DEPARTMENT OF CIVIL ENGINEERING

CE410-CIVIL ENGINEERING DESIGN

PRELIMINARY DESING REPORT

Project: Design of a Pedestrian Bridge Over Kızılırmak (STR-1)

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1. INTRODUCTION

1.1. Description of the Project

This pedestrian bridge project is conducted specifically for the Shaking Bridge which is located in Avanos, Nevsehir by Versatile Engineering. Avanos is located in the Central Anatolia region of Turkey and situated in the historic Cappadocia region. Kizilirmak river, which is the longest river in the Turkey, separates Avanos from Cappadocia. As a result of being such a tourist destination any problematic situation caused by this bridge will cause a great deal of economic issue in this region, since the major livelihood of this region depends on the touristic attributions.

Shaking Bridge is a pedestrian bridge, which was built around 1974 and since than it has been unsteady. It provides a connection between two sides of the Kizilirmak. As it can be seen in Figure 1. It is obvious that while being constructed, no one expected it to last that longer and be a trademark of Avanos. The unstable situation of the bridge makes it a tourist destination and Avanos is benefitting from this current situation. However, it has been realized that as much as it benefits the district, the unstable behavior of the bridge under pedestrian walking makes it a time bomb, not knowing when will it explode.

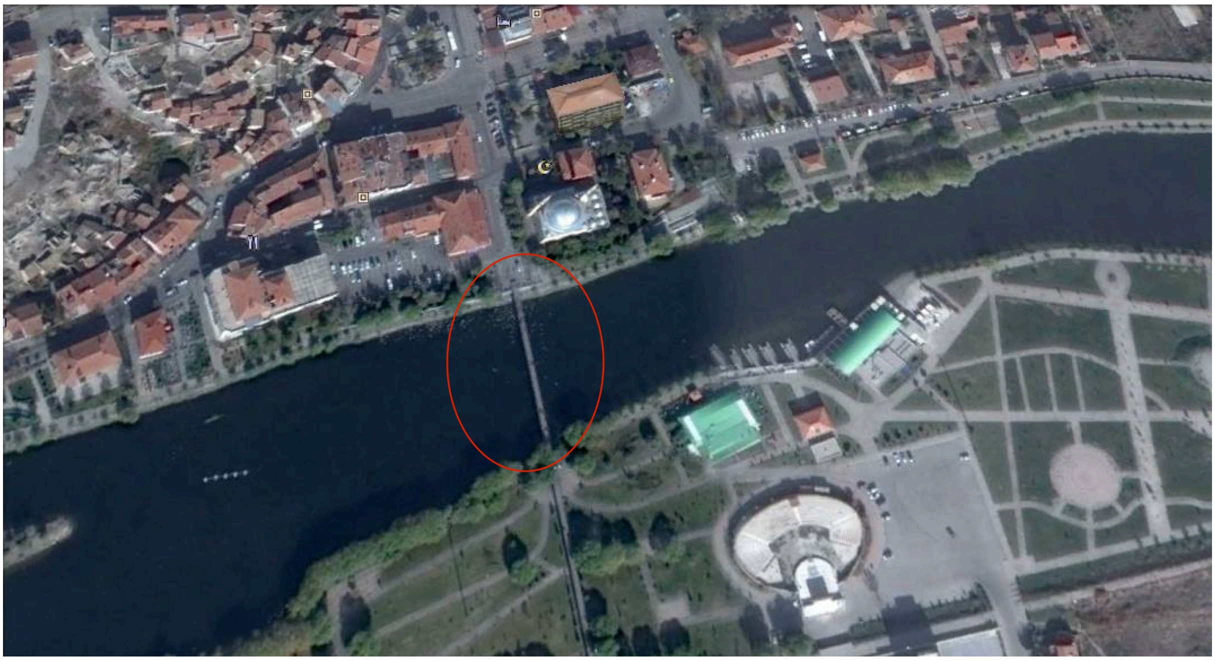

Figure 1. Satellite View of the Shaking Bridge in Avanos

1.2. Aim and Scope of the Project

Aim of this project is determined to be stabilizing the bridge while allowing some vibration in order not to block the tourist attention if it is possible or designing a new suspension bridge similar to the old one without any safety issue, upon the request of the mayor of Avanos. High density of tourists increases the risk of failure and creates major serviceability problems, by creating nausea like sensation while passing over the bridge. Upon the site visit it has been observed that even 3 people walking with the same period creates an incredible amount of wobbling effect on the bridge.

The Shaking Bridge is a suspension bridge, which is one of the oldest engineering techniques, that are used by the primitive people, as shown in Figure 2 and got stronger with the use of iron in the cables in 4th century by Indians. Moreover, suspension bridges provide an economic solution over long spans of navigable channels (Encyclopedia Britannica, n.d.).

Figure 2. Primitive Suspension Bridge made by Incans

Shaking Bridge in Avanos is constructed as a suspension bridge in order to enable the gondola service with ease and minimizing the cost. Shaking Bridge consist of two parts one on the Kizilirmak river and in the continuation of that part there exist a part on the land. The problematic part of the bridge is the former one, since the portion that is stretching out on the land is stabilized by connecting the bridge to the land. Moreover, it should be noted that same solution is tried to be applied to the problematic portion as well by putting rock blocks and connecting the bridge to these blocks by spring ended cables (see Figure 3), but this solution did not work, in addition architect of the municipality believes that this solution technique caused some deformation on the middle span of the bridge, which as a result increases the vibration problem in the long run.

Figure 3. Unsuccessful Solution for the Vibration Problem by Municipality

1.3. Vibration Phenomenon

Vibration is the periodic motion of any particle in an elastic body, which is back and forth. This phenomenon occurs when a body is disturbed from its equilibrium, and allowed to react to the external forces that tend to reconstruct its equilibrium position (Encylopedia Brittannica , 2016), This could best be understood with the spring-mass example as it can be seen in Figure 4. If the system is disturbed from its equilibrium position it tends to go back to the equilibrium position, this back and forth motion continuous until all the kinetic and potential energy is lost due to frictional forces. This motion of the spring-mass system is defined as the simple harmonic motion. If there are no frictional forces acting on the system, it will move in a sinusoidal motion through time as it can be seen from Figure 4.

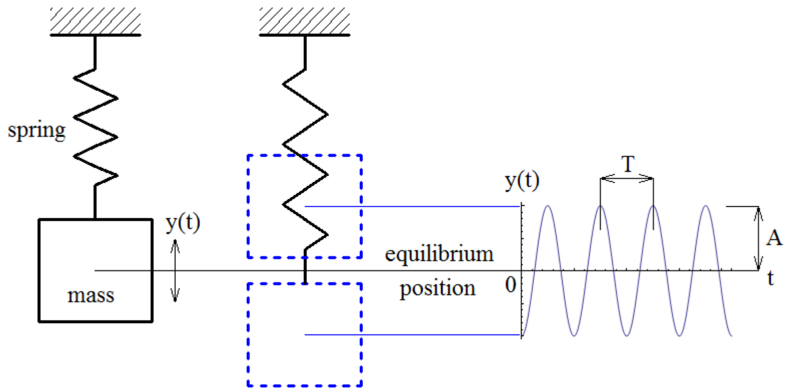

Figure 4. Simple Spring Mass System Equilibrium and Disturbed Positions

The main damper of this kind of systems are frictional forces and these forces amortise the energy and enable the spring mass system to go back to his equilibrium position. However, it should be noted that the dampers do not avoid the object to go into undisturbed state it just enables the system to absorbed the energy induced by the external forces, the dampers amortizing ability will depend on the dampers internal features and the external forces' magnitude. Moreover, natural frequency of this spring motion can be calculated using the equation given below (CHAN);

$$f = \frac{1}{2\pi} * \sqrt{\frac{k}{m}}$$

It should be noted that according to this equation without changing the stiffness of the object or the mass of the object it could be impossible to change its natural frequency.

Every rigid object has a structural resonance frequency, which is a frequency when entered the particles in that object wants to vibrate more, the best example of this situation is the urban myth of an opera singer, shattering a goblet with her voice alone. This situation can be explained in comprehensible way; the sound of the person is the result of the vibration of the vocal chords of that person, which travels in the air with a certain frequency if that frequency is in the range of the goblet the particles in the goblet starts to vibrate, and when the frequency of the soundwave reaches exactly the natural frequency of the goblet, it starts to vibrate more and more. If this state could be provided a few seconds theoretically the goblet particles depart from each other, which leads to the shattering of the goblet. This state of the object is called resonance. It should be noted that this phenomenon is almost analogous for the large objects like buildings as well, which could enter resonance with wind and earthquake forces. In order to avoid dangerous situations big buildings are made by considering these kind of effects by engineering and architectural expert cooperation (Things of Interest, 2011).

In order to, understand the concept of resonance, natural frequency and effect of damper see Figure 5.

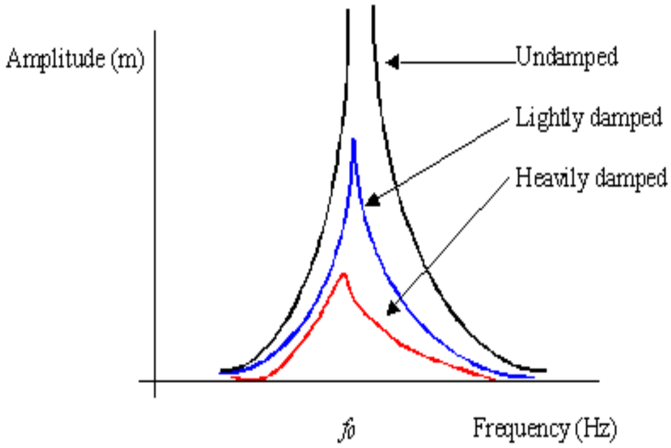

Figure 5. Effect of Damper on the System

As it can be seen from the Figure 5 if the frequency range of one object coincides with the other objects natural frequency range they both start to act as one and vibrate together. Moreover, damper usage only decreases the amplitude of the vibration in spring case deformation of the spring, but it does not create full absorption of the exerted force, which means dampers could only be used in order to decrease the effect of vibration, not for wiping out the vibration problem.

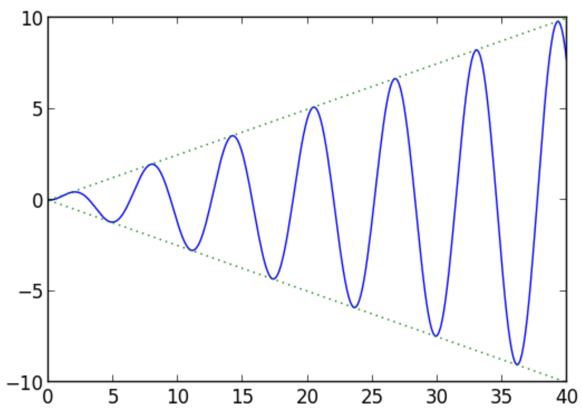

Figure 6. Behaviour of Undamped Vibration

Damper, behaviour is also should be known in order to limit the problems caused by the vibration. As it can be seen in the Figure 5 if there is no damper the vibration amplitude goes to infinity, which means the total energy cannot be consumed. Another visualisation for better understanding the phenomenon is provided in Figure 6. However, when the damper is used the amplitude of the vibration can be fixed to a value and vibration could be taken under control until a certain point. Vibration behaviour could be observed in Figure 7.

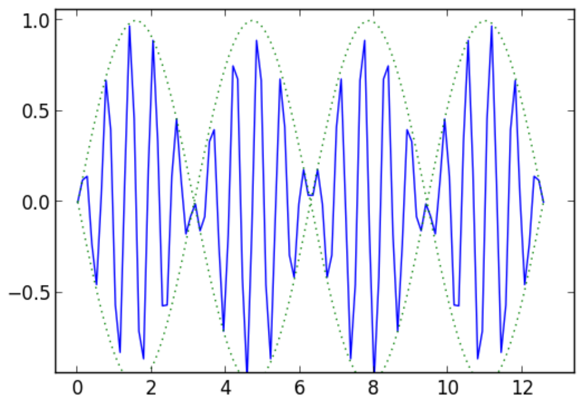

Figure 7. Behaviour of Damped Vibration

As it can be seen the amplitude of the vibration is fixed to a value and it never exceeds that fixed value.

1.3.1. Vibration Problem in Pedestrian Bridges

In recent past there a lot of pedestrian bridges that have experienced large amplitude pedestrian- induced vibrations. Main cause of this phenomenon is the low mass, stiffness and damping. The most recent and known example of this phenomenon is the Millennium Bridge in London, which has closed immediately after the vibration complaints. Vibration in that case was the result of the large number of people passing the bridge, which starts synchronized walking motion after a while (Ricciardelli F. , 2007).

It is obvious that pedestrian movement is a very complex motion which introduces a complex system of forces on three dimensions, which are changing in space and time, depending on the physical attributes of the pedestrian (Ricciardelli F. , 2016). It should be noted that the Shaking Bridge in Avanos is designed in such a way that the human induced forces are assumed to be acting as structural loads by ignoring the dynamic behavior. Moreover, it has been observed that bridge motion modifies the actions that are exerted by the pedestrians. The basic effect of this vibration on the pedestrians are discomfort nausea like symptoms, which are serviceability issues of the structures. Furthermore, it should be noted that what makes modelling of the pedestrian walking on the bridge so complex is taking into consideration of both inter-pedestrian and pedestrian-bridge interactions, in addition to that there are synchronized and incoherent groups of people at the same time while some people just try to force the bridge to vibrate (Ricciardelli F. , 2016).

Recently, modern and architecturally appealing bridges are being constructed all over the world, but while being modern and aesthetic their weights are reduced significantly, moreover, this weight reduction is caused by the increased material capacities too. Thus, these lightweight bridges have low mass inertia, which in turn drastically decreases the natural frequency of the structure. It is known that if the natural frequency of the structure line up with the frequency of the excitation a resonance phenomenon occurs. Pedestrians induce an unsteady, transient and swaying excitation on the bridge in a small range of frequency, which is between 1 and 2 hertz (Hz). Moreover, vibrations are effective in three dimensions (RWTHAACHEN University).

Thus, according to the subjects mentioned above the natural frequency of the structure should not match with the pedestrian induced frequency range in order to be stable in 3 directions.

1.3.2. Analysis of the Current Bridge

A field trip has been conducted with the Professor Alp Caner and his PhD. student Nefize Saban, in order to observe the current situation of the bridge and found the natural frequency of the bridge with the help of accelerometers.

After taking measurements and conducting a Fast Fourier Transform (FFT) Analysis, Nefize Saban had given the graphs in Figure 8,Figure 9 and Figure 10 that shows the natural dominant frequencies of the bridge in 3 directions. It should be noted that FFT analysis coverts a signal from time and space in to the frequency. It should be noted that a1, a2 and a3 indicates the longitudinal, transverse and vertical direction respectively.

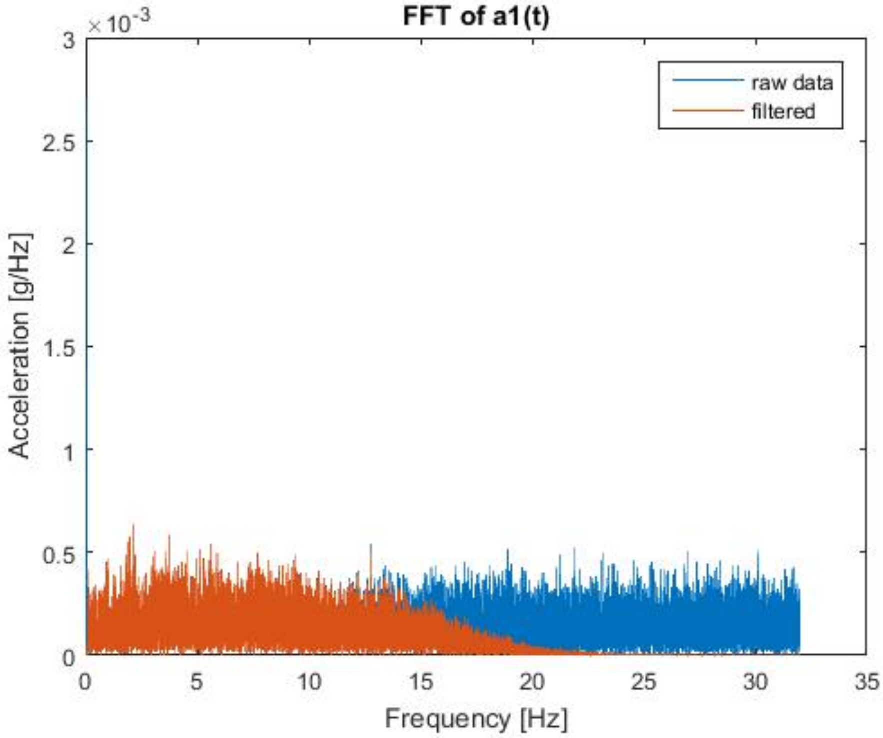

Figure 8. Longitudinal Frequency Vs Acceleration Graph

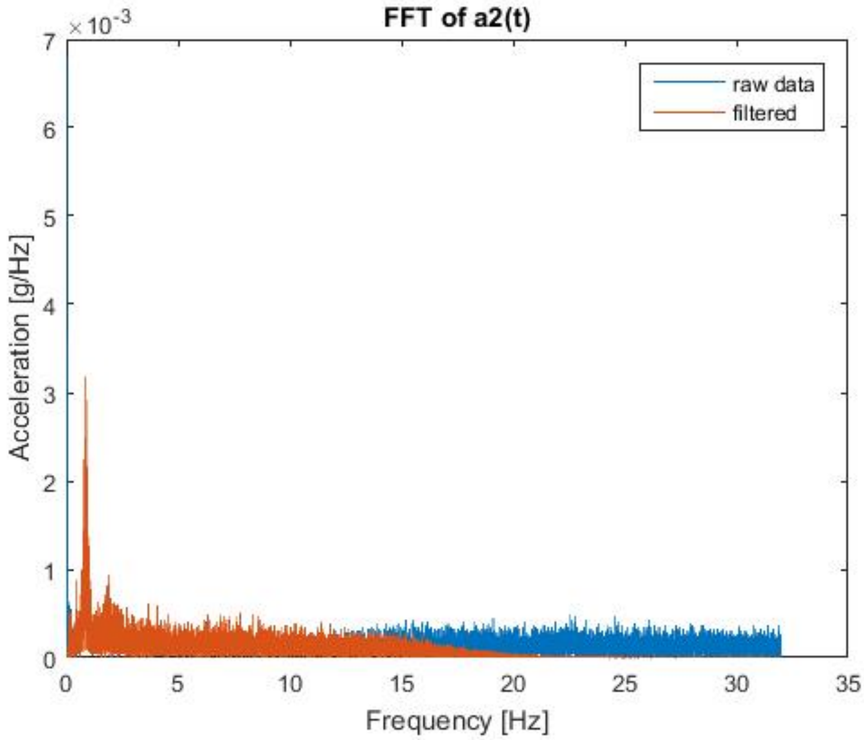

Figure 9. Transverse Frequency Vs Acceleration Graph

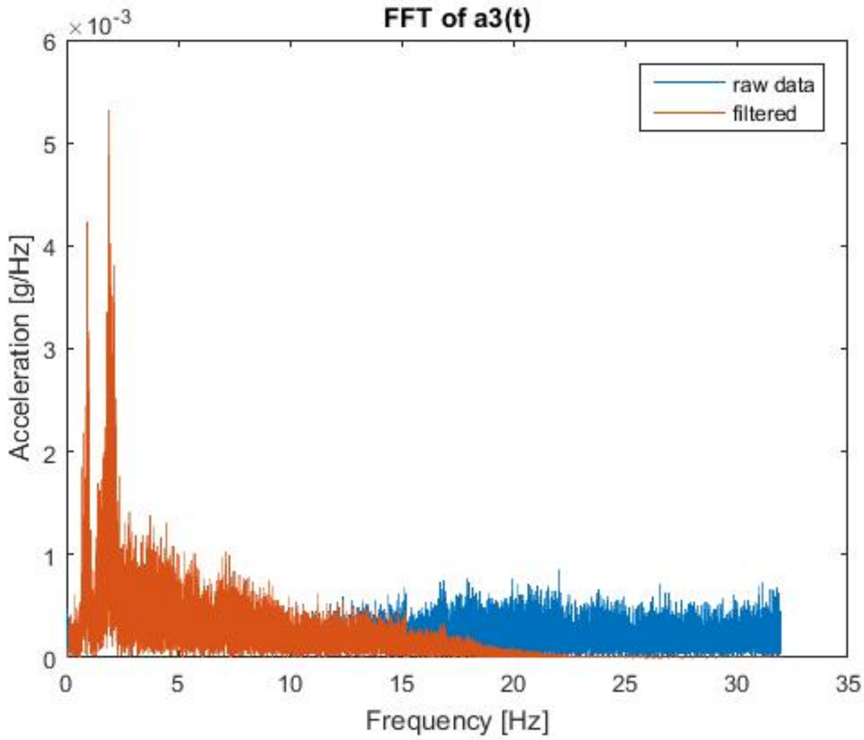

Figure 10. Vertical Frequency Vs Acceleration Graph

As it can be seen from the above graphs the dominant Frequencies can be listed as shown in Table 1 ,

Table 1. Dominant Frequencies in Three Dimensions

Direction	Frequency(Hz)
Longitudinal	2.1162
Transverse	0.82715
Vertical	1.9092

As it can be seen in Table 1, dominant frequency of the bridge is found between 1 and 2 Hz, which means natural frequency of the bridge lines up with the pedestrian walking frequency. This situation will cause the bridge to wobble with the people walking on it. In addition to this fact, the bars that, that connects I Beams under the deck to the suspended ropes of the bridge, are subjected to 1800 cycle in 250 seconds. The cycle number that leads to failure is approximately 10^7 cycles according to ASM International Fatigue Resistance of steels handbook. So,

$$Cycle\ per\ day = \frac{1800\ (cycle) * 3600\ (seconds) * 24\ (hours)}{250\ (seconds) * 1\ (hours)} = 622080\ cycles/d$$

$$Days\ until\ Failure = \frac{10^7\,(cycles)}{622080\ (cycles/d)} = 16\ days$$

As it can be seen from the calculations one bar fails in approximately 16 days which is a very optimistic number, since the mayor of Avanos had stated that at least one bar fails in a week.

After the calculation of the natural frequencies of the bridge, in order to see how it would react to damping another measurement is made with hanging 4 bags of 10 kg additional weight 20 on one side and 20 on the other side. Vibration measurements are taken from the bridge as it can be seen in Figure 11, Figure 12, and Figure 13. Moreover, the dominant frequencies can be found in Table 2.

Table 2. Dominant Frequencies in Three Dimensions (with mass damping)

Direction	Frequency(Hz)
Longitudinal	2.1162
Transverse	0.82715
Vertical	1.9092

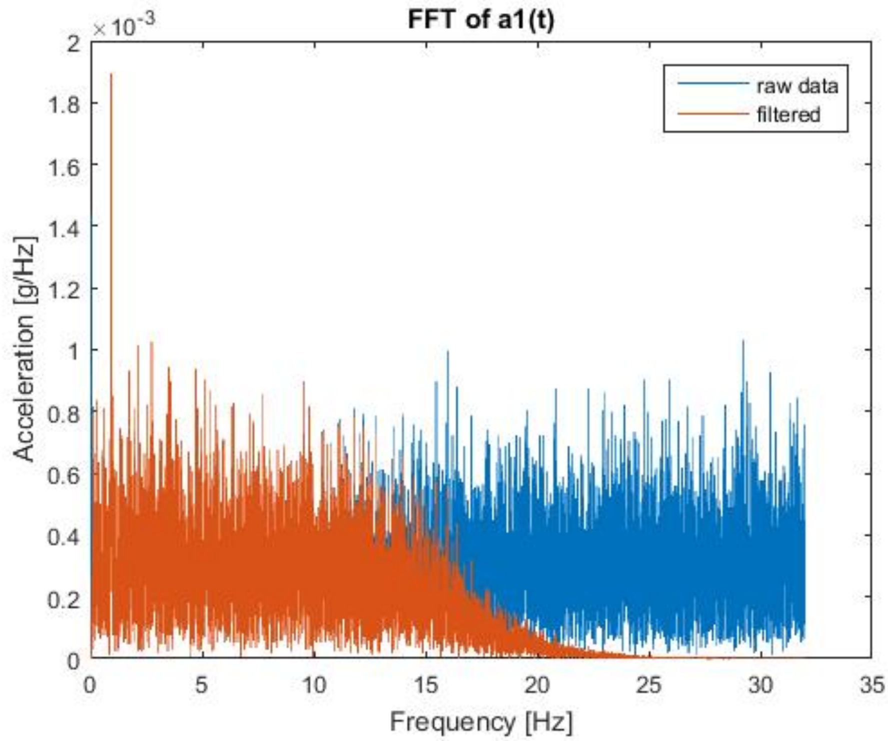

Figure 11. Longitudinal Frequency Vs Acceleration Graph (with mass damping)

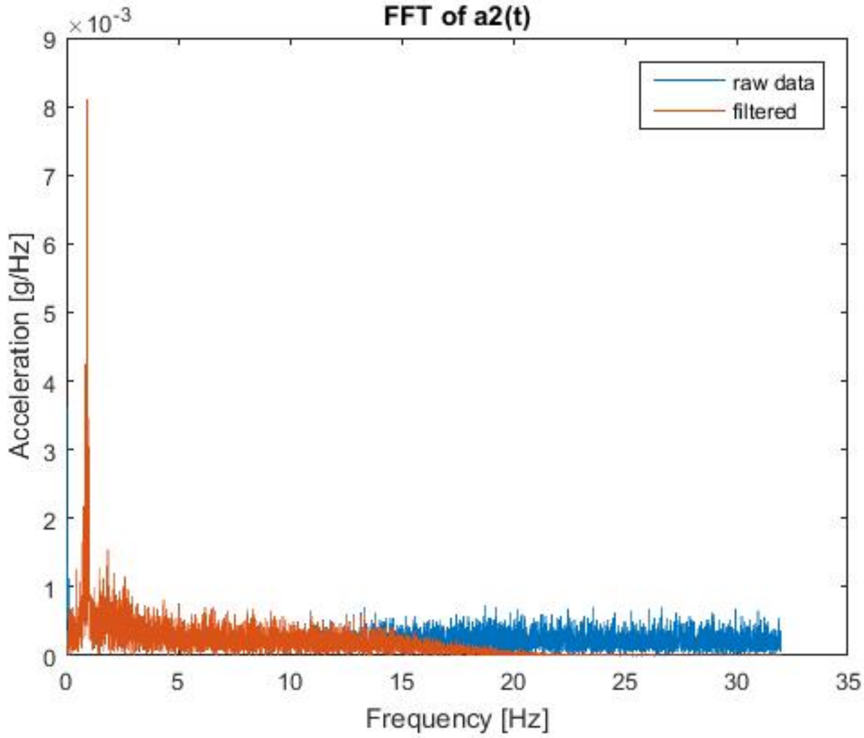

Figure 12. Transverse Frequency Vs Acceleration Graph (with mass damping)

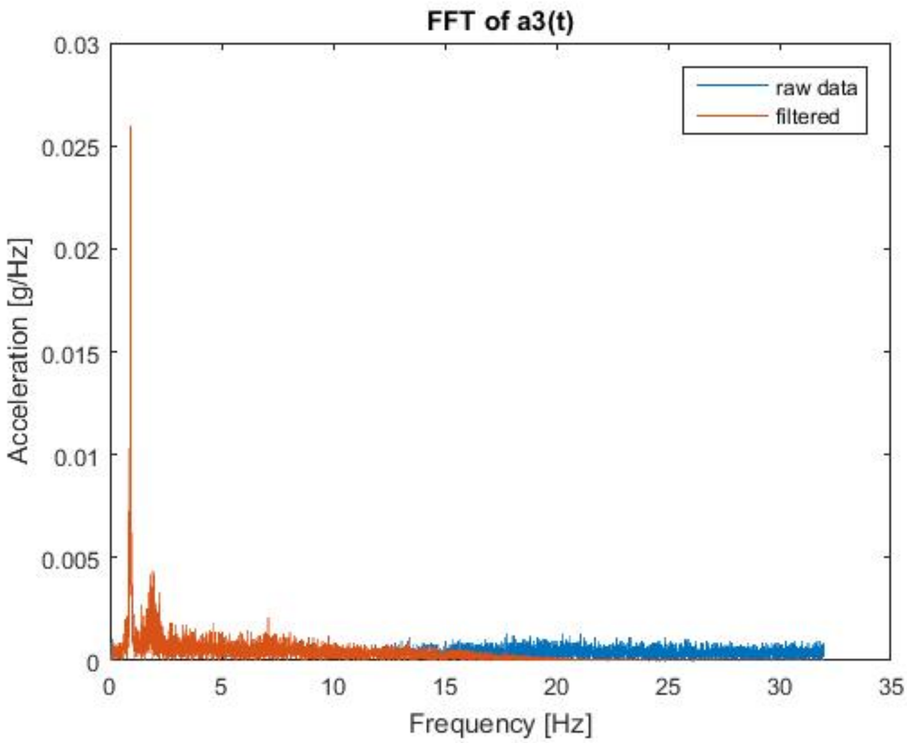

Figure 13. Vertical Frequency Vs Acceleration Graph (with mass damping)

As it can be seen in although the mass dampers are added the dominant frequency could not be changed, so it is concluded that the mass damping solution for the vibration problem cannot solve the existing vibration problem. Moreover, there is a limit for mass damping since the load carrying capacity of the bridge has a limit.

Other than natural frequency and fatigue of the steels, current suspension bridge pylons are 3.8 meters which is very small. Moreover, it should be noted that there is not any written information about the bridge, so the rope strengths, tension in the ropes, or bars are not known, in addition; the connections used in the bridge in order to carry the deck are very susceptible to fatigue which do not enable any motion in the longitudinal direction as it can be seen in Figure 14.

Figure 14. Connections for Carrying Deck

As stated above the dominant natural frequency of the current suspension bridge is found between 1-2 Hz, which coincides with the pedestrian walking frequency causing the bridge to vibrate, during the bridge crossing of the pedestrian. The first thought was to add dampers and minimize the amplitude of the vibration, however, this application only reduces the amplitude of the vibration. So, this would not be a solution to the bridge vibration, moreover; after some point the usage of the dampers will negatively affect the bridge, since the mass of the dampers should be manageable in order to be carried by the bridge itself. It is evident that without playing with the mass or stiffness of the bridge, frequency of the bridge cannot be increased and separated from the frequency range of the walking pedestrians.

By considering all the mentioned factors above it has been decided to design a new bridge from start in order to create a more stable and safe transportation for all the pedestrians that are passing through the bridge every day. The construction year and lack of document existing about the bridge makes increasing the mass or playing with the stiffness of some elements. Thus, the final verdict is that, design and construction of a new bridge will be a more effective solution, than trying to solve the vibration problem of the current suspension bridge with the dampers. This project will also help the municipality since, it has been stated that, municipality is planning to build 3 more bridges across the Kizilirmak. So, from this point on this paper will focus on the design of a new pedestrian bridge in 90-meter length in order to provide safe and stable transportation for the people.

1.4. Structural Features of the Bridge

It has been decided that construction of another suspension bridge will not disturb the aesthetic and authentic visualization of the area and it will create a little vibration compared to other bridge techniques like cable stayed bridges, moreover, it will definitely be less costly, since it this bridge technique does not utilizes the new technologies like pre-tensioning or post tensioning. Furthermore, it has been decided that the bridge will be very similar to the old one in order to be cost effective and familiar to the local residents. During the field trip it has been observed that the local residents like passing thru this bridge, although it wobbles.

It should be noted that there will be no piers in the stream in order to provide sufficient room for gondolas to pass underneath the bridge, which is another reason why the suspension bridge is selected.

Advantages of suspension bridges are as follows (ConnectUS, 2015);

- The main advantage of the suspension bridges is the fact that its construction is very inexpensive, the materials used are very basic which are anchorages connections and ropes, considering the budget limitation of the municipality it is very crucial to make a safe bridge with the least cost.

- Ease of maintenance work is another advantage of the suspension bridges; this type of bridges does not need any major constructional maintenance work over long time periods which again makes it appealing considering the budget of the municipality.
- Bearing the earthquake loads is another important feature of the suspension bridges, their flexible features enable the bridge to vibrate and wobble during the earthquake, freely.

Disadvantages of suspension bridges are as follows;

- Suspension bridges are very susceptible to wind loads because of their flexibility.
- Limited load carrying capacity compared to other type of bridges is the main disadvantage of this type of bridges, however; for a small province like Avanos this disadvantage becomes problematic when the tourist number is very high.

By considering all the advantages and disadvantages in addition to the current situation of the Avanos, cable stayed bridge construction seams very feasible.

2. DESIGN CRITERIA

2.1. Introduction

This document includes the codes and standards and criteria that will be used in the CE-410 Project of the Versatile Group, which has a footbridge project in Avanos, that has a vibration problem. More particular information about the project will be given in the final report of the project.

2.2. Codes and Standards

The design and control of this engineering project will be done according to the design specifications of the pedestrian bridge which is published by the American Association of State Highway and Transportation Officials. In case of any missing or conflicting information requirements of the more conservative documents will be used in order to be on the safe side.

- LRFD Guide Specifications for the Design of Pedestrian Bridges
- AASHTO LRFD
- AASHTO Signs
- Suspended Pedestrian Bridge Design and Analysis

2.3. Structural Design Criteria

2.3.1. Design Loads

- Dead Loads;

Dead loads include the complete weight of the structure and all the equipment that is used for fastening or connecting structural elements.

- Pedestrian Load (Live Load);

Live loads include the load of the people that are passing through the bridge, since the bridge is not suitable for vehicle entrance only pedestrian loads will be considered.

AASHTO LRFD specifications uses a base nominal loading of 85 psf without considering the influence area, since we are using LRFD specifications these accepted pressures are factored using LRFD load factor which is 1.75 (American Association of State Highway and Transportaion Officials, 2009).

$$1.75 * 85 = 149\, psf$$

It should be noted that there is a physical limitation for the maximum pedestrian loading which is taken as 150 psf. Pedestrian bridges are to be designed for a uniform loading of 90 psf – 4.31 kPa, this amount of load is patterned on the bridge to produce the maximum load effects. Combining this with load factor;

$$1.75 * 90 = 158\, psf$$

It should be noted that taking the nominal live load capacity as 90 psf gives borderline, but sufficient pressure estimation compared with the maximum credible pedestrian loading which is taken as 150 psf –7.18kPa. (American Association of State Highway and Transportaion Officials, 2009)

- Wind Loads;

Wind load is taken as the force caused by pressure of 0.02 ksf over the full deck width which is applied at the same time, facing the wind at the quarter point of the deck width.

2.4. Design Bases

2.4.1. Allowable Stresses

Calculated stresses in any member of the structure caused by different loading combinations will not exceed the maximum allowable stresses permitted by the applicable codes.

2.4.2. Load Factors and Combinations

It is stated in the LRFD Guide Specification for the Design of the Pedestrian Bridges that no factor is going to be taken, dead load and live load will not be factorized.

3. SYSTEM GEOMETRY

From what we have seen when we went to the bridge, the width of the bridge which was 2,3 m was not enough when large group of people are crossing. Therefore, we wanted to enlarge it to 3,5 meters. Our bridge has 90 meters' span length. Some of the suspension bridges in our country has these longest span/width ratios shown in the Table 3. According to average of these ratios our bridge should be 3 meters. However, we wanted it to be wider in order to ease the crossing of crowded groups and also we wanted our bridge to be heavier to make it period higher.

Table 3. Width to Span Ratios of Suspension Bridges

Bridges	Longest Span	Width	Ratio
Fatih Sultan Mehmet	1090	39	0.036
Yavuz Sultan Selim	1408	58.5	0.041
Bosphrous	1074	33.4	0.031
Osman Gazi	1550	35.93	0.023
Avanos	90	2.96	0.032

3.1. Dimensions

The height of the suspension bridge towers above deck is dependent on the sag to span ratio which can vary from about 1:8 to1:12. A good preliminary value is about 1:10 (Wai-Fah Chen, 1999). Therefore, we selected our bridge towers height above the deck as 9 meters and below the deck it is 3.5 meters as shown in the Figure 15.

The sag ratio is taken as L/20 (Bridges to Prosperity, 2009). Then at the middle point of our bridge, the height of the main rope from the deck is 4,5 m.

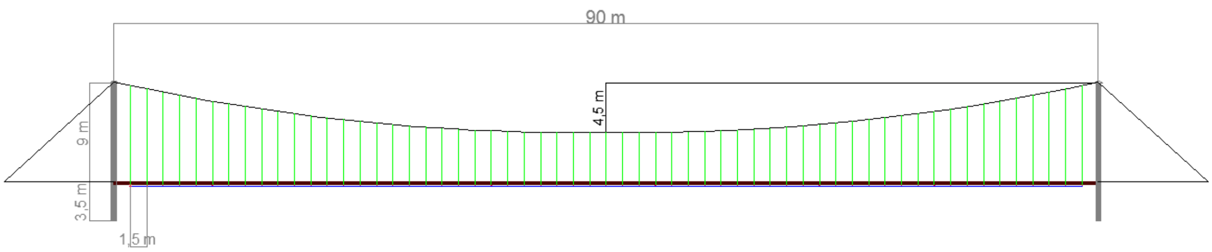

Figure 15. Side View of the Avanos Bridge

General dimensions of our bridge is tabulated in the Table 4.

Table 4. General Dimensions of Avanos Bridge

Avanos Bridge General Dimensions	
Span Lengths (m)	90
Pylon Heights (m)	12,5
Pylon Type	H
Width (m)	3,5
Sag Length (m)	4,5

Our bridge has upper timbers that pedestrians walk through. Another timber section that is under upper timber that will be nailed to the upper timber and placed on the IPN beams, in order to keep upper timbers together and to spread the load to the IPN sections. We selected IPN 200 steel sections, which have 1,5 m spacing between them, will be connected to main ropes of our bridge. Under these IPN 200 sections we will weld rectangular bars to keep them together. Section view of our bridge at the middle point is shown in the Figure 16.

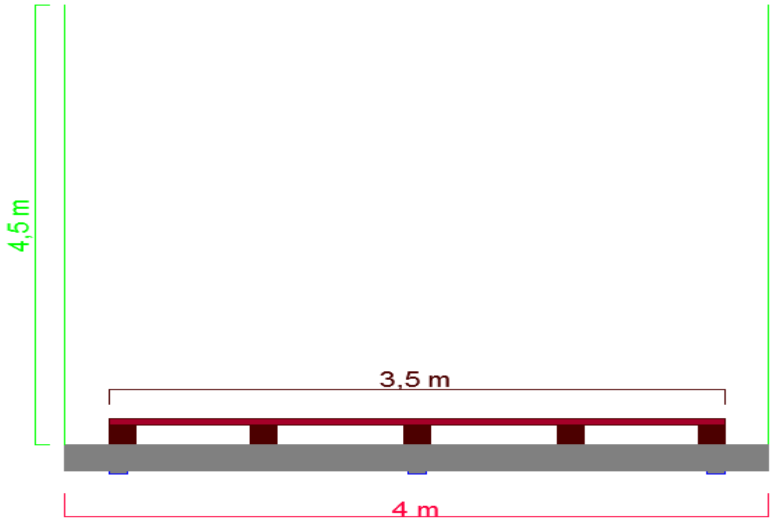

Figure 16. Section View of the Bridge

3.1.1. Detailed Drawings of the Members in the Bridge

- Upper Timbers

Figure 17. Side and Section View of Upper Timber

Upper timber is shown in the Figure 17. It has 5 cm depth, 10 cm width and 4 meters length. It is the layer that people will step on it. Whole cross section of the bridge deck will be layered with this timber. It will be nailed to the timbers underneath.

- Timber

Figure 18. Side and Section View of Timber

Timber will be placed under the upper timber. As it is shown in the Figure 18 , it has square section with 15x15cm. Its length will be 3 meters and it will be attached to the top of the IPN 200 sections. It will distribute the load to the IPN 200 sections. Five pieces of them will be used along with the whole span as it is shown in the Figure 16.

- IPN 200

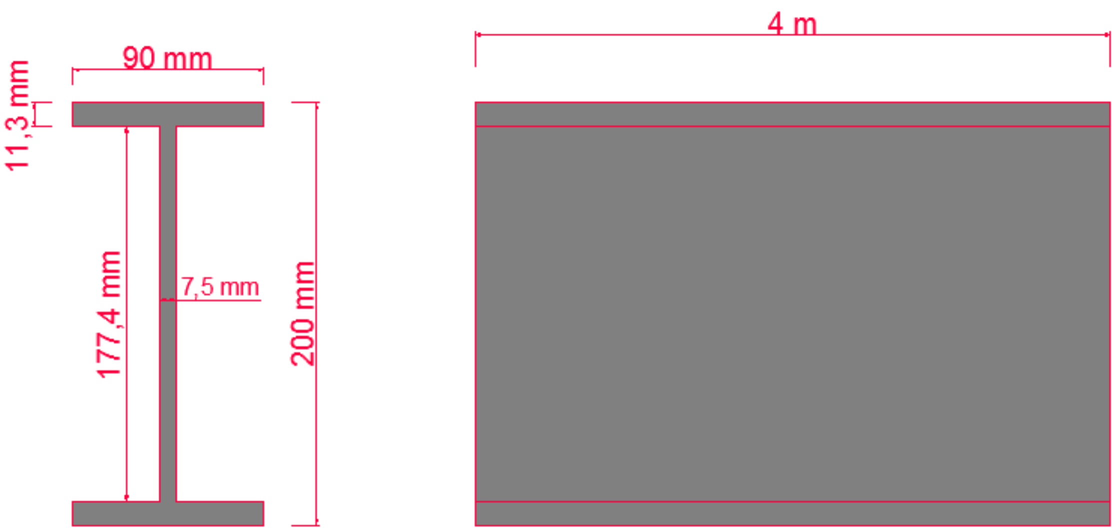

Figure 19. Side and Section View of the Beams

As it is shown in the Figure 19 we will be using IPN 200 sections. Its length is 4 meters. Flange thickness is 11,3 mm and it has 7,5 mm web thickness. The height of the section is 200 mm.

This section will carry the load and transfer it to the main rope by vertical ropes. There will be fifty-nine of them in our bridge. They will be placed with 1.5 m spacing.

- Bottom Steel

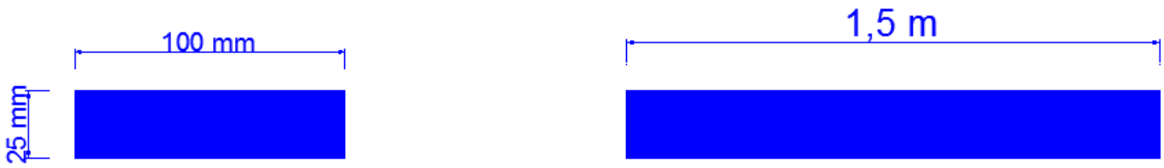

Figure 20. Side and Section View of the Bottom Steel

Bottom steel is in Figure 20. It has 25 mm thickness and 100 mm width. These steels will be welded to the bottom of the IPN 200 sections in order to keep them compact. At the sides and at the middle of the IPN 200 sections, this sections will be attached as shown in the Figure 20.

- Pylons

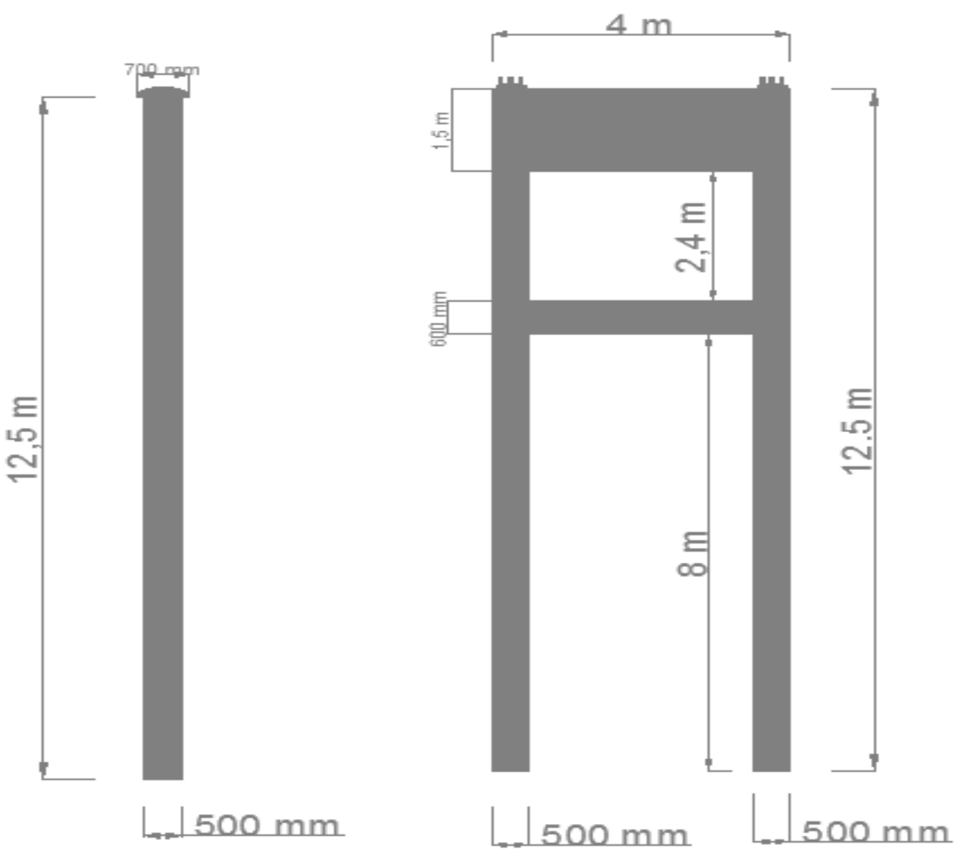

Figure 21. Side and Section View of the Pylons

In the Figure 21, our pylons' dimensions are given. Its total height is 12.5 meters. 9 meters of the height will be above the deck and 3.5 meters will be below. The feet of it have square cross section with 50 cm thickness. There are places at the top of our pylons for the main ropes to seat them.

- Connections

Figure 22. Connection of IPN200 to Main Cables

This equipment in Figure 22 will be used to connect IPN 200 sections and main rope. A plate with a hook on it will be bolted to the sides of the IPN section and this equipment will be attached to that hook then with vertical ropes it will be connected to main rope. Dimensions of this equipment is not decided yet, however this is the type that we are thinking about using in order to prevent the loss of ropes in longitudinal direction vibration, which if fastened exposed to a lot of cycles, as a result fails in 1 or 2 weeks.

4. STRUCTURAL ANALYSIS & DESIGN

It should be noted that there are several things to be checked for the preliminary design which are forces in the main rope, stresses in the I Beams are the main focus in this preliminary calculations. For these calculations to be made the dead load and the live load should be determined meticulously. However, it should be noted that these calculations are very rough and the results of these calculations will only be used for having a rough information about the general behavior and the founding the needed strengths of the materials that ae going to be selected.

Calculations in this point are very rough as stated and done only statically. Detailed calculations will be done with the help of a software, while investigating the dynamic behavior of the structure, and the results of this second part will be presented in the final report.

4.1. Dead Load Calculation

For calculation of the dead loads unit weight of the selected steel parts and density of the wood is used.

- I-Beams;

As stated before IPN-200 type of I Beams are selected, Basic Properties of the selected beam is shown in Table 5.

Table 5. IPN-200 Sectional Properties

I Beam					
IPN 200	S_x (m^3)	Quantity (#)	Length (m)	Total Length (m)	Unit Weight (kg/m)
	0,000214	59	4	236	26,2

$$Total\ Length = Quantity * Length = 59 * 4 = 236\ (m)$$
$$Total\ Weight = Total\ Length * Unit\ Weight = 236 * 26{,}2 = 6183.2\ (kg)$$

- Timber on the Beams;

Basic properties of the Timber are shown in Table 6.

Table 6. Timber Properties

Timber				
Quantity (#)	Height (m)	Width (m)	Length (m)	Density (kg/m^3)
5	0,15	0,15	90	670

$$Volume\ of\ Timber = Height * Width * Length = 0.15 * 0.15 * 90 = 2.025\ m^3$$

$$Total\ Weight = Volume * Density = 2.025 * 670 = 6783.75(\ k)g$$

- Bottom Steel;

Basic properties of the selected bottom steel are given in Table 7.

Table 7. Bottom Steel Properties

Bottom Steel		
Quantity (#)	Length (m)	Unit Weight (kg/m)
3	90	19,62

$$Total\ Weight = Quantity * Length * Unit\ Weight$$

$$Total\ Weight = 3 * 90 * 19.62 *= 5297.4(kg)$$

- Upper Timber;

Basic properties are given in

Table 8. Upper Timber Properties

Upper-Timber			
Height (m)	Width (m)	Length (m)	Density (kg/m^3)
0,05	3,5	90	670

$$Volume\ of\ Timber = Height * Width * Length = 0.05 * 3.5 * 90 = 15.75\ (m^3)$$

$$Total\ Weight = Volume * Density = 15.75 * 670 = 10552.5\ (kg)$$

As a result of these calculations the total force acting on 1 m^2 area can be calculated as follows;

$$Total\ Weight\ (kg) = Total\ Weight\ of\ Timbers(kg) + Total\ Weight\ of\ Steel(kg)$$

$$Total\ Weight\ (kg) = 6183.2 + 6783.75 + 5297.4 + 10552.5 = 28816.85\ (kg)$$

$$Total\ Dead\ Load\ Weight\ (kN) = Total\ Dead\ Load\ (kg) * g\ (m/s^2) * 10^{-3}$$

$$Total\ Dead\ Load\ (kN) = 28816.85\ \ (kg) * 9.81\ \left(\frac{m}{s^2}\right) * 10^{-3} = 282.7\ (kN)$$

Calculations in order to find the stress caused by all of the dead load on the deck, are shown below;

$$Stress = \frac{Total\ Dead\ Load}{Area} = \frac{282.7}{90 * 3.5} = 0.897\ (kPa)$$

It should be noted that the beams cover 4 meters, but the deck that people can walk is 3.5 meters long, so the stress is calculated by using 3.5 meters.

4.2. Live Load Calculation

It is stated in the design criteria that the live load should be 150 psf, which is equal to 7.18 kPa. Detailed explanation can be found in design criteria section of this report.

4.3. Stress Calculation on the I Beams

It should be noted that the dead load is factorized with a rough coefficient in order to include the weight of the safety rail and the connecting cable weights.

Total stress on the deck is calculated as,

$$Total\ Stress\ on\ the\ Deck = 1.2 * Dead\ Load + Live\ Load$$

$$Total\ Stress\ on\ the\ Deck = 1.2 * 0.897 + 7.18 = 8.26\ (kpa)$$

Note that 1.2 is a factorization for the LRFD Guide;
In order to find the stress caused by the flexure, finding the distributed load acting on one beam is a must, so

$$Distributed\ Load\ on\ a\ I\ Beam = Total\ Stress\ on\ the\ Deck * Length\ of\ the\ Deck$$

$$Total\ Distributed\ Load\ on\ \ I\ Beams = 8.26\ kPa * 90\ m =\ 743.12\ (kN/m)$$

$$Total\ Distributed\ Load\ on\ a\ I\ Beam = \frac{743.12}{59} = 12.59\ (kN/m)$$

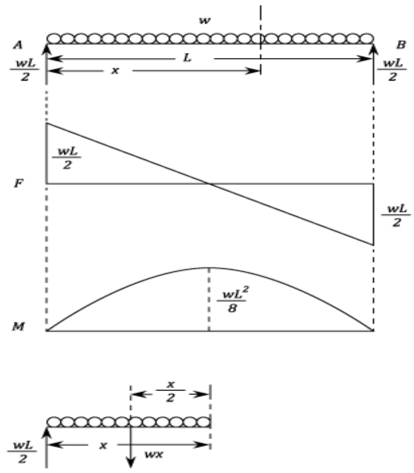

Figure 23. Moment and Shear Diagrams Under Distributed Loading

As shown in Figure 23 Beams are under distributed loading and the maximum moment occurs in the middle of the beam and can be calculated as follows;

$$M_d = \frac{(W_g + W_q) * L^2}{8}$$

$$M_d = \frac{12.59 * 4^2}{8} = 25.18\ (kN * m)$$

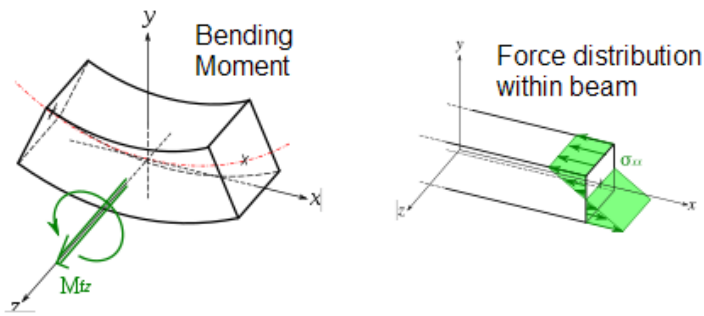

Figure 24. Stress Distribution on a Beam Under Flexure

After, moment calculation the maximum stresses, which are compression on top and tension on the bottom, as it can be seen from the Figure 24, that will develop on the top fibers of the beam could be calculated as follows;

$$\sigma_{max} = \frac{M_d}{S_x}$$

$$\sigma_{max} = \frac{25.18 * 10^3}{214 * 10^{-6}} = 117.66\ MPa$$

So, if the Steel with yielding capacity higher than 117.66 MPa is selected the beam will be safe under the most extreme dead load and live loads.

4.4. Calculation of the Forces Developing on the Main Ropes

There is tension in the main ropes and compression on the pylons, there is 2 components of the tension force on the rope which are vertical and horizontal, as shown in Figure 25.

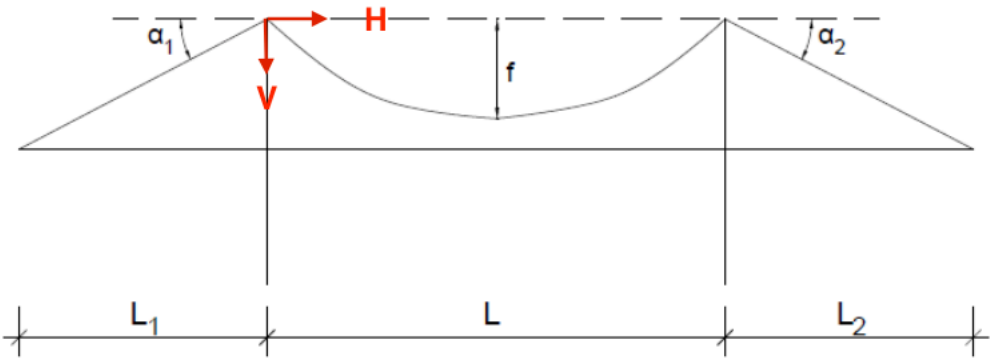

Figure 25. Suspension Bridge Forces in Main Ropes

It should be noted that the forces on the main ropes can be calculated using the following formulations (Bridge Turkey Organization, 2016);

$$V = \frac{(W_g + W_q) * L}{2}$$

$$H = \frac{(W_g + W_q) * L^2}{8 * f}$$

$$T\ (Resultant\ Force) = \sqrt{V^2 + H^2}$$

Distributed load in the 90 meters' span should be found first to be used in this calculation so,

$$Distributed\ Load\ on\ 90\ m\ Span = Total\ Stress\ on\ the\ Deck * Width\ of\ the\ Deck$$

$$Distributed\ Load\ on\ 90\ m\ Span\ (W_g + W_q) = 8.26\ (kPa) * 3.5(m) = 28.92\ (\ kN/m)$$

So the total forces on the main ropes can be calculated as shown below, moreover; it should be noted that it has been decided that four main ropes will be used in this bridge so the total force is divided equally between 4 ropes

$$V_{Total} = \frac{28.92 * 90}{2} = 1301.4\ (kN)$$

$$V_{One\ Rope} = \frac{1301.4\ (kN)}{4} = 325.35\ (kN)$$

$$H_{Total} = \frac{28.92 * 90^2}{8 * 4.5} = 6507\ (kN)$$

$$H_{One\ Rope} = \frac{6507\ (kN)}{4} = 1626.86(kN)$$

The resultant force in one rope can be calculated as follows;

$$T = \sqrt{1626.86^2 + 325.35^2} = 1659.08\ (kN)$$

So, if all the ropes are selected with the design load greater than 1659.08 kN the bridge safely carries all the loads applied on it.

4.5. Load Calculations on the Ropes Connected to Main Cables

Total load of the deck and the live load is transferred to the main cable of the suspension bridge, with the help of these ropes, and they share the load equally between them, so the load on one rope can be calculated as follows;

$$T_{vertical} = \frac{Total\ Stress\ on\ the\ Deck * Area\ of\ the\ Deck}{\#\ of\ Ropes}$$

As mentioned in the bridge geometry part of this report there will be ropes connected to the all I beam on either sides of the beam, so

$$\#\ of\ Ropes = \#\ of\ Beams * 2$$

$$\#\ of\ Ropes = 59 * 2 = 118\, ropes$$

Thus, load on one vertical rope can be calculated as;

$$T_{vertical} = \frac{8.26(kPa) * (3.5 * 90)}{118} = 21.89\ (kN/rope)$$

So, if the design load of the selected cable is higher than 22 kN, ropes will safely hold the deck and the people on it in the most extreme conditions.

It should be noted that the above calculations are very raw calculations, since this report is a preliminary report, which has the aim to give general concepts of and these calculations are going to be detailed in the final report.

5. GEOTECHNICAL DESIGN

In general, foundations have vital role in structures, and its importance is originated from that they transfer loads on the structures to soil underneath them. However, to fulfill this duty in a trouble-free way, all possible scenarios of relationship between the structure and foundation should be considered and corresponding foundation design should be made according to the most critical case(s) of these possibilities, that is to say, load combinations most of the times. In addition to that, relation of the foundation and soil profile should also be examined i.e. settlement calculations etc.

5.1. Load Combination

Generally speaking, approximately 80% of the total vertical load on bridges is originated from the self-weight of the bridges. However, this bridge is an example of suspension bridge and timber used as platform, thus live load in most critical case can be greater than dead load. In addition, thickness of bridge is so small, wind load can be neglected for all stages. To narrow gap;

1.2D+1L used as load combination where;
D: Dead load
L: Live load

5.2. Soil Profile

In order to get idealized soil profile of the site, Standard Penetration Test (SPT) was introduced through 5 different boreholes. However, for all 5 of these boreholes, there are so limited SPT-N values because of that formation of the soil does not allow more penetration. In addition to this, those SPT-N values are around 50. This situation can be interpreted as that soil is too hard to penetrate. This clue may imply something about the features of the soil. In addition to that, since SPT cannot proceed throughout the profile some undisturbed samples were taken via drilling core to analyze them in the laboratory and result in more precise result. With the results of these laboratory tests (Toker Sondaj ve İnşaat, 2012) all relevant features and parameters of the whole soil formations and idealized soil profile were determined by using USCS system (see Figure 27.) According to conducted laboratory tests, soil profile consists of thin fill layer, a layer that contains clayey sand, sandy clay and at the bottom part clayey gravel and clay/silty clay (Ankara Clay) as indicated in Figure 26 . Clay parameters are shown on Figure 28. and Table 11. Further findings of these tests also indicate that there is a stable water table 11 m under the ground surface, so there is no need to use ground water table in foundation design since our foundation does not reach there. (See Appendix for further information.)

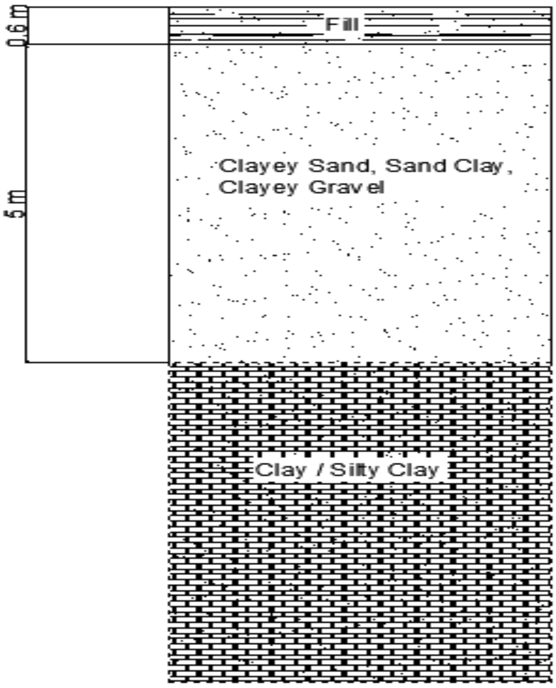

Figure 26. Soil Profile

Table 9. Soil Parameters

Group Symbol (USCS Classification)	**CH,CL**
Fine Material Ratio	F (%) = 51.5~98.8 (F_{rep}=83)
Gravel Ratio	G (%) = 0~5.5 (E_{rep}=1)
Water Content	w_n (%) = 16.7~31.3 ($w_{n\ rep}$= 25)
Liquid Limit	LL (%) = 24.8~87.6 (LL_{rep} =59)
Plastic Limit	PL (%) = 14.3~28.5 (PL_{rep} =21)
Plasticity Index	PI (%) = 10.5~66.5 (PI_{rep} =38)

COARSE-GRAINED SOILS
(more than 50% of material is larger than No. 200 sieve size.)

	Clean Gravels (Less than 5% fines)	
GRAVELS More than 50% of coarse fraction larger than No. 4 sieve size	GW	Well-graded gravels, gravel-sand mixtures, little or no fines
	GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines
	Gravels with fines (More than 12% fines)	
	GM	Silty gravels, gravel-sand-silt mixtures
	GC	Clayey gravels, gravel-sand-clay mixtures
	Clean Sands (Less than 5% fines)	
SANDS 50% or more of coarse fraction smaller than No. 4 sieve size	SW	Well-graded sands, gravelly sands, little or no fines
	SP	Poorly graded sands, gravelly sands, little or no fines
	Sands with fines (More than 12% fines)	
	SM	Silty sands, sand-silt mixtures
	SC	Clayey sands, sand-clay mixtures

Figure 27. USCS Soil Classification for Coarse Grained Soil

FINE-GRAINED SOILS
(50% or more of material is smaller than No. 200 sieve size.)

SILTS AND CLAYS Liquid limit less than 50%	ML	Inorganic silts and very fine sands, rock flour, silty of clayey fine sands or clayey silts with slight plasticity
	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
	OL	Organic silts and organic silty clays of low plasticity
SILTS AND CLAYS Liquid limit 50% or greater	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
	CH	Inorganic clays of high plasticity, fat clays
	OH	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS	PT	Peat and other highly organic soils

Figure 28. USCS Soil Classification for Fine Grained Soil

Table 10. Clay Parameters

Short Term	Long Term
C_u=230 kPa	c=20 kPa
$\Phi=0^o$	$\Phi'=25^o$
γ= 19.5 kN/m^3	-
$m_v=0.75*10^{-4}$ m^2/kN	-
E_s=70000 kPa	-

5.3. Foundation System

Pylons of bridge have H shape, thus, for each pylon there are 2 piers linked to the foundations. To avoid possible differences in settlement amounts for those 2 piers, it is decided to select combined footing system. Raft foundation cannot be used because piers are too close to each other and from economical view of aspect combined footing system is more reasonable than raft foundation system. Dimensions of foundations determined as 5*5*2 m. According to these dimensions, settlement and bearing capacity calculations will be conducted and results of these calculations will be revised if findings are not in acceptable limits.

5.4. Gross and Net Foundation Pressure

To find out bearing capacity and settlement values, the net pressure that soil would carry should be known. The total dead load of bridge is 284.399 kN and live load is 2261.7 kN as mentioned earlier. Load combination is 1.2D+ 1 L used as mentioned on load combination part. Since the structure is symmetrical, soil under the foundations of pylons share this load equally and this vertical load is calculated as follows.

$$Vertical\ Load = \frac{1.2 * 282.7 + 2261.7}{2} = 1300.47\ kN$$

Gross foundation pressure and net foundation pressure can be calculated from the equation below.

$$Gross\ Foundation\ \text{Pressure} = \frac{\text{Vertical Load + Self Weight of Foundation}}{Area}$$

$$Net\ Foundation\ \text{Pressure}\ (q_f) = \frac{Vertical\ \text{load + self weight of foundation} - \text{weight of excavated soil}}{\text{Area}}$$

Self-weight of foundation can be calculated as;

$$\text{Self Weight of Foundation} = \frac{(2400 * 5 * 5 * 2)}{1000} = 120\text{kN}$$

Thus gross foundation pressure is calculated as follows;

$$\text{Gross Foundation Pressure} = \frac{(1300.47 + 120)}{(5 * 5)} = 56.82\ \text{kPa}$$

So, net foundation pressure can be calculated as follows;

$$\text{Net Foundation Presusre} = \frac{(1300.47 + 120 - 2 * 19.5 * 5 * 5)}{(5 * 5)} = 17.9\ \text{kPa}$$

Please note that effects of the eccentric loadings and moments created by those force (if there will be any) will be discussed in the final report.

5.5. Bearing Capacity

Bearing capacity of foundations on clay is calculated from the Skempton's equation below. (Birand, 2011)

$$q_f = c_u N_c + \gamma D$$

N_c values are found from Skempton's Chart (1951)Figure 29.

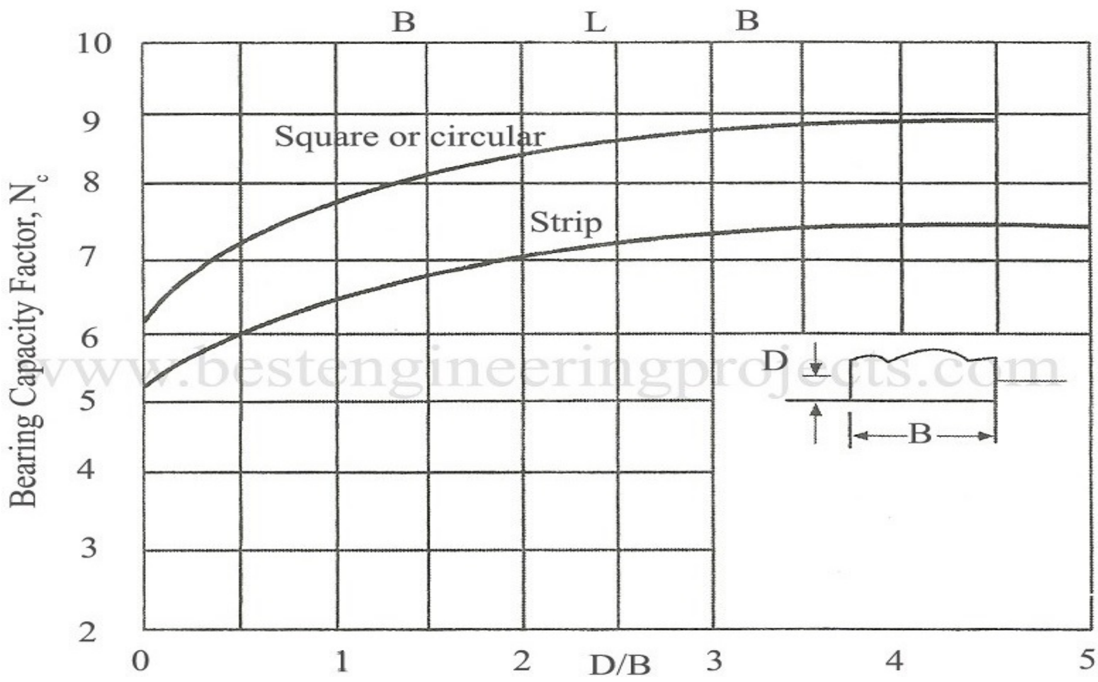

Figure 29. Skempton's N_c Values

$$q_{net} = 1288 \text{ kPa}$$

and,

$$q_{all} = 468.33 \text{ kPa}$$

Since bearing capacity of soil is much larger than net foundation pressure, dimensions of footings and depth of penetration are revised for economic reasons. Using 5x5x2 footings is unnecessary.

5.6. Bearing Capacity and Net Foundation Pressure for Revised Footing

Bearing capacity and net foundation pressure are calculated again for 5x1x1 m footings and results are on table below.

Table 11 : Capacity and Pressures of Revised Footings

Gross Foundation Pressure (kPa)	284.094
Net Foundation Pressure (kPa)	264.594
Bearing Capacity (kPa) q_{net}	1265
Allowable Bearing Capacity(kPa) q_{all}	441.17

Revised version of footings is more reasonable than first assumption.

5.7. Settlement

Settlement is one of the main the criterion in the design of foundations for safety and serviceability. Immediate settlement and consolidation settlement is calculated.

5.7.1. Immediate Settlement

According to Carrier's equation (1978), immediate Settlement of foundations on clay is calculated as follows. This equation presents average vertical displacement under a flexible area carrying a uniform pressure. (Birand, 2011)

$$S_i = \mu_0 * \mu_1 * \mathrm{q} * (\mathrm{B}/\mathrm{E})$$

Coefficients of immediate settlement is found from Figure 30. $\mu 0$ of Immediate Settlement .

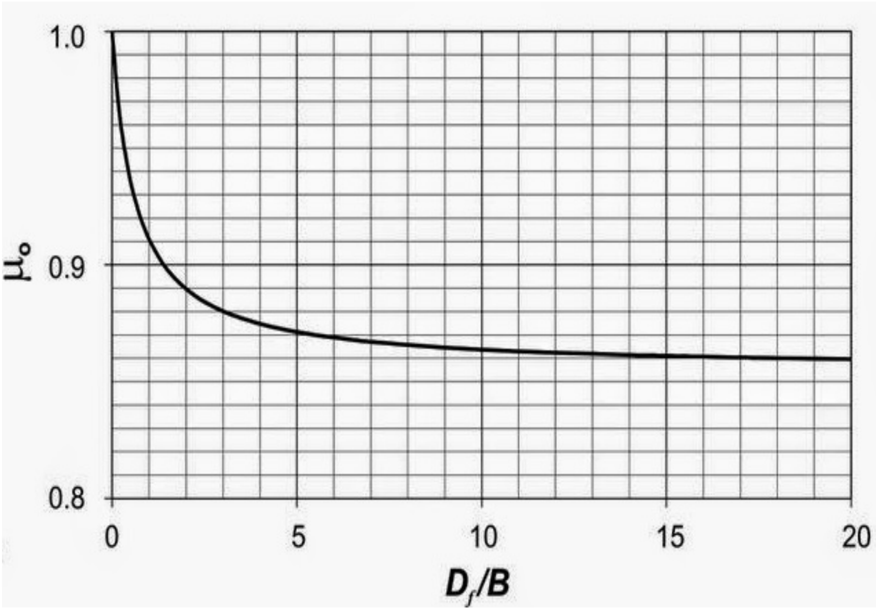

Figure 30. μ_0 of Immediate Settlement

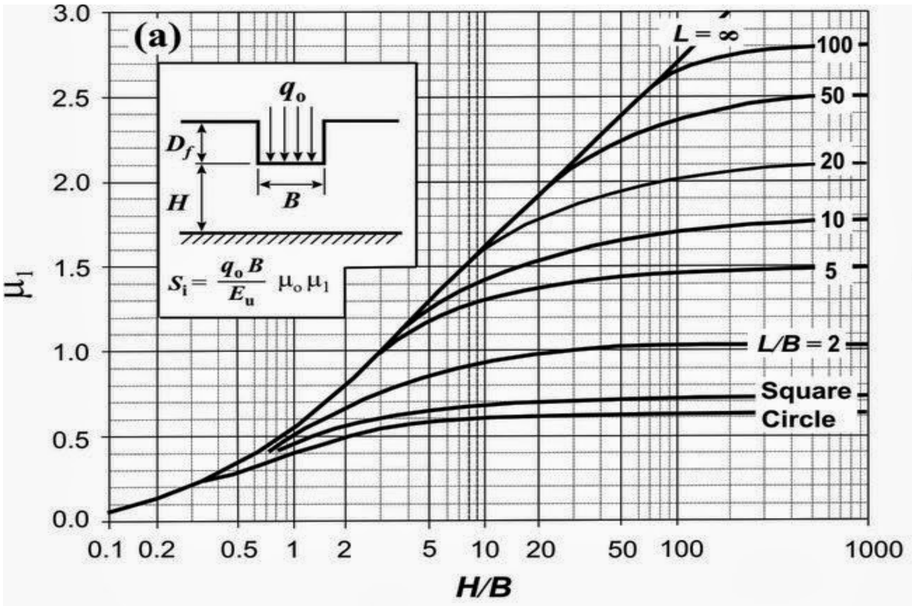

Figure 31. μ_1 of Immediate Settlement

$$\mu o = 0.92$$

and,

$$\mu 1 = 0.3$$

for 5x1x1 footing. Average vertical displacement (S_i) is found as 4.3 mm.

5.7.2. Consolidation Settlement

One-dimensional compression (S_{oed}) of cohesive soils (such as clay) is calculated on the basis of parameters obtained in the oedometer test. Following expression is recommended:

$$S_{oed} = \sum H . m_v . \Delta\sigma'$$

m_v is given on Figure 28 and $\Delta\sigma'$ is calculated from 2V:1H horizontal approach.

By using above approach S_{oed} is found as 34.11 mm. However, this is for one-dimensional. Skempton and Bjerrum observed that actual consolidation settlement of the structures were generally less than the values computed based on oedometer tests. They proposed the following expression: (Birand, 2011)

$$S_c = \mu . S_{oed}$$

μ is taken as 0.5 as a representative value and S_c is found as 17.05 mm. Settlement values are in acceptable limit (New Mexico Tech.). See appendix for further information about consolidation settlement.

5.8. Anchorage

Main ropes of suspension bridge are tied to an anchorage block like in old bridge (see Figure 32: Anchorages of Old Structure). Overturning and sliding is concerned while designing this blocks. Though, calculations of anchorage blocks will be done in the final part.

Figure 32: Anchorages of Old Structure

6. MATERIAL SELECTION

Material selection is done according to the found moments and the forces, in order to have the adequate strength.

- Concrete (Foundation) C30 20m^3 - for foundation
- Steel (Beams + Bottom Steel) S235
- Timber (Upper + Bottom)
- Main Cable (220 kgf/cm^2) 6x36 Werrington-Seale Steel Core with Minimum Rupture Load of 173.14 tonnes $\phi 47^{mm}$
- Hanger Cables (180 kgf/cm^2) 1x6 Monotron Cable with Minimum Rupture Load of 2.4 tonnes $\phi 5^{mm}$

7. COST ESTIMATION

For the rough cost estimation all the volumes of the main equipment have been calculated and the unit prices are found. Unit price of the timber is found to be 507,4 TL/m^3 from the Kayalar Kereste and concrete unit price is found to be 153.4 TL/m^3 from Sebeltas A.S. Cable prices are Copuroglu Steel for main cable 56.64 TL/m and the vertical hanging rope unit price is obtained to be 0.354 TL/m and finally, the steel unit price is found to be 1.7 kg/m^3 LME Steel. Note that most of the unit prices are provided through direct phone calls with the related companies. The price calculations are carried out in Excel Spreadsheet and the results are provided below in tables, and the Total Cost of this project is Estimated to be almost 300000 TL.

Table 12. Cost Estimation of IPN 200 Beams Used in the Project

I Beam					
t_f (m)	**t_w (m)**	**h_w (m)**	**b_f (m)**	**Area (m^2)**	**Length (m)**
0,0113	0,0075	0,1774	0,09	0,0033645	4
Volume (m^3)	**Density of Steel (kg/m^3)**	**Mass (kg)**	**Unit Price (TL/kg)**	**Quantity (#)**	**Total Price (TL)**
0,013458	8050	108,3369	1,7	59	10866,19107

Table 13. Bottom and Upper Timber Volume Calculations

Bottom Timber				
Height (m)	**Width (m)**	**Length (m)**	**Quantity (#)**	**Total Volume (m^3)**
0,15	0,15	90	5	10,125
Upper Timber				
Height (m)	**Width (m)**	**Length (m)**	**Volume (m^3)**	**Total Volume (m^3)**
0,05	3,5	90	15,75	25,875

Table 14. Timber Cost Estimation

Timber	
Unit Price (TL/m^3)	**Total Price (TL)**
507,4	13128,975

Table 15. Bottom Steel Cost Estimation

Bottom Steel			
Height (m)	**Width (m)**	**Length (m)**	**Quantity (#)**
0,025	0,1	90	3
Total Volume (m^3)	**Density of Steel (kg/m^3)**	**Unit Price (TL/kg)**	**Total Price (TL)**
0,675	8050	1,7	3667,78125

Table 16. Pylon Beams Volume Calculation

H Pylon			
Bottom Beams			
Height (m)	**Width (m)**	**Length (m)**	**Volume (m^3)**
0,6	0,5	3	0,9
H Pylon			
Upper Beams			
Height (m)	**Width (m)**	**Length (m)**	**Volume (m^3)**
1,5	0,5	3	2,25

Table 17. Pylon Column Volume Calculations

H Pylons				
Columns				
Height (m)	Width (m)	Length (m)	Quantity (#)	Volume (m^3)
12,5	0,5	0,5	2	6,25

Table 18. Pylon Cost Estimation

H Pylon				
Total				
Quantity (#)	Volume (m^3)	Density of Steel (kg/m^3)	Unit Price (TL/kg)	Total Price (TL)
2	18,8	8050	1,7	257278

Table 19. Main Cable Cost Estimation

Main Cable			
Quantity (#)	Back Length-1 (m)	Back Length-2 (m)	Main Span (m)
4	13,53756	13,53714	90,668
Total Length (m)	Unit Price (TL/m) without KDV	Unit Price (TL/m) with KDV	Total Price (TL)
117,7427	48	56,64	26675,78611

Table 20. Vertical Cable Cost Estimation

Vertical Cables			
Total Length (m)	Unit Price (TL/m) without KDV	Unit Price (TL/m) with KDV	Total Price (TL)
717,64	0,3	0,354	254,04456

Table 21. Concrete used in Foundation Cost Estimation

Concrete		
Volume of Concrete (m^3)	**Unit Price (TL/m^3)**	**Total Price (TL)**
20	153,4	3068

Table 22. Total Cost Estimation of the New Bridge

TOTAL COST (TL)
288262,9919

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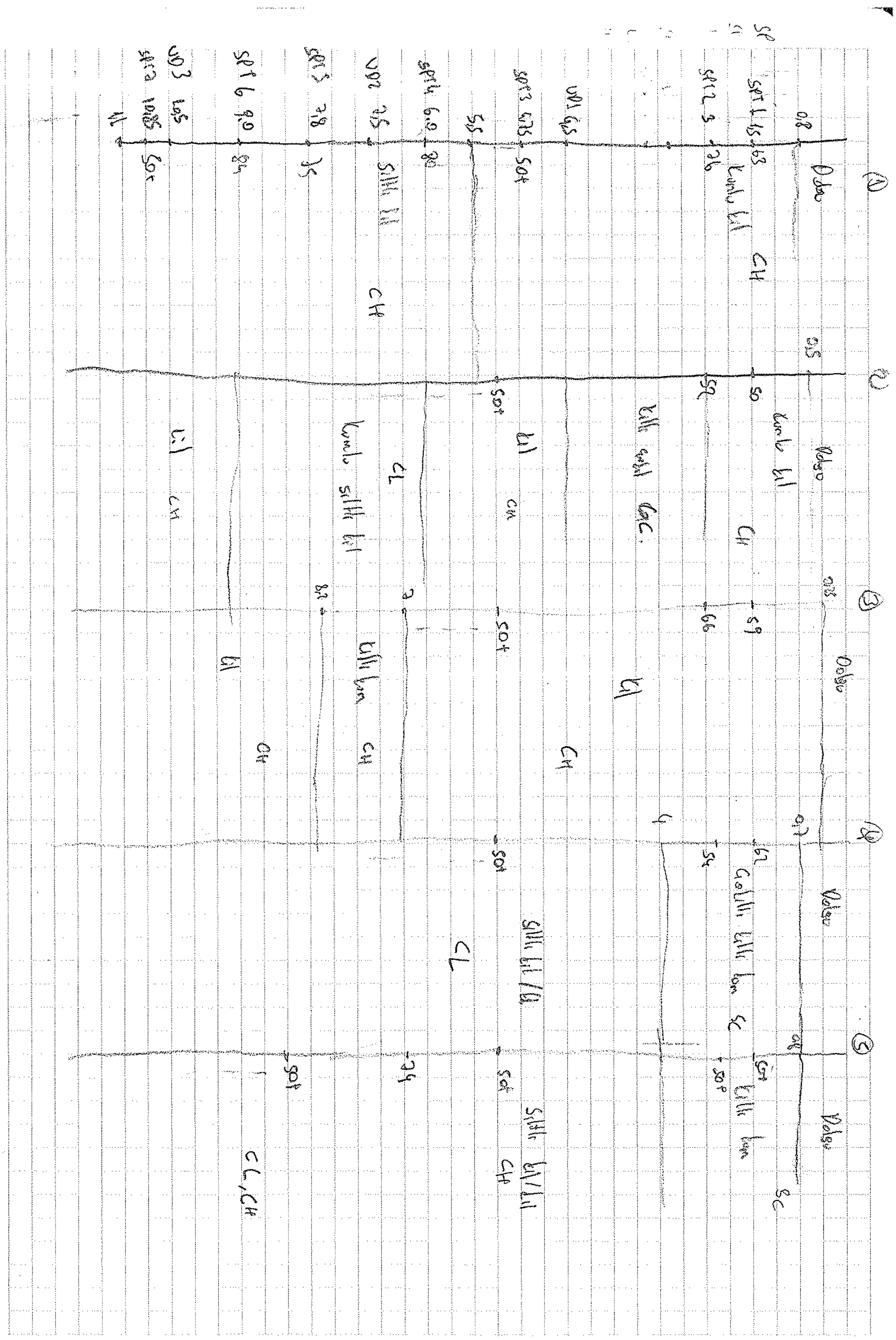
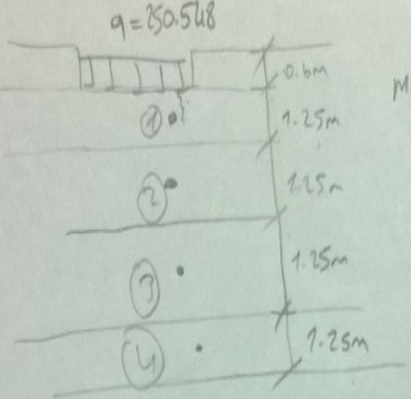
APPENDIX



$S_{oed} = m_v \, H \, \Delta q$

$S_1 = 0.75 \times 10^{-4} \times \frac{1.25}{2} \times \frac{5 \times 264.594 \times 1}{5.625 \times 1.625}$

$S_1 = 6.78 \text{ mm}$



$m_v = 0.75 \times 10^{-4} \, m^2/kN$

1H:2V approximation..

$\Delta q = \frac{B q L}{(B+z)(L+z)}$

$B = 5m$

$L = 1m$

$S_2 = 0.75 \times 10^{-4} \times 1.875 \times \frac{5 \times 264.594 \times 1}{6.875 \times 2.875}$

$S_2 = 9.41 \text{ mm}$

$S_3 = 0.75 \times 10^{-4} \times 3.125 \times \frac{5 \times 264.594 \times 1}{8.125 \times 4.125}$

$S_3 = 9.25 \text{ mm}$

$S_4 = 0.75 \times 10^{-4} \times 4.375 \times \frac{5 \times 264.594 \times 1}{9.375 \times 5.375} =$

$S_4 = 8.61 \text{ mm}$

$S_1 + S_2 + S_3 + S_4 = 34.11 \text{ mm}$

$S_c = \mu \times S_{oed} = \mu_g \times 34.113 = 0.5 \times 34.11$

$= 17.055 \text{ mm}$