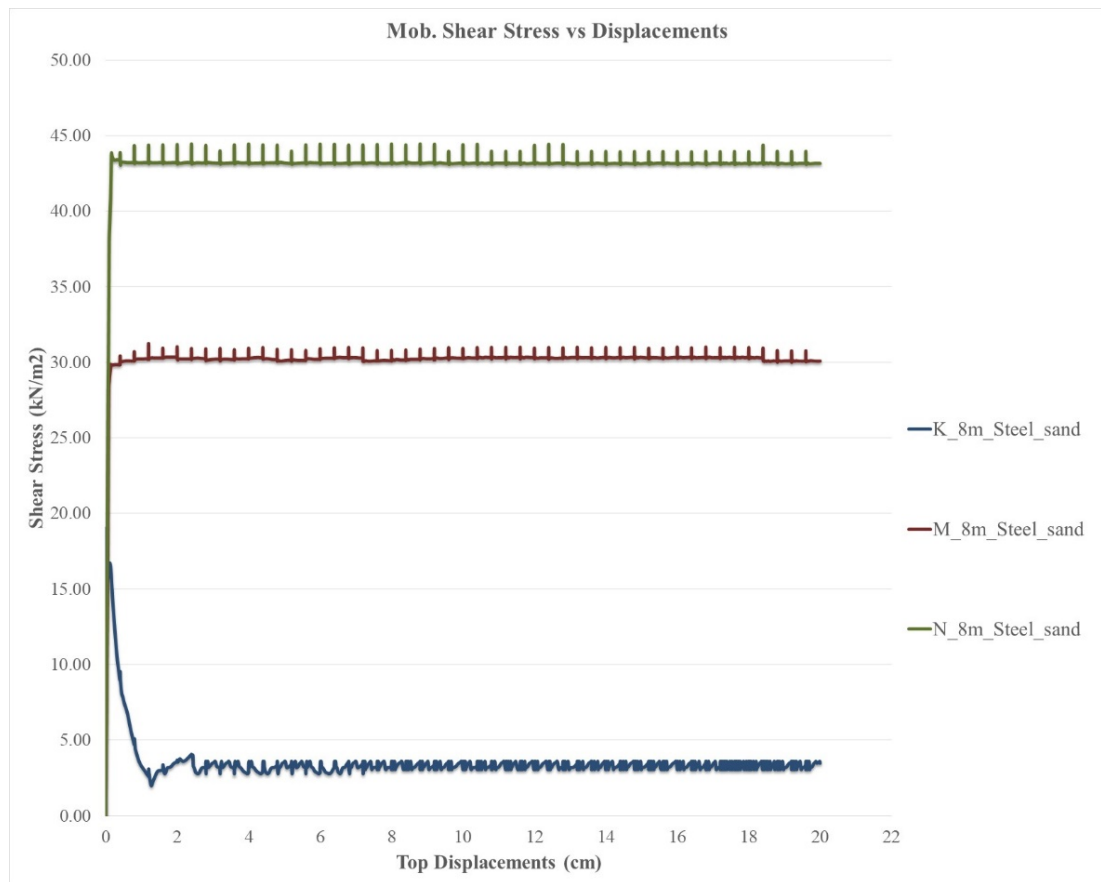


Figure 6.60. Mobilized Shear Stress vs Displacement plot for points K, M and N in the FE mesh.

The mobilized shear stress on the point which is at the top of the fixed length K, has reached the peak shear stress almost instantly as the displacement continues, it dropped to a residual value as expected. The medium point M, and the distal point N, has reached the peak stress value almost at the same deformation level with the point K and stayed at these values until failure.

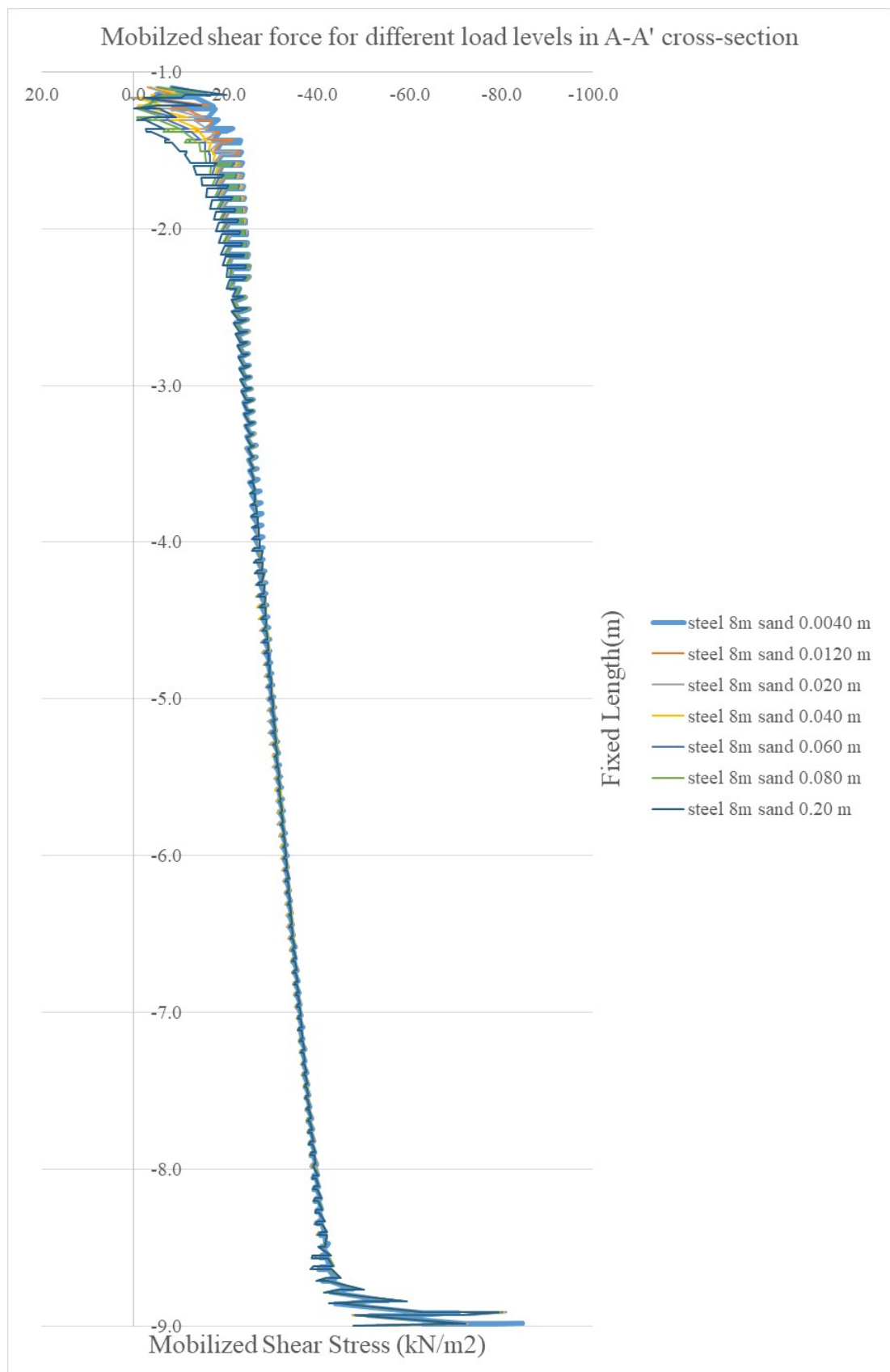
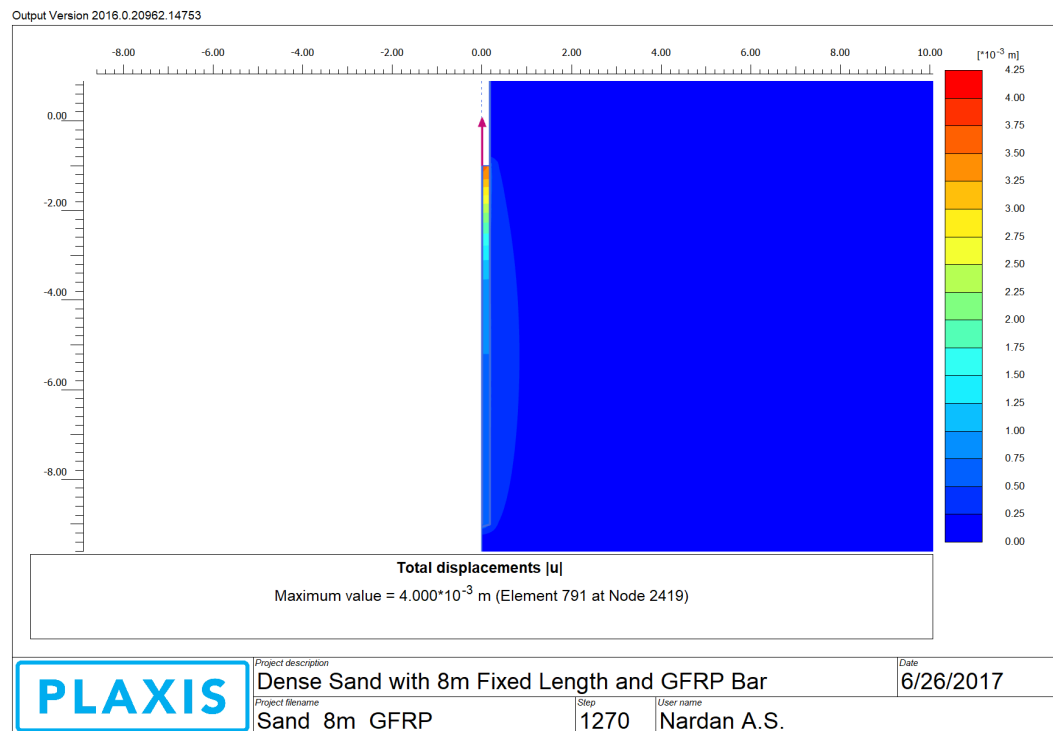


Figure 6.61. Mobilized shear stress for different load levels along A-A' cross-Section.

As can be seen from Figure 6.61, the shear stress doesn't change remarkably along the fixed length zone. The Maximum pull-out force mobilized in 0.22 cm deformation, no remarkable difference is apparent but the change in the stress at the top of the fixed length decreased to residual level again like in previous case.

6.2.2.2. Ground Anchor with GFRP Bar. Pull-out test of a ground anchor with GFRP bar is simulated for the same conditions with the previous model. In this analysis, the transition from elastic phase to plastic phase occurred after 0.4cm pull out (at 0.45cm) likewise the previous model. Therefore the deformation and relative shear plots for 0.4cm and 0.8cm pull out will be presented in Figure 6.62-Figure 6.65.



inin

Figure 6.62. Deformation plot for 0.4 cm pull out.

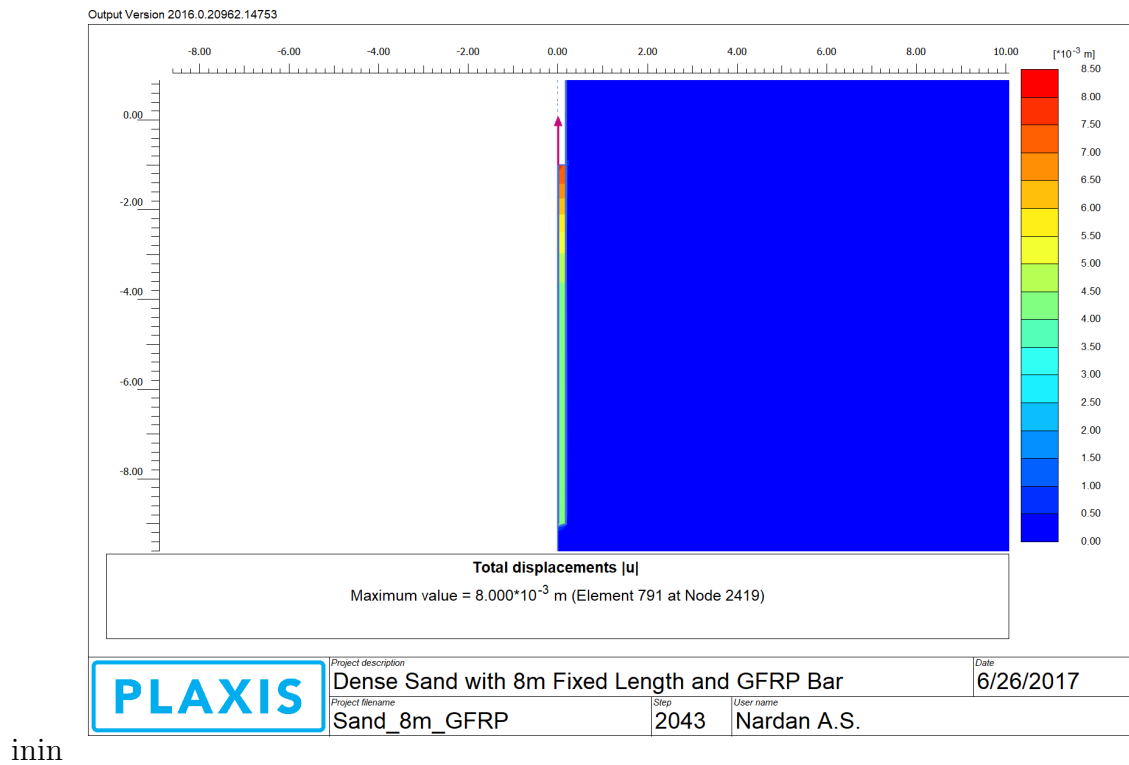


Figure 6.63. Deformation plot for 0.8cm pull out.

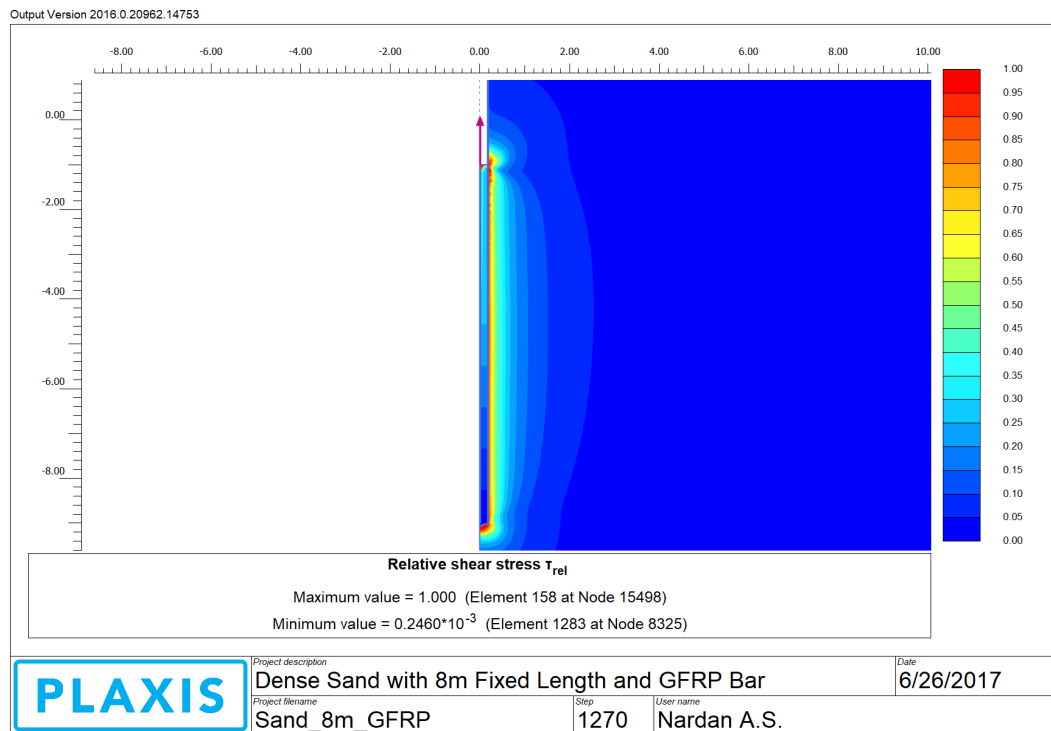


Figure 6.64. Relative Shear plot for 0.4 cm pull out.

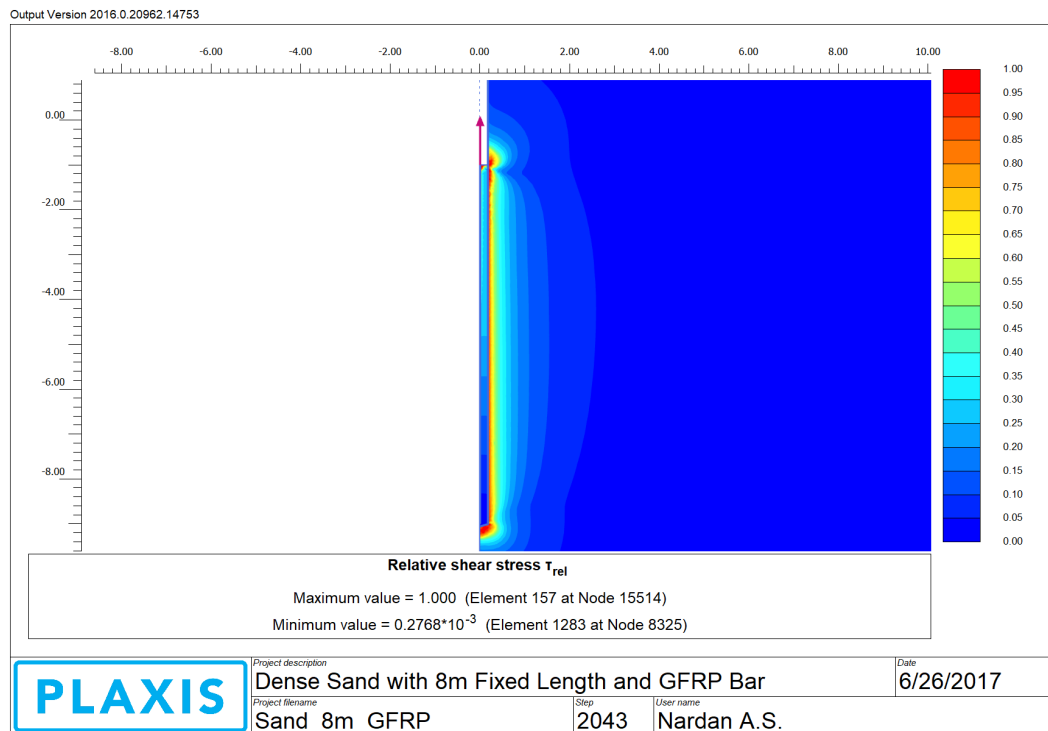


Figure 6.65. Relative Shear plot for 0.4 cm pull out.

For this case, the transition from elastic phase to plastic is occurred following the 0.4 cm pull out (0.45 cm). This situation is apparent from the contour colors (mostly light blue) in Figure 6.62 however, in Figure 6.63 anchor capacity is reached before all of the deformation on the top haven't transferred to the end yet. The anchor bearing capacity is the same with the previous one.

Graphs for the load vs displacement and mobilized shear to “deflection on the top of the fixed length” may be seen in Figure 6.66 and Figure 6.67.

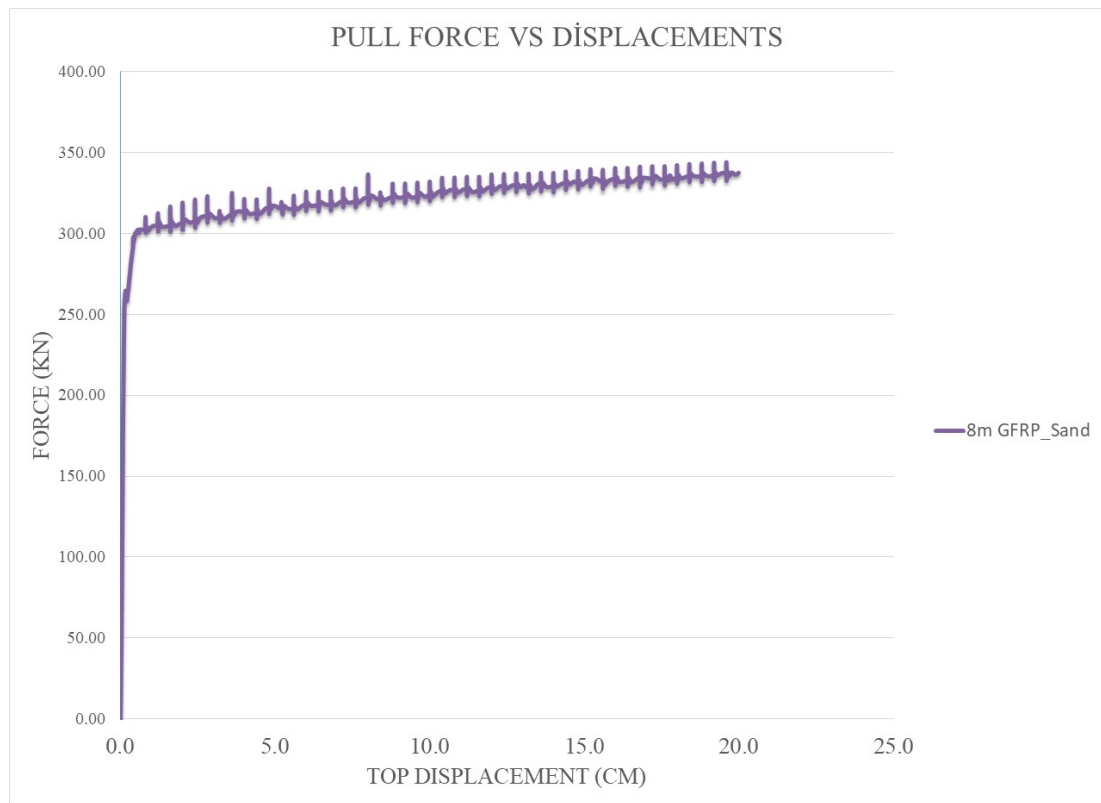


Figure 6.66. Load vs Displacement plot.

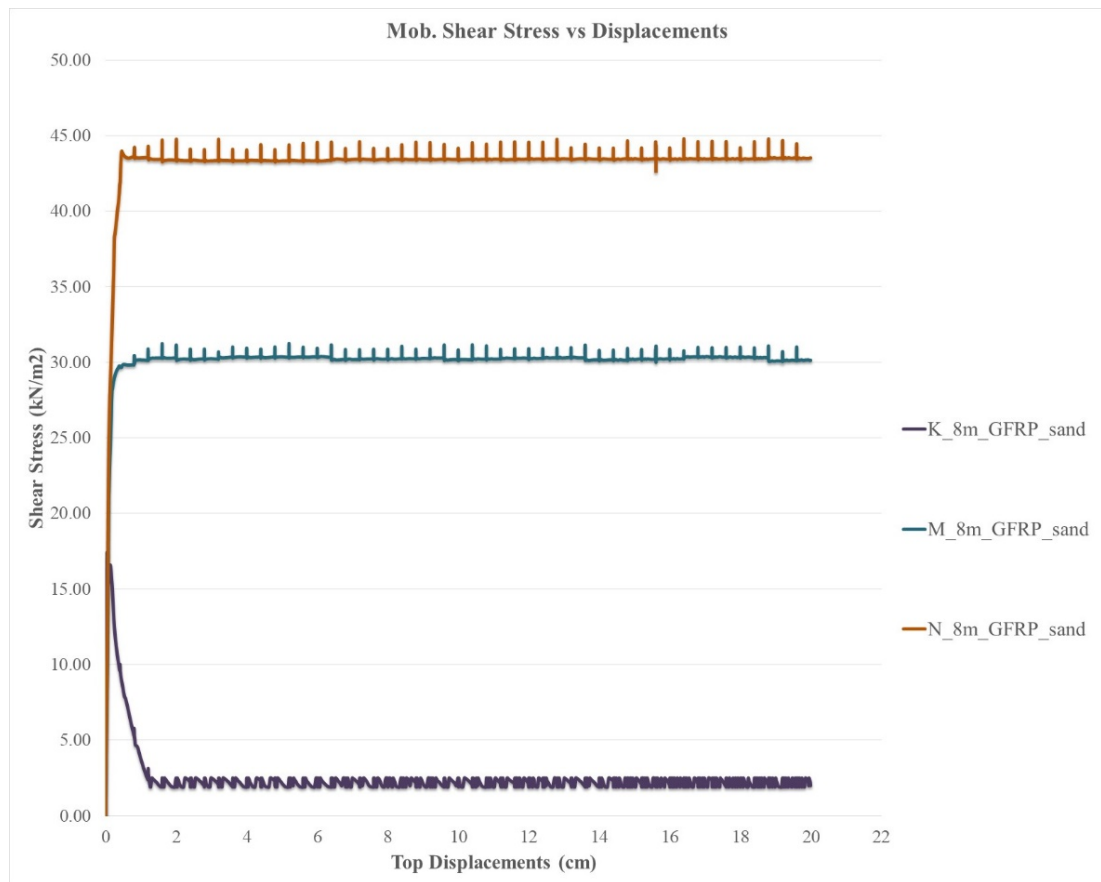


Figure 6.67. Mobilized Shear Stress vs Displacement plot for points K, M and N in the FE mesh.

As can be anticipated, behaviours of points; K, M, N for the ground anchor with GFRP bar and the steel bar are similar for this case, too. The mobilized shear stress on the point which is at the top of the fixed length K, has reached the peak shear stress almost instantly and as the displacement continues, it dropped to a residual value as expected. The medium point M, and the distal point N, has reached the peak shear stress value after the point K and stayed at these values until the failure. It can very easily be observed that there is no more shear stress mobilization after (almost) 0.45 cm deformation, however there is an increase in force with the increasing deformation. The reason for this behaviour will be addressed in the next subchapter. As can be seen from Figure 6.53, the shear stress doesn't change remarkably along the fixed length. The peak value reached at 0.45 cm deformation at the top, no difference is apparent but the change in the stress at the top of the fixed length decreased to residual level.

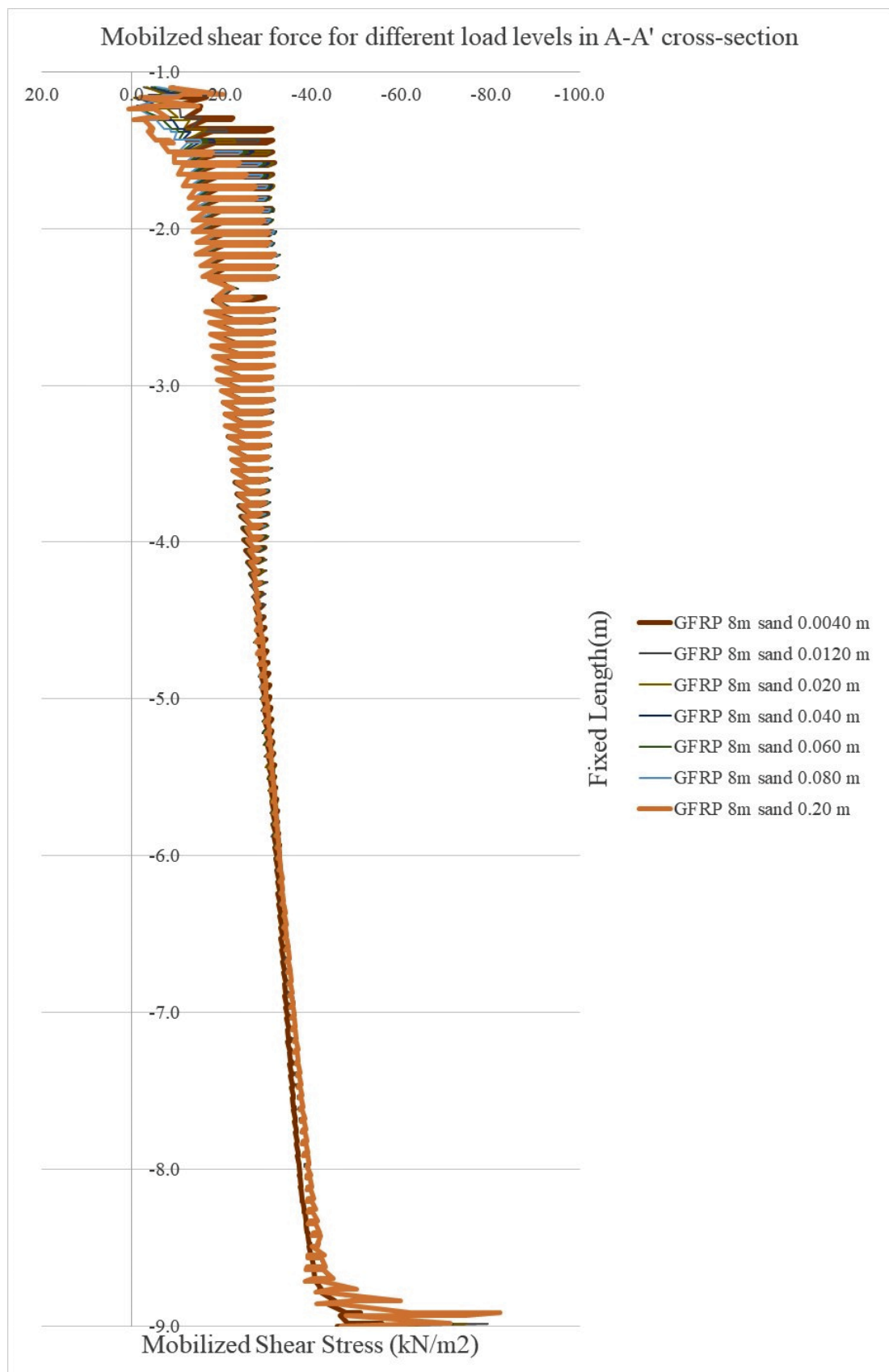


Figure 6.68. Mobilized shear stress for different load levels along A-A' cross-Section.

6.2.2.3. Comparison of the Results. As it can be anticipated, at this stress level in dense sand material with 8m fixed length, there is no significant difference between ground anchors with GFRP bar and conventional steel bar. However, in this case (dense sand) the load-displacement behaviour is not perfectly plastic but both ground anchor models (conventional steel and GFRP) experienced an increase in capacity with increased pull-out displacement. Because both of the material experienced the same behaviour and there is no change in resulting ultimate load reached in the analysis. This behaviour will be addressed in this Chapter. B-B' cross-Section may be seen in Figure 6.69;

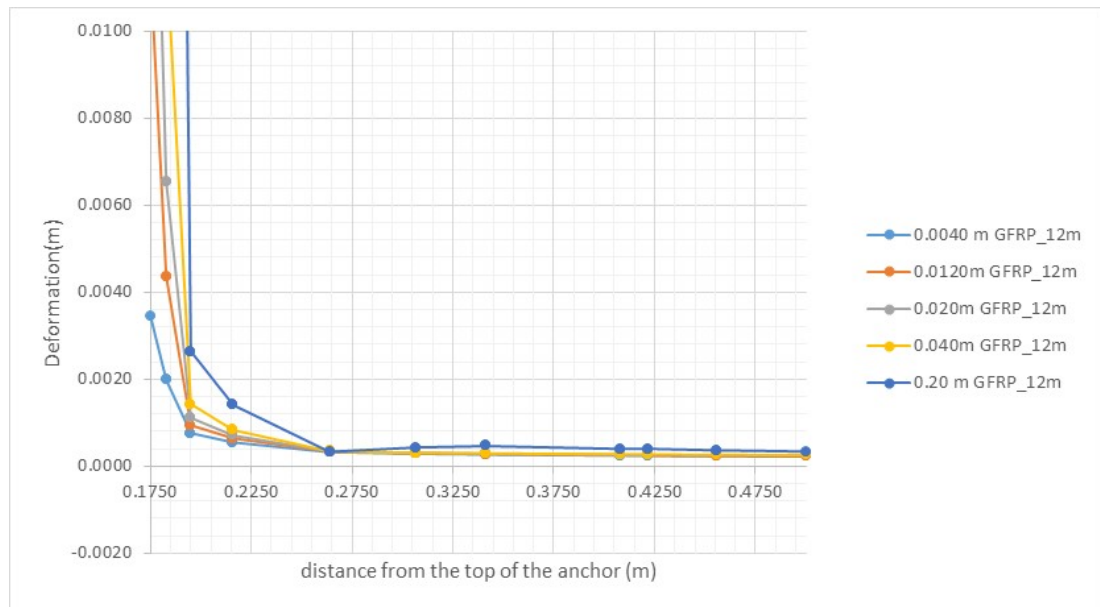


Figure 6.69. Mobilized ground deformation for different deformations on top along B-B' cross-Section.

As it can be seen from Figure 6.69, after full mobilization of the shear stress at grout-ground interface, the shear zone thickness around the anchor increases progressively. The rise with low inclination in the pull-out force/deformation graph is the result of this behaviour. This behaviour on the other hand is very similar in both cases, so it is not distinctive. The comparison of conventional steel versus GFRP at pull force-displacement graph may be seen in Figure 6.70 and Figure 6.71. As can be anticipated from the graphs, the difference between the GFRP and conventional Steel bar in ground anchorage is almost none. Minor variations in mobilized shear stress will

have no effect on the overall behaviour or stability of anchored structures.

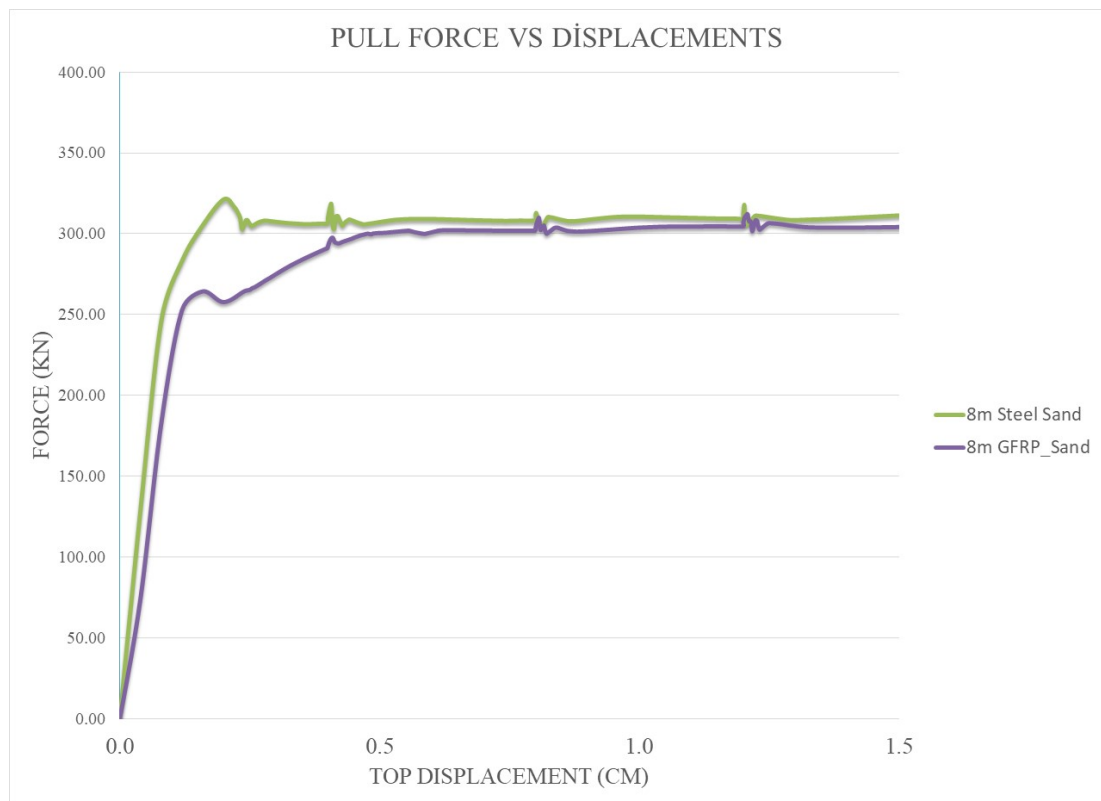


Figure 6.70. Load vs Displacement plot for the comparison of GFRP and Steel Bar.

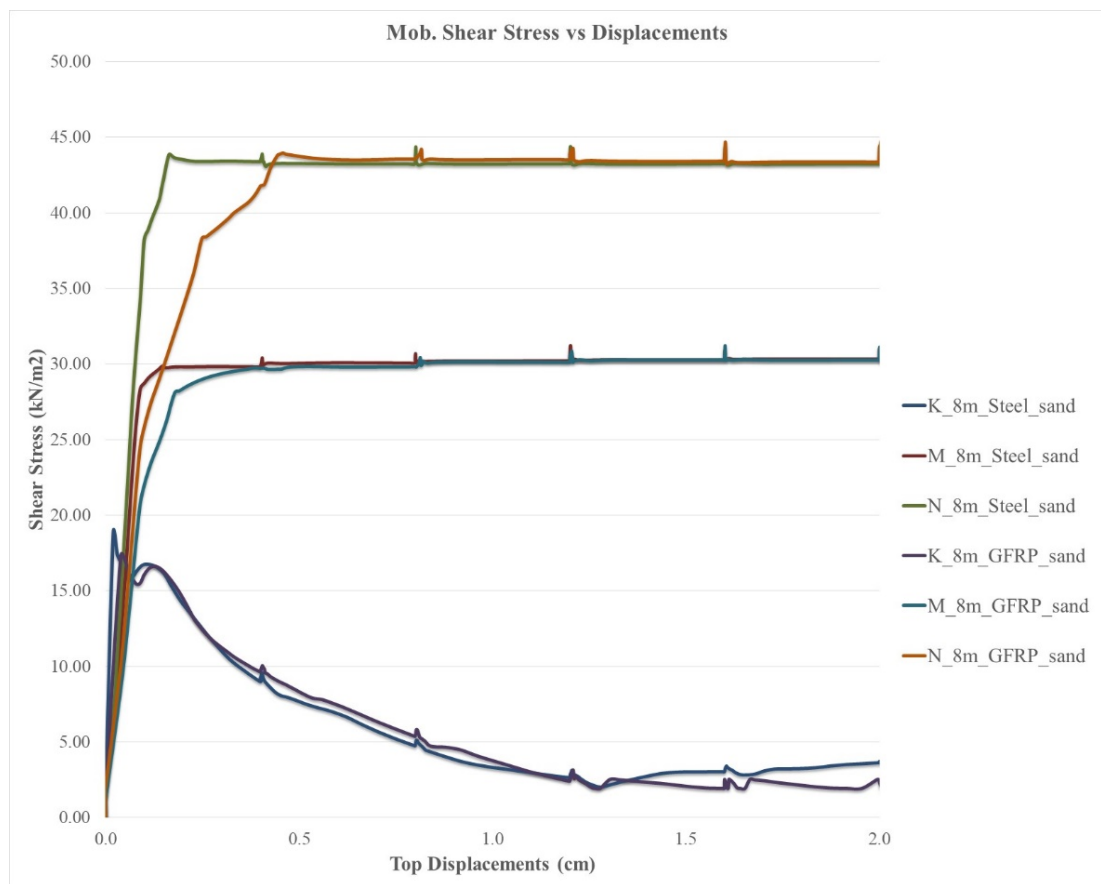


Figure 6.71. Mobilized Shear Stress vs Displacement plot for the comparison of GFRP and Steel Bar.

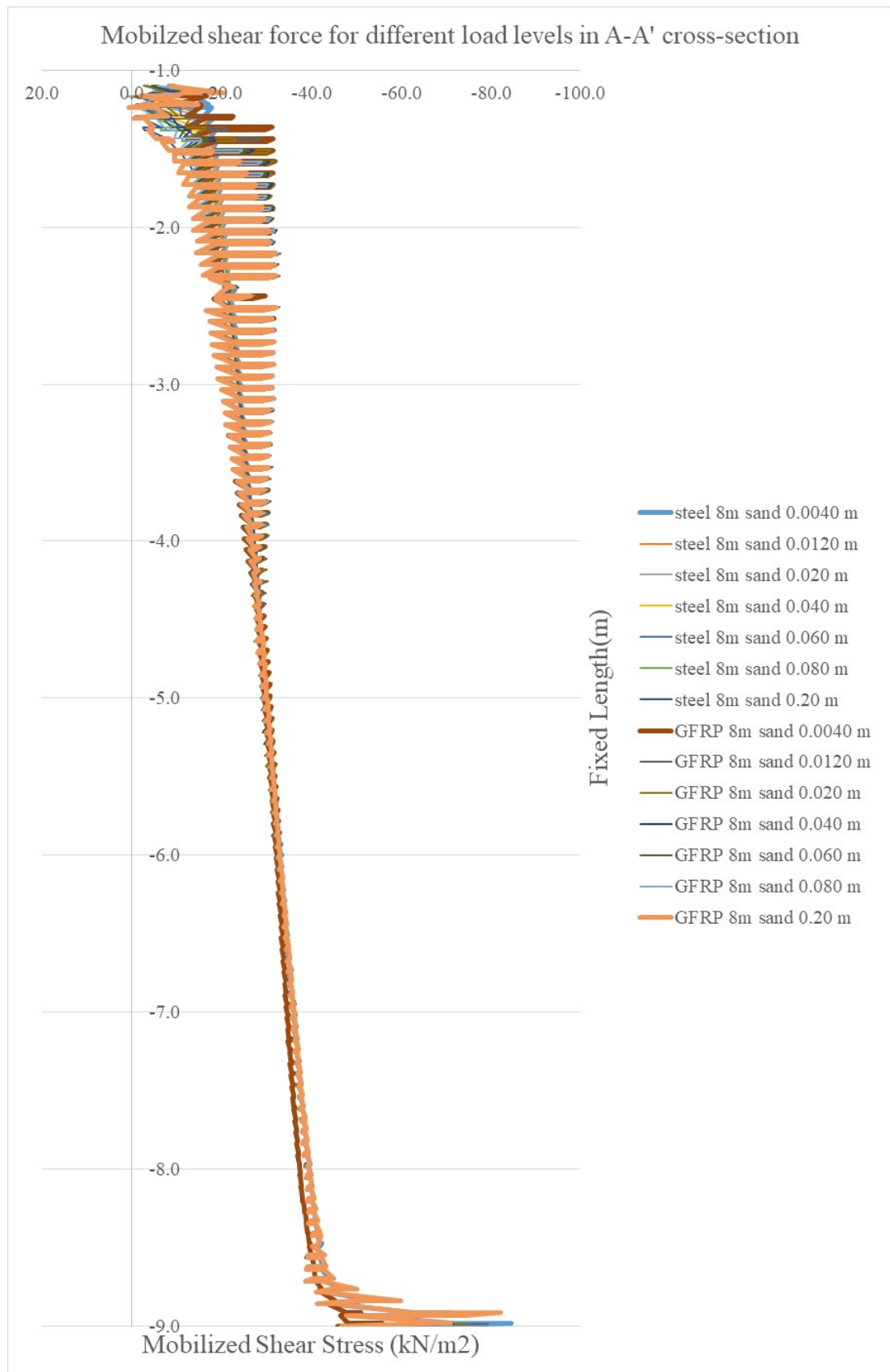


Figure 6.72. Mobilized shear stress for different load levels along A-A' cross-Section for the comparison of GFRP and Steel Bar.

6.2.3. 12m Fixed Length

6.2.3.1. Ground Anchor with Steel Bar. Pull-out test of a ground anchor with steel bar embedded in dense sand soil is simulated in this model. The fixed length of the ground anchor is 12m. In this analysis, the transition from elastic phase to plastic phase occurred after 0.4cm pull out (at 0.75cm). Therefore the deformation and relative shear plots for 0.4cm and 0.8cm pull out will be presented in Figure 6.73-Figure 6.76.

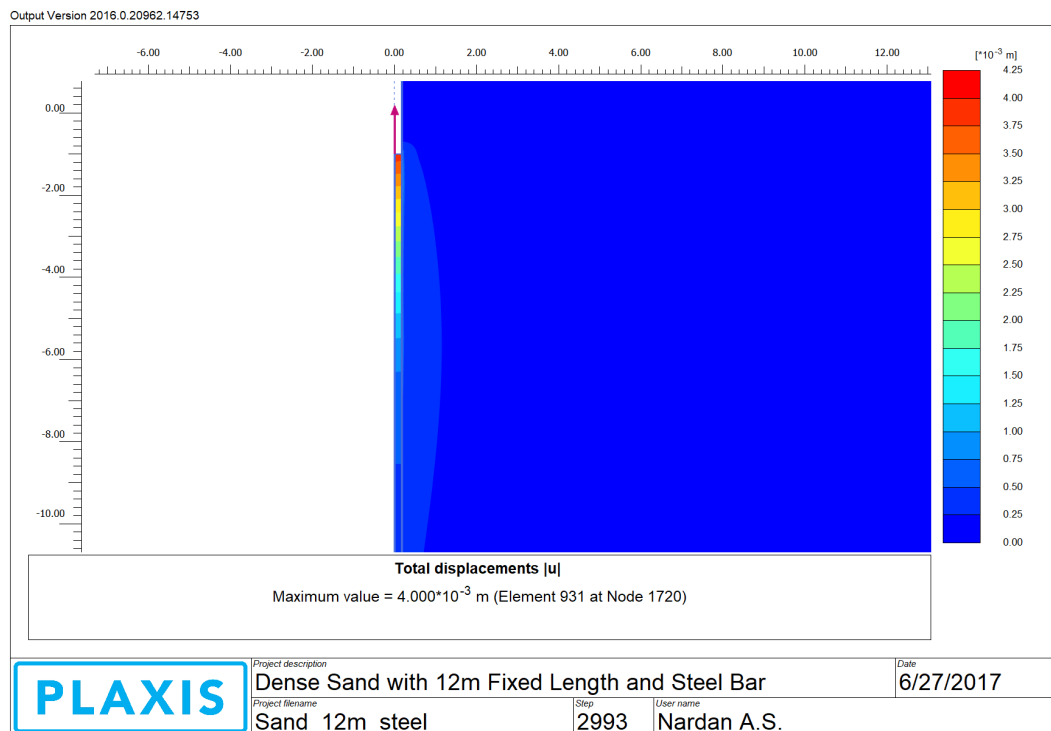


Figure 6.73. Deformation plot for 0.4 cm pull out.

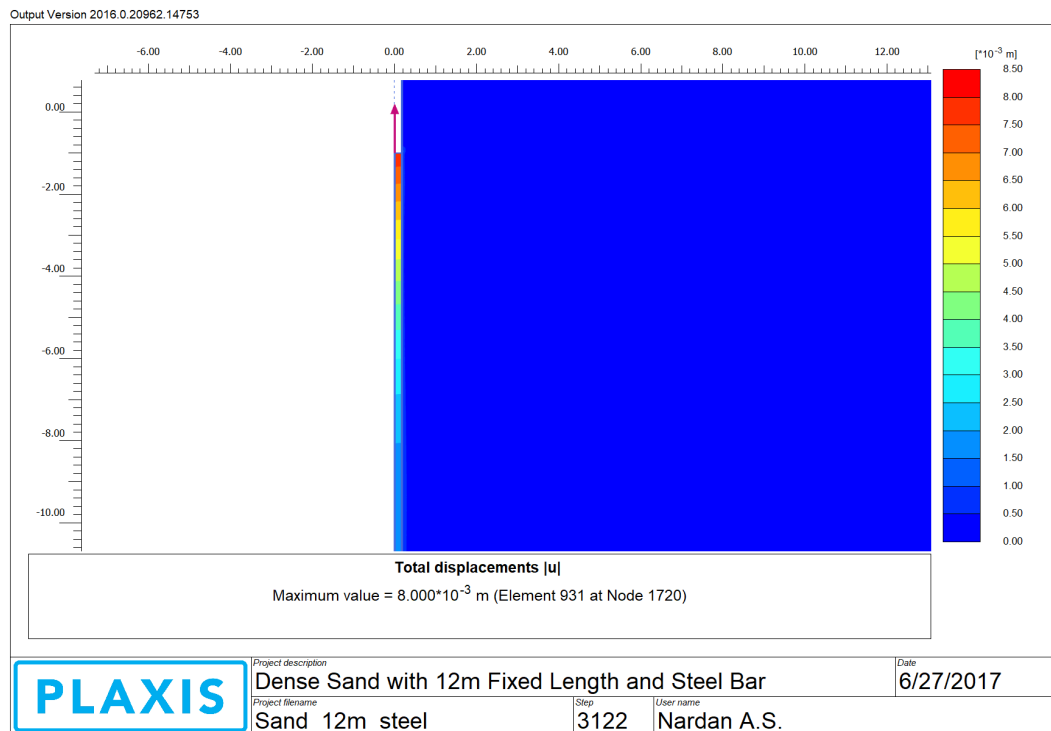


Figure 6.74. Deformation plot for 0.8cm pull out.

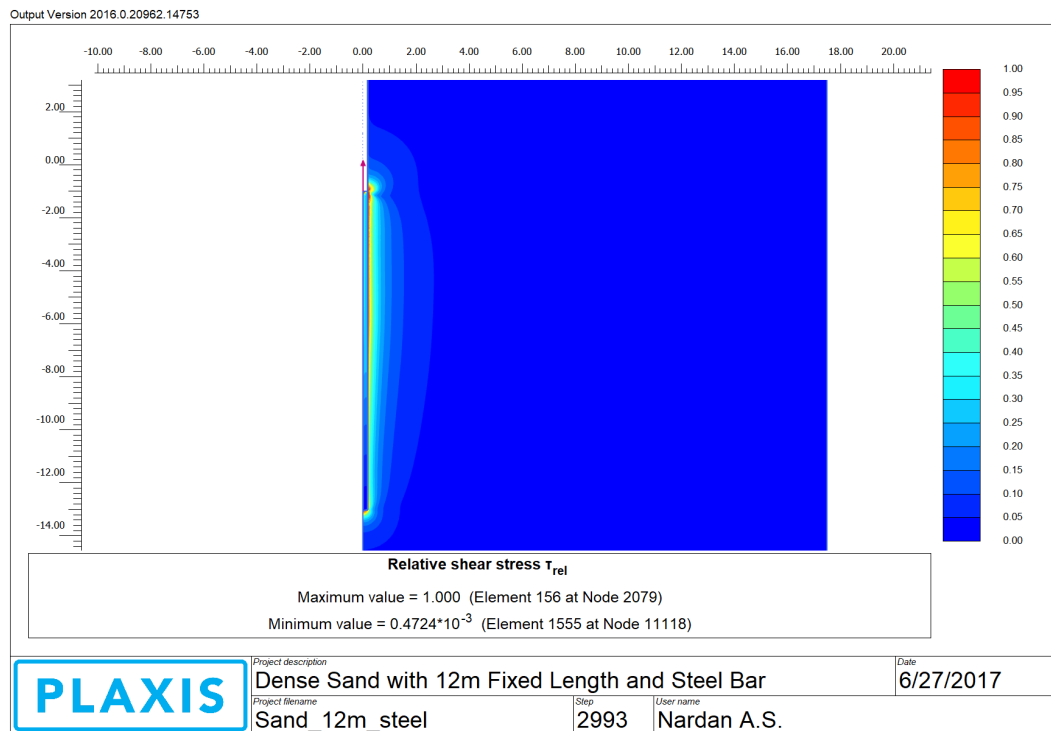


Figure 6.75. Relative Shear plot for 0.4 cm pull out.

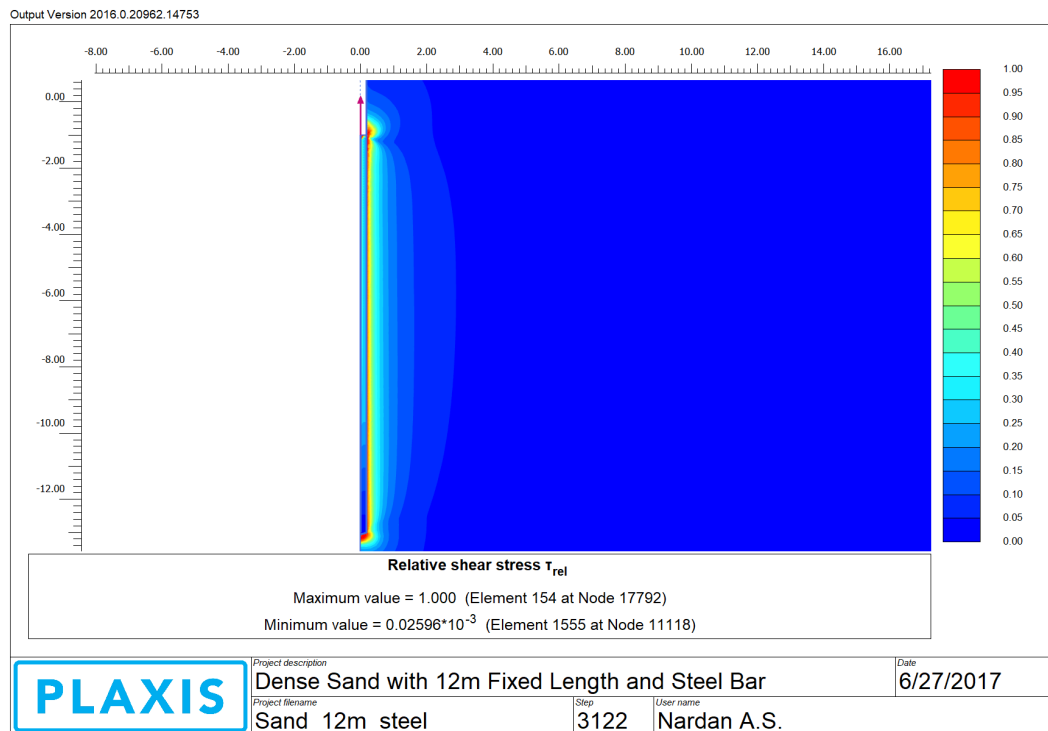


Figure 6.76. Relative Shear plot for 8mm pull out.

It can be seen from the contour diagram the transmission of the deformations from the proximal to the distal part of the fixed length slightly slower than ground anchor with 12m fixed length. This situation is apparent in Figure 6.73 and Figure 6.74 anchor capacity is reached before all of the deformation on the top haven't transferred to the end yet. The pull-out resistance is around 1.67 times larger than ground anchor with 8m fixed length and 2.5 times larger than the ground anchor with 8m fixed length. Graphs for the load-displacement and mobilized shear stress to "deflection on the top of the fixed length" may be seen in Figure 6.77 and Figure 6.78.

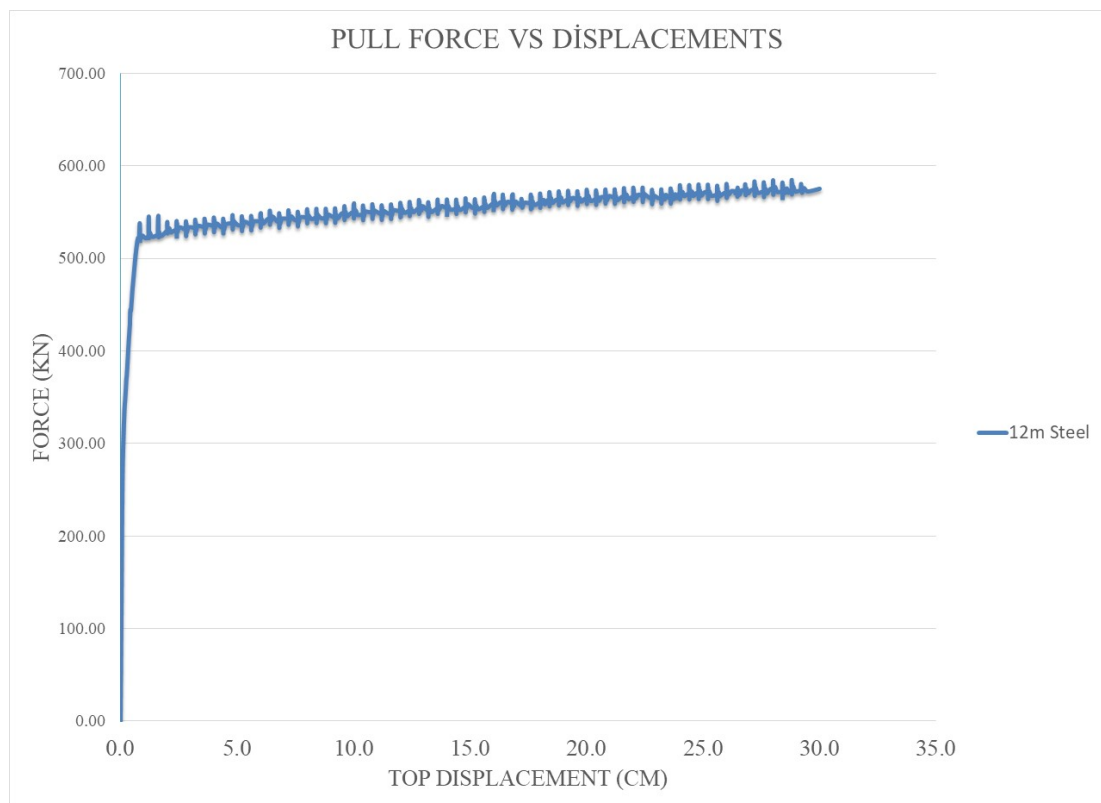


Figure 6.77. Load vs Displacement plot.

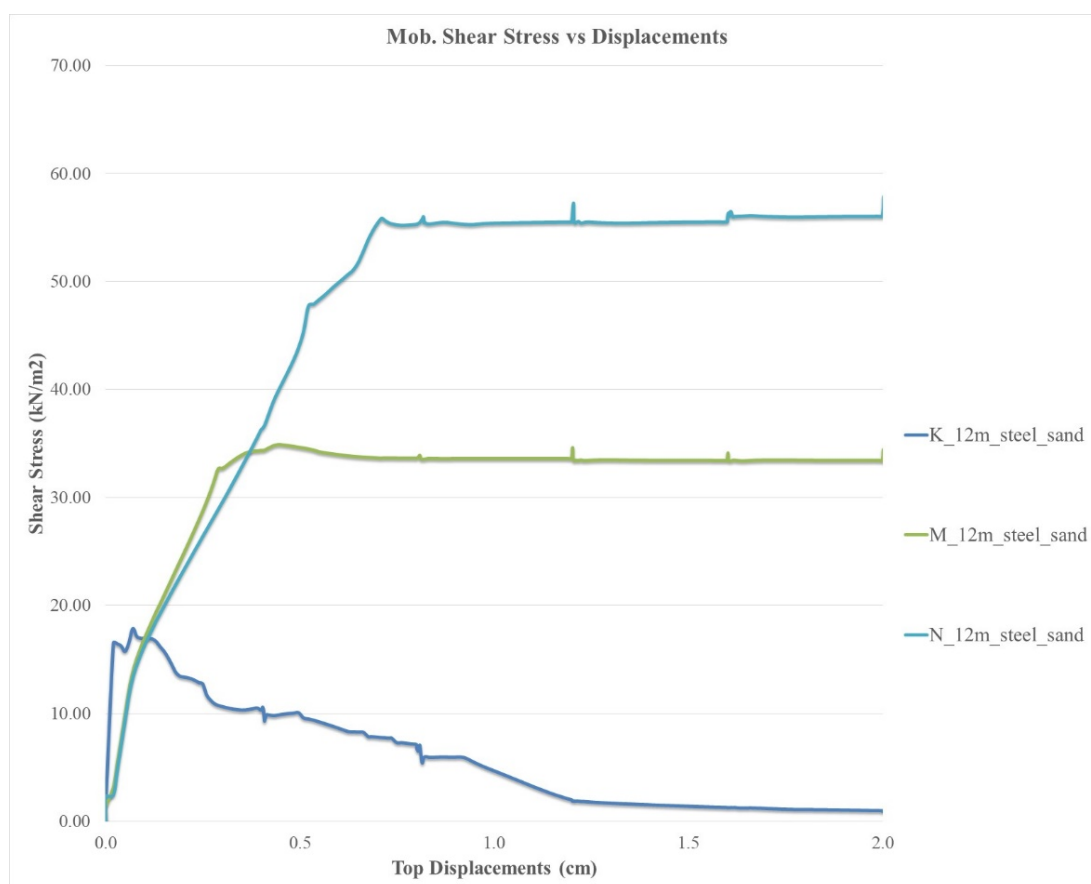


Figure 6.78. Mobilized Shear Stress vs Displacement plot for points K, M and N in the FE mesh.

The mobilized shear stress on the point which is at the top of the fixed length K, has reached the peak shear stress almost instantly as the previous ones but more deformation is needed for it to drop to a residual value. The medium point M, and the distal point N, has reached the peak stress values after the point K and stayed at these values until failure. It can be easily observed from Figure 6.79, the shear stress fully mobilized after 0.76cm of deformation at the top of the ground anchor. This is a distinction in the behaviour of the pull-out resistance of ground anchor by comparing with 8m and 6m fixed length ground anchors. After 0.8cm deformation at the top there is still change in the stress at the top of the fixed length, it decreased to residual level.

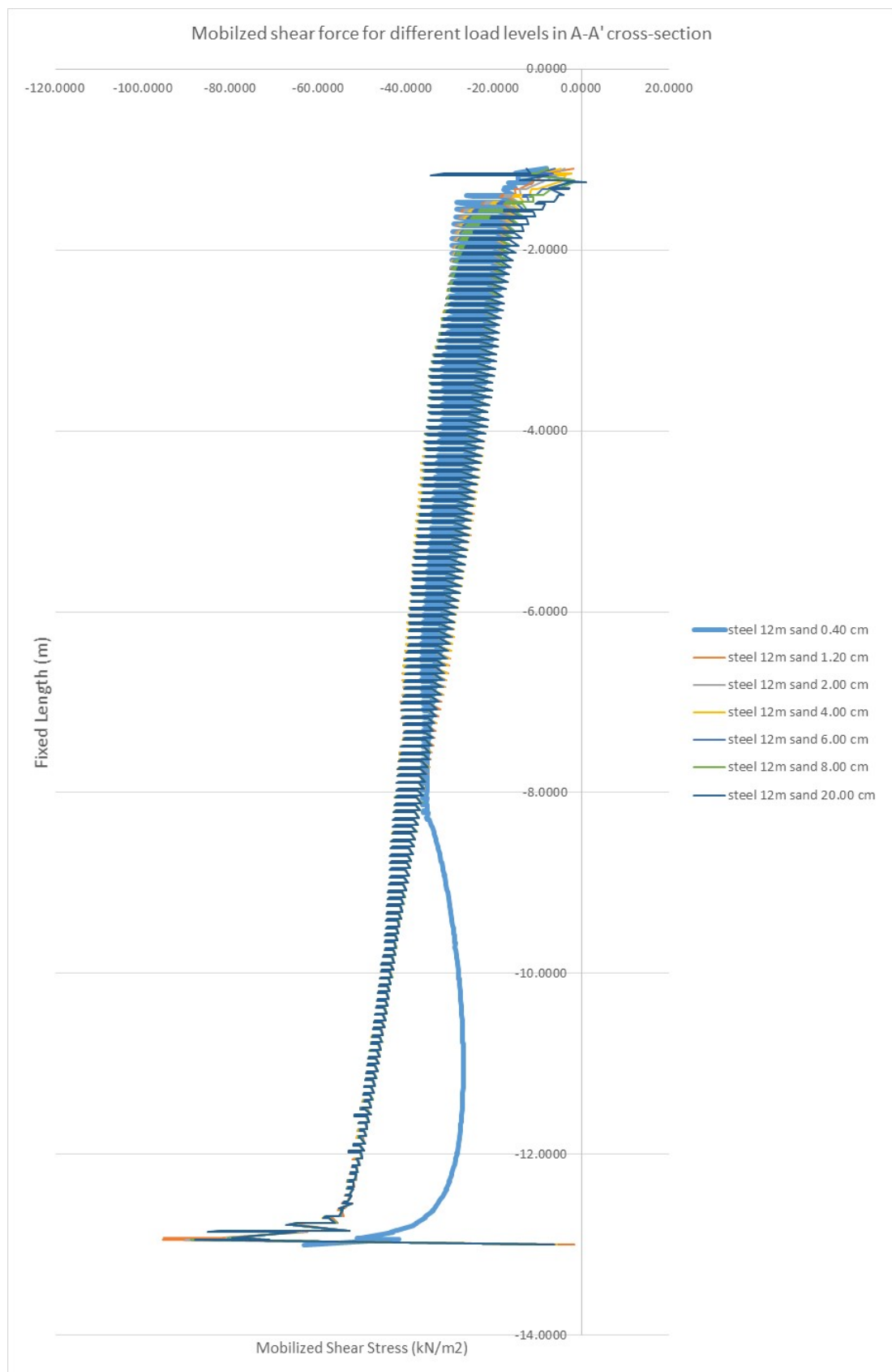


Figure 6.79. Mobilized shear stress for different load levels along A-A' cross-Section.

6.2.3.2. Ground Anchor with GFRP Bar. Pull-out test of a ground anchor with GFRP bar is simulated for the same conditions with the previous model. In this analysis, the transition from elastic phase to plastic phase occurred after to 0.4 cm pull out (at 2.8cm) likewise the previous model. Therefore only the deformation and relative shear plots for 0.4cm pull out will be presented in Figure 6.80-Figure 6.83.

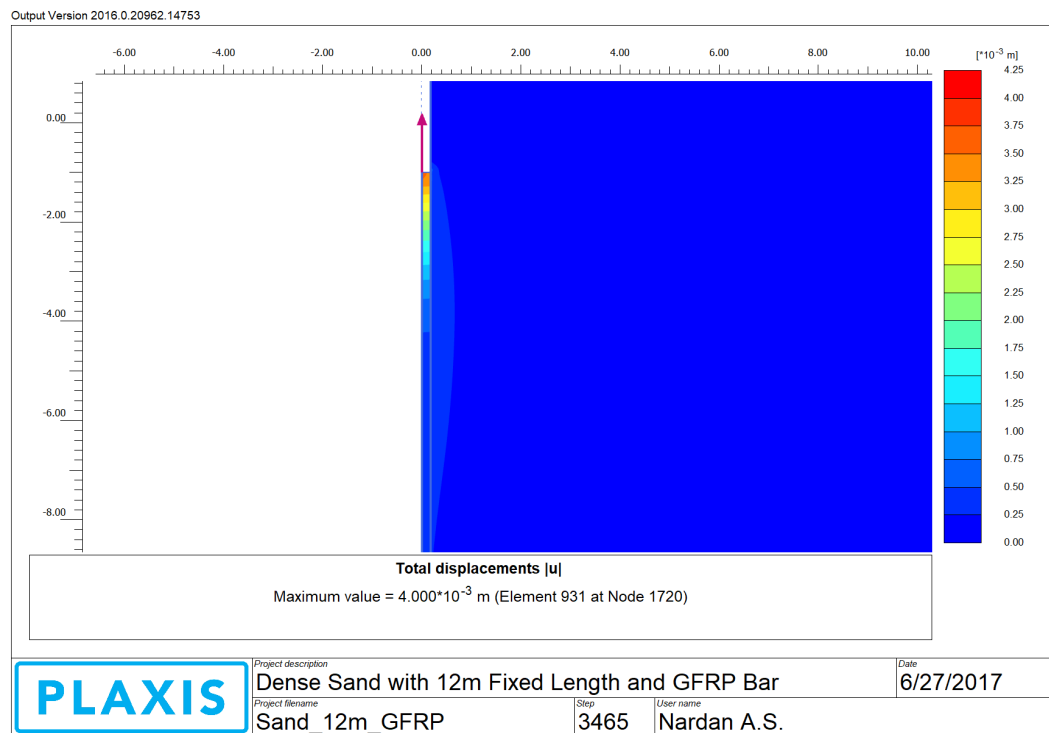


Figure 6.80. Deformation plot for 0.4 cm pull out.

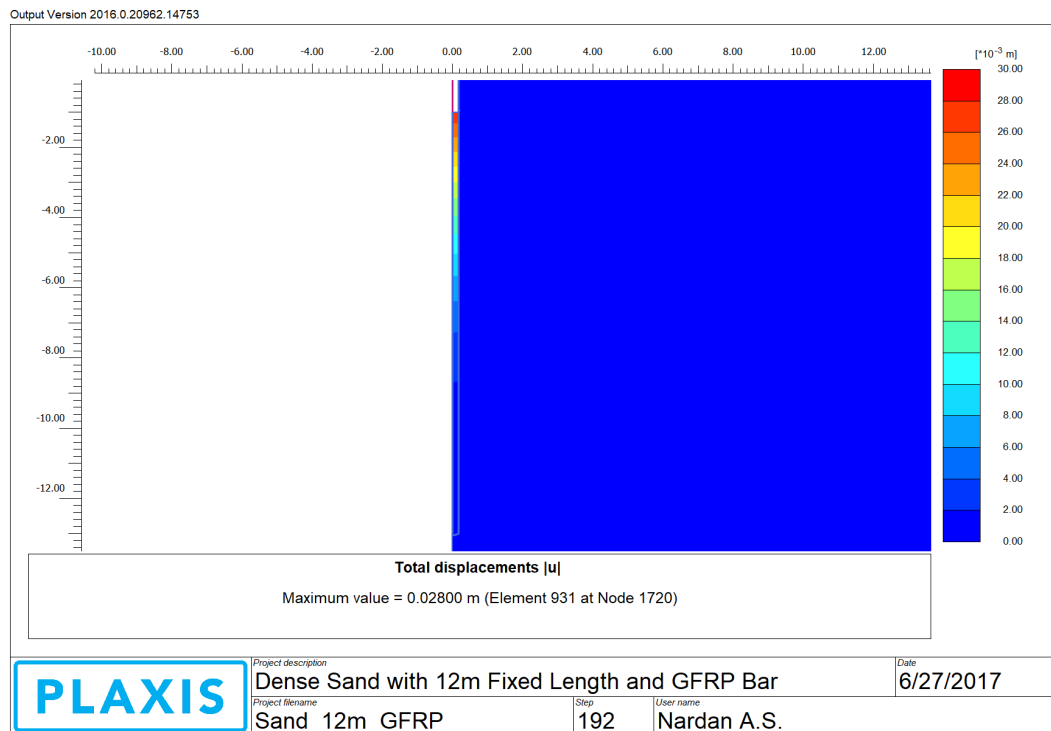


Figure 6.81. Deformation plot for 2.8cm pull out.

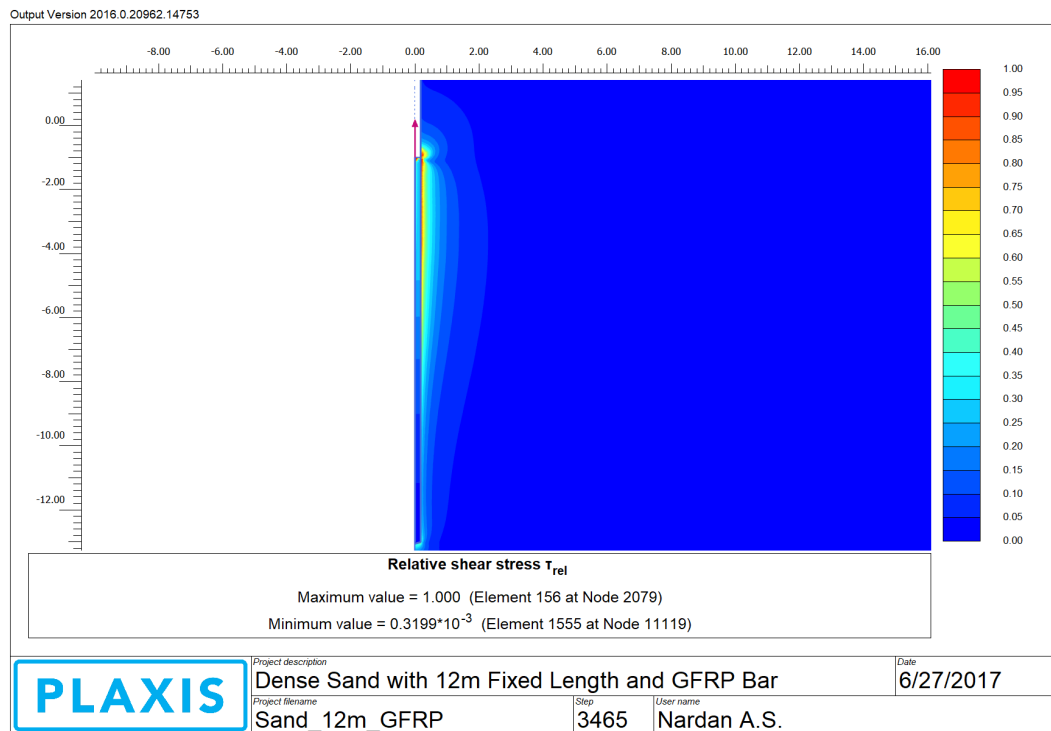


Figure 6.82. Relative Shear Stress plot for 0.4 cm pull out.

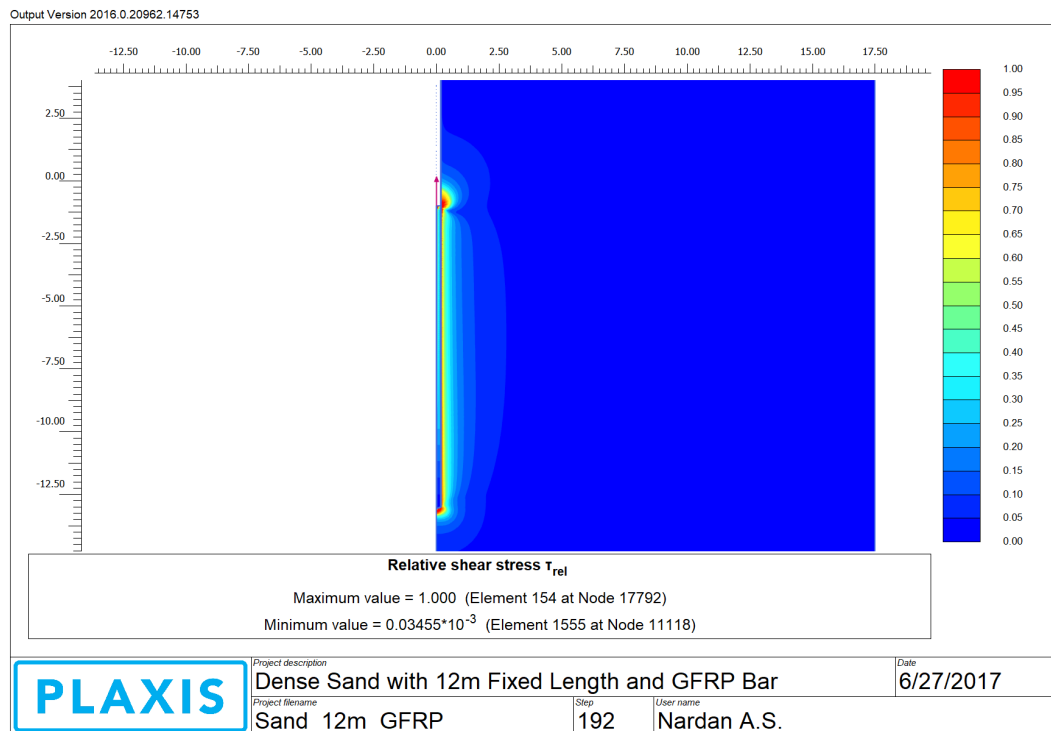


Figure 6.83. Relative Shear plot for 2.8cm pull out.

For this case, the transition from elastic phase to plastic phase is occurred at 2.8 cm pull out (2.8cm). This situation is more distinct in this case with GFRP bar. It is very clear from the Figure 6.84 and Figure 6.85, the maximum load is reached before all the deformations at the top isn't yet transferred to the medium level due to the lower young's modulus of GFRP bar. Graphs for the load vs displacement and mobilized shear vs "deflection on the top of the fixed length" may be seen in Figure 6.84 and Figure 6.85.

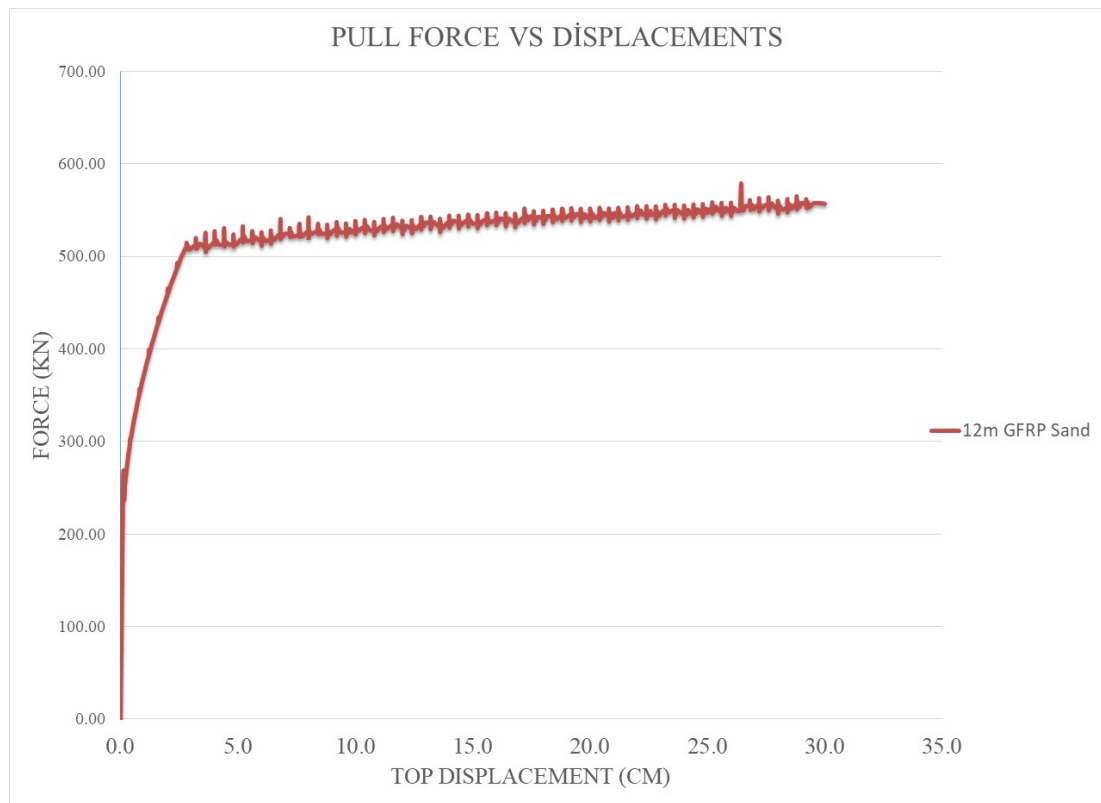


Figure 6.84. Load vs Displacement plot.

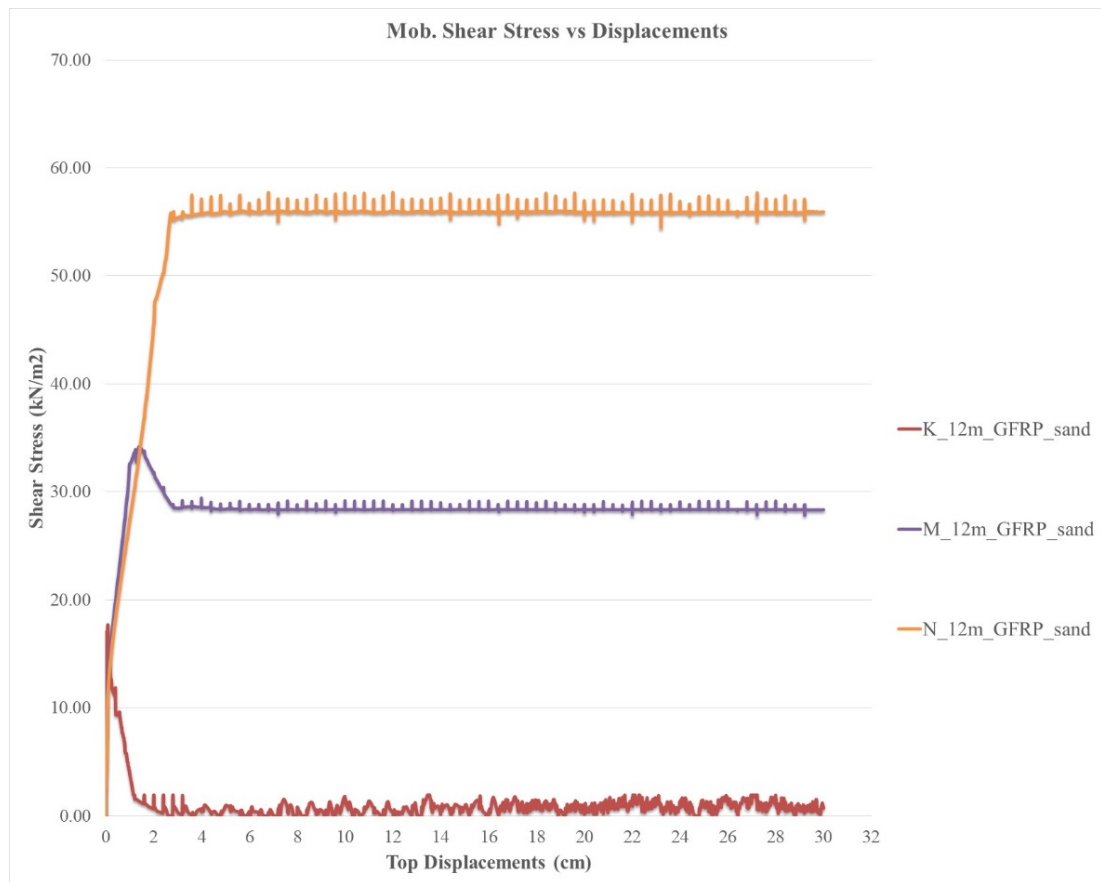


Figure 6.85. Mobilized Shear Stress vs Displacement plot for points K, M and N in the FE mesh.

The mobilized shear stress on the point which is at the top of the fixed length K, has reached the peak shear stress almost instantly but more deformations are needed for it to drop to residual value. The medium point M, and the distal point N, has reached the peak shear stress value after the point K and stayed at these values until the failure. There is also a slight increase in pull out force (without any increase in mobilized shear stress) like the one in previous (8m fixed length) sub-Chapter. This will be addressed again in comparison sub-Chapter. As can be seen from the Figure 6.86, the shear stress fully mobilized after 2.8cm of deformation at the top of the ground anchor and it is more clearly progressive than the other ones (6m and 8m fixed length). The shear stress doesn't change remarkably along the shear zone at fixed length after 2.8cm and the same residual value may be observed from the graphes like the previous ones.

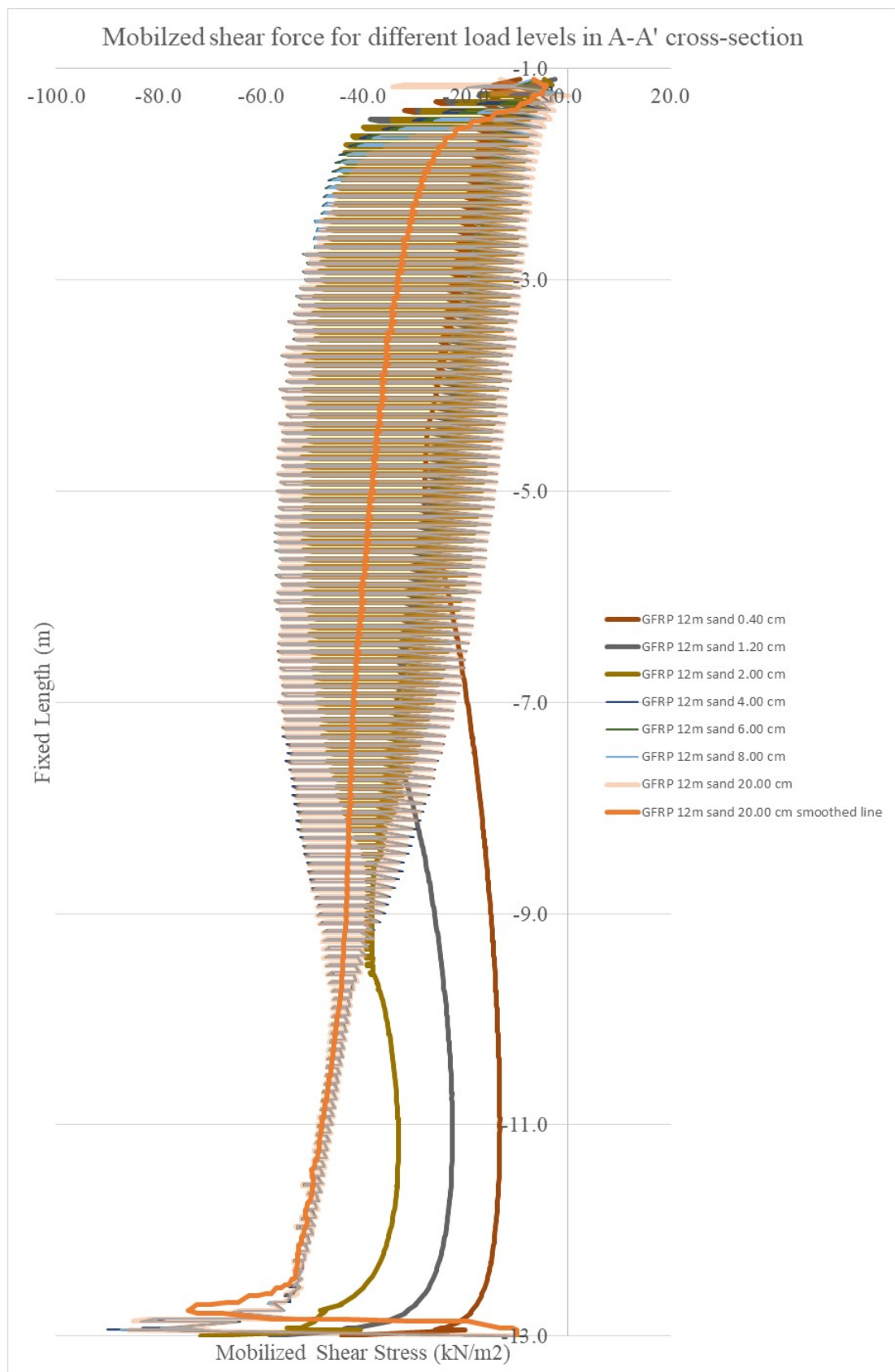


Figure 6.86. Mobilized shear stress for different load levels along A-A' cross-Section.

6.2.3.3. Comparison of the Results. As it can be seen from both three graphs, more deformations are needed for 12m fixed length ground anchor with GFRP bar to mobilize the full pull-out resistance force comparing with the steel. Even though the mobilization of the shear force at the top (point K) of the fixed length is identical in two distinct cases, the rate of deformation which is needed for the mobilization of the full shear stress at the shear zone in distal end (of the fixed length), three times more for GFRP when comparing with Steel. (The mobilized shear stress at 0.4cm for steel is in the same order of magnitude with mobilized shear stress at 1.2cm for GFRP). Even this proportion seems extreme, the deformations are still comparatively low for practical purposes. Therefore the difference between the GFRP and Steel is not so significant in practice. In both two cases, in the 8m and 6m fixed length cases, the mobilized shear stress in the grout/ground zone did not increase after a specific deformation value but the load in pull force vs deformation graph increases slightly. It is not a distinctive feature for the present research since as asserted the behaviour is same in both of the models. However this will be addressed in this Chapter regardless. The B-B' cross-Section may be seen in Figure 6.87.

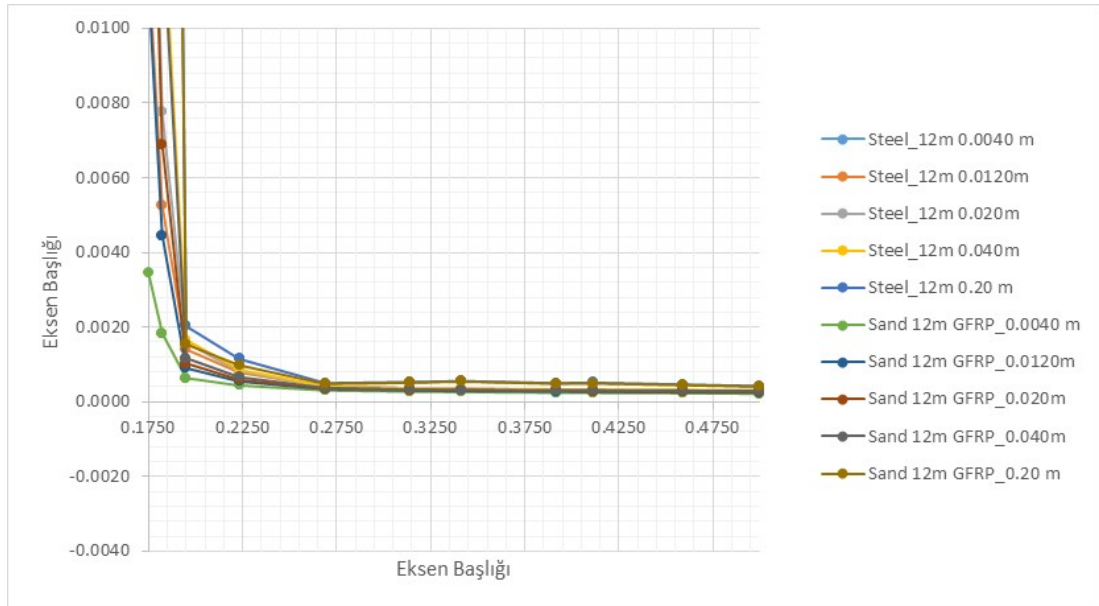


Figure 6.87. Mobilized ground deformation for different deformations on top along B-B' cross-Section.

As it can be seen from Figure 6.88 and Figure 6.89, after full mobilization of

the shear stress at grout-ground interface, the shear zone thickness around the anchor increases progressively. The rise with low inclination in the pull-out force/deformation graph is the result of this behaviour. This behaviour on the other hand is very similar in both cases, so it is not distinctive. The comparison of conventional steel versus GFRP at pull force-displacement may be seen in Figure 6.88 and Figure 6.89.

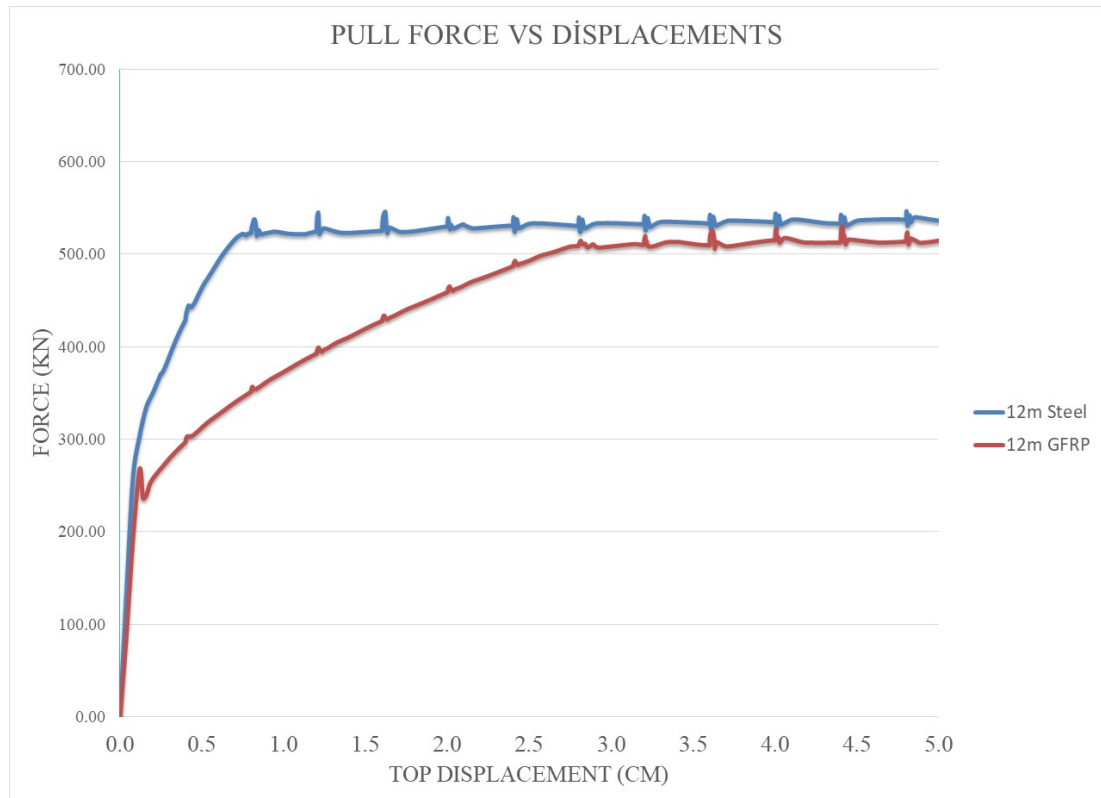


Figure 6.88. Load vs Displacement plot for the comparison of GFRP and Steel Bar.

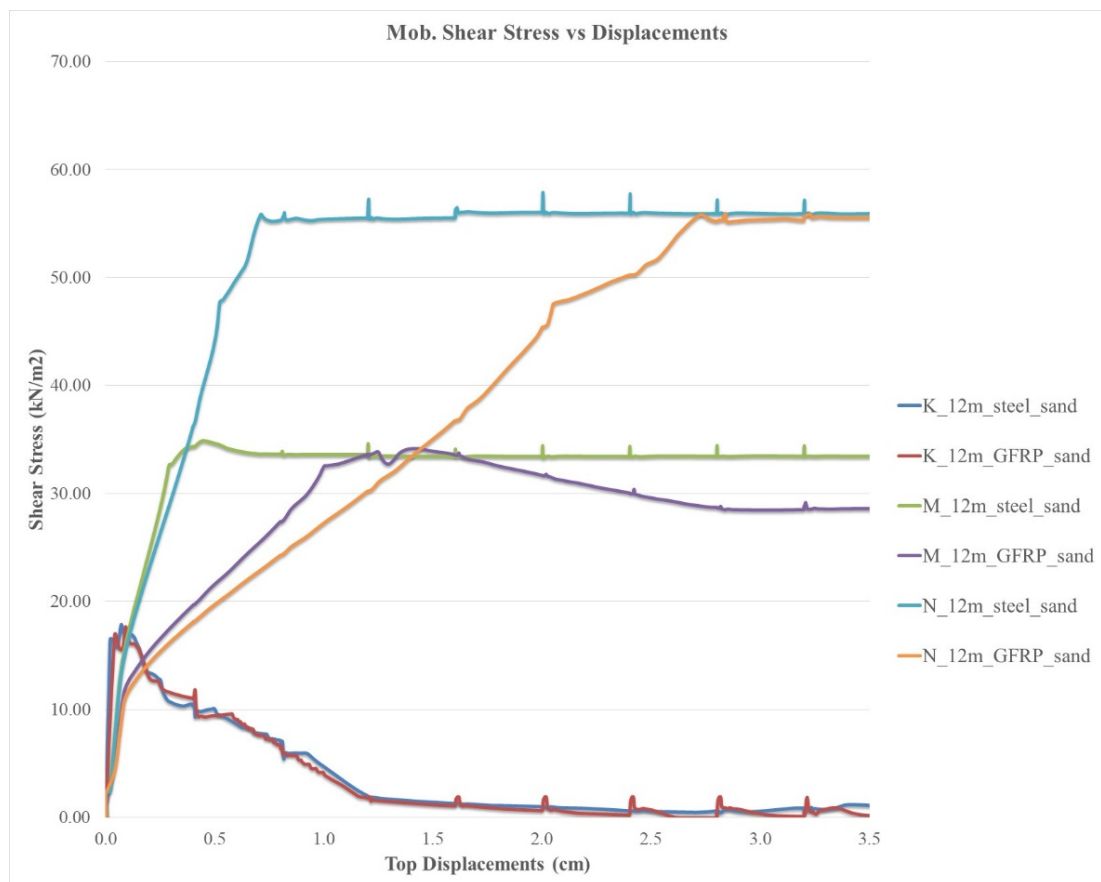


Figure 6.89. Mobilized Shear Stress vs Displacement plot for the comparison of GFRP and Steel Bar for points K, M and N in the FE mesh.

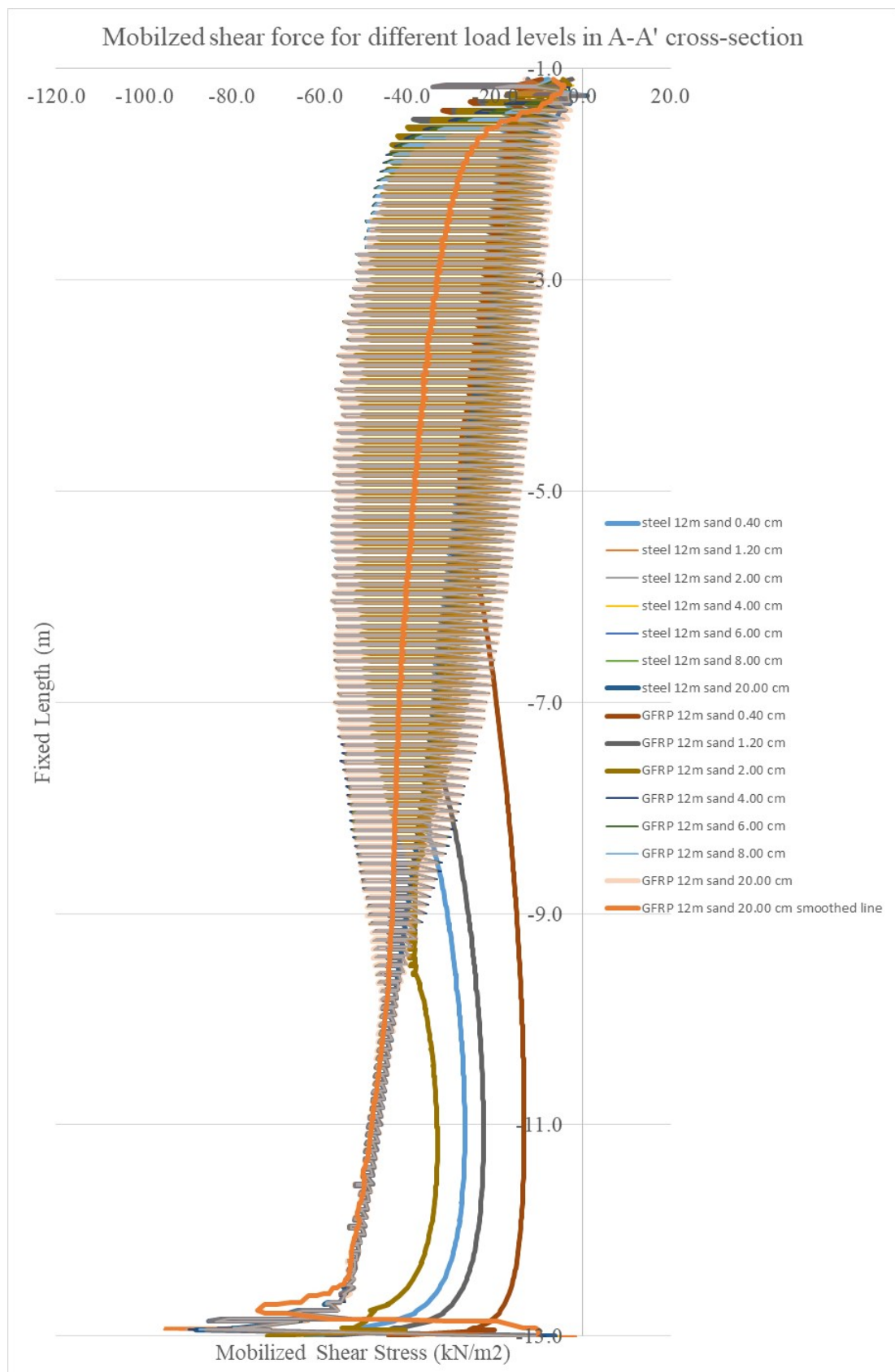


Figure 6.90. Mobilized shear stress for different load levels along A-A' cross-Section for the comparison of GFRP and Steel Bar.

6.3. Rock

This sub-Chapter will present the results of the analyses with the change in fixed lengths respectively (6m, 8m, and 12m). Analyses for the each particular length is utilized by changing tendon material (Steel and GFRP). First, Plaxis analysis outputs; deformation and relative shear plots at 0.4 cm, several prescribed pull out values will be presented. Later, graphs for the points and cross-Sections described in Section 5.2.4 will be presented respectively.

6.3.1. 6m Fixed Length

6.3.1.1. Ground Anchor with Steel Bar. Pull-out test of a ground anchor with steel bar embedded in rock is simulated in this model. The fixed length of the ground anchor is 6m. In this analysis, the transition from elastic phase to plastic phase occurred prior to 0.4 cm pull out (at 0.2cm). Therefore only the deformation and relative shear plots for 0.4cm pull out will be presented in Figure 6.91 and Figure 6.92.

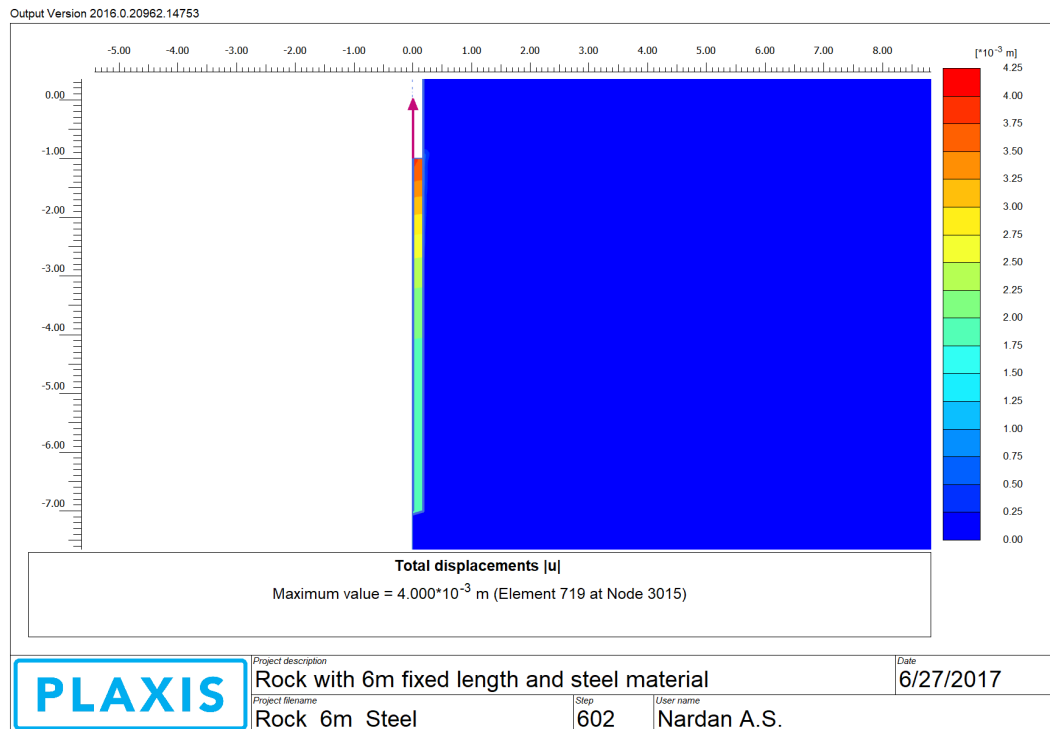


Figure 6.91. Deformation plot for 0.4 cm pull out.

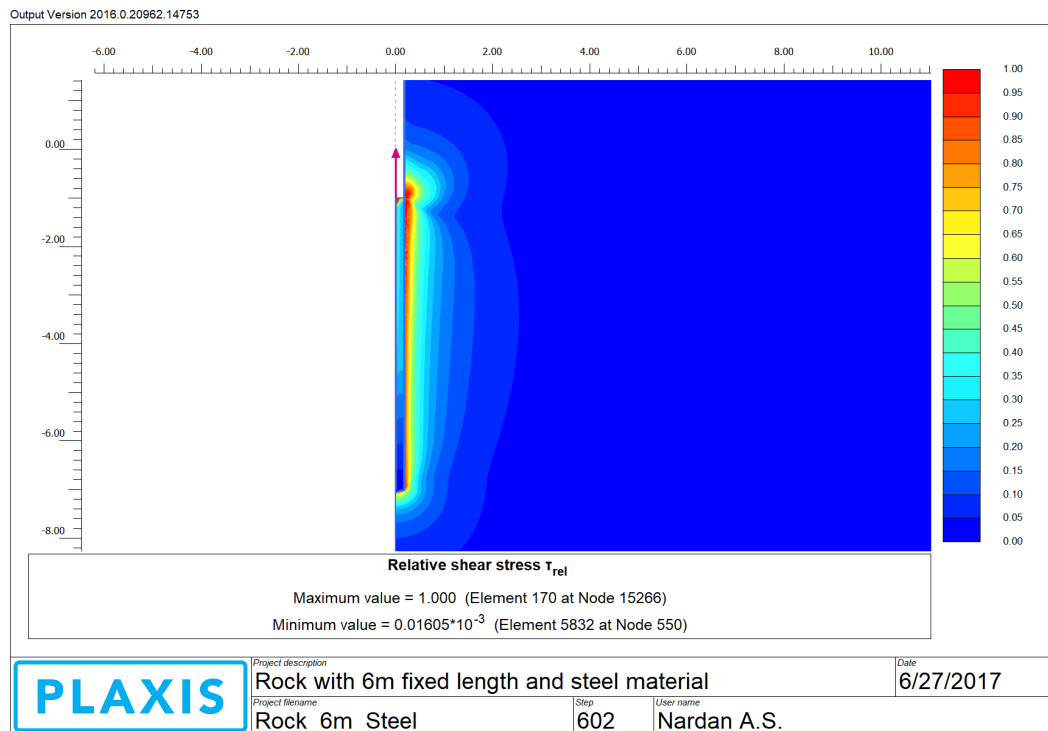


Figure 6.92. Relative Shear plot for 0.4 cm pull out.

Graphs for the load-displacement and mobilized shear versus “deformation on the top of the fixed length” may be seen in Figure 6.93 and Figure 6.94.

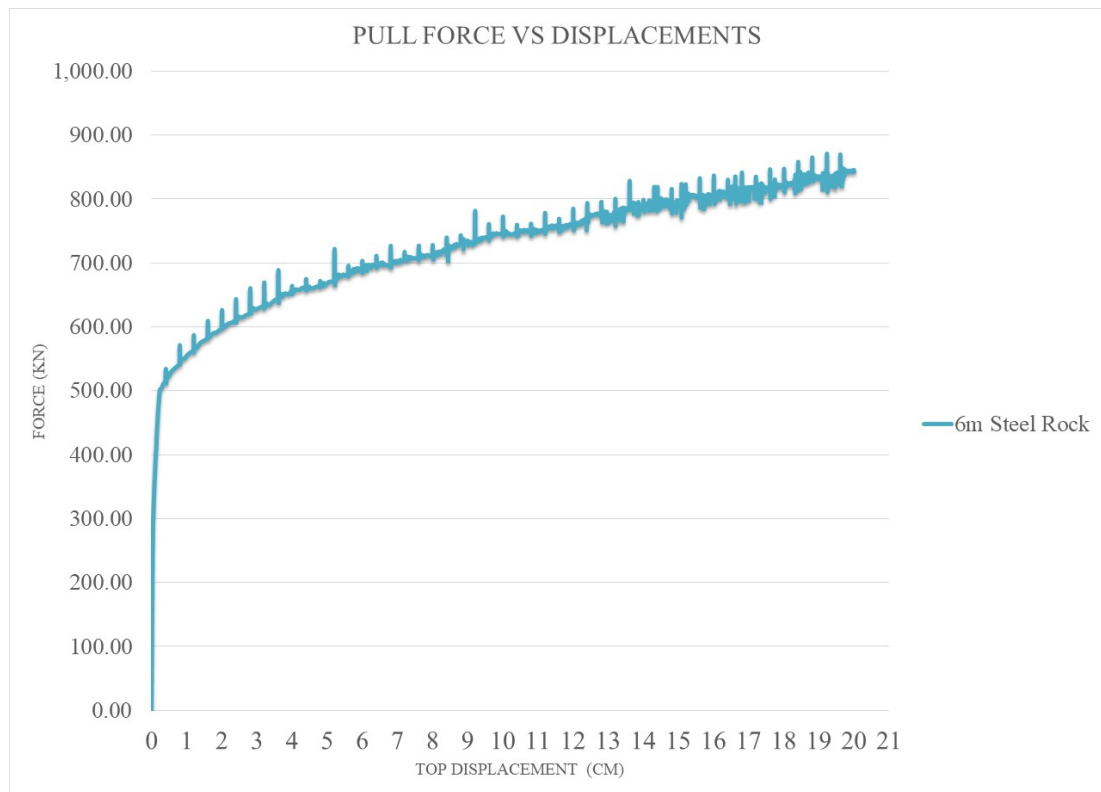


Figure 6.93. Load vs Displacement plot.

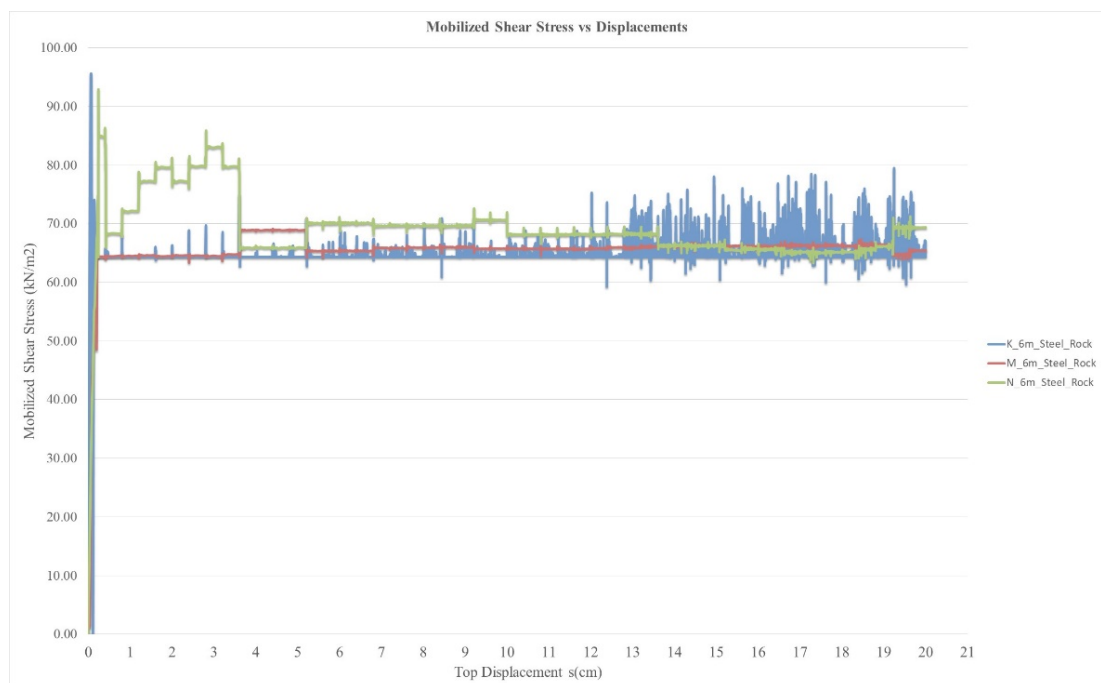


Figure 6.94. Mobilized Shear Stress vs Displacement plot for points K,M and N in the FE mesh.

The mobilized shear stress on the points K, M and N has reached the peak shear stress almost at the same displacement. Those three points mobilized the same shear stress due to high stiffness of the rock material for a short anchor length. This circumstance is more visible in the graph below;

As can be seen from Figure 6.95, the shear stress doesn't change remarkably along the fixed length zone. The Maximum pull-out force mobilized in 0.23 cm deformation, no remarkable difference is apparent between the shear stress graphs at different deformation levels.

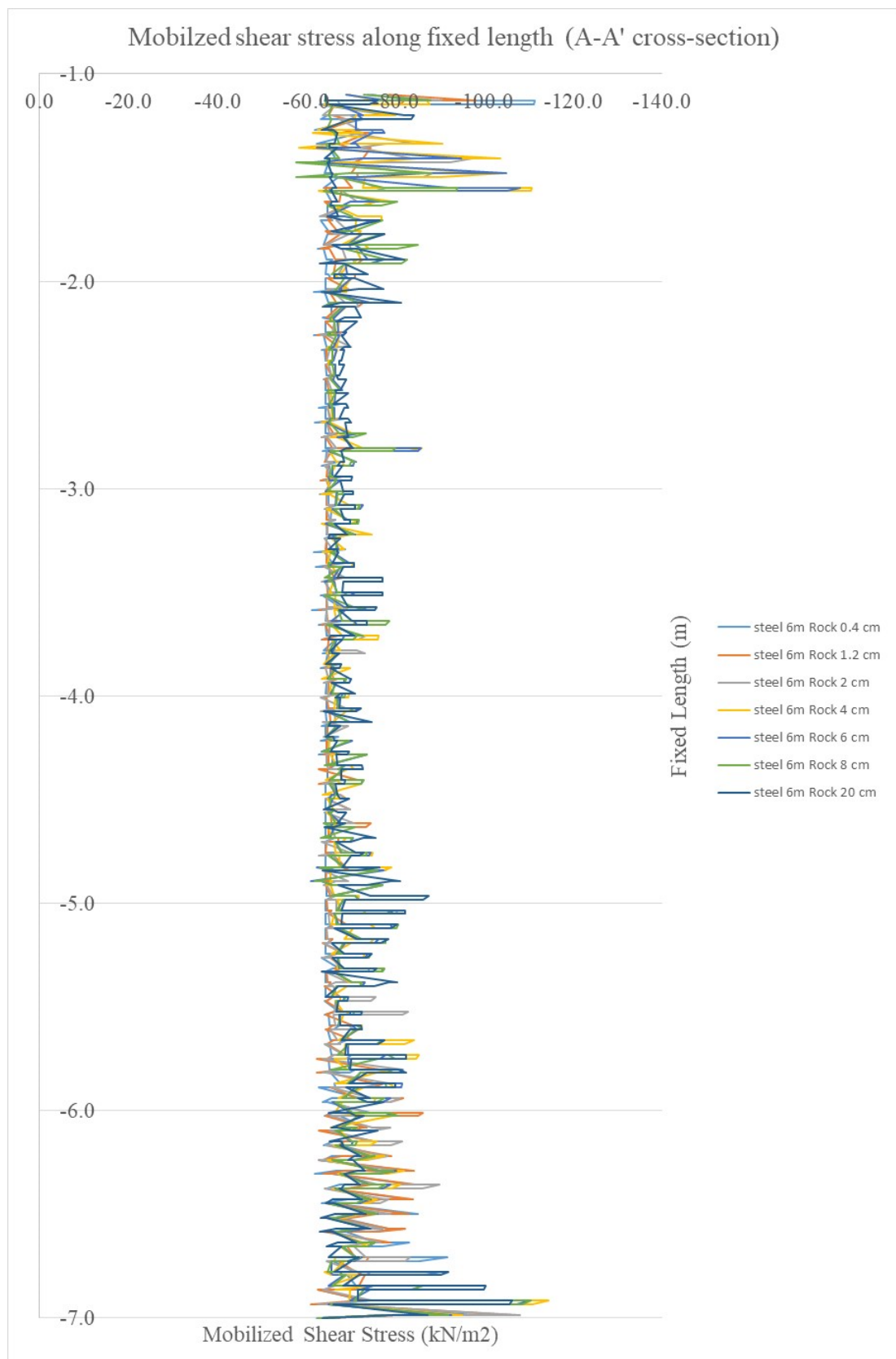


Figure 6.95. Mobilized shear stress for different deformation levels along A-A' cross-Section.

6.3.2. Ground Anchor with GFRP Bar

Pull-out test of a ground anchor with GFRP bar is simulated for the same conditions with the previous model. In this analysis, the transition from elastic phase to plastic phase occurred after 0.4cm pull out (at 0.92cm) likewise the previous model. Therefore the deformation and relative shear plots for 0.4cm and 1.2cm pull out will be presented in Figure 6.96-Figure 6.99.

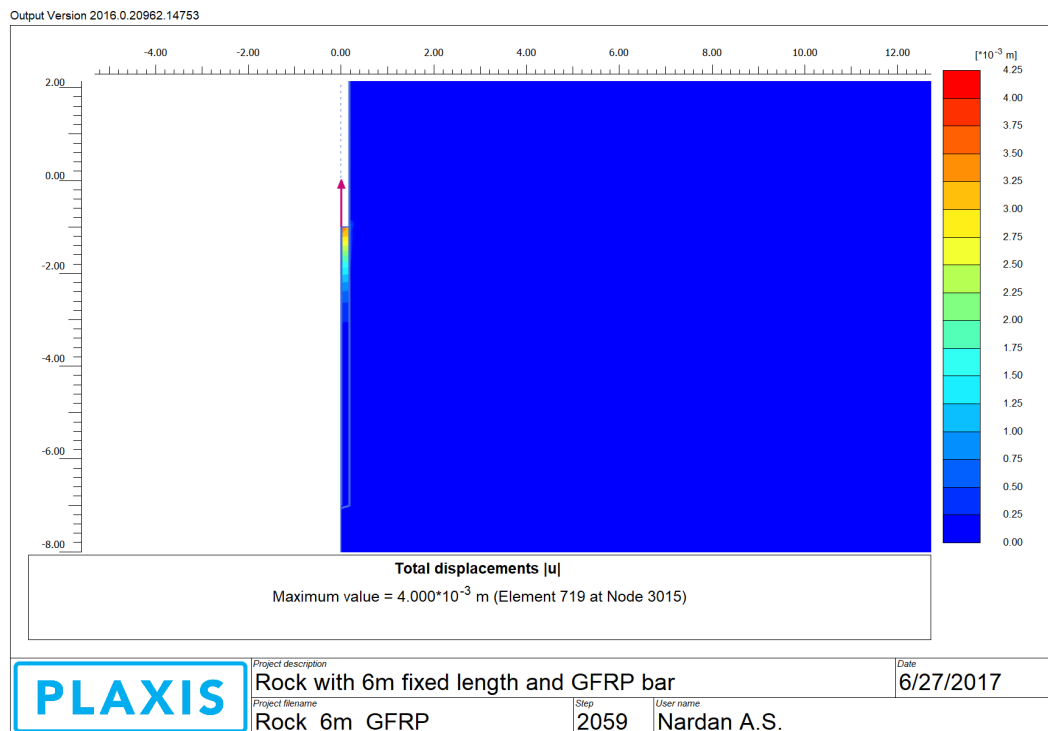


Figure 6.96. Deformation plot for 0.4 cm pull out.

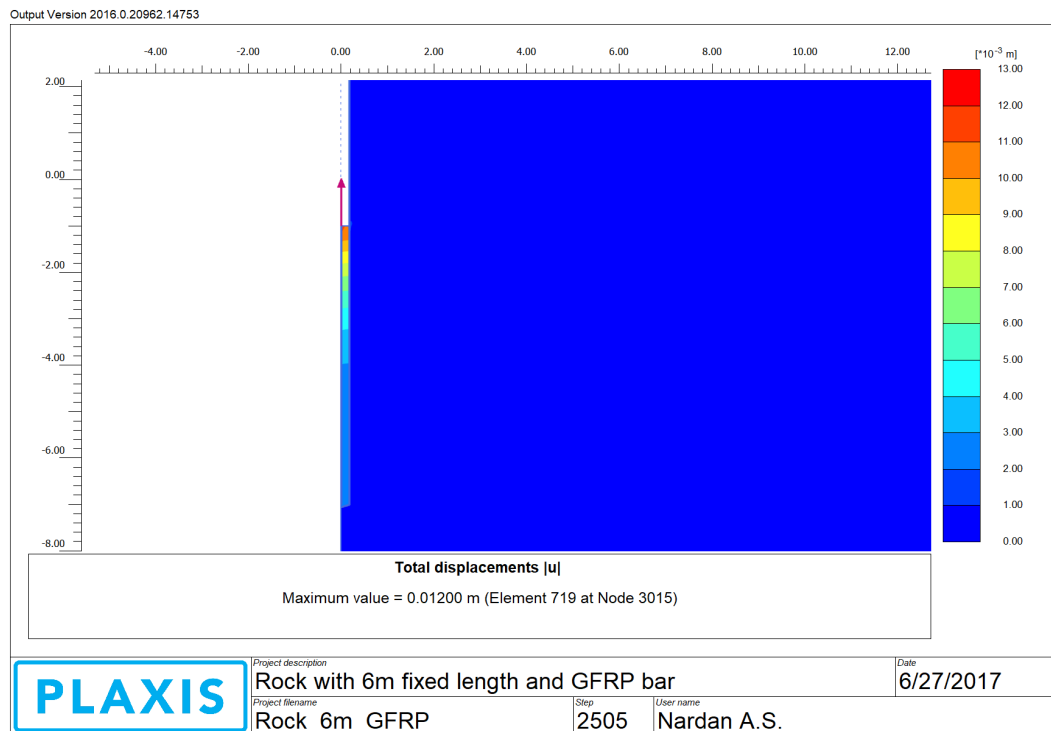


Figure 6.97. Deformation plot for 1.2 cm pull out.

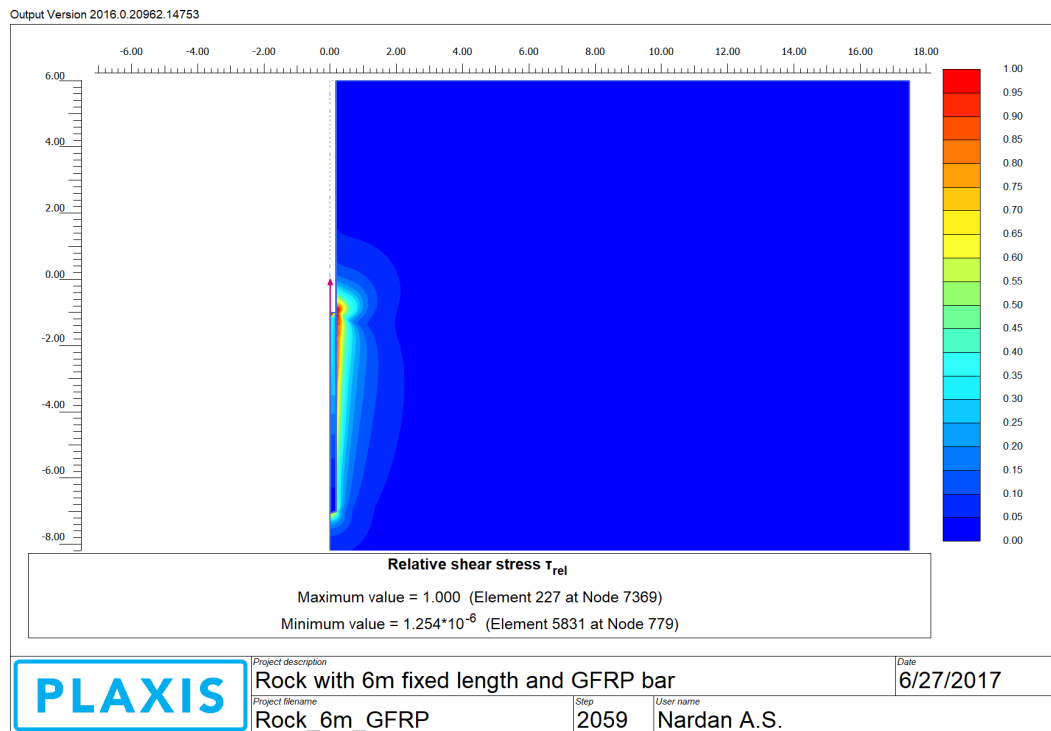


Figure 6.98. Relative Shear plot for 0.4 cm pull out.

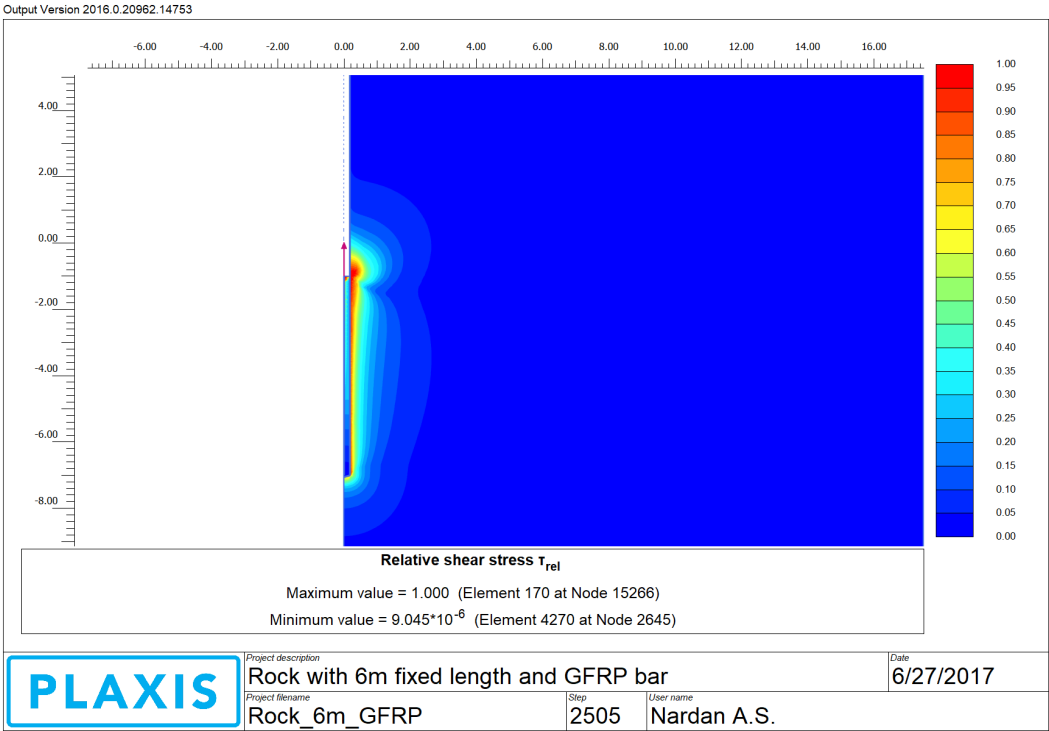


Figure 6.99. Relative Shear plot for 1.2 cm pull out.

Graphs for the load vs displacement and mobilized shear versus “deflection on the top of the fixed length” may be seen in Figure 6.100 and Figure 6.101.

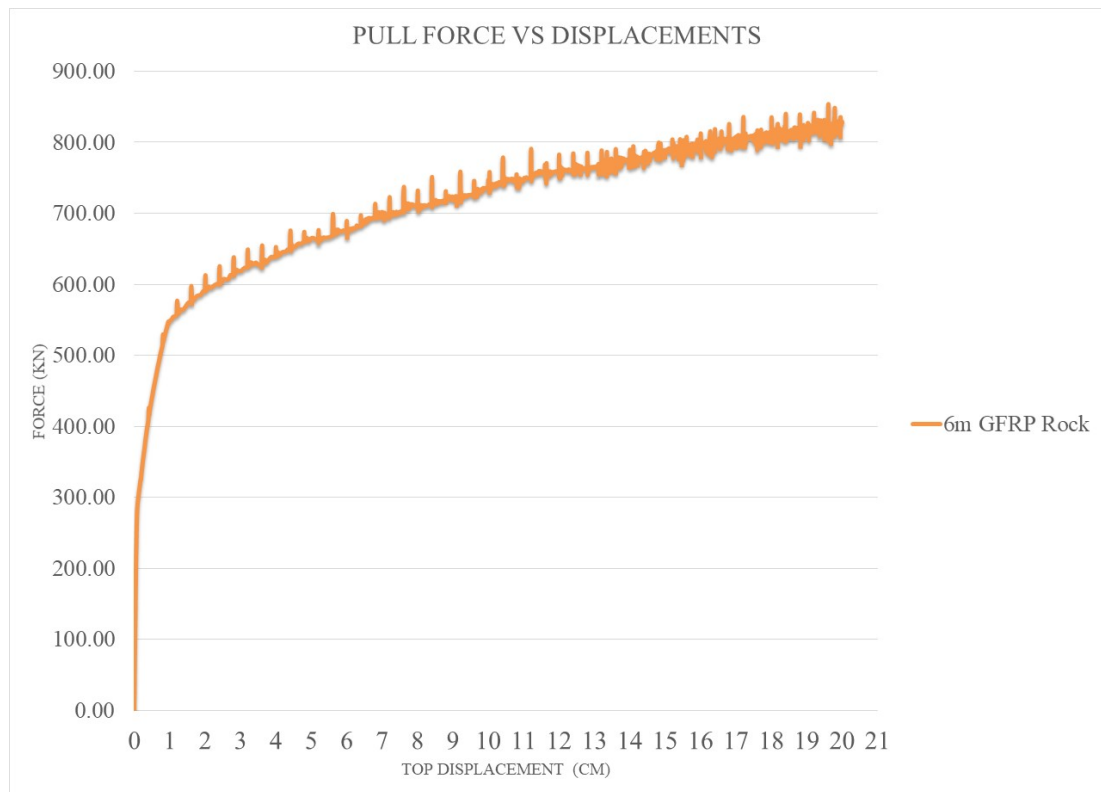


Figure 6.100. Load vs Displacement plot.

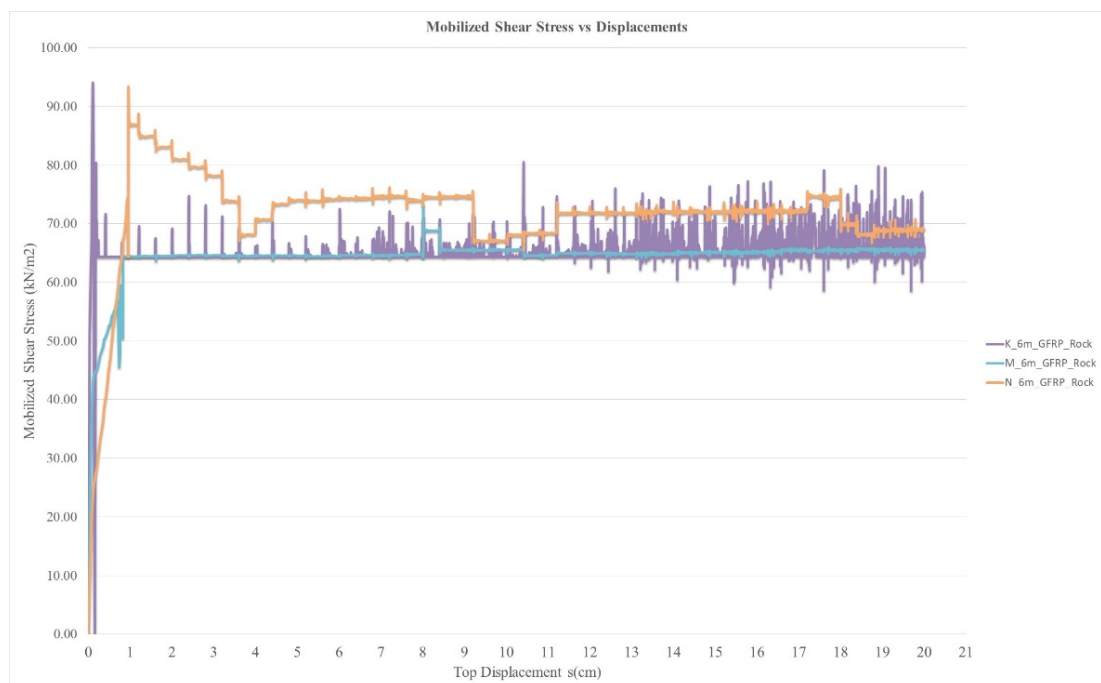


Figure 6.101. Mobilized Shear Stress vs Displacement plot for points K, M and N in the FE mesh.

As it can be expected, the behaviours of points; K, M, N for the ground anchor with GFRP bar and the steel bar are similar but deformation needed to reach those peak values are quite different from each other. The mobilized shear stress on the point which is at the top of the fixed length K, has reached the peak shear stress quickly and dropped to a residual value with a very steep gradient. Medium point M, reached the peak after 1.2cm deformation and stay at that value until failure. Distal point N on the other hand, mobilizes a peak value and then dropped to 80% of its shear stress value until failure. Those three points however mobilized almost same shear stress due to high stiffness of the rock material for a short anchor length. As can be seen from Figure 6.102, the shear stress has reached its peak value at 1.2cm of deformation. No difference is apparent with the increasing deformation levels, there is almost same shear stress mobilized all along the fixed length just same as in former case (steel bar).

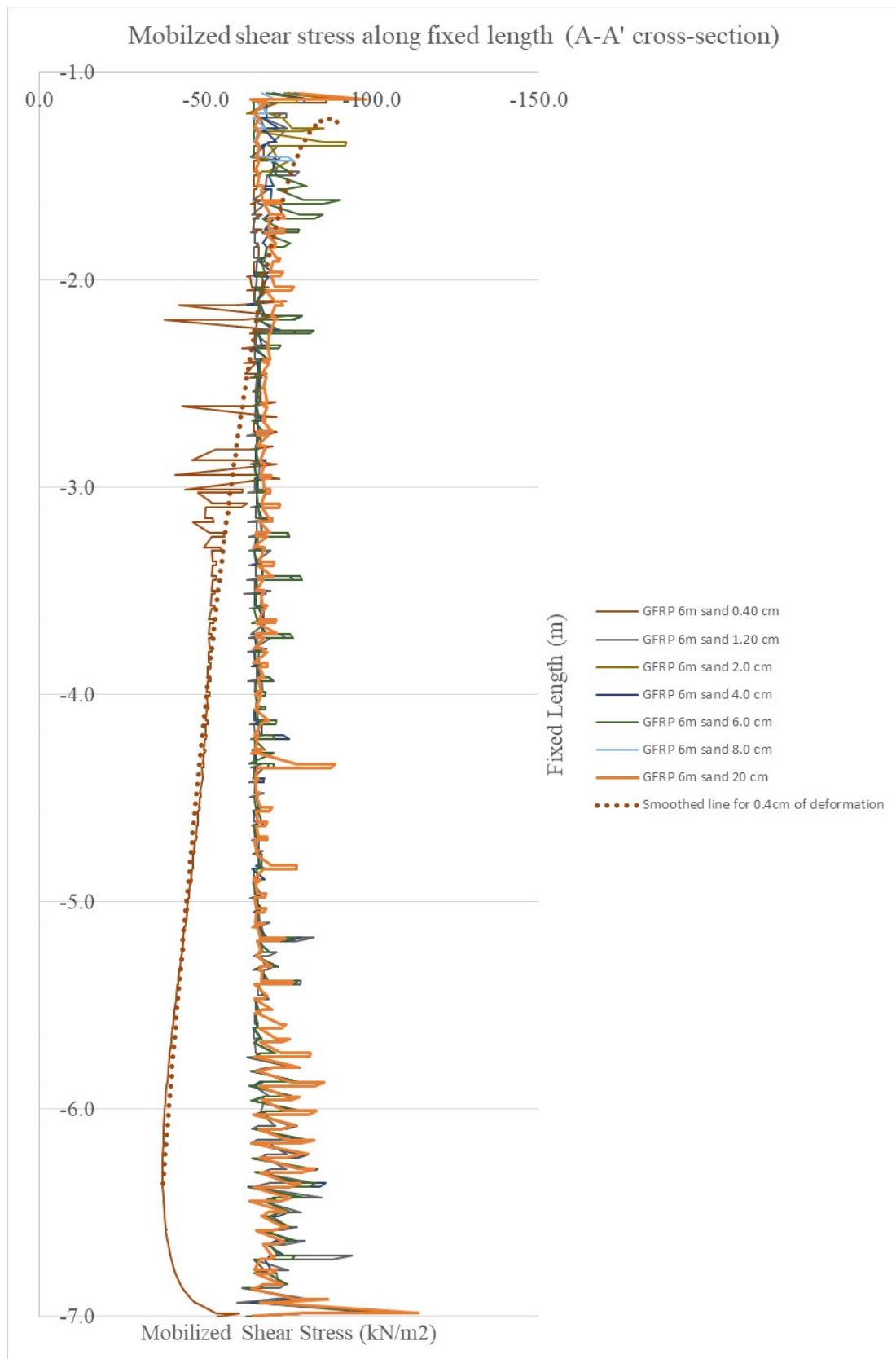


Figure 6.102. Mobilized shear stress for different load levels along A-A' cross-Section.

6.3.2.1. Comparison of the Results. As it can be anticipated, at this stress level in the rock material case with the 6m fixed length, there is a minor difference in maximum pull-out force between ground anchors with GFRP bar and conventional steel bar but it is insignificant for this case. Because like in dense sand, both ground anchor models (Conventional steel and GFRP) experienced an increase in capacity with increased pull-out displacement. Just as both of the material experienced the same behaviour and resulting ultimate load is not significantly different, this behaviour is not a distinctive feature for the present research. However this will be addressed in 8m fixed length and 12m fixed length anchor cases. The comparison of conventional Steel versus GFRP at the pull force-displacement graph may be seen below in Figure 6.103.

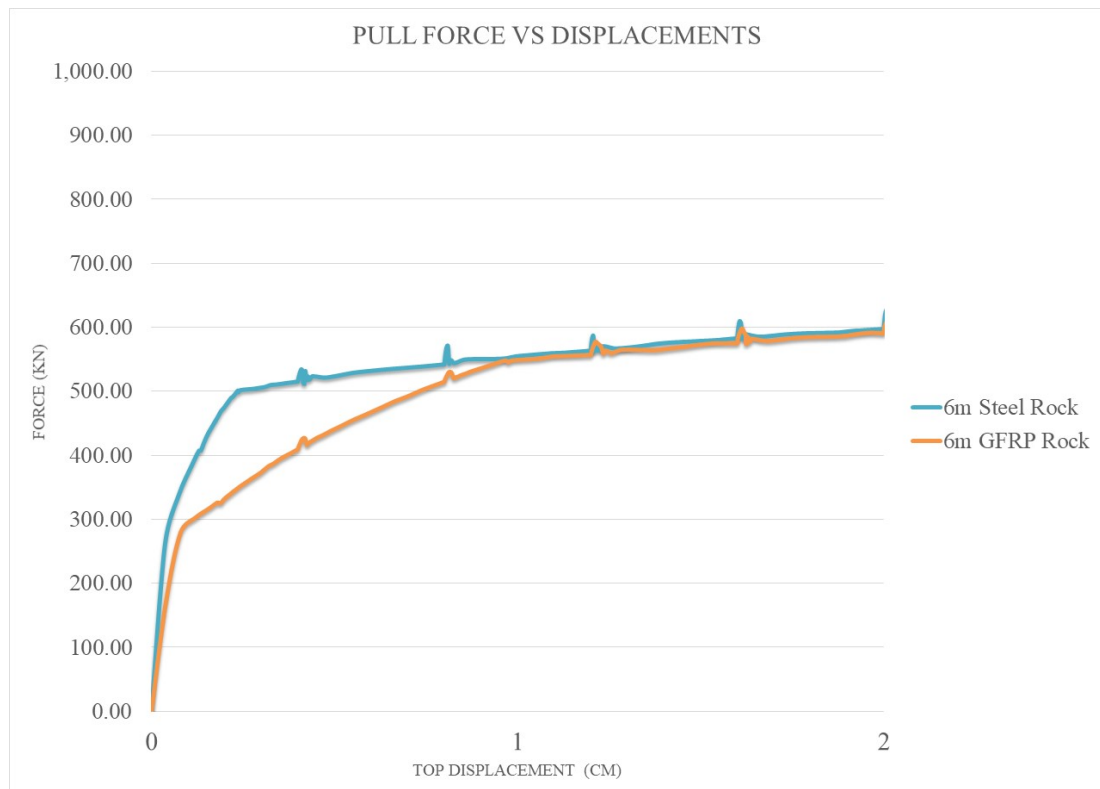


Figure 6.103. Load vs Displacement plot for the comparison of GFRP and Steel Bar.

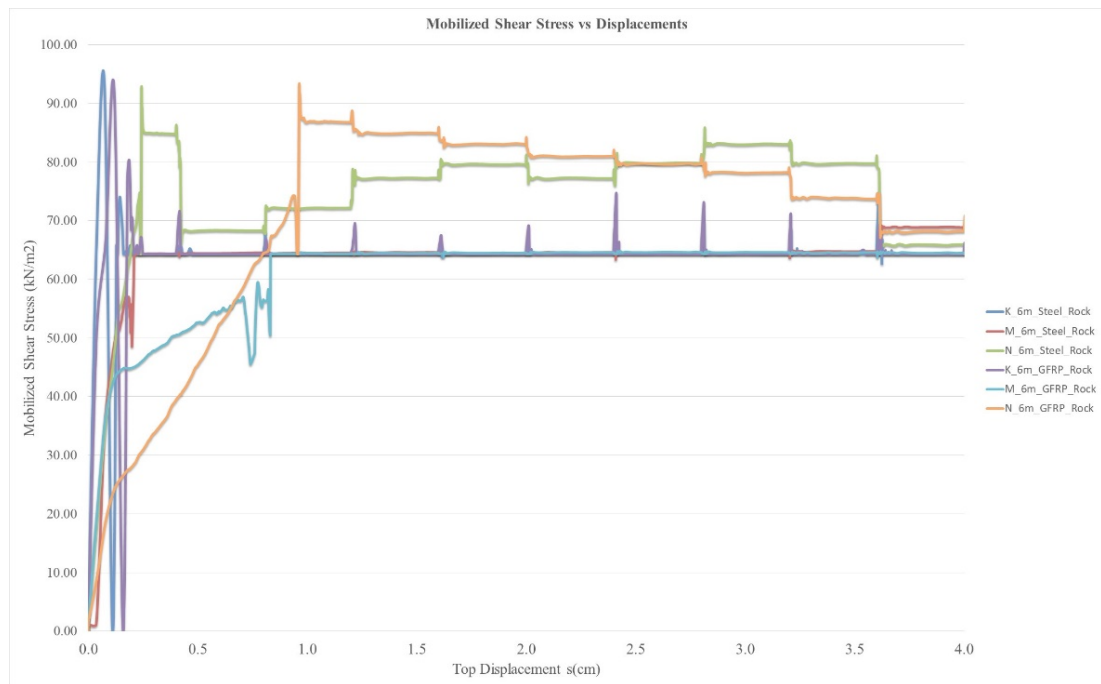


Figure 6.104. Mobilized Shear Stress vs Displacement plot for the comparison of GFRP and Steel Bar.

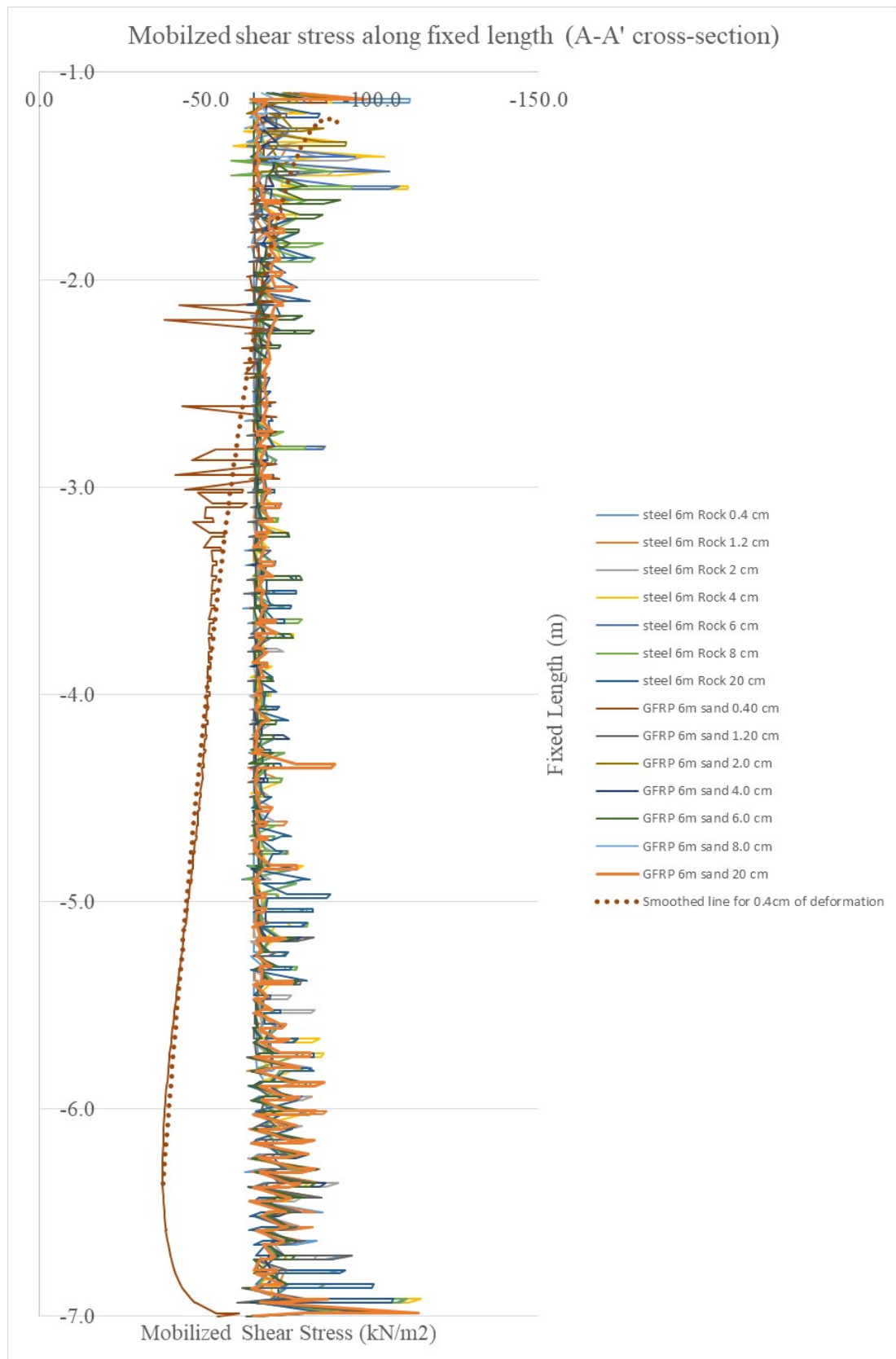


Figure 6.105. Mobilized shear stress for different load levels along A-A' cross-Section for the comparison of GFRP and Steel Bar.

6.3.3. 8m Fixed Length

6.3.3.1. Ground Anchor with Steel Bar. Pull-out test of a ground anchor with steel bar embedded in rock is simulated in this model. The fixed length of the ground anchor is 8m. In this analysis, the transition from elastic phase to plastic phase occurred after 0.4 cm pull out (at 0.56cm). Therefore only the deformation and relative shear plots for 0.4cm and 0.8cm pull out will be presented in Figure 6.107-Figure 6.111.

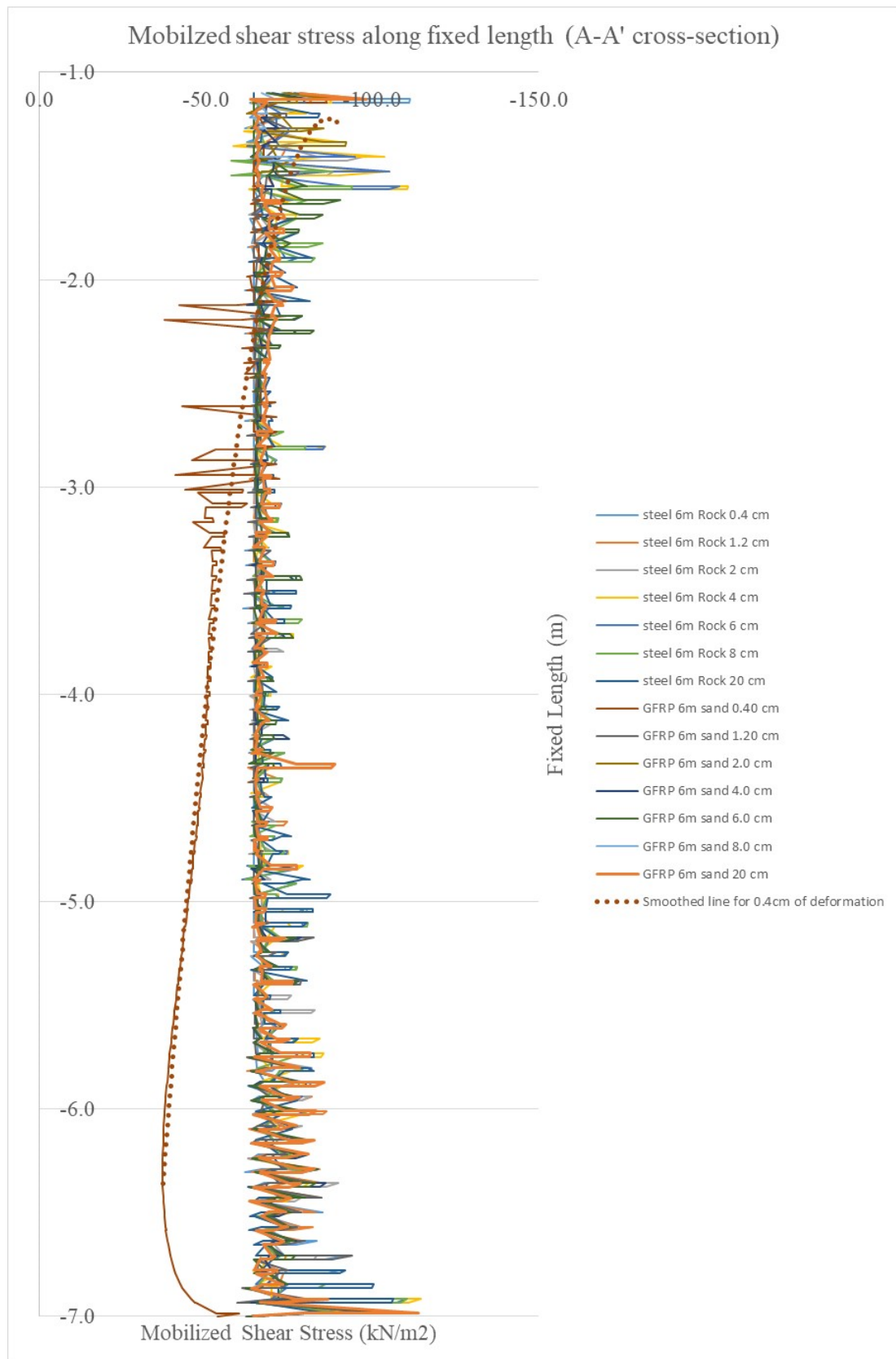


Figure 6.106. Mobilized shear stress for different load levels along A-A' cross-Section for the comparison of GFRP and Steel Bar.

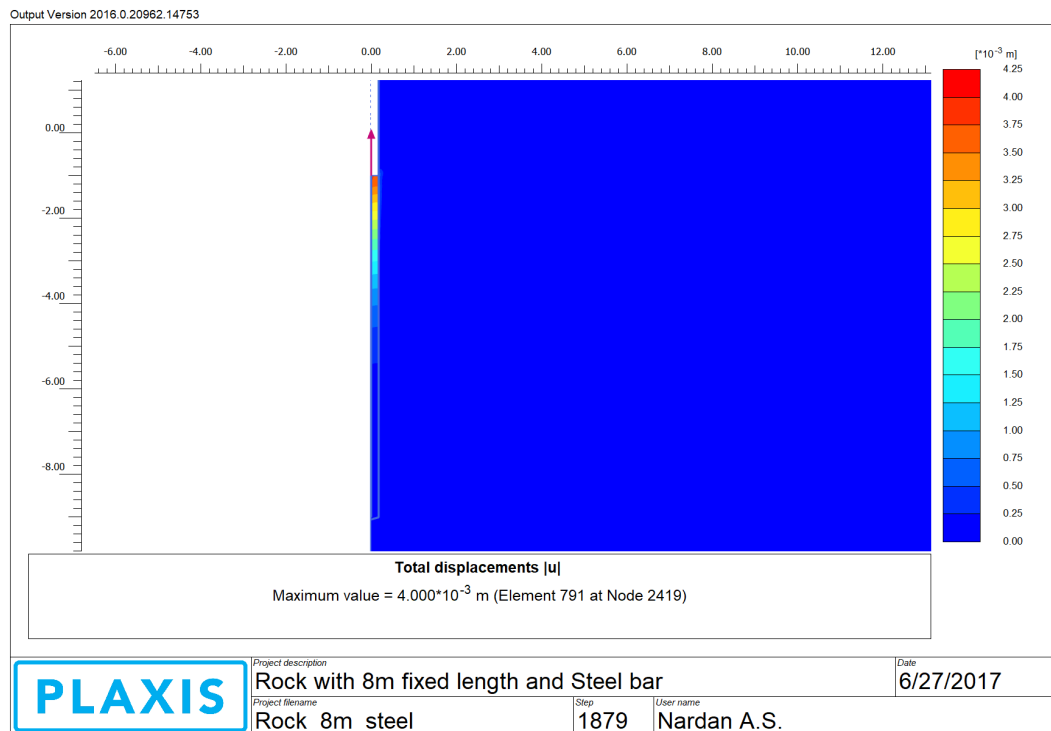


Figure 6.107. Deformation plot for 0.4 cm pull out.

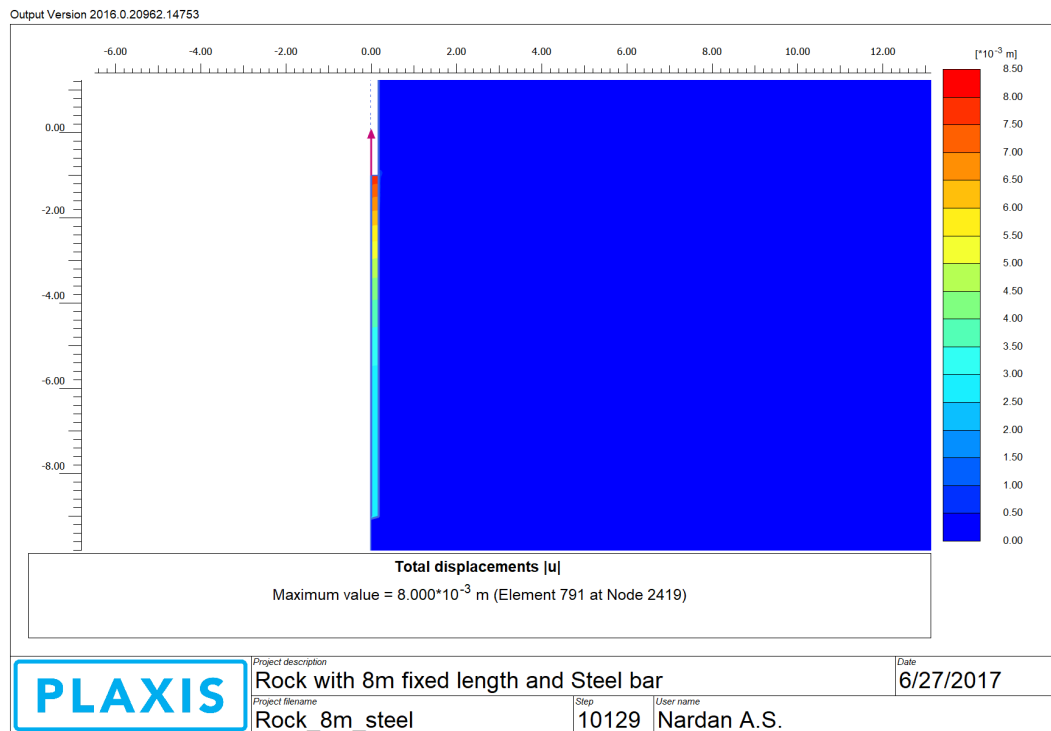


Figure 6.108. Deformation plot for 8mm pull out.

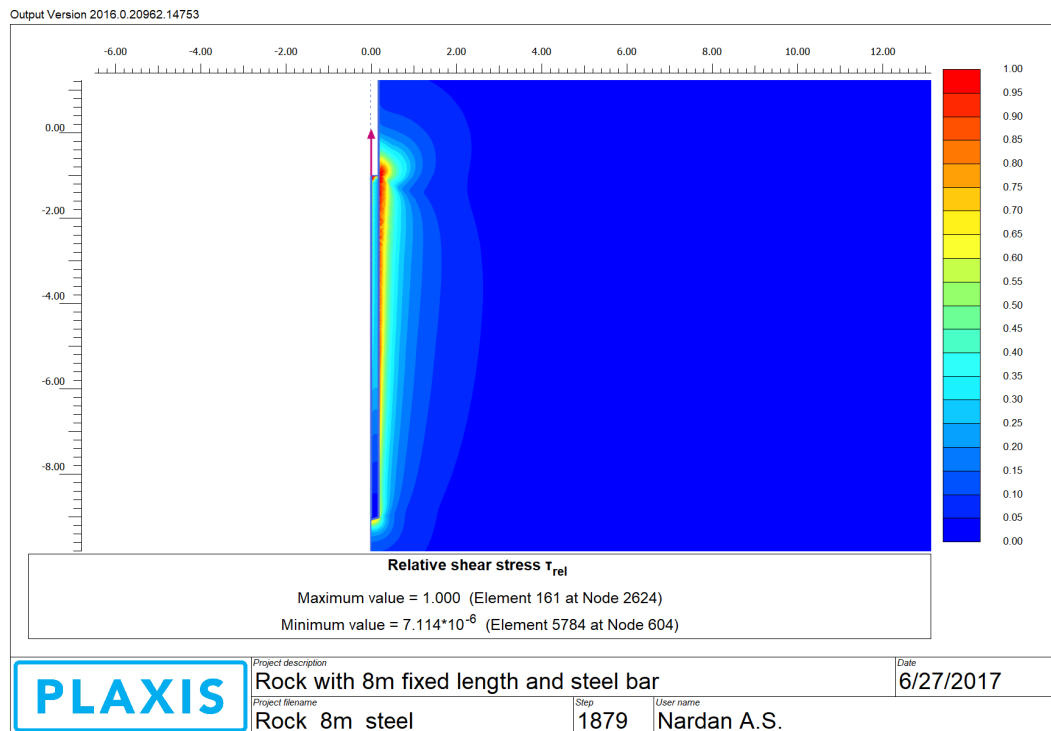


Figure 6.109. Relative Shear plot for 0.4 cm pull out.

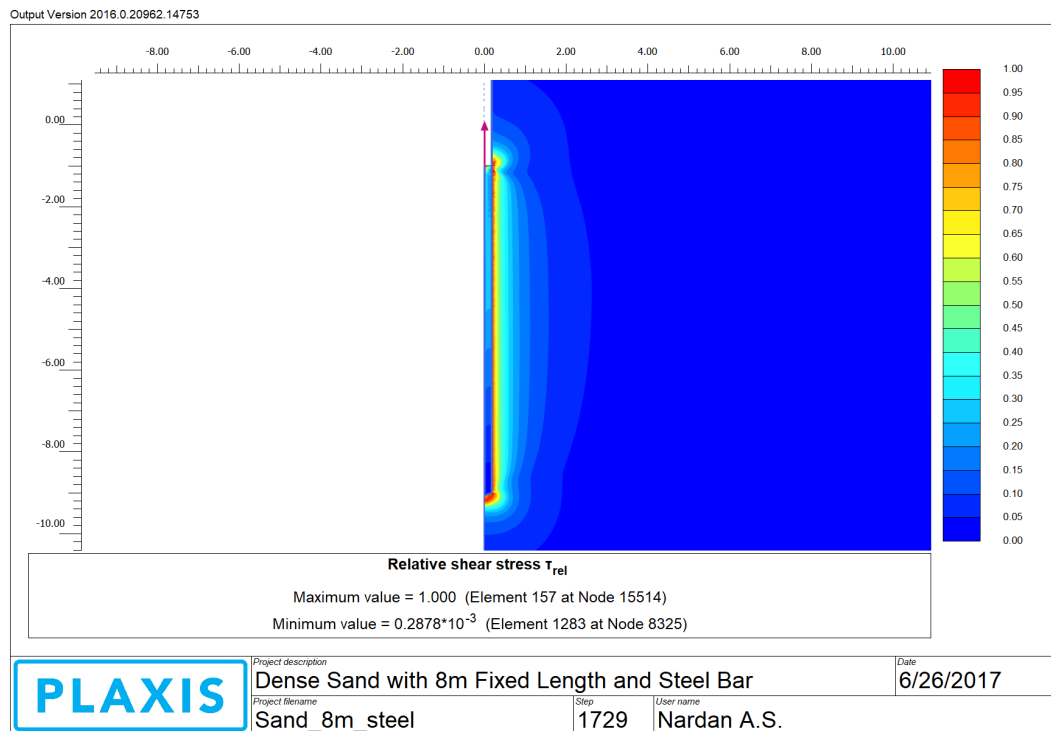


Figure 6.110. Relative Shear plot for 0.4 cm pull out.

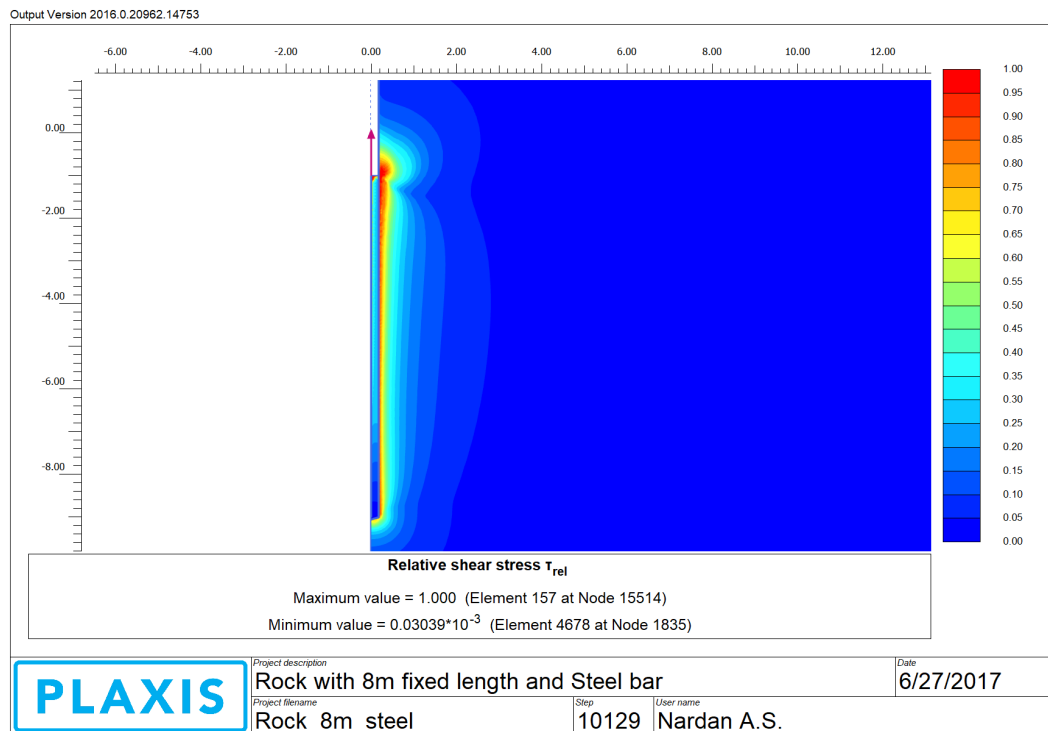


Figure 6.111. Relative Shear plot for 8mm pull out.

Pull-out resistance is around 1.4 times larger than ground anchor with 6m fixed length. Graphs for the load vs displacement and mobilized shear versus “deflection on the top of the fixed length” may be seen below. Also, it can be seen from the contour diagram the transmission of the deformations from the proximal to the distal part of the fixed length slightly slower than ground anchor with 6m fixed length.

Graphs for the load-displacement and mobilized shear versus “deflection on the top of the fixed length” may be seen in Figure 6.112 and Figure 6.113. The shear stress on point K is mobilized after 0.12cm of deformation. Points M and N, are mobilized after 0.42cm and 0.56cm deformation respectively. As can be seen from Figure 6.114, the shear stress has reached its peak value at 0.56cm of deformation. No difference is apparent. However, mobilized shear stress increased with the increase in depth.

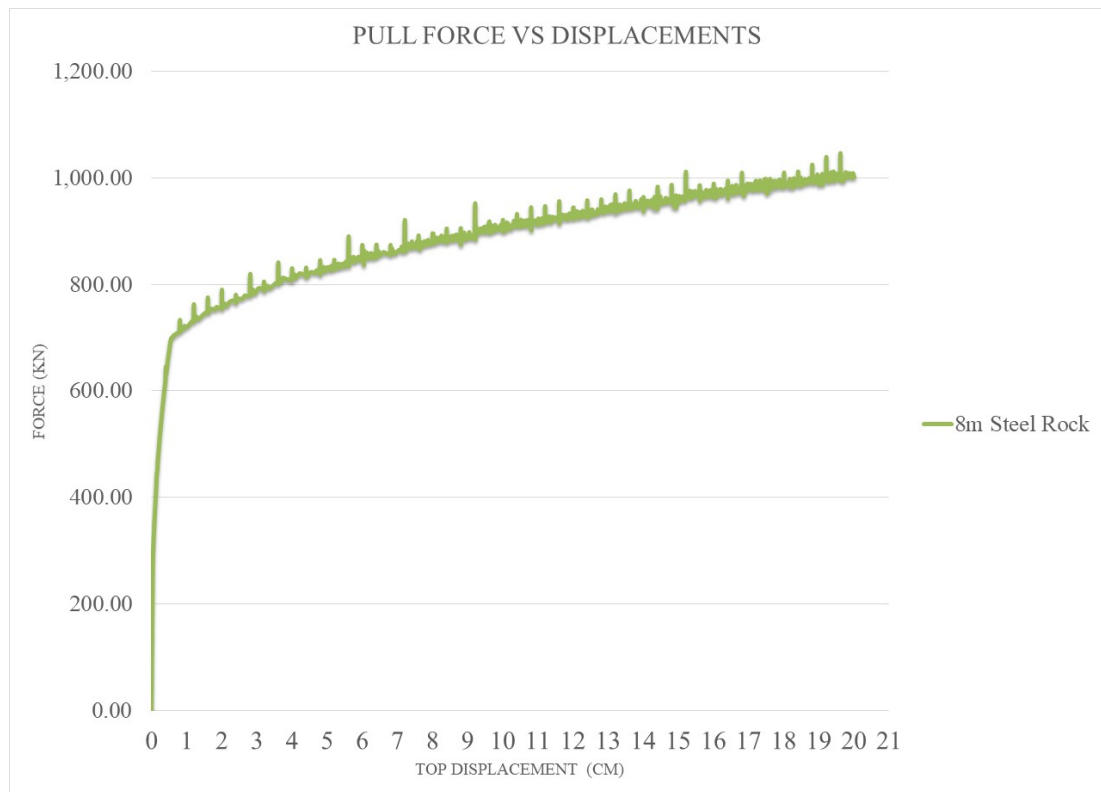


Figure 6.112. Load vs Displacement plot.

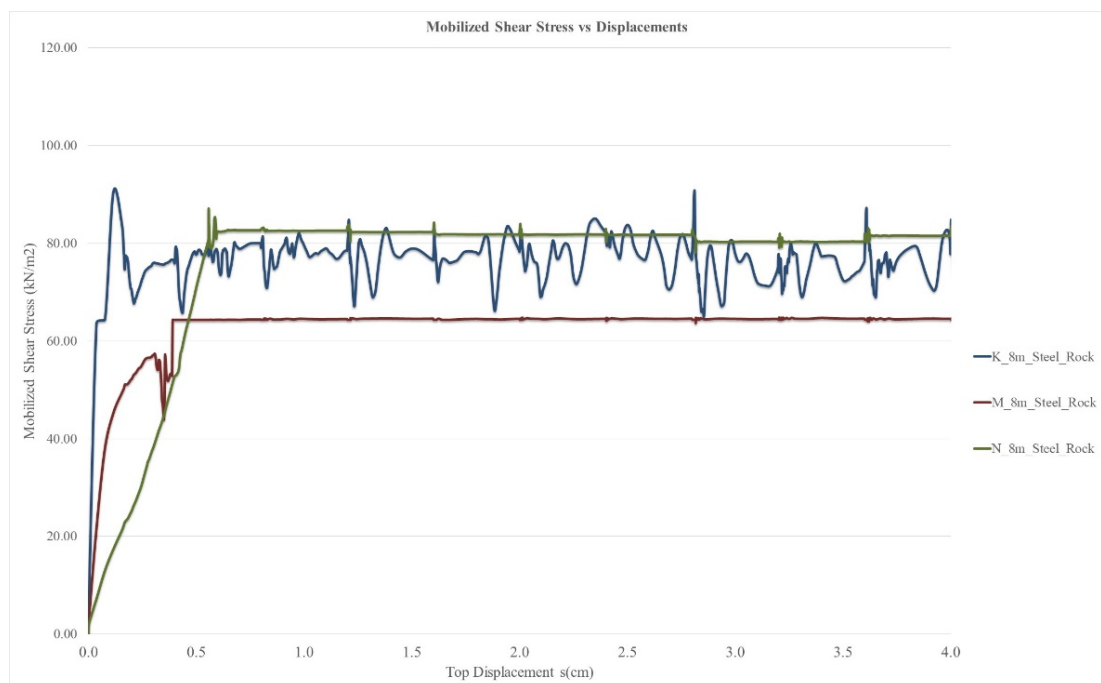


Figure 6.113. Mobilized Shear Stress vs Displacement plot for points K, M and N in the FE mesh.



Figure 6.114. Mobilized shear stress for different load levels along A-A' cross-Section.

6.3.3.2. Ground Anchor with GFRP Bar. Pull-out test of a ground anchor with GFRP bar is simulated for the same conditions with the previous model. In this analysis, the transition from elastic phase to plastic phase occurred after 0.4cm pull out (at 2.53cm). Therefore, the deformation and relative shear plots for 0.4cm and 2.8cm pull out will be presented in Figure 6.115-Figure 6.119. As anticipated, in this model, the transfer of the deformations from proximal to the distal part of fixed length is very slow. The deformations which is developed from the uplift in top of the fixed length isn't transferred to the end in no portion until 3.6cm of pull out deformation. This can be easily seen from the Figure 6.116 and Figure 6.117.

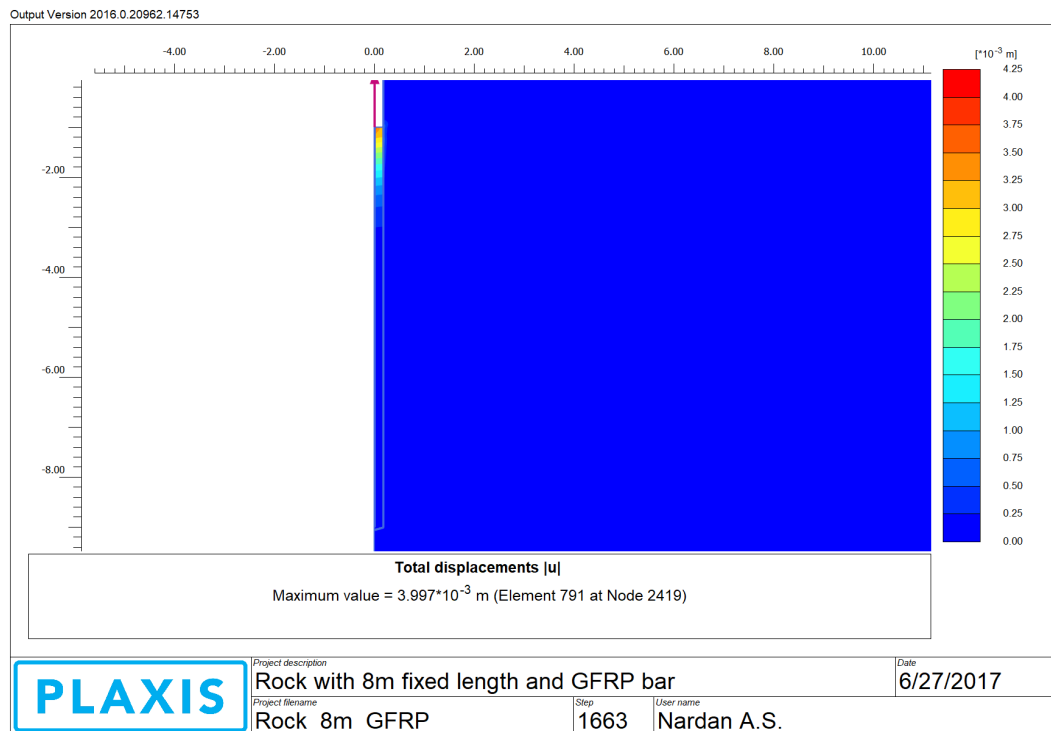


Figure 6.115. Deformation plot for 0.4 cm pull out.

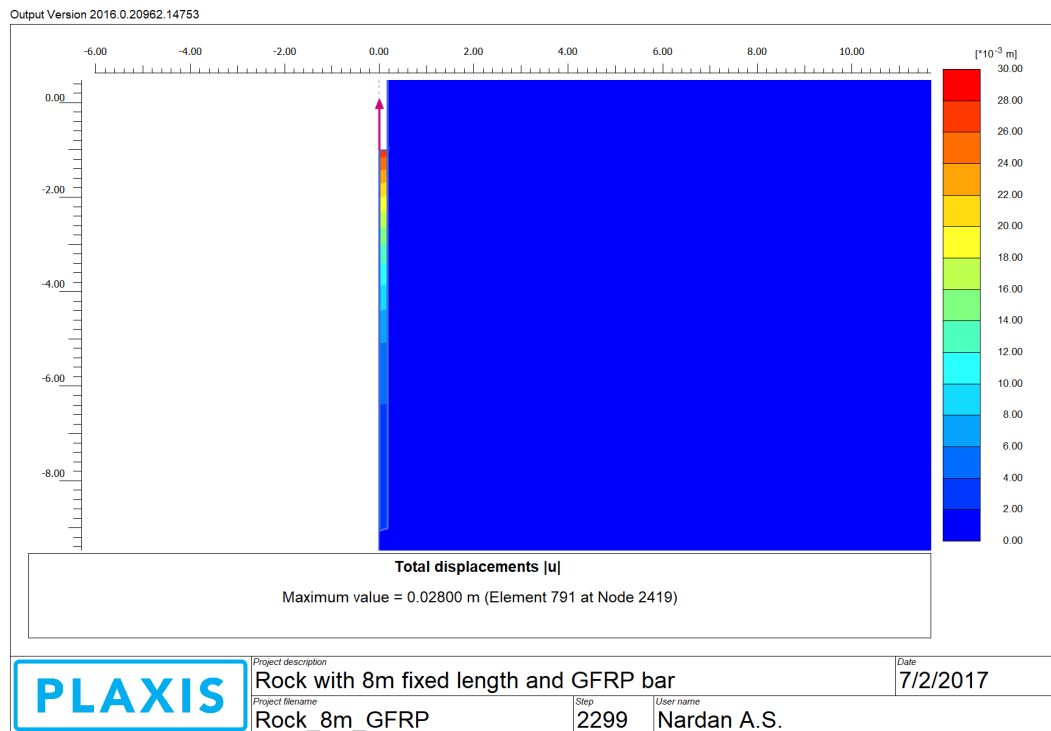


Figure 6.116. Deformation plot for 2.8 cm pull out.

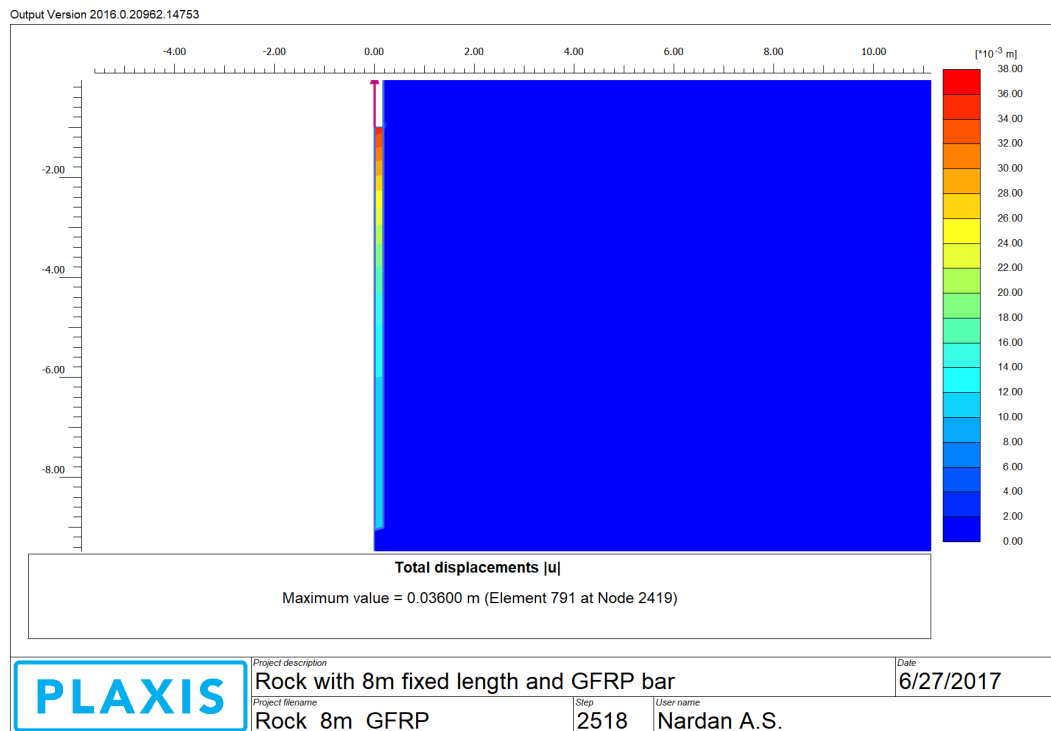


Figure 6.117. Deformation plot for 3.6cm pull out.

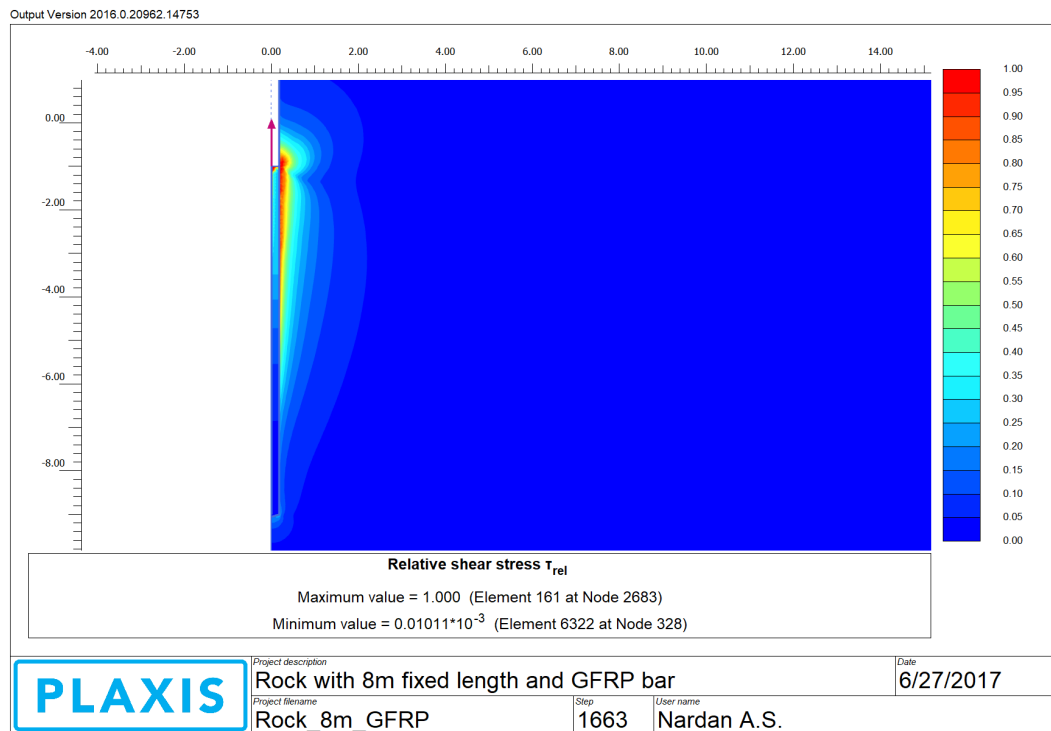


Figure 6.118. Relative Shear plot for 0.4 cm pull out.

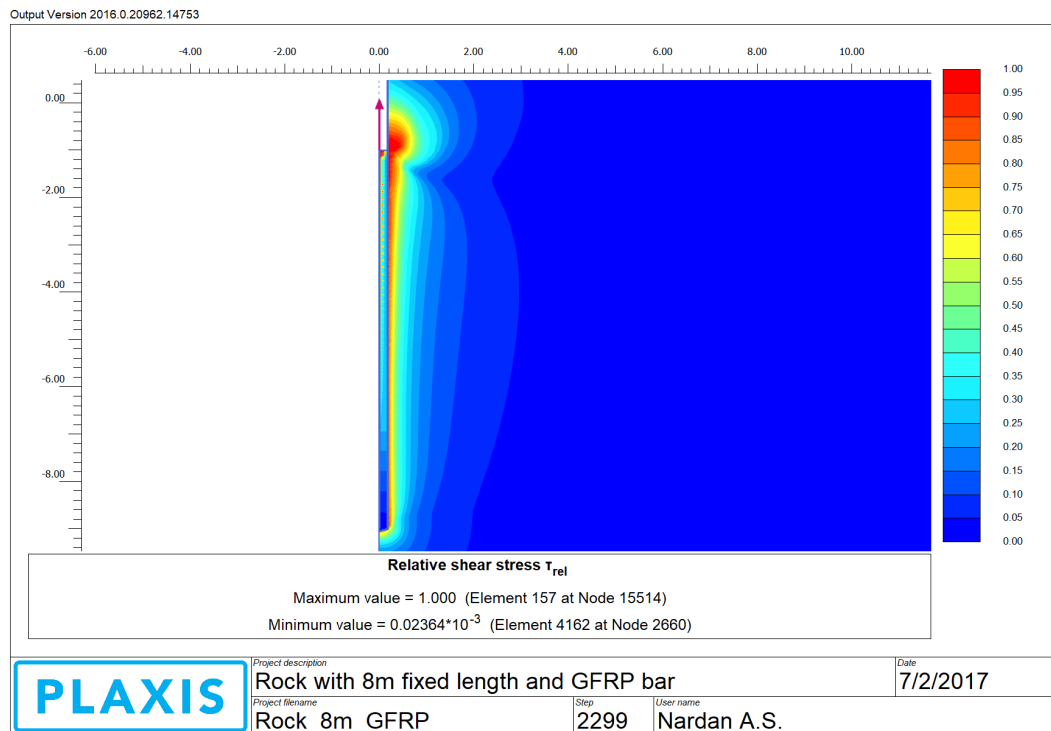


Figure 6.119. Relative Shear plot for 2.8 cm pull out.

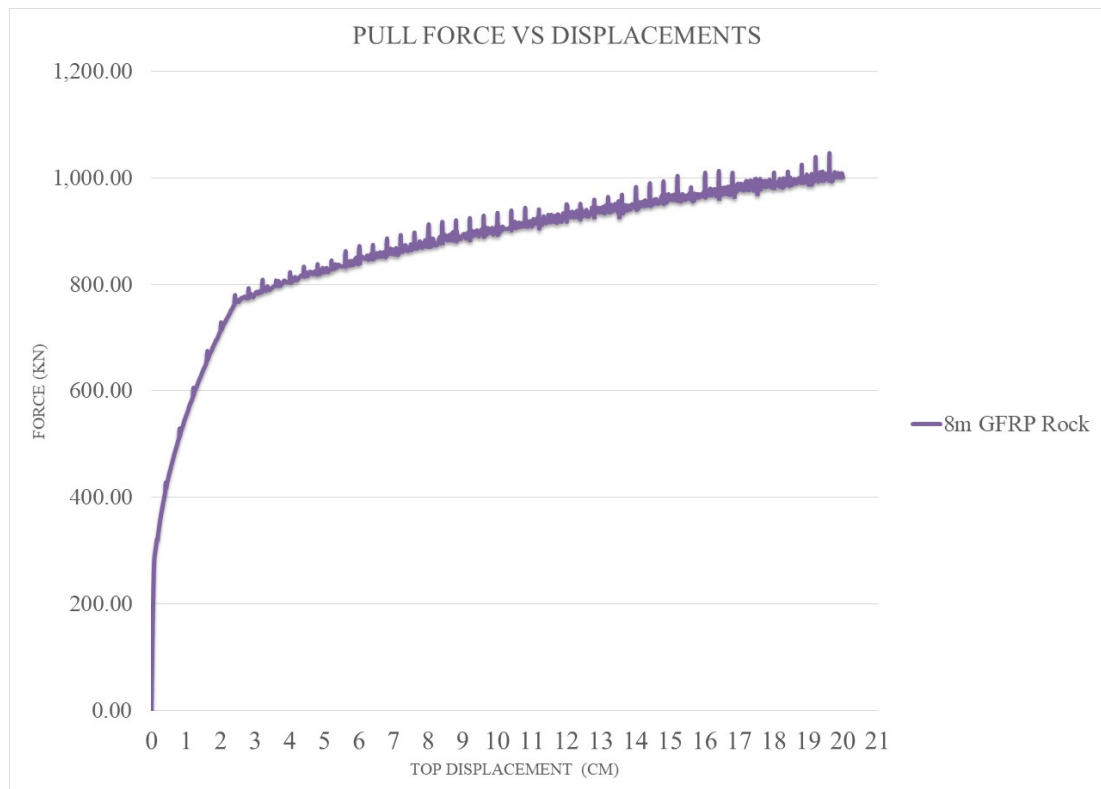


Figure 6.120. Load vs Displacement plot.

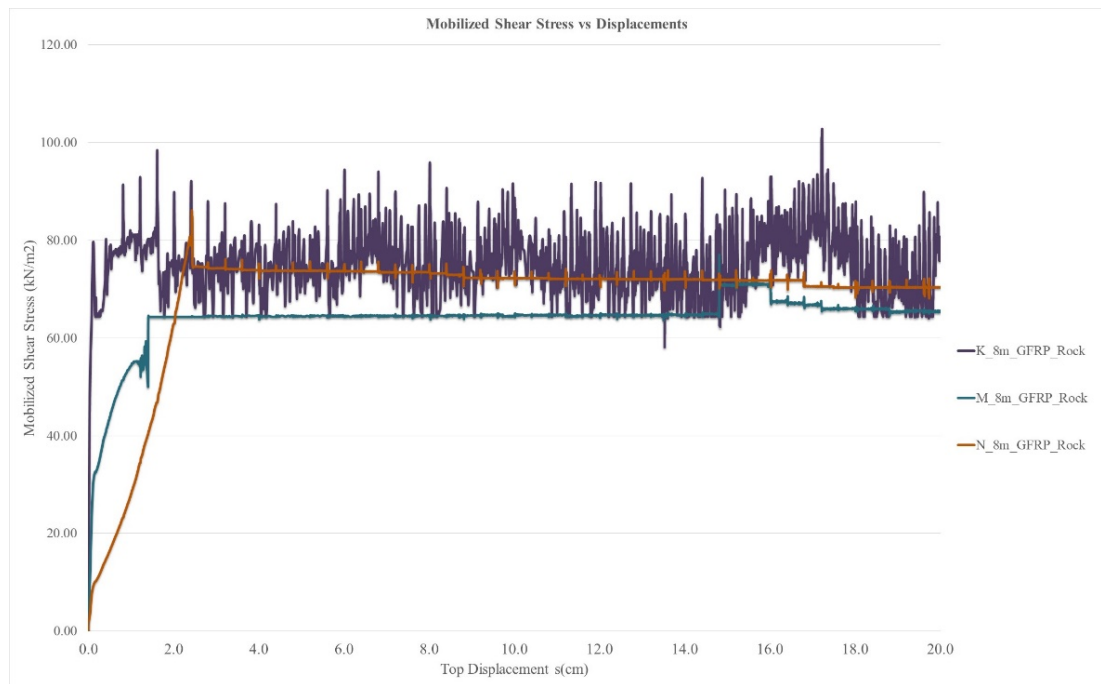


Figure 6.121. Mobilized Shear Stress vs Displacement plot.

Graphs for the load vs displacement and mobilized shear to “deflection on the top of the fixed length” may be seen in Figure 6.120 and Figure 6.121. The shear stress on point K is mobilized full shear stress after 0.12cm of deformation. Points M and N are mobilized full shear stress after 1.43cm and 2.53cm deformation respectively. As it can be seen from Figure 6.122, the shear stress didn’t change remarkably along the shear zone at fixed length. The peak value reached at 2.53 cm deformation at the top almost instantly, no difference is apparent but the change in the stress at the top of the fixed length decreased to residual state likewise in the previous case.

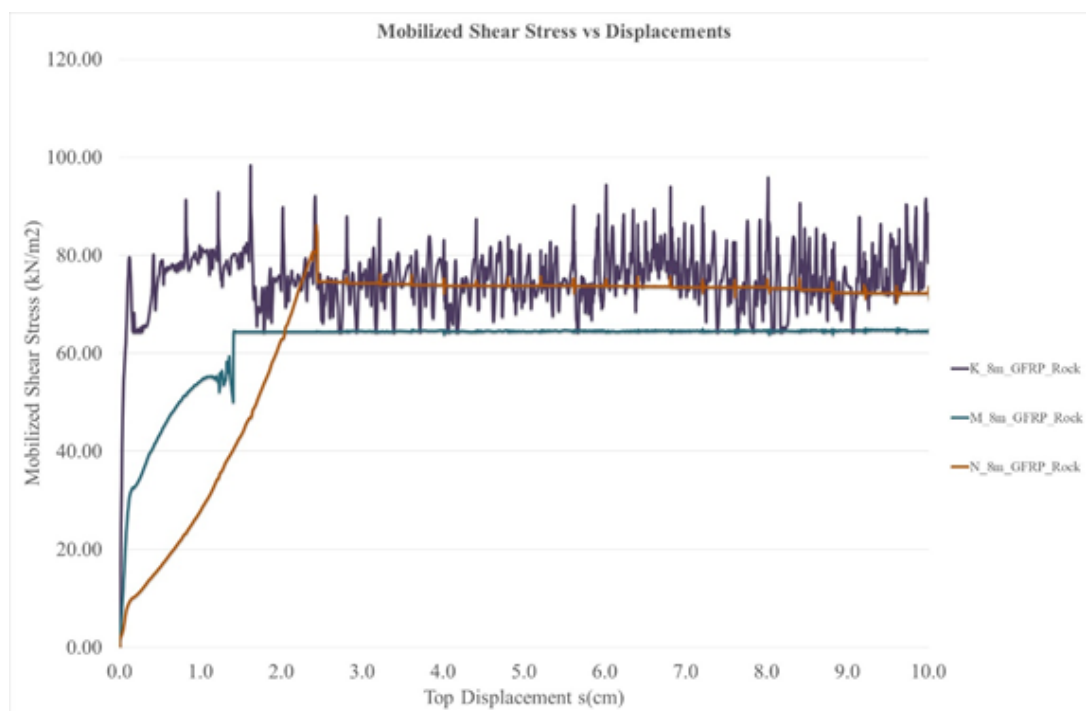


Figure 6.122. Mobilized shear stress for different load levels along A-A' cross-Section.

6.3.3.3. Comparison of the Results. As it can be seen from both three graphs, more deformations are needed for 8m fixed length ground anchor with GFRP bar to mobilize the full pull-out resistance force comparing with the steel. Even though the mobilization of the shear force at the top (point K) of the fixed length is identical in two distinct cases, the rate of deformation which is needed for the mobilization of the full shear stress of the medium and distal end of the fixed length, five times more for GFRP when comparing with steel (the mobilized shear stress at 0.56 cm top deformation for steel is the same with the mobilized shear stress at almost 2.53 cm top deformation for GFRP). This proportion would be extreme if there was a need for ultimate load capacity for design purposes. However, factor of safety for tendon failure is commonly taken as 2 and specifications foresee a factor of safety of 3 for bond failure for permanent purposes. This leads us to a need of 1/3 of the ultimate load. Therefore, smaller deformations will be needed to mobilize the allowable design load. By using this assumption for design purposes, the deformation needed to mobilize allowable load is around 0.05 cm and is not significant at all.

In both two cases, like in the 8m and 6m fixed length cases, the mobilized shear stress in the grout/ground zone stays in the same value after a specific deformation value but the load in pull force vs deformation graph increases slightly. It is not a distinctive feature for the present research since as asserted the behaviour is identical in both of these models. However this will be addressed again for this model regardless. The B-B' cross-Section may be seen in Figure 6.123;

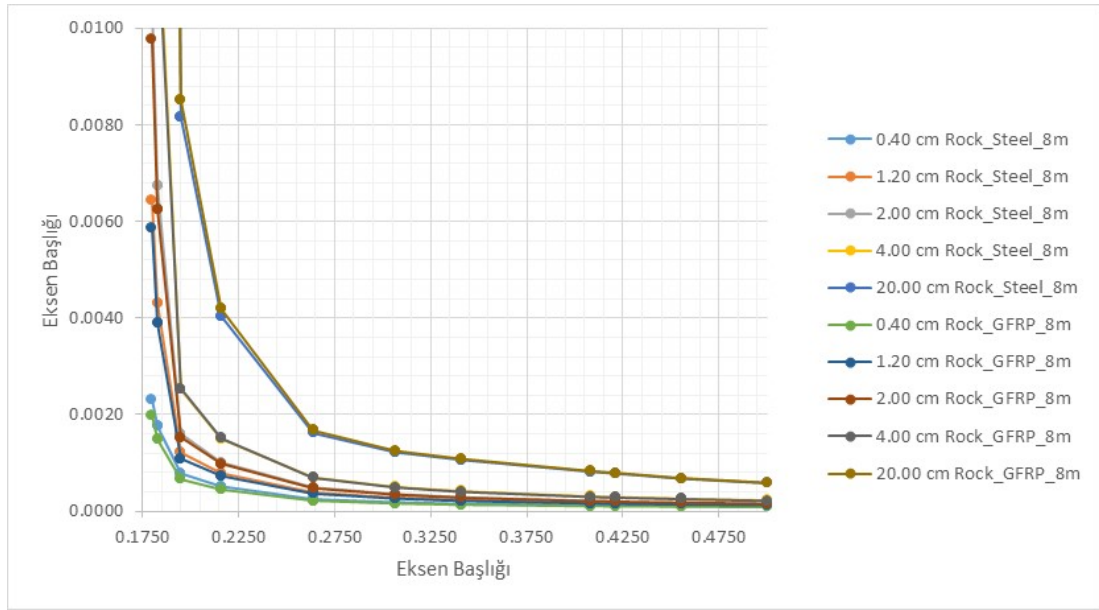


Figure 6.123. Mobilized ground deformation for different deformations on top along B-B' cross-Section.

As it can be seen from Figure 6.123, after full mobilization of the shear stress at grout-ground interface, the shear zone thickness around the anchor increases progressively. The rise with low inclination in the pull-out force/deformation graph is the result of this behaviour. This behaviour on the other hand is very similar in both cases, so it is not distinctive. The comparison of conventional steel versus GFRP at pull force-displacement may be seen in Figure 6.124 and Figure 6.125. As can be anticipated from Figure 6.124 and Figure 6.125, the difference between the GFRP and conventional Steel bar in ground anchorage is significant at distal part (point N) however, by using allowable load levels the difference between Steel and GFRP in the rate of deformation needed to mobilize full shear stress, is decreased to an insignificant gap.

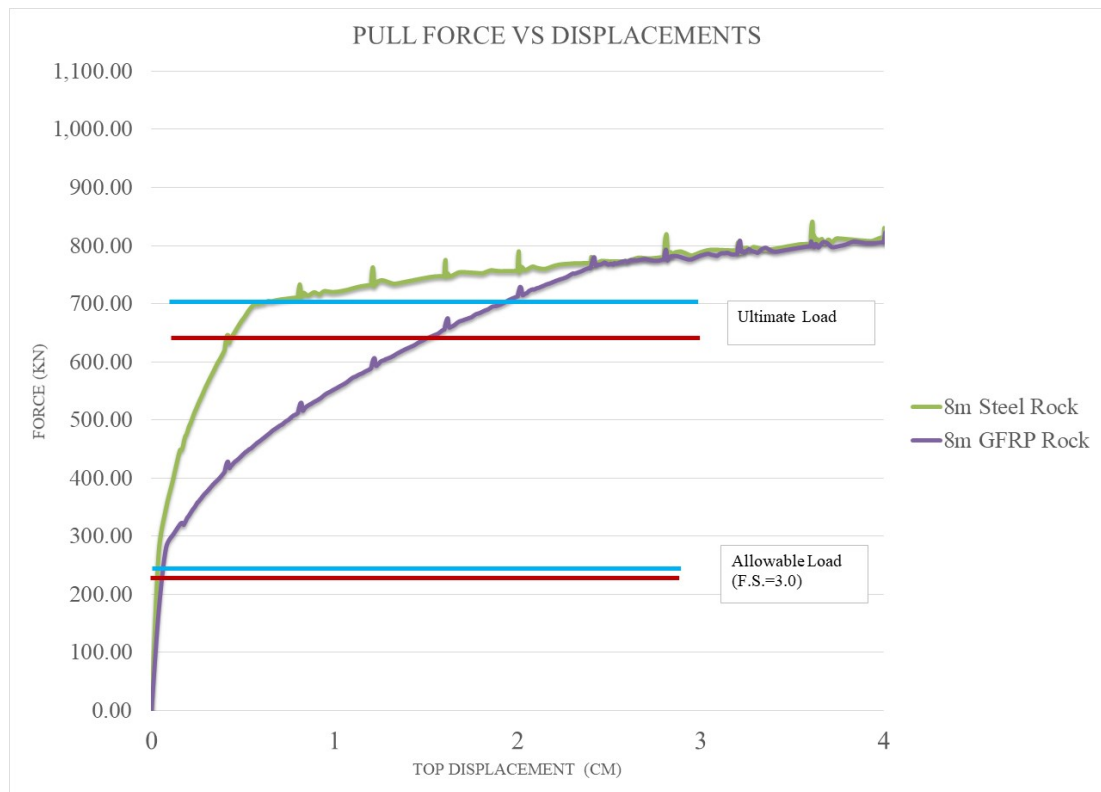


Figure 6.124. Load vs Displacement plot for the comparison of GFRP and Steel Bar.

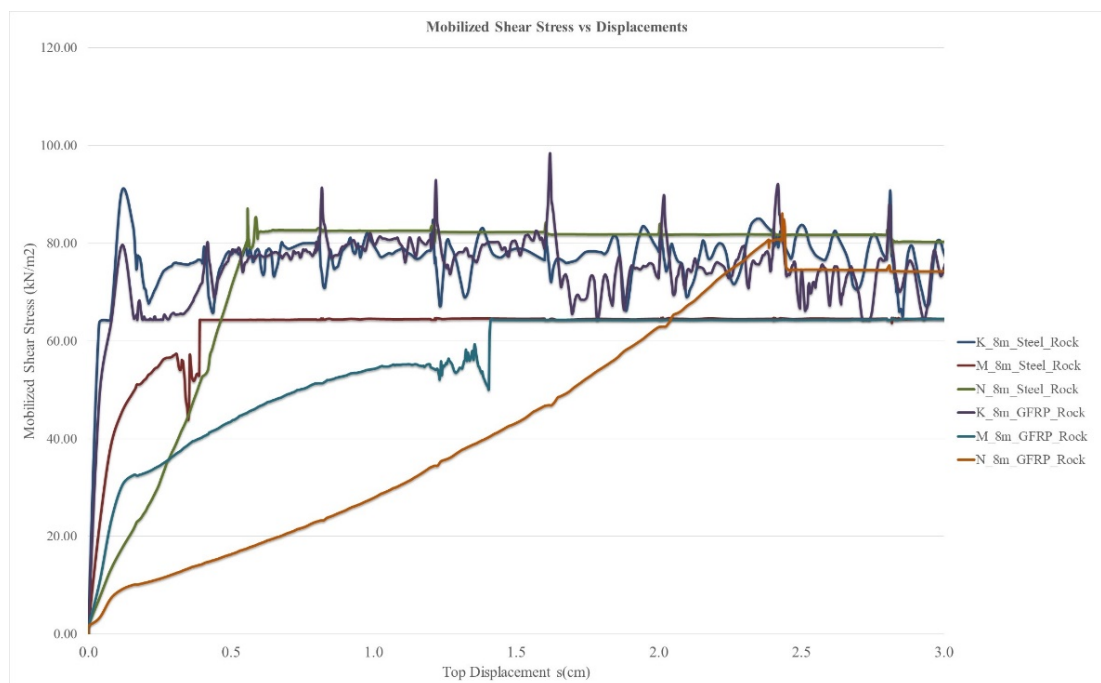


Figure 6.125. Mobilized Shear Stress vs Displacement plot for the comparison of GFRP and Steel Bar for points K, M and N in the FE mesh.



Figure 6.126. Mobilized shear stress for different load levels along A-A' cross-Section for the comparison of GFRP and Steel Bar.

6.3.4. 12m Fixed Length

6.3.4.1. Ground Anchor with Steel Bar. Pull-out test of a ground anchor with steel bar embedded in rock is simulated. The fixed length of the ground anchor is 12m. In this analysis, the transition from elastic phase to plastic phase occurred after 0.4 cm pull out (at 1.51cm). Therefore the deformation and relative shear plots for 0.4cm and 1.6cm pull out will be presented in Figures Figure 6.127-Figure 6.131. As anticipated, in this model, the transfer of the deformations from proximal to the distal part of fixed length is very slow. The deformations which is developed from the uplift in top of the fixed length isn't transferred to the end in no portion until 2.4 cm pull out deformation. This can be easily seen from the Figure 6.128 and Figure 6.129.

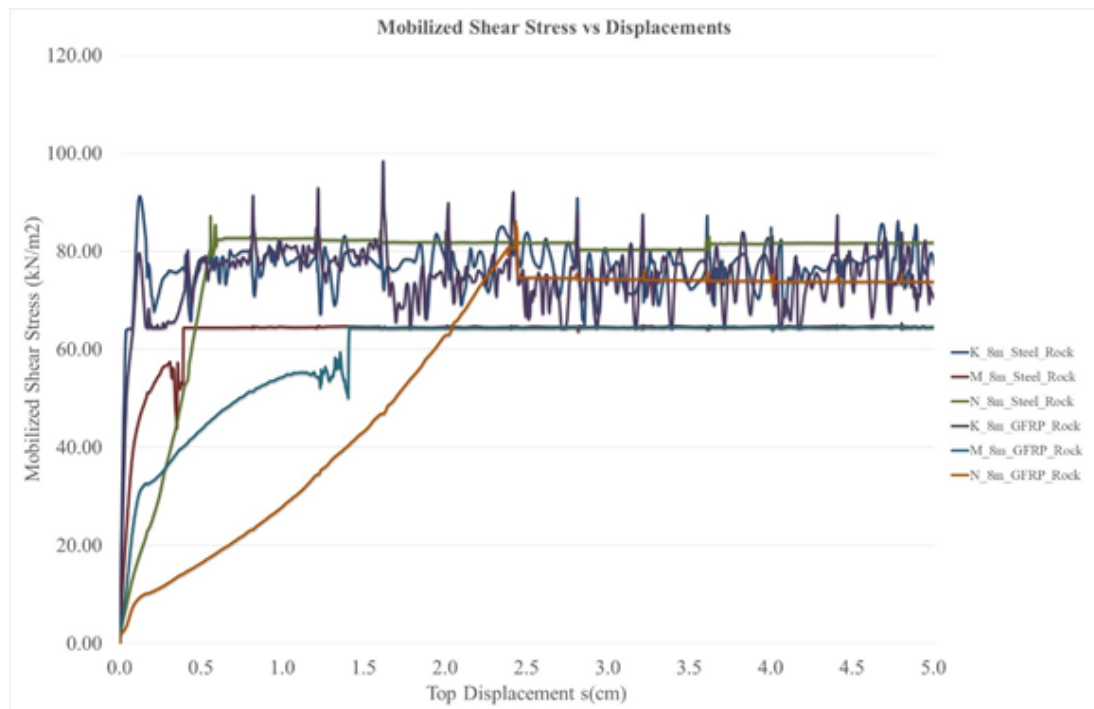


Figure 6.127. Deformation plot for 0.4 cm pull out.

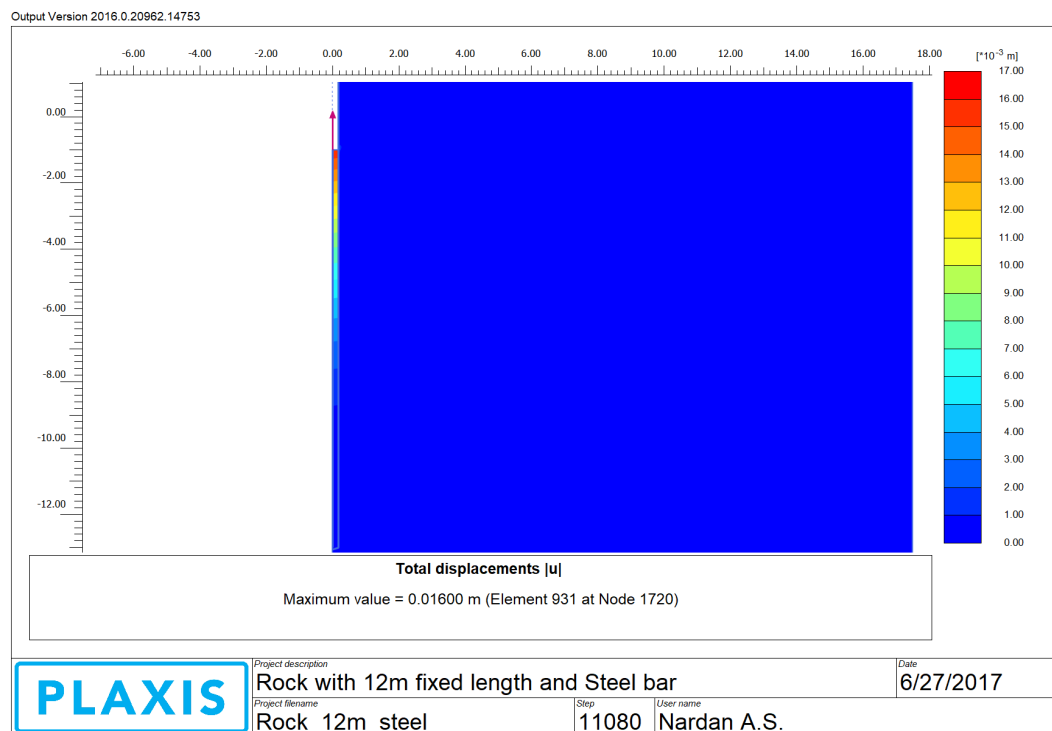


Figure 6.128. Deformation plot for 1.6cm pull out.

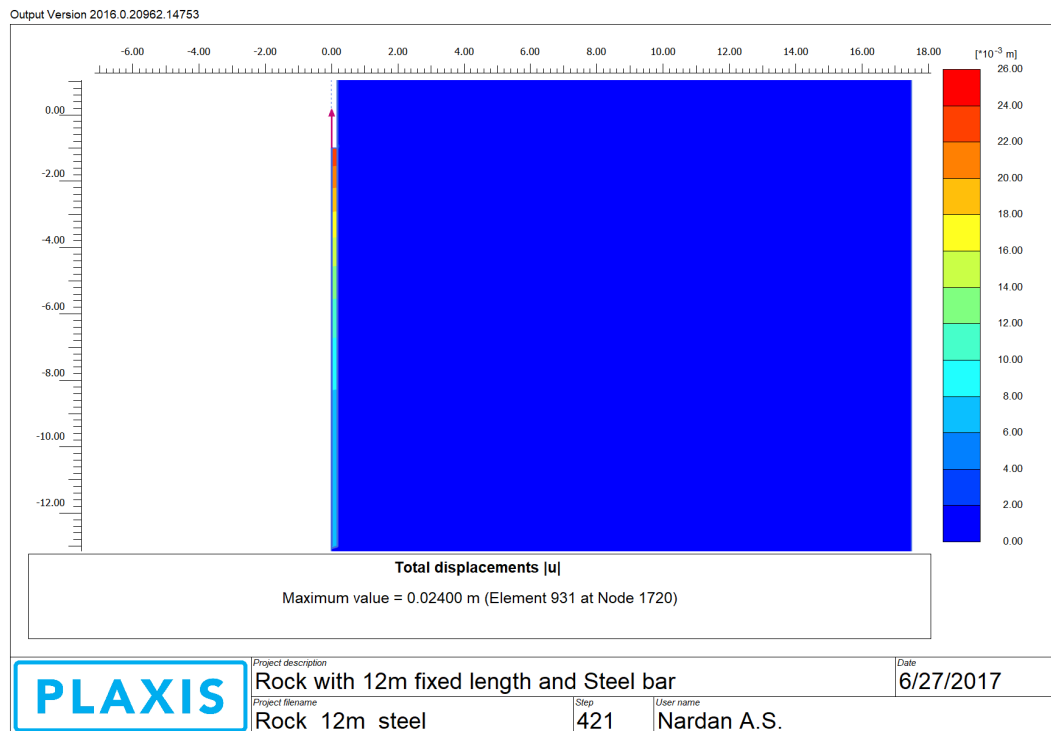


Figure 6.129. Deformation plot for 2.4 cm pull out.

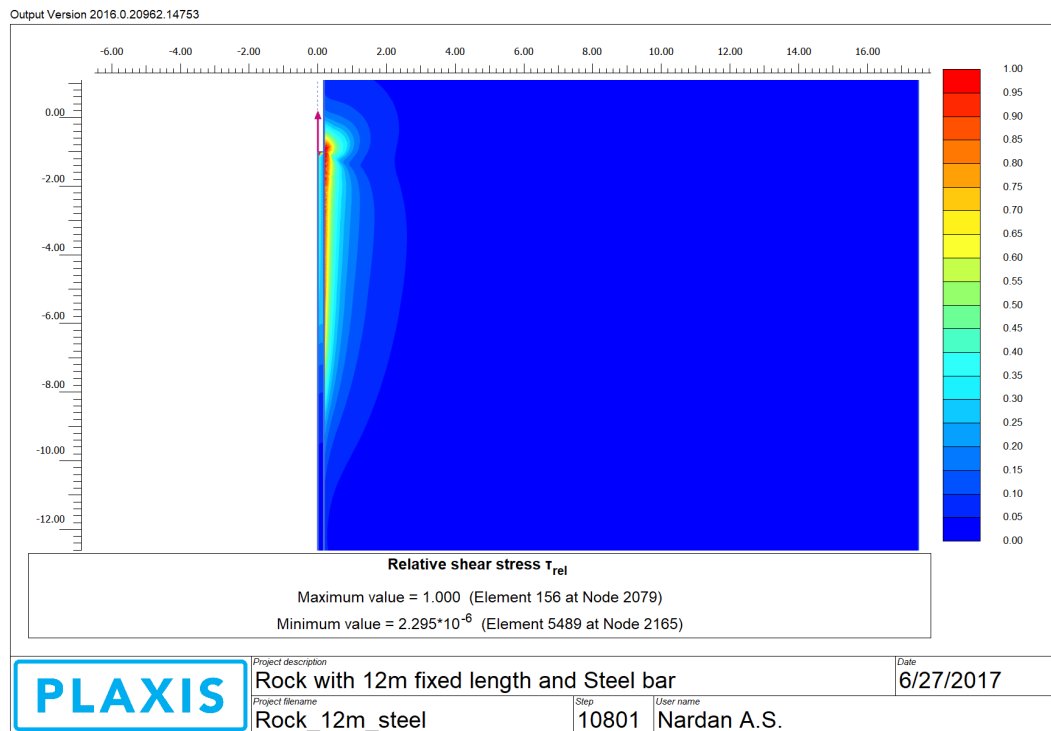


Figure 6.130. Relative Shear plot for 0.4 cm pull out.

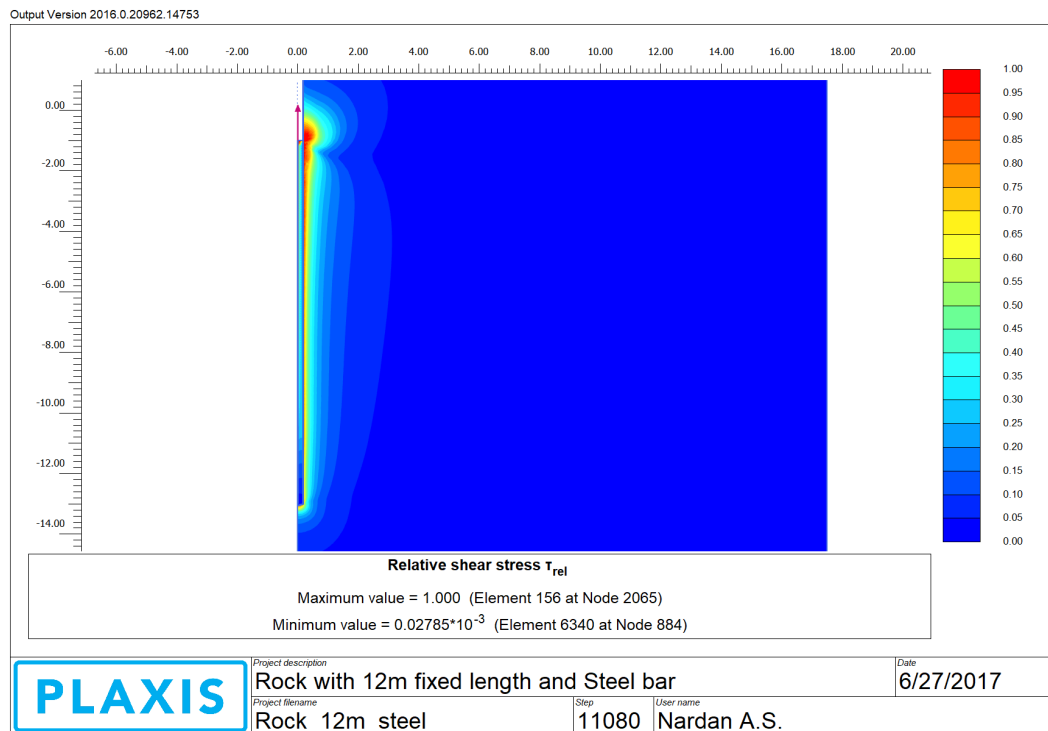


Figure 6.131. Relative Shear plot for 0.8cm pull out.

The pull-out resistance is around 1.67 times larger than ground anchor with 8m fixed length and 2.5 times larger than the ground anchor with 6m fixed length. Graphs for the load-displacement and mobilized shear to “deflection on the top of the fixed length” may be seen in Figure 6.132 and Figure 6.133. The mobilized shear stress on the points K and M has reached the peak shear stress almost at the same deformation level (0.18cm and 0.69cm). However, point N has reach the peak after 1.8cm of deformation and its peak value is more than 1.5 times more comparing with the other two points. This circumstance is more visible in Figure 6.134; It can be easily observed from Figure 6.134, the shear stress fully mobilized after 1.8cm of deformation at the top of the ground anchor. This is a distinction in the behaviour of the pull-out resistance of ground anchor by comparing with 8m and 6m fixed length ground anchors.

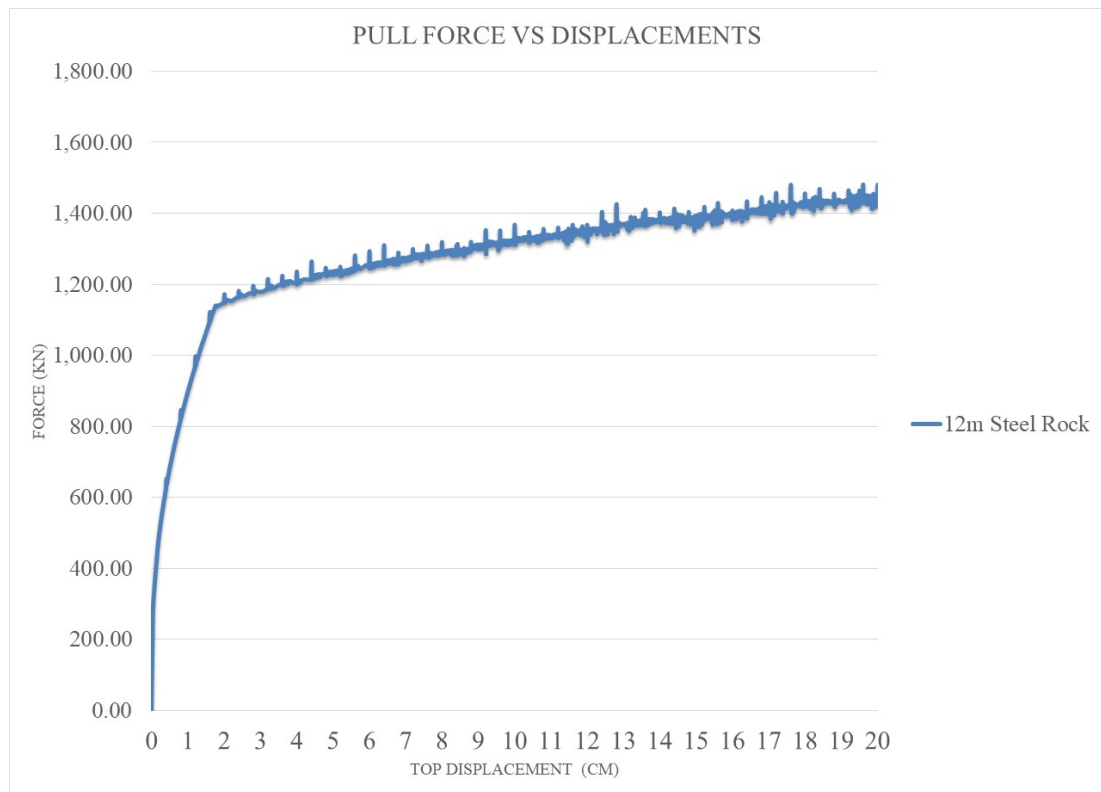


Figure 6.132. Load vs Displacement plot.

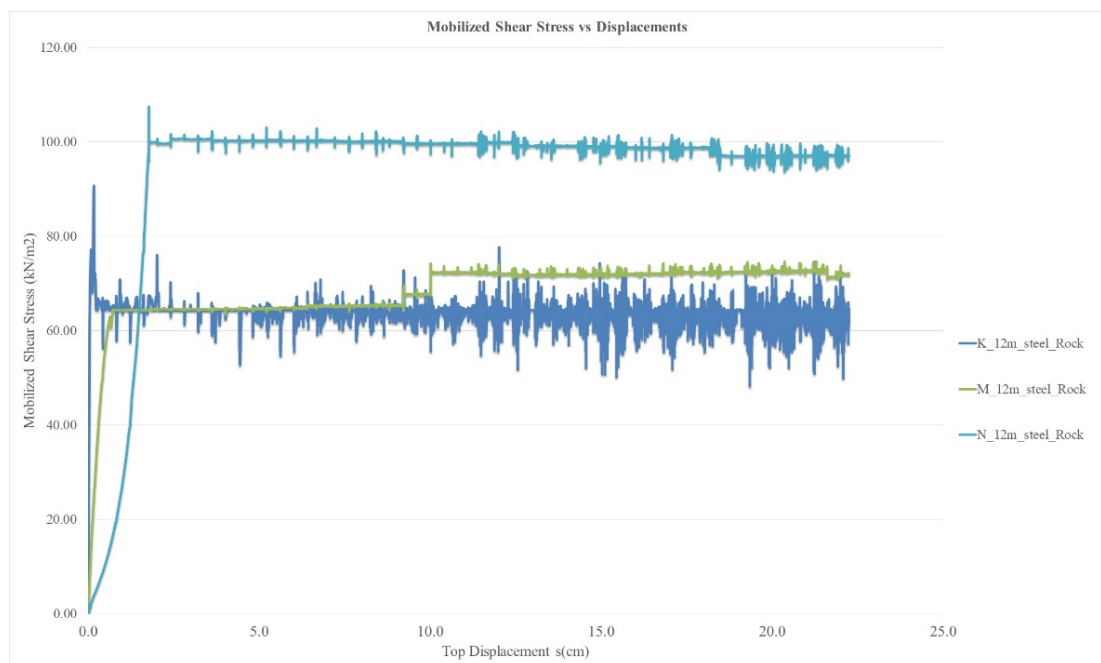


Figure 6.133. Mobilized Shear Stress vs Displacement plot.

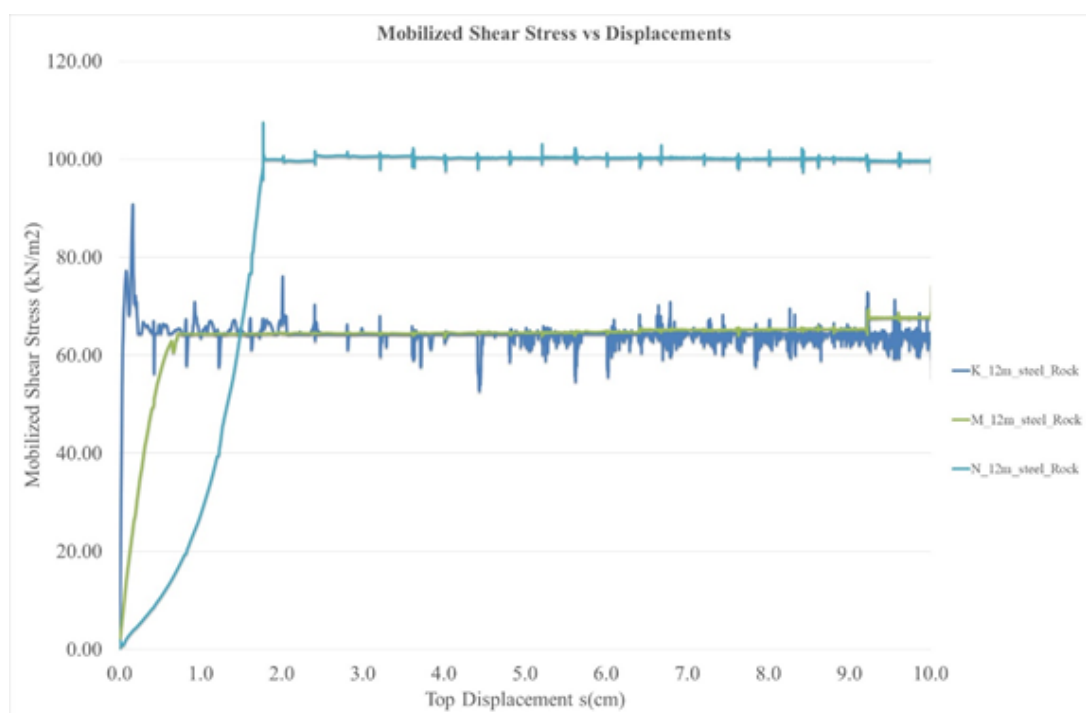


Figure 6.134. Mobilized shear stress for different load levels along A-A' cross-Section.

6.3.4.2. Ground Anchor with GFRP Bar. Pull-out test of a ground anchor with GFRP bar is simulated for the same conditions with the previous model. In this analysis, the transition from elastic phase to plastic phase occurred after 0.4cm pull out (at 7.9cm). Therefore the deformation and relative shear plots for 0.4cm and 8.0cm pull out will be presented in Figure 6.135-Figure 6.139. As anticipated, in this model, transfer of the deformations from proximal to the distal part of fixed length is very slow. The deformations which is developed from the uplift in top of the fixed length isn't transferred to the end in no portion until 11.2 cm pull out in top of the fixed length. This is somehow extreme comparing all the previous models. This can be easily seen from the Figure 6.136 and Figure 6.137.

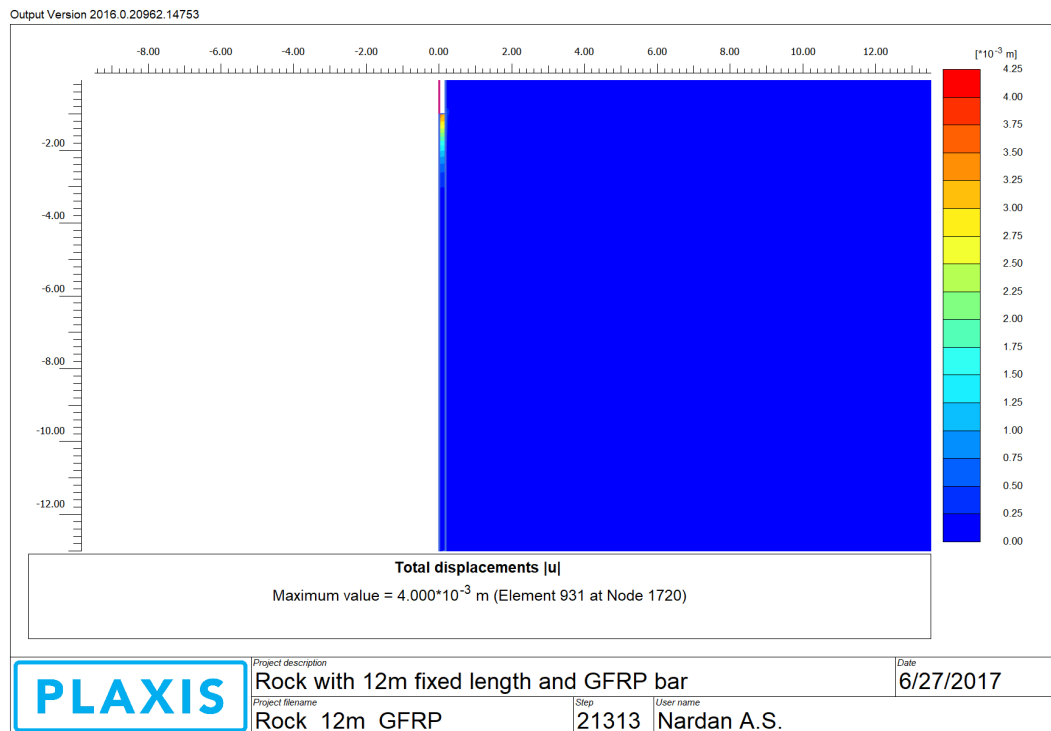


Figure 6.135. Deformation plot for 0.4 cm pull out.

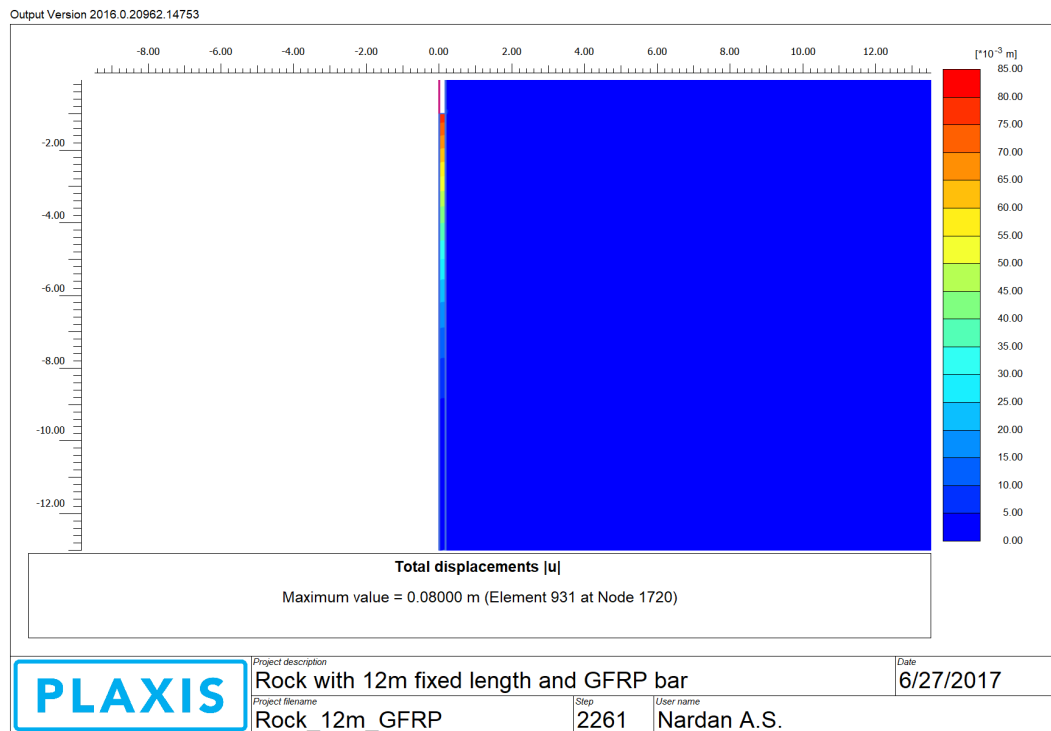


Figure 6.136. Deformation plot for 80mm pull out.

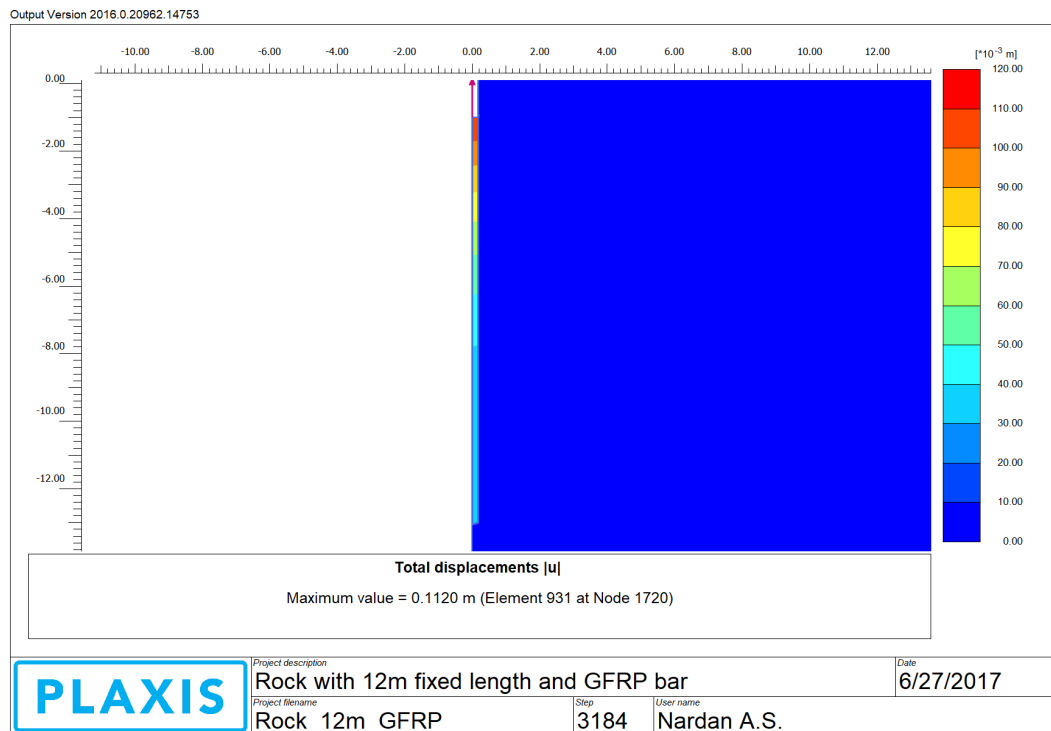


Figure 6.137. Deformation plot for 112mm pull out.

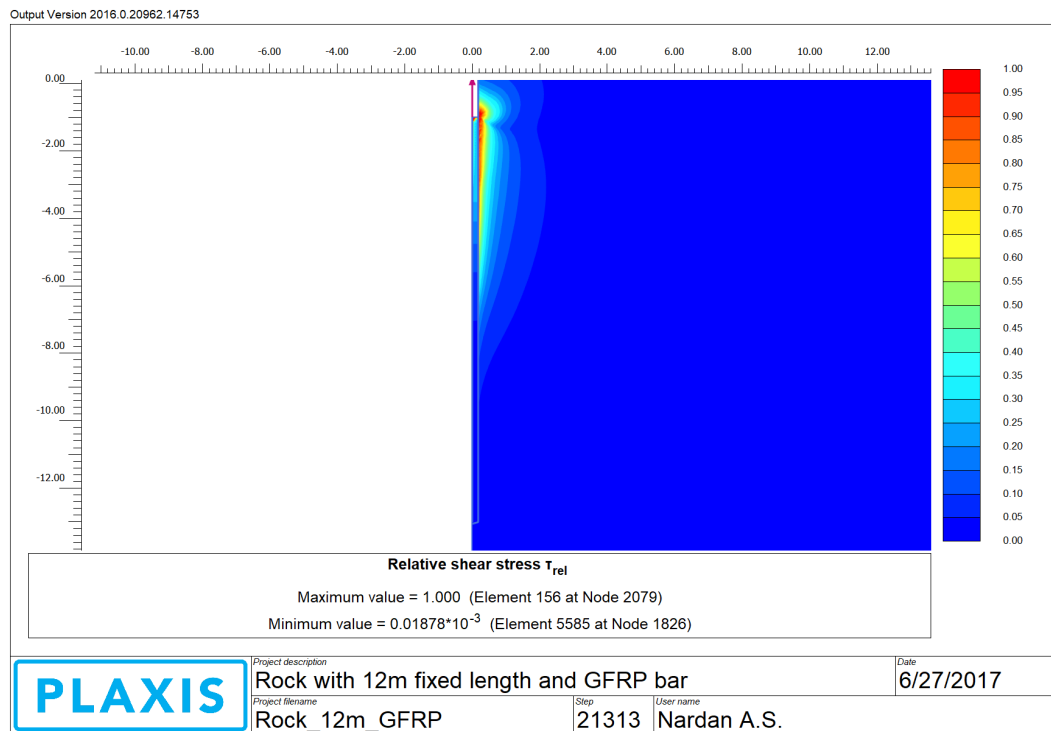


Figure 6.138. Relative Shear Stress plot for 0.4 cm pull out.

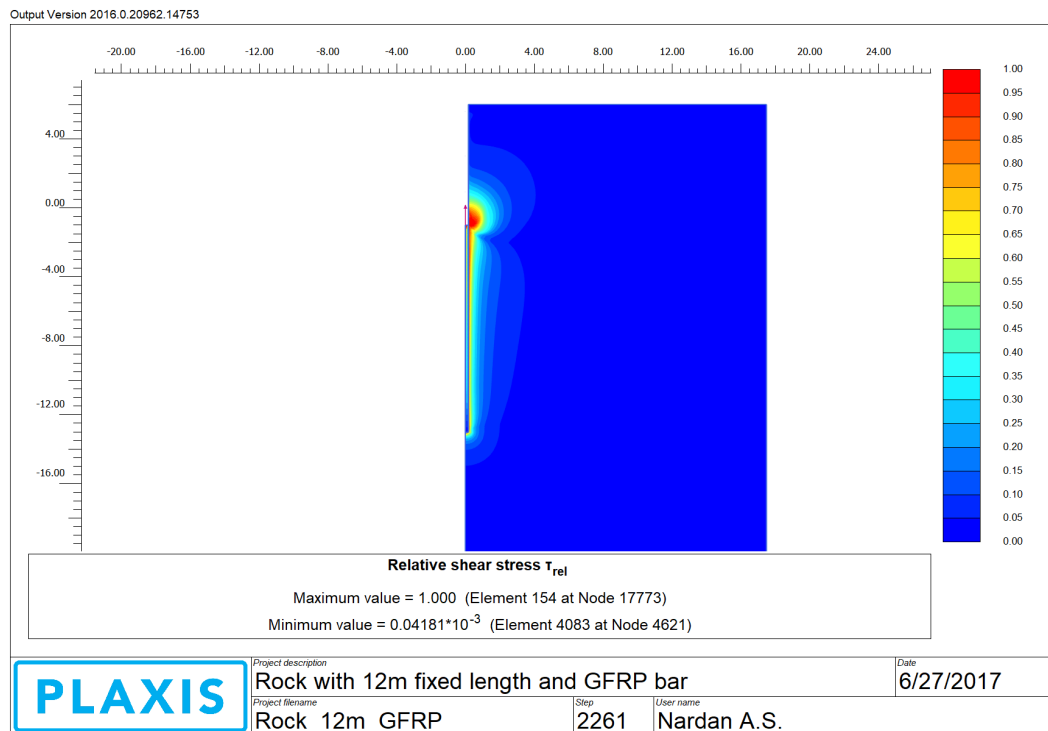


Figure 6.139. Relative Shear plot for 8.0cm pull out.

It is very clear from Figure 6.135-Figure 6.138, the deformations on the top is far more slowly being transferred to the end of the fixed length comparing with the all of other models. Although the maximum load is reached approximately at 8.0cm, the deformations at the top haven't transferred yet to the medium part even due to the lower young's modulus of GFRP bar. This situation can be comprehended by following the dark blue to light blue color in the deformation plot.

The pull-out resistance (reached after a 8.0cm more deformation than the steel one) is 100 kPa higher than the one with steel bar. Graphs for the load vs displacement and mobilized shear vs "deflection on the top of the fixed length" may be seen in Figure 6.140 and Figure 6.141. The mobilized shear stress on the point which is at the top of the fixed length K, has reached the peak shear stress after 0.6cm of deformation. The medium point M, mobilized full shear stress at 2.78cm of deformation and the distal point N, mobilized the full shear stress after 8.0cm of deformation there is also a 10% increase in pull force (without any increase in mobilized shear stress). This will be addressed again in comparison sub-Chapter.

As it can be seen from the Figure 6.142, the shear stress fully mobilized after 8.0cm of deformation at the top of the ground anchor and it is more clearly progressive than the other ones (6m and 8m fixed length). The shear stress didn't change remarkably along the shear zone at fixed length after 8.0cm of deformation and the shear stress is at the same value in all along the fixed length zone up to 10m.

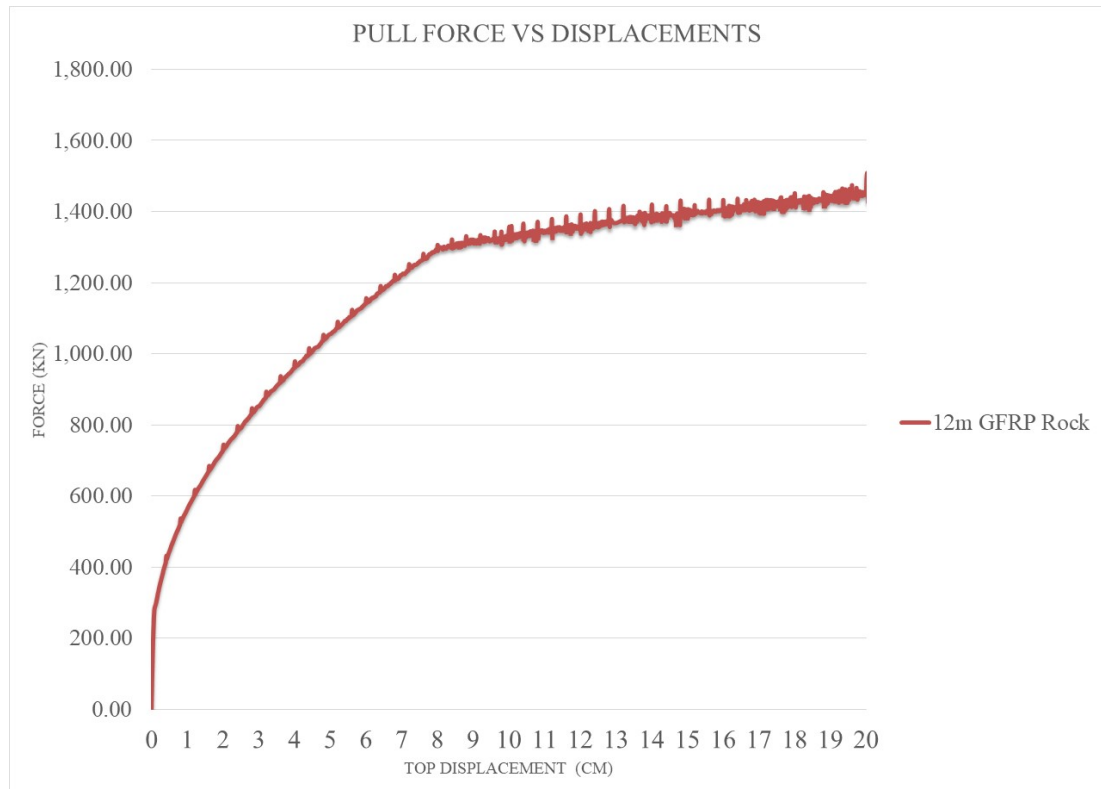


Figure 6.140. Load vs Displacement plot.

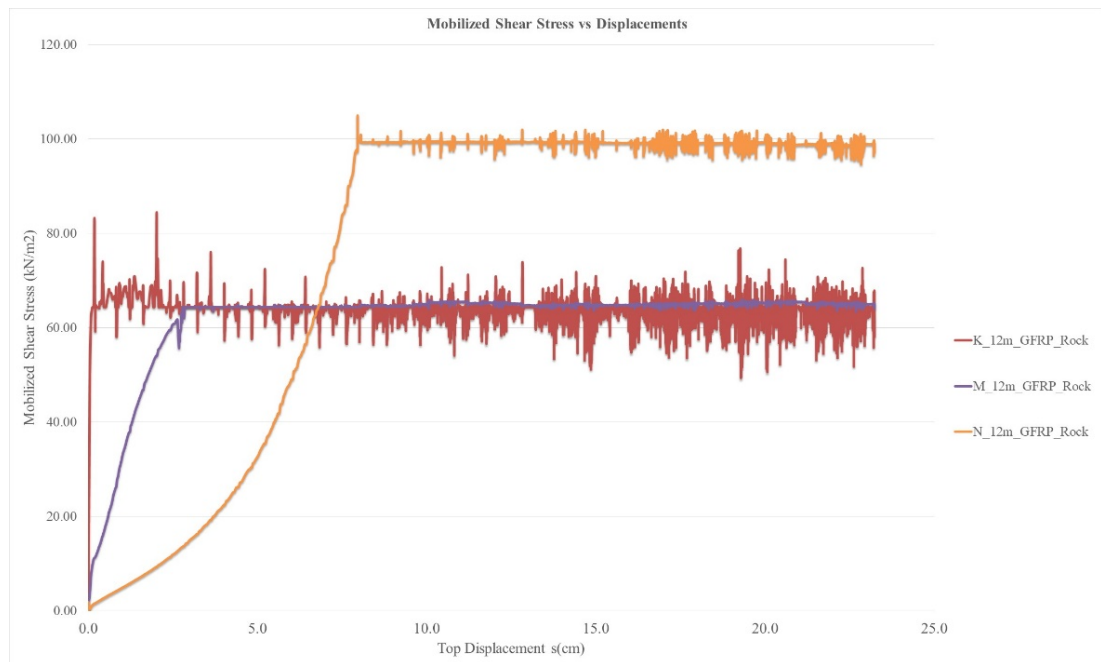


Figure 6.141. Mobilized Shear Stress vs Displacement plot for points K, M and N in the FE mesh.

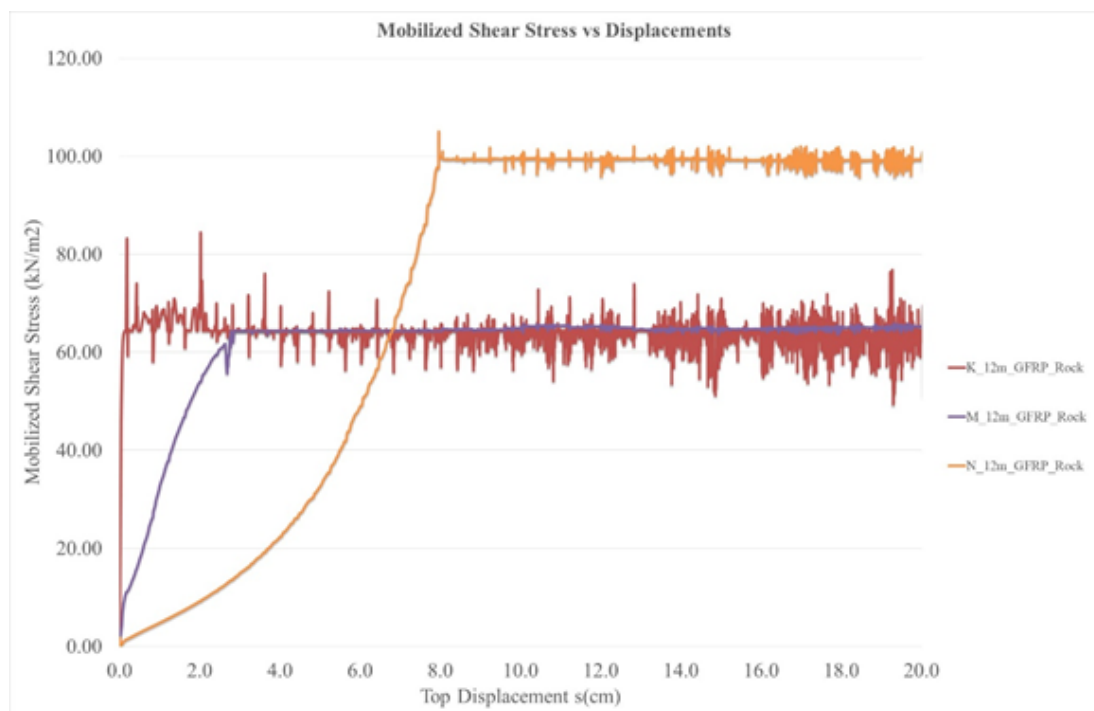


Figure 6.142. Mobilized shear stress for different load levels along A-A' cross-Section.

6.3.4.3. Comparison of the Results. As it can be seen from both three graphs, more deformations are needed for 12m fixed length ground anchor with GFRP bar to mo-

mobilize the full pull-out resistance force comparing with the steel. Even though the mobilization of the shear force at the top (point K) of the fixed length is the identical in two distinct cases, the rate of deformation which is needed for the mobilization of the full shear stress at the shear zone in medium and in the distal end (of the fixed length), five times more for GFRP when comparing with steel (the mobilized shear stress at 1.6cm for steel is in the same order of magnitude with the mobilized shear stress at 8.0cm for GFRP). This proportion would be extreme if there have been a need for ultimate load capacity for design purposes. However, factor of safety for tendon failure is commonly taken as 2 and specifications foresee a factor of safety of 3 for bond failure for permanent purposes. This leads us to a need of 1/3 of the ultimate load. Therefore, smaller deformations will be needed to mobilize the allowable design load. By using this assumption for design purposes, the deformation which is needed to mobilize allowable load is around 0.43 cm and is not significant at all.

In both three cases, like in the 12m, 8m and 6m fixed length cases, the mobilized shear stress in the grout/ground zone stays in the same value after a specific deformation value but the load in pull force vs deformation graph increases slightly. It is not a distinctive feature for the present research since as asserted the behaviour is identical in both of these models. However this will be addressed again for this model regardless. The B-B' cross-Section may be seen in Figure 6.143;

As it can be seen from the Figure 6.143, after full mobilization of the shear stress at grout-ground interface, the shear zone thickness around the anchor increases progressively. The rise with low inclination in the pull-out force/deformation graph is the result of this behaviour. This behaviour on the other hand is very similar in both cases, so it is not distinctive. The comparison of conventional steel versus GFRP at pull force-displacement may be seen in Figure 6.144 and Figure 6.145.

As can be anticipated from Figure 6.144 and Figure 6.145, the difference between the GFRP and conventional Steel bar in ground anchorage is significant at distal part (point N) however, by using allowable load levels the difference between Steel and GFRP in the rate of deformation needed to mobilize full shear stress, is decreased to

an insignificant gap.

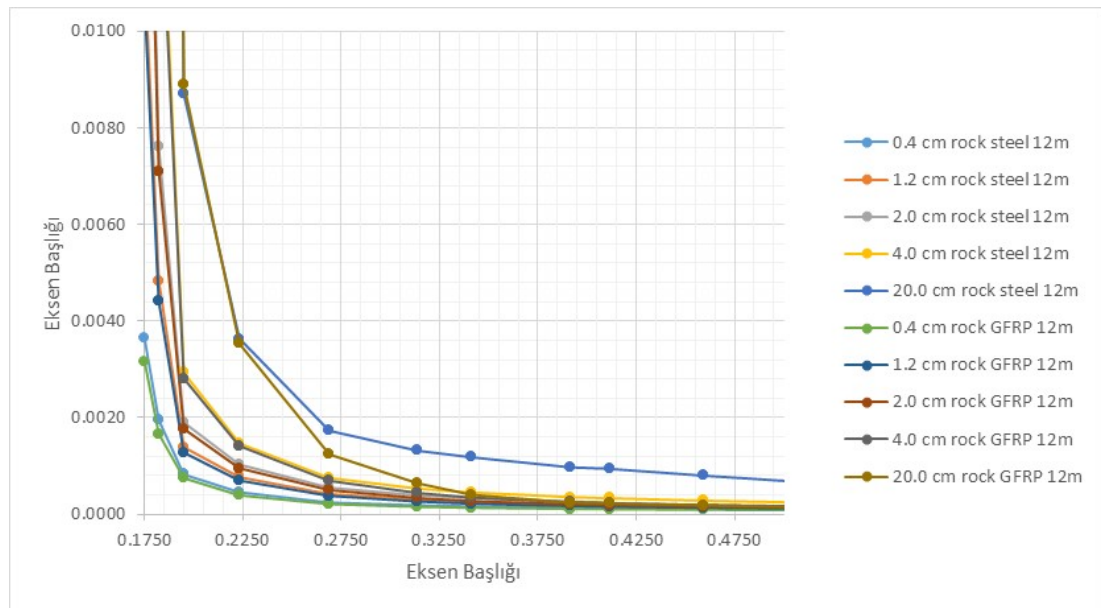


Figure 6.143. Mobilized ground deformation for different deformations on top along B-B' cross-Section.

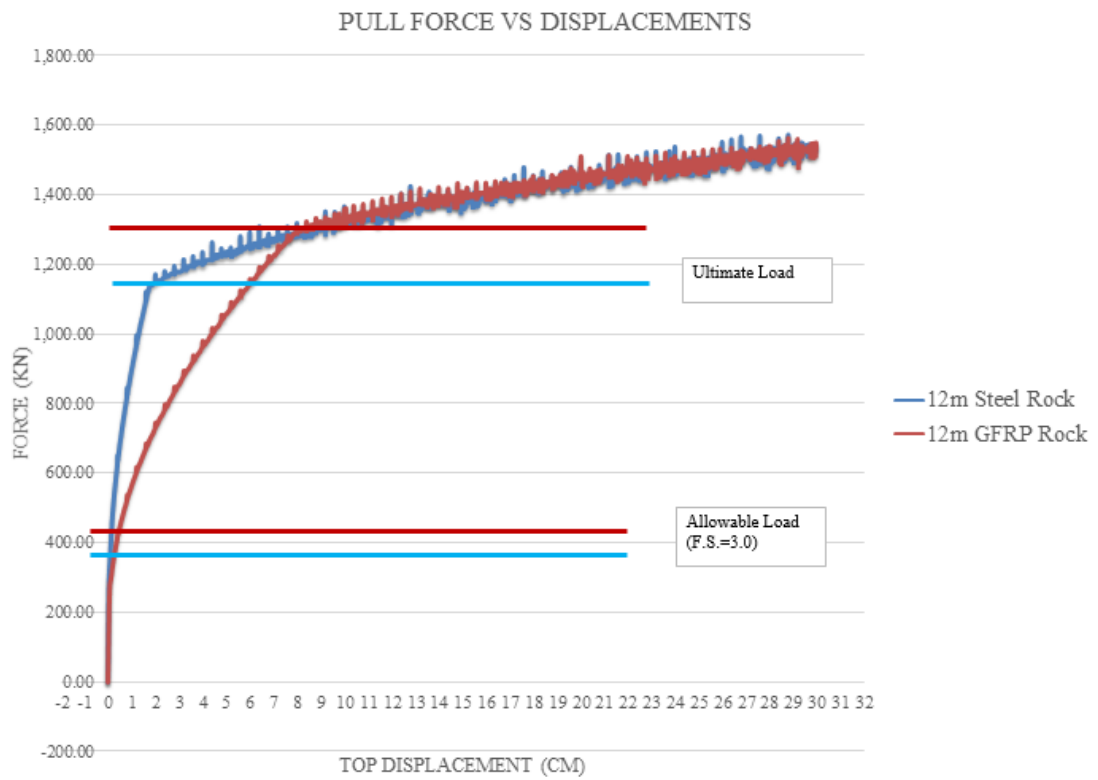


Figure 6.144. Load vs Displacement plot for the comparison of GFRP and Steel Bar.

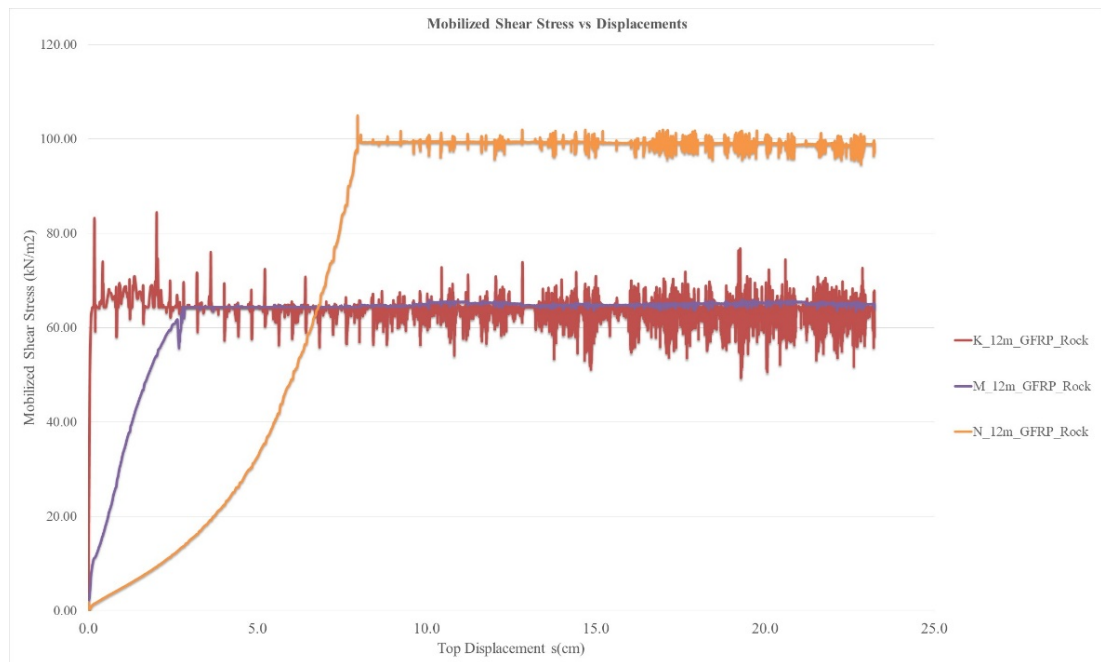


Figure 6.145. Mobilized Shear Stress vs Displacement plot for the comparison of GFRP and Steel Bar for points K, M and N in the FE mesh.

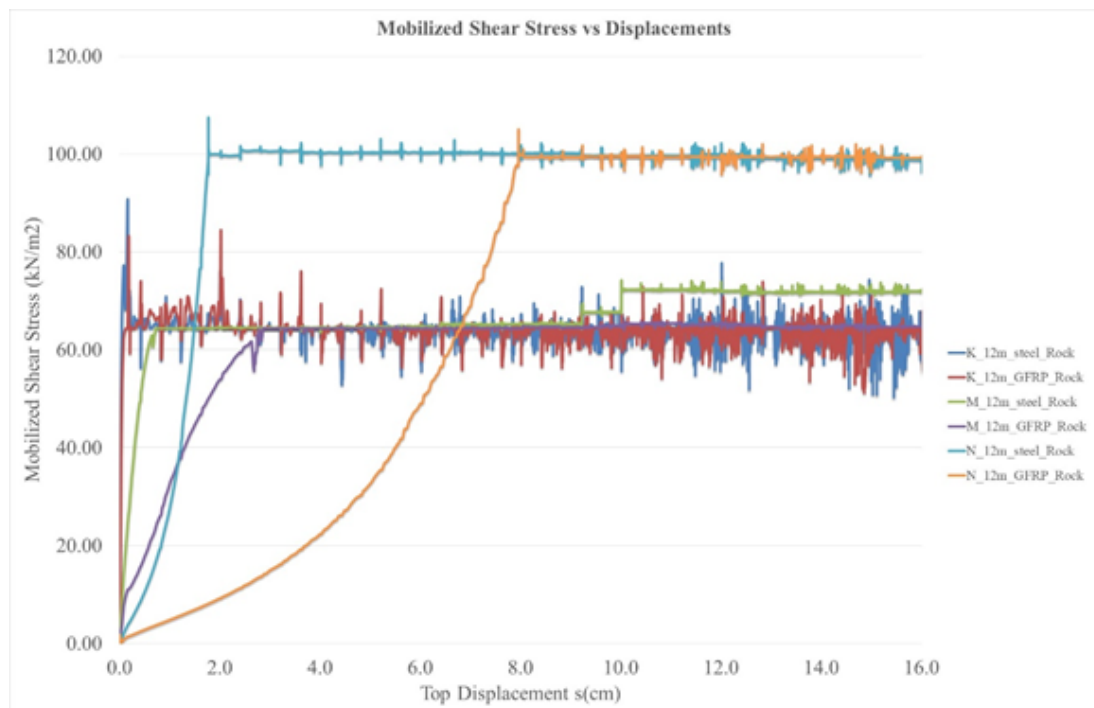


Figure 6.146. Mobilized shear stress for different load levels along A-A' cross-Section for the comparison of GFRP and Steel Bar.

6.4. Summary of the Comparisons of the Simulated Models

In this Chapter, highlights on the results of the models with GFRP and Steel tendon will be summarized. Pull out of a ground anchor with three distinctive fixed lengths (6m, 8m and 12m) are conducted. In the FE models, these ground anchors are embedded in three different geo-materials (Over consolidates clay, dense sand and rock). Tendon material (GFRP and Steel) is investigated for each soil type and fixed length. In total, 18 different pull-out models are simulated. The overburden stress is identical in Dense Sand and Over-consolidated Clay. However, it is almost 30% more for Rock material because of the difference in Rock's actual unit weight comparing to over consolidated Clay and Dense Sand (5.2.1.2). In general concern, there are two distinct characteristics when comparing those models;

Practically, there is no apparent difference in the behaviour of 6 m and 8 m fixed length ground anchors with any tendon type in any geo-material. Therefore, 12m fixed length ground anchors will be used for comparison purposes. The difference in the behaviour of pull-out force to deformation (in top) and mobilized shear stress at medium and end point may be seen in Figure 6.147 and Figure 6.148 for all 12m fixed length ground anchors;

As it can be seen from the Figure 6.148, the inclination angle of force vs displacement graph is getting steeper as the stiffness of the geo material increases. Although the mobilized ultimate force is increasing too with this condition and the deformations which is needed to reach that force is also increasing, the gap of deformation between the GFRP and Steel tendon grows significantly (five times compared to steel). However, as explained in previous Chapters, the design of anchors always performed by using allowable loads and the significant condition is permanent conditions. Therefore, the safety factor for these conditions is 3 for bond failure. Therefore by using this safety factor, the gap between Steel and GFRP turns into insignificant (it is 0.05 cm for 8m fixed length with GFRP tendon in rock and 0.48 cm for 12m fixed length with GFRP tendon in rock).

The behaviour of point K at the very beginning of the anchor is more or less similar in OC clay and dense sand when comparing the behaviour of GFRP vs Steel. However, the difference between GFRP and steel becomes more distinct towards the end of the fixed length. Therefore it is important to analyze the points M and N. When analyzing Figure 6.147 and Figure 6.148, it can be anticipated, that there are no notable differences. The deformation which is needed to mobilize the peak shear stress is in very low value. On the other hand, for point N, deformations needed to mobilize maximum shear stress for anchorages with GFRP tendon is five times higher than the steel one and the exact value is 8.00 cm. However, by using allowable design capacity the deformations which is needed to mobilize pull out force decreases dramatically as it explained in Chapters 6.3.2 and 6.3.3.

The increase in the pull-out force in the plastic phase is also discussed in each comparison sub-Chapter as it is a considerable behaviour. Graph of the Deformations at the geo-medium in which grouted ground anchor is embedded may be seen in Figure 6.150. The graph presents the vertical displacements around the head of the fixed length at 4cm and 20cm top deformation levels for 12m fixed lengths. The shear zone around the anchor increases progressively for dense sand and rock but not for over-consolidated clay. Therefore, the difference between the ultimate pull-out force value in anchors with GFRP and Steel tendons (the anchorage with GFRP tendon had 10% higher ultimate pull-out force value than the one with the steel tendon) is not significant.

In all these cases analyzed, it is anticipated that the tendon (Steel or GFRP) will not fail. Similarly, it is assumed that the grout surrounding the tendon will not fail either. So, the only failure investigated is the shear failure between grout and ground (which is the dominant failure type) and the effect of the tendon material on this behaviour.

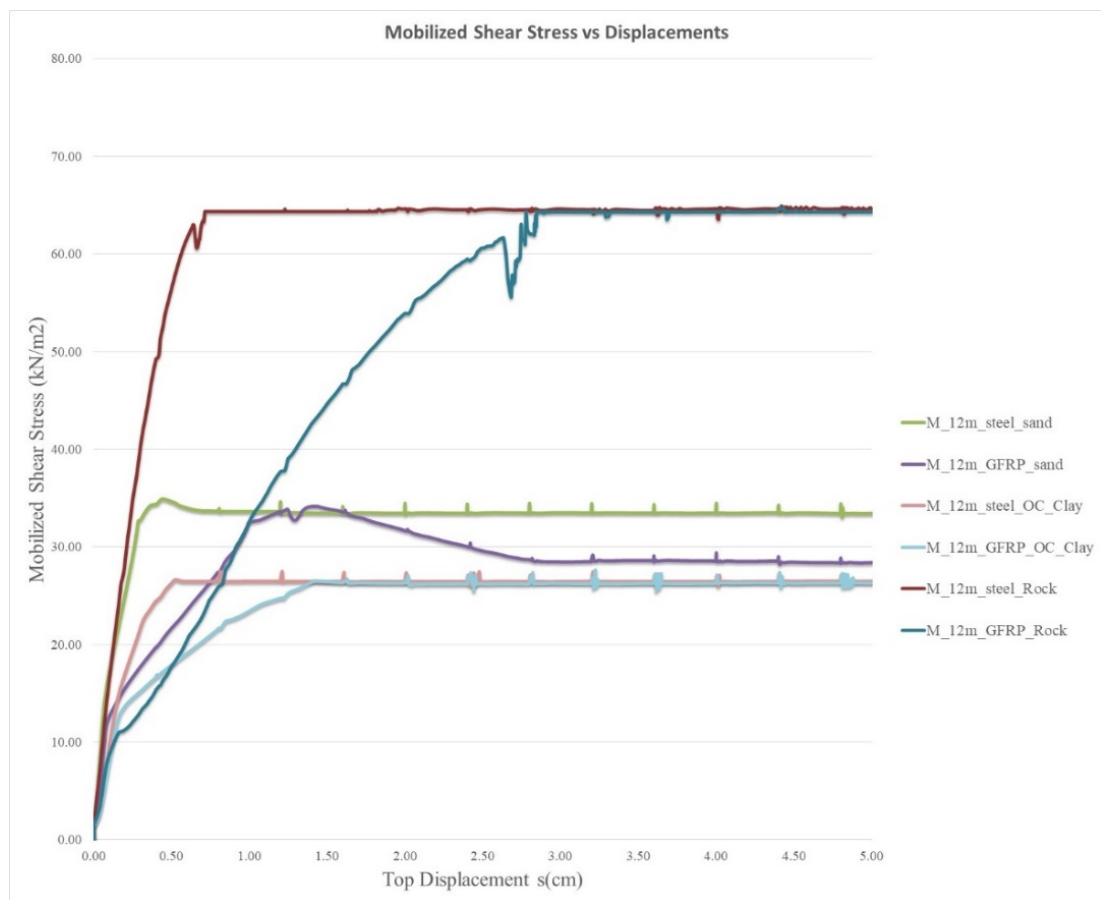


Figure 6.147. Comparison between medium points (M) of ground anchors with 12m fixed length.

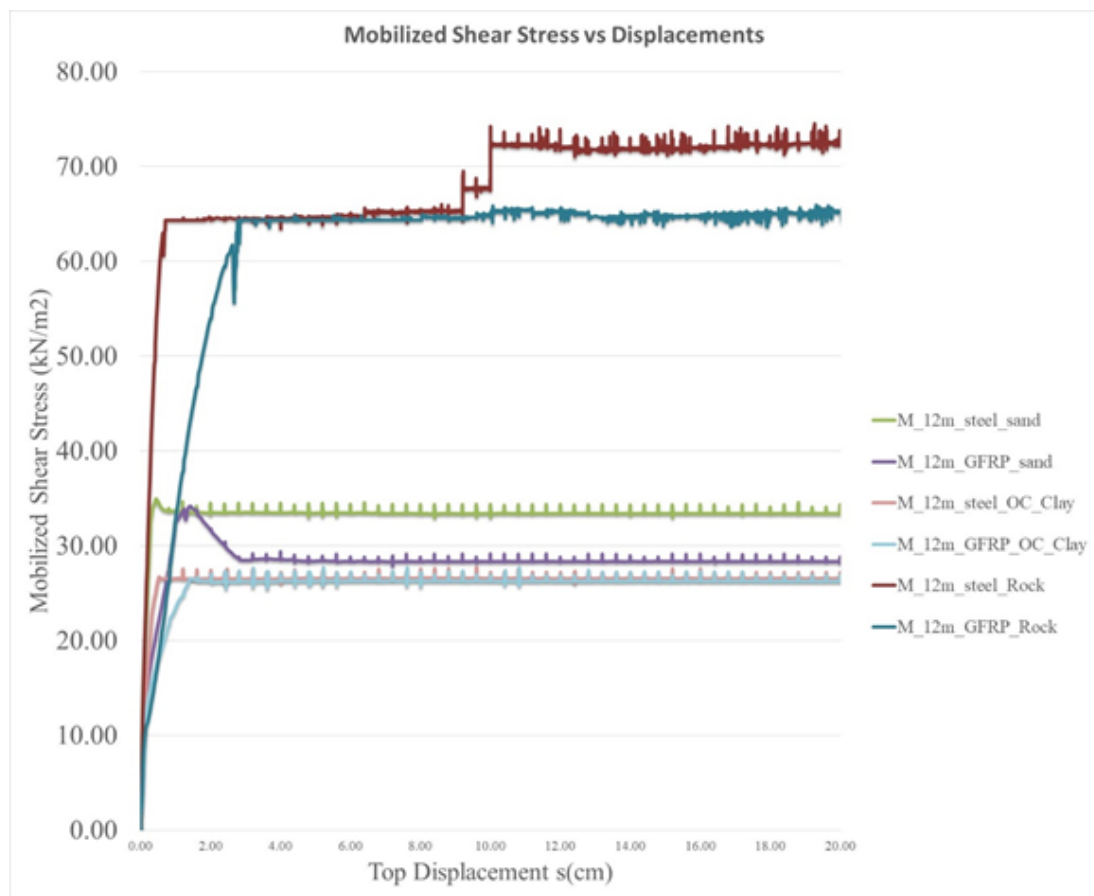


Figure 6.148. Comparison between medium points (M) of ground anchors with 12m fixed length.

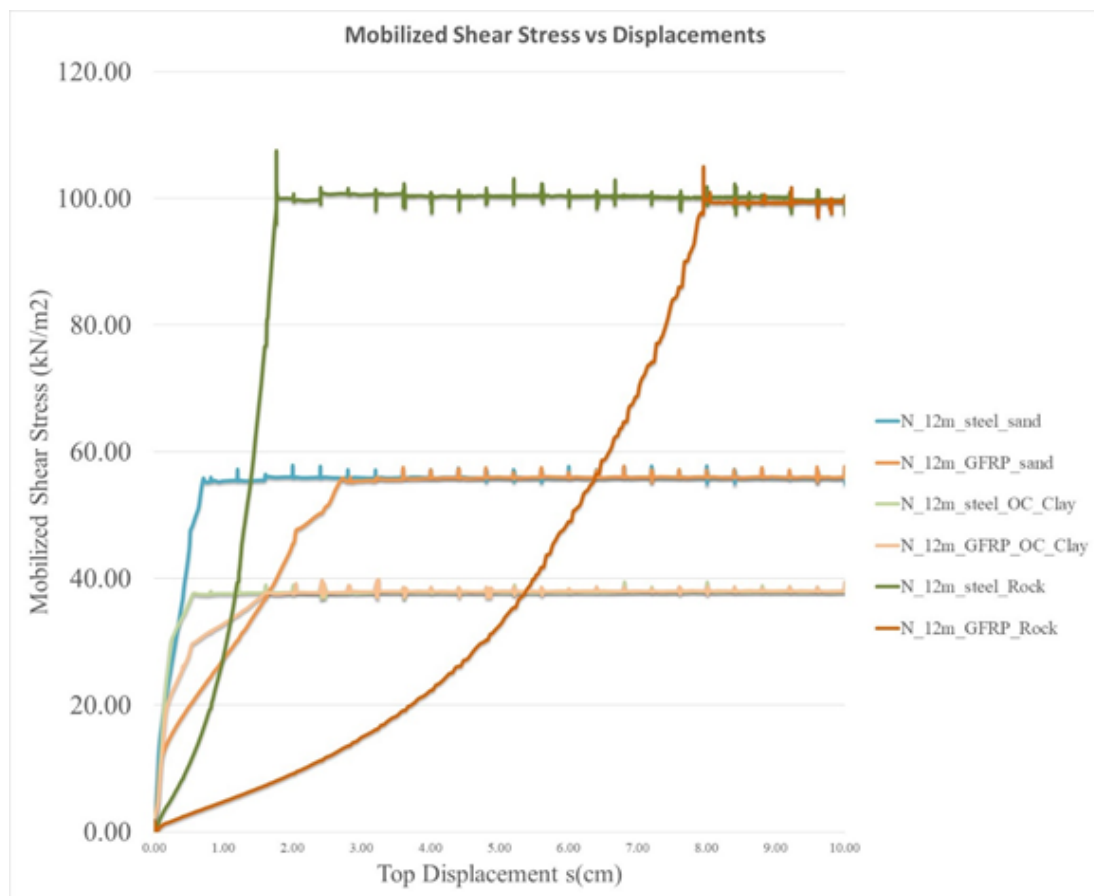


Figure 6.149. Comparison of pull-out force/top disp. behaviour of ground anchors with 12m fixed length.

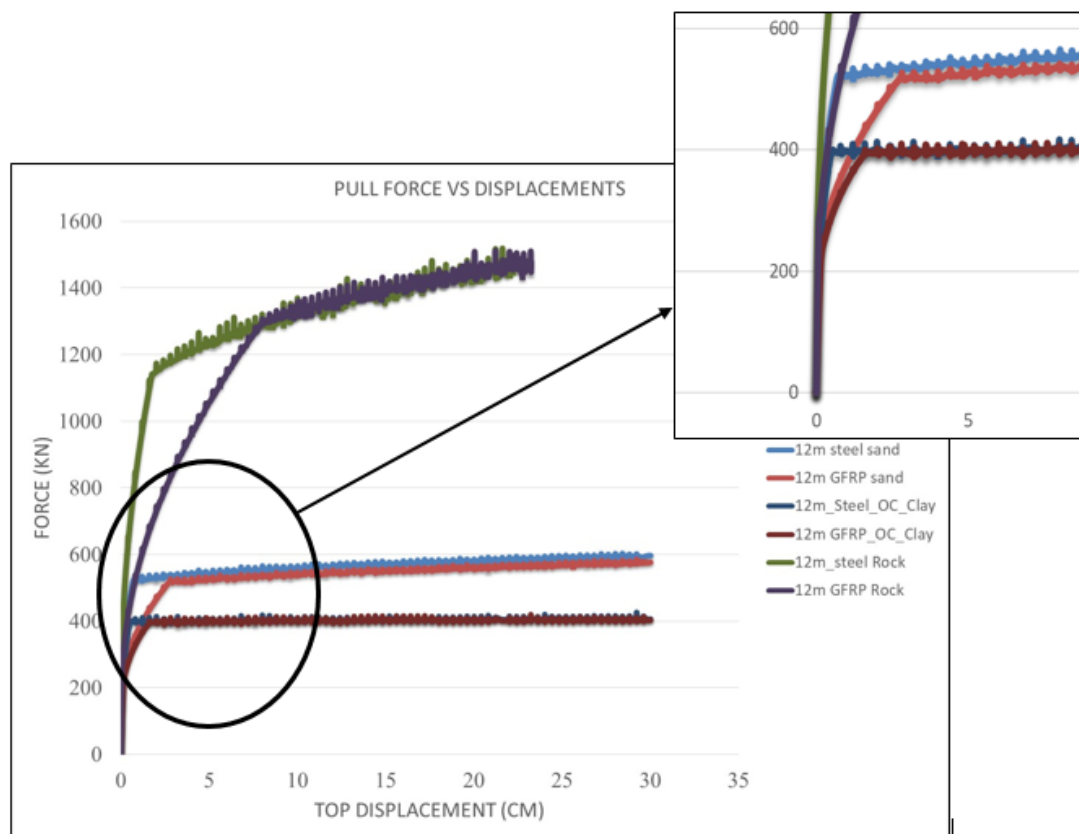


Figure 6.150. Mobilized ground deformation for different deformations on top along B-B' cross-Section for ground anchors with 12m fixed length.

7. CONCLUSION

In this thesis, first a literature review is given on the history of FRPs (fibre reinforced polymers). Types of FRPs and FRP bars also their benefits-drawbacks and standards, codes are summarized. Later, ground anchoring technique is presented with detailed data about its history and types. The detailed information about its fields of application and assembly is introduced. Following that the approaches for estimating the pull-out force (bearing capacity) with design method, codes and standards are presented by reviewing briefly current and universally accepted factor of safety values. Later, the numerical modelling and finite element method are briefly reviewed and calculation model geometries, simplifications and analysis procedures are described.

In the present thesis, pull out of a ground anchor with three distinctive fixed lengths (6m, 8m and 12m) are analyzed. In the FE models, ground anchors are embedded in three different geo-materials (over consolidates clay, dense sand and rock). As tendon material GFRP and Steel was used separately. All combinations of soil, tendon type and length have been analyzed. In total, 18 different pull-out models are simulated. The overburden stress is identical in Dense Sand and Over-consolidated Clay. However, it is almost 30% more for Rock material because of the difference in rock's actual unit weight compared with over-consolidated clay and dense sand. The fundamental aim of this study is to investigate the feasibility of using the newly emerging GFRP material for reinforcing in situ soil (particularly in grouted ground anchorages) As explained in Chapter three, permanent condition is the significant and most problematic condition for grouted ground anchors. Therefore, the simulated models are conducted to investigate the use of GFRP tendons in grouted ground anchorages for permanent conditions in highly harmful environments due to their corrosion resistance.

By using numerical simulations the effect of following aspects on the behaviour of grout-ground shear zone in ground anchors which is reinforced with GFRP and Steel bar is investigated;

- (i) The Length of the grouted zone of ground anchors (Fixed length),
- (ii) The Stiffness of surrounding soil-mediums,

In all of these 18 FE-models, the tensile capacity of grout and tendon apart from type haven't been investigated.

Based on the results of numerical simulation models, the following observations are made for the use of GFRP bar for reinforcing in situ soil;

The grouted ground anchor with GFRP bar has exhibited an excessive displacements to mobilize the ultimate pull-out resistance. However, the pull-out displacements are not so significant for shorter fixed lengths (6m and 8m). The displacements of ground anchors with GFRP bar, is low for practical purposes for 6 m and 8 m fixed lengths. On the other hand, for the ground anchors with 12m fixed length, the pull-out behaviour is sensitive to the tendon material and surrounding soil mediums. Even so, for less stiff soil mediums, developed pull-out deformations are lower since the mobilized pull-out capacity is lower. For stiff soil mediums (Rock) the deformations of GFRP bar is very high. However, considering a factor of safety of 3 for bond failure in permanent purposes, the mobilized deformations remains in order of magnitudes which are feasible for practical purposes. Although the grout-ground skin friction capacity is highly site-specific, increase in stiffness of the soil or overburden stress increases the skin friction capacity. The results of the FE-models revealed that, the gradient in pull-out force to deformation curve got steeper as the soil medium stiffness increases. This behaviour was observed for Dense Sand and Rock. It is apparent when observing the FE-Models of ground anchors with GFRP tendons in Dense Sand and Rock. By using allowable load capacity (Factor of safety; 3 in permanent purposes), the rate of deformations to mobilize design loads gets even lower. Even though, the tensile capacity of GFRP nail is superior when comparing with the conventional steel (Gr 50), the effect of creep and rupture in GFRP material needs to be investigated none the less.

By taking account of the results of this study and the literature findings, it can be concluded that in aggressive environmental conditions, ground anchors with

GFRP tendon material gets more feasible as the stiffness of surrounding soil-mediums increases and as the fixed length of the ground anchor decreases. GFRP tendon mobilizes excessive deformations comparing with the steel due to its low Young's modulus for longer fixed lengths but this situation becomes insignificant when using allowable load capacities (displacement control) with universally accepted factor of safety values for practical applications.

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