Infrastructure Design Project

ENG7142

Bridge Design Report

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About the bridge

Haw Bridge is a bridge across the River Severn on the B4213 road at The Haw, near Tirley. Gloucestershire. It was originally built in the 1824, demolished by a canal barge in 1958 and subsequently rebuilt in its present form in 1961(*Haw Bridge - Roader's Digest: The SABRE Wiki*, 2021). The approximate length of the bridge is 120 meters, which is divided into three spans. The approximate width of the bridge is 11.5 meters with a pedestrian crossing on one side.

Figure 1 Haw Bridge actual view

Figure 2 Footpath of haw bridge

Area Information

The bridge construction area is rural and has 133 houses. There are a total of 287 people living in the area, with an average age of 39. Also, in terms of economic activities, 232 people are active in this region, and the figure 1 can be seen(*Interesting Information for Haw Bridge, Tirley, Gloucester*).

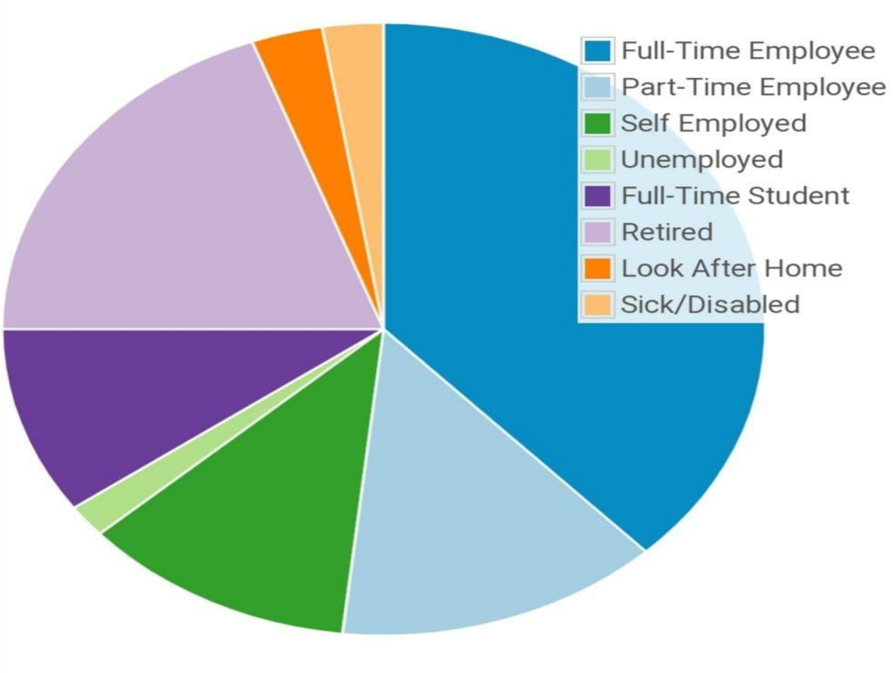


Figure 3 People employed in the area

Water level

From the beginning of the measurement of water heights at the Haw Bridge the height of the river water in 90% of cases has been between 0.53 to 4.50 meters. But in the last 12 months, the results show a relative increase in water level compared to previous years. So that the minimum water has increased to 0.72 meters and the maximum to 5.28 meters. Also in July 2007, the water level in this area reached its highest level of 6.23 meters(*River Severn at Haw Bridge :: the UK River Levels Website*, 2021).

This bridge is the main road for many cars from nearby areas such as Evesham, Gloucster, Cheltenham and Ledbury. Although there are adjacent roads to access these areas, as shown in Figure 2 this bridge will provide the closest connection between these cities(*Society Roam*,2021).

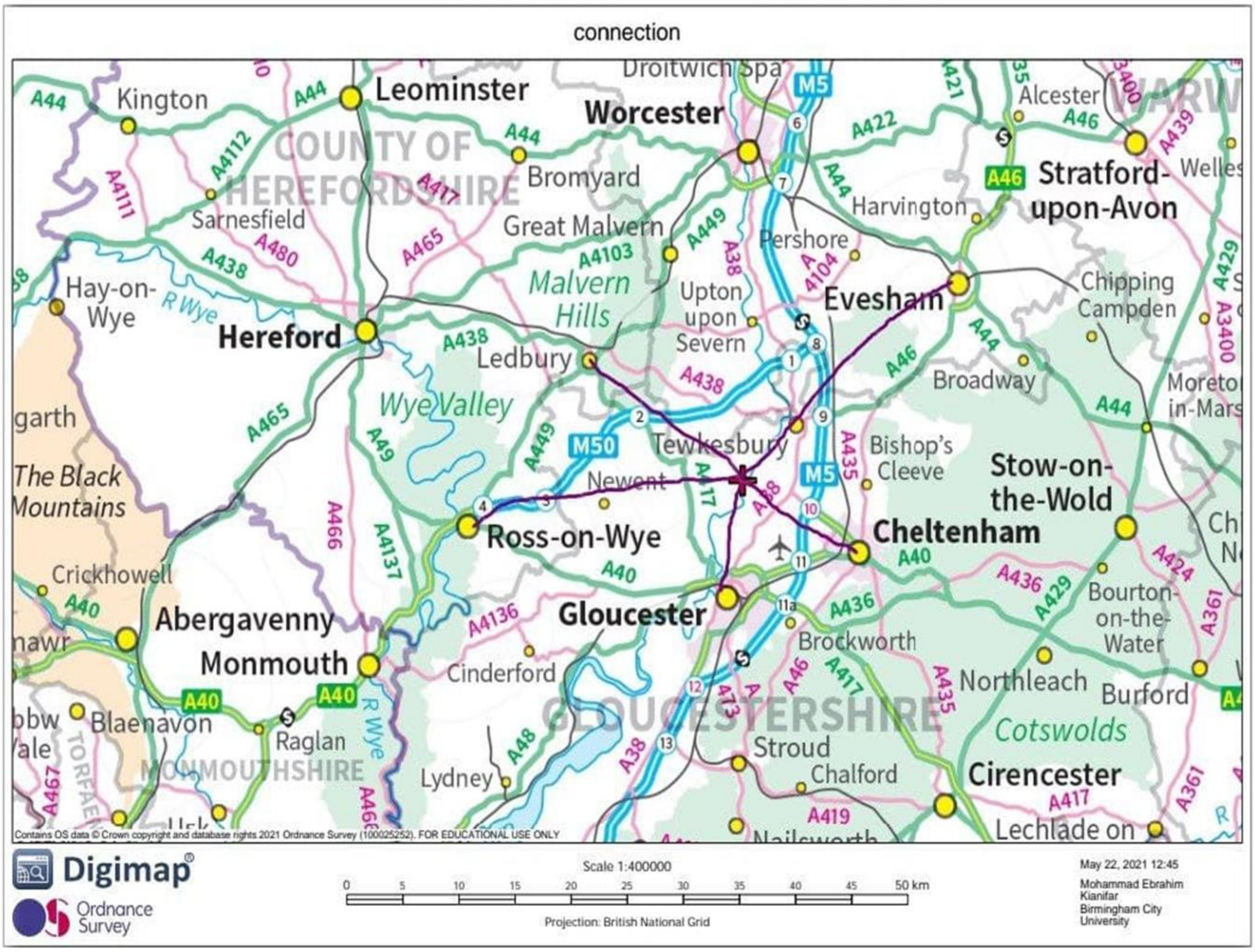


Figure 4 Road map

Land use

On both river sides, the project is bordered by multiple housing, stores, and bars. Coniferous and non-coniferous trees make up the natural vegetation cover, and the water body has rough grassland, marsh reeds, and saltmarsh, as well as other mixed plants**.** The area surrounding the planned bridge is primarily made up of improved grassland, farms, agriculture lands, and freshwater; this neighborhood can be described as rural living. Figure 3 depicts the local land and its many uses.

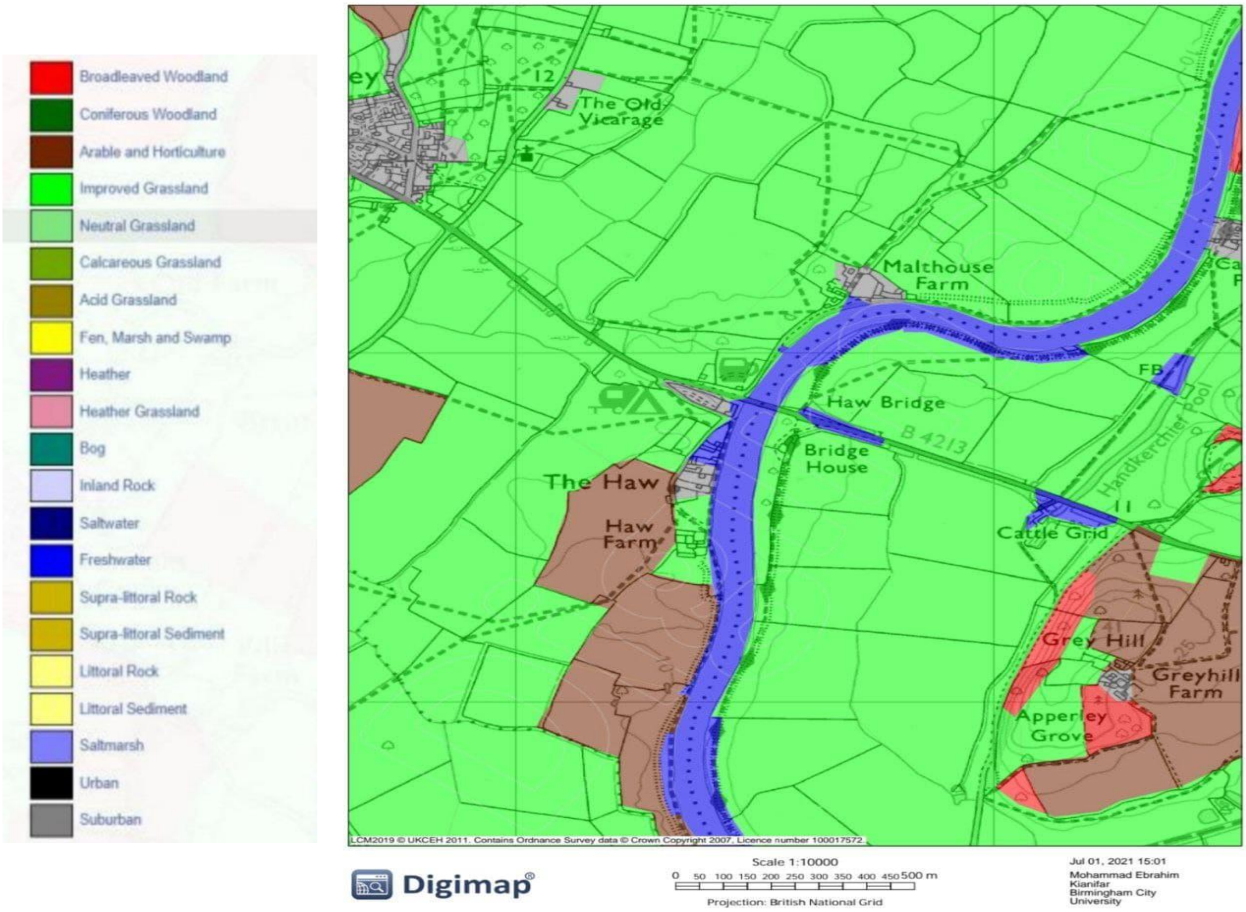


Figure 5 Land use

Economy values of the bridge

The making of a bridge has a significant impact on the economy. Bridges have been responsible for allowing transportation from one place to the other. The transport network can be enhanced by constructing a healthy bridge system. The bridge has an influence on the traffic, business ventures, development of cities and market places and what not. It has been found through research that people generally like to stay in areas where there are large bridges which symbiosis economical and financial stability of a place or country. Rapid communication can take place through these bridges which is why bridges have quite an impact on the economy.

The Haw bridge connects the two major cities of Birmingham and Cardiff and serves as one of the key transportation links between them. The trade of products between both metropolises is enhanced by this bridge, and in terms of traffic volume, it improves the balance on the roadways connecting the two cities. It also makes it easy to travel between the two cities on a small scale. It also makes connecting the two cities with smaller towns like Gloucester and Evesham easier. Furthermore, the majority of indigenous people in the bridge-building area work in the industrial sector, and industrial sites are located in surrounding cities. By lowering the distance between the region's towns and cities, this route will undoubtedly lead to greater economic growth.

Social impact

Bridges allow people to visit places like shopping malls and stores. It allows agencies and companies to trade materials with one another. In some cases, it acts as the symbol of a state or a country which shows that bridges does have an economical aspect to it as well. When a bridge falls or collapses, economic activities come to a halt and can have major consequence on the lives of people and on the environment.

Risk Control Plan

Since this project will be carried out on the river, as well as other crossing projects have numerous obstacles, there is a lot of potential for human and financial dangers in the current project. Many factors should be considered before executing the project, some of which are listed below (*Risk Management in Construction | process of managing risk*, no date).

- To limit the hazards of darkness in the project environment, start the implementation phase in the spring.
- Make use of qualified workers who have health and safety certifications.
- Use of safety equipment suitable for forces such as shoes and helmets.
- Install fences and safety signs in high-risk locations.
- Visitors will be shown health and safety training movies in the project prior to entering the project.
- Use lighting equipment suitable for night work.

Sustainable Development Plan

One of the most significant aspects of the current project is sustainable development, which must be addressed independently in the suggested plan and specifications. However, in summary, the following are some of the most important planning topics:

Material selection

In choosing the material, many points should be considered that their use in the long run and short term have fewer negative effects on the environment. For example, materials should be renewable and packaging should be removed as much as possible to prevent the production of plastic waste. One of the operational plans in this section will be the purchase of cementitious materials and liquids with pallet packaging. In general, the study of the most optimal economic and environmental conditions, the selection of materials and its preparation locations will be done with the help of Life Cycle Assessment (LCA)(Vieira, Calmon and Coelho, 2016), and the amount of pollutant gases emitted in each case will be examined. To do this, Sima Pro software is provided, which will provide us with a good database. Figure 4 shows the inputs and outputs of Sima Pro software. In this project, we will use the following materials:

- Use of slag in concrete to reduce cement consumption and increase the durability of concrete.
- Use of water reducing additives in concrete.
- Construction of non-structural components with the help of concrete containing recycled aggregates**.**
- Use of sealing materials in concrete elements as well as stainless steel for steel parts.

Environment

The execution of the project may have long- and short-term harmful consequences on the region's ecosystem, however precautions have been taken to prevent and mitigate damage.

- Informing the people of the area about the start of the project and its completion.
- During the process, predicting side roads for passing autos.
- Avoidance to carry out activities that cause noise pollution at night in order to protect the residents' comfort.
- Prevent garbage and wastewater from entering the river and green environment around the project.
- Project water purification and reuse.
- Protecting the plant tissue of the area during the project and repairing the damaged parts after the project.
- Separation of renewable and non-recyclable waste during the project.

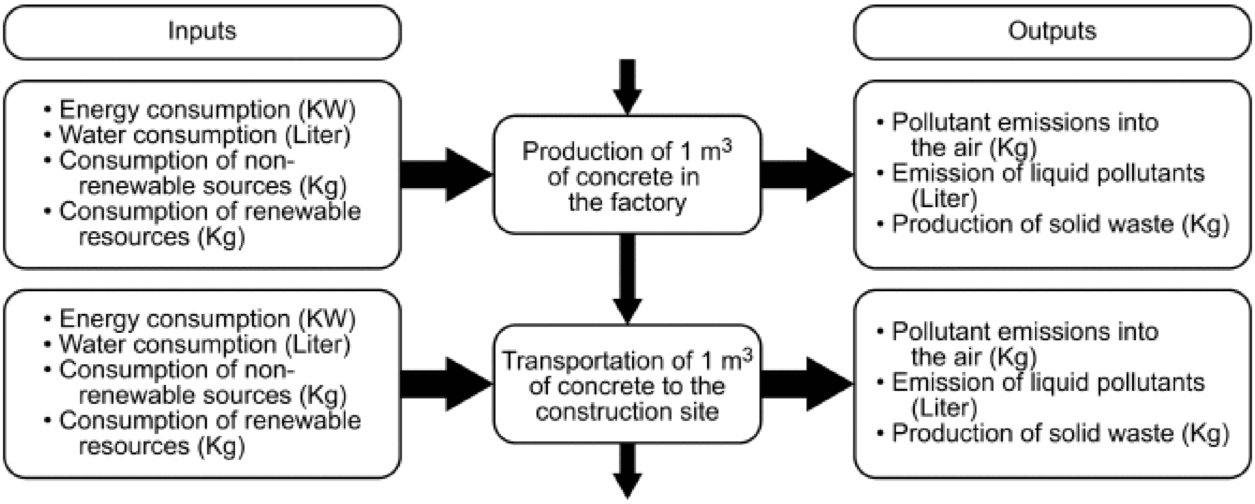


Figure 6 LCA inventory inputs and outputs(Vieira, Calmon and Coelho, 2016).

Material

Steel

Steel is often used as the material for making the girder of the complex structure as these materials have proven to give long results with simple maintenance. Steel is a well-renowned form of iron, which is a mixture of alloys with iron. Various types of steel are available for industrial practice and choosing the right one is done based on the design requirement. As per the loading and types of load that will be applied and data structure will carry, the material for the bridge making is decided. The materialistic properties of steel such as strength as well as ductility play crucial roles in the design process (Yepes *et al.* 2017). The internal properties such as strength as well as ductility of the steel depending on the chemical composition that is put in while manufacturing. The use of steel is done as per the demand on the various components of the bridge elements. While using steel in the bridge elements, the ropes need to have elastic properties as this will carry the elastic loads. To meet this demand, additional tensile strength is to be fed into the steel. Due to its easy-to-use properties, several studies have been conducted and as per the studies, the behavior of the steel is found out.

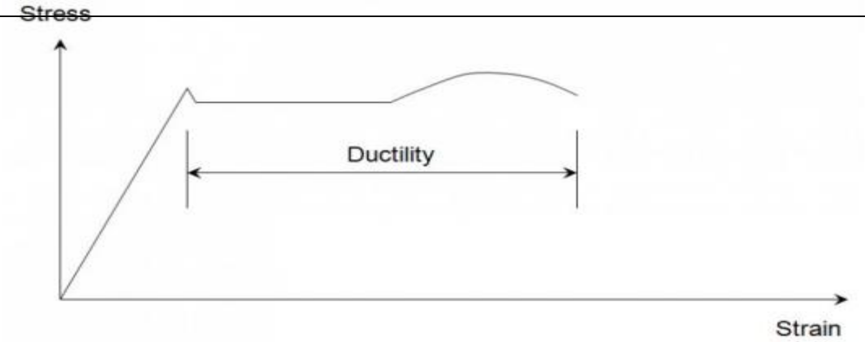


Figure 7 Typical properties of steel

(Source: Yepes *et al*. 2017)

Concrete

The most common structural material for bridges is concrete, and prestressed concrete is commonly used. The advantages of this type of bridge over more conventional designs were studied in the 1940s for use on heavy-duty bridges. The advantages of this type of bridge over more traditional designs were that it was faster to build, more economical, and longer-lasting, and the bridge was less lively. The Adam Viaduct, a railway bridge built in 1946 in the United Kingdom, was one of the first bridges built in this manner. In the United Kingdom, prestressed concrete bridges had increasingly replaced reinforced concrete bridges by the 1960s, with box girders being the most common form.

Prestressing is typically used in short-span bridges with spans of 10 to 40 meters (30 to 130 feet) in the form of precast pre-tensioned girders or planks. Precast-segmental, in-situ balanced-cantilever, and incrementally-launched designs are commonly used for medium-length structures of about 40 to 200 meters (150 to 650 feet). Prestressed concrete deck systems are often used in cable-stayed designs for the longest bridges.

Concrete grades for beams and slab are C32/40 and C40/50 respectively.

Structural concrete mix design

The design of the concrete mixture of this project is based on the ACI-211. As mentioned in the sustainable development program, after designing the concrete mixing plan, it can be examined using LCA software (Vieira, Calmon and Coelho, 2016).

Grading Aggregates

The grading of aggregates is designed based on the modified relations of Fuller-Thomson, the formula used can be seen below. The grading of aggregates for the concrete is designed for self-compacting concrete to provide more freedom of action in the use of reinforcement in elements. As can be seen in the graph..., aggregates smaller than 10 mm have been used in this concrete.

P = 100% * [(d/D)n - (0.075/D)n] / [1-(0.075/D)n]

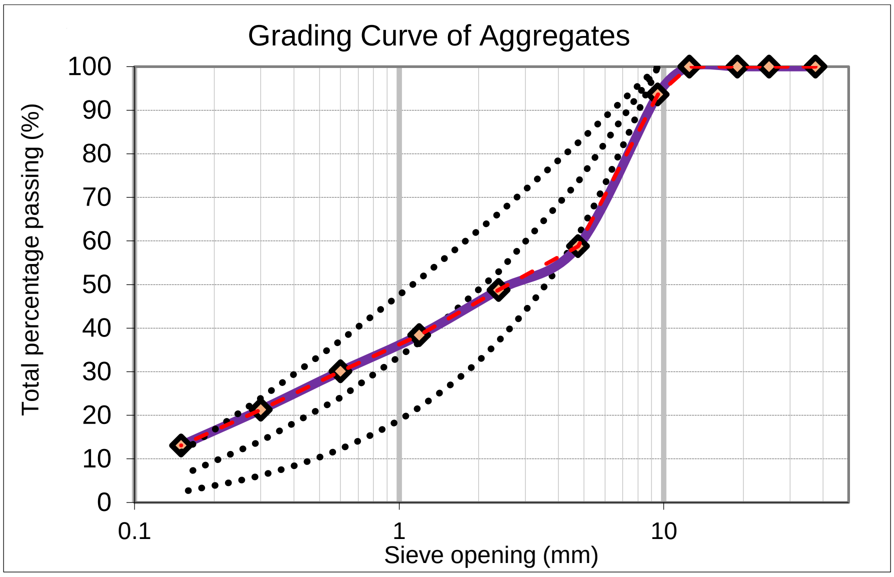


Figure 8 Grading Curve of Aggregates

Mix Design

In table ... the specifications of the concrete mixing plan for one cubic meter are shown. The proposed super lubricant used in the project can be purchased from SIKA company, which adds many capabilities to concrete. This material is designed for self-compacting concrete and can be produced in the UK. As mentioned in the section on sustainable development, slag has been used to increase its durability and help the environment. The optimal amount for the use of this material is 30 to 60%(Hussain, Kaur and Hussain, 2020), which in this project will be 30%. Table… shows the mix design for C32/40 grade and table… illustrates grade C40/50.

Table 1 Mix design for grade C32/40

Mix Design									Other Pozzolans	
w/c	Cement (kg/m3)	Slag (GGBS) Percentage (%)	Slag (GGBS) Weight (kg/m3)	Water (Lit)	Air Content (%)	Aggregates Volume (Lit)				
0.350	280	30	120.00	140.00	2.0	693				
Additives		**Company**	**Type**		**Solid Content**	**Percentage (%)**	**Density**	**Weight kg/m3**	Pozzolan 1	
		Sika SF-2000			45%	1	1.15	4.000	Percentage	Weight
					35%		1.15	0.000	0	0.00
					35%		1.15	0.000	Pozzolan 2	
					35%		1.15	0.000	Percentage	Weight
					35%		1.15	0.000	0	0.00
	Additives (kg)					Concrete Volume (lit)			**Whole Cementitious Materials(kg/m3)**	
	4.00						999		400	

Table 2 Mix design for grade C40/50

Mix Design									Other Pozzolans	
w/c	Cement (kg/m3)	Slag (GGBS) Percentage (%)	Slag (GGBS) Weight (kg/m3)	Water (Lit)	Air Content (%)	Aggregates Volume (Lit)				
0.330	322	30	138.00	151.80	2.0	659				
Additives		**Company**	**Type**		**Solid Content**	**Percentage (%)**	**Density**	**Weight kg/m3**	Pozzolan 1	
		Sika SF-2000			45%	1	1.15	4.600	Percentage	Weight
					35%		1.15	0.000	0	0.00
					35%		1.15	0.000	Pozzolan 2	
					35%		1.15	0.000	Percentage	Weight
					35%		1.15	0.000	0	0.00
	Additives (kg)					Concrete Volume (lit)			**Whole Cementitious Materials(kg/m3)**	
	4.60						999		460	

Figure 9 Bridge Elevation

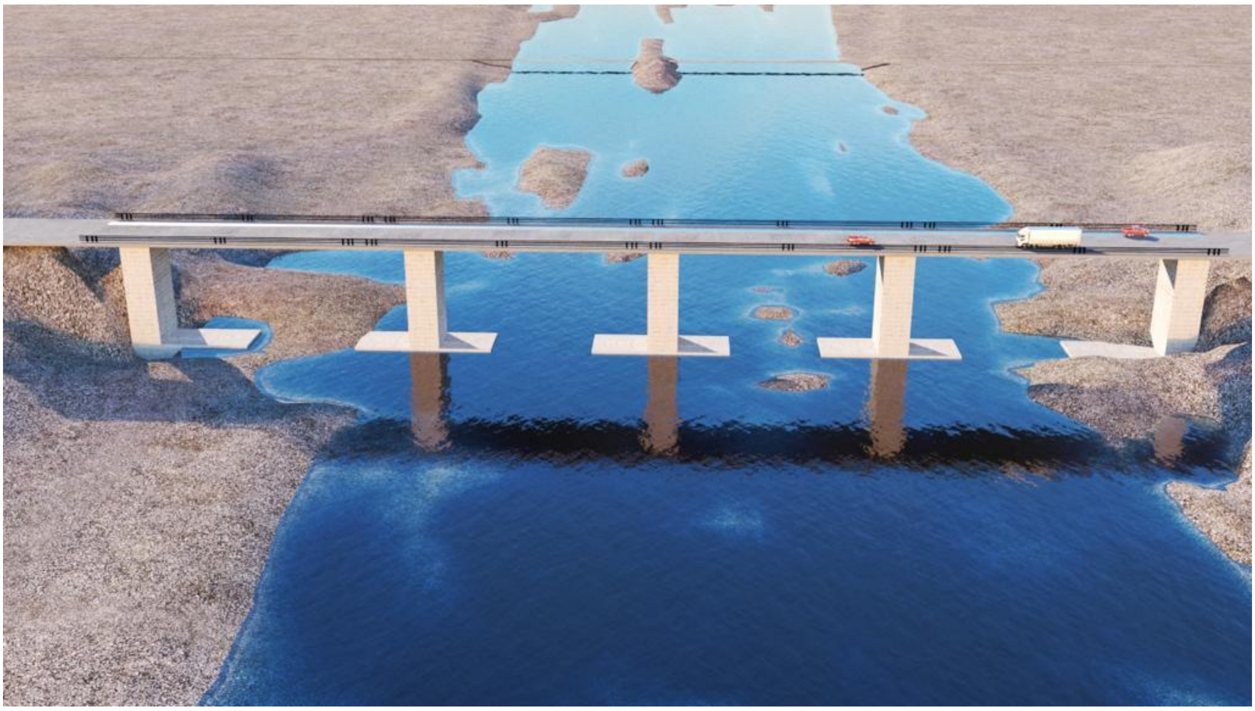


Figure 10 Bridge Elevation Side View

Figure 11 Bridge Elevation Front View

Figure 12 Bridge Elevation Top View

Structural design

Bridge Loading

To start we have taken the relevant information from the final design and outlined this below

Initial Road Data:

Lane Width	-	4 mt + 1 mt Carriage Way (both way)
Span	-	30 mt
Bridge Length	-	120 mt
Surface Thickness	-	100 mm
Slab Thickness	-	150 mm
Beam	-	Y5 Beam (Pre-stressed Beam)
		Beam to Beam Distance 1mt

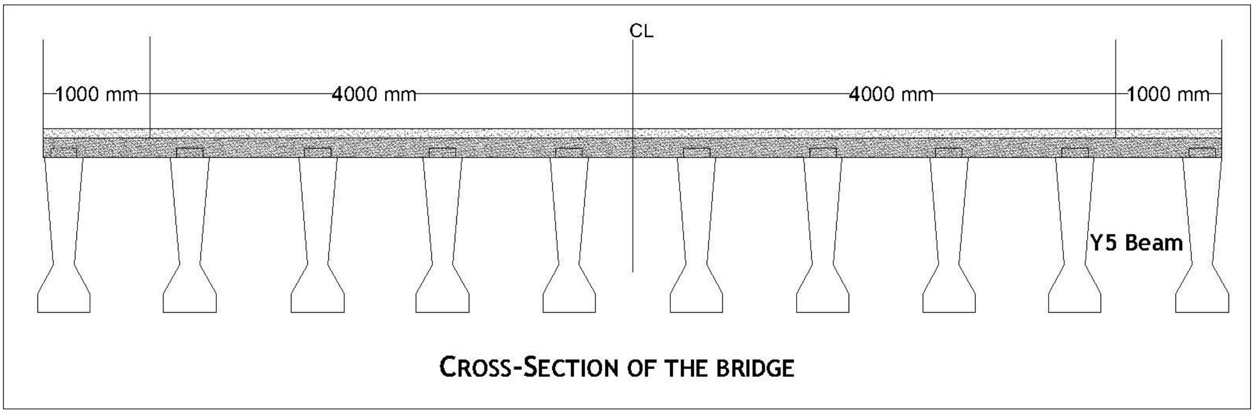


Figure 13 Cross-Section of the bridge

Dead Load: -

$$\text{Slab Dead Load} = \text{Slab Self Weight X Slab Height X Beam to Beam Distance}$$

$$= 25 \text{ X } 0.150 \text{ X } 1$$

$$= 3.75 \text{ KN/mt}$$

Surface Dead Load = Surface Self Weight X Surface Height X Beam to Beam Distance

= 24 X 0.100 X 1

= 2.4 KN/mt

$$Y5\ Beam\ Self\ Weight\ =\ 11.23\ KN/mt (As\ per\ MACRETE - \ standard\ Pre - stressed\ Beam\ Section)$$

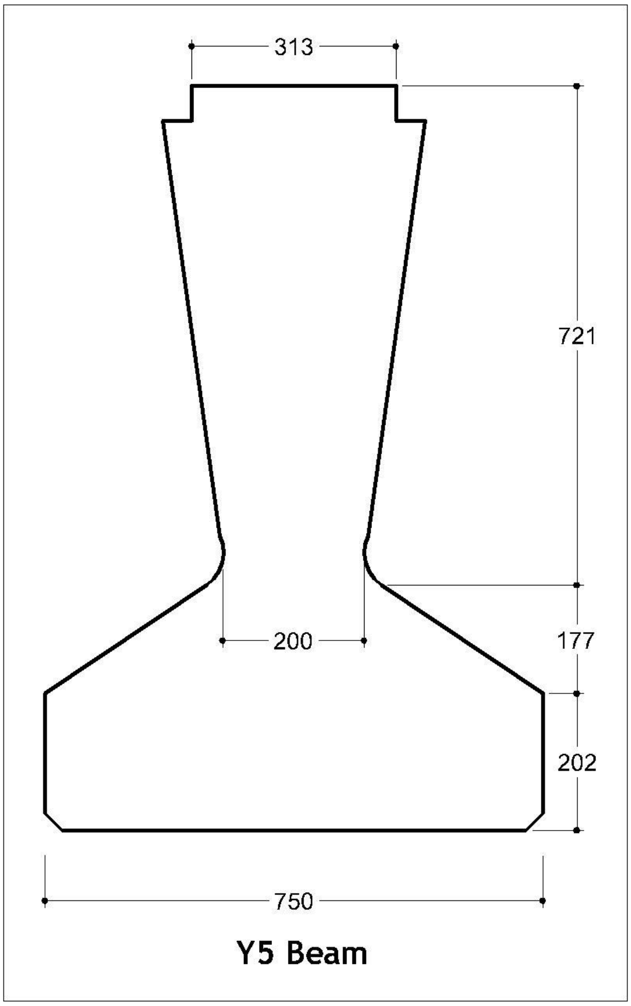


Figure 14 Section of Y5 Beam

Live Load: -

$$As\ the\ Bridge\ is\ Multi\ Span, 2\ HA\ UDL's\ will\ be\ calculated$$

$$HA\ UDL\ =\ (336\ X\ (\frac{1}{L})^{0.67})bt \quad (L\ =\ Span\ Length)$$

$$HA\ UDL\ =\ 34.41\ KN/mt$$

$$HA\ UDL\ on\ Each\ Beam\ =\ 34.41/4\ =\ 8.60\ KN/mt$$

$$AND$$

$$HA\ UDL\ =\ (336\ X\ (\frac{1}{L})^{0.67})bt \quad (L\ =\ Span\ Length)$$

$$HA\ UDL\ =\ 13.59\ KN/mt$$

$$HA\ UDL\ on\ Each\ Beam\ =\ 13.59/4\ =\ 3.40\ KN/mt$$

$$Lane\ Factor\ \beta\ As\ Per\ BS\ 5400, Part - 2\ and\ Table\ 14$$

$$\beta =\ 0.0137\ X\ (b_L(40 - L) +\ 3.65\ X\ (L - 20)$$

$$\beta = 1.05$$

$$Final\ HA\ UDL\ =\ 8.60\ X\ 1.05\ =\ 9.03\ KN/mt$$

$$AND$$

$$=\ 3.40\ X\ 1.05\ = 3.57\ KN/mt$$

$$KEL\ =\ \frac{\beta\ X\ 120}{b_L}\ KN$$

$$=\ \frac{1.05\ X\ 120}{4}\ KN$$

$$=\ 31.5\ KN$$

$$Principal\ Road$$

$$HB\ Load\ per\ axle\ for\ ULS\ =\ 37.5\ X\ 10\ =\ 375KN$$

$$25 - unit\ HB\ to\ be\ consider\ at\ SLS\ for\ Load\ Combination\ 1\ only\ (BS\ 5400 - 4, cl. 4.2.2)$$

$$=\ 25\ X\ 10\ =\ 250KN$$

Load Factor:

Concrete Grades:

Beam: C40/50:

f_{cu} = 50 N/mm^2

f_{ci} = 40 N/mm^2

Slab: C32/40:

f_{cu} = 40 N/mm^2

f_{ci} = 32 N/mm^2

Table 3 Load Factor

Load Type		Limit State	Coefficients (γ_{fL}) for Load Combinations	
			1	3
Dead	Concrete	SLS	1.00	1.00
		ULS	1.15	1.15
Superimposed Dead	Deck Surfacing	SLS	1.20	1.20
		ULS	1.75	1.75
Temperature Difference		SLS	-	0.8
		ULS	-	1.00
Highway Bridges Live Load	HA	SLS	1.20	1.00
		ULS	1.50	1.25
	HB	SLS	1.10	1.00
		ULS	1.30	1.10

Table 4 Section Properties:

Property	Beam Section	Composite (Beam+Slab) Section
A (mm^2)	449.22×10^3	599.22×10^3
Centroid (mm)	456	623
I (mm^4)	52.905×10^9	103.515×10^9
Z @ Level 1 (mm^3)	116.020×10^6	166.156×10^6
Z @ Level 2 (mm^3)	89.066×10^6	242.424×10^6
Z @ Level 3 (mm^3)	-	179.402×10^6

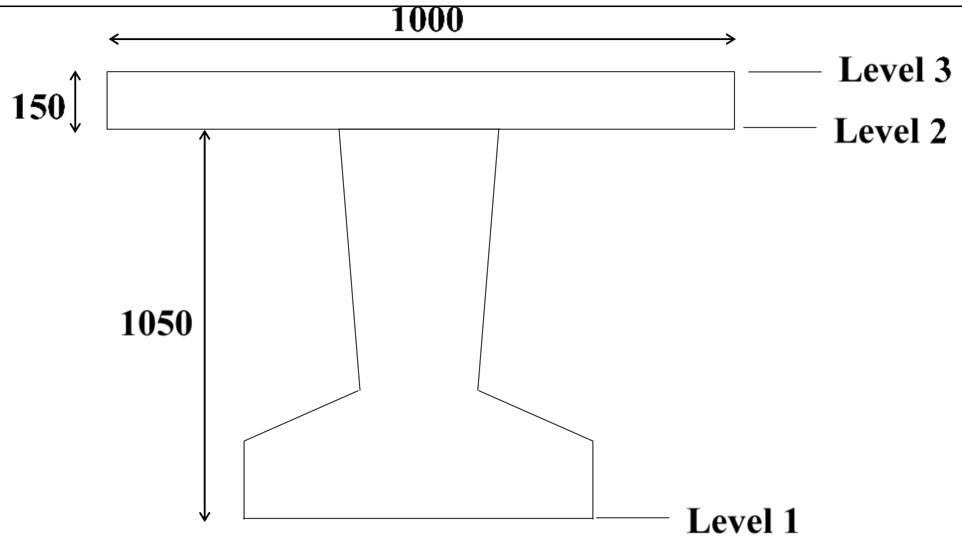
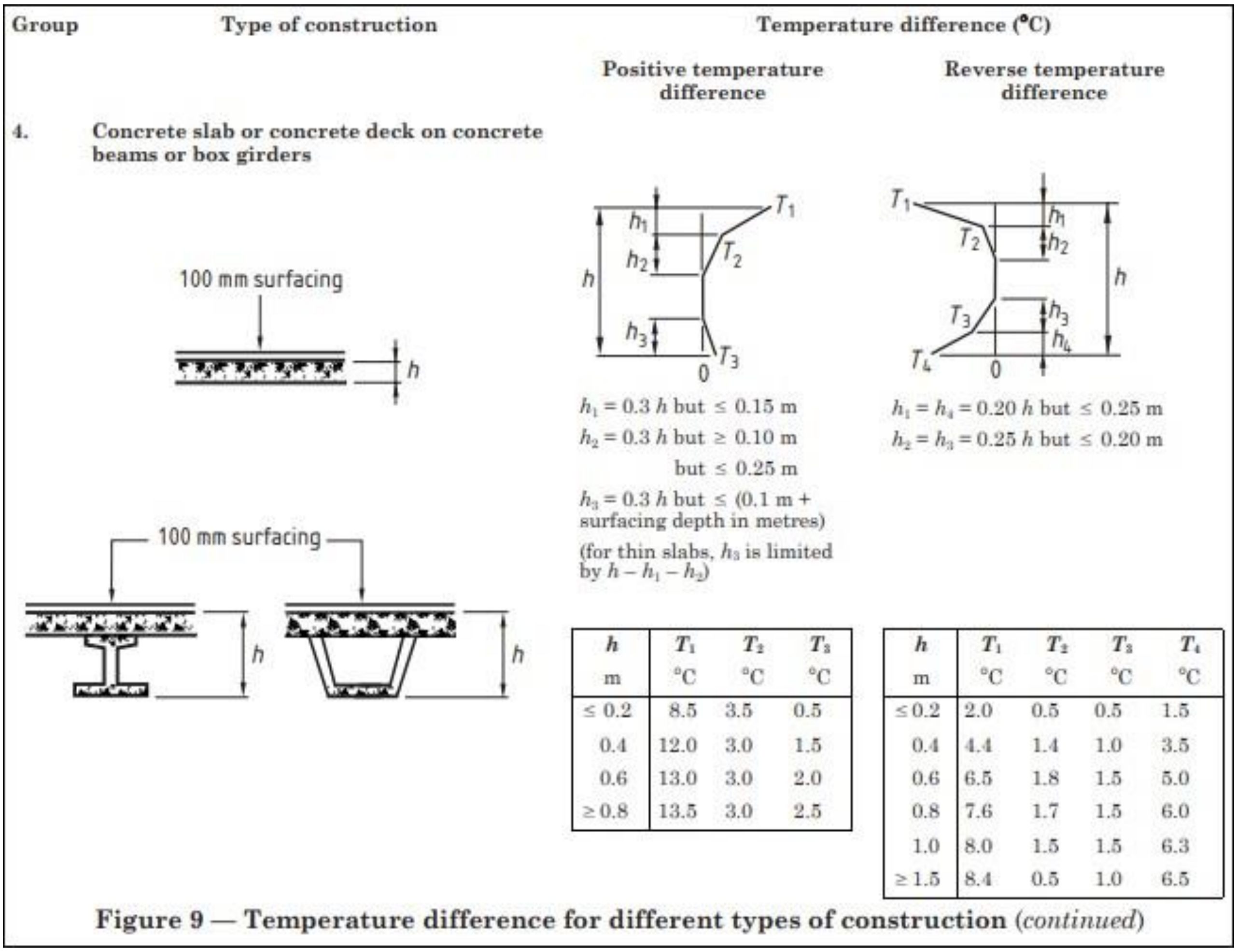


Figure 15 Section of Y5 Beam



Group	Type of construction	Temperature difference (°C)	
		Positive temperature difference	Reverse temperature difference
4.	Concrete slab or concrete deck on concrete beams or box girders		

Positive temperature difference:

$h_1 = 0.3\ h$ but ≤ 0.15 m

$h_2 = 0.3\ h$ but ≥ 0.10 m but ≤ 0.25 m

$h_3 = 0.3\ h$ but $\leq$ (0.1 m + surfacing depth in metres)

(for thin slabs, h_3 is limited by $h - h_1 - h_2$)

Reverse temperature difference:

$h_1 = h_4 = 0.20\ h$ but ≤ 0.25 m

$h_2 = h_3 = 0.25\ h$ but ≤ 0.20 m

h m	T_1 °C	T_2 °C	T_3 °C
≤ 0.2	8.5	3.5	0.5
0.4	12.0	3.0	1.5
0.6	13.0	3.0	2.0
≥ 0.8	13.5	3.0	2.5

h m	T_1 °C	T_2 °C	T_3 °C	T_4 °C
≤ 0.2	2.0	0.5	0.5	1.5
0.4	4.4	1.4	1.0	3.5
0.6	6.5	1.8	1.5	5.0
0.8	7.6	1.7	1.5	6.0
1.0	8.0	1.5	1.5	6.3
≥ 1.5	8.4	0.5	1.0	6.5

Figure 9 — Temperature difference for different types of construction (*continued*)

Figure 16 Temp different for different types of construction

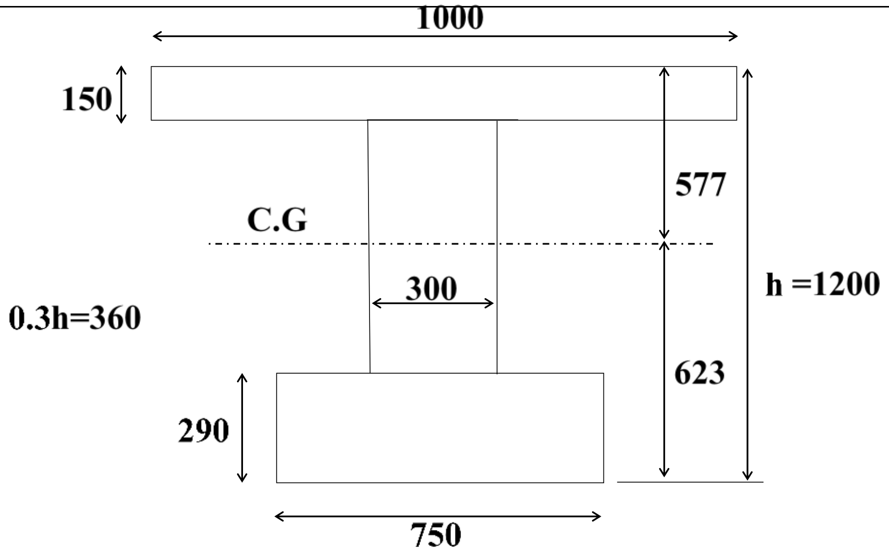


Figure 17 Y5 Section with Dimensions

$$Cl.5.4.6\ state\ the\ coefficient\ of\ thermal\ expansion\ =\ 12\ X\ 10^{-6}\ per\ ^\circ C$$

$$BS\ 5400-4\ TABLE\ 3, E_c\ =\ 34KN/mm^2\ For\ f_{cu}\ =\ 50\ N/mm^2$$

$$Restrained\ Stress\ due\ to\ temperature\ per\ ^\circ C\ =\ 34\ \ X\ \ 10^3\ \ X\ \ 12\ X10^{-6}$$

$$=\ 0.408\ N/mm^2$$

Positive Temperature stresses:

shear stress = Generlised width X Temperature X Height of Section

This must be done for each shown region:

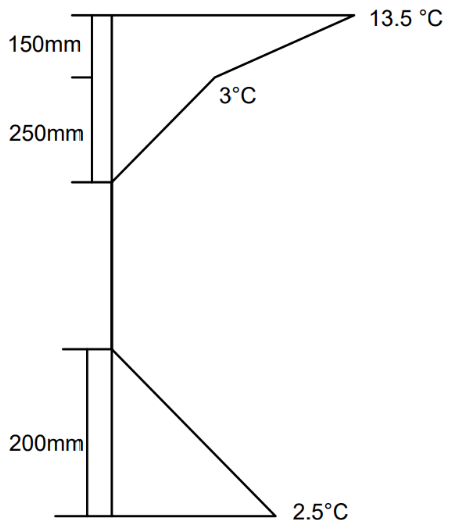


Figure 18 Positive Temperature Diagram

$$shear\ stress\ =\ Generlised\ width\ X\ Temperature\ X\ Height\ of\ Section$$

$$Positive\ Temprature\ \ at\ TOP\ (Region\ 1)\ =\ 0.408\ \times\ 1000\ \times\ 150\ \times\ \ 5.25\ \times\ 10-3$$

$$=\ 321.300\ KN$$

$$Positive\ Temprature\ at\ TOP\ (Region\ 2)\ =\ \ 0.408\ \times\ 1000\ \times\ 150\ \times\ 3.0\ \ \times\ 10-3$$

$$=\ 183.600KN$$

$$Positive\ Temprature\ \ at\ MIDDLE\ (Region\ 3)\ =\ \ 0.408\ \times\ 300\ x\ 250\ x1.5\ \times\ 10-3$$

$$=\ 45.900KN$$

$$Positive\ Temprature\ at\ BOTTOM\ (Region\ 4)\ =\ \ 0.408\ \times\ 750\ x\ 200\ x\ 1.25\ \times\ 10-3$$

$$=\ 76.500KN$$

$$Total\ Force\ F\ to\ restrain\ temperature\ strain\ =\ 627.30\ KN$$

$$Momentstress =\ DistancefromNuetralAxis \times\ shearstress \times\ restrainedstress$$

$$Positive\ Temprature\ Moment\ at\ Region\ 1(Above\ from\ Neutral\ axis)$$

$$=\ \ 0.408\ \times\ 1000\ \times\ 150\ \times\ \ 5.25 \times\ 10-6x\ 527$$

$$=\ 169.3251\ KN$$

$$Positive\ Temprature\ Moment\ at\ Region\ 1(Above\ from\ Neutral\ axis)$$

$$=\ \ 0.408\ \times\ 1000\ \times\ 150\ \times\ \ 3.0 \times\ 10-6\ x\ 502$$

$$=\ 92.1672\ KN$$

$$Positive\ Temprature\ Moment\ at\ Region\ 1(Above\ from\ Neutral\ axis)$$

$$=\ \ 0.408\ \times 300\ \times 250\ \times\ \ 1.5 \times\ 10-6\ x\ 344$$

$$=\ 15.7896\ KN$$

$$Positive\ Temprature\ Moment\ at\ Region\ 1(Above\ from\ Neutral\ axis)$$

$$=\ \ 0.408\ \times 750\ \times 200\ \times\ \ 1.25\ \times\ 10-6\ x\ \ -556$$

$$=\ -42.534\ KN$$

$$Moment\ M\ about\ centroid\ of\ section\ to\ restrain\ curvature\ due\ to\ temperature\ strain\ \ = 234.8KN$$

$$Totalstress =\ Restrainedstress +\ \ Axialforce + Bendingmoment$$

$$Restrainedstressat = Restrainedstressesduetotemperatureper°C \times T°C$$

$$Axial\ Force\ =\ Force\ /\ Area$$

$$Bending\ Moment\ =\ Moment\ /\ Section\ Modulus$$

Table 5 Positive Stress Result:

Restrained stresses	Axial Force	Bending	Stress due to +ve Temp.
1.02	-1.04686	1.4131298	1.38627
0	-1.04686	0.9547800	0.09208
0	-1.04686	-0.5148973	-1.56176
1.224	-1.04686	-0.9685509	-0.79141
5.508	-1.04686	-1.3087925	3.15234

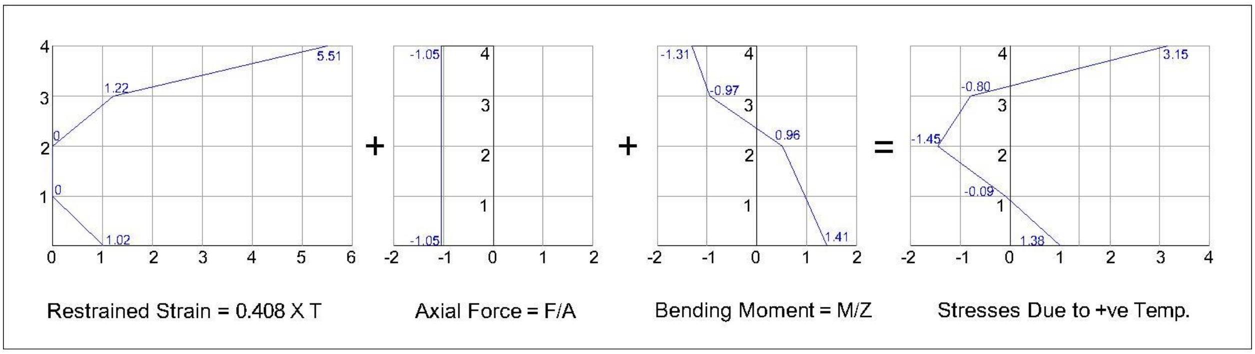


Figure 19 Positive Stress Diagram

Negative Temperature Stresses:

shear stress = Generlised width X Temperature X Height of Section

This must be done for each shown region**:**

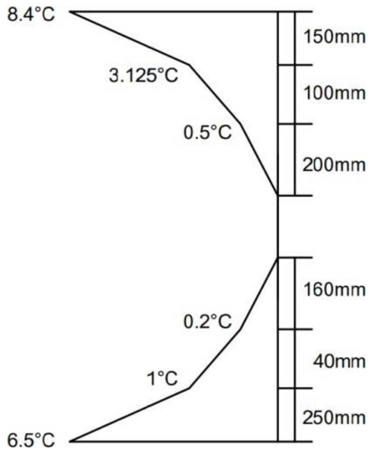


Figure 20 Negative Temperature Diagram

$$Negative\,Temprature\;\;at\,TOP\,(Region\,1) \;=\; -0.408\,\times\,1000\,\times\,150\,\times\,3.6\,\times\,10-3$$

$$=\;-220.32\,KN$$

$$Negative\,Temprature\,at\,TOP\,(Region\,2) \;=\; 0.408\,\times\,1000\,\times\,150\,\times\,2.3\,\times\,10-3$$

$$= -140.76\,KN$$

$$Negative\ Temprature\ \ at\ MIDDLE\ (Region\ 3)\ =\ -0.408\ \times\ 300\,x\,90\,x\,0.9\ \times\ 10-3$$

$$=\ -9.9144\,KN$$

$$Negative\ Temprature\ \ at\ MIDDLE\ (Region\ 4)\ =\ -0.408\ \times\ 300\,x\,90\,x\,1.35\ \times\ 10-3$$

$$=\ -14.8716\,KN$$

$$Negative\ Temprature\ \ at\ MIDDLE\ (Region\ 5)\ =\ -0.408\ \times\ 300\,x\,200\,x\,0.45\ \times\ 10-3$$

$$=\ -11.016\,KN$$

$$Negative\ Temprature\ \ at\ MIDDLE\ (Region\ 6)\ =\ -0.408\ \times\ 300\,x\,150\,x\,0.45\ \times\ 10-3$$

$$=\ -8.262\,KN$$

$$Negative\ Temprature\ at\ BOTTOM\ (Region\ 7)\ =\ -0.408\ \times\ 750\,x\,50\,x\,0.9\ \times\ 10-3$$

$$=\ -13.77\,KN$$

$$Negative\ Temprature\ at\ BOTTOM\ (Region\ 8)\ =\ -0.408\ \times\ 750\,x\,50\,x\,0.15\ \times\ 10-3$$

$$=\ -2.295\,KN$$

$$Negative\ Temprature\ at\ BOTTOM\ (Region\ 9)\ =\ -0.408\ \times\ 750\,x\,240\,x\,1.2\ \times\ 10-3$$

$$=\ -88.128\,KN$$

$$Negative\ Temprature\ at\ BOTTOM\ (Region\ 10)\ =\ -0.408\ \times\ 750\,x\,240\,x\,2.6\ \times\ 10-3$$

$$=\ -190.944\,KN$$

$$Total\ Force\ F\ to\ restrain\ temperature\ strain\ =\ -700.281\ KN$$

$$Momentstress =\ DistancefromNuetralAxis \times\ shearstress \times\ restrainedstress$$

$$Negative\ Temprature\ \ Moment\ at\ Region\ 1(Above\ from\ Neutral\ axis)$$

$$=\ \ 220.32\,X\,502 \times\ 10-3 =\ -110.601\,KN$$

$$Negative\ Temprature\ Moment\ at\ Region\ 2(Above\ from\ Neutral\ axis)$$

$$=\ \ -140.76\,X\,527 \times\ 10-3 =\ -74.1805\,KN$$

$$Negative\ Temprature\ \ Moment\ at\ Region\ 3(Above\ from\ Neutral\ axis)$$

$$=\ \ -9.9144\,X\,382 \times\ 10-3 =\ -3.7873\ KN$$

$$Negative\ Temprature\ \ Moment\ at\ Region\ 4(Above\ from\ Neutral\ axis)$$

$$=\ -14.8716\ KN\ X\,397 \times\ 10-3\ =\ -5.90403\,KN$$

$$Negative\ Temprature\ \ Moment\ at\ Region\ 5(Above\ from\ Neutral\ axis)$$

$$=\ -11.016\ KN\ X\,270 \times\ 10-3\ =\ -2.97432\,KN$$

$$Negative\ Temprature\ \ Moment\ at\ Region\ 6(Above\ from\ Neutral\ axis)$$

$$=\ -8.262\ KN\ X\,283 \times\ 10-3\ =\ -2.33815\,KN$$

$$Negative\ Temprature\ Moment\ at\ Region\ 7(Below\ from\ Neutral\ axis)$$

$$=\ -13.77\ KN\ X\ \ -358 \times\ 10-3\ =\ 4.92966\ KN$$

$$Negative\ Temprature\ Moment\ at\ Region\ 8(Below\ from\ Neutral\ axis)$$

$$=\ -2.295\ KN\ X\ \ -366 \times\ 10-3 =\ 0.83997\,KN$$

$$Negative\ Temprature\ Moment\ at\ Region\ 9(Below\ from\ Neutral\ axis)$$

$$= -88.128\ KN\ X\ -503 \times\ 10-3\ =\ 44.32838\ KN$$

$$Negative\ Temprature\ Moment\ at\ Region\ 10(Below\ from\ Neutral\ axis)$$

$$= -190.944\ KNX\ -543 \times\ 10-3\ =\ 103.6826\ KN$$

$$Moment\ M\ about\ centroid\ of\ section\ to\ restrain\ curvature\ due\ to\ temperature\ strain\ = -46.0043\ KN$$

$$Totalstress = Restrainedstress + Axialforce + Bendingmoment$$

$$Restrainedstressat = Restrainedstressesduetotemperatureper℃ \times T℃$$

$$Axial\ Force\ =\ Force\ /\ Area$$

$$Bending\ Moment\ =\ Moment\ /\ Section\ Modulus$$

Table 6 Negative Stress Result:

Restrained stresses	Axial Force	Bending	Stress due to +ve Temp.
-3.4272	1.16854	0.248730	-2.00982
-1.275	1.16854	0.248730	0.142384
-0.204	1.16854	0.168054	1.132708
-0.0816	1.16854	-0.170478	0.916576
-0.408	1.16854	-0.090629	0.670025
-2.652	1.16854	-0.230365	-1.71371

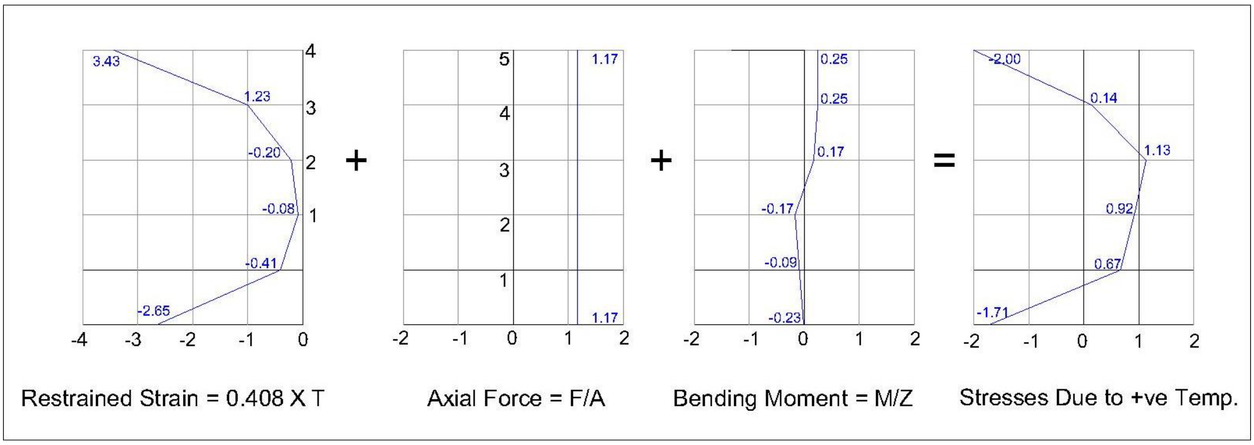


Figure 21 Negative Stress Diagram

Differential Shrinkage Stress:

$BS\ 5400-4, cl. 7.4.3.4, use\ cl. 6.7.2.4, Table\ 29: Total\ shrinkage\ of\ in-situ\ concrete\ =\ 300\ \times\ 10-6$

$Assume\ that\ 2$
$/3\ of\ the\ total\ shrinkage\ of\ the\ precast\ concrete\ takes\ place\ before\ the\ deck\ slab\ is\ cast\ and\ that\ the\ residu$
$\times\ 10-6, hence\ the\ differential\ shrinkage\ is\ 200\ \times\ 10-6.$

$$Cl. 7.4.3.5\ Force\ to\ restrain\ differential\ shrinkage: F\ =\ -\varepsilon diff\ \times\ Ecf\ \times\ Acf\ \times\ \varphi$$

$$= -200 \times 10 - 6 \times 34 \times 1000 \times 150 \times 0.43$$

$$= -439\ kN$$

$$Eccentricity\ acent\ = 502mm$$

$$Restraint\ moment\ Mcs\ = -439 \times 0.502 = -220.4\ kN.m$$

Table 7 Differential Shrinkage Stress:

Restrained stresses	Axial Force	Bending	Stress due to shrinkage
-2.924	0.7319515	1.227299	-0.9647495
-2.924	0.7319515	0.908243	-1.2838055
0	0.7319515	0.908243	1.6401945
0	0.7319515	-1.322283	-0.5903315

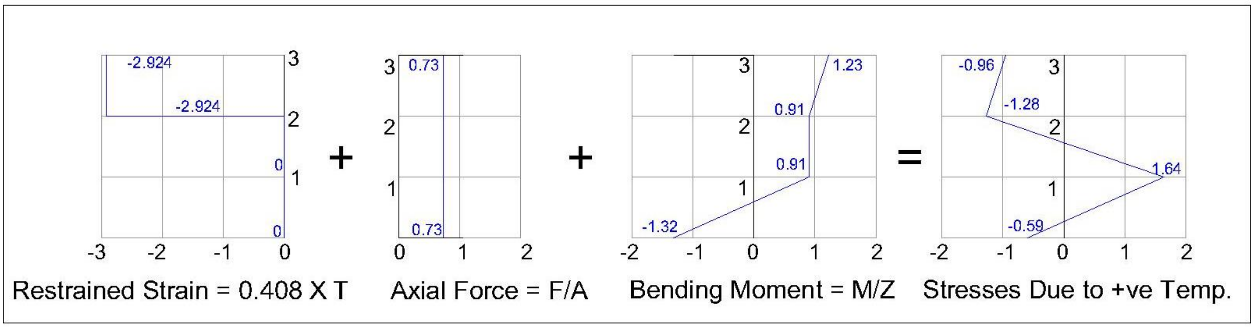


Figure 22 Shrinkage Stress Diagram

Calculation of Maximum Moment and maximum Shear

We used beam analysis and moving load analysis spreadsheet for below calculation:

Table 8 Moment and Shear:

Moments	
Dead Load	1444.5
Super imposed dead load	231.429
HA load	496.125
HB load	1662.071
Shear	
Dead Load	272.85
Super imposed dead load	43.7143
HA load	85.8375
HB load	335.326

Table 9 Moment Combinations:

Moment	**Combination 1**	**Combination 3**
Dead Load (SLS)	1444.5	1444.5
(ULS)	1661.175	1661.175
Super imposed dead		
load (SLS)	277.7148	277.7148
(ULS)	405.00075	405.00075
HA load (SLS)	595.35	
(ULS)	**744.1875**	620.15625
HB load (SLS)	1828.2781	**1662.071**
(ULS)	2160.6923	1828.2781

Table 10 Shear Combinations:

Shear	**Combination 1**	**Combination 3**
Dead Load (ULS)	313.7775	313.7775
Super imposed dead load (ULS)	76.500025	76.500025
HA load (ULS)	128.75625	107.296875
HB load (ULS)	435.9238	368.8586

Table 11 Load Configuration:

Load configuration		
Moments	**Combination 1**	**Combination 3**
Super + HA(SLS)	873.0648	773.8398
Super + HB(SLS)	2105.9929	1939.7858
Super + HA(ULS)	1149.18825	1025.157
Super + HB(ULS)	2565.69305	2233.27885
Shear		
Dead + Super + HA(ULS)	519.033775	497.5744
Dead + Super + HB(ULS)	826.201325	759.136125

Table 12 Summary of loading:

Summary of loading			
Combinations	**Moment**	**Moment**	**Shear**
	SLS kNm	ULS kN	ULS kN
Dead loading	1444.5		
Combination 1	2105.9929	1939.7858	826.201325
Combination 3	1939.7858	2233.27885	759.136125

Serviceability Limit State Design

Table 13 Beam Section Data:

Beam Section modulus, top fibre	$Z_{bt} = -\frac{I_b}{y_{bt}}$	116019736
Beam Section modulus, bottom fibre	$Z_{bb} = -\frac{I_b}{y_{bb}}$	-89065656
Beam Section area	A_b	449220
The applied moment at transfer	M_t	1644.75
The ratio of prestress after transfer to prestress before any loss, P_j	a_t	
Eccentricity of the tendon's centroid from the neutral axis for beam section	e_b	456

Composite Section Modulus, Top Fiber	$Z_{ct} = -\frac{I_b}{y_{bt}}$	179402079
Composite Section Modulus, Bottom Fiber	$Z_{cb} = -\frac{I_b}{y_{bb}}$	-166155698
Composite Section Area	A_c	599220
Applied Moment at Service	M_s	2066.625
The ratio of prestress in service to prestress before any loss, P_j	a_s	
Eccentricity of the tendon's centroid from the neutral axis for beam section **from neutral axis of beam, eb**	e_c	623

Maximum Stress

$$BS\ 5400\ part\ 4\ [9], cl. 6.3.2.2, cl. 6.3.2.4\ and\ cl. 7.4.3.2$$

$$The\ beam\ will\ be\ classified\ as\ class\ 1$$

$$Maximum\ Stresses$$

$$At\ transfer\ ftt\ -1\ N/mm2$$

$$At\ transfer\ ftc\ 20\ N/mm2$$

$$In\ service\ fsc\ 0\ N/mm2$$

$$In\ service\ fst\ 20\ N/mm2$$

$$Stress\ at\ transfer\ (Beam\ Only, N/mm2):$$

$$Moment\ Due\ to\ Self-Weight\ = \frac{wl^2}{8}$$

$$= \frac{11.23\ x\ 30^2}{8}$$

$$= 1263.375\ KNm$$

$$Dead\ Load\ =\frac{Moment\ due\ to\ self\ weight}{Beam\ Section\ Modulus\ at\ fiber\ location}$$

Table 14 Stress at Transfer:

At Bottom Level -1		
	Combination 1	Combination 3
Dead Load	-10.89	-10.89
At Top Level - 2		
	Combination 1	Combination 3
Dead Load	14.18	14.18

Stress in service (SLS, composite section, N/mm2):

$$Moment\ Due\ to\ Self\ Weight\ =\frac{wl^2}{8}$$

$$=\frac{(11.23+3.75)\ x\ 30^2}{8}$$

$$=\ 1685.25\ KNm$$

Table 15 Stress in Service Bottom level:

At Bottom Level -1		
	Combination 1	**Combination 3**
Dead Load(M/Z)	-10.14258	-10.14258
Super & Live Load(M/Z)	-12.674792	-13.44086
Positive Temp. $=\gamma fL\times 1.38627 = 0.8\times 1.38627$	-	-1.109016
Differential shrinkage	0.5903315	0.5903315
Total Stress	**-22.22704**	**-24.10212**

Table 16 Stress in Service top Level:

At Top Level -3		
	Combination 1	**Combination 3**
Dead Load (M/Z)	9.393708	9.393708
Super & Live Load(M/Z)	11.738959	12.448461
Positive Temp. $=\gamma fL\times 3.15234 = 0.8\times 3.15234$	-	2.521872
Differential shrinkage	0.9647495	0.9647495
Total Stress	**22.09741**	**25.32879**

Magnel Diagram

For Magnel Diagram, a feasible region is found in which points of $1/P_j$ and e simultaneously satisfy all four stress limits.

$$\frac{1}{P_j} \geq \left[\frac{\alpha_t / Z_{bt}}{f_{tt} - M_t / Z_{bt}}\right] e_b + \left[\frac{\alpha_t / A_b}{f_{tt} - M_t / Z_{bt}}\right] \qquad (1)$$

$$\frac{1}{P_j} \geq \left[\frac{\alpha_t / Z_{bb}}{f_{tc} - M_t / Z_{bb}}\right] e_b + \left[\frac{\alpha_t / A_b}{f_{tc} - M_t / Z_{bb}}\right] \qquad (2)$$

$$\frac{1}{P_j} \leq \left[\frac{\alpha_s / Z_{ct}}{f_{sc} - M_s / Z_{ct} - \sigma_{Temperature} - \sigma_{Shrinkage}}\right] e_c + \left[\frac{\alpha_s / A_c}{f_{sc} - M_s / Z_{ct} - \sigma_{Temperature} - \sigma_{Shrinkage}}\right] \qquad (3)$$

$$\frac{1}{P_j} \leq \left[\frac{\alpha_s / Z_{cb}}{f_{st} - M_s / Z_{cb} - \sigma_{Temperature} - \sigma_{Shrinkage}}\right] e_c + \left[\frac{\alpha_s / A_c}{f_{st} - M_s / Z_{cb} - \sigma_{Temperature} - \sigma_{Shrinkage}}\right] \qquad (4)$$

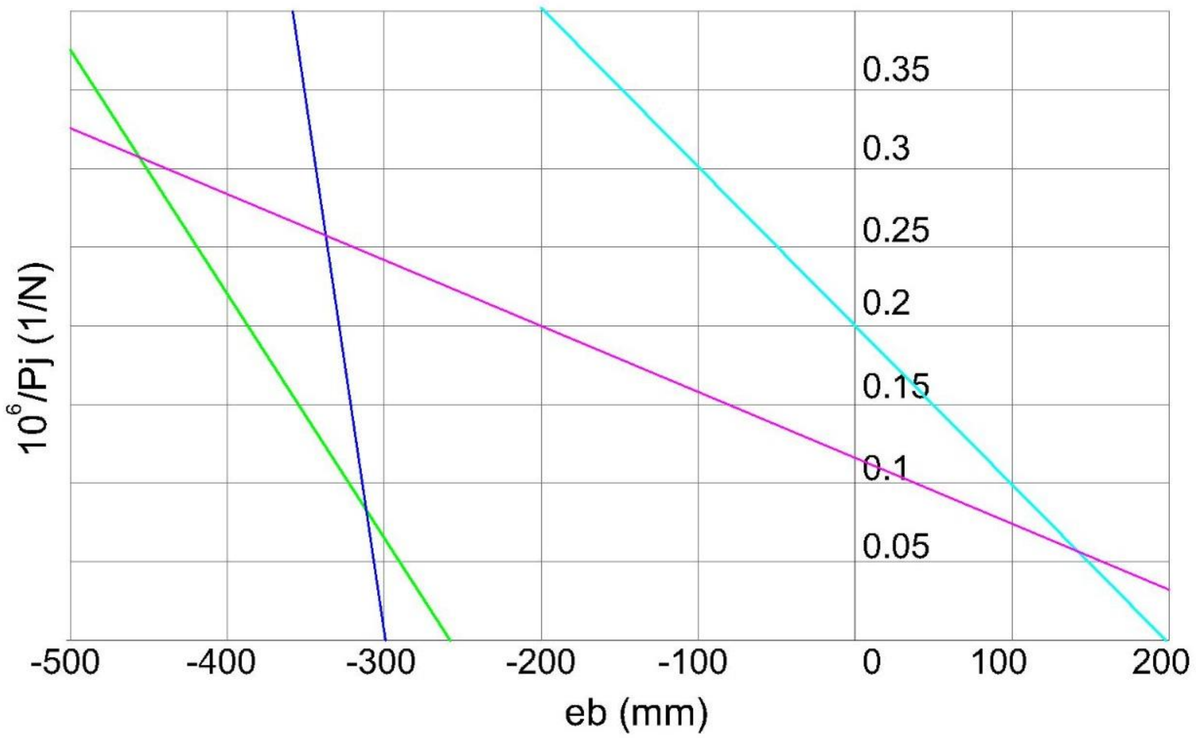


Figure 23 Magnal Diagram

$$The\ Jacking\ force\ is\ \frac{10^6}{0.17x10^3} = 3891.504\ kN$$

Pre-stressing force and Eccentricity

The tendons are now selected from BS 5896. Additionally, Class selection of relaxation is provided. For the design, a relaxation class of 1 and 15.7mm 7-wire super was selected. Below is a table of technical data for the selected wire

Table 17 Wire Data

Percentage loss	0.8	%
Load	4.5	
Diameter	15.2	mm
Breaking load	300	kN
Area	165	mm^2
Normal tensile strength, f_{pu}	1820	n/mm^2

Next, the max initial force is calculated by the following equation;

$$Max\ initial\ force = \frac{Percentage\ loss}{Breaking\ load}$$

$Max\ initial\ force = 240\ kN$

Next the minimum of strands, Strand centre and Strand layout number is calculated by:

$$Minimum\ No^{*}\ of\ strands = \frac{Jacking\ force}{Max\ initial\ force}$$

$$Minimum\ No^{*}\ of\ strands = 17\text{ nos}$$

(To note, the minimum number of strands is always rounded up)

$$Strand\ Centre = Centroid\ of\ beam + e_b$$

$$Strand\ Centre = 145.411\ mm$$

$$Strand\ layout\ No^{*} = Strand\ Centre \times Minimum\ No^{*} of\ strands$$

$$Strand\ Layout\ No^{*} = 2472$$

Designing a layout of strands requires that we use a similar number of strands and strand layout number. Below shows the decided layout

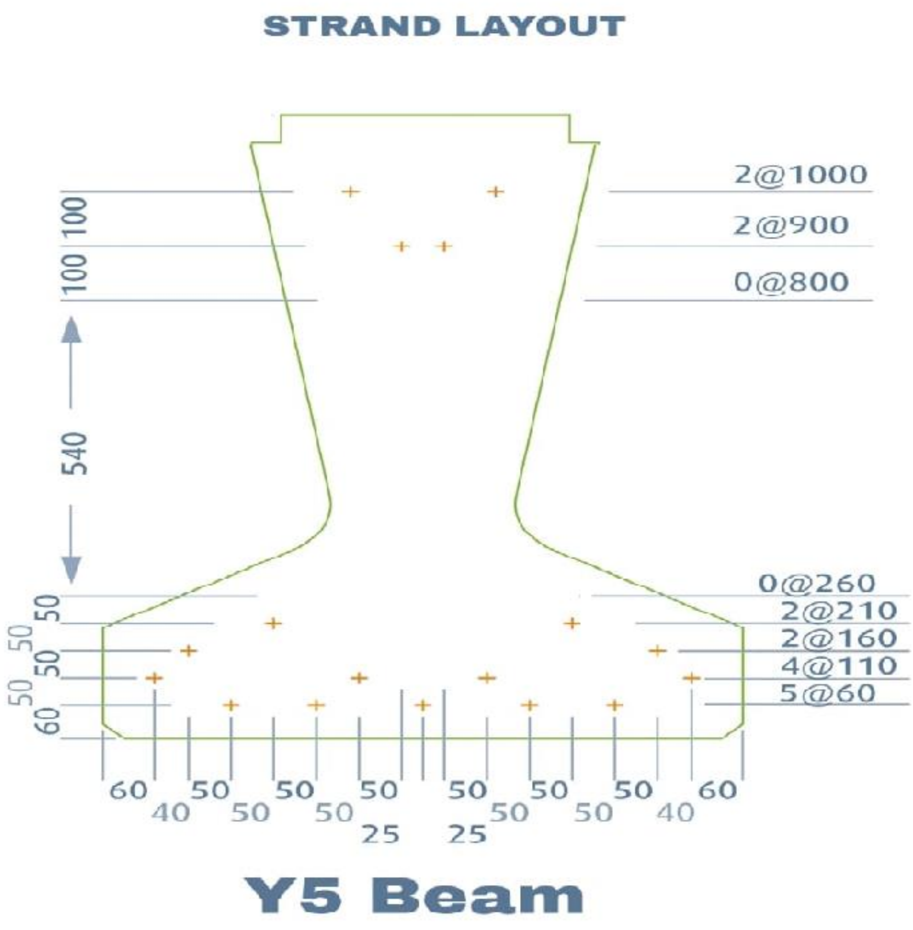


Figure 24 Strand Layout

Table 18 Strand Layout

Soffit	allowed	In layout	
1000	4	2	2000
900	4	2	1800
800	2	0	0
260	6	0	0
210	10	2	420
160	12	2	320
110	14	4	440
60	11	5	300
		Sum of Strands: 17	Strand layout Centre:310.588

Relaxation loss during transfer

BS 5400-part 4 cl. 6.7.2.3 [9] states that an allowed relaxation of loss in steel before transfer is 2.25%. Using the equation below, we can calculate loss of pre-stress during transfer and elastic shorting:

$$\Delta P_{\varepsilon} = \frac{A_p E_p}{E_c}\left(\frac{P_j}{A_b} + \frac{P_j e_b^2}{I_b} + \frac{M_t e_b}{I_b}\right)$$

$$\Delta P_{\varepsilon} = 272.55\ kN$$

$$Calculation\ of\ loss\ percent\ requires\ use\ of\ the\ equations\ below;$$

$$P_j = Minimum\ No * of\ strands \times breaking\ force$$

$$P_j = 4080\ kN$$

$$Pre - stress\ loss\ before\ transfer$$
$$= 1 - allowed\ relaxation\ before\ transfer + \Delta P\varepsilon / Pj$$

$$Pre - stress\ loss\ before\ transfer = 0.079$$

$$This\ is\ an\ 7.9\%\ loss\ in\ pre - stress\ before\ transfer.$$

Relaxation loss after transfer

BS 5400part 4 cl. 6.7.3.3 [9] states the other half of loss of pre-stress occurs after transfer

cl. 6.7.2.4 states pre-stress loss due concrete shrinkage is $\varepsilon cs \times Es \times As$ where $\varepsilon cs = 300 \times 10-6$ and cl. 6.7.2.5 states pre-stress loss due concrete specific creep is $ct \times fci3 \times Es \times As$ where $ct = 48 \times 10-6\ per\ N/mm2$

Table 19 Relaxation Losses After Transfer

steel relaxation	0.0125	
concrete shrinkage pre-stress loss	164.934	KN
Concrete Creep pre-stress loss	351.859 2	KN

$$Calculation\ of\ loss\ percent\ requires\ use\ of\ the\ equations\ below;$$

$$Prestress\ loss\ after\ transfer$$
$$= Prestress\ loss\ before\ transfer + Steel\ relaxation$$
$$+ Prestress\ loss\ due\ to\ concrete/Pj$$
$$+ Prestress\ loss\ due\ to\ concrete\ creep/Pj$$

$$Prestress\ loss\ after\ transfer = 0.2184$$

$$This\ is\ a\ 21\%\ loss\ in\ pre - stress\ after\ transfer.$$

Total loss and Final force

$$Final\ pre - stress\ force\ after\ all\ losses\ P_e = 1 - Prestress\ loss\ after\ transfer \times P_j$$

$$P_e = 1 - 0.24 \times 5936 = 3188.656\ kN$$

$$As\ a\ percentage\ this\ is\ P_e / P_j = 0.7815\ or\ 78.15\%$$

Ultimate Limit State Design

Ultimate design Moment

The design ultimate moment is calculated by taking the highest ULS moment found and applying the partial factor of safety from BS 5400-4 of 1.1. The ultimate design moment is;

$$Ultimate\ design\ moment = \gamma_{f3} \times M$$

$$Ultimate\ design\ moment = 1.1 \times 2233.28$$

$$Ultimate\ design\ moment = 2456.608\ kN$$

Additionally, as the tendons are under stress, we need to obtain the relationship of stress to strain through creating 3 equations that will describe this relation in accordance with BS 5400-4. Below is the BS 5400-4 graph and our calculated graph with labelled equations according to region

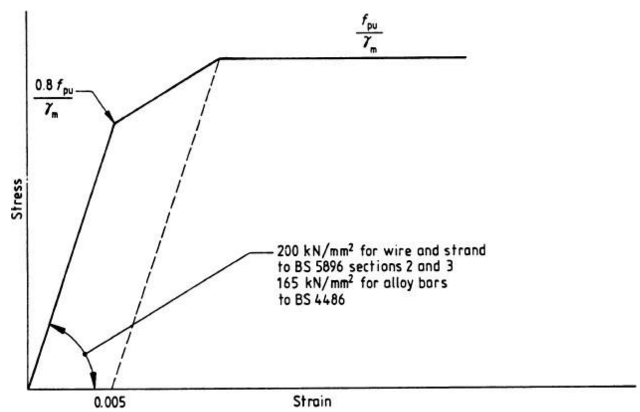


Figure 25 Stress Strain Graph for Steel

$$Region\ equations:$$

$$A)\ f_p = 200{,}000\varepsilon_p;\ 0 \leq \varepsilon_p \leq 0.00616$$

$$B)\ fp = 47{,}799.06(\varepsilon p - 0.00616) + 936.86;\ 0.00616 \leq \varepsilon p \leq 0.0126$$

$$C)\ f_p = 1539.13;\ \varepsilon_p \geq 0.0126$$

Stress in tendon

$$Stress\ in\ each\ tendon\ after\ all\ losses\ is\ calculated\ as:$$

$$fpe = (3188.65\ x\ 1000)\ /\ (17\ x\ 165)$$
$$fpe = 1136.77\ N/mm2$$

$$Prestrain\ is\ found\ by\ the\ following\ equation:$$

$$\varepsilon pe = fpe\ /Es$$
$$= 1136.77/\ 200 \times 10\text{^}3$$
$$= 0.0057$$

Total force in tendons

$$x = 0.6h$$
$$= 0.6 \times 1200$$
$$= 720mm$$

$$Use\ of\ the\ following\ equations\ will\ be\ required\ to\ find\ the\ total\ force\ in\ tendons:$$
$$effective\ depth = deck\ thickness - soffit$$

$$Tendon's\ strain, \varepsilon p$$
$$= (Effective\ depth$$
$$- Depth\ to\ neutral\ axis)\ \times Ultimate\ strain\ of\ concrete$$
$$/\ Depth\ to\ neutral\ axis$$
$$Total\ strain, \varepsilon p + \varepsilon pe = Tendon'strain + prestrain$$
$$Tendons\ stress, fsp\ = one\ of\ the\ three\ region\ equations\ depending\ on\ Total\ strain$$
$$Total\ Tendon\ Force, Fp$$
$$= No * strands\ at\ that\ soffit\ \times\ Tendon's\ stress\ \times\ Tendon\ area(m)$$

Applying all equations results in the table below:

Table 20 Force in Tendons

No of strands	Soffit	Dp	Ep	Ep+Epe	Fsp	Fp
2	1000	200	-0.0025278	0.003156102	631.220481	208.302759
2	900	300	-0.0020417	0.003642214	728.442704	240.386092
0	800	400	-0.0015556	0.004128325	1086.75288	0
0	260	940	0.00106944	0.006753325	1204.912	0
2	210	990	0.0013125	0.00699638	1215.85266	401.231378
2	160	1040	0.00155556	0.007239436	1226.79332	404.841796
4	110	1090	0.00179861	0.007482491	1237.73398	816.904427
5	60	1140	0.00204167	0.007725547	1248.67464	1030.15658
17						3101.82303

$$Compressive\ force:$$
$$We\ must\ compare\ this\ against\ total\ compression\ within\ the\ concrete\ and\ check\ if\ total\ tendon\ force\ is\ large$$

$$First\ the\ compressive\ force\ is\ calculated\ from:$$

$$Compressive\ Force\ in\ Flange, Ff\ = 0.4\ \times\ fcu\ \times\ area\ of\ flange$$

$$Ff\ = 0.4\ \times\ 40\ \times\ 1000\ \times\ 150\ \times\ 10 - 3$$

$$Ff \quad = \; 2400\; kN$$

$$Compressive\ force\ within\ the\ web\ above\ the\ neutral\ axis$$

$$Fw = 0.4 \times fcu \times area\ of\ web$$
$$= \; 3545.688\; KN$$

$$Adding\ these\ forces\ gives\ us\ the\ total\ compression\ force\ in\ concrete:$$

$$F_W = 0.4\ x\ 50\ x\ 593\ x\ 2\ x \frac{570}{671}\ x\ 570\ x\ 10^{-3}$$
$$Fw \quad = \; 3545.688\; kN$$

$$Adding\ these\ forces\ gives\ us\ the\ total\ compression\ force\ in\ concrete:$$

$$Fc \quad = \; 2400 \; + \; 3545.688$$
$$Fc \quad = \; 5945.688$$

Force check

As the total compression force is larger than the total tendon force, the design fails.

$$Fp \leq Fc$$

To correct this, the depth to neutral axis will be changed. The optimum depth where total tendon force is slightly larger than total compression force is 380 mm. Below is the recalculated table and total compression force

Table 21 Corrected Force In Tendons

No of strands	soffit	Dp	Ep	Ep+Epe	Fsp	Fp
2	1000	200	-0.0016579	0.004025985	805.19709	265.71504
2	900	300	-0.0007368	0.004947038	1123.60563	370.789856
0	800	400	0.00018421	0.005868091	1165.06497	0
0	260	940	0.00515789	0.010841775	1388.94541	0
2	210	990	0.00561842	0.011302301	1409.67509	465.192778
2	160	1040	0.00607895	0.011762828	1430.40476	472.03357
4	110	1090	0.00653947	0.012223354	1451.13443	957.748722
5	60	1140	0.007	0.01268388	1471.8641	1214.28788
17						3745.76785

$$Fc \quad = \; 3721.60$$
$$Fp \quad > \; Fc$$
$$OK$$

Total moment due to concrete

Calculation of the total moment requires use of this equation:

$Mp = Distance\ from\ neurtal\ axis \times Fp\ at\ each\ soffit$

Applying this to each soffit gives the table below:

Table 22 Moment Force in Tendons

Fp KN	Distnce From NA	Mp KNm
265.7150396	-180	-47828.707
370.7898563	-80	-29663.189
0	20	0
0	560	0
465.1927782	610	283767.595
472.0335697	660	311542.156
957.7487223	710	680001.593
1214.287881	760	922858.79
		2120.67824

Moment capacity

$$For\ Design\ Check\ we\ calculate\ Moment\ Capacity\ using\ below\ equations:$$

$$Compressive\ force\ in\ concrete\ web = 0.4 * 50 * (393 + 393 + (200 - 393) * (380 - 150)/671)/2 * (380 - 150) * 0.001$$

$$= 3721.61\ KN$$

$$Total\ Moment\ in\ web\ \ w\ = Fw \times lever\ arm$$

$$Mw = 3545.69 * (380 - 191)/1000$$

$$Mw = 703.38\ kNm$$

$$compressive\ force\ in\ flange = 0.4 * 40 * 1000 * 150 * 0.001$$

$$= 2400\ KN$$

$$Total\ Moment\ in\ Flange, f = Ff \times lever\ arm$$

$$Mf == 2400 * (380 - 150/2)/1000$$

$$Mf = 732\ kNm$$

$$Adding\ these\ moments\ provides\ the\ total\ moment\ due\ to\ the\ concrete:$$

$$Mu = MF + Mw + Mp$$

$$Mu = 2120.68 + 703.38 + 732$$

$$Mu = 3556.06\ kNm$$

$$\boldsymbol{Moment\ check}$$

$$Checking\ this\ against\ the\ ultimate\ design\ moment;$$

$$Mu\ >\ \gamma f3\ \times\ M$$

$$3556.06\ kN\ \ >\ 2456.61\ kNm\quad \boldsymbol{Hence\ OK}$$

$$\boldsymbol{Outermost\ tendon\ check}:$$

$$cl.\,6.3.3.1\ states\ that\ outmost\ strain\ of\ the\ tendons\ must\ be\ greater\ than\ 1.15$$

$$\frac{Mu}{Yf3\ X\ M}\ >\ 1.15$$

3556.06/2456.61> 1.15

1.45 > 1.15 **Hence OK**

Outermost tendons provide greater than 25% of the total tendon area > 0.25

=5/17

= 0.29 > 0.25 **Hence OK**

Ultimate Shear capacity

For the final stage in design analyse, we need to check the beam in shear to determine shear links required.

cl. 6.3.4.1 states that we must check the shear resistance of the composite in both uncracked and cracked.
Case I – Uncracked

at the support: M = 0, V = 826.20 kN

$$fcp = 0.87(\frac{P_e}{A_c})$$

$$fcp = 0.87(\frac{3188.67\ X\ 1000}{599220})$$

$$fcp = 4.63\ N/mm^2$$

$$The\ ultimate\ shear\ capacity\ in\ an\ uncracked\ section\ in\ flexture, V_\infty\ is\ calculated\ using$$

$$V_\infty = 0.67bh\sqrt{{f_t}^2 + f_{cp}f_t}$$

$$V_\infty = 0.67 * 200 * 1200\sqrt{1.69^2 + 4.63 * 1.69} * 10^{-3}$$

$$V_\infty = 487.08\ KN$$

$$Next\ we\ check\ V_\infty\ \ against\ V$$

$$V = \ 1.1\ X\ 826.20$$

$$= \ 908.82KN \quad < \quad 487.08\ KN$$

$$So\ not\ consider\ shear\ links\ for\ a\ uncracked\ section$$

$$\boldsymbol{Case\ II}: \boldsymbol{Cracked}$$

$$f_{st} = 0.87(\frac{P_e}{A_C} + \frac{P_e{y_t}^2}{I_c})$$

$$f_{st} = 0.87(\frac{3188.67}{599220} + \frac{3188.67 * 1000 * (623 - 60)^2}{103515000000})$$

$$f_{st} = 9.10\ N/mm^2$$

$$M_{cr} = \left(0.37\sqrt{f_{cu}} + \ f_{st}\right)I/y$$

$$M_{cr} = \left(0.37\sqrt{50} + \ 9.10\right)182216/(780.57 - 60)$$

$$M_{cr} = 3044.79\ KNm$$

$$V_{cr} = 0.037bd\sqrt{f_{cu}} + \frac{M_{cr}}{M}V \quad \geq 0.1bd\sqrt{f_{cu}}$$

$$V_{cr} = 0.037 * 200 * (1200 - 202.9)\sqrt{50} + \frac{3044.79}{2233.28} * 241.04$$

$$V_{cr} = 380.80\ KN$$

$$V_{cr} = 380.80\ N \ \geq 0.1bd\sqrt{f_{cu}}$$

$$V_{cr} = 380.80\ \ KN \ \geq 0.1 * 200(1200 - 202.9)\sqrt{50}$$

$$V_{cr} = 380.80 \quad \geq 141.01\ \boldsymbol{Hence\ OK}$$

cl. 6.3.4.4 allows us to determine if the shear links are minimal or normal. This is done by comparing V multiplied by a partial safety factor against V_{cr}

$$V \ = \ 1.1\ X\ 241.04$$

$$= \ 265.144\ KN \ < \ 380.80\ KN\ Hence\ Minimum\ shear\ links\ requied$$

$$(\frac{A_{sv}}{S_v})_{min} = \left(\frac{0.4b}{0.87f_{yv}}\right)$$

$$(\frac{226.3}{S_v})_{min} = \left(\frac{0.4 * 200}{0.87 * 500}\right)$$

$$S_v = 1230.51mm$$

$$USE\ No12\ shear\ link\ at\ 1100\ mm$$

Abutment and Pier Design:

Below Table Value calculate using the excel spreadsheet for beam analysis, influence line and moving load analysis.

Table 23 Reaction of pier and Abutment

Loaded Spans		HA UDL	Nominal Reactions	
			Abutment	Pier
Single Span Loaded		34.4	405.4	626.6
Both Spans Loaded		13.6	160.3	466.3

Load type	UDL	Nominal Reactions	
		Abutment	Pier
Concrete Deck	14.5	176.55	513.6
Surfacing	2.4	28.29	82.29

Table 24 Data for pier

Critical Reaction Under One Beam (kN)		
Load Type	Nominal	Ultimate
Concrete Deck	514	650
Surfacing	82	158
HA UDL+KEL	187	308
37.5 units HB	418	597

Total Reaction on Pier (kN)		
Load Type	Nominal	Ultimate
Concrete Deck	5136	6497
Surfacing	823	1584
HA UDL+KEL	1363	2248
37.5 units HB	2924	4181

Table 25 Data for Abutment

Abutment					
Critical Reaction Under One Beam			Total Reaction on Abutment		
Load Type	Nominal	Ultimate	Load Type	Nominal	Ultimate
Concrete Deck	177	223	Concrete Deck	1766	2233
Surfacing	28	54	Surfacing	283	545
HA UDL+KEL	131	217	HA UDL+KEL	959	1582
37.5 units HB	343	490	37.5 units HB	2400	3432

Table 26 Pier Geotechnical properties

Pier Geometry (m)	
Tw	1.2
Hw	6
Wb	5.5
Hb	1.5
Wt	2.15
Wh	2.15

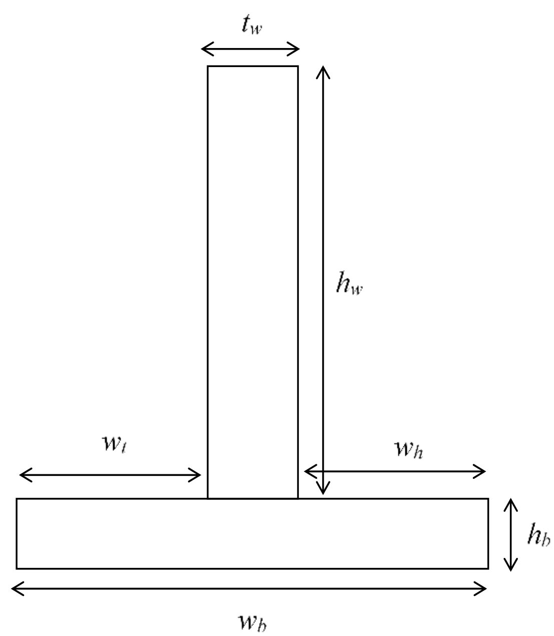


Figure 26 pier section

Table 27 Abutment Geotechnical properties

Abutment Geometry (m)	
tw	1.2
hw	6
wb	5.5
hb	1.5
wt	1.2
wh	3.1

Table 28 Backfill Properties

Backfill Properties	
Density (kN/m3)	18
Friction angle	37

Table 29 Allowable Bearing Pressure

Allowable Bearing Pressure	
Pier (kN/m3)	500
Abutment (kN/m3)	500

Table 30 Angle of Friction for Base

Angle of Friction for Base	
Pier	32
Abutment	32

Load cases

$$Case\ 1: Backfill\ +\ Construction\ surcharge\ wall\ backfilled\ up\ to\ bearing\ shelf\ level\ only$$

$$Case\ 2: Backfill\ +\ HA\ surcharge\ +\ Deck\ dead\ load\ +\ Deck\ contraction$$

$$Case\ 3: Backfill\ +\ HA\ surcharge\ +\ Braking\ behind\ abutment\ +\ Deck\ dead\ load$$

$$Case\ 4: Backfill\ +\ HB\ surcharge\ +\ Deck\ dead\ load$$

$$Case\ 5: Backfill\ +\ HA\ surcharge\ +\ Deck\ dead\ load\ +\ HB\ on\ deck$$

$$Case\ 6: Backfill\ +\ HA\ surcharge\ +\ Deck\ dead\ load\ +\ HA\ on\ deck\ +\ Braking\ on\ deck$$

$$Equations:$$

$$Stem: W\ =\ 25KN/m3\ \times\ tw\ \times\ hw$$

$$Base: W\ =\ 25KN/m3\ \times\ Wb\ \times\ hb$$

$$Backfill: W\ =\ 18KN/m3\ \times\ hw\ \times\ Wh$$

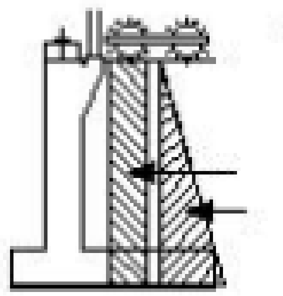
$$Stem: Lever\ arm\ =\ (tw\ +\ Wt)/2$$

$$Base: Lever\ arm\ = Wb/2$$

$$Backfill: Lever\ arm\ = (Wt\ +\ tw\ +\ Wh)/2$$

$$Moment\ =\ W\ \times\ lever\ arm$$

Case 1 :



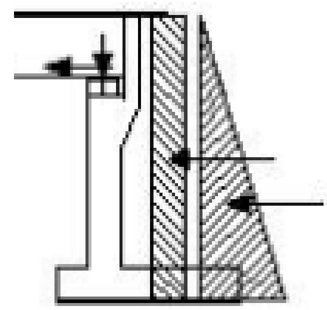
CASE 1

Case 1- Stability Check-Abutment			
Load Type	W (kN/m)	Lever arm (m)	Moment kN.m/m)
Stem	180	1.80	324
Base	206	2.75	567
Backfill	335	3.95	1322
Surcharge	37	3.95	147
Sum	**758**		**2361**
Backfill	126	2.5	315
Surcharge	55	3.75	208
Sum	**181**		**523**
		4.5	**> 2 OK Overturning Check**
Frictional force	474	**2.6**	**>2 OK Sliding Check**
Pressure at the toe	**187**	**< 500 OK Bearing Pressure Check**	
Pressure at the heel	**89**	**< 500 OK Bearing Pressure Check**	

Case 1- Stability Check-Pier			
Load Type	W (kN/m)	Lever arm (m)	Resisting Moment kN.m/m)
Stem	180	2.75	495
Base	206	2.75	567
Sum	**386**		**1062**
Sum	**0**		**0**
		NA	**> 2 OK Overturning Check**
Frictional force	241	**NA**	**>2 OK Sliding Check**
Pressure at the toe	**70**	**< 800 OK Bearing Pressure Check**	
Pressure at the heel	**70**	**< 800 OK Bearing Pressure Check**	

Case 2:

Case 2- Stability Check-Abutment			
Load Type	W (kN/m)	Lever arm (m)	Resisting Moment kN.m/m)
Stem	180	1.80	324
Base	206	2.75	567
Backfill	335	3.95	1322
HA Surcharge	31	3.95	122
Deck Dead Load	228	1.80	410
Sum	**980**		**2746**
Backfill	126	2.5	315
HA Surcharge	46	3.75	173
Deck Contraction	67	7.5	502
Sum	**239**		**990**
		2.8	**> 2 OK Overturning Check**
Frictional force	612	**2.6**	**>2 OK Sliding Check**
Pressure at the toe	**364**	**< 500 OK Bearing Pressure Check**	
Pressure at the heel	**-8**	**< 500 OK Bearing Pressure Check**	

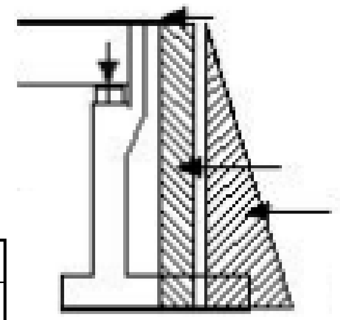


Case 2- Ultimate Shear and Moment-Abutment Wall										
Shear (kN)						Moment (kN.m)				Ultimate Axial Load (kN)
Section	Depth (m)	Backfill	Surcharge	Deck Contraction	Ultimate	Backfill	Surcharge	Deck Contraction	Ultimate	308.7
H/8	0.8	2.0	7.5	67	111.3	0.5	2.8	50.2	77.2	
H/4	1.5	8.1	15.0	67	133.7	4.0	11.3	100.3	168.7	
3H/8	2.3	18.1	22.5	67	162.7	13.6	25.3	150.5	279.4	
H/2	3.0	32.3	30.0	67	198.4	32.3	45.0	200.6	414.4	
5H/8	3.8	50.4	37.5	67	240.7	63.0	70.3	250.8	578.6	
3H/4	4.5	72.6	45.0	67	289.6	108.9	101.3	301.0	777.0	
7H/8	5.3	98.8	52.5	67	345.2	172.9	137.8	351.1	1014.7	
H	6.0	129.0	60.0	67	407.5	258.0	180.0	401.3	1296.6	

Case 2- Stability Check-Pier			
Load Type	W (kN/m)	Lever arm (m)	Resisting Moment kN.m/m)
Stem	180	2.75	495
Base	206	2.75	567
Deck Dead Load	662	2.75	1821
Sum	**1048**		**2883**
Deck Contraction	133	7.5	994
Sum	**133**		**994**
		2.9	**> 2 OK Overturning Check**
Frictional force	655	**4.9**	**>2 OK Sliding Check**
Pressure at the toe	**388**	**< 800 OK Bearing Pressure Check**	
Pressure at the heel	**-7**	**< 800 OK Bearing Pressure Check**	

Case 2- Ultimate Shear and Moment-Pier Wall						
		Shear (kN)		Moment (kN.m)		Ultimate Axial Load (kN)
Section	Depth (m)	Deck Contraction	Ultimate	Deck Contraction	Ultimate	897.9
H/8	0.8	133	189.6	99.4	142.2	
H/4	1.5	133	189.6	198.9	284.4	
3H/8	2.3	133	189.6	298.3	426.6	
H/2	3.0	133	189.6	397.8	568.8	
5H/8	3.8	133	189.6	497.2	711.0	
3H/4	4.5	133	189.6	596.7	853.2	
7H/8	5.3	133	189.6	696.1	995.4	
H	6.0	133	189.6	795.5	1137.6	

Case 3:



CASE 3

Case 3- Stability Check-Abutment			
Load Type	W (kN/m)	Lever arm (m)	Resisting Moment kN.m/m)
Stem	180	1.80	324
Base	206	2.75	567
Backfill	335	3.95	1322
HA Surcharge	31	3.95	122
Deck Dead Load	228	1.80	410
Sum	**980**		**2746**
Backfill	126	2.5	315
HA Surcharge	46	3.75	173
Braking Behind Abutment	134	7.5	1008
Sum	**307**		**1496**
		1.8	**> 2 OK Overturning Check**
Frictional force	612	**2.0**	**>2 OK Sliding Check**
Pressure at the toe	**465**	**< 500 OK Bearing Pressure Check**	
Pressure at the heel	**-108**	**< 500 OK Bearing Pressure Check**	

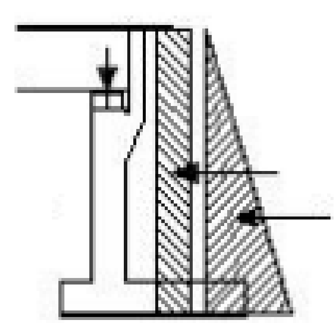
Case 3- Ultimate Shear and Moment-Abutment Wall										
		Shear (kN)				Moment (kN.m)				Ultimate Axial Load (kN)
Section	Depth (m)	Backfill	Surcharge	Braking	Ultimate	Backfill	Surcharge	Braking	Ultimate	308.7
H/8	0.8	2.0	7.5	134	200.6	0.5	2.8	100.8	144.1	
H/4	1.5	8.1	15.0	134	222.9	4.0	11.3	201.7	302.5	
3H/8	2.3	18.1	22.5	134	251.9	13.6	25.3	302.5	480.2	
H/2	3.0	32.3	30.0	134	287.6	32.3	45.0	403.3	682.1	
5H/8	3.8	50.4	37.5	134	329.9	63.0	70.3	504.2	913.2	
3H/4	4.5	72.6	45.0	134	378.9	108.9	101.3	605.0	1178.5	
7H/8	5.3	98.8	52.5	134	434.5	172.9	137.8	705.8	1483.1	
H	6.0	129.0	60.0	134	496.7	258.0	180.0	806.7	1831.9	

Case 3- Stability Check-Pier			
Load Type	W (kN/m)	Lever arm (m)	Resisting Moment kN.m/m)
Stem	180	2.75	495
Base	206	2.75	567
Deck Dead Load	662	2.75	1821
Sum	**1048**		**2883**
Sum	**0**		**0**
		NA	**> 2 OK Overturning Check**
Frictional force	0	**NA**	**>2 OK Sliding Check**
Pressure at the toe	**191**	**< 800 OK Bearing Pressure Check**	
Pressure at the heel	**191**	**< 800 OK Bearing Pressure Check**	

Case 3- Ultimate Shear and Moment-Pier Wall: since there is no horizontal load, then, there is no moment and shear for this load case.	Ultimate Axial Load (kN)
	897.9

Case 4:

Case 4- Stability Check-Abutment			
Load Type	W (kN/m)	Lever arm (m)	Resisting Moment kN.m/m)
Stem	180	1.80	324
Base	206	2.75	567
Backfill	335	3.95	1322
HB Surcharge	50	3.95	196
Deck Dead Load	228	1.80	410
Sum	**998**		**2819**
Backfill	126	2.5	315
HB Surcharge	74	3.75	277
Sum	**200**		**592**
		4.8	**> 2 OK Overturning Check**
Frictional force	624	**3.1**	**>2 OK Sliding Check**
Pressure at the toe	**284**	**< 500 OK Bearing Pressure Check**	
Pressure at the heel	**79**	**< 500 OK Bearing Pressure Check**	



CASE 4

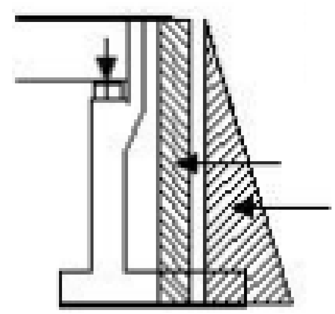
Case 4- Ultimate Shear and Moment-Abutment Wall								
		Shear (kN)			Moment (kN.m)			Ultimate Axial Load (kN)
Section	Depth (m)	Backfill	Surcharge	Ultimate	Backfill	Surcharge	Ultimate	308.7
H/8	0.8	2.0	12.0	23.1	0.5	4.5	8.3	
H/4	1.5	8.1	24.0	52.9	4.0	18.0	36.4	
3H/8	2.3	18.1	36.0	89.3	13.6	40.5	89.3	
H/2	3.0	32.3	48.0	132.4	32.3	72.0	172.0	
5H/8	3.8	50.4	60.0	182.2	63.0	112.5	289.6	
3H/4	4.5	72.6	72.0	238.5	108.9	162.0	446.9	
7H/8	5.3	98.8	84.0	301.6	172.9	220.5	649.0	
H	6.0	129.0	96.0	371.3	258.0	288.0	900.9	

Case 4- Stability Check-Pier			
Load Type	W (kN/m)	Lever arm (m)	Resisting Moment kN.m/m)
Stem	180	2.75	495
Base	206	2.75	567
Deck Dead Load	662	2.75	1821
Sum	**1048**		**2883**
Sum	**0**		**0**
		NA	**> 2 OK Overturning Check**
Frictional force	0	**NA**	**>2 OK Sliding Check**
Pressure at the toe	**191**	**< 800 OK Bearing Pressure Check**	
Pressure at the heel	**191**	**< 800 OK Bearing Pressure Check**	

Case 4- Ultimate Shear and Moment-Pier Wall: since there is no horizontal load, then, there is no moment and shear for this load case.	Ultimate Axial Load (kN)
	897.9

Case 5:

Case 5- Stability Check-Abutment			
Load Type	W (kN/m)	Lever arm (m)	Resisting Moment kN.m/m)
Stem	180	1.80	324
Base	206	2.75	567
Backfill	335	3.95	1322
HA Surcharge	31	3.95	122
HB on Deck	267	1.80	480
Sum	**1019**		**2816**
Backfill	126	2.5	315
HA Surcharge	46	3.75	173
Sum	**172**		**488**
		5.8	**> 2 OK Overturning Check**
Frictional force	637	**3.7**	**>2 OK Sliding Check**
Pressure at the toe	**279**	**< 500 OK Bearing Pressure Check**	
Pressure at the heel	**91**	**< 500 OK Bearing Pressure Check**	



CASE 5

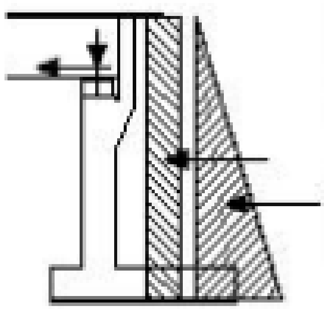
Case 5- Ultimate Shear and Moment-Abutment Wall								
		Shear (kN)			Moment (kN.m)			Ultimate Axial Load (kN)
Section	Depth (m)	Backfill	Surcharge	Ultimate	Backfill	Surcharge	Ultimate	381.3
H/8	0.8	2.0	7.5	15.7	0.5	2.8	5.5	
H/4	1.5	8.1	15.0	38.1	4.0	11.3	25.2	
3H/8	2.3	18.1	22.5	67.1	13.6	25.3	64.2	
H/2	3.0	32.3	30.0	102.7	32.3	45.0	127.5	
5H/8	3.8	50.4	37.5	145.0	63.0	70.3	220.0	
3H/4	4.5	72.6	45.0	194.0	108.9	101.3	346.7	
7H/8	5.3	98.8	52.5	249.6	172.9	137.8	512.6	
H	6.0	129.0	60.0	311.9	258.0	180.0	722.7	

Case 5- Stability Check-Pier			
Load Type	W (kN/m)	Lever arm (m)	Resisting Moment kN.m/m)
Stem	180	2.75	495
Base	206	2.75	567
HB on Deck	325	2.75	893
Sum	**711**		**1956**
Sum	**0**		**0**
		NA	**> 2 OK Overturning Check**
Frictional force	0	**NA**	**>2 OK Sliding Check**
Pressure at the toe	**129**	**< 800 OK Bearing Pressure Check**	
Pressure at the heel	**129**	**< 800 OK Bearing Pressure Check**	

Case 5- Ultimate Shear and Moment-Pier Wall: since there is no horizontal load, then, there is no moment and shear for this load case.	Ultimate Axial Load (kN)
	464.6

Case 6:

Case 6- Stability Check-Abutment			
Load Type	W (kN/m)	Lever arm (m)	Resisting Moment kN.m/m)
Stem	180	1.80	324
Base	206	2.75	567
Backfill	335	3.95	1322
HA Surcharge	31	3.95	122
Deck Dead Load	228	1.80	410
HA on Deck	107	1.80	192
Sum	**1086**		**2938**
Backfill	126	2.5	315
HA Surcharge	46	3.75	173
Braking on Deck	45	7.5	336
Sum	**217**		**824**
		3.6	**> 2 OK Overturning Check**
Frictional force	679	**3.1**	**>2 OK Sliding Check**
Pressure at the toe	**371**	**< 500 OK Bearing Pressure Check**	
Pressure at the heel	**24**	**< 500 OK Bearing Pressure Check**	



CASE 6

Case 6- Ultimate Shear and Moment-Abutment Wall										
Shear (kN)						Moment (kN.m)				Ultimate Axial Load (kN)
Section	Depth (m)	Back fill	Surcharge	Brakig on Deck	Ultimate	Back fill	Surcharge	HA on Deck	Ultimate	175.8
H/8	0.8	2.0	7.5	45	79.8	0.5	2.8	33.6	53.5	
H/4	1.5	8.1	15.0	45	102.1	4.0	11.3	67.2	121.3	
3H/8	2.3	18.1	22.5	45	131.1	13.6	25.3	100.8	208.4	
H/2	3.0	32.3	30.0	45	166.8	32.3	45.0	134.4	319.7	
5H/8	3.8	50.4	37.5	45	209.1	63.0	70.3	168.1	460.3	
3H/4	4.5	72.6	45.0	45	258.1	108.9	101.3	201.7	635.1	
7H/8	5.3	98.8	52.5	45	313.7	172.9	137.8	235.3	849.0	
H	6.0	129.0	60.0	45	376.0	258.0	180.0	268.9	1107.3	

Case 6- Stability Check-Pier			
Load Type	W (kN/m)	Lever arm (m)	Resisting Moment kN.m/m)
Stem	180	2.75	495
Base	206	2.75	567
Deck Dead Load	662	2.75	1821
HA on Deck	151	2.75	416
Sum	**1048**		**2883**
Braking on Deck	45	7.5	336
Sum	**45**		**336**
		8.6	**> 2 OK Overturning Check**
Frictional force	655	**14.6**	**>2 OK Sliding Check**
Pressure at the toe	**257**	**< 800 OK Bearing Pressure Check**	
Pressure at the heel	**124**	**< 800 OK Bearing Pressure Check**	

Case 6- Ultimate Shear and Moment-Pier Wall						
		Shear (kN)		Moment (kN.m)		Ultimate Axial Load (kN)
Section	Depth (m)	Braking on Deck	Ultimate	Braking on Deck	Ultimate	249.8
H/8	0.8	45	61.6	33.6	462.2	
H/4	1.5	45	61.6	67.2	96.1	
3H/8	2.3	45	61.6	100.8	144.2	
H/2	3.0	45	61.6	134.4	192.3	
5H/8	3.8	45	61.6	168.1	240.3	
3H/4	4.5	45	61.6	201.7	288.4	
7H/8	5.3	45	61.6	235.3	336.4	
H	6.0	45	61.6	268.9	384.5	

Reinforcement design of pier

Table 31 Vertical Reinforcement for pier

Critical Ultimate Shear, Moment, and Axial Force per width-Pier										
Vertical Reinforcement										
0.1fcuAc (kN)	4800	> 545.4	Design as slab							
Concerete Cover (mm)	60	εy	0.004175							
Section	Depth (m)	Moment (kN.m)	Reinforcment Length	c/c	Bar Size (mm)	As (mm2)	d (mm)	x(mm)	εs	Mu (kN.m)
H/8	0.75	142.2	0-1.5 m	250	16	804.2	1132.0	22.1	0.1756	392.2
H/4	1.5	284.4	1.5-3 m	250	25	1963.5	1127.5	54.0	0.0696	940.0
3H/8	2.25	426.6								
H/2	3	568.8	3-4.5 m	250	32	3217.0	1124.0	88.5	0.0410	1511.0
5H/8	3.75	711.0								
3H/4	4.5	853.2	4.5-6 m	250	40	5026.5	1120.0	138.2	0.0249	2297.8
7H/8	5.25	995.4								
H	6	1137.6								

Table 32 Horizontal Reinforcement for pier

Horizontal Reinforcement												
Section	Depth (m)	Shear (kN)	Reinforcment Length	Shear Stress (N/mm2),v	ξs	Ultimate Shear Stress (N/mm2),vc	ξsvc	Asv/sv,min	Bar Size (mm)	As (mm2)	Sv (mm)	Asv/sv
H/8	0.75	189.60	0-6 m	0.17	0.82	0.66	0.54	0.92	12	226.19	200.00	1.13
H/4	1.5	189.60			0.82	0.89	0.73					
3H/8	2.25	189.60										
H/2	3	189.60			0.82	1.05	0.86					
5H/8	3.75	189.60										
3H/4	4.5	189.60			0.82	1.22	1.00					
7H/8	5.25	189.60										
H	6	189.60										

Reinforcement design of abutment

Table 33 Vertical Reinforcement for Abutment

Critical Ultimate Shear, Moment, and Axial Force per width-Abutment										
Vertical Reinforcement										
0.1fcuAc (kN)	4800	> 332.1	Design as slab							
Concerete Cover (mm)	60	εy	0.004175							
Section	Depth (m)	Moment (kN.m)	Reinforcment Length	c/c	Bar Size (mm)	As (mm2)	d (mm)	x(mm)	εs	Mu (kN.m)
H/8	0.75	144.1	0-1.5 m	250	12	452.4	1134.0	12.4	0.3155	221.9
H/4	1.5	302.5	1.5-3 m	250	20	1256.6	1130.0	34.6	0.1109	608.3
3H/8	2.25	480.2								
H/2	3	682.1	3-4.5 m	250	25	1963.5	1127.5	54.0	0.0696	940.0
5H/8	3.75	913.2								
3H/4	4.5	1178.5	4.5-6 m	250	40	5026.5	1120.0	138.2	0.0249	2297.8
7H/8	5.25	1483.1								
H	6	1831.9								

Table 34 Horizontal Reinforcement for Abutment

Horizontal Reinforcement												
Section	Depth (m)	Shear (kN)	Reinforcment Length	Shear Stress (N/mm2),v	ξs	Ultimate Shear Stress (N/mm2),vc	ξsvc	Asv/sv,min	Bar Size (mm)	As (mm2)	Sv (mm)	Asv/sv
H/8	0.75	200.56	0-1.5 m	0.18	0.81	0.54	0.44	0.92	12	226.19	200.00	1.13
H/4	1.5	222.92	1.5-3 m	0.22	0.82	0.77	0.62					
3H/8	2.25	251.92										
H/2	3	287.58	3-4.5 m	0.29	0.82	0.89	0.73					
5H/8	3.75	329.89										
3H/4	4.5	378.85	4.5-6 m	0.44	0.82	1.22	1.00					
7H/8	5.25	434.46										
H	6	496.73										

STRUCTURAL DESIGN OF ELASTOMERIC BEARING PADS

Table 35 Temperature Effect

Minimum effective temp.	-12
Maximum effective temp.	36
Temp. range	48
Total deck movement (mm)	21.9
Ultimate thermal movement of deck	15.6
Installation of bearings at midrange temp.	12

Table 36 Bearing of Pier

Bearing of pier	
Total Ultimate reaction (kN)	1405
Thermal movement of pier (mm)	7.8
Use Bearing EKR17 (610 × 420 × 12 mm)	
Maximum Load (kN)	1448
Shear deflection (mm)	8.4
Shear stiffness (kN/mm)	23.79
Bearing horizontal force (kN)	186.2
Bearing horizontal force (kN)	130.2

Table 37 Bearing of Abutment

Bearing of abutment	
Total Ultimate reaction (kN)	768
Thermal movement of pier (mm)	3.9
Use Bearing EKR6 (500 × 320 × 6 mm)	
Maximum Load (kN)	904
Shear deflection (mm)	4.2
Shear stiffness (kN/mm)	24
Ultimate Bearing horizontal force (kN)	93.9
Nominal Bearing horizontal force (kN)	65.7

$$Traction\ and\ Braking\ Load\ -\ BS\ 5400\ Part\ 2\ Clause\ 6.10:$$

$$Nominal\ Load\ for\ HA\ =\ 8kN/m\ \times\ 120\ m\ +\ 250kN\ =\ 1210\ kN$$

$$Nominal\ Load\ for\ HB\ =\ 25\%\ of\ 30\ units\ \times\ 10kN\ \times\ 4axles\ =\ 300\ kN$$

$$1210\ >\ 300, hence\ HA\ braking\ is\ critical.$$

Braking load on each pier/abutment = 1210 / 3 = 403.3 kN/m.

When this load is applied on the deck it will act on two abutments and pier. Thus, pier and each abutment portion: 403.3 kN/m

Skidding Load - BS 5400 Part 2 Clause 6.11:

Nominal Load = 300kN < 403.3kN, hence braking load is critical in the longitudinal direction.

Table 38 Pier and Abutment Breaking load

Braking Load (kN)	HA	1210	1210
	HB	375	
Nominal braking load at pier (kN)	403.3		
Nominal braking load at abutment (kN)	403.3		

Geotechnical Design

Site investigation

A number of factors are needed to be checked when constructing a bridge on a particular site. These are listed as below:

- The engineers should get to know as much as possible about the site such as geological and historical background. One can gather grid location on this by using OS maps. Getting maps and survey report from the local authority can help in getting information about land use and recent developments.
- Geological structure investigation is required to make measurements such as depth or bedrock or pit.
- Soil testing is required to understand the quality of the soil. Factors such as mining, natural cavities and disturbed strata also needs to be identified when doing site investigation. Special in-situ tests should be carried out for this purpose.

Boreholes

Based on the studies conducted in the project area, there are 5 boreholes, which we will examine in the following.

S082NW/1

From 0.3 to 3.7 meters, the borehole revealed upper mad ground of soft brown clay and soft silty blue clay, alluvium of brown black sand with black silty and clean gravel from 11.5 to 17.4 meters, and mostly hard grey green marl from 17.5 to 20.11 meters.

S082NW/3

Water level begins at 1.5 meters, with soil types ranging from medium soft to soft silty brown and Gerry mottled clay at a depth of 6.8 meters. From 6.8 to 11.5 meters, small and medium Gravel with a moisture content of 24 percent, hard brown marl or undistributed sedimentary rock with a moisture content of 26.2 percent to 32 percent

S082NW/4

The level of water commences at 1.3 m, and the moisture content ranges from 1.6 to 6.2 m, with a moisture content of 26 to 33 percent. The soil type ranges from 0.3 to 7.1 m, with medium soft to very soft brown mottled Garry and silty clayey with organic matter, 7.1 to 8.1 meters, sand with brown silt and little Gravels, and 8.1 meters decreasing tough layer of sand and gravel.

S082NW/5

The soil texture remains as loose sand with gravel from 2.1 to 6.2 meters, followed by stiff red marl from 6.2 to 10.6 meters, with soil types ranging from medium to soft and loose with little clay brown silty clay accordingly.

S082NW/7

At a depth of 2.3 to 4.9 meters, soft brown clay begins, followed by different constituents of brown clayey gravel and brown black silty sand, from 4.9 to 12.8 meters, it is brown black silty sand with gravel and stone, and at a depth of 12.5 to 12.8 meters, it is soft blue clay and silty gravel, and at a depth of 14.9 meters, it is hard grey green marl.

GEOLOGICAL SURVEY OF GREAT BRITAIN

RECORD OF SHAFT OR BORE FOR MINERALS

(For Survey use only)
6-inch Map Registered No. 50/82NW/1

Name of Shaft or Bore given by Geological Survey: 1

Name and Number given by owner: Haw Bridge B.H.

Nat. Grid Reference c. 846.278

For whom made

Town or Village Deerhurst County Glouc.

Exact site at site of east abutment of the old bridge (Attach a tracing from a map, or a sketch-map, if possible.)

1"N.S. Map No.	1"O.S. Map No.	Confidential or not
216		No.

Purpose for which made

Ground Level at shaft/bore relative to O.D. 48.7 ft. (14.81 m) If not ground level give O.D. of beginning of shaft/bore

Made by Date of sinking ?1960

Information from Glouc. C.C. Date received

Examined by

SPECIMEN NUMBERS AND ADDITIONAL NOTES

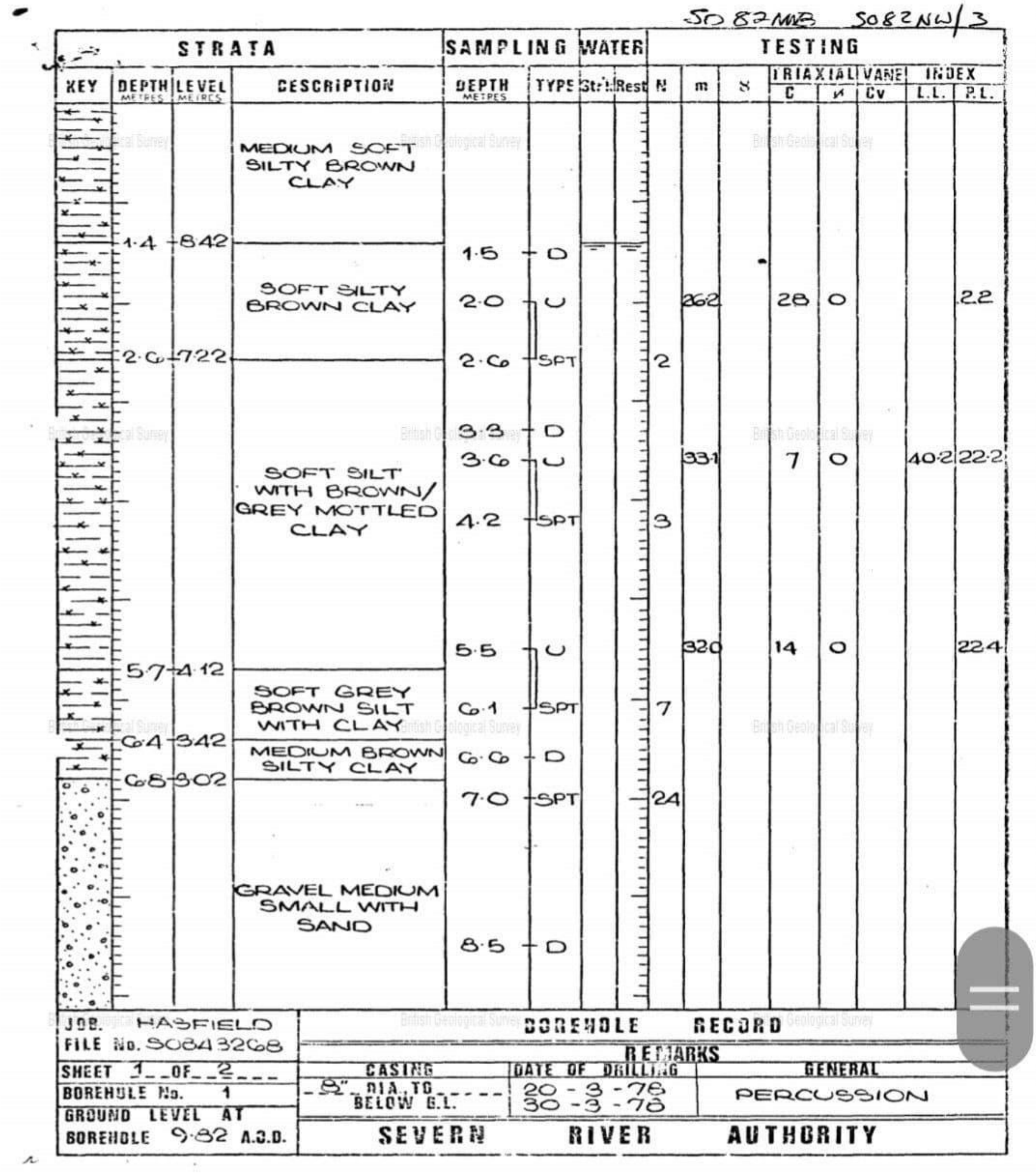
This figure may be too high O.D.
If the clay to 3.7m is in situ alluvium then the S.L. should be around 11–12m O.D. (?)
RAE

(For Survey use only) Geological Classification	Description of Strata	Thickness Ft	Thickness In.	Depth Ft	Depth In.
	Made ground	1 0·4	66	1 0·4	6 6
	Soft brown clay	6 1·9	8 6	8 2·4	0
? Made Ground	Soft silty blue clay	4 1·2	20	12 3·7	0
	Clean gravel	16 4·8	8 0	28 8·5	0
? Alluvium	Brown - black sand	10 3·0	5 0	38 11·6	0
	Brown - black silty sand	1 4·3	0	52 15·8	0
	Clean gravel	1 0·3	0	53 16·2	0
	Soft blue clay	3 0·9	0	56 17·1	0
?	Soft brown clay	1 0·5	6	57 17·5	6
? Arden Sst	Hard grey - green marl	0 0·2	6	58 17·7	0
	Hard shaly green marl	1 0·3	0	59 18·0	0
	Softer patchily green marl	0 0·2	6	59 18·1	6
	Hard shaly green marl	7 2·1	0	?66 20·3	6
				20·27m	

RAE
March 81

C.B.H. 34519

Figure 27 Borehole S082NW/1



SO 82NW3 SO82NW/3

Depth (metres)	Level (metres)	Description
1.4	8.42	MEDIUM SOFT SILTY BROWN CLAY
2.6	7.22	SOFT SILTY BROWN CLAY
5.7	4.12	SOFT SILT WITH BROWN/GREY MOTTLED CLAY
6.4	3.42	SOFT GREY BROWN SILT WITH CLAY
6.8	3.02	MEDIUM BROWN SILTY CLAY
		GRAVEL MEDIUM SMALL WITH SAND

Sampling depth (metres)	Type	N	m	Triaxial C	Triaxial φ	Index L.L.	Index P.L.
1.5	D						
2.0	U		262	28	0		22
2.6	SPT	2					
3.3	D						
3.6	U		331	7	0	40.2	22.2
4.2	SPT	3					
5.5	U		320	14	0		22.4
6.1	SPT	7					
6.6	D						
7.0	SPT	24					
8.5	D						

JOB. HASFIELD
FILE No. SO843268
SHEET 1 OF 2
BOREHOLE No. 1
GROUND LEVEL AT BOREHOLE 9.82 A.O.D.

BOREHOLE RECORD

REMARKS

CASING	DATE OF DRILLING	GENERAL
8" DIA. TO BELOW G.L.	20 - 3 - 78 30 - 3 - 78	PERCUSSION

SEVERN RIVER AUTHORITY

Figure 28 Borehole S082NW/3

So 82NW/4

STRATA			SAMPLING		WATER		TESTING							
DEPTH METRES	LEVEL METRES	DESCRIPTION	DEPTH METRES	TYPE	Strk	Resd	N	m %	γ wet Kg/m³	TRIAXIAL C KN/m²	TRIAXIAL φ	VANE Cv	INDEX L.L.	INDEX P.L.
0.3	9.50	TOP SOIL												
1.8	8.00	MED. BROWN MOTTLED CLAY	1.2	D		DAY 2								
2.3	7.50	SOFT BROWN GREY SILTY CLAY				DAY 3								
3.5	6.40	VERY SOFT GREY BROWN SILTY CLAY	2.5	U				29	1860	27	5		42	24
			3.1	D SPT			2							
5.1	4.70	VERY SOFT GREY SILT WITH ORGANIC MATTER,+ CLAY	4.5	U				33	1750	3	6		53	27
			5.0	SPT			1							
			5.1	D										
5.6	4.20	V. SOFT GREY SILT WITH ORGANIC MATTER SHELLS AND CLAY	5.5	D										
6.2	3.60	SOFT BROWN SILTY CLAY	5.8	U				26	1900	42	6		41	23
			6.0	D										
7.1	2.70	MEDIUM BROWN SILTY CLAY WITH ORGANIC MATTER	6.4	D										
			6.5	SPT			6							
8.1	1.70	SAND WITH BROWN SILT AND LITTLE GRAVEL												
		SAND AND GRAVEL	8.2	D										
			8.3	SPT			1							

JOB. HASFIELD	BOREHOLE RECORD	
FILE No. SO84302705	REMARKS	
SHEET 1 OF 2	CASING	DATE OF DRILLING / GENERAL
BOREHOLE No. 2	8" DIA. TO ____ BELOW G.L.	19-6-78 / 23-6-78 / PERCUSSION
GROUND LEVEL AT BOREHOLE 9.8m A.O.D.	SEVERN RIVER AUTHORITY	

Figure 29 Borehole S082NW/4

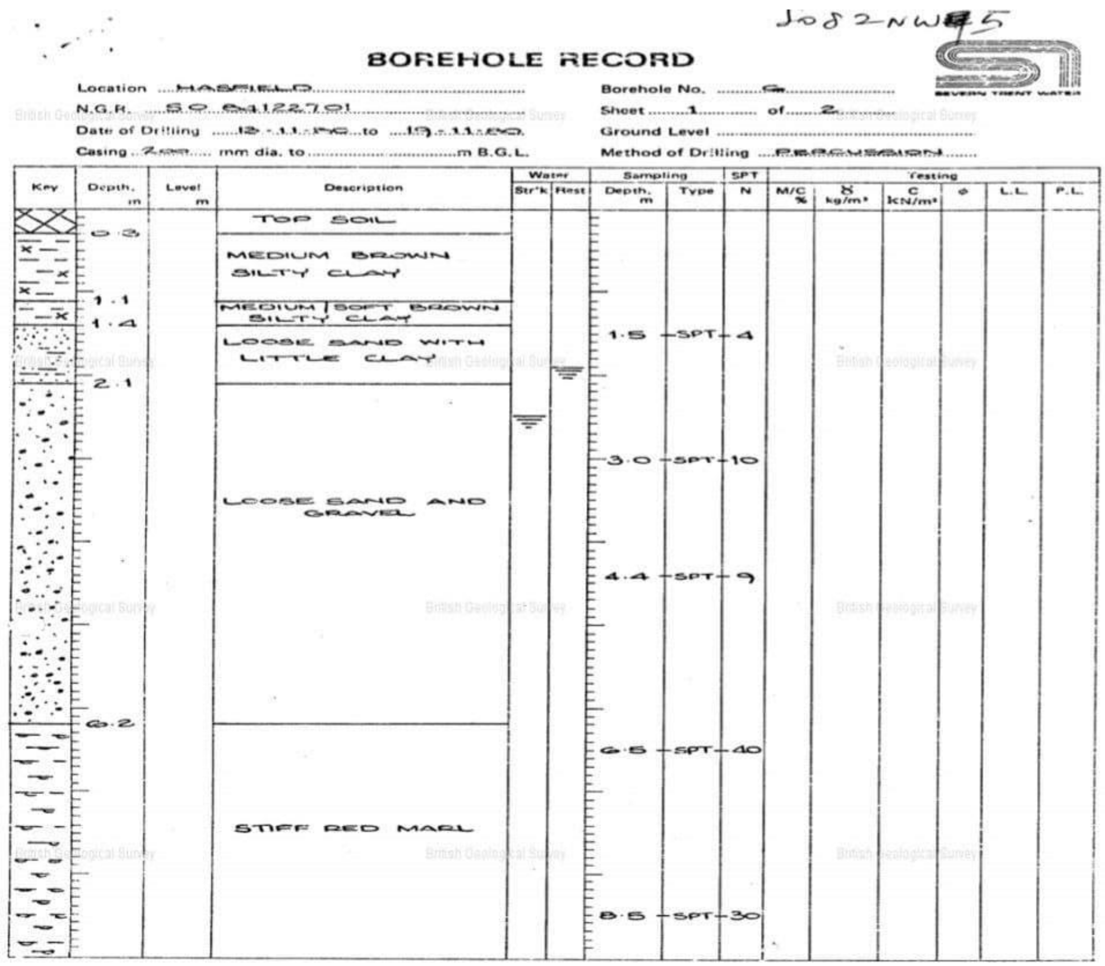


Figure 30 Borehole S082NW/5-1

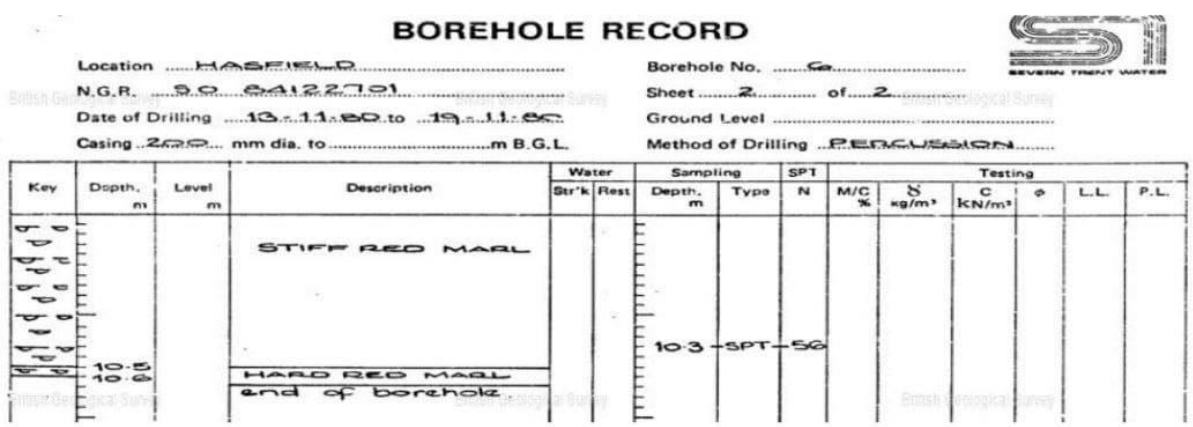


Figure 31 Borehole S080NW/5-2

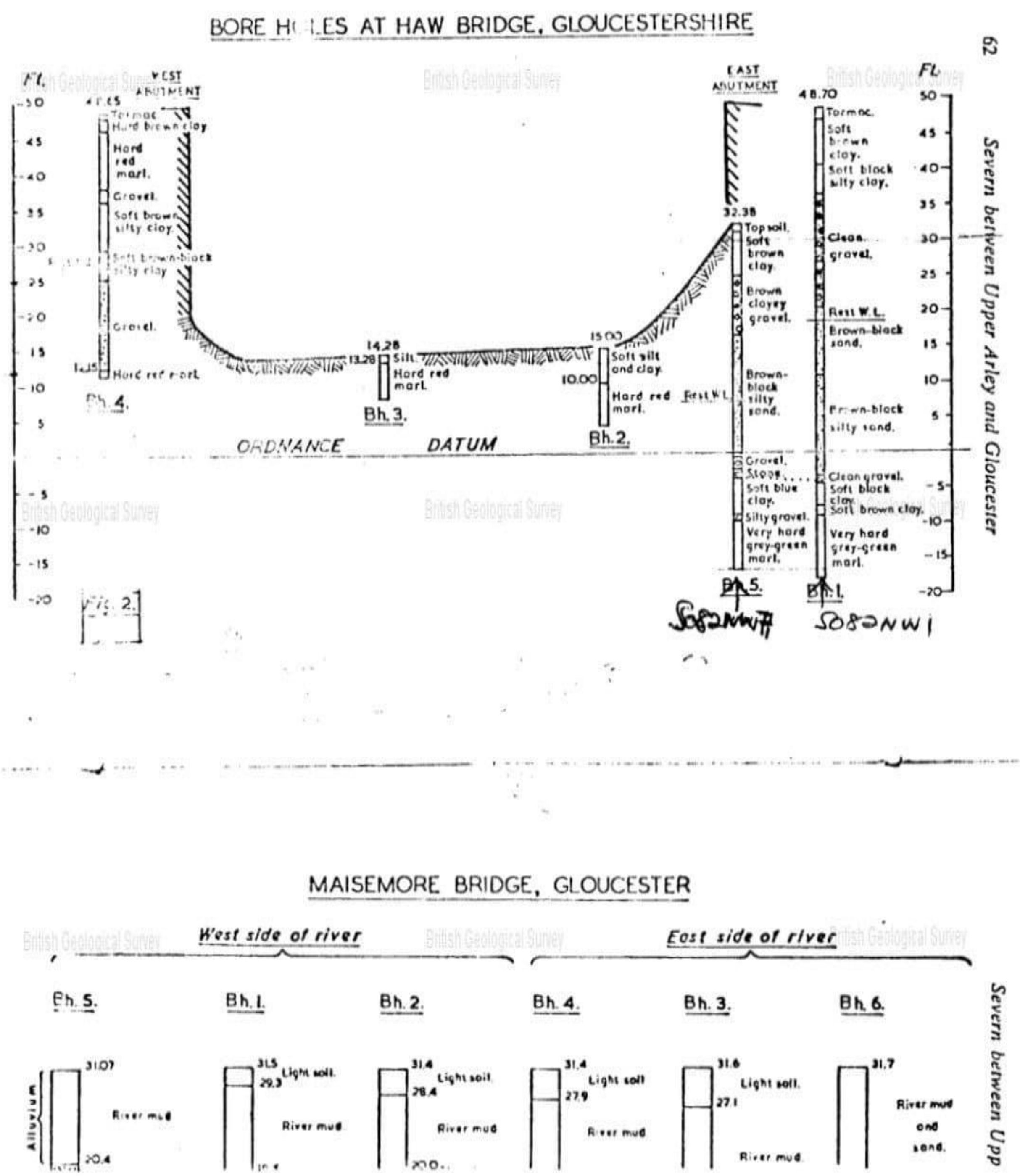


Figure 32 S082NW/7

Ground model

After comparing different boreholes, the borehole S082NW/7 was selected as the grand model, which incorporates other boreholes.

Soil properties

In Ground Investigation design stage, there are some in situ tests such as SPT and CPT test which can present soil specification. In the present project, after reviewing the records of the bridge construction area, 5 boreholes close to the project were used as the design basis. According to the study of project elevation points and assumptions, N=30 was considered as SPT the soil of the foundation and abutment area. Also SPT = 10 for sandy soil (Backfill) is obtained based on boreholes and its height is 5 meters in the calculations. The calculated results of soil characteristics are as follows:

Marl

SPT N value = 30
Density for Marl γ=21kN/m^3 ,(*Parameters*, geotech data, 2021)
Calculation of the effective stress at the depth of the test:
Z = 8.5 m
Total Vertical Stress: $\sigma_v = \gamma Z$ = 21 × 8.5 = 178.5 kN/m2
Pore Water Pressure: $u = \gamma'' Z$ = 9.81 × 6.5 = 63.76 kN/m2
Vertical Effective Stress: $\sigma = \sigma! - u$ = 178.5 – 63.76 = 114.7 kN/m2

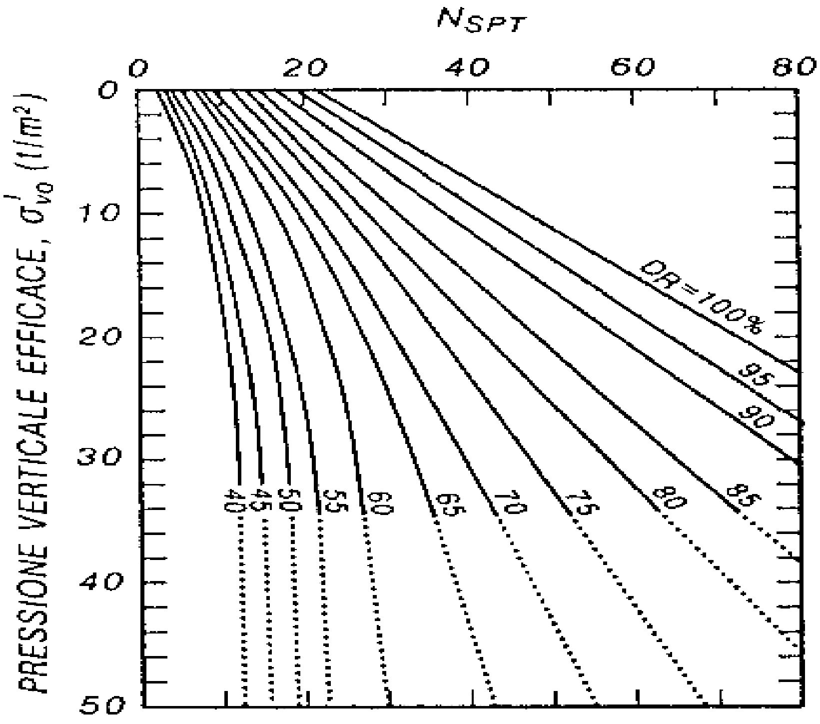


Figure 33 Relative density (D_R)

- The relative density: D_R= 80%

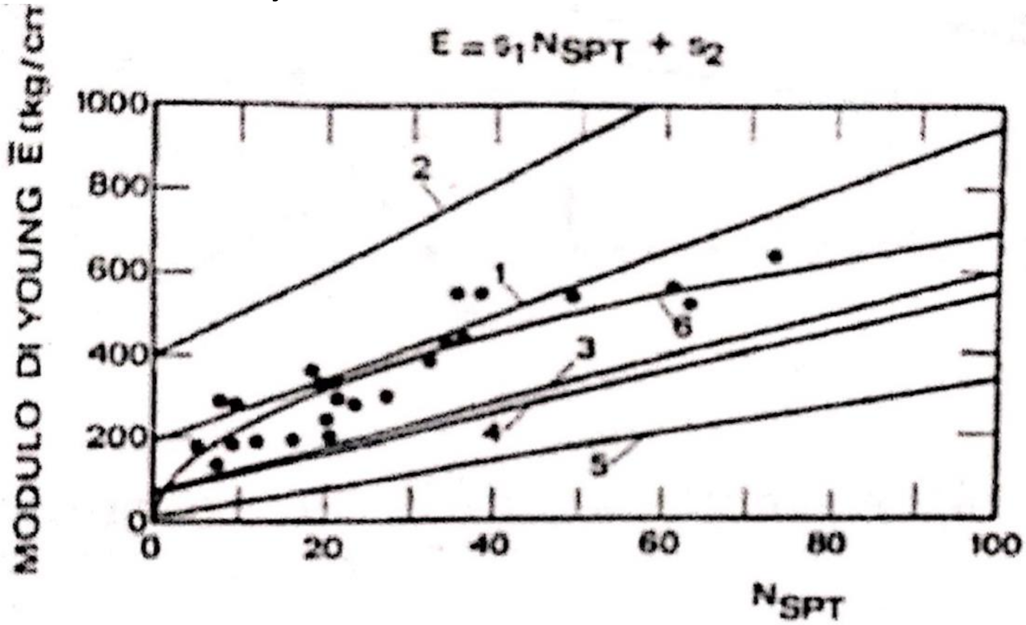


Figure 34 Young's Modulus (E)

- $Young\ Modules$:

$$E = s1N + s2$$
$$E = 68MPa$$

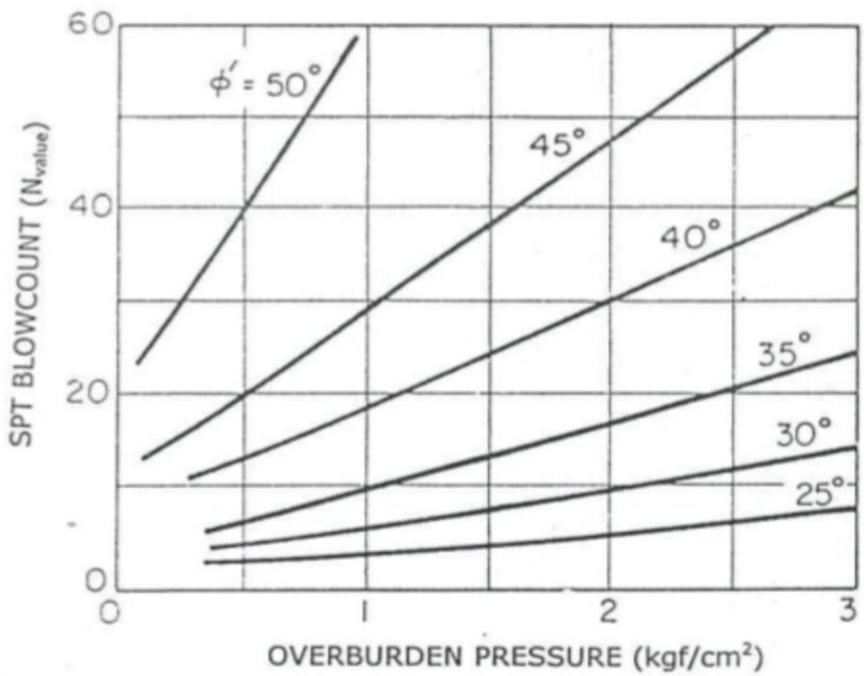


Figure 35 De Mello's correlations for the angle of shearing resistance

- The angle of shear resistance: Φ′= 45 (De Mello formula, 1971)

Backfill sand

$$SPT\ N\ value = 10$$
$$Density\ for\ Sand\ \ \gamma = 19kN/m3\ \ , (\text{Parameters, geotech data, 2021})$$
$$Calculation\ of\ the\ effective\ stress\ at\ the\ depth\ of\ the\ test:$$
$$Z = 5\ m$$
$$Total\ Vertical\ Stress: \sigma v = \gamma Z = 19 \times 5 = 95\ kN/m2$$
$$Pore\ Water\ Pressure: u = \gamma"Z = 9.81 \times 3 = 29.43\ kN/m2$$
$$Vertical\ Effective\ Stress: \sigma = \sigma! - u = 95 - 29.43 = 65.57\ kN/m2$$

- $The\ relative\ density: DR = 50\%$
- $Young\ Modules$:

$$E = s1N + s2$$
$$E = 25MPa$$

- $The\ angle\ of\ shear\ resistance: \Phi' = 35\ (\ De\ Mello\ formula, 1971)$

Pier and Abutment Geometry

Table 39 Pier and Abutment Geometry

Pier Geometry (m)	
tw	1.2
hw	6
wb	5.5
hb	1.5
wt	2.15
wh	2.15

Abutment Geometry (m)	
tw	1.2
hw	6
wb	5.5
hb	1.5
wt	1.2
wh	3.1

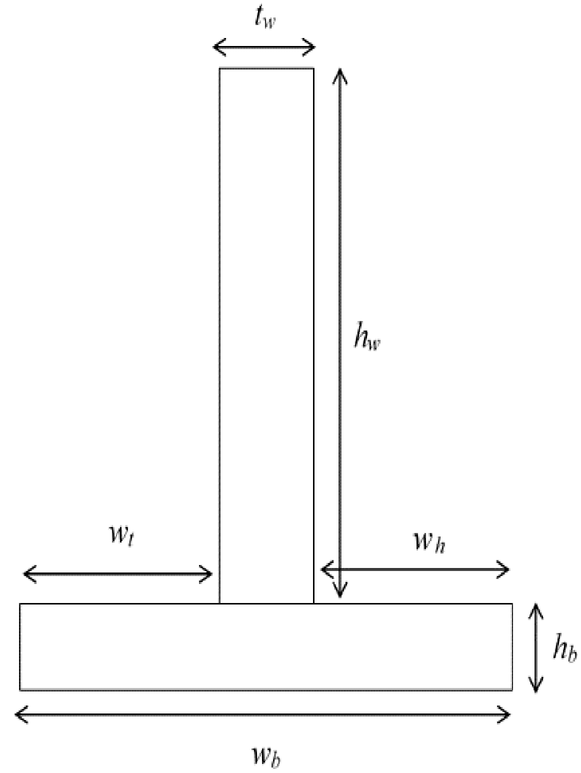


Figure 36 Temperature Effect

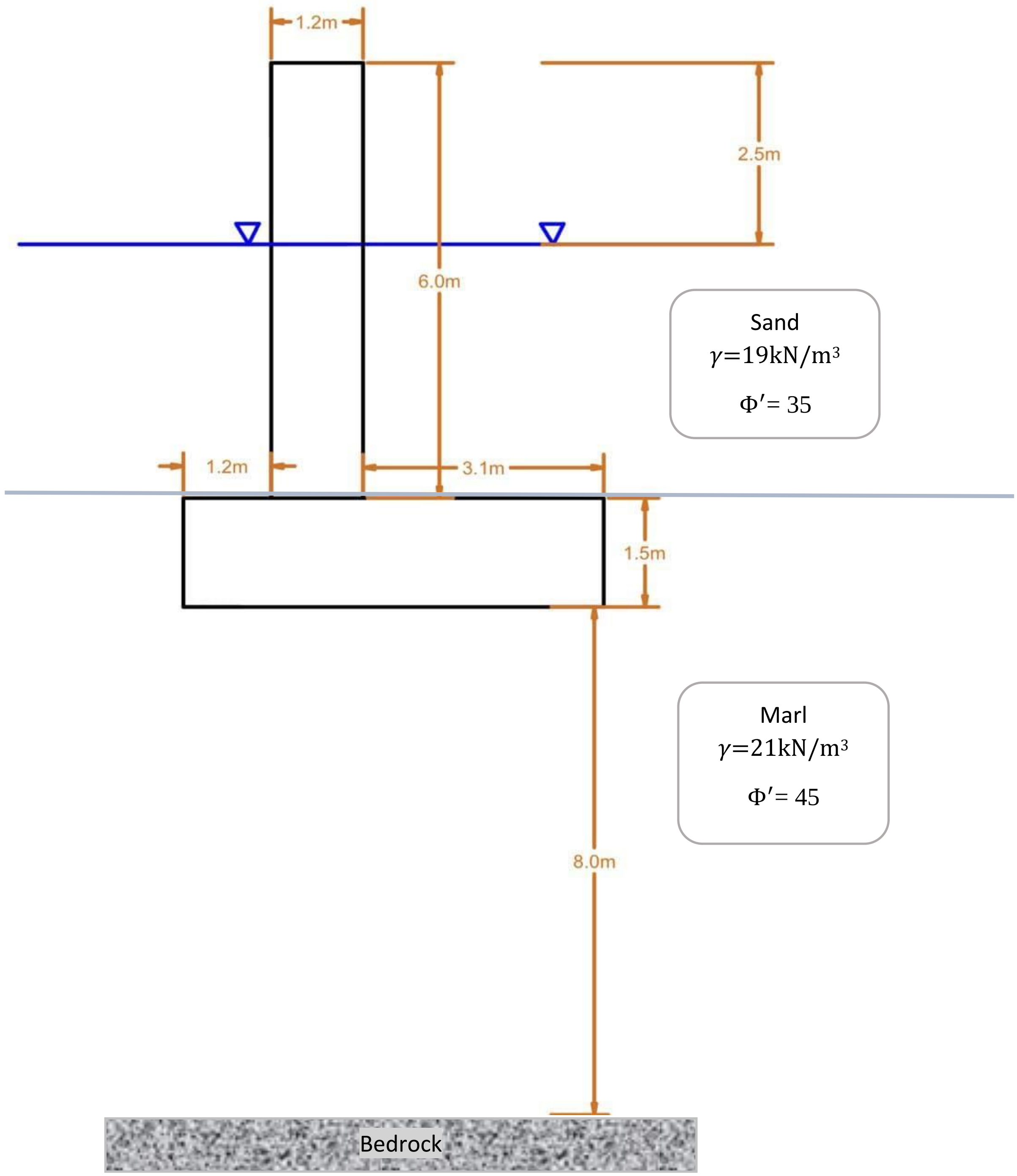


Figure 37 Abutment dimension and its ground model

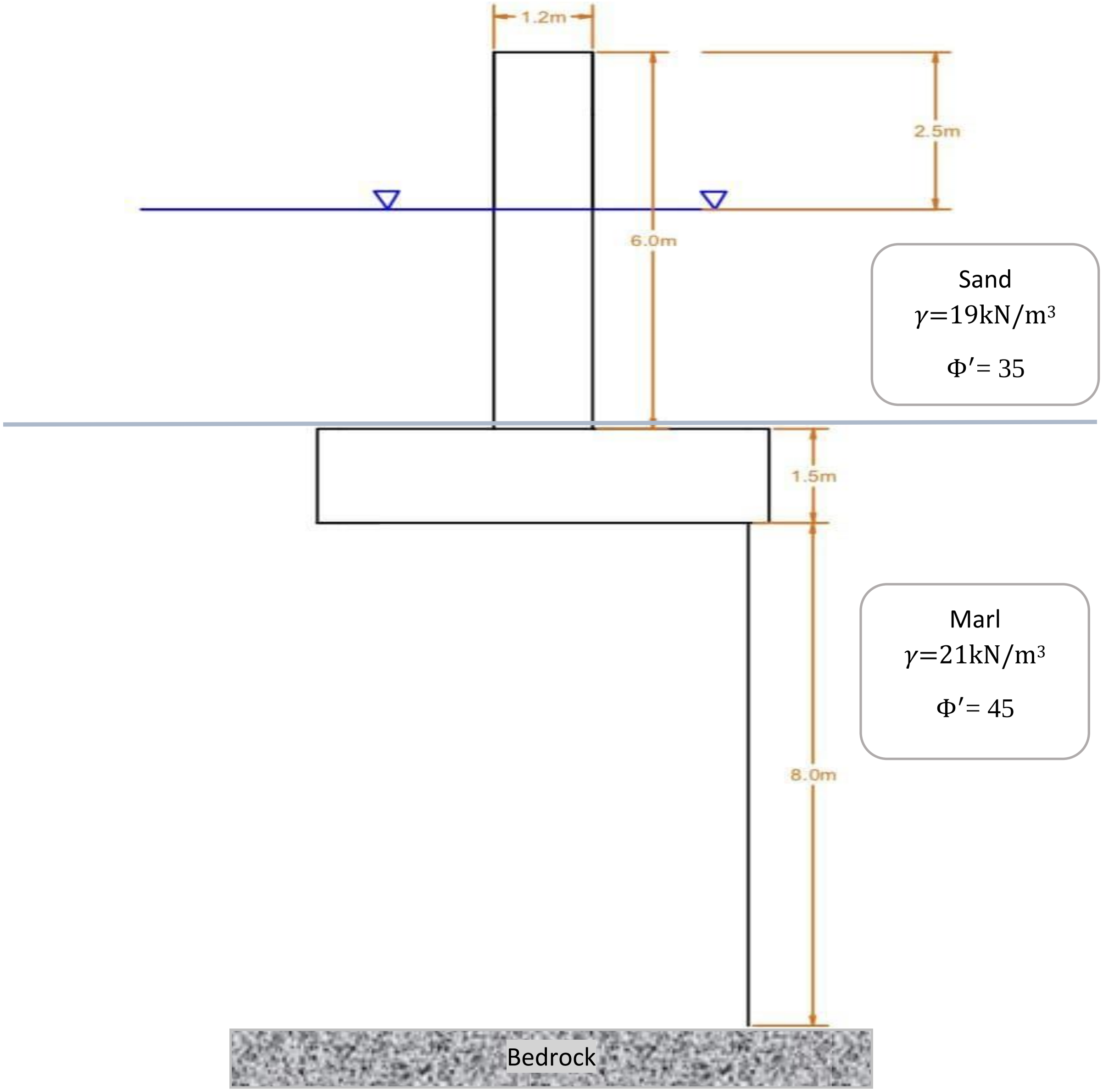


Figure 38 Pier dimension and its ground model

Primary consolidation settlement

Granular sand relations have been used to calculate the structure foundation settlement. It should be noted that according to the type of soil, there is no need to calculate other types of settlements.

z = 8m
According to the figure…:

$z/B = (8\text{-}D) / (5.5/2) = 2.36$

$L / B = 1$

$r = 0.13$

$\Delta q = q - \sigma`(D)$= 225.45 – (21 – 9.81) * 1.5 = 208.665 KN/m^2

$\Delta\sigma` = 5.5 * 208.66 * 0.13 = 149.2\ KN/m^2$

$E = 68MPa$

$\varepsilon vi = (\frac{\Delta\sigma`}{E}) = 2.19*10^{-3}$

$\Delta Hi = 8*2.19*10^{-3} = 0.017m = 17mm$

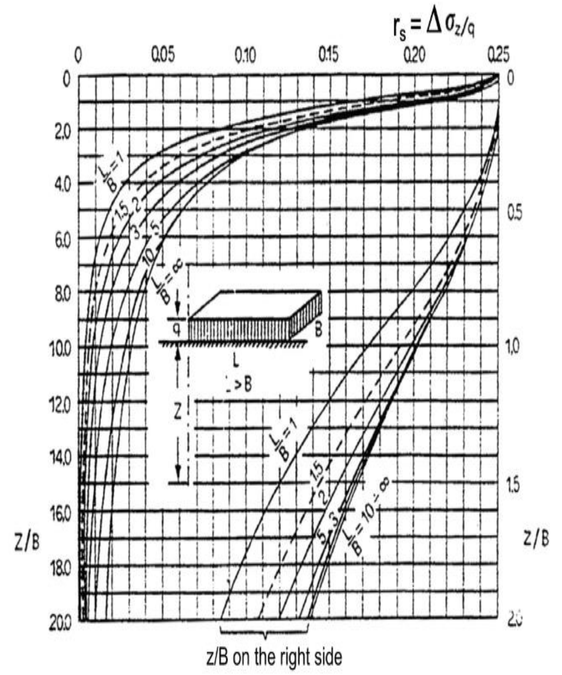


Figure 39 Temperature Effect

According to the table…:

Maximum settlement = 50mm

17mm < 50mm

Design is verified.

Tolerable differential settlement of buildings, mm*

Recommended maximum values in parentheses

Criterion	Isolated foundations	Rafts
Angular distortion (cracking)	1/300	
Greatest differential settlement		
Clays	45 (35)	
Sands	32 (25)	
Maximum settlement		
Clays	75	75–125 (65–100)
Sands	50	50–75 (35–65)

*After MacDonald and Skempton (1955) but see also Wahls (1981).

Figure 40 Tolerable maximum and differential settlements of buildings (from Bowles, 1997)

According to the soil types, It can be considered as undrained loading on foundation.

$$\rho i = I\rho \Delta q B (\frac{1-\nu^2}{Eu})$$

$$\rho t = \rho i$$

Δq: *load applied to the foundation level.*
B: *characteristic dimension of the foundation* ($B = 5.5m$).
Ip: *shape and rigidity factor* ($Ip = 1.12$).
Eu: *undrained elastic modulus of the soil.*
νu: *undrained Poisson's ratio of the soil* ($\nu u = 0.5$).
ρi *is a function of the applied load, the foundation geometry and the undrained elastic properties of the soil.*

$$Load\ transmitted\ to\ foundation\ =\ 1240\ KN/m\ /5.5m\ =\ 225.45KN/m2$$

$$\rho t\ =\ 1.12\ *\ 208.66\ *\ 5.5\ ((1-0.52)\ /\ 68000\ =\ 0.0141m\ =\ 14.1mm$$

Bearing Capacity

According to the ground model, the foundation soil is made of marl, which has a high resistance. For this reason, we use the special relation of rocks to determine the bearing capacity.

$$q_u = cNc + \frac{1}{2}\gamma BN\gamma + \gamma DNq$$

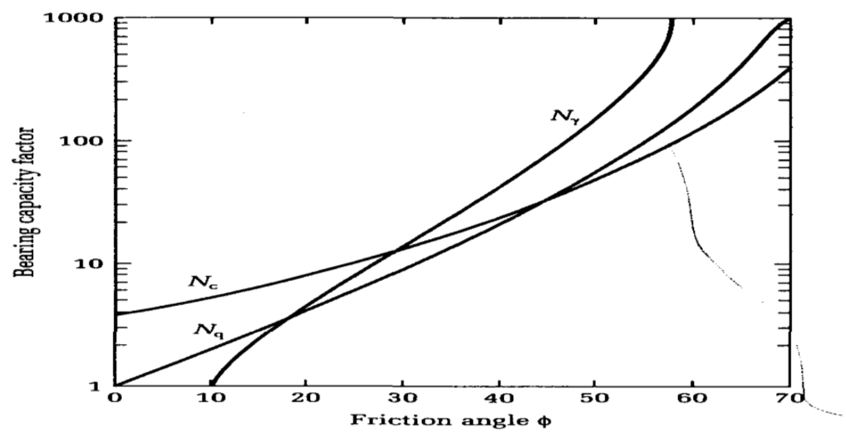


Figure 41 Bearing capacity for foundations on rocks (from Tomlinson, 2001)

$$c\ =\ 100KPa\ ,\ (\text{Vlastelica, Miščević and Pavić, 2016})$$
$$Nc\ =\ 9$$
$$Nq\ =\ 4$$
$$N\gamma\ =\ 4$$

$$qu\ =\ 100\ *\ 9\ +\ 0.5\ *\ 21*\ 5.5\ *\ 4\ +\ 21\ *\ 1.5\ *\ 4\ =\ 1257KPa$$

$$Qstruct\ =\ 1240KN/m\ *\ 5.5m\ =\ 6820KN$$

$$\gamma rc\ =\ 25KN/m3$$

$$Weight\ of\ the\ soil\ backfill\ on\ the\ foundation:$$

$$WS1\ =\ ysVs1\ =\ 19*\ 2.5\ *\ 2.15\ =\ 102.13KN/m$$

$$WS2 = y`sVs2 = 9.19 * 3.5 * 2.15 = 69.15KN/m$$

$$Distance\ of\ the\ WS1\ and\ WS2\ from\ the\ center\ of\ foundation:$$

$$DWS = 1.675m$$

$$Weight\ of\ foundation:$$

$$WCS = 6 * 1.2 * 1 * 25 *= 180KN$$

$$WCB = 5.5 * 1.5 * 1 * 25 = 206.25KN$$

$$V = QStruct + (WCS + WCB) * L + (WS1 + WS2) * L$$
$$V = 6820 + 386.25 * 5.5 + 171.28 * 5.5 = 9886KN$$

Bearing Capacity Checks

K0=1-SinØ`= 1 – Sin(35◦) = 0.426

$P_{1} =$ 0.5 * 19 * 0.426 * 2.5^2 = 25.3KN/m

$P_{2} =$ 0.426 * 19 * 2.5 * 1.75 = 35.4KN/m

$P_{3} =$ 0.5 * 19 * 2.5 * 1.75 * .875 = 36.4KN/m

Total thrust from backfill:

$P_{T} = P_1 + P_2 + P_3$ = 97.1 KN/m

H = P_T * L = 97.1 * 5.5 = 534KN

Distance of the horizontal thrust from the foundation level:

D_{PT} = (P_1 * 5.5 + P_2 * 3.25 + P_3 * 2.67) ÷ (P_T) = 3.61m

My = (97.1* 3.61 – 171.28 * 1.675) * 5.5 = 350KN.m

e_X = 350 ÷ 9886 = 0.035m

B`= 5.5 – 2* 0.035 = 5.43m
L = 5.5m

Factor of Safety = 2.5

FoS=qu/q

$q = V/A = 9886 \div (5.5 * 5.43) = 331 KN/m^2$

FoS = 1257 ÷ 331 = 3.79 > 2.5

The foundation design is verified.

Verification of stability of wall against sliding translation

FoS= 2.5

$W_{CS} = 6 * 1.2 * 25 = 180KN/m$

$W_{CB} = 5.5 * 1.5 * 25 = 206.25KN/m$

Weight of the wall = 386.25KN/m

δ=θ=35°

$P_{restoring} = W * \tan(35°) = 270KN/m$

$P_a = 97.1KN/m$

FoS=Prestoring/Ptranslating=270/97.1=2.8>2.5 Design is verified

Fluid Structure Analysis

The interaction of a moveable or deformable structure with a fluid flow is known as fluid-structure interaction (FSI) (Bungartz 2006). FSI, in instance, is traditionally thought to be a two-way coupling in which the structure is influenced by the fluid's pressure and/or viscous forces, while the fluid is influenced by the structure's shape and velocity (CD-Adapco (2013))..

Many branches of engineering are interested in the dynamic study of structural structures submerged in fluid media. In the earthquake response of bridge piers, fluid-structure interaction occurs. Through the reliance of the effective mass, damping, and stiffness of these systems on the acceleration, velocity, and displacement of the fluid field, respectively, the dynamic response of such immersed structural systems is related to the fluid response. The principal action of the fluids in the range of frequencies characteristic of free vibration of these systems is to increase the effective mass of the structural system. The effects of the fluid on the effective damping and stiffness of the system may be significant in the examination of the forced or transient response of these systems.

The current research focuses on the analysis of fluid-structure systems in which the fluid's physical behavior, to a first approximation, is similar to that of an acoustic medium. The fluid is inviscid, irrotational, compressible, and only sensitive to minor displacements from or oscillations about some equilibrium point, according to the constraining assumptions. The structural systems' behavior is assumed to be linearly elastic. In the presence of fluid free surfaces, it is generally accepted that using the simpler, high frequency approximation to the free surface condition, which corresponds to the lack of gravity waves, is consistent with the above assumptions. Within these limitations, the analytical methods provided in this paper can be used to approximate the vibratory response of submerged bridge piers on a regular basis. Piers create equivalent vibrations in the water in contact with the pier surfaces as they respond to earthquake ground motions. This causes changes in the water pressure acting on the piers, which affects the piers' dynamic reaction. Fluid-structure interaction is the name given to this coupled hydro elastic phenomena

Fluid simulation

Now We are making Solid portion of the model which shown in below figure

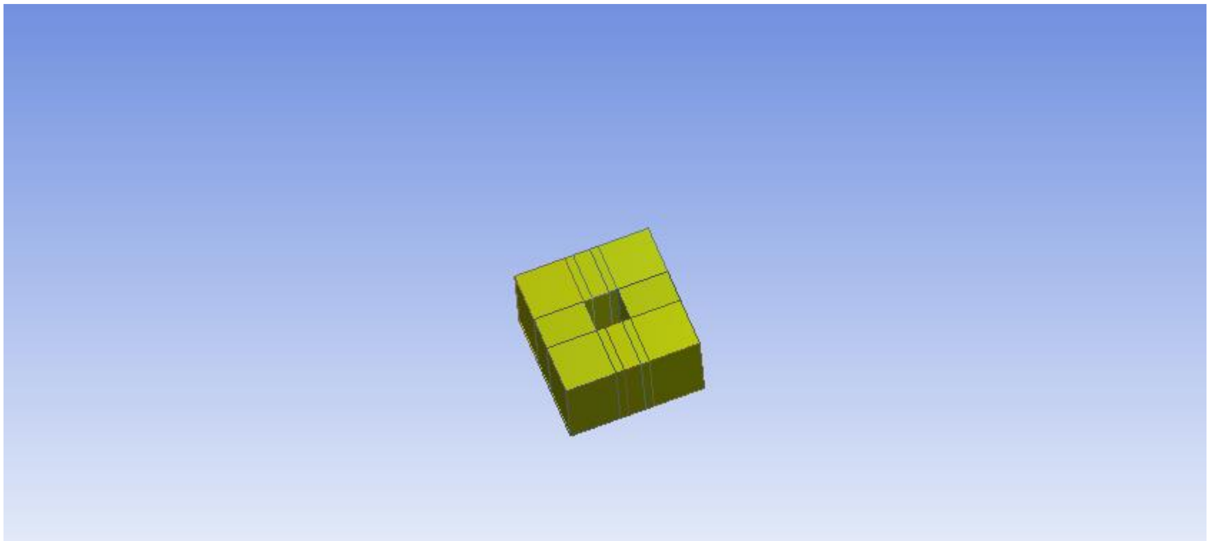
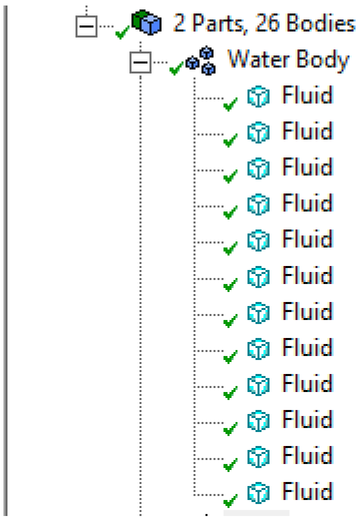


Figure 42 Fluid model



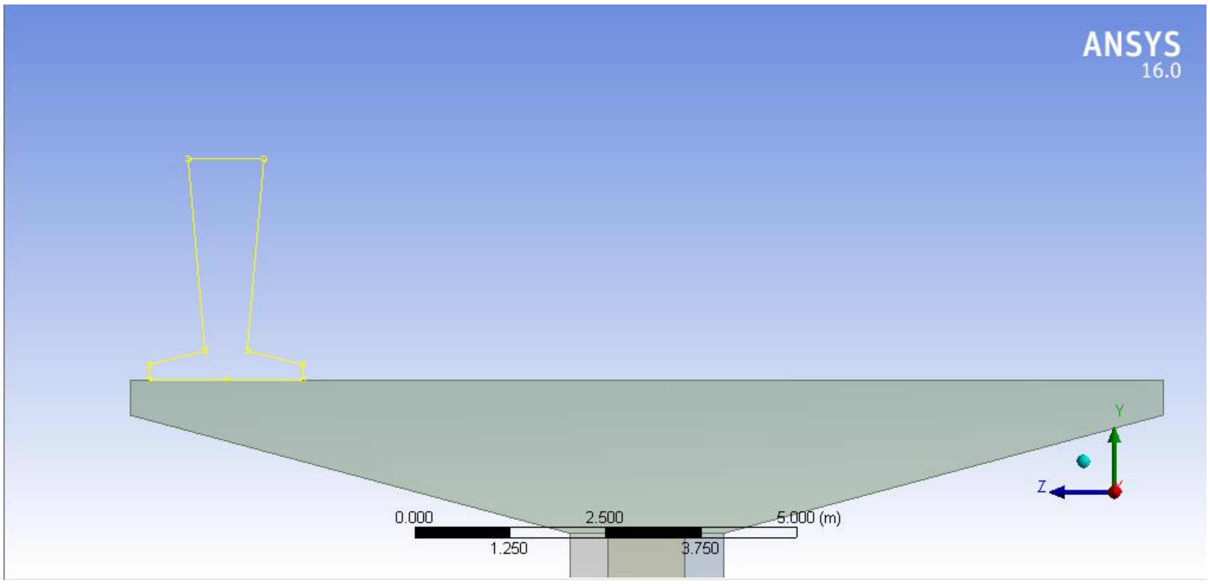


Figure 43 Beam location on pile cap

Here it was created bridge model with solid supression, bridge car lane, pier and beam

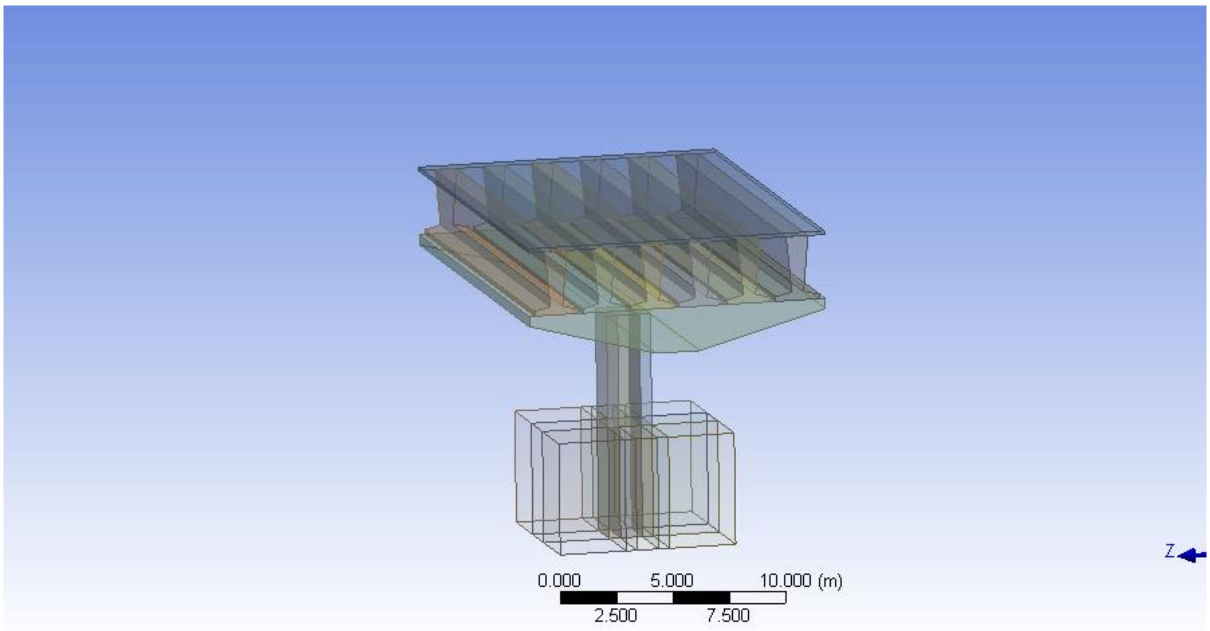
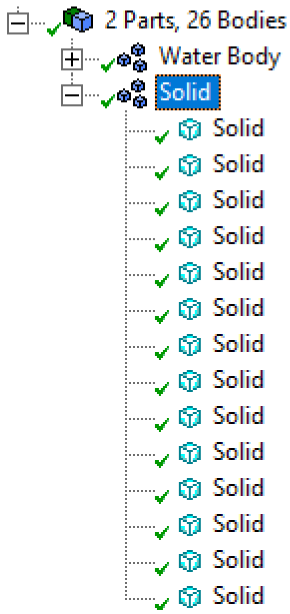


Figure 44 Front View of bridge model


2 Parts, 26 Bodies
Water Body
Solid
Solid
Solid
Solid
Solid
Solid
Solid
Solid
Solid
Solid
Solid
Solid
Solid
Solid
Solid

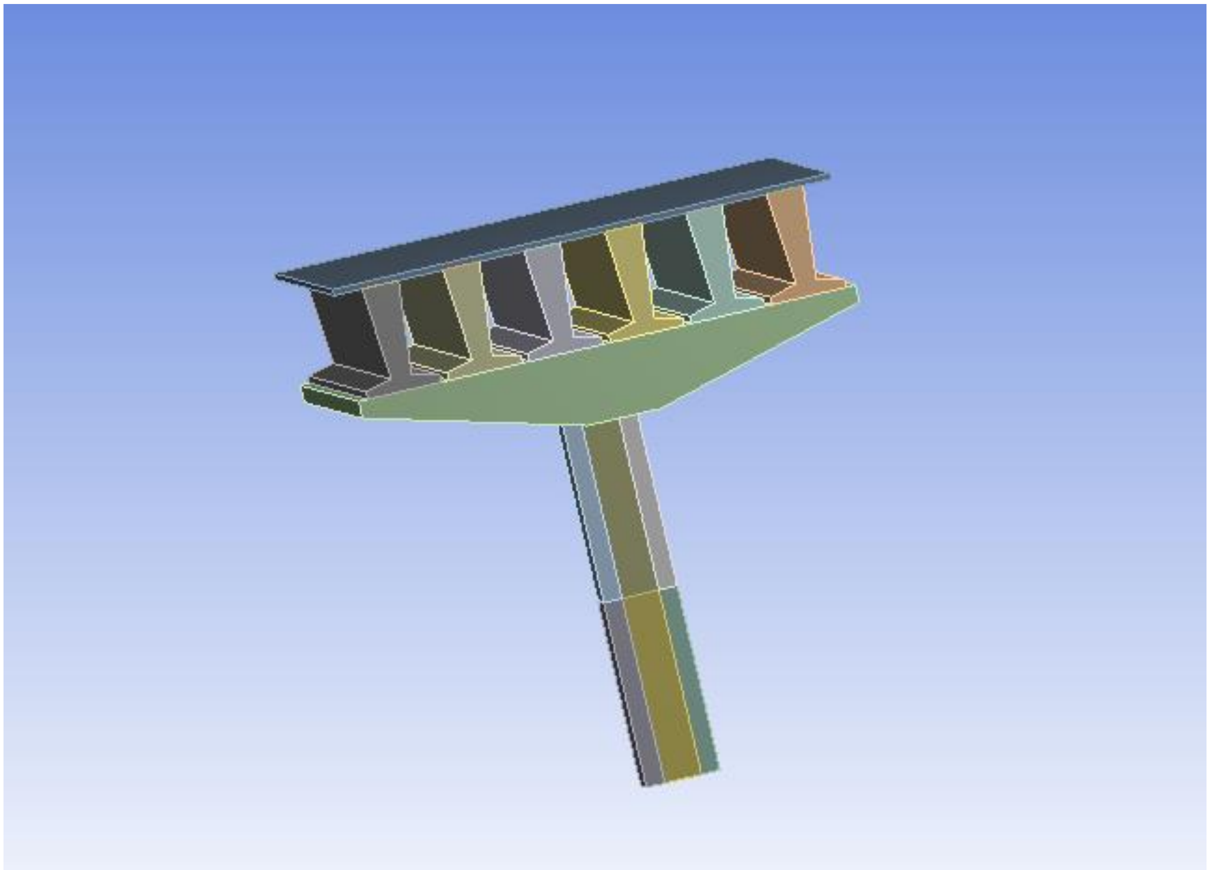


Figure 45 Isometric View of bridge

Meshing:

After making model of solid supression we was doing a mashing on model which shows in below image

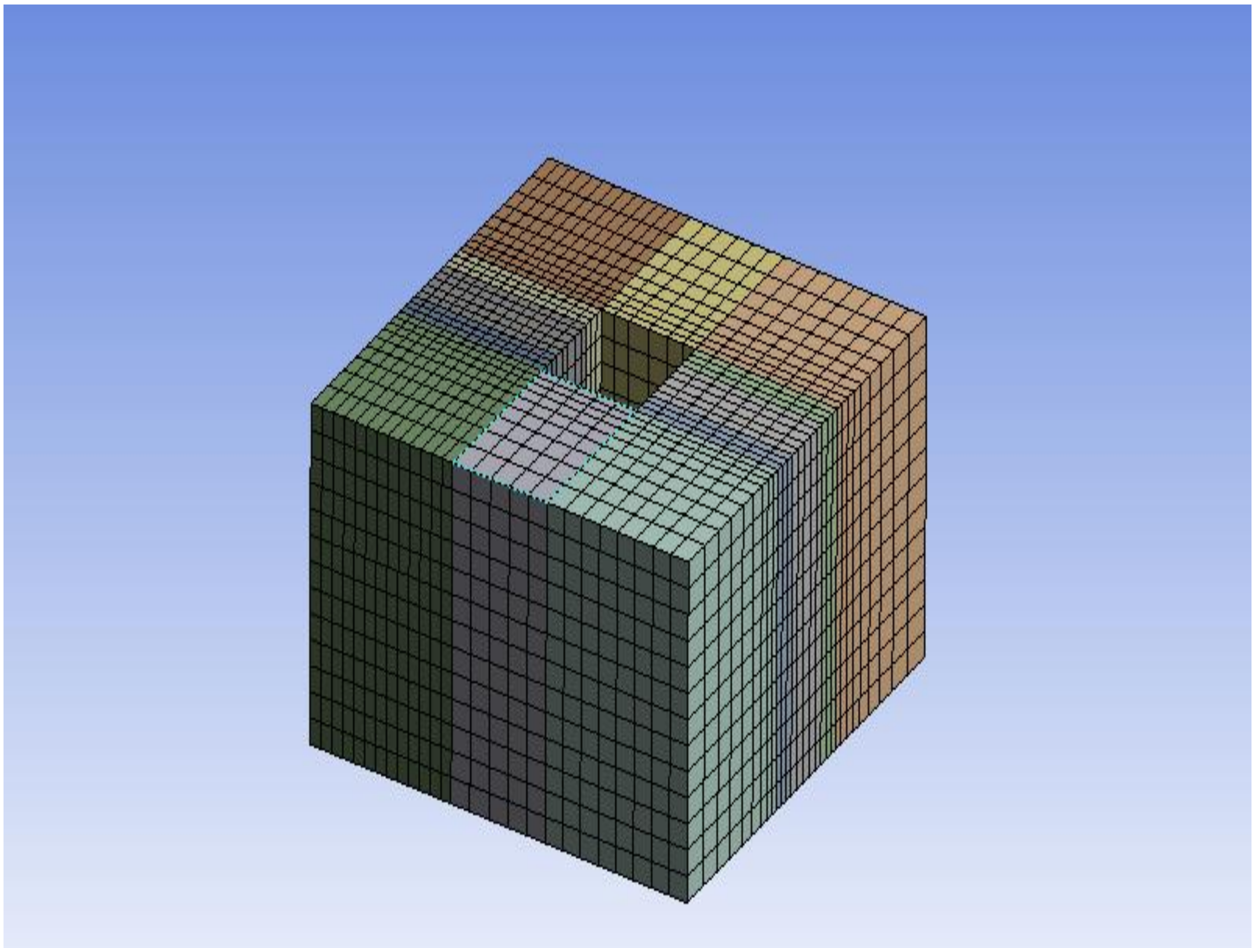


Figure 46 Meshing front view

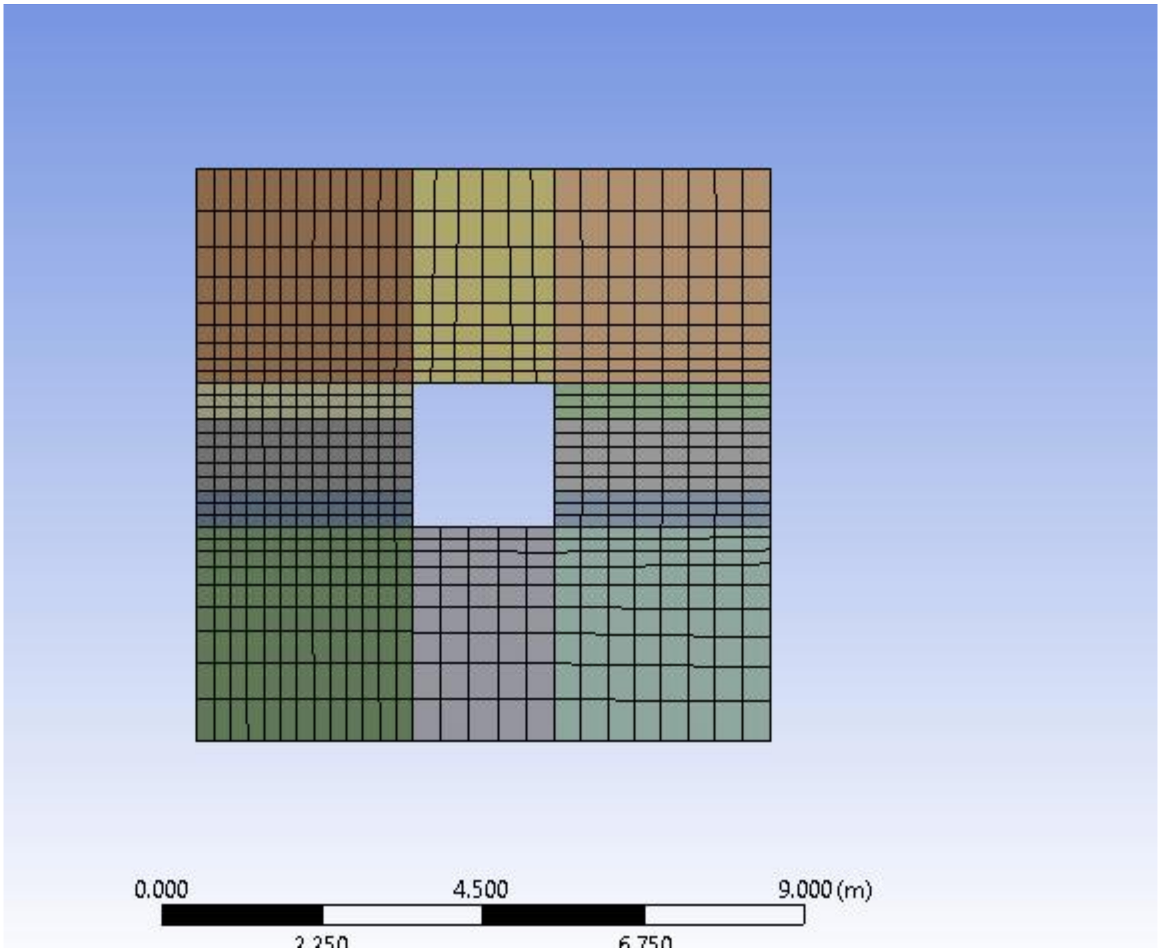
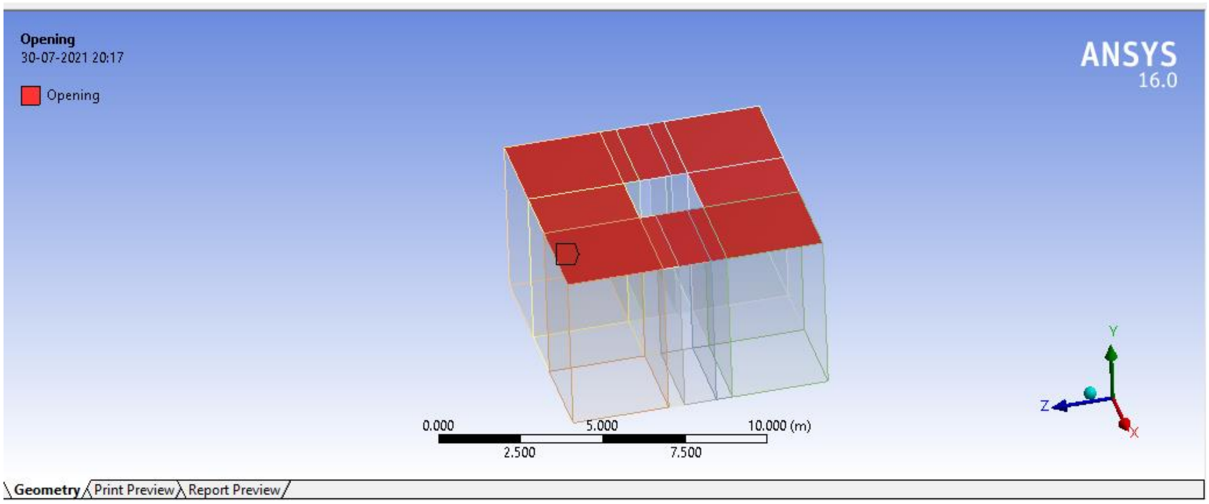


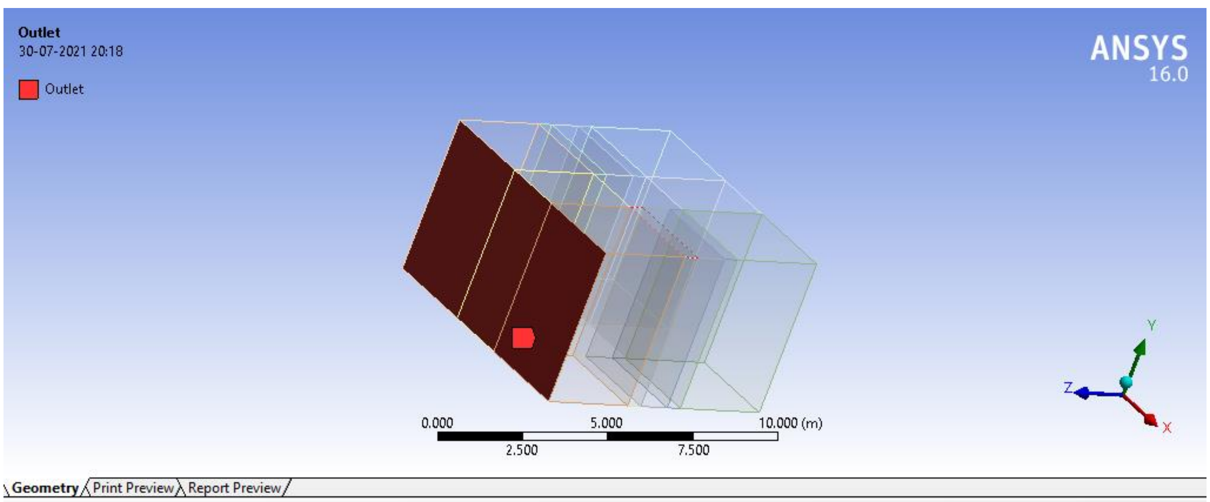
Figure 47 Meshing of top view

Name Sections:-

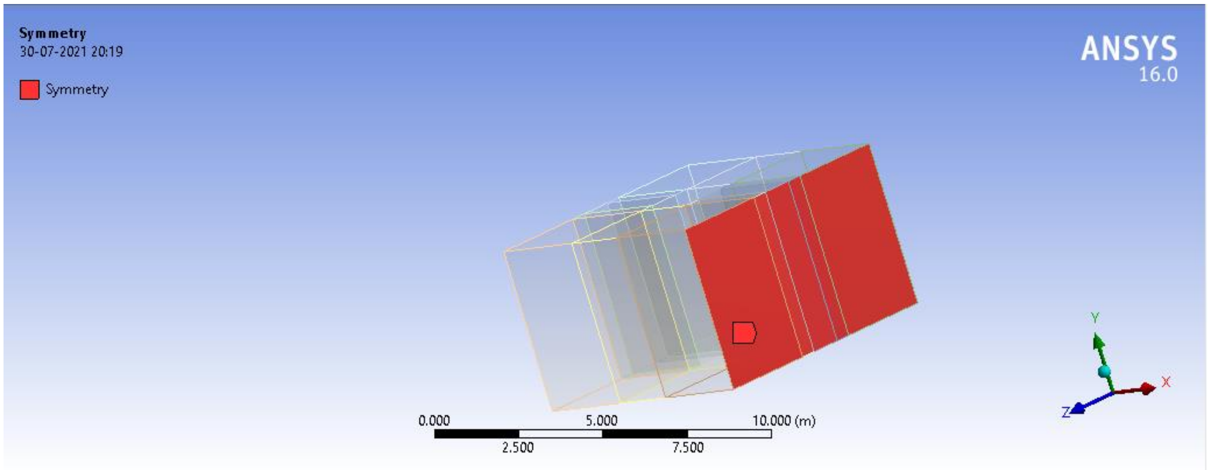
1) Opening:-



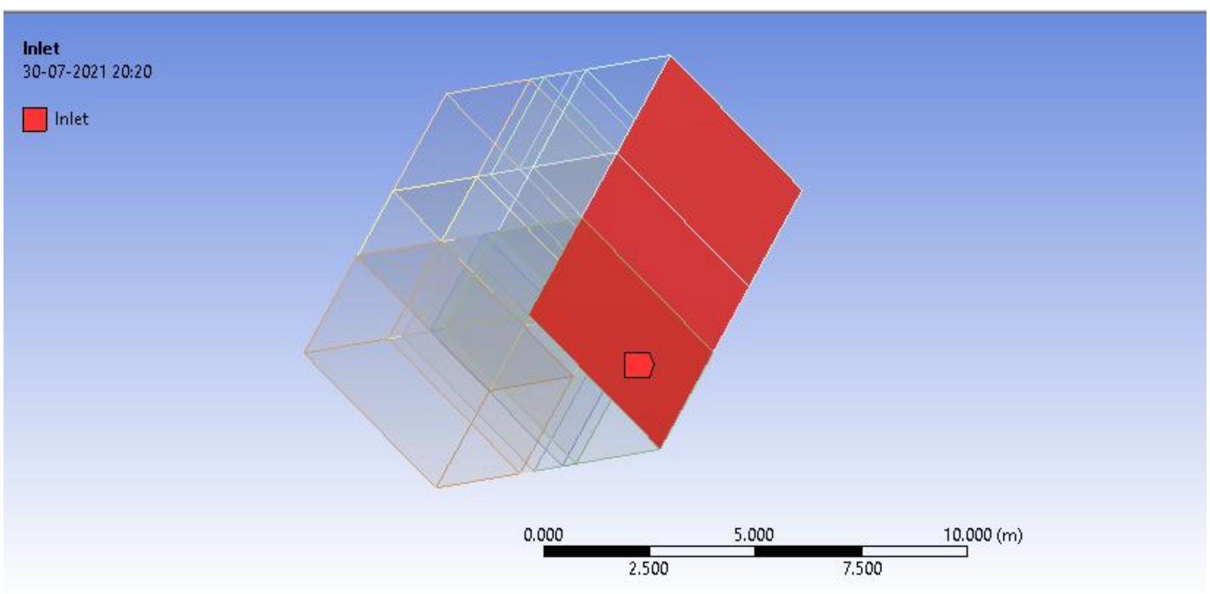
2) Outlet:-



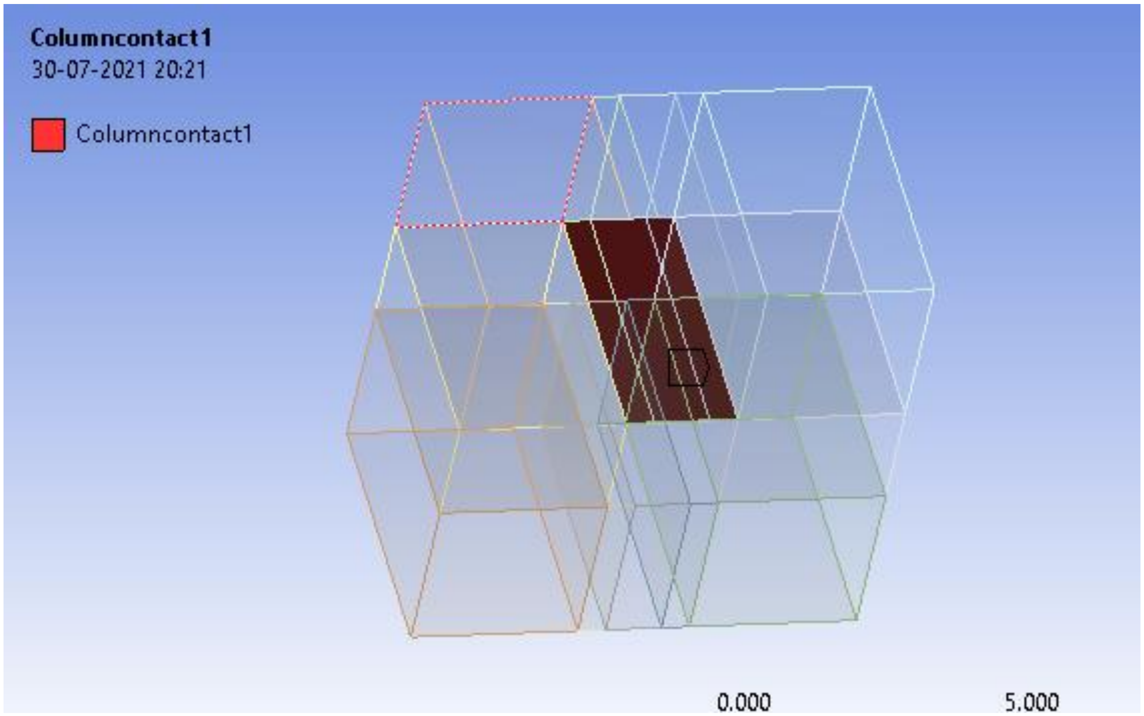
3) Symmetry:-


Symmetry
30-07-2021 20:19
Symmetry
ANSYS
16.0
0.000
5.000
10.000 (m)
2.500
7.500
Y
X
Z

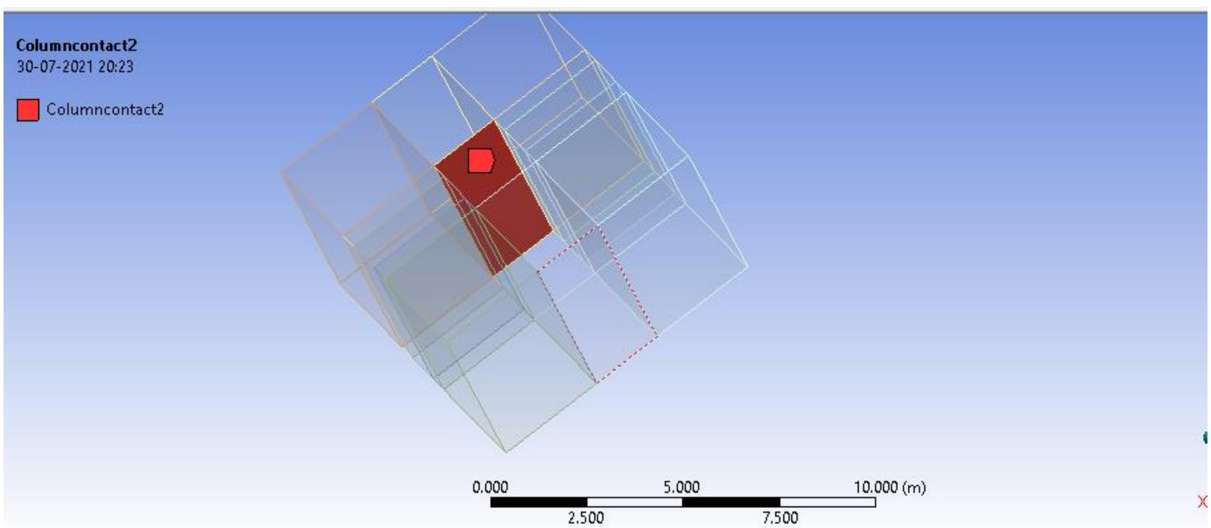
4) Inlet:-


Inlet
30-07-2021 20:20
Inlet
0.000
5.000
10.000 (m)
2.500
7.500

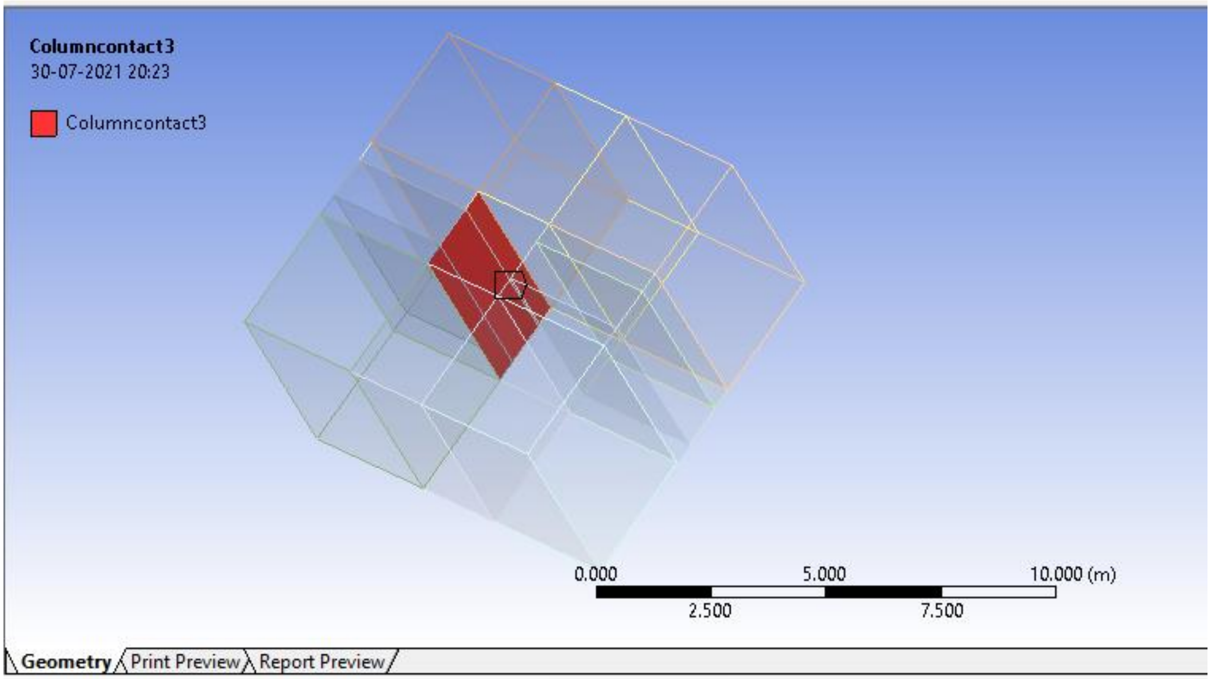
5) Columncontact 1:-



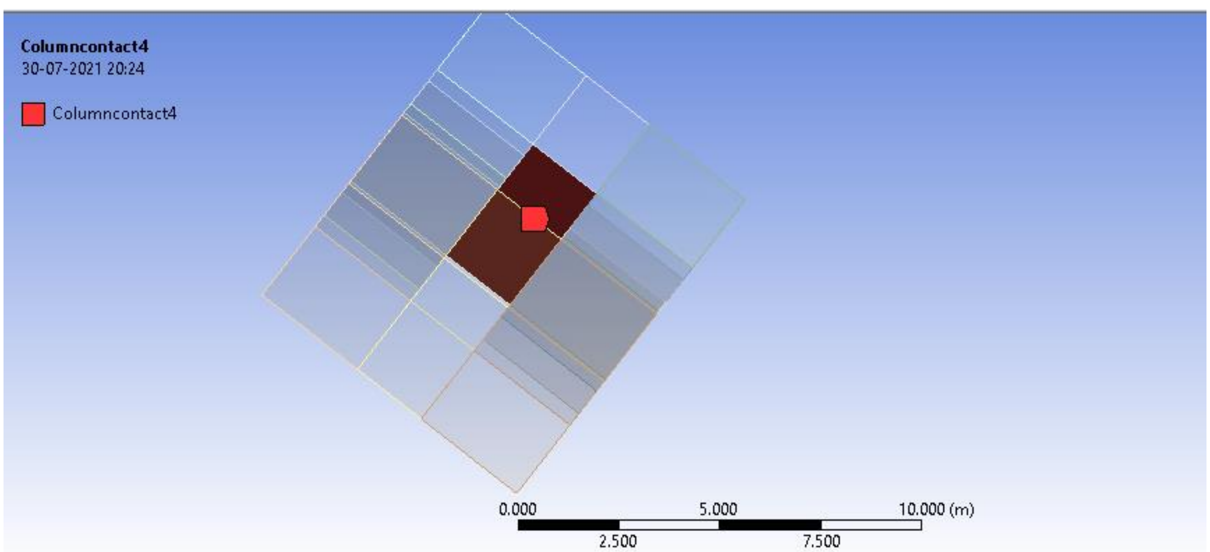
6) Columncontact 2:-



7) Columncontact 3:-



8) Columncontact 4:-



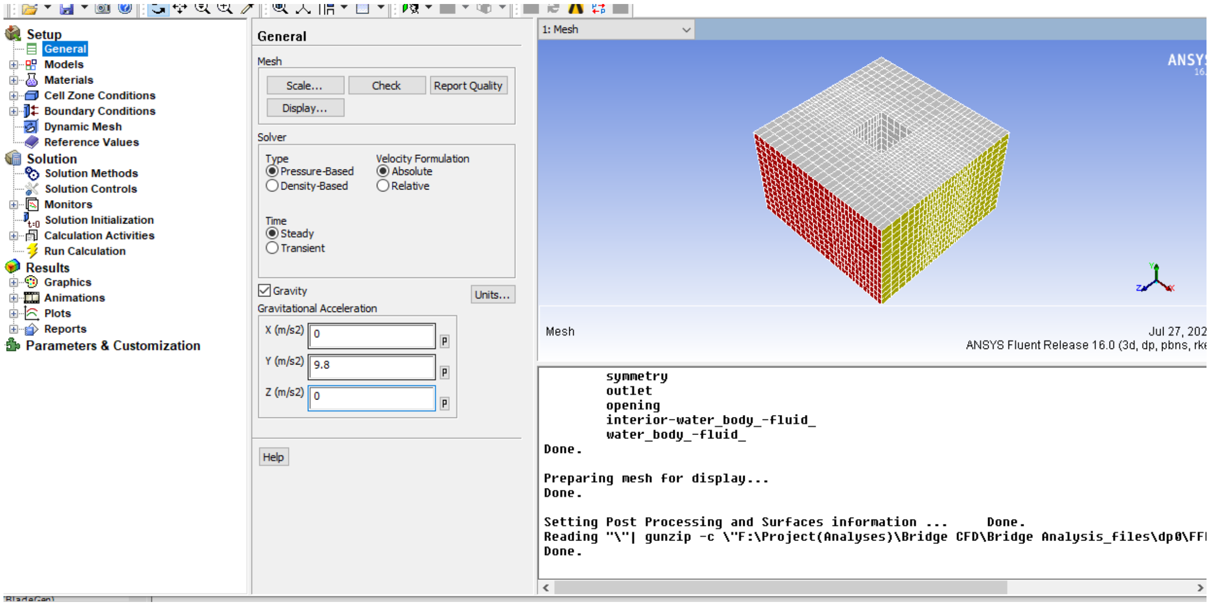


Figure 48 Setup for Flow

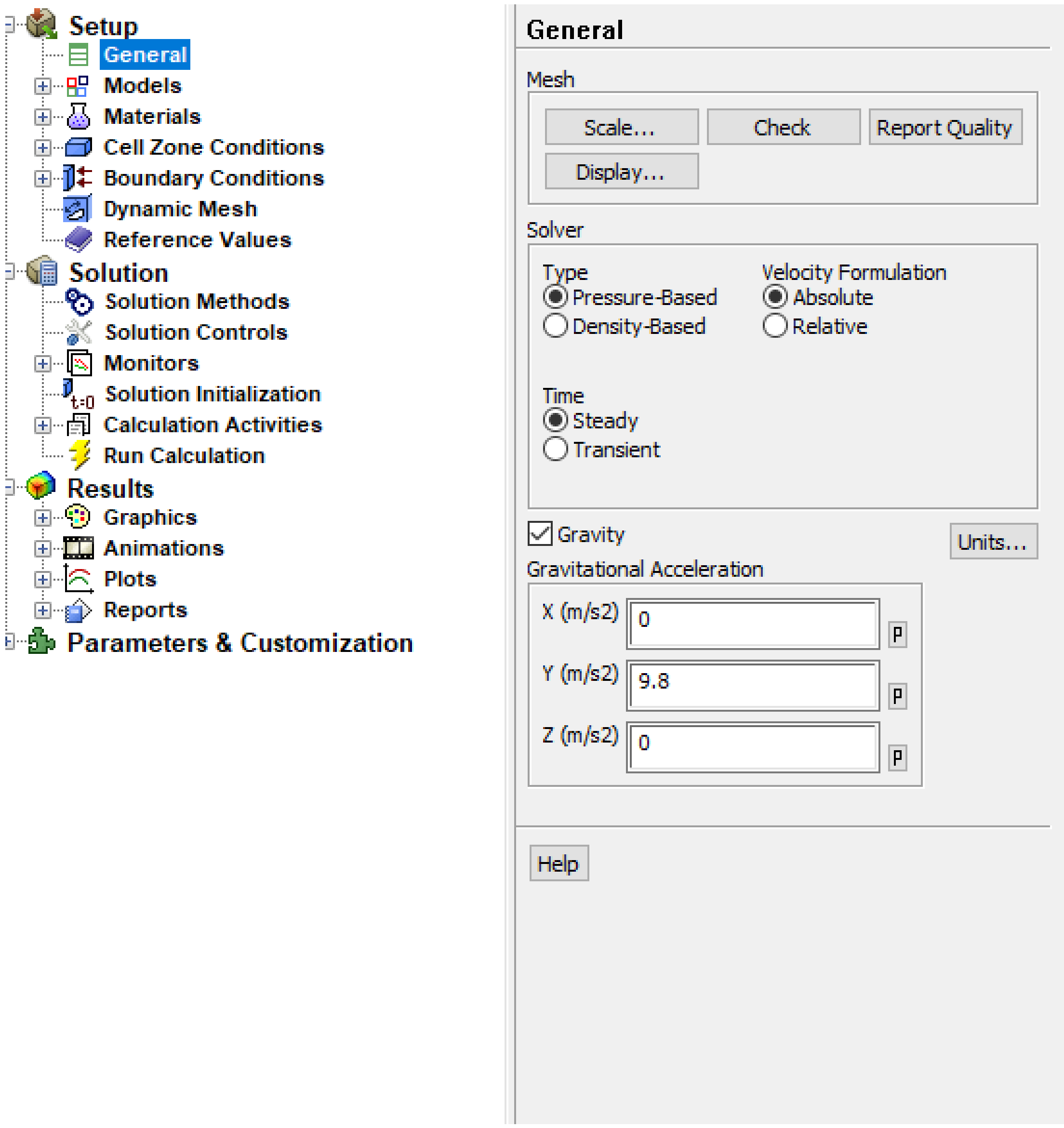


Figure 49 Setup

SET UP OF TURBULENT FLUID

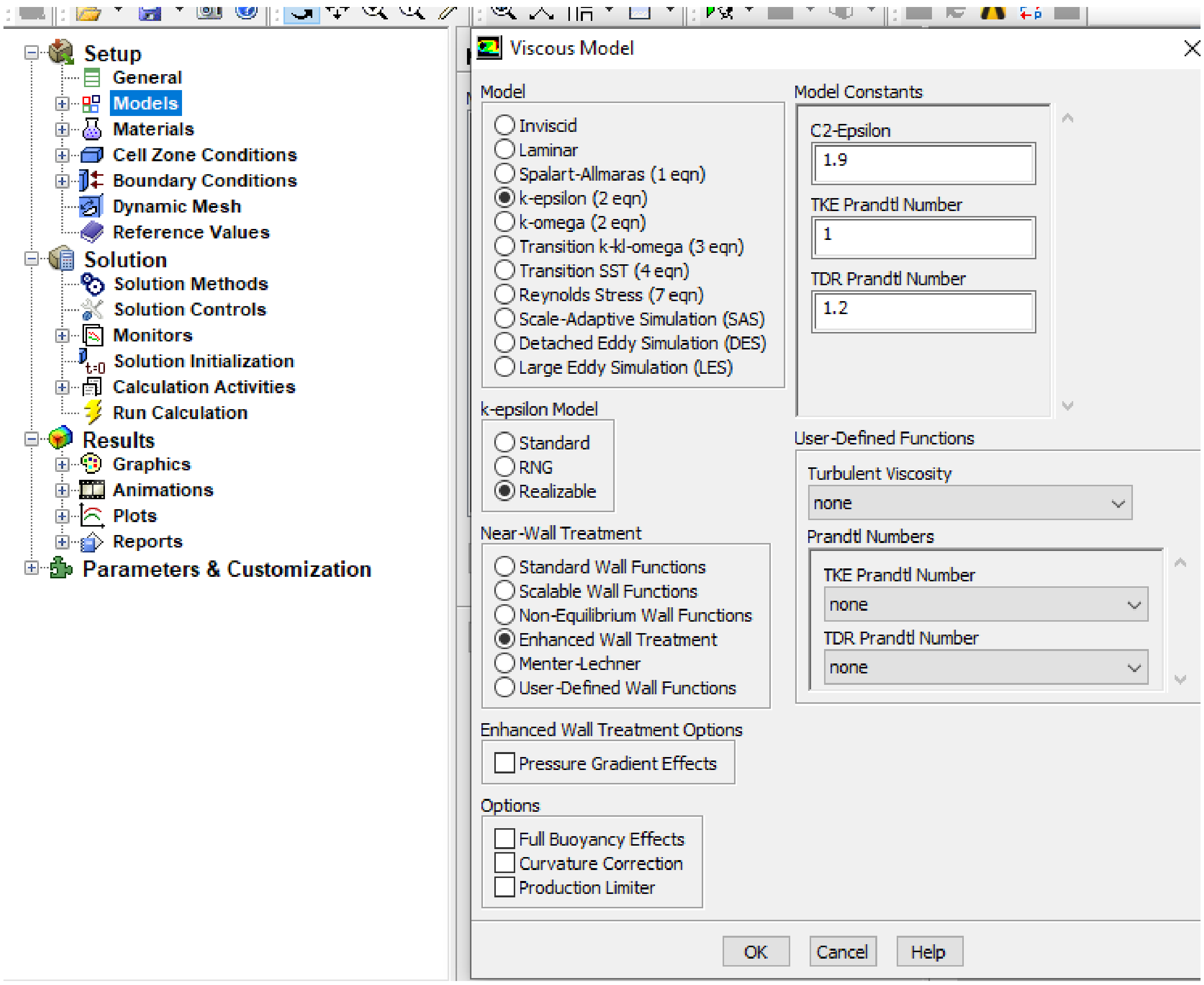


Figure 50 Set Up of Turbulent Fluid

SET UP PARAMETERS OF WATER

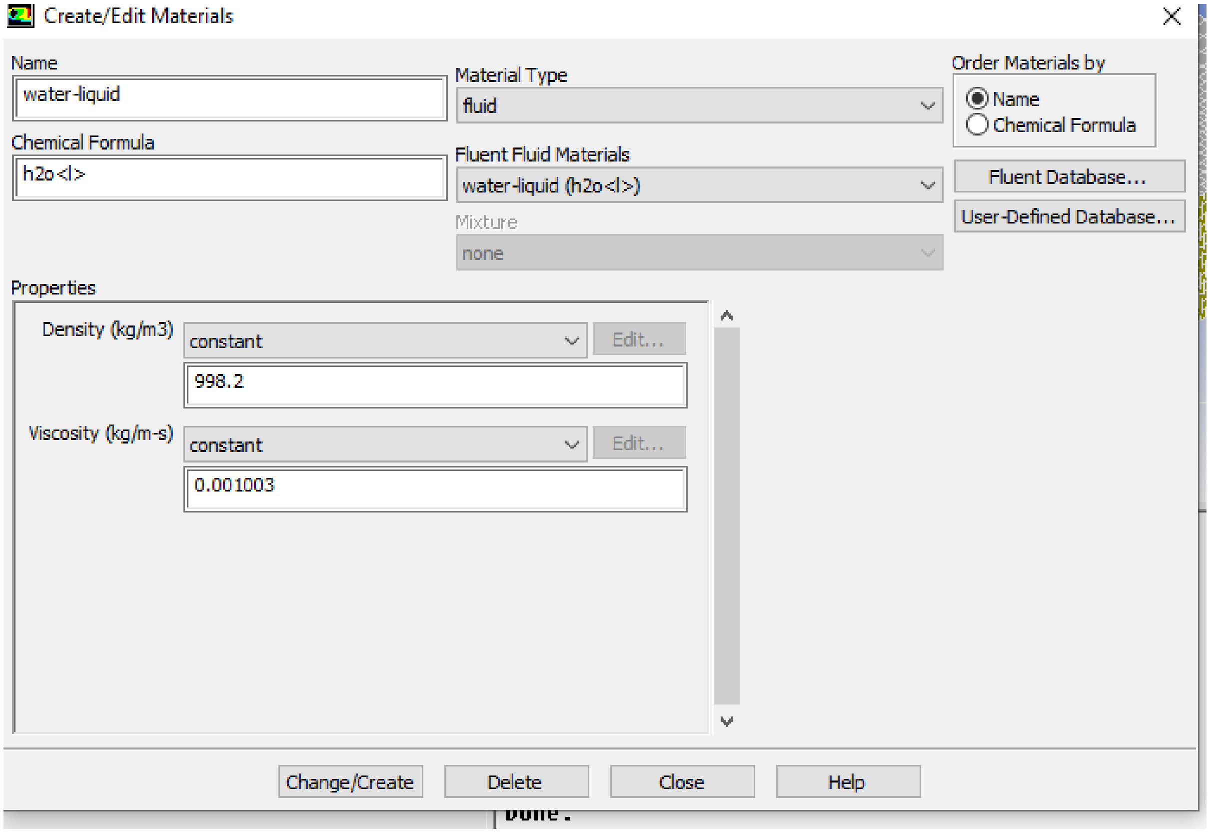


Figure 51 set up parameter of water

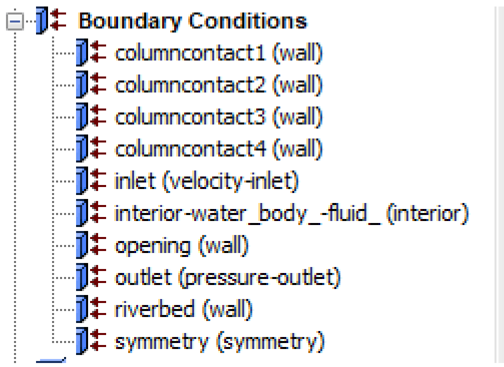


Figure 52 Boundary conditions

Solutions: -

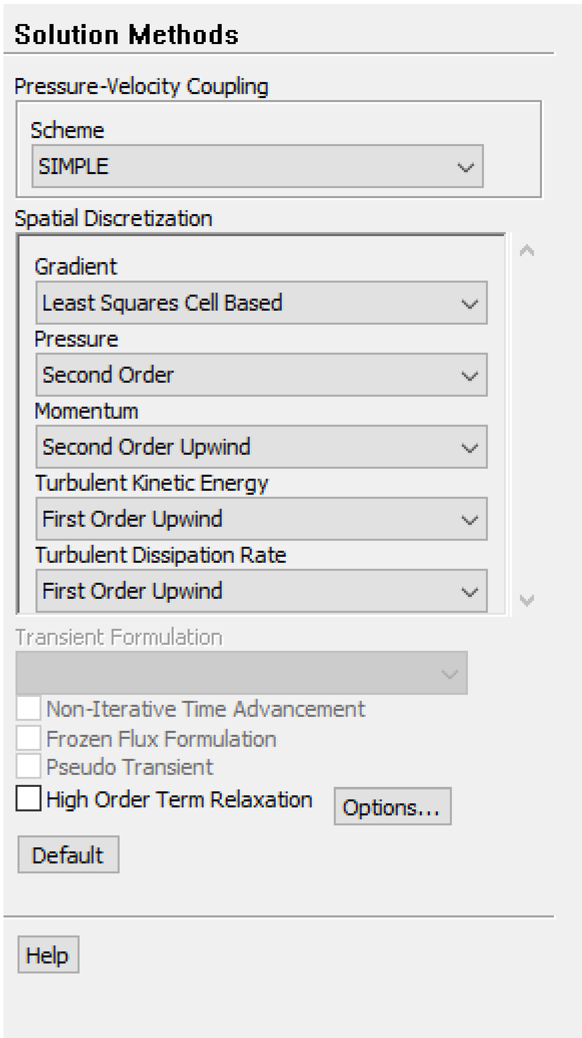


Figure 53 solution method

Solution Controls

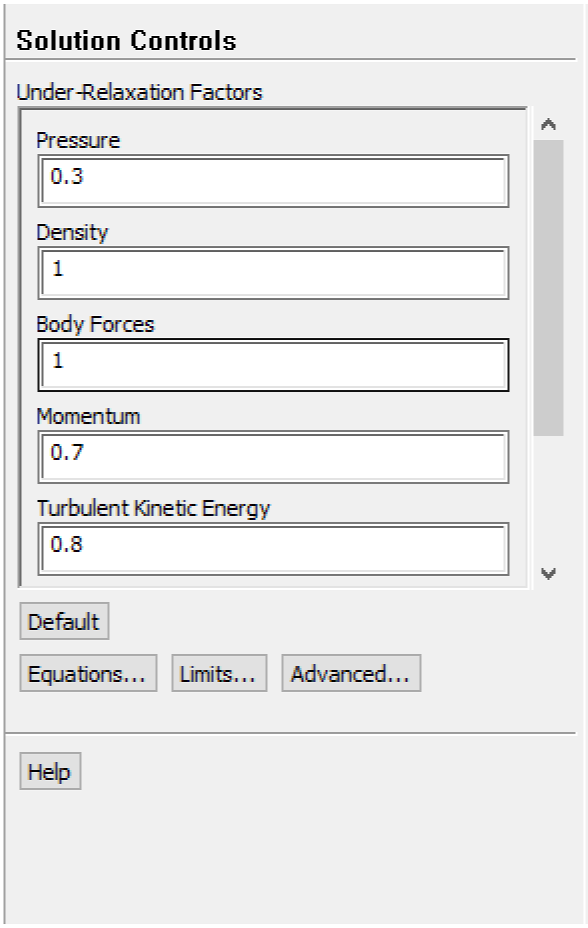


Figure 54 Solution Control

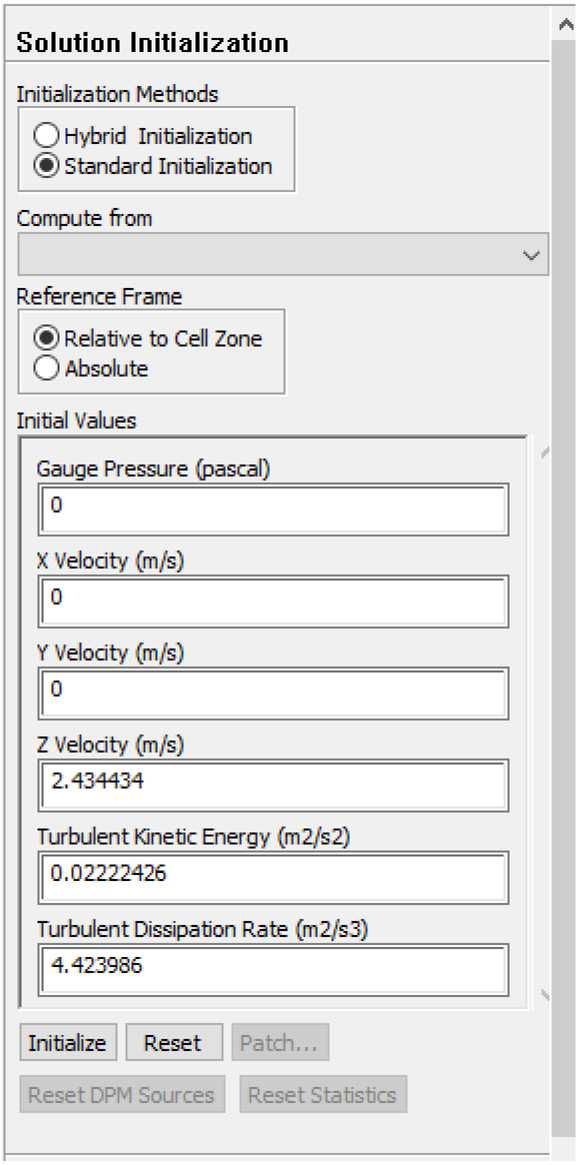


Figure 55 Solution initialization

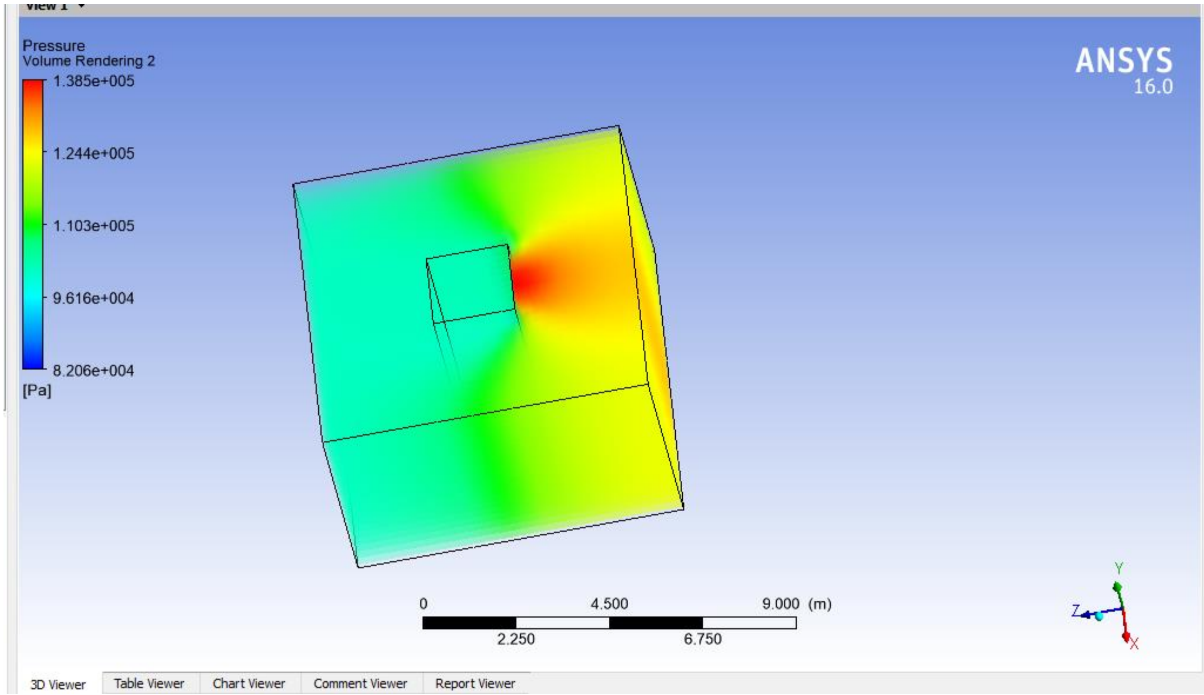


Figure 56 Pressure Volume

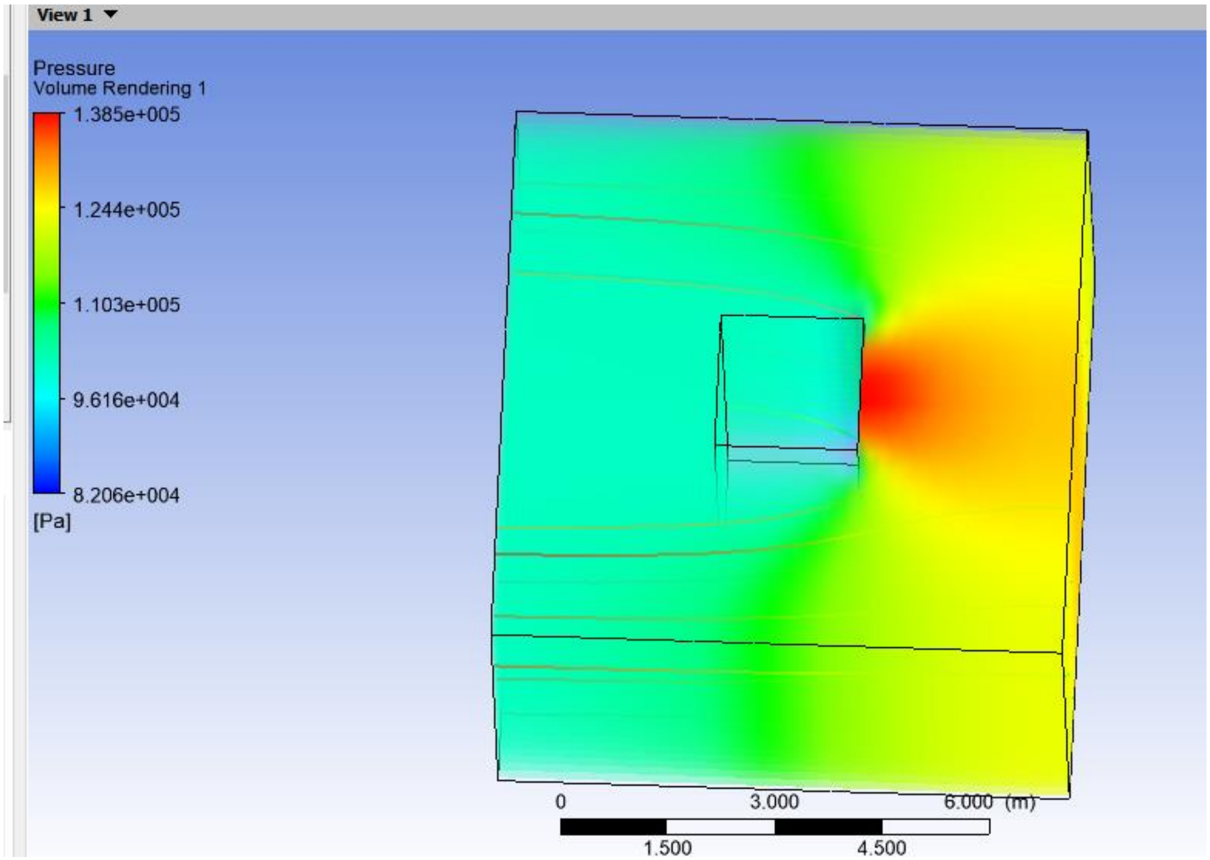


Figure 57 Presure Volume top view

Isometric View of Fluid Velocity

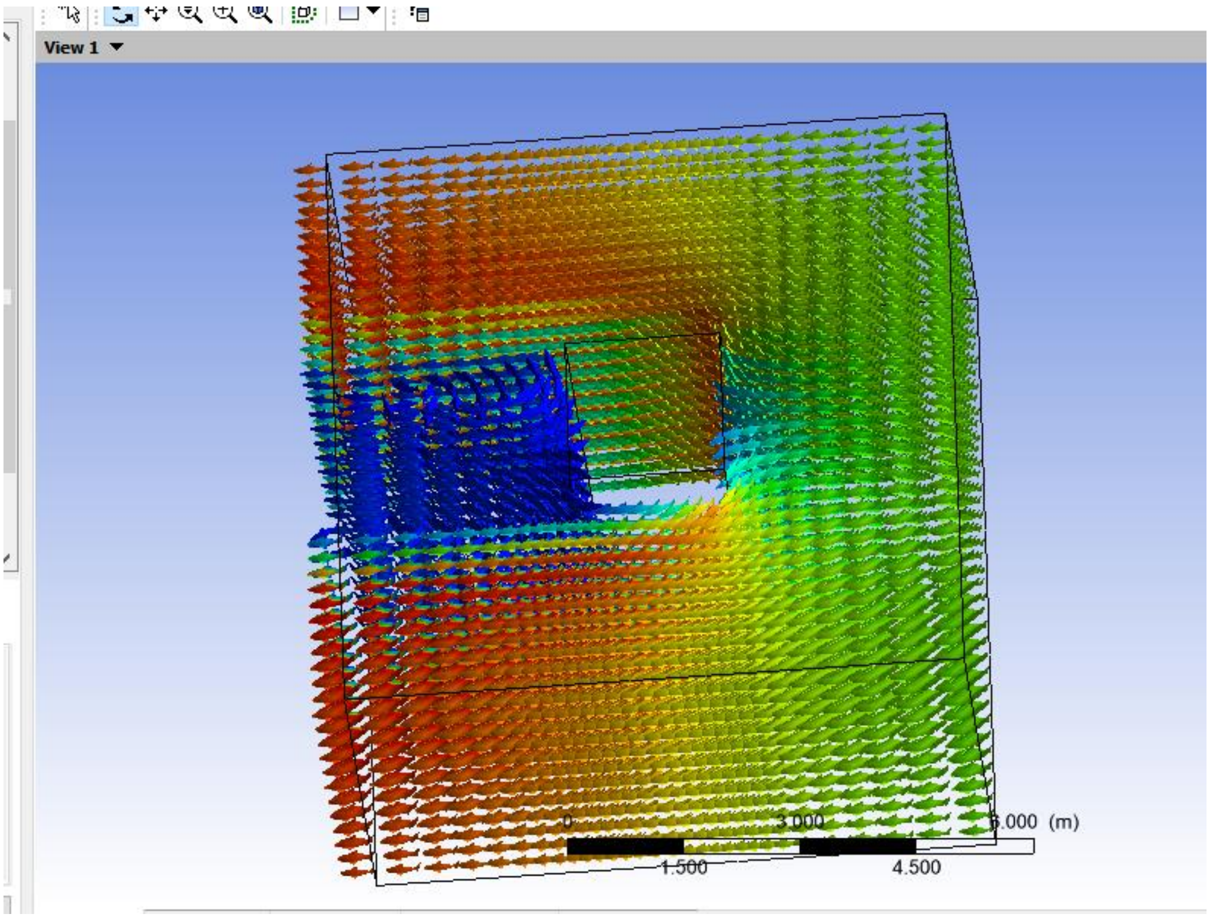


Figure 58 Isometric view of Fluid velocity

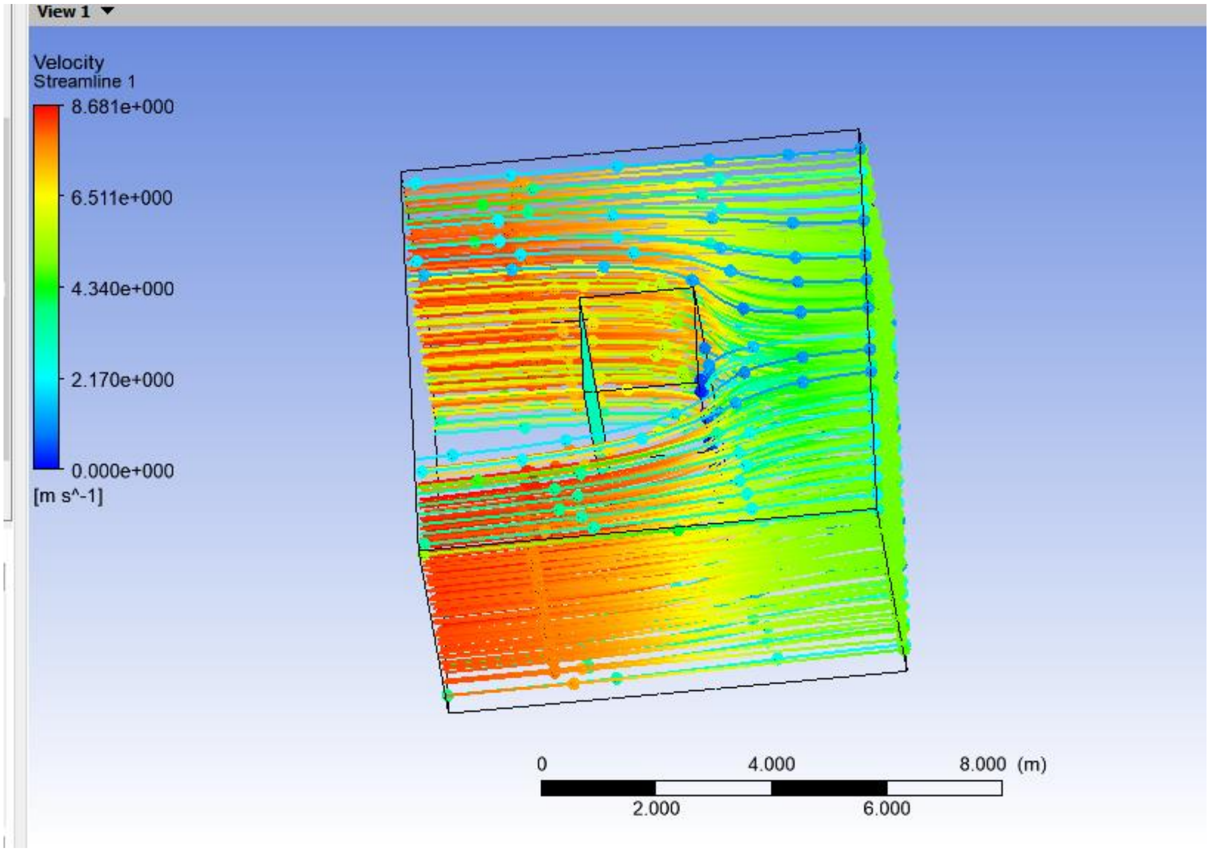


Figure 59 Velocity Streamline 1

Velocity Streamline 2

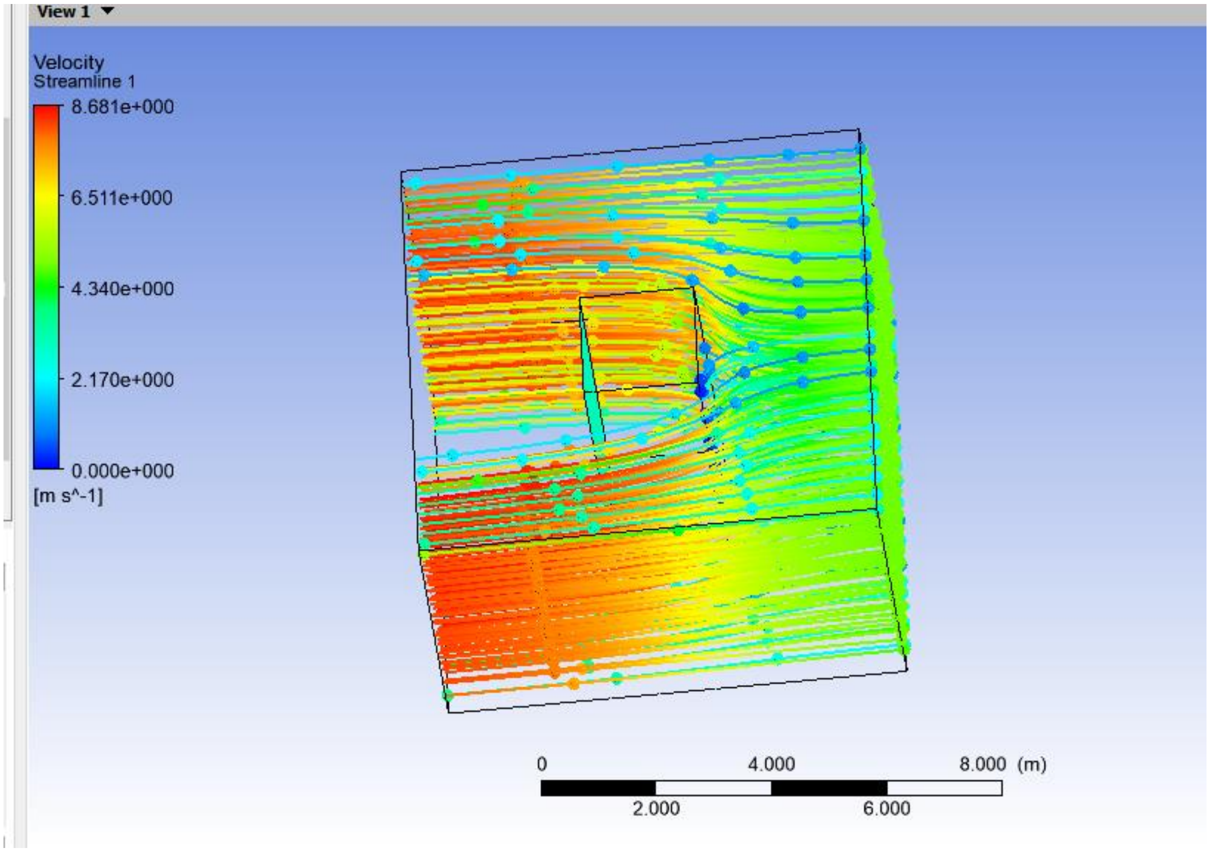


Figure 60 Velocity Streamline 2

Velocity Volume Rendering 1

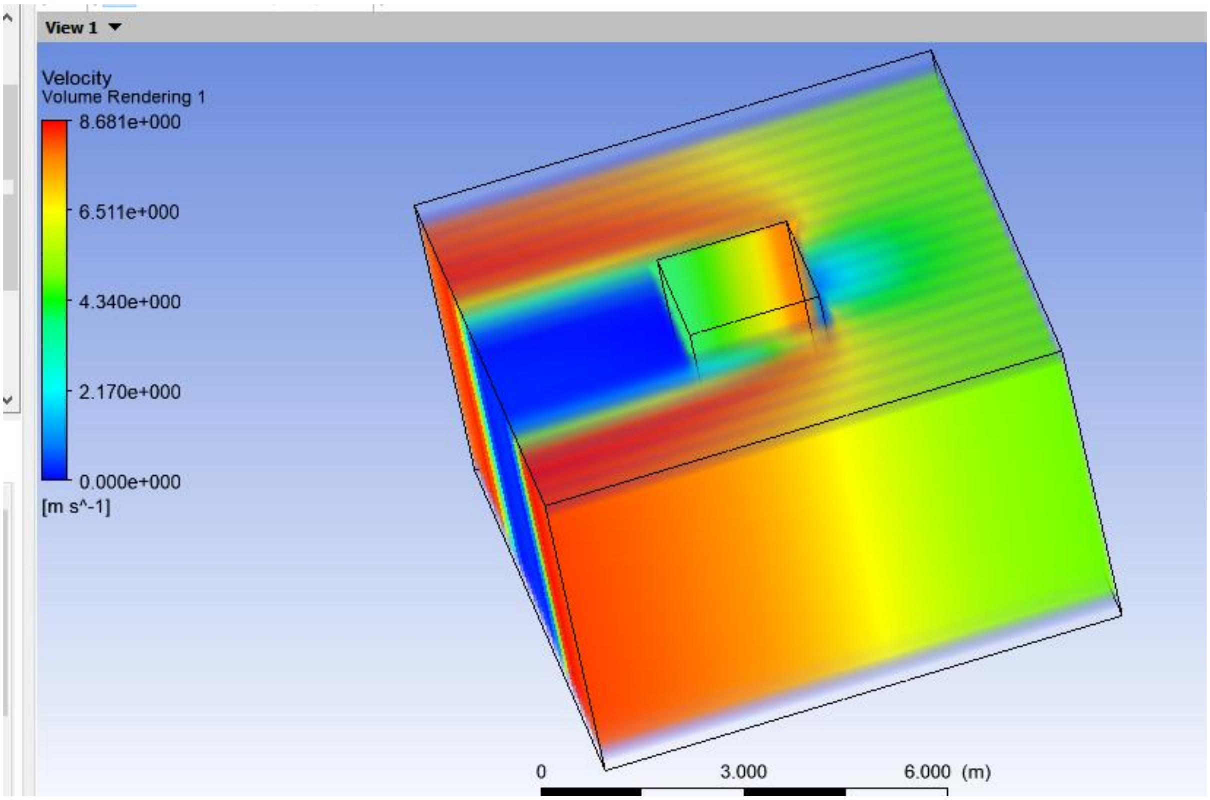


Figure 61Velocity Volume rendering 1

Model Tree

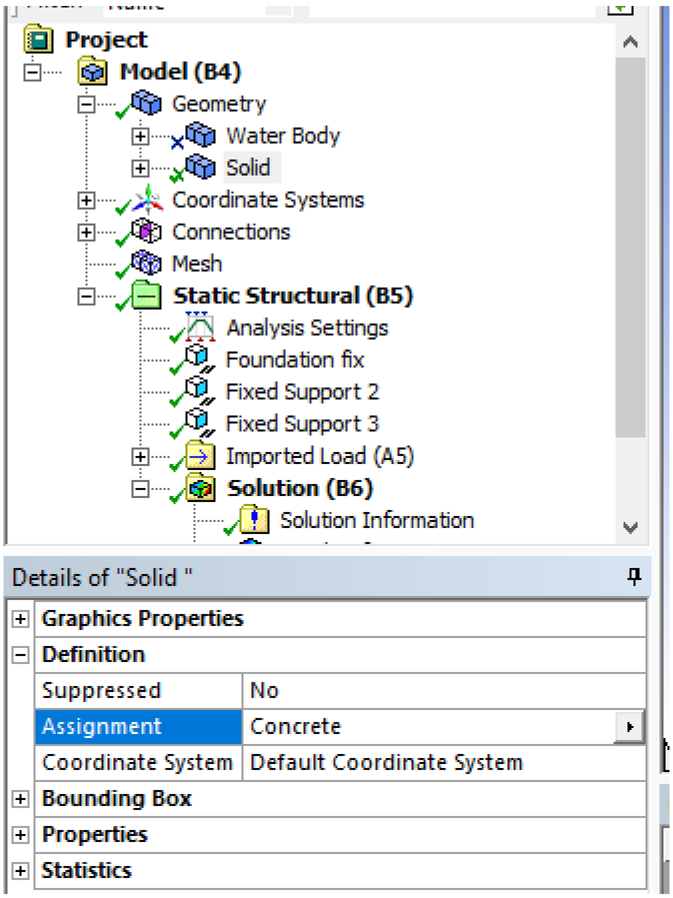


Figure 62 Model Tree

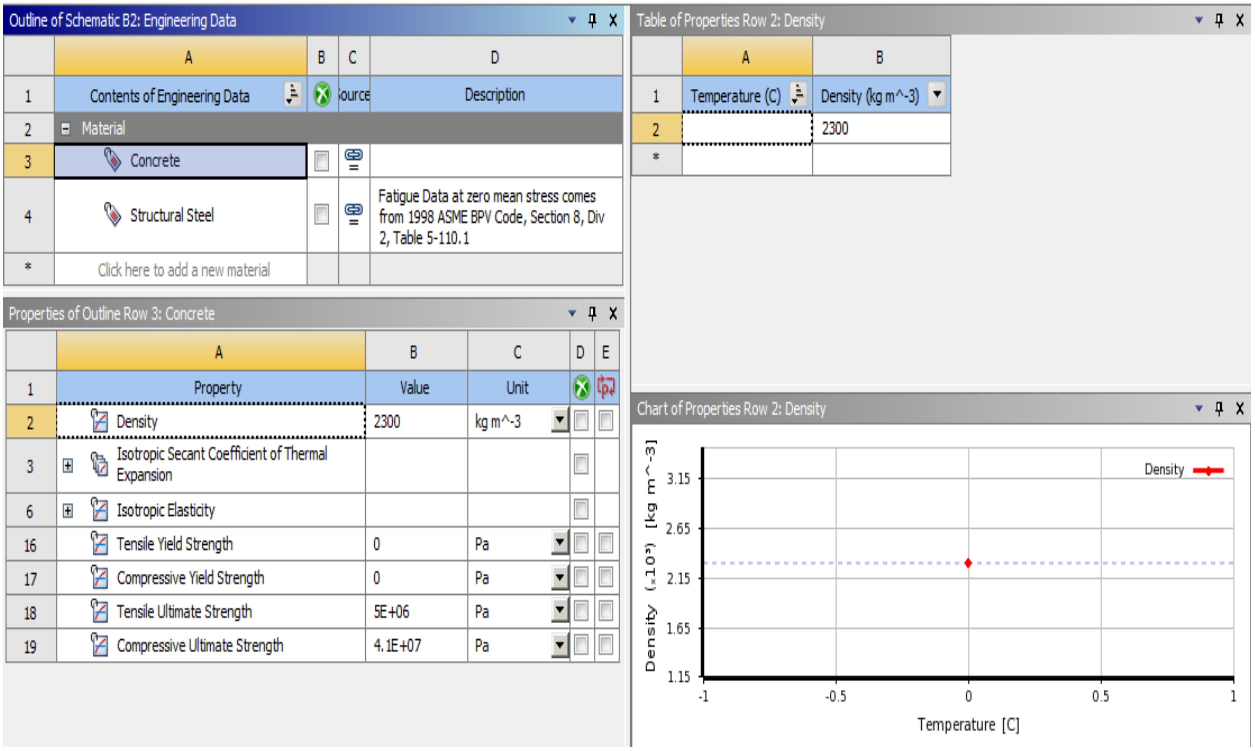


Figure 63 Material Property (concrete)

The second part of Fluid is to simulate the structures reaction to the pressure force from the water. From this we can obtain the maximum deformation and maximum normal stress in all axis.

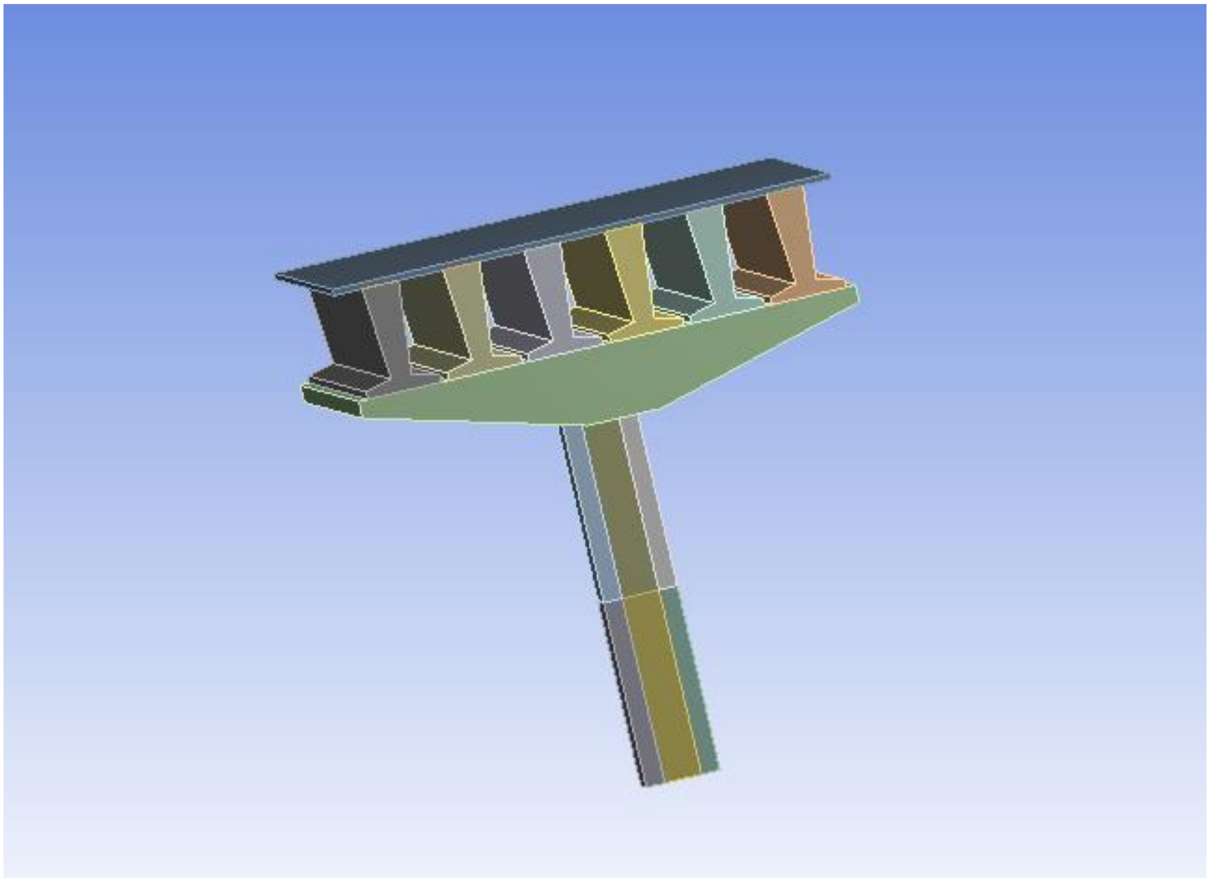


Figure 64 Isometric View of Bridge

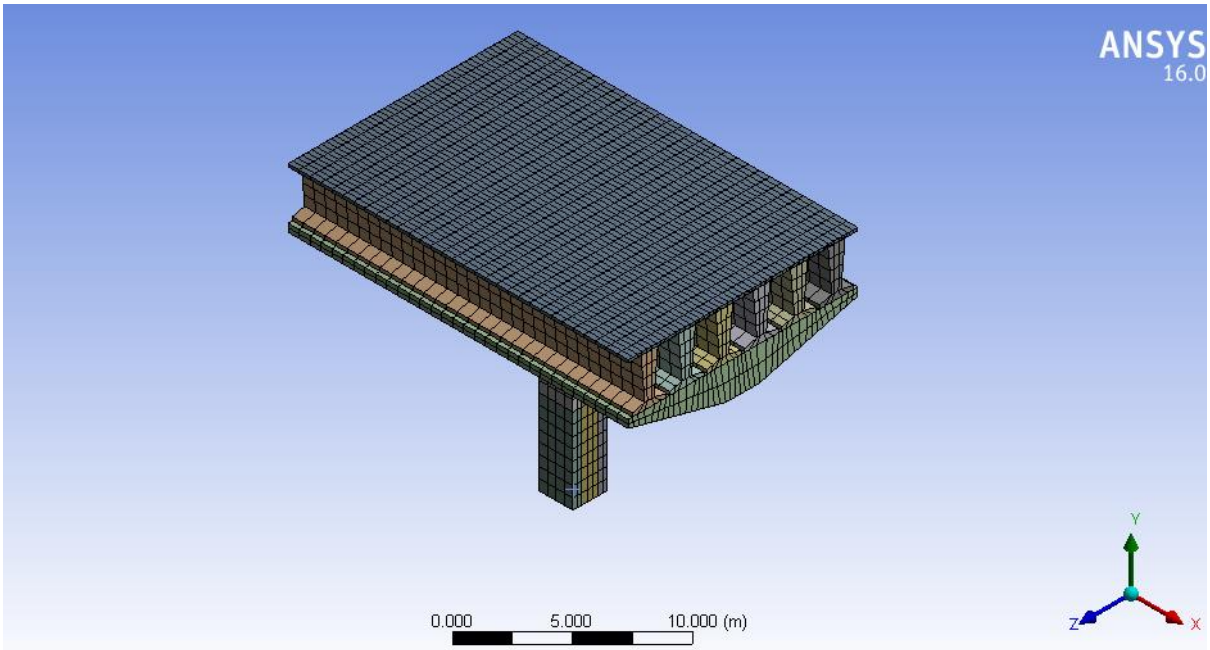


Figure 65 Meshing of solid Bridge

PIER FIXED SUPPORT

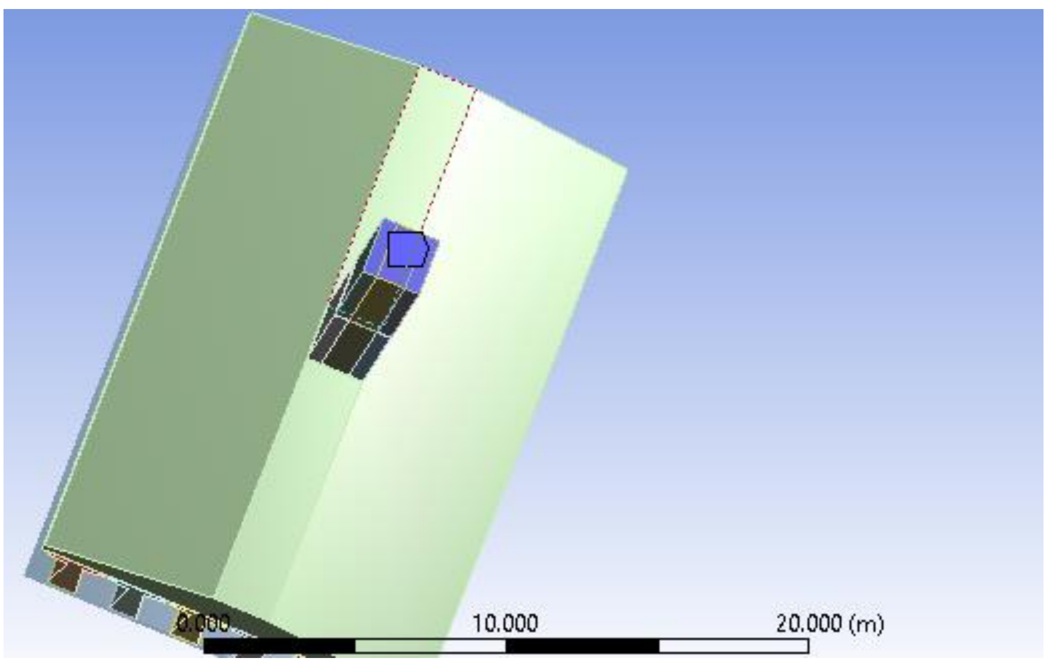


Figure 66 Pier Fix Support

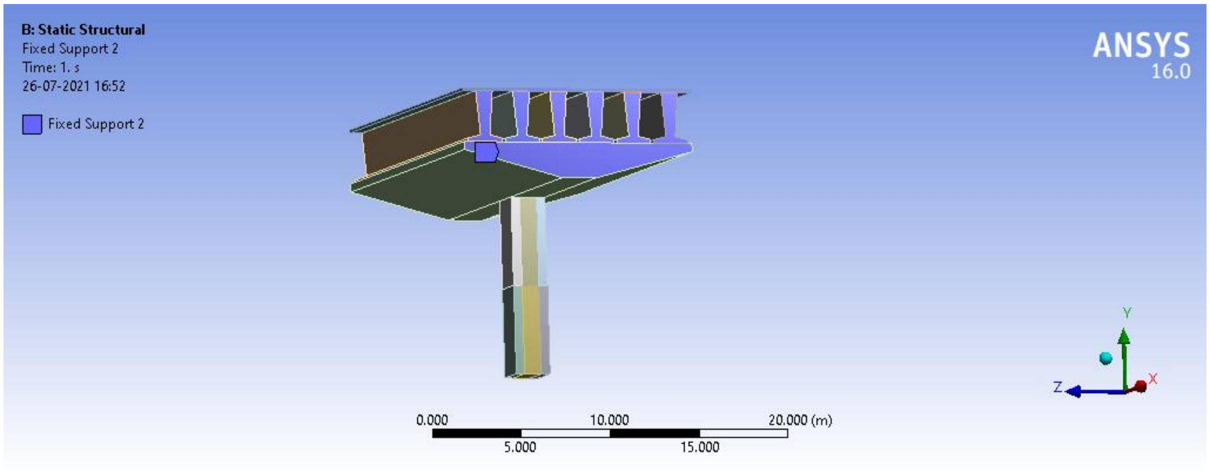


Figure 67 Left face fixed Support

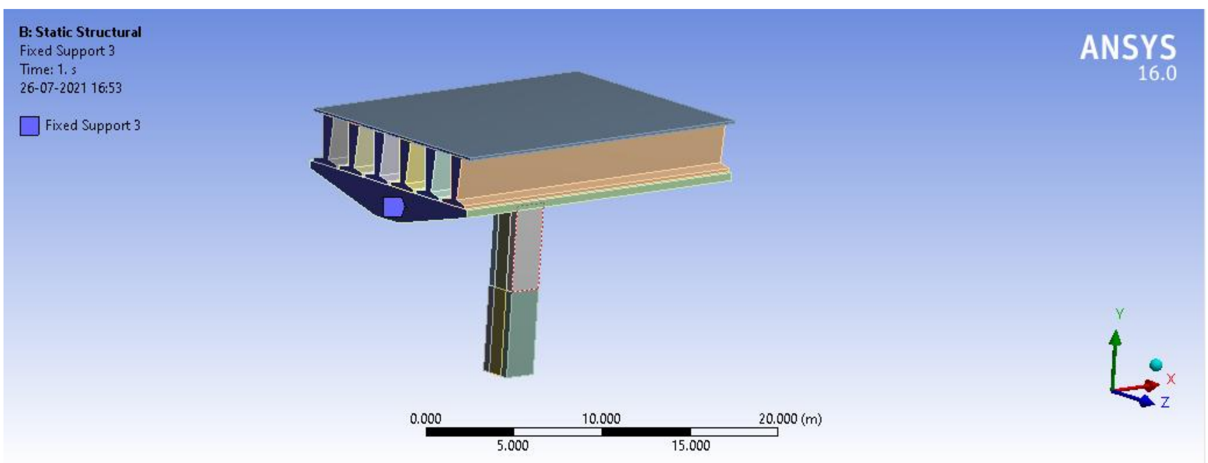


Figure 68 Right face fix support

Results

Finally, we execute the solve command to obtain the maximum deformation and maximum normal stress in all axis. And the result shown in below.

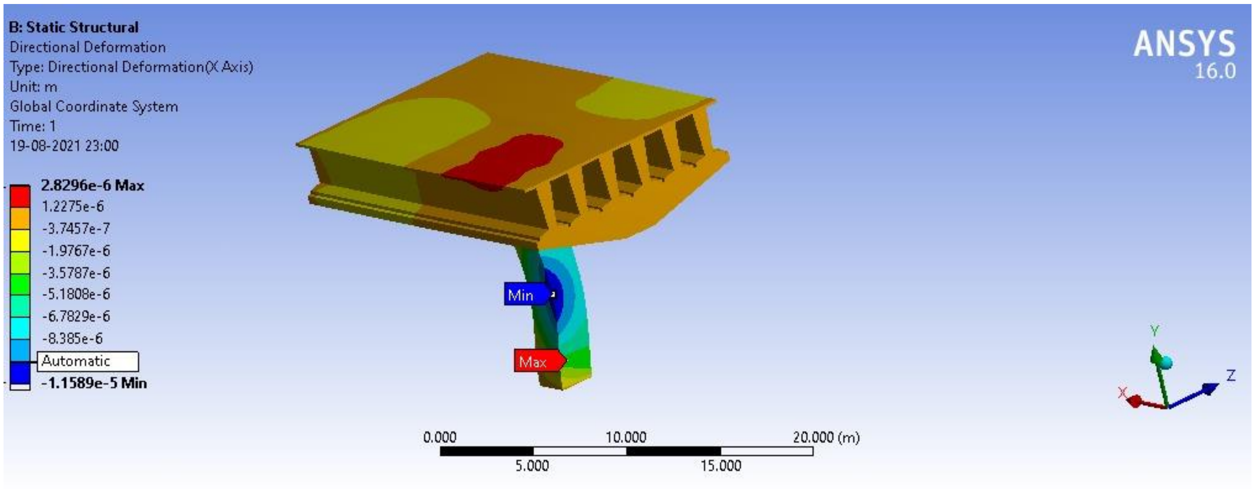


Figure 69 flood water Deformation, X axis

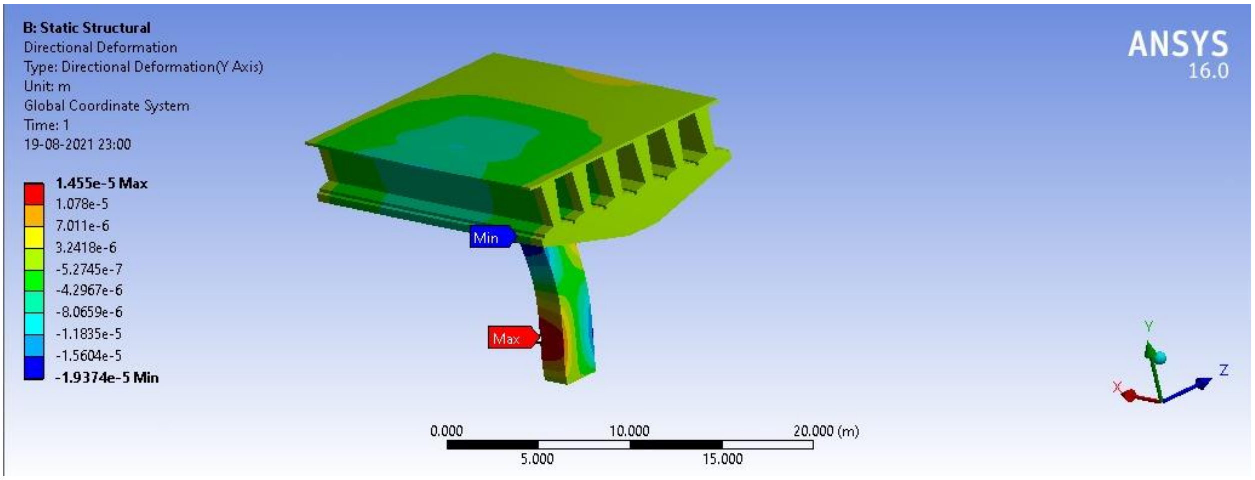


Figure 70 flood water Deformation, Y axis

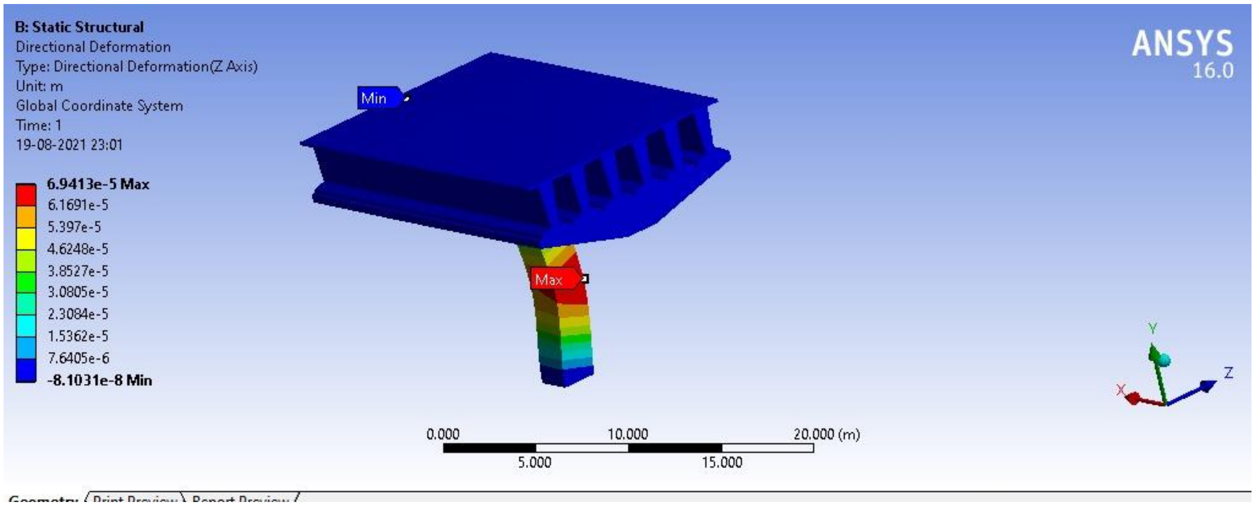


Figure 71 flood water Deformation, Z axis

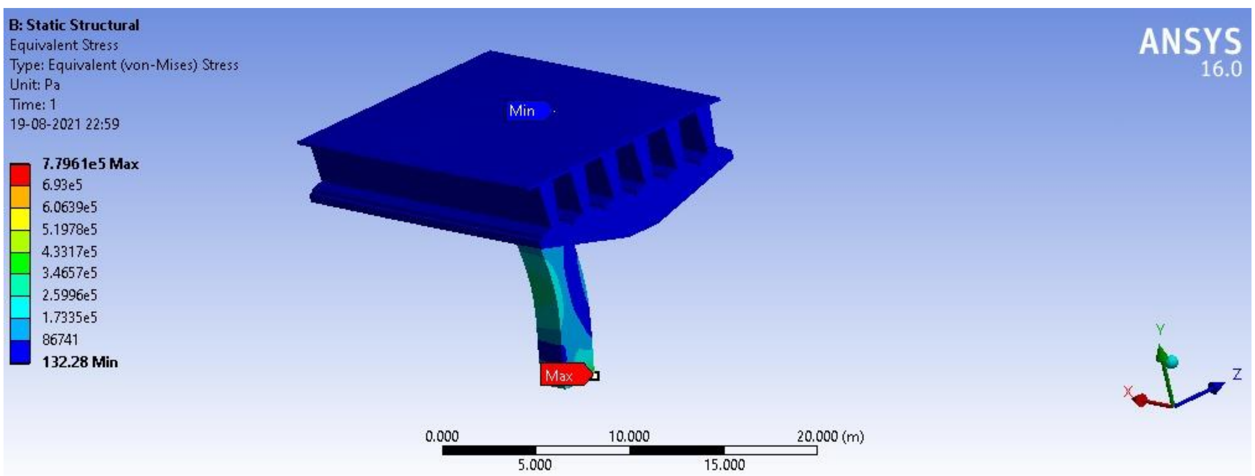


Figure 72 Equivalent stress

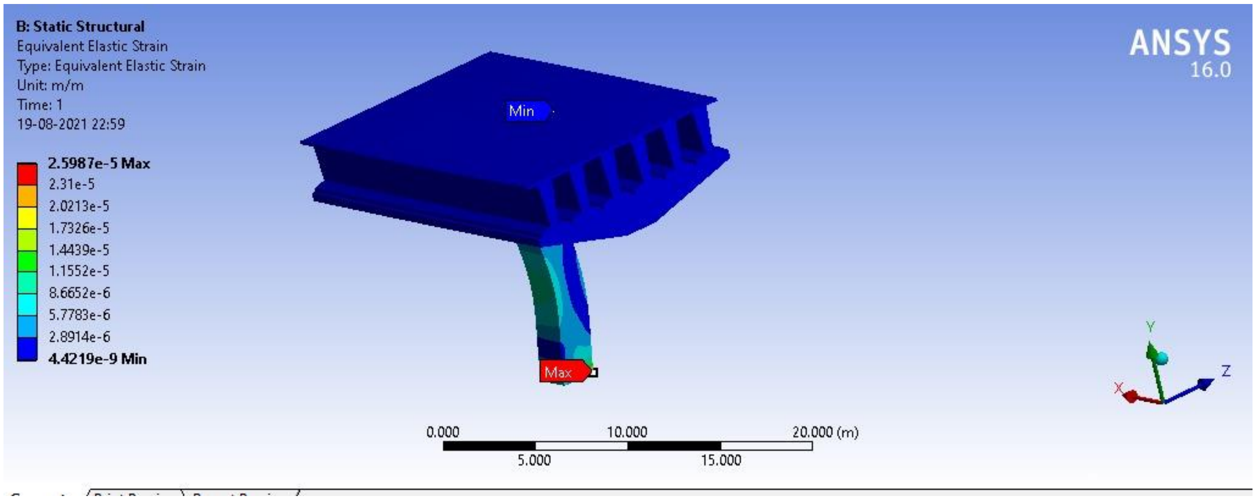


Figure 73 Equivalent Elastic Strain

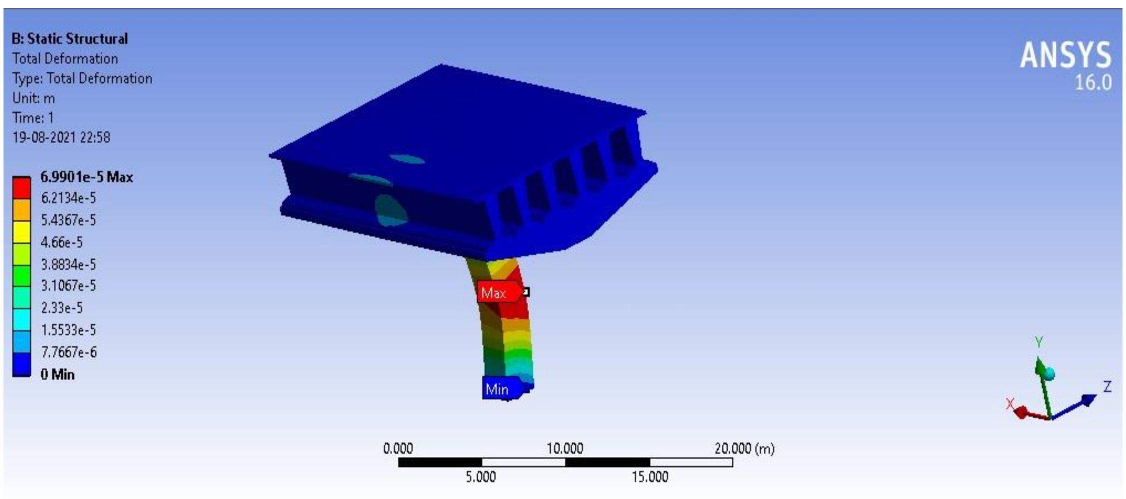


Figure 74 Total Deformation

Summery of model Result

Table 40 – Summery of Result

Total Deformation	6.990 x 10^-5
Equivalent elastic Strain	2.5987 x 10^-5
Equivalent stress	7.7961 x 10^ 5
Directional Deformation	
X axis	2.8296 x e^-6
Y axis	1.455 x e^-5
Z axis	6.9413 x e^-5

Rerences

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(BS 5400.4, 2006)

(BS 5400.2, 1996)

Standard, B. (2006) *BS 5400.4*, British Standard.

-Standard, B. (1996) *BS 5400.2, British Standard.*

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