Department of Civil Engineering

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CE402

The final report of a Multi-Disciplinary Design Project

Prepared in partial fulfilment of the requirements of

the degree of Bachelor of the Science of Engineering

DESIGN OF A NEW BRIDGE TO REDUCE TRAFFIC CONGESTION IN PERADENIYA JUNCTION

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EXECUTIVE SUMMARY

The existing Peradeniya Bridge is one of the most prominent bridges on Kandy - Colombo, and Kandy – Gampola roads in Sri Lanka. Current traffic congestion on Peradeniya Bridge has become severe during peak hours and therefore development of infrastructure facilities to ease the traffic movement through the Peradeniya Bridge has become an urgent requirement. Considering all these facts, a basic design study has been conducted to design and construction of a new bridge parallel to the existing bridge, addressing all possible issues which may arise with the new project.

Within the study boundary in Peradeniya, the existing situation of project location was studied by frequent field visits and reading previous proposals. Then, suitable alternatives were identified which were best suited to solve the existing problem. After that, most viable solution was selected for the detailed design. In the selected solution, a new two lane bridge parallel to the existing bridge was proposed along with the widening of A1 and A5 roads from Gannoruwa junction to Penideniya junction. Moreover, a new approach road was designed to connect A1 road with the new bridge. Additionally, the riverbank slope protection was designed using adequate measures. Furthermore, to overcome the traffic congestion problem at the Peradeniya junction a signalized traffic control network and an underpass were designed.

Finally, a detailed Bill of Quantity was prepared to estimate the construction cost of the project and well-established environmental impact assessment was carried out to minimize the environmental impacts. This design was created to reduce the traffic congestion and to improve the commuter's comfort and transform Peradeniya to a more aesthetic place.

ACKNOWLEDGEMENTS

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Furthermore, we wish to acknowledge the help and support offered by Mr. Wijenayake the design engineer of Road Development Authority, Kandy for the valuable directive information given to us about the existing bridge and the factors we should consider when we are implementing the project.

We are also grateful for the immense support and assistance given by Prof. Jayawardana and Dr. Atapattu by giving the geological data of the existing rock and soil data near the Peradeniya Bridge.

Finally, we would like to offer our gratitude to others whom we could not mention here, but helped us by assisting in numerous ways.

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LIST OF ABBREVIATIONS

2D	Two Dimensional
2W	Two-wheeler
3D	Three Dimensional
3W	Three-Wheeler
ABC	Aggregate Base Course
CBR	California Bearing Ratio
DEM	Digital Elevation Model
ESA	Equivalent Standard Axle
LM	Load Model
LOS	Level of Service
LWAF	Lane Width Adjustment Factor
OMC	Optimum Moisture Content
PAF	Pedestrian Adjustment Factor
PCE	Passenger Car Equivalents
PCU	Passenger Car Units
PCV	Passenger Car Volume
RCC	Reinforced Cement Concrete
RDA	Road Development Authority
RQD	Rock Quality Designation
RT	Right Turn
SLS	Serviceability Limit State
SSD	Stopping Sight Distance

STM	Strut and Tie Model
TRB	Transportation Research Board
TS	Tandem System
UDL	Uniformly Distributed Load
ULS	Ultimate Limit State
WBM	Water bound macadam

LIST OF SYMBOLS

A’	Effective area
Ac,eff	Effective concrete area
As	Reinforcement area
As,min	Minimum reinforcement area
As,prov	Provide reinforcement area
B’	Effective width
d	Effective depth
dc	Depth to neutral axis
e	Eccentricity
I	Second Moment of area
L’	Effective length
MEd	Design bending moment
Mqp	Moment due to long term action
MSLS	Serviceability bending moment
Mst	Moment due to short term action
MULS	Ultimate bending moment
ϕ	Diameter
u	Control perimeter
VRd,c	Shear resistance without shear reinforcement
Wk	Crack width
σc	Concrete stress

CHAPTER 1

INTRODUCTION

1.1 PROJECT OVERVIEW

The national transport infrastructure is an essential component of the social and economic development of Sri Lanka. Roads account for more than 90% of transport in Sri Lanka. Most of the existing roads were constructed more than 50 years ago. Road maintenance, rehabilitation, and construction have not kept abreast of the rapid growth of transport demand, which resulted negatively on user costs, road safety, and economic development. Therefore, projects that can improve the quality and condition of the existing transport infrastructure is an utmost requirement.

Peradeniya is located 6km away from Kandy and bounded by Mahaweli River on the east and Peradeniya Mountain on the west. The town center has been evolved due to the location at a key road intersection of A1 road and A5 road, where physical development is inevitable with high road connectivity. The Peradeniya Bridge which is a key link for transportation as both A1 and A5 roads connect the Kandy city through it. Figure 1.1 shows the location of the existing Peradeniya Bridge

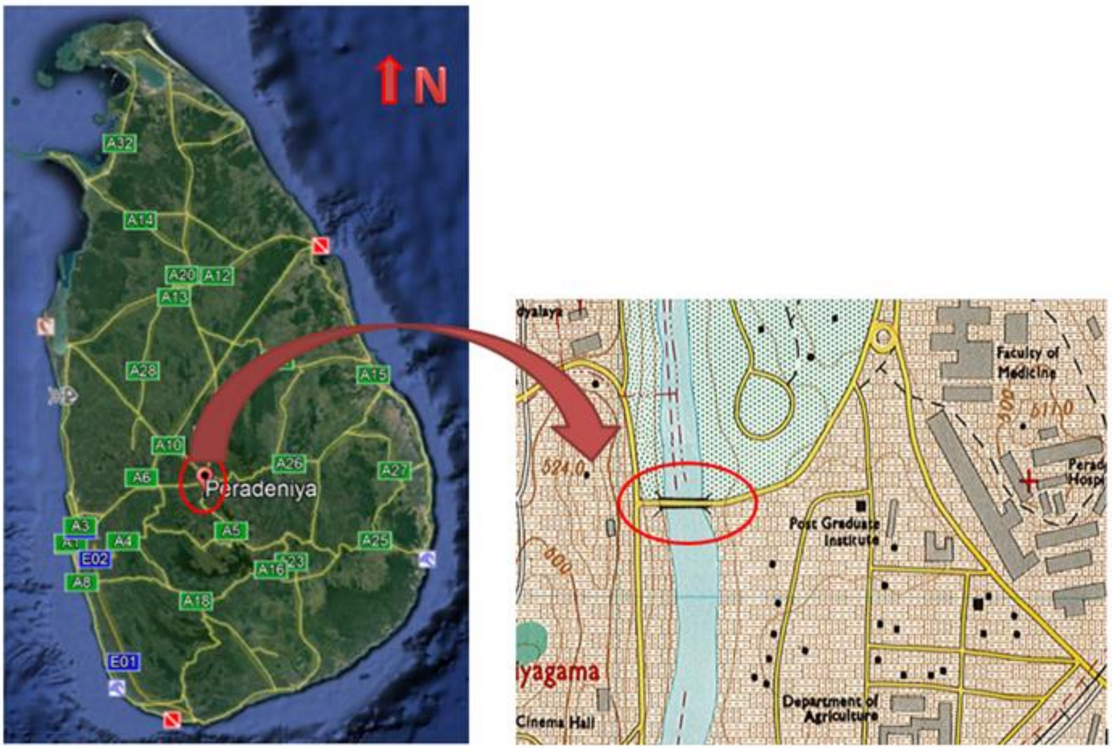

Figure 1.1 Location of the Peradeniya Bridge

The Peradeniya Bridge was initially constructed over the Mahaweli River in 1833, using timber and not a single nail has been used for assembling. The arch consisted of a 62 m span and a total width of 6.7 m. Therefore, this design and construction became unique among other bridges in Sri Lanka. Figure 1.2 shows an image of the old Peradeniya Bridge (Henry, W., Jackson (1843-1942) Satinwood Bridge. Available at: http://lankapura.com/2008/10/satinwood-bridge-mahaweli-ganga-river-near-peradeniya/ Accessed: 30 May 2020).

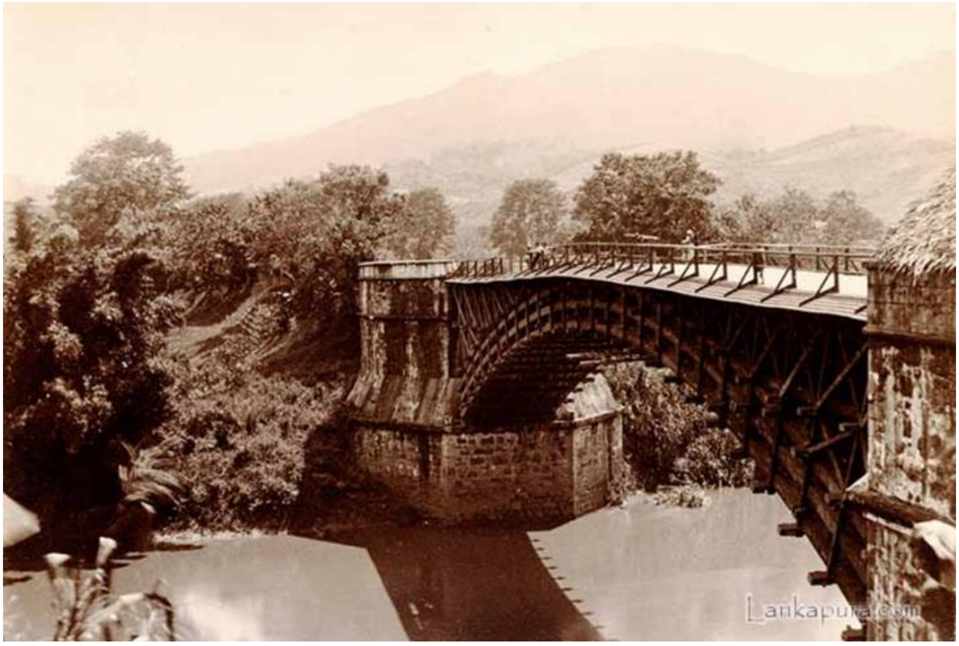

Figure 1.2 Old Peradeniya bridge over the Mahaweli River in 1894

The main part of the structure had been prefabricated in Colombo and transported to the site. The Construction of the bridge had taken more than 6 years. The old bridge remained for 72 years and then replaced by the current steel arch bridge in 1905. The bridge consists of three 3 bridge spans with 22 m in length. (Takaura, H. (2017), 'Ancient bridges – evidence of a proud history ', Daily news E-Paper. 29 December). Figure 1.3 shows an image of the existing Peradeniya Bridge.

Figure 1.3 The existing Peradeniya Bridge

Peradeniya junction has becomes severely congested due to vehicular traffic during morning, afternoon and evening rush hours (Figure 1.4). Rapid increase of traffic over the years, sudden contraction of the A1 road towards the bridge and not having a proper traffic control system are the main reasons for this heavy traffic congestion. Traffic congestion can cause several problems such as economic costs resulting from delayed travel times, increase fuel consumption, air pollution and accidents. Therefore, efforts were made to quantify this congestion and formulate appropriate measures to mitigate it.

Figure 1.4 Traffic congestion in Peradeniya Junction

Considering the above facts, a basic design study has been conducted in order to ease the traffic movements over the Peradeniya Bridge, by designing and constructing a new bridge parallel to the existing bridge, addressing the major issues arising with the new project. Initially, a preliminary survey was conducted in the Peradeniya bridge location to investigate the current traffic conditions and geological conditions in the area. With the implementation of the new bridge, the existing A1 and A5 roads have to be widened to four lane roads. Due to the widening, the slope of the left bank has to be protected with adequate measures. On the other hand, the existing bridge is a historical valuable structure; hence the new bridge has to be constructed at least 20 m away from the existing bridge to minimize the structural disturbance while construction of the new bridge. The following objectives were formulated after considering all the above aspects.

1.2 OBJECTIVES

- To resolve the traffic congestion in Peradeniya Junction
- To improve mobility on the national A1 and A5 road network
- To facilitate the regional development and enhance the stability of people's livelihood

1.3 SCOPE

The project area is centered over Peradeniya town, namely from Gannoruwa junction to Penideniya junction and Galaha junction. The project is mainly based on improving the transportation facilities to cater to the needs of passengers and pedestrians. Possible solutions that ease the traffic congestion in Peradeniya junction are discussed. On the other hand, improving the stability of existing slopes that are susceptible to erosion and making Peradeniya town an aesthetically appealing area is also considered in the scope of the project.

1.4 OUTLINE OF THE PROJECT

The New Peradeniya Bridge has been planned to be constructed on the southern side of the existing bridge where the environmental impact from the project and the damage to the existing structures are minimum. The followings are the sections covered in this design project.

- Construction of a new 75 m long, 9 m width bridge, with Y6 Prestressed concrete Girder with 3 spans, 2 carriageways, and foot walks on one side.
- Construction of a new approaching road from Galaha Junction towards new Peradeniya Bridge with appropriate filling design.
- Widening of existing A1 and A5 road stretch from Gannoruwa junction on A1 road to Penideniya junction on A5 road.
- Improvement of side slope on the left river bank which is subjected to additional load due to road widening.
- Improvement of traffic congestion by implementing a traffic color light system.
- Construction of an underpass for safe pedestrian crossing.
- Construction of four new bus halts along the new widened road.
- Proposing a new shopping complex to relocate the shops along project boundary.

Figure 1.5 shows the proposed solution on a map of Peradeniya.

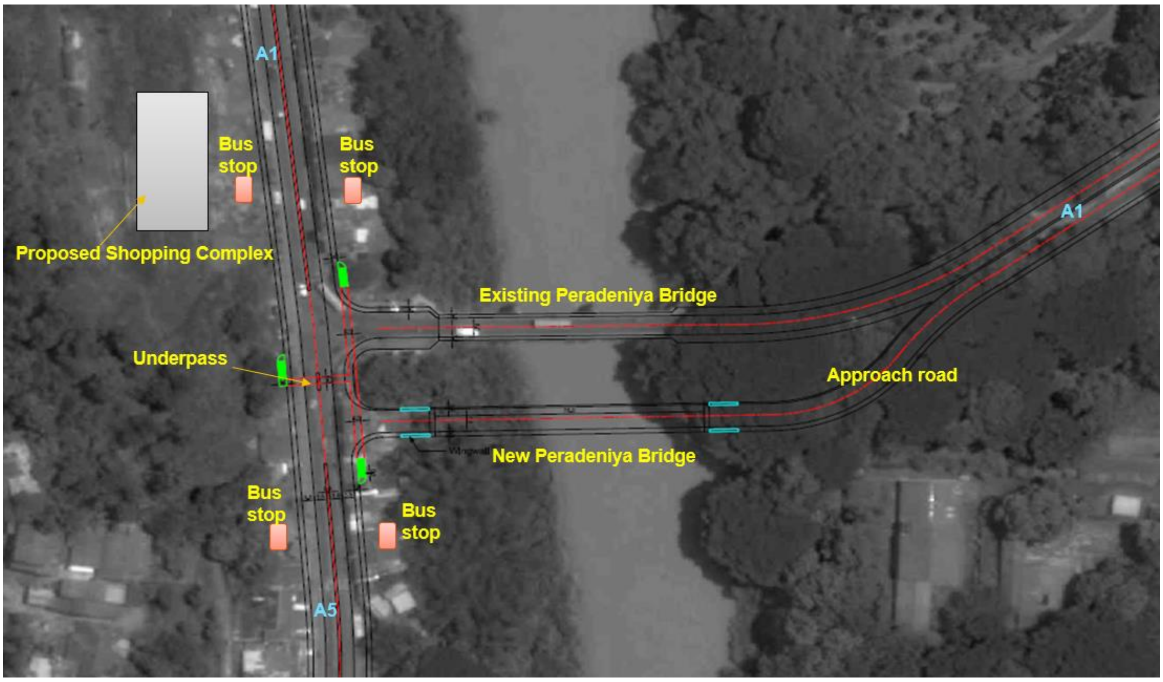

Figure 1.5 Proposed solution

CHAPTER 2

REVIEW OF THE PROJECT BACKGROUND

2.1 INTRODUCTION

Existing condition of Peradeniya Junction was investigated and evaluated in order to verify whether Peradeniya junction needed improvement. Initially, a site survey was carried out to acquire the dimensions of the existing bridge. Then, a traffic survey was carried out in Peradeniya junction considering all six directional traffic movement. In order to find a solution to this traffic congestion, the capacities of the roads were evaluated by compiling data related to prevailing vehicular flows and collecting other supportive information. From the capacity calculations made from field data it is evident that the bridge would fail to cater to the traffic demands.

On the other hand, the safety and ease of the pedestrians that travel through Peradeniya were investigated during a walkthrough survey. The major findings from this survey were,

- Only one Pedestrian crossing is available from the Gannoruwa junction to Penideniya junction resulting in a lot of pedestrians crossing the road in undesignated crossing places (figure 2.1).
- Walking space on both sides of the road was not adequate for the safe and comfortable movement of pedestrians as figure 2.2.

Thus, the existing condition of the safety and ease of commuting for pedestrians is not adequate and has to be much improved.

Figure 2.1 Unsafe crossing of the road

Figure 2.2 Inadequate walking space

Furthermore, due to the large number of buses that go through Peradeniya, traffic congestion is inevitably increased by the constant stop-start action of buses. This is further heightened by the lack of properly designed bus bays in the existing Peradeniya town. It was noticed during the walkthrough survey that most of the buses stopped in the middle of the road for the collection of passengers. (Figure 2.3)

Figure 2.3 Improper design of bus-bays

In addition to that, two places of religious significance were identified in Peradeniya. Those are Buddhist shrine and Bodhi tree (Figure 2.4) and Jumma Muslim mosque (Figure 2.5). It was noted that the relocation of these two places might be problematic due to the cultural importance of the two religious places. This issue was discussed with the design engineer in RDA and he stated that permission for relocation can be obtained by discussing with the relevant parties.

Figure 2.4 Buddhist shrine and Bodhi tree

Figure 2.5 Peradeniya Jumma mosque

By evaluating the aforementioned details and observations it was understood that the prevailing conditions at Peradeniya junction are need to be improved and the aesthetic appearance has to be enhanced.

2.2 DATA COLLECTION

The existing bridge is a steel-concrete composite arc bridge and the dimensions of the bridge are shown in the figure 2.6.the pier height data was given by RDA and Pier is 21 m in height from the bedrock. From the data available in the geotechnical laboratory, it was found that the bedrock is at a depth of 5m below the river bed, The left river bank soil data are given in section 2.2.1.

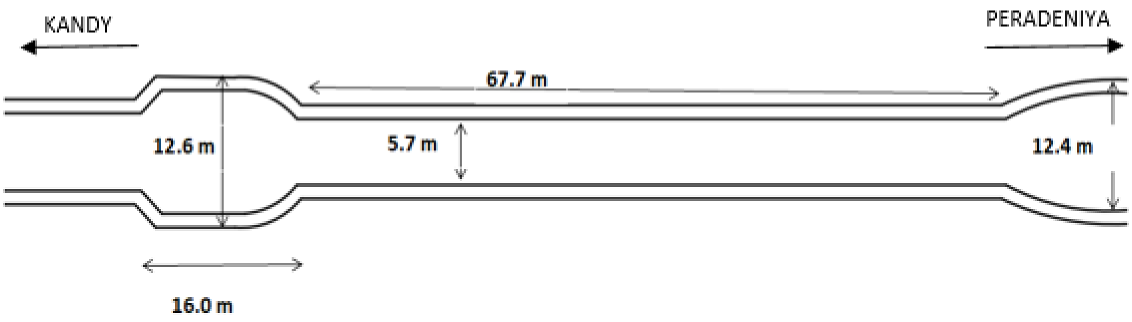

Figure 2.6 Dimensions of the existing bridge

2.2.1 GEOLOGICAL DATA

The left bank of the Mahaweli River is proposed to stabilize. Therefore, the required data for the left river bank was obtained. There are two types of soil layers in the left bank of the river. Figure 2.7 shows the location where soil data was collected and figure 2.8 shows the cross-section of that slope in X-X. Table 2.1 shows the soil parameters.

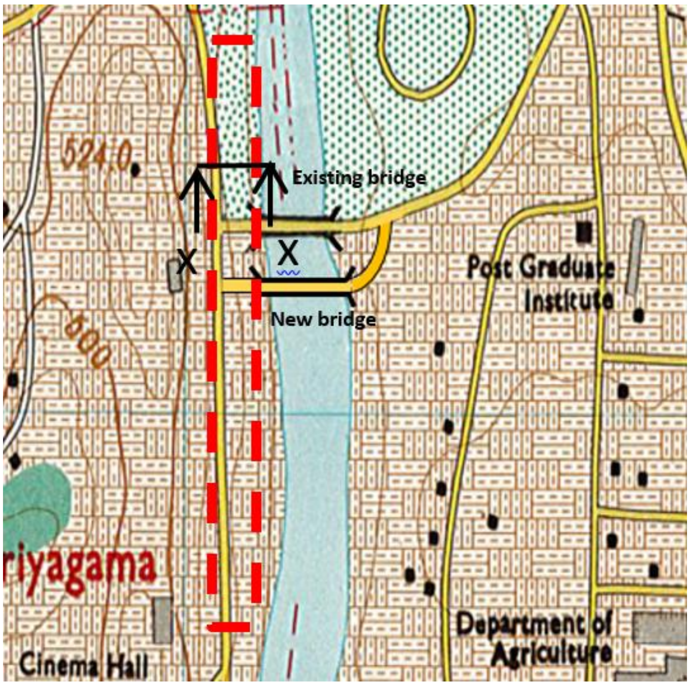

Figure 2.7 Location of the following soil data.

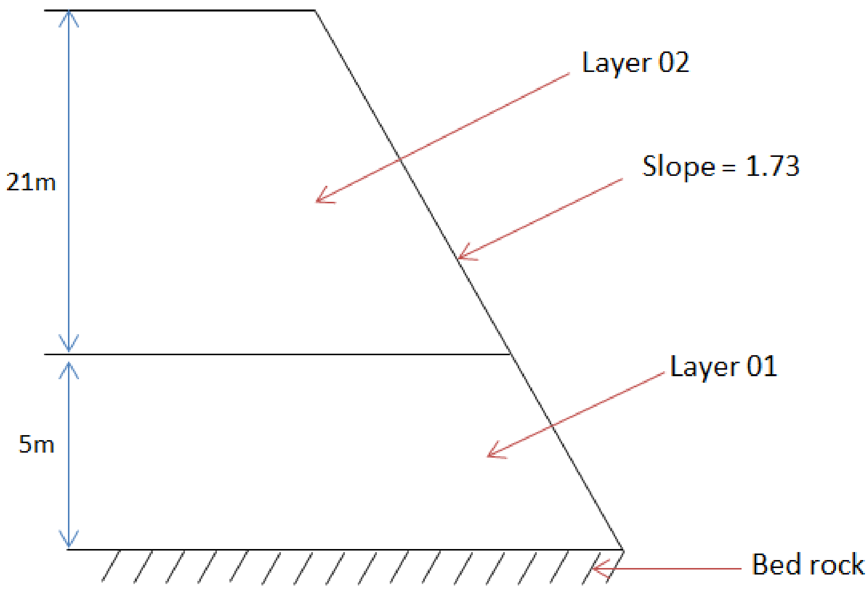

Figure 2.8 Section properties of X-X section of the riverbank

Table 2.1 Soil parameters of the left riverbank of Mahaweli River at Peradeniya

Soil parameters	**Layer 01**	**Layer 02**
Cohesion (c) / kPa	0	20
Friction angle (Φ)	44°	28°
Liquid limit (LL%)	56	80
Plastic limit (PL%)	31	46
Clay %	9	30.4
Silt %	20.8	56.7
Sand %	53.4	9.8
Gravel %	16.8	3.1

2.2.2 HYDROLOGICAL DATA

The daily average discharge values of Mahaweli River at Peradeniya junction were obtained from the Department of Irrigation. Considering peak daily average discharge from 69 years of data (1943-2012),

$$Q_{average} = 818.97\ m^3/s$$

$$\approx \underline{820\ m^3/s}$$

$$Q_{max} = 1660\ m^3/s$$

2.2.3 TRAFFIC DATA

The traffic flow along a road does not remain constant throughout a day or week but varies with both space and time. The peak hour represents the most critical period for operations and has the highest capacity requirements for a given location. A traffic survey was carried out at the Peradeniya junction on 26th February 2020 from 6.30 a.m. to 8.30 a.m. Traffic volumes were taken in fifteen minute intervals. The traffic survey results are given in Appendix H.

On the other hand, three crossing conflict points were observed at the Peradeniya junction which causes huge traffic congestion in peak hours (Figure 2.3).

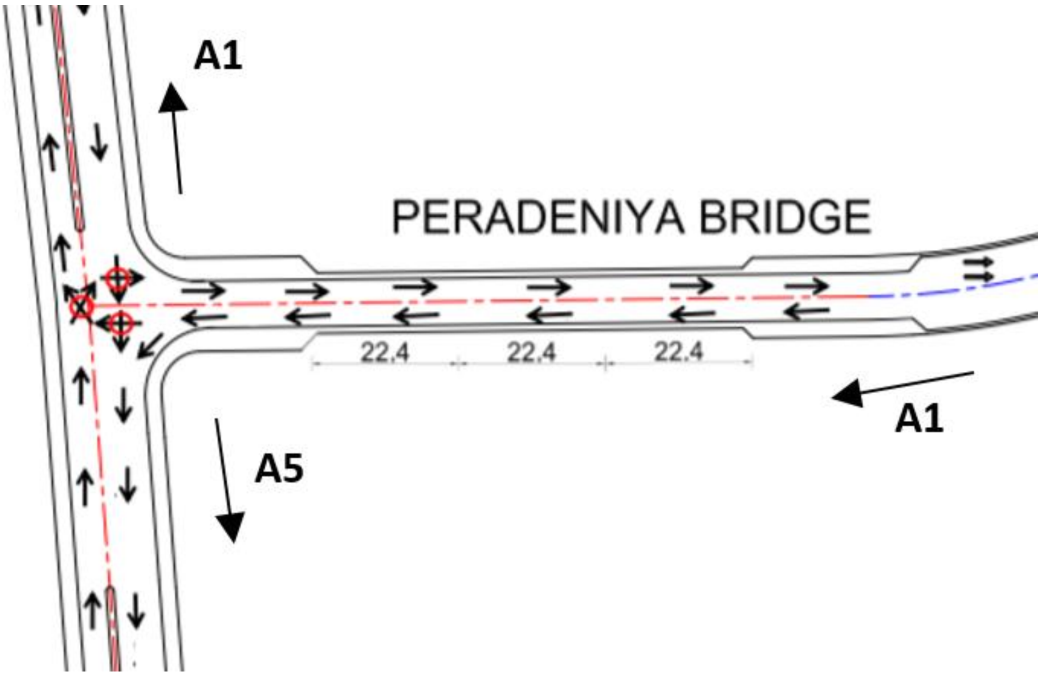

Figure 2.9 Conflict points in Peradeniya junction

2.3 TRAFFIC DATA ANALYSIS

The aim of this study is to observe whether the existing bridge capacity is adequate to cater to the current traffic volumes.

- **Passenger Car Unit (PCU)Values**

A Passenger Car Unit is a measure used primarily to assess highway capacity, for modelling purposes. Different vehicles are assigned different values, according to the space they take up. A car has a value of 1; smaller vehicles will have lower values, and larger vehicles will have higher values. To ascertain estimates of roadway capacity, Passenger Car Equivalent (PCE) factors are vital as they provide mechanism through which vehicles are converted into reference vehicle (i.e. Car)

The total number of vehicles travel from Kandy to Peradeniya and Peradeniya to Kandy through Peradeniya Bridge is given in table2.2 and table 2.3.

MC – Motorcycles

3W – three-wheelers

C/V/J – Car /Van /Jeep

Table 2.2 Traffic from Kandy to Peradeniya Junction direction

TIME (a.m.)	MC	3W	C/V/J	BUS	LORRY	TOTAL
6.30-6.45	66	45	60	15	11	197
6.45-7.00	88	97	55	33	9	282
7.00-7.15	93	102	90	30	7	322
7.15-7.30	94	115	87	27	8	331
7.30-7.45	110	121	71	26	10	338
7.45-8.00	96	118	68	29	10	321
8.00-8.15	81	99	61	31	10	282
8.15-8.30	81	78	68	30	7	264

Table 2.3 Traffic from Peradeniya Junction to Kandy direction

TIME (a.m.)	MC	3W	C/V/J	BUS	LORRY	TOTAL
6.30-6.45	142	120	125	69	13	469
6.45-7.00	172	139	82	47	7	447
7.00-7.15	121	97	66	22	7	313
7.15-7.30	171	114	99	28	4	416
7.30-7.45	214	121	102	31	7	475
7.45-8.00	202	134	108	31	6	481
8.00-8.15	223	143	114	38	7	525
8.15-8.30	208	134	121	44	9	516

Peak hour volume for the traffic

After that, the passenger car units (PCU) were found for each 15 min period using passenger car equivalent factors (PCEFs) in Table 2.4. Since the bridge is two-lane PCEFs as follows (Transport Research Board, 1984, "Highway CapacityManual").

Table 2.4 Passenger car equivalent factors (PCEFs)

	MC	3W	C/V/J	BUS	LORRY
PCEFs	0.40	0.80	1.00	1.80	3.00

Using the above factors PCU was calculated for both directions. Then using that values total passenger equivalent units were calculated as in Table 2.5.

Consider 6.30 a.m. to 6.45 a.m. in Kandy to Peradeniya junction direction,

PCU = 0.4x66 + 0.8x45 + 1x60 + 1.8 x 15 + 3x11

= 182.4

Table 2.5 Passenger Car Unit values for the bridge

TIME (a.m.)	PCU for Peradeniya junction direction	PCU for Kandy direction	Total PCU
6.30-6.45	182.4	441.0	623.4
6.45-7.00	254.2	367.6	621.8
7.00-7.15	283.8	252.6	536.4
7.15-7.30	289.2	321.0	610.2
7.30-7.45	288.6	361.2	649.8
7.45-8.00	283.0	369.8	652.8
8.00-8.15	258.4	407.0	665.4
8.15-8.30	237.8	417.6	655.4

Then using the above data passenger car units per hour (PCU/h) were calculated as in table 2.6. From the table 2.6 peak hour was found as 7.30 – 8.30 a.m.

Consider 6.30 a.m. to 7.30 a.m. traffic volumes,

Total vehicles = 197 + 282 + 322 + 331+ 469 +447 + 313 + 416

= 2777 veh/h

PCU s = 623.4 + 621.8 + 536.4 + 610.2

= 2391.8 PCU/h

Table 2.6 Passenger car units per hour and vehicle per hour

TIME (a.m.)	Vehicles per hour	PCU per hour
6.30 - 7.30	2777	2391.8
6.45 - 7.45	2924	2418.2
7.00 - 8.00	2997	2449.2
7.15 - 8.15	3169	2578.2
7.30 - 8.30	3202	2623.4
7.45 - 8.45	2389	1973.6

Peak hour = 7.30 – 8.30 a. m.

Peak hour volume on both directions = 2623.4 PCU/h

Total vehicles to both directions in the peak hour = 3202 veh/h

Two way Hourly volume is higher than 1900

So, peak hour factor (PHF) = 0.96 (Transport Research Board, 1984, "Highway Capacity Manual")

Adjustment factor of directional distribution for the bridge

Consider 6.30 – 6.45 a.m.,

Total vehicles from Kandy to Peradeniya junction direction = 197

Total vehicles from Peradeniya junction to Kandy direction = 469

Total vehicles for both directions = 666

Percentage of vehicles for Kandy to Peradeniya junction direction = (197x100)/660

= 30 %

Similarly, time slot total percentage vehicles were calculated and presented in the Table 2.7.

Table 2.7 Percentage of vehicles to Peradeniya junction direction

TIME (a.m.)	Total vehicles to Peradeniya junction	Total vehicles to Kandy direction	Total vehicles to both directions	Percentage of vehicles to Peradeniya junction direction (%)
6.30-6.45	197	469	666	30
6.45-7.00	282	447	729	39
7.00-7.15	322	313	635	51
7.15-7.30	331	416	747	44
7.30-7.45	338	475	813	42
7.45-8.00	321	481	802	40
8.00-8.15	282	525	807	35
8.15-8.30	264	516	780	34

Average percentage of vehicles for Peradeniya junction direction = 39.375 % = 40 %

So, the directional split of vehicles on the bridge = 40 /60.

Therefore, adjustment factor for directional distribution, f_d = 0.94

Level of Service (LOS) on the bridge

- **Level of Service Criteria (LOS)**

Level of service (LOS) is a mechanism used to determine how well a transportation facility is operating from a traveler's perspective. Typically, six levels of service are defined and each is assigned a letter designation from A to F, with LOS A representing the best operating conditions, and LOS F the worst. (Transport Research Board, 1984, "Highway Capacity Manual)

- **Service flow, (SF)**

The maximum volume that can be carried at any selected level of service is referred to as the service volume or the service flow rate for the level.

The bridge is located in level terrain and no passing zones are 100% due to a two way two lane bridge.

Lane width = 2.76 m

Shoulder width = 0 m

Therefore,

Adjustment factor for narrow lanes and restricted shoulders f_w

For LOS A – D = 0.49

For LOS E = 0.66

Directional split factor f_d = 0.94

Consider LOS A,

Volume / Capacity (V/C) ratio = 0.04

Service flow, SF = 2800 * f_w * f_d * v/c

= 2800 * 0.49 * 0.94 * 0.04

= 51.59 PCU/h

Likewise, for each LOS, SF was calculated for LOS **A** to LOS **E** as in table 2.8.

Table 2.8 Service flow rates for each LOS

LOS	f_w	f_d	V/C ratio	SF (PCU/h)
A	0.49	0.94	0.04	51.59
B	0.49	0.94	0.16	206.35
C	0.49	0.94	0.32	412.70
D	0.49	0.94	0.57	735.12
E	0.66	0.94	1.00	1737.12

Peak hour volume = 2623.4 PCU/h

Peak flow rate for the traffic = peak hour volume / PHF

= 2623.4 / 0.96

= 2732.71 PCU/h > 1737.12 PCU/h (SF for LOS **E** from table 2.8)

So, Bridge operates in LOS **E**.

The above analysis shows that the condition of the existing bridge is in Level of Service **E**. This indicates that the existing bridge cannot carry this vehicle capacity and a need for an additional method to divert the traffic entering the existing bridge. Thus, five alternative solutions were proposed to the current traffic issue at the Peradeniya junction.

CHAPTER 3

FORMULATION OF CONCEPTUAL DESIGN ALTERNATIVES

By considering all the aspects mentioned in the previous chapters five alternative solutions were formulated for the construction of the new bridge.

3.1 CONCEPTUAL DESIGN ALTERNATIVES

3.1.1 ALTERNATIVE 1

The first alternative is the construction of a new bridge parallel to the existing bridge and allow only Kandy-Colombo direction traffic movements in the existing bridge and the new bridge is used for Kandy- Gampola traffic movements as shown in figure 3.1. As the new bridge is parallel to the existing bridge, the length of the new bridge is around 75m.

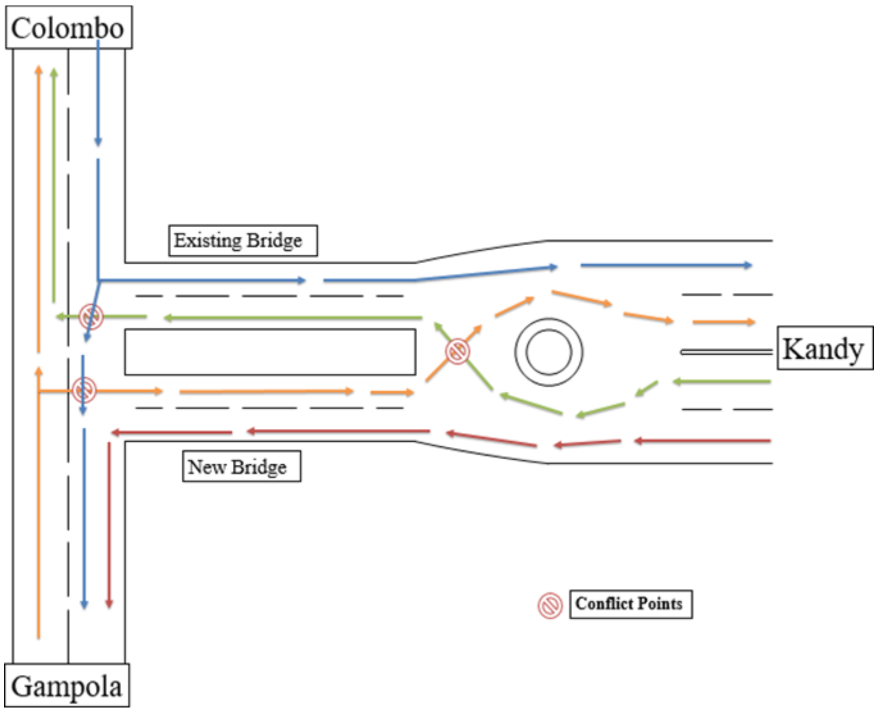

Figure 3.1 Sketch showing the new bridge, existing bridge and traffic directions in alternative 1

In this alternative, three major crossing conflict points were found. A roundabout is introduced to smooth the traffic flow into the bridge. This alternative solves the traffic congestion of the existing bridge and the congestion will be minimized at the junction and helps to effectively transport with reduced delays.

To implement this alternative the existing A1 and A5 roads need to be widened to four lane roads. Therefore, the existing shops in Peradeniya town from both sides of the Gannoruwa Junction to Penideniya junction have to be relocated. The level of Serviceability of the roads can be increased with this alternative. This design has less impact on the Peradeniya Botanical garden but has to acquire some land from the University of Peradeniya.

3.1.2 ALTERNATIVE 2

The second alternative is the construction of a new bridge parallel to the existing bridge and only allowing one-way traffic movements on each bridge as shown in figure 3.2. . As the new bridge is parallel to the existing bridge, the length of the new bridge is around 75m.

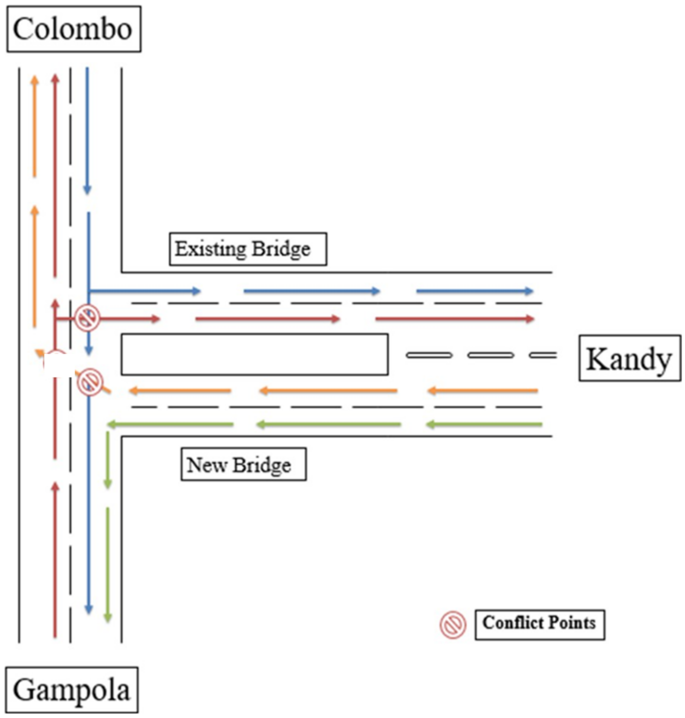

Figure 3.2 Sketch showing the new bridge, existing bridge and traffic directions in alternative 2

In this alternative, two major crossing conflict points were found. From this, the traffic congestion on the existing bridge and at the junction will be minimized because of fewer conflict points and this will help for smooth traffic movements with reduced delays.

To implement this alternative the existing A1 and A5 roads need to be widened to four-lane roads. Therefore, the existing shops in Peradeniya town in Peradeniya town from both sides of the Gannoruwa Junction to Penideniya junction have to be relocated. The level of Serviceability of the roads can be increased with this alternative. This design has less impact on the Peradeniya Botanical garden but has to acquire some land from the University of Peradeniya.

3.1.3 ALTERNATIVE 3

The third alternative is the construction of a new flyover connecting Galaha junction and Penideniya junction as shown in figure 3.3. The new flyover length is around 700m.

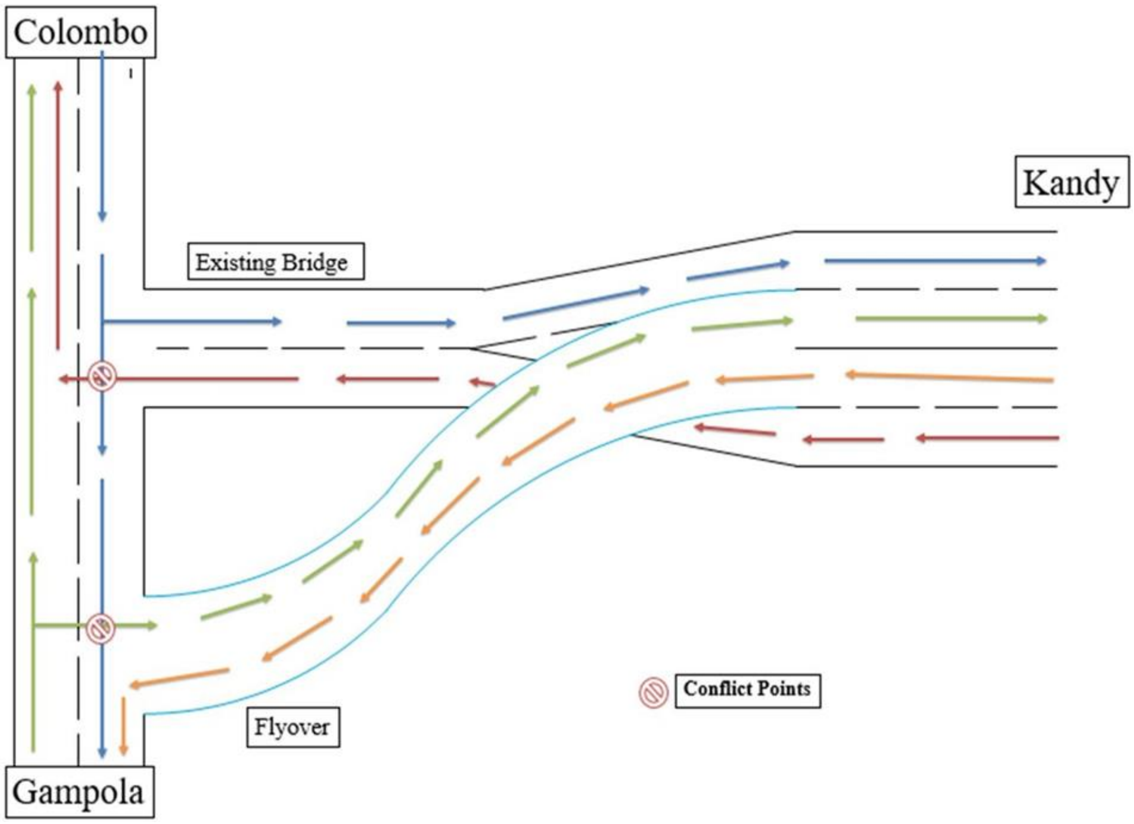

Figure 3.3 Sketch showing the new bridge, existing bridge and traffic directions in alternative 3

In this flyover design, there are only two conflict points and they are not occurring at the same place. Therefore, there is a very low chance of traffic congestion in this alternative. This design is more durable and suitable for growing traffic demand. Due to the land availability in Galaha junction and

Penideniya junction as shown in figure 3.4 and 3.5, this design can be implemented without obtaining Botanical garden land but the lands of the university, Mahaweli Authority, and RDA will be required. The Flyover starting and ending points are shown on the survey map in figure 3.6. In this alternative, the existing A1 and A5 roads do not need any widening, and demolition of the shops also not needed.

Figure 3.4 Flyover starting point (Galaha junction)

Figure 3.5 Flyover ending point at Peradeniya city in A5 road (near the Cinema hall)

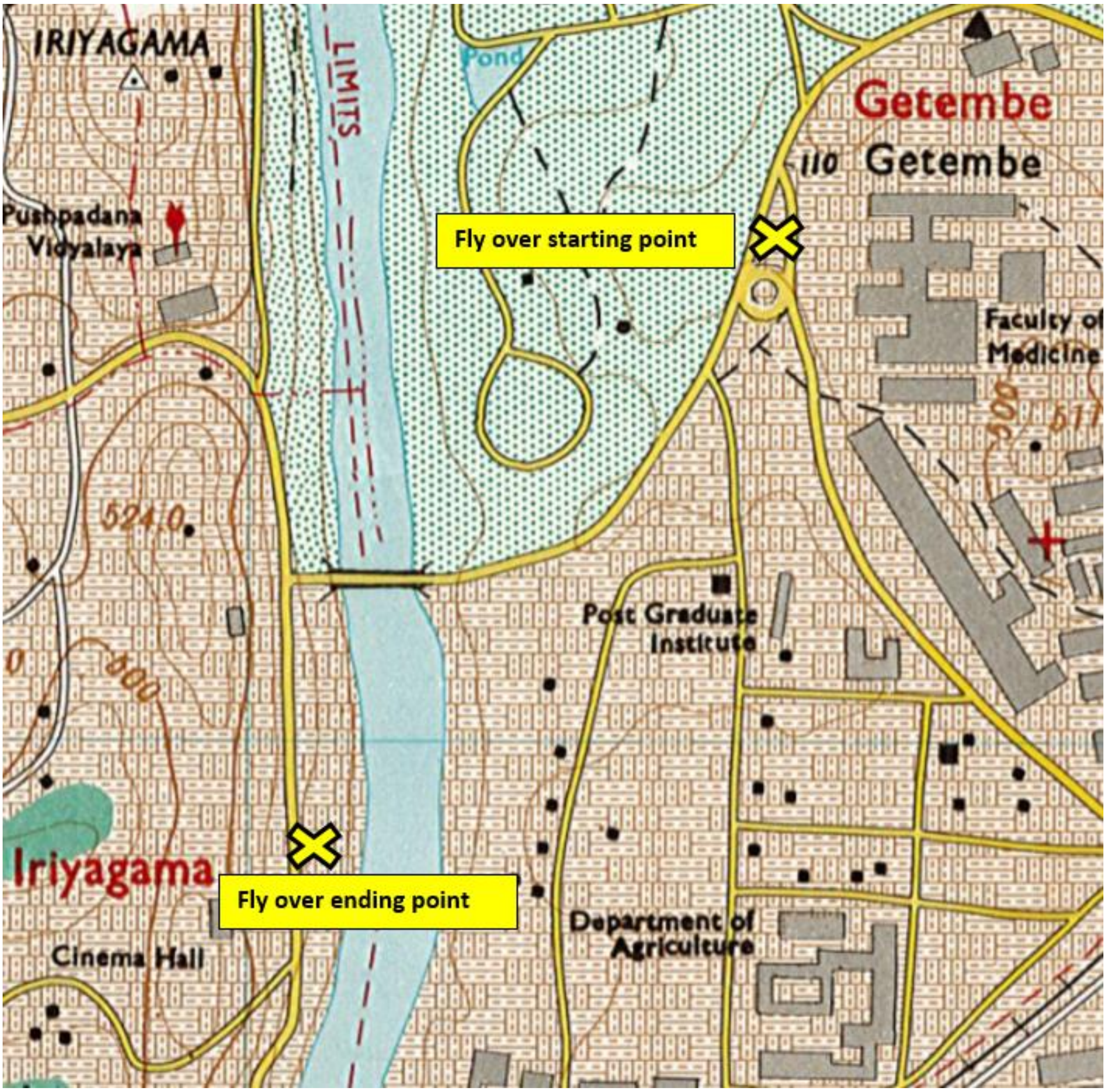

Figure 3.6 Flyover starting and ending points on the survey map

‘

3.1.4 ALTERNATIVE 4

The fourth alternative is the construction of a new 4 lane bridge parallel to the existing bridge as shown in figure 3.6. The new bridge length is around 700m. In this design alternative, the existing bridge will only be used for pedestrian passing. The new bridge length is around 75 m.

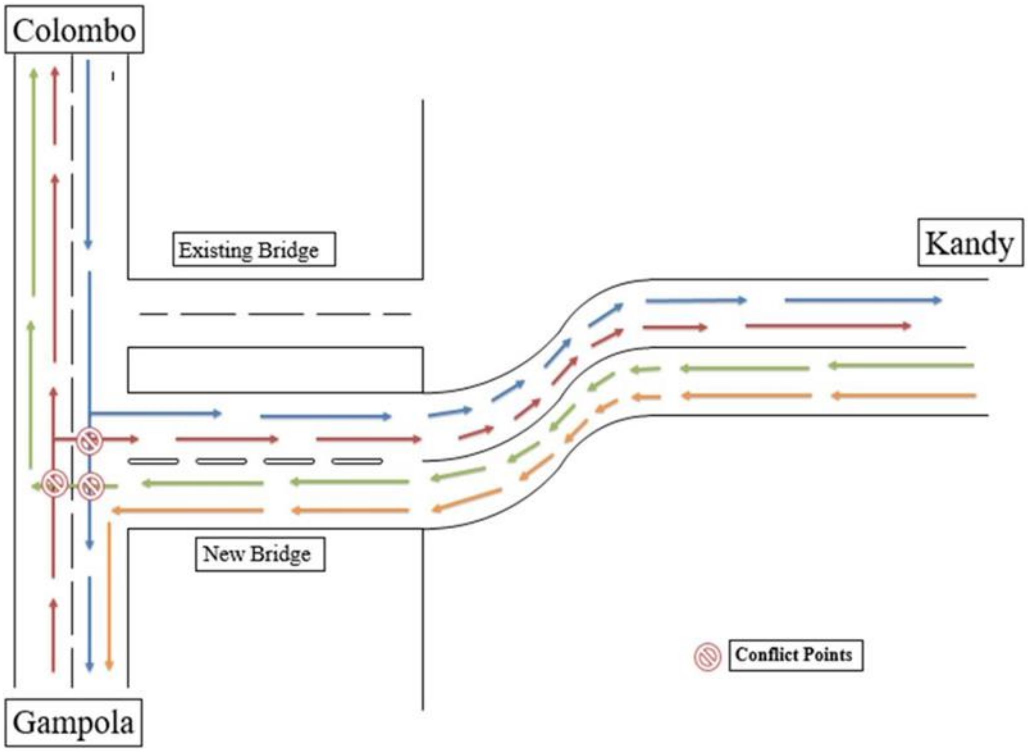

Figure 3.7 Sketch showing the new bridge, existing bridge and traffic directions in alternative 4

In this alternative, three major crossing conflict points were found. From this alternative, the traffic congestion of the existing bridge is no longer occurring and the congestion is minimizing at the junction and that helps in effective transport with reduced delays.

To implement this alternative the existing A1 and A5 roads need to be widened to four lane roads. Therefore, the existing shops in Peradeniya town have to be relocated. The level of Serviceability of the roads can be increased with this alternative. This design has less impact on the Peradeniya Botanical garden but has to acquire some land from the University of Peradeniya.

3.1.5 ALTERNATIVE 5

The fifth alternative is the construction of a new flyover connecting Gannoruwa junction and Penideniya junction crossing the existing bridge perpendicularly as shown in figure 3.7. The new flyover length is around 600m.

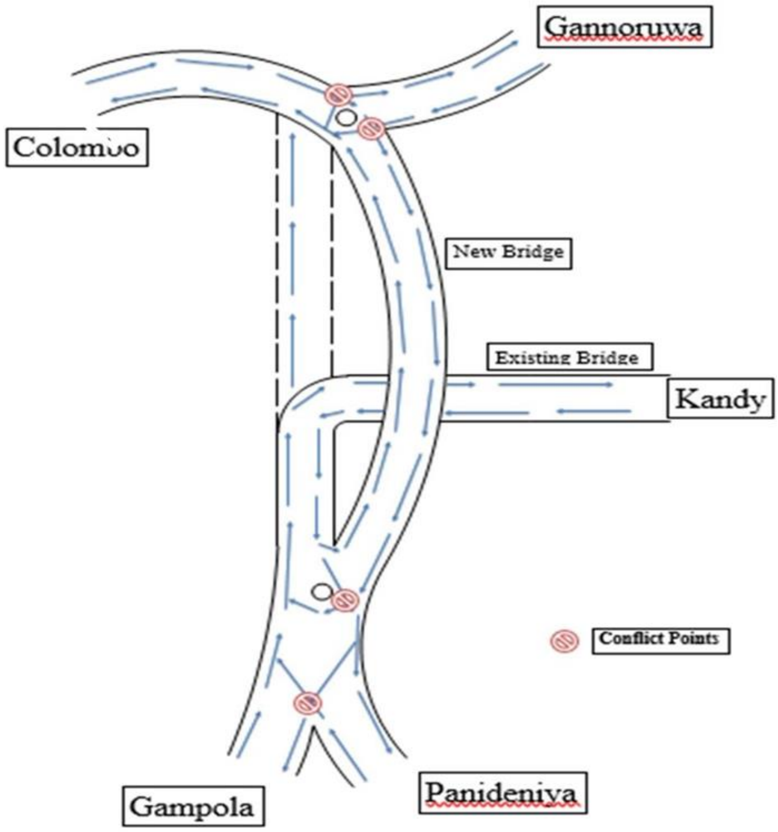

Figure 3.8 Sketch showing the new bridge, existing bridge and traffic directions in alternative 5

In this alternative, there are zero conflicts along the existing Peradeniya Bridge. Other four conflicts act in much greater length, therefore less impact on the traffic flow. Due to the smooth traffic flow in the existing bridge, traffic congestion will be reduced. This design is more durable and suitable for growing traffic demand. This design can be done without obtaining university land, and Botanical garden land. In this design alternative, the existing A1 and A5 roads do not need any widening and demolition of the shops also not required.

3.2 COMPARISON OF ALTERNATIVES

Table 3.1 Comparative study of Alternative solutions

	DESIGN LENGTH	WIDTH	CONFLICT POINTS	ADVANTAGES	DISADVANTAGES
ALTERNATIVE 1	75m	9m	3	**Only the Colombo to Gampola traffic movement will be interrupted. The cost of this bridge is low compared to alternatives 3, 4, and 5. Less Environmental Impacts on Botanical garden**	**Need to obtain land from Peradeniya University to widen the existing road. Need to change the existing A1 and A5 road to 4 lane roads. Less Driver Comfort**
ALTERNATIVE 2	75m	9m	2	**The cost of this bridge is low compared to alternatives 3, 4, and 5. Fewer conflict points. Less Environmental Impacts on Botanical garden**	**Need to obtain land from Peradeniya University to widen the existing road. Need to change the existing A1 and A5 road to 4 lane roads.**
ALTERNATIVE 3	700m	9m	2	**Fewer conflict points A durable design for the Rapid increase of traffic over the years. Less traffic congestion Smooth traffic flow**	**The cost of the flyover will be higher than alternatives 1,2 and 4. Need to obtain land from the university of Peradeniya, Mahaweli authority and RDA. Pier heights will be very high. Considerable environmental impacts**
ALTERNATIVE 4	75m	18m	3	**Less environmental impacts compared to alternative 3 and 5. Durable Less traffic congestion Smooth traffic flow**	**The cost of this bridge will be higher than the alternatives 1 and 2. Existing A1 and A5 road has to be widened to 4 lanes. Considerable environmental impacts**
ALTERNATIVE 5	600m	9m	4	**Due to the smooth traffic flow in the existing bridge the traffic congestion will be reduced. Existing roads can be used as it is**	**Considerable environmental impacts during construction. High cost compared to other alternatives.**

When we consider the cost of construction, the duration of the construction, socioeconomic factors, reducing traffic congestion, aesthetic appearance, space for expansion, and effects for the environment, alternative no. 2 has the most advantages among other alternatives. Therefore, alternative no.2 was selected as the final design.

CHAPTER 4

PRELIMINARY DESIGN CONSIDERATIONS

4.1 PROPOSED BRIDGE

A new bridge is proposed to construct parallel to the existing bridge allowing Kandy- Gampola/ Kandy-Colombo traffic movements. Based on the requirements of RDA, a distance of at least 20 m has to be maintained between the two bridges. An approach road is needed to be designed to connect the new bridge with the existing road. Figure 4.1 shows the layout plan with the proposed bridge.

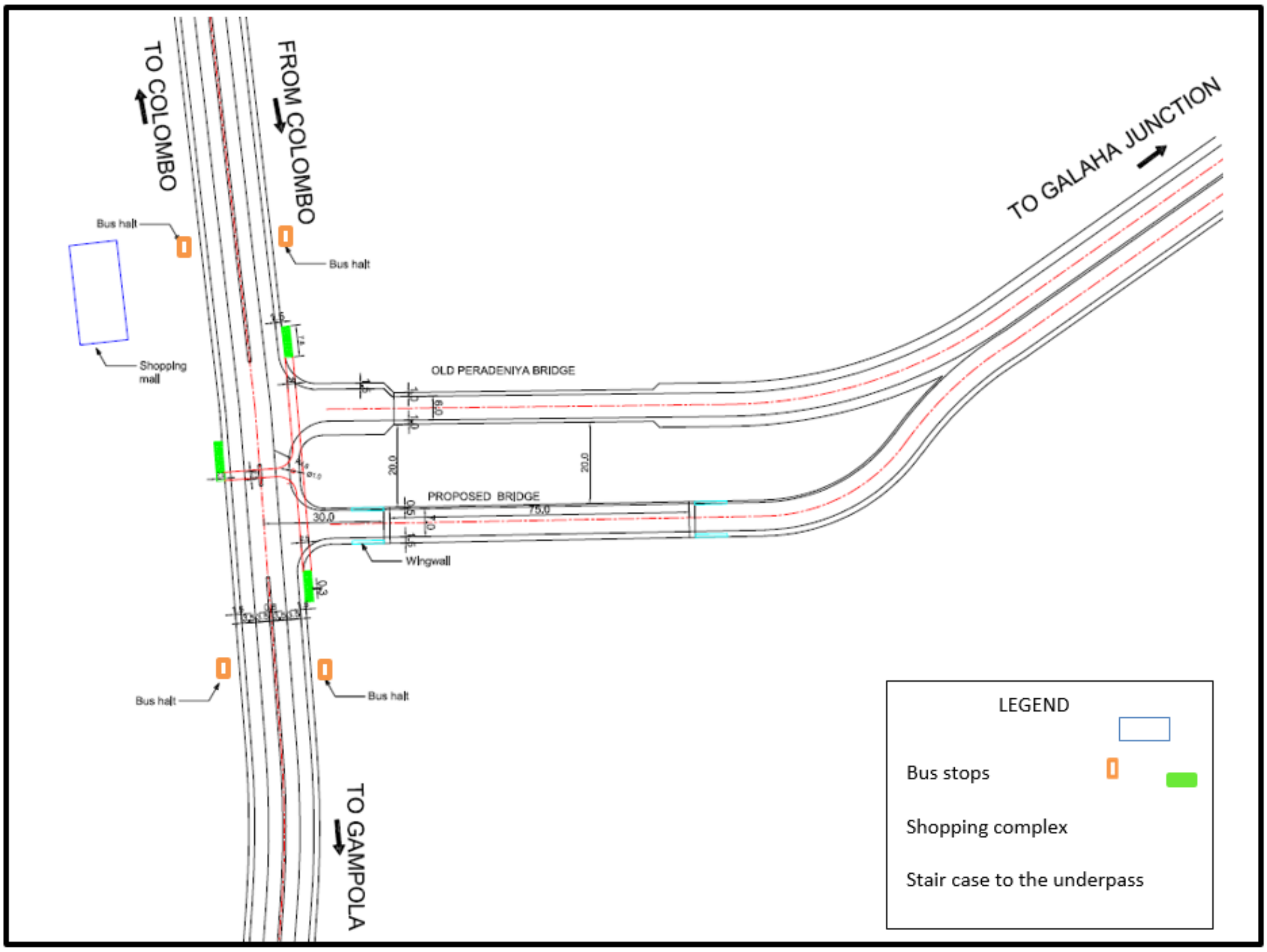

Figure 4.1 The layout plan with the proposed bridge

The existing bridge has three right angle crossing conflicts which led to huge traffic congestion in peak hours. In this new design crossing conflicts were reduced to two. Since the existing junction does not have any traffic management method, this design will be implemented a signalized traffic control system to control the traffic movements.

On the other hand, the existing A1 and A5 roads are to be widened to four lanes. For this reason shops from Gannoruwa junction to 50 m away from existing Peradeniya Bridge, along A1 and A5 roads, will be demolished. Therefore, a shopping complex is proposed on the opposite side to the existing Peradeniya Bridge to relocate the demolished shops.

For the problem of inadequate walking space, a walking path along the river bank is proposed which includes adequate space for jogging as well. To provide safe passenger crossing, an underpass implementation is proposed which will improve the appearance of the town.

When the roads are widened, the slope stability of the left riverbank of the Mahaweli River has to be implemented. To obtain the slopes of this area and contour data, a contour map of the area was developed.

4.2 DEVELOPING THE CONTOUR MAP AND DIGITAL ELEVATION MODEL

To represent elevation data on 2D maps, contour plots are used. A contour line joins all the points of equal height from the mean sea level. A Digital Elevation Model (DEM) is a specialized database that represents the relief of a surface between points of known elevation. By interpolating known elevation data from ground surveys a rectangular digital elevation model can be created. ArcGIS software was used to achieve the contour map in Peradeniya. Hence, to develop an accurate contour map, data indicated below were required.

- Road network data
- Stream network data
- Contour data

Contour data and stream network were obtained from the maps available in the survey laboratory of faculty of Engineering of university of Peradeniya. Then, the road network and then the map was developed according to the following steps.

1. Coordinates of the required points and the route in the location were obtained using GPS coordinate machine.

2. An excel file was created including all latitude and longitude coordinates.
3. The coordinates table was imported to ArcGIS
4. The contour map and the DEM were developed according to the Kandawala coordinates system.
5. After adding contour data, stream network and tracking points were inserted to ArcGIS as shown in figure 4.2

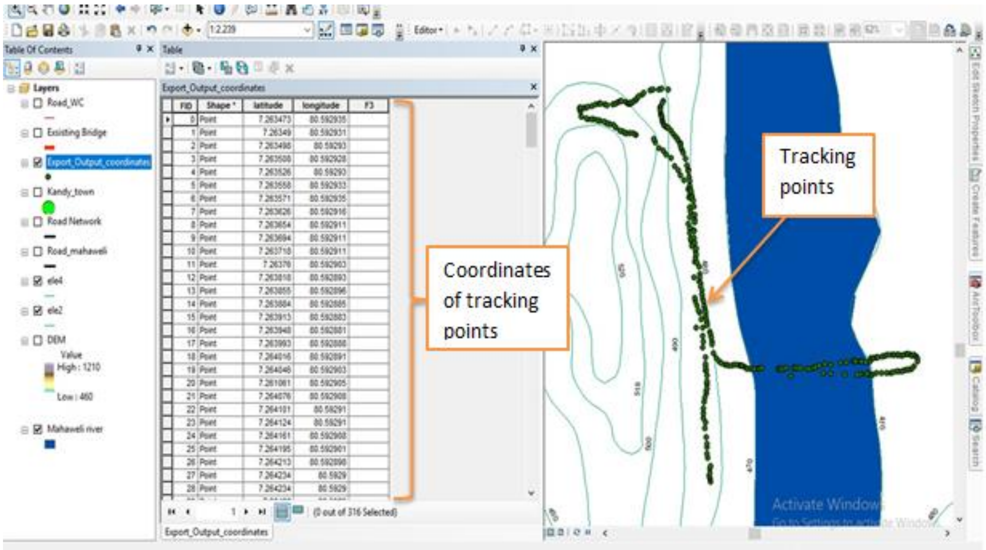

Figure 4.2 Adding contour data, stream network and tracking points to ArcGIS

6. Then, the road network was drawn along with the tracking points in figure 4.3.

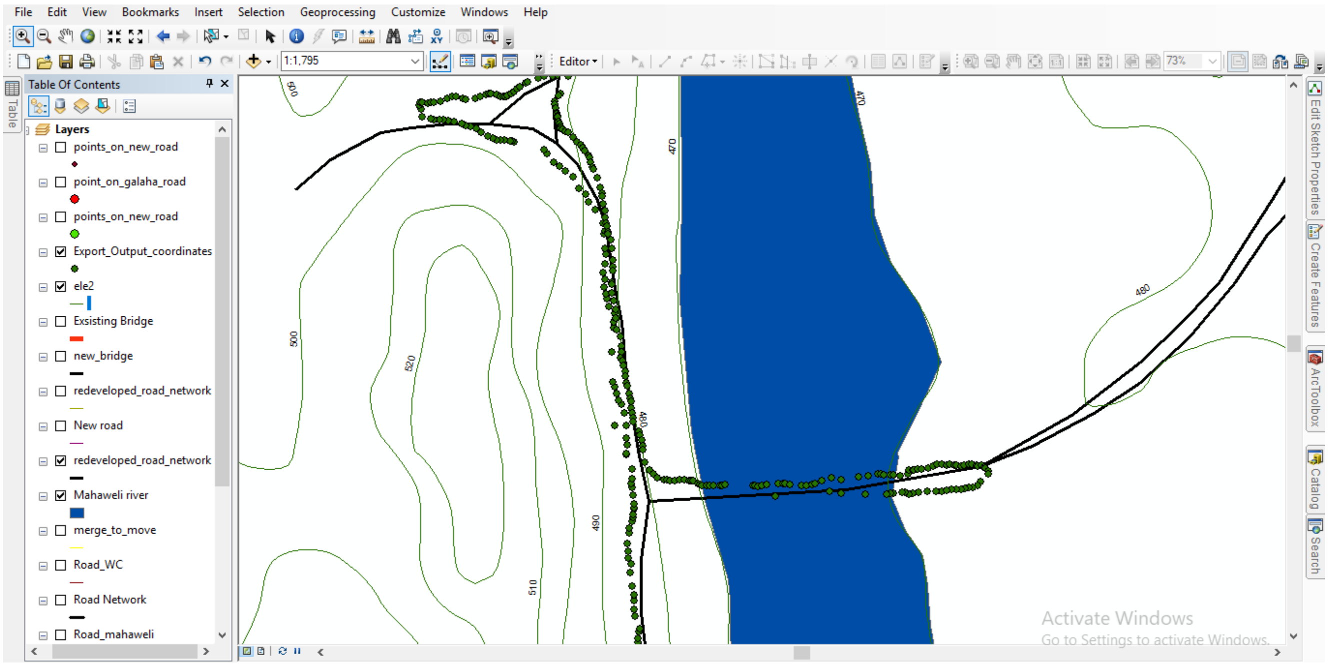

Figure 4.3 Developing the road network

Finally, an accurate road network, Mahaweli river, and contour profile were achieved for the same coordinates system. To develop a proper map, a base map was added to the worksheet in ArcGIS software for the same coordinates system. Then, that base map was matched to the developed features in ArcGIS. Then the proper contour map and the DEM for our site were created as shown in figure 4.4.

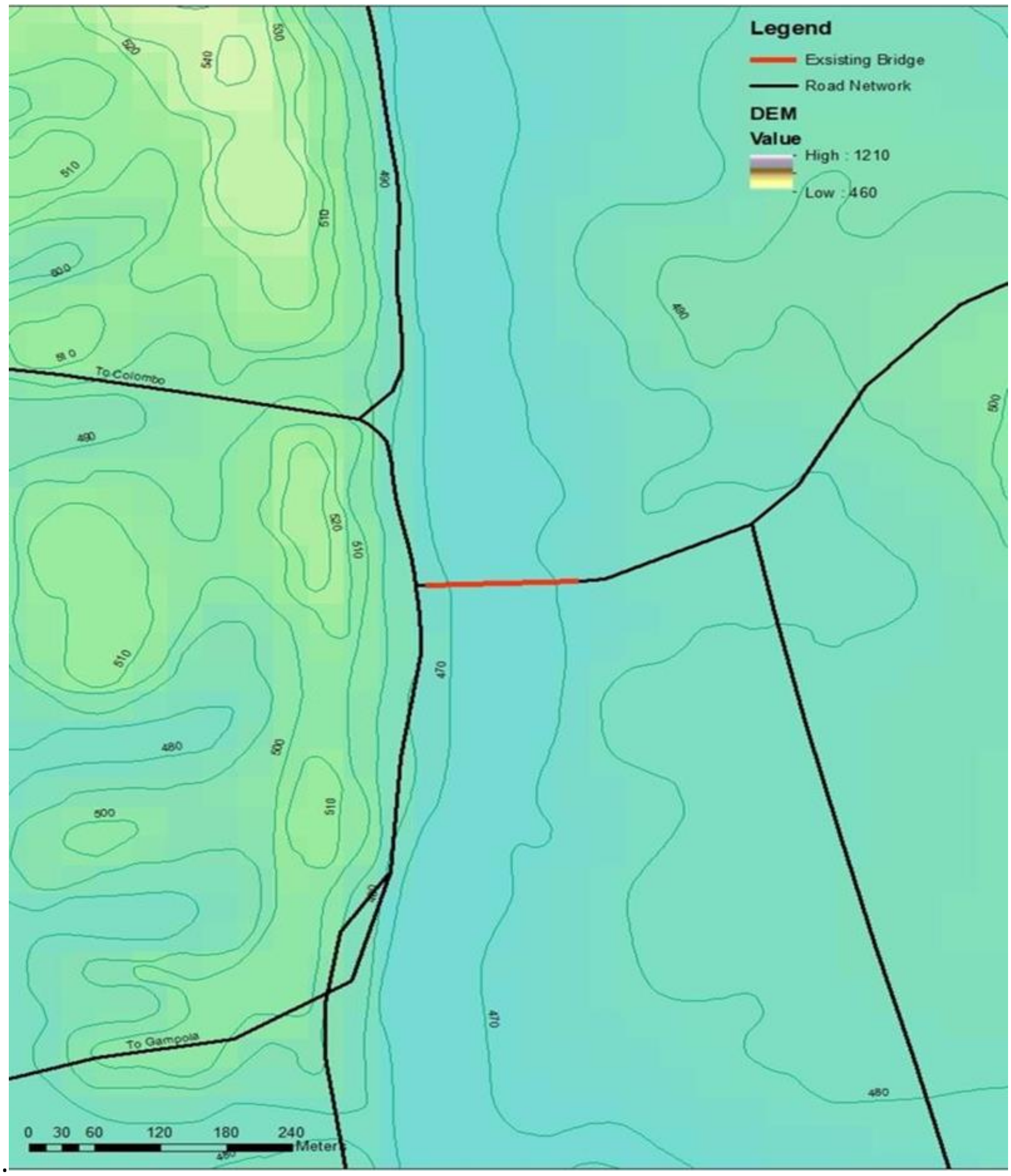

Figure 4.4 Digital elevation model (DEM) at the project area

4.3 DEVELOPING THE LONGITUDINAL PROFILE ACROSS THE BRIDGE

After the generation of the contour map for Peradeniya town, the longitudinal profile along the bridge was created. Longitudinal profile across the proposed bridge is very vital to identify the required heights of bridge components such as piers and abutments, etc. With the prevailing situation of COVID-19 in the country, it was difficult to do a site survey. Hence, data obtained from the RDA and the generated contour map was used to obtain the longitudinal profile along the new bridge.

Initially, the distance of two known locations along the bridge alignment was found using google earth software. Next, the spans of the existing bridge, pier heights, and elevation details in soil and bedrock layers were collected from the RDA and the Geotechnical laboratory of faculty of engineering, University of Peradeniya. The Proposed new bridge elevation, existing bridge elevation, and other required elevations were obtained from the developed contour map as explained in section 4.2. Then, an elevation profile of ground and bedrock was developed by interpolating the elevations using NUM XL interpolating software. Figure 4.6 shows the generated longitudinal profile of soil and bed rock.

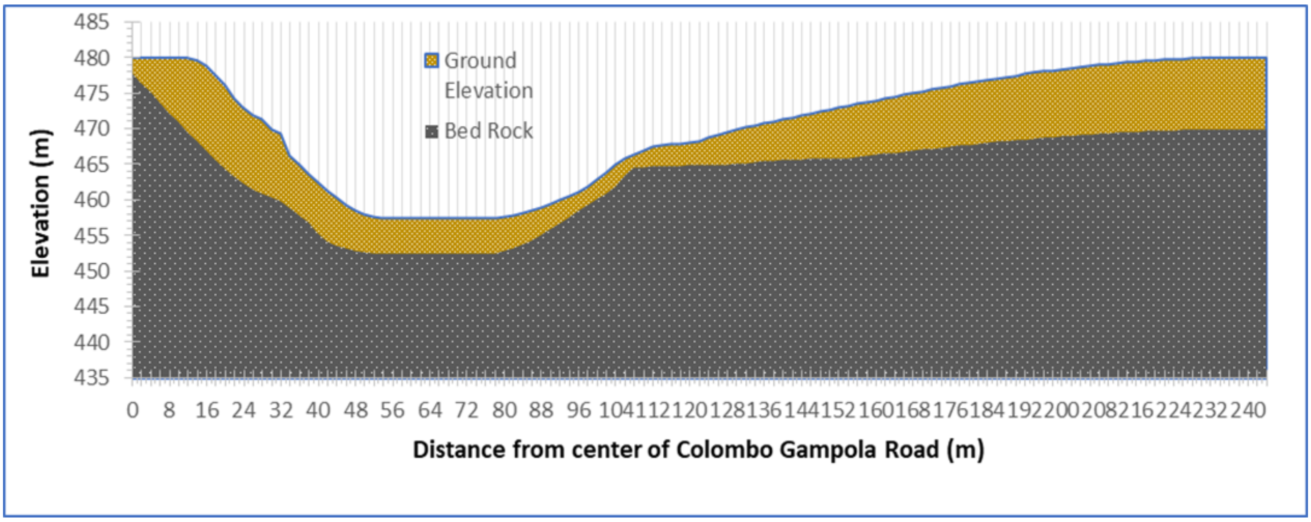

Figure 4.5 Longitudinal profile of the soil and bedrock

4.4 DESIGN CONSIDERATIONS

The basic purpose of a bridge is to carry traffic over an opening or discontinuity in the landscape. An opening can occur over a highway, a river, a valley, or any other type of physical obstacle. The need to carry traffic over such an opening defines the function of a bridge. The design process of a bridge can be divided into four basic stages: conceptual, preliminary, detailed, and construction design. The purpose of the conceptual design is to come up with various feasible bridge schemes and to decide on one or more final concepts for further consideration The purpose of the preliminary design is to select the best scheme from these proposed concepts and then to ascertain the feasibility of the selected concept. The purpose of the detailed design is to finalize all the details of the bridge structure so that the document is sufficient for tendering and construction. Finally, the purpose of the construction design is to provide step-by-step procedures for building the bridge.

In this project, five alternatives were considered as conceptual designs and alternative 2 was selected as the proposed solution. The details of the Proposed Bridge is given below.

Superstructure

- Material - Prestressed pre-tensioned concrete girders and in-situ concrete slab
- Fabrication - Precast Pre-tensioned Y6 girders , cast in-situ reinforced concrete slab
- Structural form- Simple slab and girder type
- Span - 3 span with each 25 m long.
- Length - 75 m
- Width - 9 m (Carriageway width of 7 m + 1.5 m sidewalk and 0.5 m on other side)
- Codes - Eurocode 2

Piers

- Material - Reinforced concrete
- Fabrication - Cast in-situ
- Type - Hammerhead pier type
- Height - 21 m
- Foundation - Pile foundation
- No. of piles - 9
- Codes - Eurocode 2 and Eurocode 7

Abutments

- Material - Reinforced concrete
- Fabrication - Cast in-situ
- Type - Closed ended seat type
- Height - Left 10.5m , Right 14 m
- Width - 10 m
- Foundation - Pile foundation
- No. of piles - 9
- Codes - Eurocode 2 and Eurocode 7

For the bridge, an asphaltic plug joints were designed as the expansion joint according to AASHTO, 2012. Then, bridge bearings and drains were design in accordance with AASHTO – LRFD specifications. Finally, asphalt pavement layer, lamp posts and hand rails were designed. The design life of bridge is 50 years.

Subsequently, an approach road was designed to divert the traffic to the new bridge from the A1 road. In this design pavement design and geometric design was done based on AASHTO highway design specification and Austroads, 2016. Moreover, a fill for the valley area between the approach road and existing roads was designed.

On the other hand, the A1 and A5 roads from Gannoruwa junction to Penideniya junction were proposed to widen to four lane roads in order to cater the additional traffic from the new bridge. Additional two lanes were designed from both sides of the existing road keeping the centre median of the road unaffected. Due to the additional load coming from widened road, a slope stability check was conducted for left river bank. Then, vegetated type slope stabilization was implemented for this design.

Furthermore, traffic signal light system was designed to control the traffic at the Peradeniya junction. The primary function of any traffic signal is to assign right-of-way to conflicting movements of traffic at an intersection. In this design 3 traffic signal lights are designed based on analysing the peak hour traffic volumes.

Moreover, an underpass system was designed as the pedestrian crossing for this design because it was observed that the Peradeniya junction does not have an adequate pedestrian crossings across the road. All the details of the design are given in chapter 5.

CHAPTER 5

DETAILED DESIGNS

A summary of all the designs of the project is given in this chapter

5.1 DESIGN OF BRIDGE LOADS

According to EN 1991, the following loads should be considered in the bridge design.

- Self-weight and imposed loads
- Wind
- Thermal actions
- Actions during execution
- Settlements
- Accidental actions (impact loads)
- Traffic loads

There are also other actions described in EN 1991, such as fire and snow loads, which are considered as irrelevant for this design. Additional actions are foreseen in other EN Eurocodes, namely:

- Concrete creep and shrinkage (EN 1992)
- Settlements and earth pressures (EN 1997)
- Seismic actions (EN 1998)

5.1.1 TRAFFIC LOADS

Traffic load models are used to calculate the traffic loads acting on bridges. Load modes are defined in BS EN 1991-2:2003, Section 4.

Traffic load models

- Vertical forces: LM1, LM2, LM3, LM4
- Horizontal forces: braking and acceleration, centrifugal, transverse centrifugal, transverse

Groups of loads

- gr1a, gr1b, gr2, gr3, gr4, gr5
- characteristic, frequent and quasi-permanent values

There are four Load models for limit state verifications except for fatigue limit states

- Load Model 1 - Concentrated and uniformly distributed loads that cover most of the effects of Lorries and cars (main model).
- Load Model. 2 - Single axle load applied on specific tire contact area which covers the dynamic effects of the normal traffic on short structural members.
- Load Model. 3 - Set of special vehicles
- Load Model 4 - Crowd loading

5.1.2 VERTICAL FORCES ON THE CARRIAGEWAY

There are six groups of loads according to BS EN 1991-2:2003 table 4.4.a, for traffic load calculations. Load model 1 and load model 3 with gr1a and gr5 groups were considered in the design.

- gr1a - Characteristic LM1(TS and UDL)
- gr5 - Frequent LM1 (TS and UDL) + Characteristic LM3 (Special vehicles)

As the special vehicle, SV80 vehicles were selected analyzing the traffic flow in Peradeniya.

The carriageway was divided in to notional lanes according to BS EN 1991-2:2003, Table 4.1

Total length of the bridge	= 75 m
No: of spans	= 3
Length of a span	= 25 m
Width of the bridge	= 9 m
Carriageway width (w)	= 7 m According to the BS EN 1991-2:2003
Notional lane width	= 3 m
No: of notional lanes	= Int (w/3) = 2
Remaining area width	= 1 m

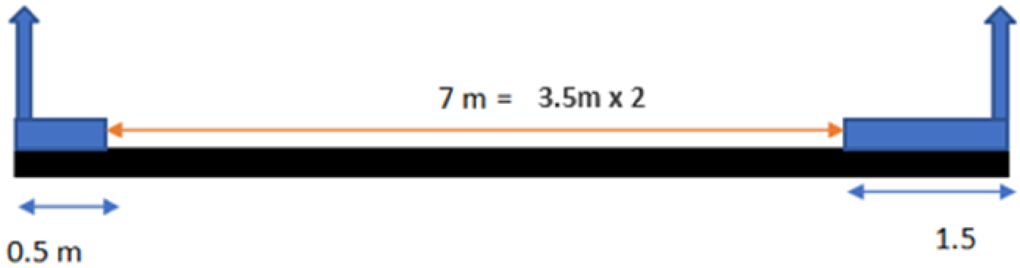

Figure 5.1 Cross-section of the bridge carriageway

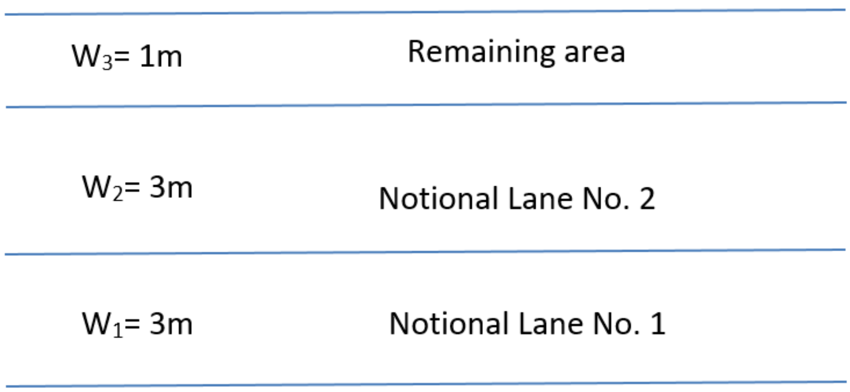

Figure 5.2 Notional lane arrangement

The detailed design of the Bridge loads are given in Appendix B. the summary of the loads are as below. Initially, Maximum bending moment and shear force for gr1a - Characteristic LM1 (TS and UDL) load combination and gr5 - Frequent LM1 (TS and UDL) + Characteristic LM3 (Special vehicles) load combinations were obtained.

gr1a - Characteristic LM1 (TS and UDL)

In gr1a load group a uniformly distributed load of 16.5 kN/m and a Tandem System load of 300 kN is acting on the bridge carriageway. Figure 5.3 shows the load along the length of one span.

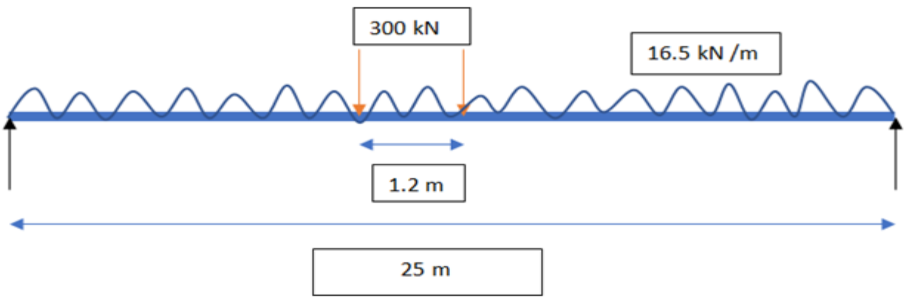

Figure 5.3 gr1a loading

- Maximum Bending Moment = 4860.65 kNm per 3m lane width occurring at 12.3 m from the left end of the beam
- Maximum shear force = 791.85 kN per 3m lane width occurring at both ends of the beam.

gr5 – Frequent LM1 (TS and UDL) + Characteristic LM3 (Special vehicles)

In gr5 load group a uniformly distributed load of 12.375 kN/m, a Tandem System load of 225 kN and SV80 vehicle load of 150.8 kN is acting on the bridge carriageway. Figure 5.4 shows the load along the length of one span. For a special vehicle, SV80 was selected. That means the weight of the maximum special vehicle is 80tons for this bridge.

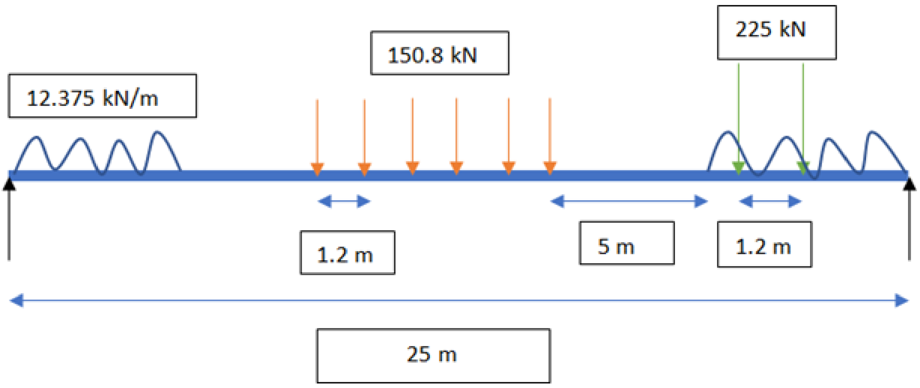

Figure 5.4 gr5 loading

- Maximum Bending Moment = 6082.59 kNm per 3m lane width occurring at 12.3 m from the left end of the beam

= 2027.53 kNm /m

- Maximum shear force = 1093.11 kN per 3m lane width occurring at both ends of the beam.

=364.37 kN/m

Considering both load groups the maximum lead was achieved using the gr5 load group. Therefore, values obtained using gr5 were used in the design.

Design BM	= 2027.53 kNm /m
Design shear force	= 364.37 kN/m

5.1.3 HORIZONTAL FORCE OF THE CARRIAGEWAY CALCULATION

There are two critical groups of loads. They are,

1. gr2 – (LM1) Axial (Breaking + Acceleration) + Lateral (Centrifugal) Forces
2. gr6 – (LM3) Axial (Breaking + Acceleration) + Lateral(Centrifugal) Forces

Since, bridge is straight centrifugal forces are not acting on the bridge.

gr2 – (LM1) Axial (Breaking + Acceleration) Forces

Longitudinal Breaking force Q_{lk} = 427.50 kN

Longitudinal Acceleration force = 213.75 kN

gr6 – (LM3 -SV80) Axial (Breaking + Acceleration) Forces

Longitudinal Breaking force Q_{lk}	= 452.4 kN
Longitudinal Acceleration force	= 470.88 kN
Lateral forces on bridge deck	= 226.2 kN

Considering both load groups,

Design Longitudinal Breaking force Q_{lk}	= 452.4 kN
Design Longitudinal Acceleration force	= 470.88 kN
Design Lateral forces on bridge deck	= 226.2 kN

5.2 DESIGN OF SUPERSTRUCTURE

The loads acting on the bridge were calculated in section 5.1. After that, the superstructure was designed. For the superstructure, pre-tensioned prestressed concrete composite beams were used. Advantages of using a prestressed concrete bridge deck is given below.

- High-strength concrete and high-tensile steel are used in prestressed beams and besides being economical, make of slender sections are aesthetically superior.
- Prestressed concrete bridges can be designed without any tensile stress under service loads, thus resulting in a crack-free structure.
- In comparison with steel bridges, prestressed concrete bridges require very little maintenance.
- Total construction time is substantially reduced when precast concrete elements are used.
- Prestressed concrete is ideally suited for composite bridge construction in which precast prestressed girders support the cast in-situ slab deck. This type of construction is very popular since it involves minimum disruption of traffic.
- Pre-tensioning in the plant is more cost-effective than post-tensioning on site. Because the precast prestressed concrete element is factory-produced and contains the bulk of reinforcement, rigorous quality control and higher mechanical properties can be achieved at relatively low cost. The cast in situ concrete slab does not need to have high mechanical properties and thus is suitable for field conditions.
- The precast prestressed concrete units are erected first and can be used to support the formwork needed for the cast in situ slab without additional scaffolding (or shoring).
- In addition to its contribution to the strength and stiffness of the composite member, the cast in situ slab provides an effective means to distribute loads in the lateral direction.
- The cast in situ slab can be poured continuously over the supports of precast units placed in series, thus providing continuity to a simple span system.

In this design, a simple slab and girder type composite section was used. The in-situ reinforced concrete slab is casted on top of the precast girders. The bridge length was decided as 75 m from layout plan and 25 m in length three spans were decided to use in the design. The width of the bridge deck was selected as 9m with a carriageway width of 7m. Only one side of the bridge is used as a side walk for pedestrian.

Prestressed Y girder beams were used in this design. The beam selection tables were reproduced from the Prestressed Concrete Association literature which are based on BS 5400 loading with 45 units of the HB vehicle, and include an allowance of 2.4kN/m^2 for finishes. For a 25 m long beam, Y6 beam was selected in the design using figure 5.5

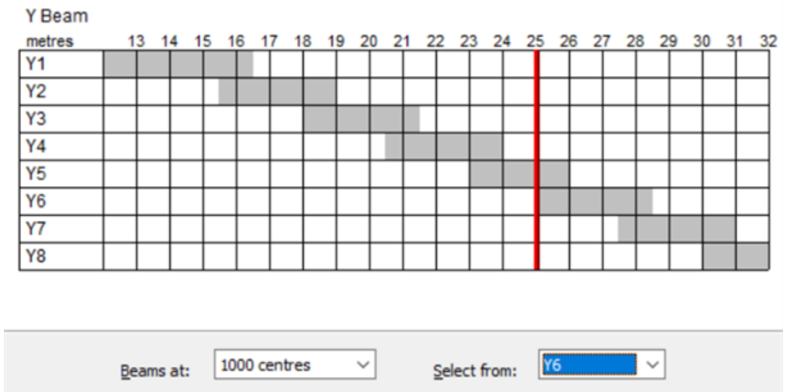

Figure 5.5 Pre-tensioned beam initial sizing graph

The Y6 beam cross section is given in figure 5.6

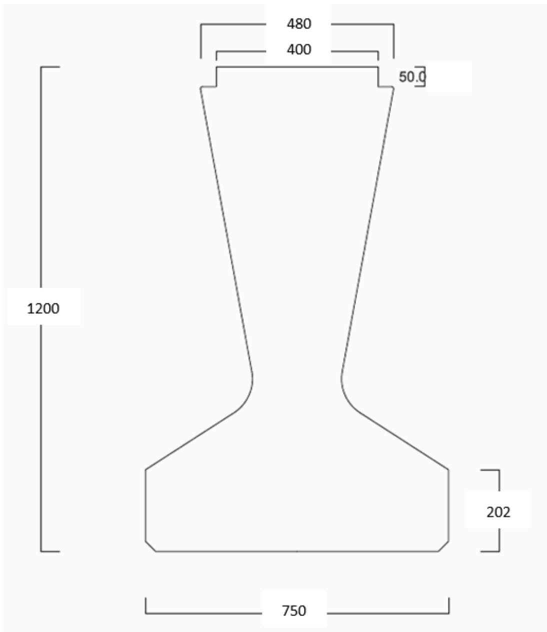

Figure 5.6 Dimensions of the pre-stress beam (all dimensions are in mm)

A 200 mm in situ concrete slab is cast on top of the Y6 beams making the beam and slab section act as a composite beam as figure 5.7.

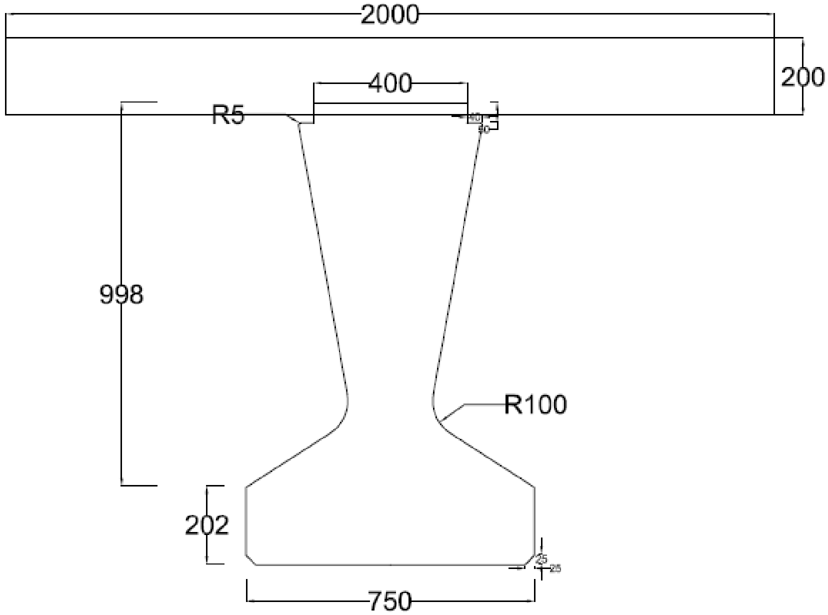

Figure 5.7 Cross-section of the composite beam (all dimensions are in mm)

In accordance with Euro code 2: Design of Concrete Structures EN1992-1-1: 200,

- Depth / Span = 1.2m/25m =6/125
- Maximum spacing = 2190 mm
- Spacing = 2000 mm spacing between 2 beams.
- No of beams = 5

The width of the bridge deck is 9m with a carriageway width of 7m. Figure 5.8 shows the cross-section of the deck with the beams.

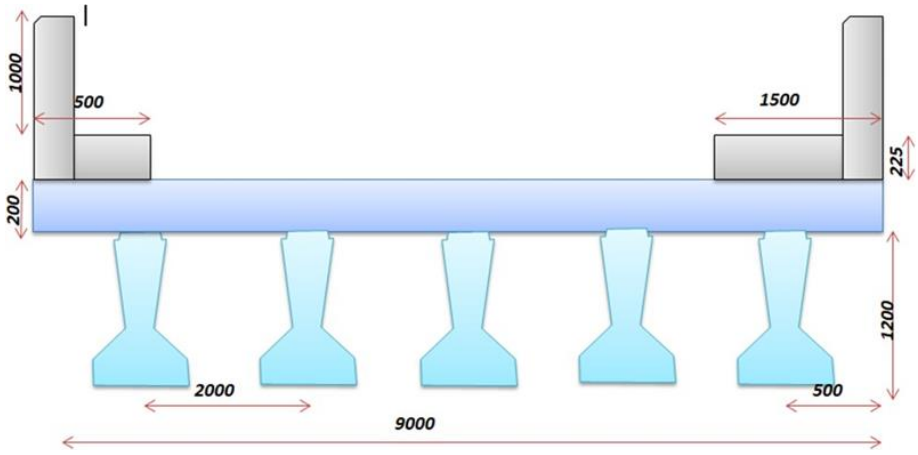

Figure 5.8 Cross-section of the beam deck

5.2.1 MATERIALS

For the prestressed pre-tensioned beam, C50/60 concrete grade with 16 mm diameter, Y186OS7 type strands were used. Figure 5.9 and figure 5.10 shows the properties of concrete properties and strand properties of prestressed beam respectively.

Concrete - Parabola-Rectangle	
Design Code Part	○ EN 1992-1-1 ◉ EN 1992-2
⊟ Characteristic Strength	
f_{ck}	50
$f_{ck,cube}$	60MPa
Modulus of Elasticity, E_{cm}	37.278GPa
Poisson's Ratio, ν	0.2
Shear Modulus, G	15.532GPa
Ultimate Compressive Strain, ε_{cu}	0.0035
Tensile Strength, f_{ctm}	-4.0716MPa
Cement Class	N: Normal and rapid hardening
Contains Silica Fume	☐
Coefficient of Thermal Expansion	1E-5/°C
Density	24kN/m³
Density Increase for Reinforcement	1kN/m³

Figure 5.9 Details of C50/60 concrete in accordance with EN 1992-2

Prestressing Steel - Inclined	
0.1% Proof Strength, $f_{p0.1k}$	1600MPa
Characteristic Tensile Strength, f_{pk}	1860MPa
Characteristic Strain Limit, ε_{uk}	0.0222
Modulus of Elasticity, E_p	195GPa
Relaxation Class	Class 2
Relaxation Loss After 1000 Hours	2.5%
Density	77kN/m³

Figure 5.10 Details of pre-stressed strands

For the in-situ concrete, grade C32/40 concrete with 25 mm reinforcement bars were used. Figure 5.11 and Figure 5.12 shows the properties of the concrete and reinforcement bars respectively.

Concrete - Parabola-Rectangle	
Design Code Part	○ EN 1992-1-1 ◉ EN 1992-2
⊟ Characteristic Strength	
f_{ck}	31.875MPa
$f_{ck,cube}$	40MPa
Modulus of Elasticity, E_{cm}	33.314GPa
Poisson's Ratio, v	0.2
Shear Modulus, G	13.881GPa
Ultimate Compressive Strain, ε_{cu}	0.0035
Tensile Strength, f_{ctm}	-3.0159MPa
Cement Class	N: Normal and rapid hardening
Contains Silica Fume	☐
Coefficient of Thermal Expansion	1E-5/°C
Density	24kN/m³
Density Increase for Reinforcement	1kN/m³

Figure 5.11 Details of C32/40 concrete in accordance with EN 1992-2

Reinforcing Steel - Inclined	
Yield Strength, f_{yk}	500MPa
$k = (f_t / f_y)$	1.08
Modulus of Elasticity, E_s	200GPa
Characteristic Strain Limit, ε_{uk}	0.05
Density	77kN/m³

Figure 5.12 Details of reinforcements

5.2.2 SECTION PROPERTIES

The composite beam's section properties are given below. In this, element ref. 1 is the prestressed beam and element ref 2 is the slab section.

Overall dimensions	height	= 1.37 m
	Width	= 2.0 m
Centroid coordinates	*y*	= 0.000mm
	z	= 849.669 mm

Cross section area = 881721.47 mm²

External surface area = 7472.7825 mm²/mm

About global centroidal axes:

Second moment of area I_{yy} = 1.9708E11 mm⁴

I_{zz} = 1.4454E11 mm⁴

Section modulus W_{yt} = 3.78755E8 mm³

W_{yb} = -2.3195E8 mm³

Initially, the tendon profile was selected for the composite beam. Then, the beam was designed for the erection loads during construction, construction stage 1 loads, temporary loads and support removals, surfacing loads and live loads. Differential temperature analysis, shrinkage and creep analysis was done to the beam, finally, Prestress losses, limiting stresses and SLS flexure and shear reinforce requirement was found. The final tendon profile is shown in figure 5.13. The detailed calculations are given in Appendix C.

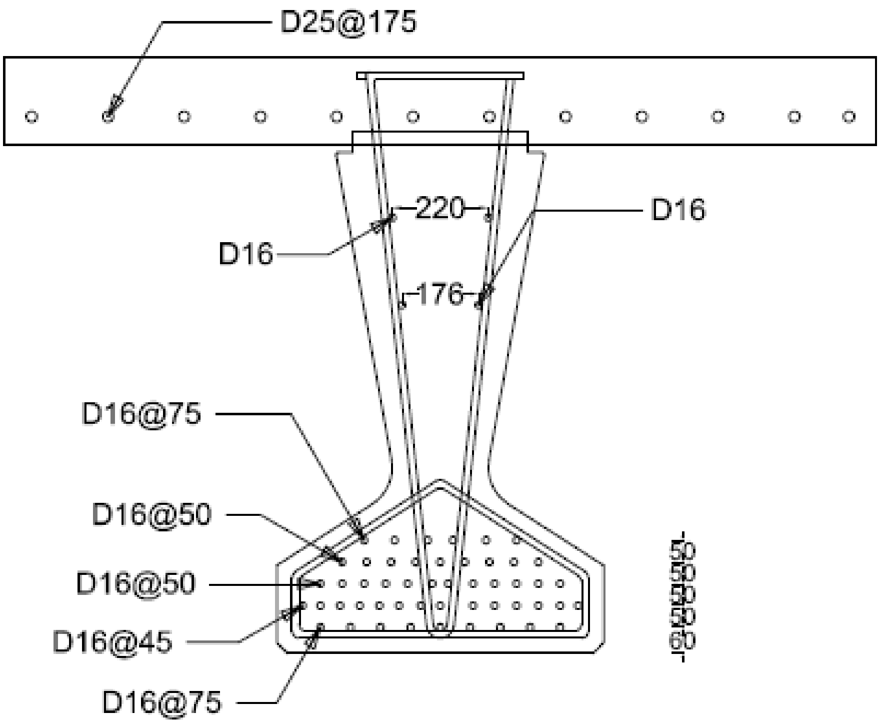

Figure 5.13 Tendon and R/F in the composite section

5.3 PIER DESIGN

5.3.1 PIER TYPE SELECTION

The selection of the type of piers for a bridge should be based on functional, structural, and geometric requirements. Aesthetic appearance is also a very important factor of selection because modern highway bridges are often a part of the landscape of a city.

The following figure 5.14 shows typical cross-section shapes of piers for river and waterway crossings. (These figures were taken from 'Bridge Engineering Handbook, Second Edition, Substructure Design', by W. Chen and L. Duan, 2014, Boca Raton, p. 37).

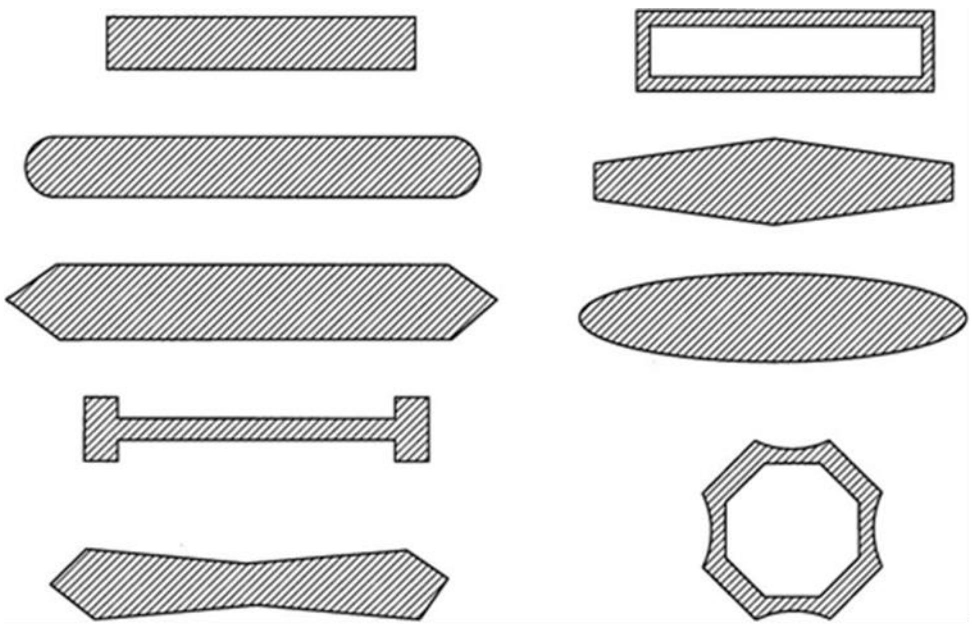

Figure 5.14 Typical cross sections of piers.

The figure 5.15 shows typical types of Piers. These figures were taken from 'Bridge Engineering Handbook, Second Edition, Substructure Design', by W. Chen and L. Duan, 2014, Boca Raton, p. 37-38.

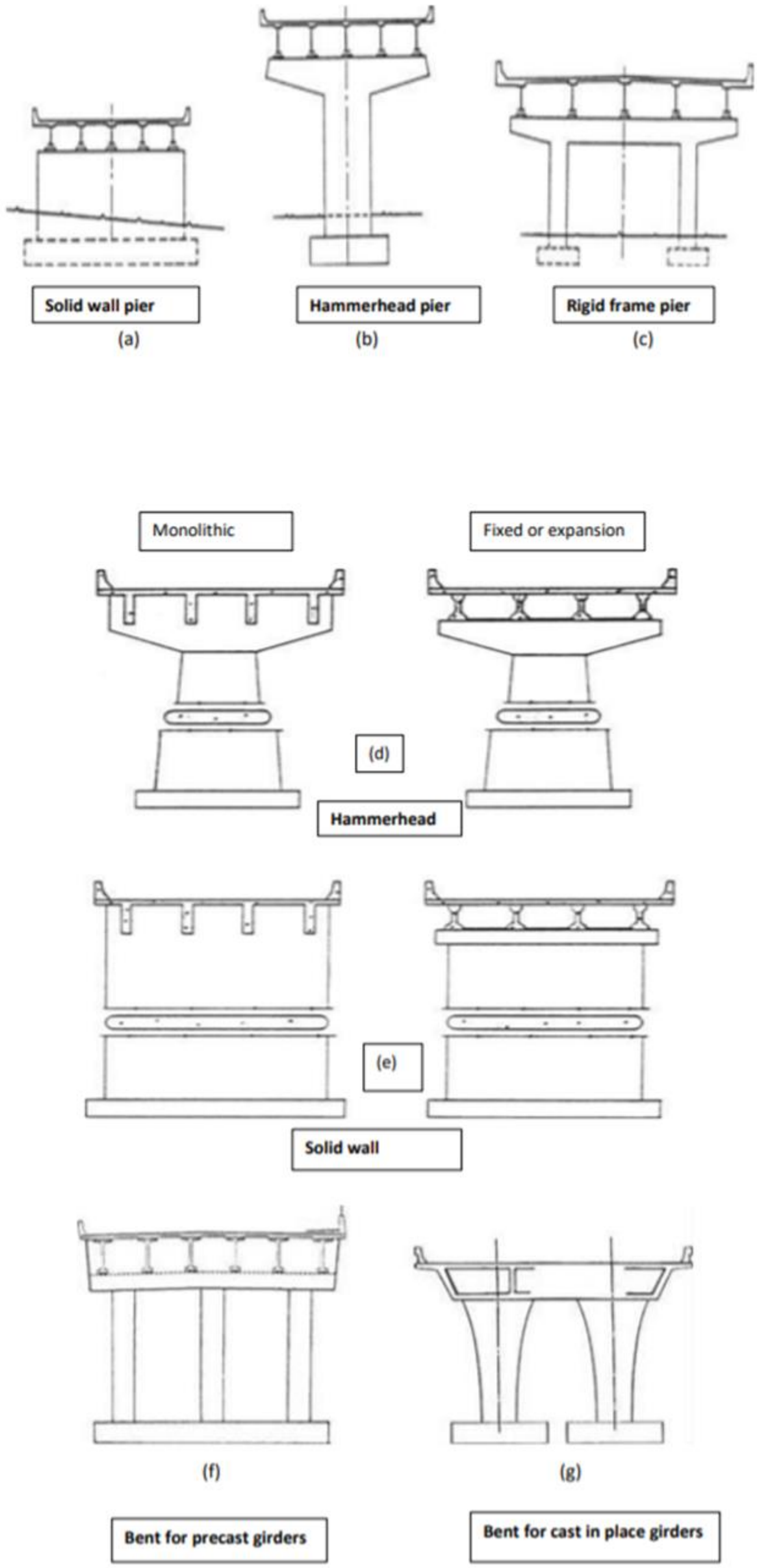

Figure 5.15 Typical types of piers.

Selection of pier type depend on the following factors,

- Type of superstructure
- Location of the bridge
- Height of piers

In this project precast prestressed concrete girders were selected as the superstructure type. According to the Bridge Engineering Handbook, Second Edition, Substructure Design 2014, there are guidelines when selecting a pier type for that type of superstructure. Those guidelines are given in table 5.6.

Table 5.1 Classification of Pier types

Location of the bridge	Tall or Short piers	Applicable pier types
Over water	Tall	b, c, d, e
	Short	a, b
On land	Tall	b, c
	Short	g

According to the longitudinal profile the required pier height is 21 m. Since the location of the bridge is over water and pier height is ***tall*** (21 m). Hammerhead pier type (figure a) was chosen. Also, this pier type is the most common type which is used in Sri Lanka when constructing expressway bridges, normal bridges and etc. According to the Bridge Engineering Handbook, Second Edition, Substructure Design 2014, pier dimensions were selected as in figure 5.16 and figure 5.17.

Advantages,

- Better in aesthetic point of view
- Generally, occupy less space
- Providing more room for the traffic underneath
- Can use for both steel girder or precast prestressed girder superstructure

5.3.2 PIER COMPONENTS

Figure 5.16 and figure 5.17 shows the pier components. Detailed design of pier is given in appendix D.

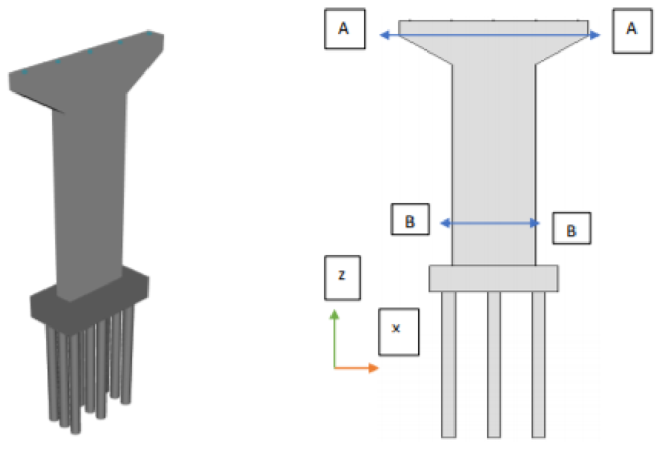

Figure 5.16 2D and 3D view of the pier

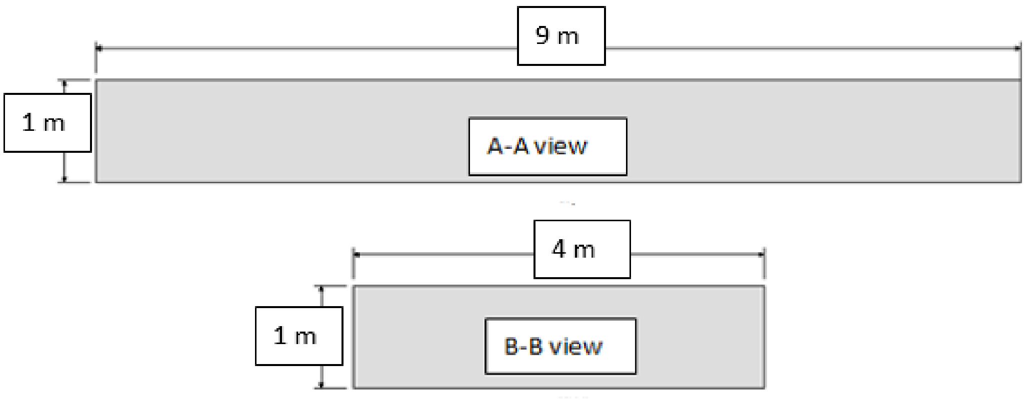

Figure 5.17 Sectional views of the pier

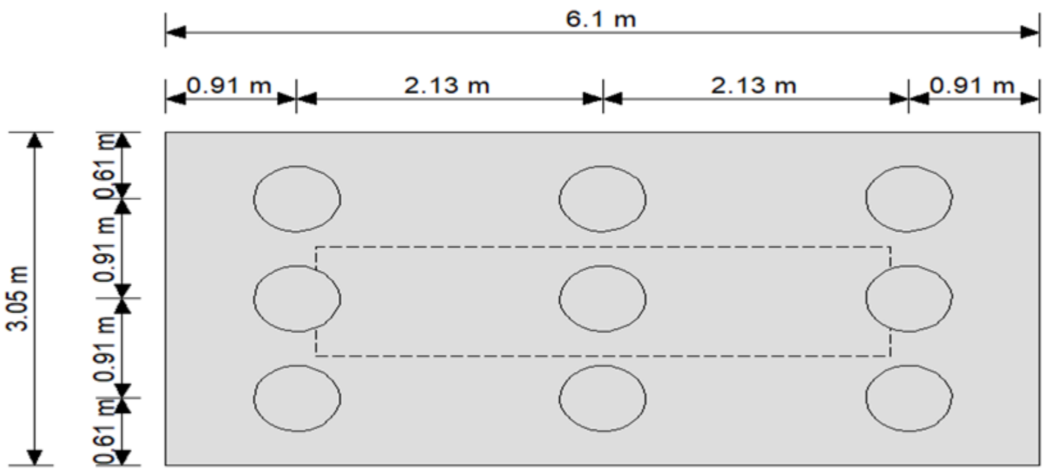

Figure 5.18 Details of pile cap

5.3.3 SCOURING EFFECT WITH PIER SHAPE

Scour is the local lowering of stream bed elevation which takes place in the vicinity around a structure constructed in flowing water. Scour takes place around bridge piers, abutments, around spur, jetties and breakwaters due to modification of flow pattern in such a way as to cause increase in local shear stress. To calculate the scouring effect with pier shape, Gampathi G. A. P. (2010) suggested as,

$$\mathbf{d_s/y = 2*K1*K2*(b/y)^{0.65}*Fr1^{0.43}}$$

Where d_s is scour depth, y is flow depth at the upstream of the pier, K1 is correction factor for pier nose shape, K2 is correction factor for the angle of attack flow, b is the pier width and Fr1 is the Froude number at upstream of the pier. L is the pier length.

L =4 m, b = 1 m, y = 4 m, v = 5 m/s

Fr1 = $v/(gy)^{0.5}$ = 0.79

K1 = 1.1 (rectangular pier shape) and L/b = 4,

K2 max = 2.5 (when angle is 90)

$d_s/y = 2*1.1*2.5*(1/4)^{0.65}*(0.79)^{0.43} = 2.01 < 2.4$ -OK

It is recommended that the limiting value of ds/y is 2.4 for Fr1≤ 0.8 and 3.0 for Fr1 >0.8.

Hence pier shape is ok.

Also, scour depth was calculated using another method using Melville and Sutherland (1988). That scour calculations are given in appendix D. According to the literature, complex model simulations is required to setup for give proper idea about sediment accumulation around piers.

5.3.4 PIER DESIGN

There are mainly two parts in a pier. Those are pier head and pier stem. In addition, there is a pile cap. Eurocode 2 was used as the design guide. Following methods was used for the design of each component.

- Pier head – STM (Strut-and-Tie Modelling) following Eurocode 2
- Pier stem (column) – Eurocode 2
- Pile cap - Strut-and-Tie Modelling following Eurocode 2

For the design, required Bridge Superstructure data are,

- Girder spacing – 2m
- Number of girders - 5
- Span length – 25m

In this design C 32/40 in situ concrete and yield steel were used.

Concrete density = 2400 kg/m^3

Concrete strength = 32 MPa

Elastic modulus of concrete = 33.314 GPa

Steel yield strength = 500 MPa

Loads on piers,

- Load of the superstructure (dead load)
- Superimposed dead loads
- Traffic loads (live loads) – UDL +tandem system
- Breaking force
- Wind load on the structure
- Stream pressure

Pier cap design

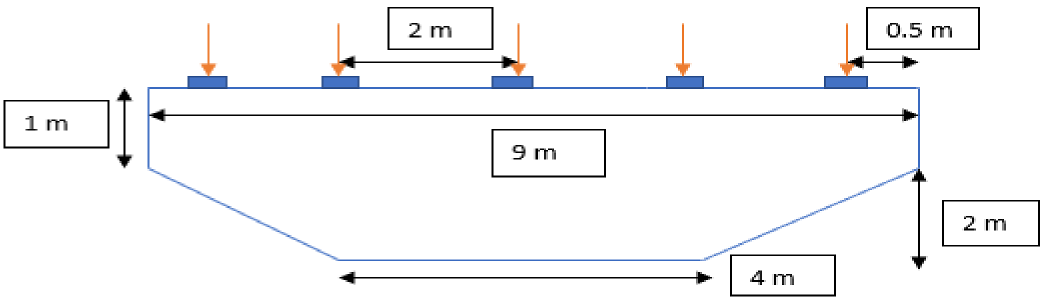

Figure 5.19 Details of pier cap

First the loads which are affecting on Piers were found. Then STM was developed using 16 nodes and 29 struts and ties with 2 supports. Strut width is 220 mm. Then forces were calculated using equilibrium of STM. Then design was carried out using EC 2. Detailed design is given in appendix D.

Reinforcement in Pier head

Main reinforcement bar sizes = 25 mm for Top and Bottom

The vertical ties represent the centroid of stirrups that will be spaced across a “stirrup band”. For this H10 4-legged stirrup bands were used as per figure 5.20.

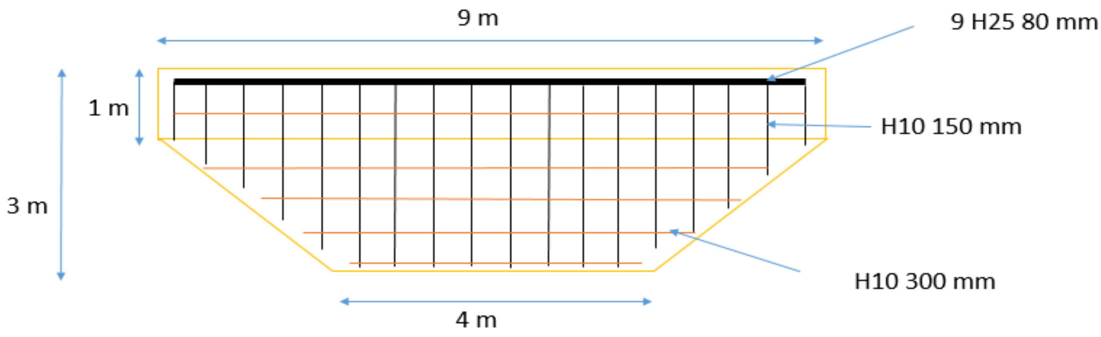

Figure 5.20 Reinforcement layout of pier head

5.3.5 PIER STEM (COLUMN) DESIGN

A rectangular pier column was designed to have a depth (y) of 1 m. The same method and materials used in Pier cap design were used for this design too. The figure 5.21 shows the final dimensions of the pier stem.

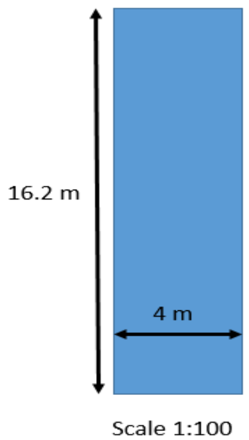

Figure 5.21 Dimensions of pier stem

Reinforcement in Pier stem

Main reinforcement – 32mm bars

Shear links – 4-legged 10mm bars used.

No of bars should be symmetric. Bars = 108

So, 4 layers with having 27 bars with 115 mm were assigned.

Transverse links spacing 225 mm.

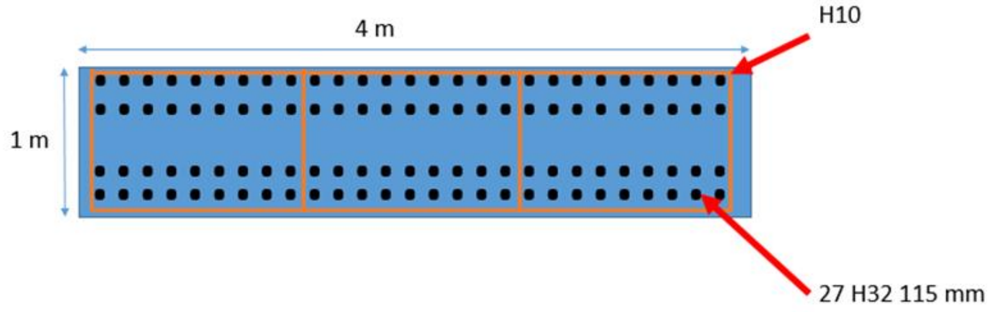

Figure 5.22 Reinforcement layout of pier stem

5.3.6 PILE CAP DESIGN

The same method and materials used in Pier cap design were used for this design too. A rectangular type pile cap was designed. Figure 5.23 shows the dimensions of the Pile cap.

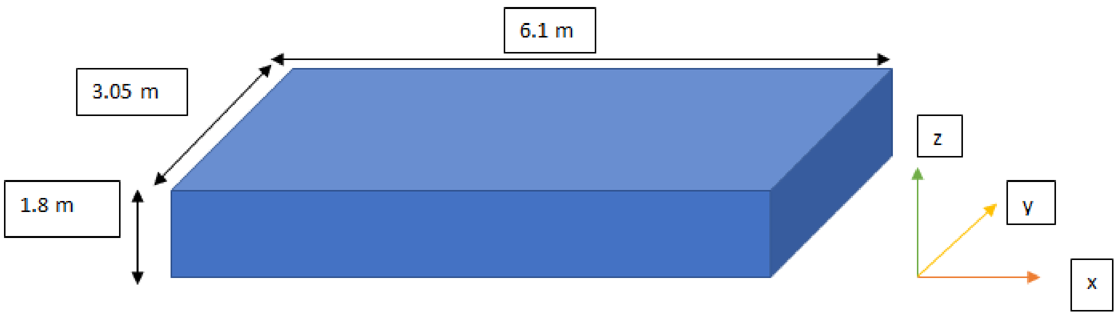

Figure 5.23 Dimensions of pile cap

In X direction, STM was designed with 7 strut and ties with 3 supports and in Z direction STM was designed with 7 strut and ties with 3 supports. Then forces were calculated using equilibrium of STM.

Reinforcement in pile cap

Main reinforcement bar size = 25 mm (for both directions)

Cover is 50 mm.

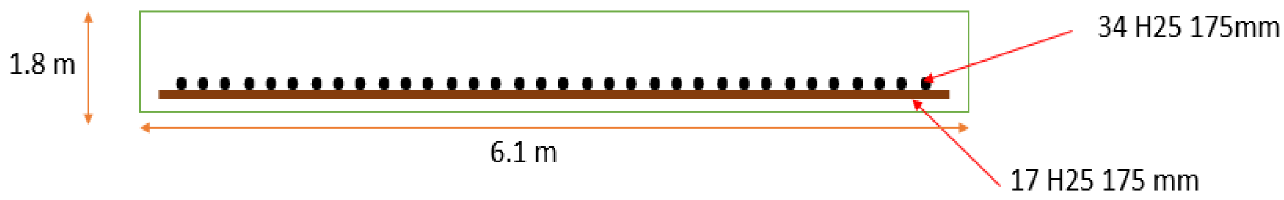

Figure 5.24 Reinforcement layout of pile cap

5.3.7 Pile design

The design Method based on Euro code 7 (Pile Design and Construction Practice book, fifth edition', by M. Tomlinson and J. Woodward, 2008, New York). The reasons for the selection pile foundation were,

- Structural loads are high. Thus, spread footing would be enormous.
- Upper soils are subjected to scouring or undermining at the pier of the bridge.

Piles can be generally classified based on the,

- Mode of installation (driven or bored)
- Degree of soil displacement during installation
- Their size (large diameter, small diameter)
- Pile material (concrete, steel, timber and composite)

Most important classification is based on the installation method. They are,

- End bearing piles
- Friction piles

According to the longitudinal profile, bed rock is located about 5 m below the river bed at pier constructing locations. So, there is a 5 m soil layer. The pile can be classified as an end bearing pile. Therefore, the pile needs to be socketed to the rock.

The term ‘socket piles’ (or rock sockets), refers to a technique that is used to embed a pile into solid rock. This is necessary to utilize the full structural capacity of the piles for both compressive and tensile forces.

Hence, bored and cast in-situ concrete piles are only used as the pile type. Bored piles are non-displacement piles. There advantages of using bored piles are,

- Minimum soil disturbance around the pile and are quiet.
- Can be used in congested urban areas.
- Complex shapes can be formed, including under reaming.

For the design following parameters are required.

- The properties of the rock.
- The presence of fractures in the rock.
- The size and spacing of any fractures.
- The degree of weathering of fractures.
- The presence of any soil within fractures.

Pile design was based on EC7 and all the design calculations are given in Appendix D.

Reinforcement for the piles

Longitudinal bar size = 25 mm

Outer ring size = 10 mm

Cover = 75 mm

Spacing of rings = 150 mm

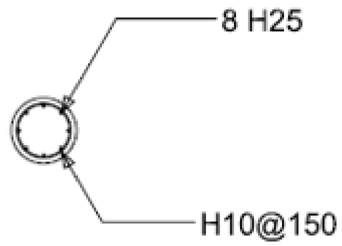

Figure 5.25 Reinforcement layout of pile cross section

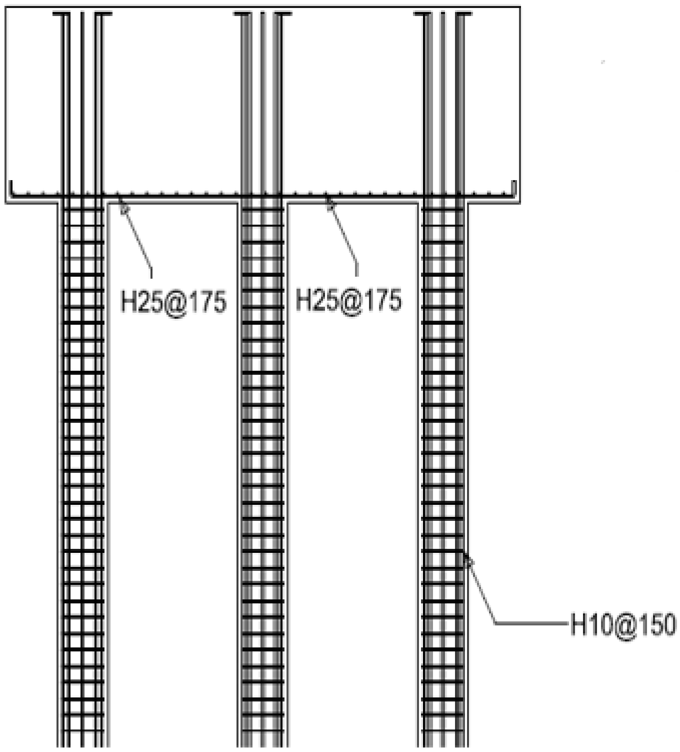

Figure 5.26 Total reinforcement layout of piles and pile cap

5.4 ABUTMENT DESIGN

5.4.1 INTRODUCTION

Abutment is used to retain the embankment and resist and transfer vertical loads and horizontal loads from the superstructure to the foundation. Mainly they can be divided in to two categories (EUR 25193 EN - 2012).

- Open end abutments
- Closed end abutments

Open ended abutments

When there is s slope between bridge abutment face and the edge of the roadway or channel as shown in figure 5.27 it is called an open end abutment. The advantages of using these type of bridge abutments are,

- Less impact to the environment
- future widening is easy by adjusting slope ratios

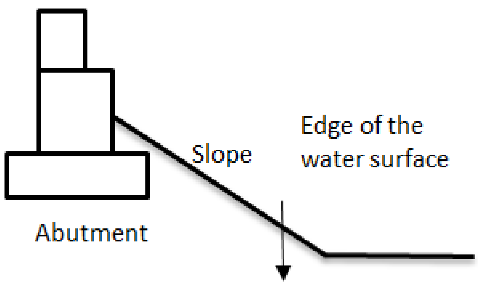

Figure 5.27 Slope between bridge abutment face and the edge of the roadway or channel

Closed end abutments

Closed end abutments are close to edge of the roadways or channels as shown in figure 5.28. Closed-end abutments have been widely used in urban areas and for rail transportation system because of the right of way restriction.

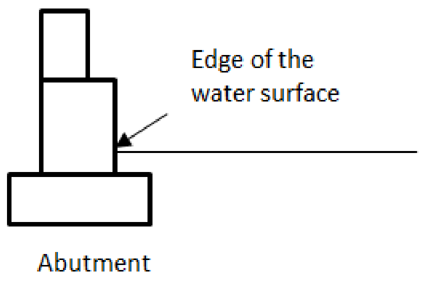

Figure 5.28 Close end abutment

Those categories can be divided further as monolithic and seat type abutments.

- Monolithic abutments – There is no relative displacement between the abutment and the superstructure.
- Seat type abutments – abutment and superstructure are constructed separately

For this project, closed end seat type abutments were selected. Parts of an abutment are shown in figure 5.29.

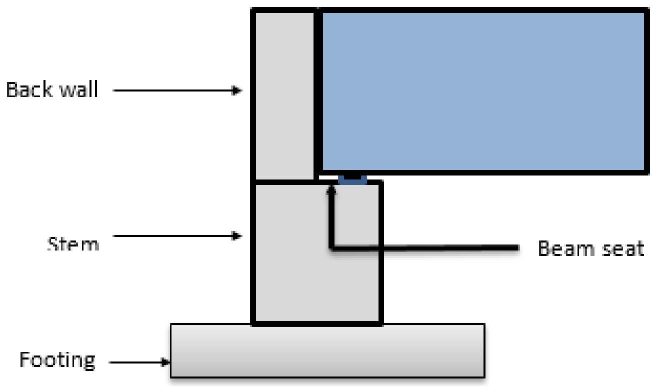

Figure 5.29 Parts of the abutment

Mainly there are three types of loads acting on the abutment,

1. Vertical reaction on supports - Self weight, Traffic UDL, Traffic TS, loads from nominal nonstructural equipment
2. Horizontal traffic action effects – Braking force, Acceleration force
3. Horizontal wind action effects

According to the longitudinal profile of the site, the height of the abutment should be 10.5m for the left abutment and 14m for the right abutment. Therefore, abutments with different heights were required for the left and right side as shown in figure 5.30.

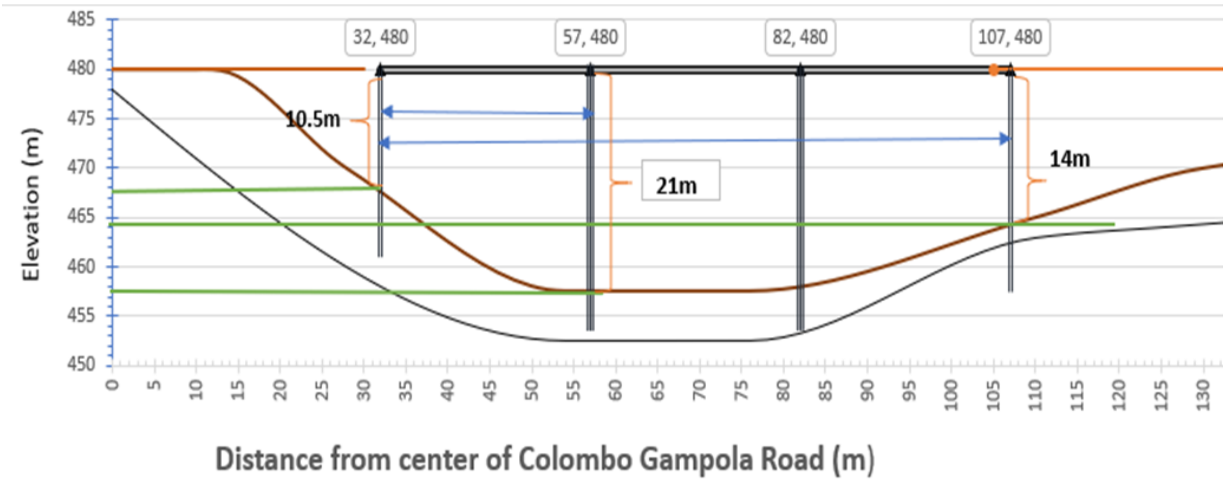

Figure 5.30 Longitudinal profile of the new bridge

5.4.2 LEFT ABUTMENT DESIGN

The height of the abutment should be 10.5m according to longitudinal profile and length (l) of the abutment should be 10m according to superstructure details. Then, dimensions were decided considering the geotechnical conditions of the site.

Initially, a spread foundation was considered. Then, sliding, bearing and settlement for the abutment was checked and the shape and size of the abutment was obtained. The shape and size, were found to be satisfactory with sliding, bearing and settlement. The calculations are given in the Appendix E.

Thus, results were not satisfactory in the geotechnical design part because, bed rock is very near to the abutment footing according to longitudinal profile. Then, a pile foundation design was done as the second trial for the abutment. The design was satisfactory and taken as the final design. Dimensions of the left abutment is given in figure 5.31.

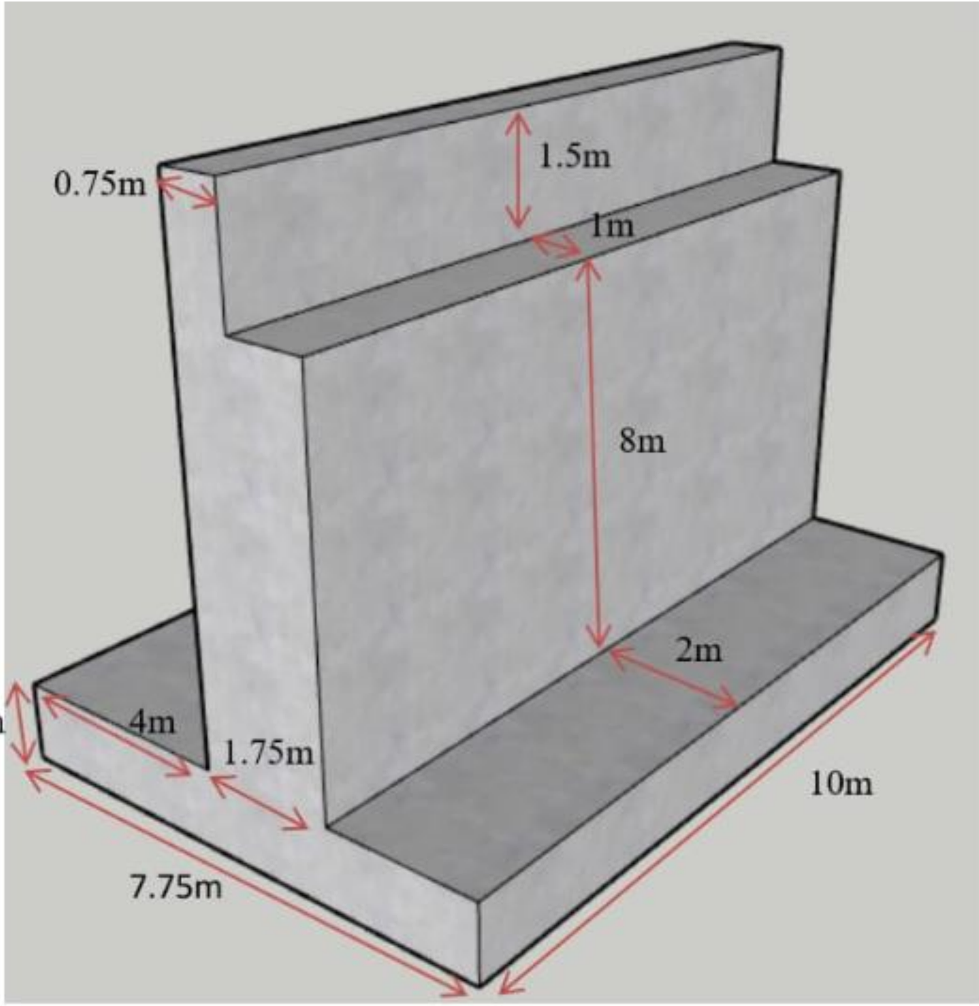

Figure 5.31 Dimensions of the left abutment

Material properties were taken as,

Cohesion(C)	= 20 kPa
Friction angle(Ø)	= 28˚
Specific gravity of soil(ϒsoil)	= 20 kN/m³
Specific gravity of concrete(ϒcon)	= 24 kN/m³

Forces acting on the left abutment are as shown in figure 5.32.

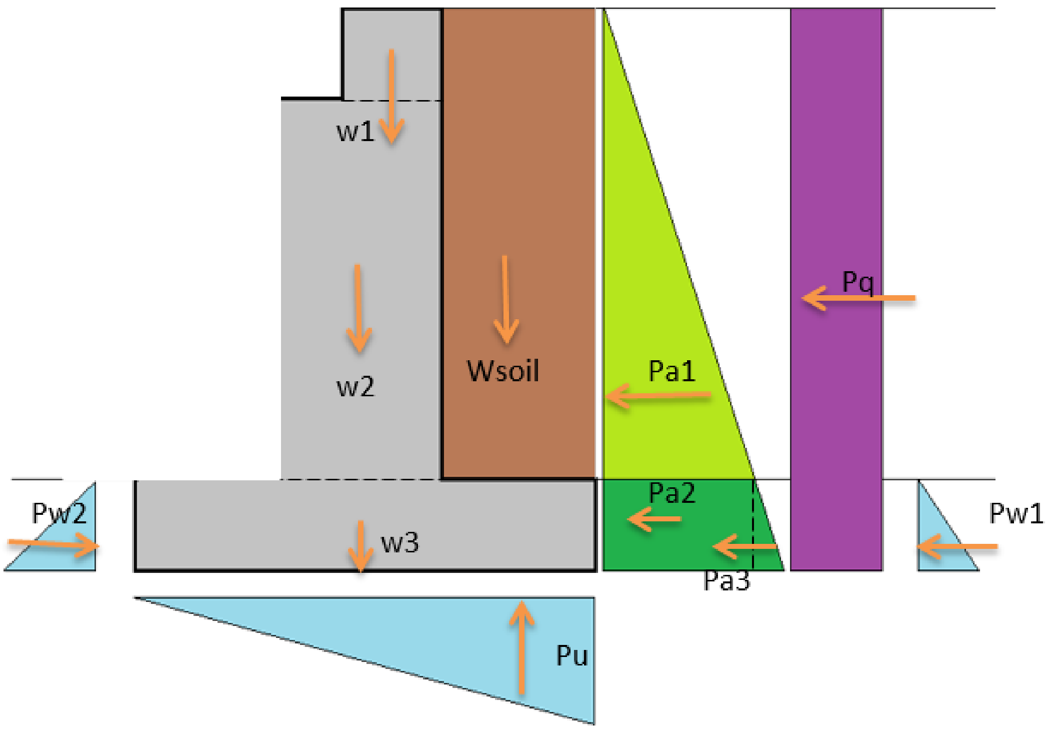

Figure 5.32 Forces acting on the abutment

Actions

Self-weight of the abutment,

W1	= 180 KN
W2	= 3360 KN
W3	= 2340 KN

Load from the soil,

Wsoil	= 9500 kN
Pa1	= 5625×K_a kN
Pa2	= 4500×K_a kN
Pa3	= 458.55×K_a kN

Surcharge,

Pq	= 26.25×K_a kN

Up thrust,

Pu	= 1434.71 kN

Water pressure,

Pw1 = 441.45 kN

Pw2 = 441.45 kN

Load from the deck,

Self-weight of the deck(Fg) = 314.345 kN

Super imposed load(Fq) = 31.25 kN

Traffic load(Ft) = 530.48 kN

Acceleration force(Fax) = 470.88 kN

Breaking force(Fbx) = 452.4 kN

Satisfactory results were obtained for bearing and sliding check for all the combinations. The relevant calculations are given in the Appendix E.

For the structural design part, the wall and footing were considered separately. In the wall design part, the three load cases were considered in calculating the critical bending moment. Values of bending moments are in table 5.2.

Table 5.2 Bending moment values for different load cases.

Load cases	Combination 01 (KNm)		Combination 02 (KNm)	
	permanent	variable	permanent	variable
Case 01	4469.49	16.92	3310.77	14.67
Case 02	3459.58	16.96	2572.13	14.67
Case 03	997.12	16.92	738.64	14.67

SLS bending moment and shear,

permanent = 3310.77 kNm

Variable = 11.28 kNm

Shear force = 906.95 kN

For the footing design part, two combinations were considered and the critical bending moment acting on the footing was obtained. Figure 5.33 shows the bending moment diagram along the footing. Reinforcement details for the left abutment is shown in figure 5.34. Footing heel and toe were considered separately.

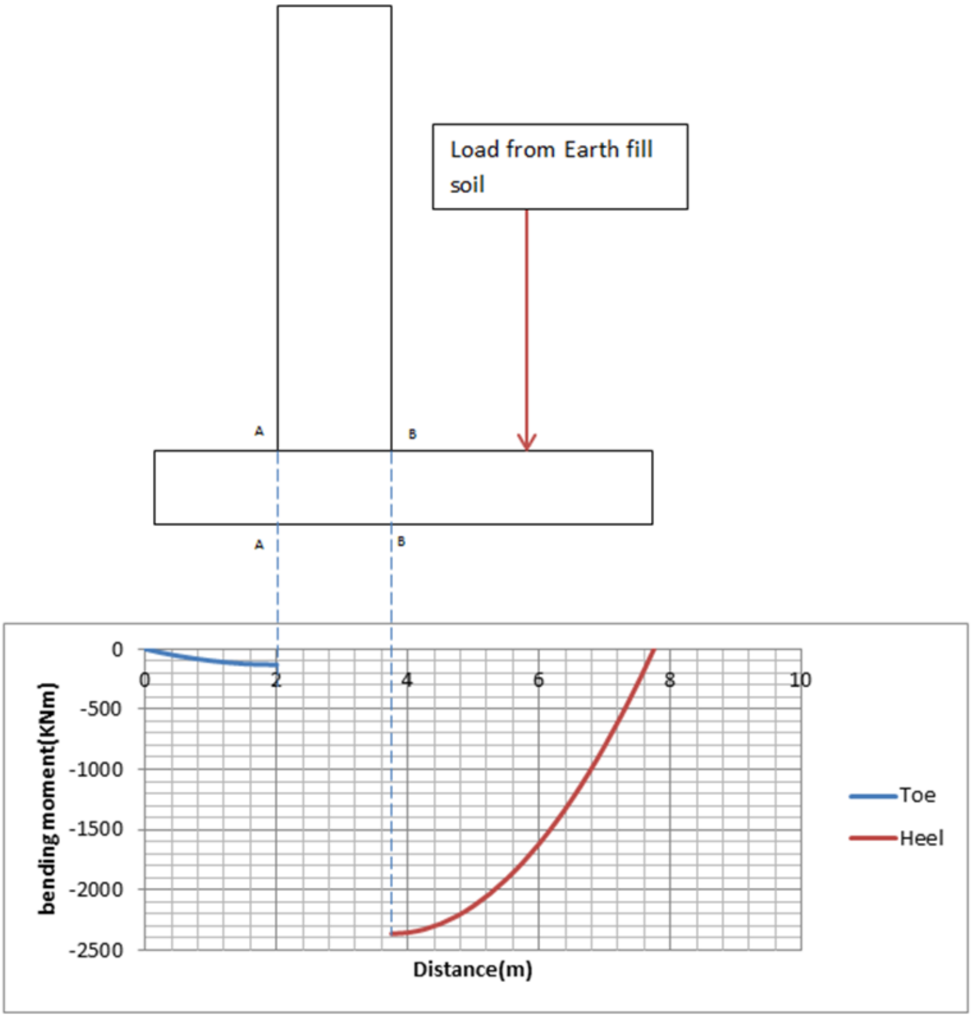

Figure 5.33 Bending moment diagram for toe and heel under combination 01

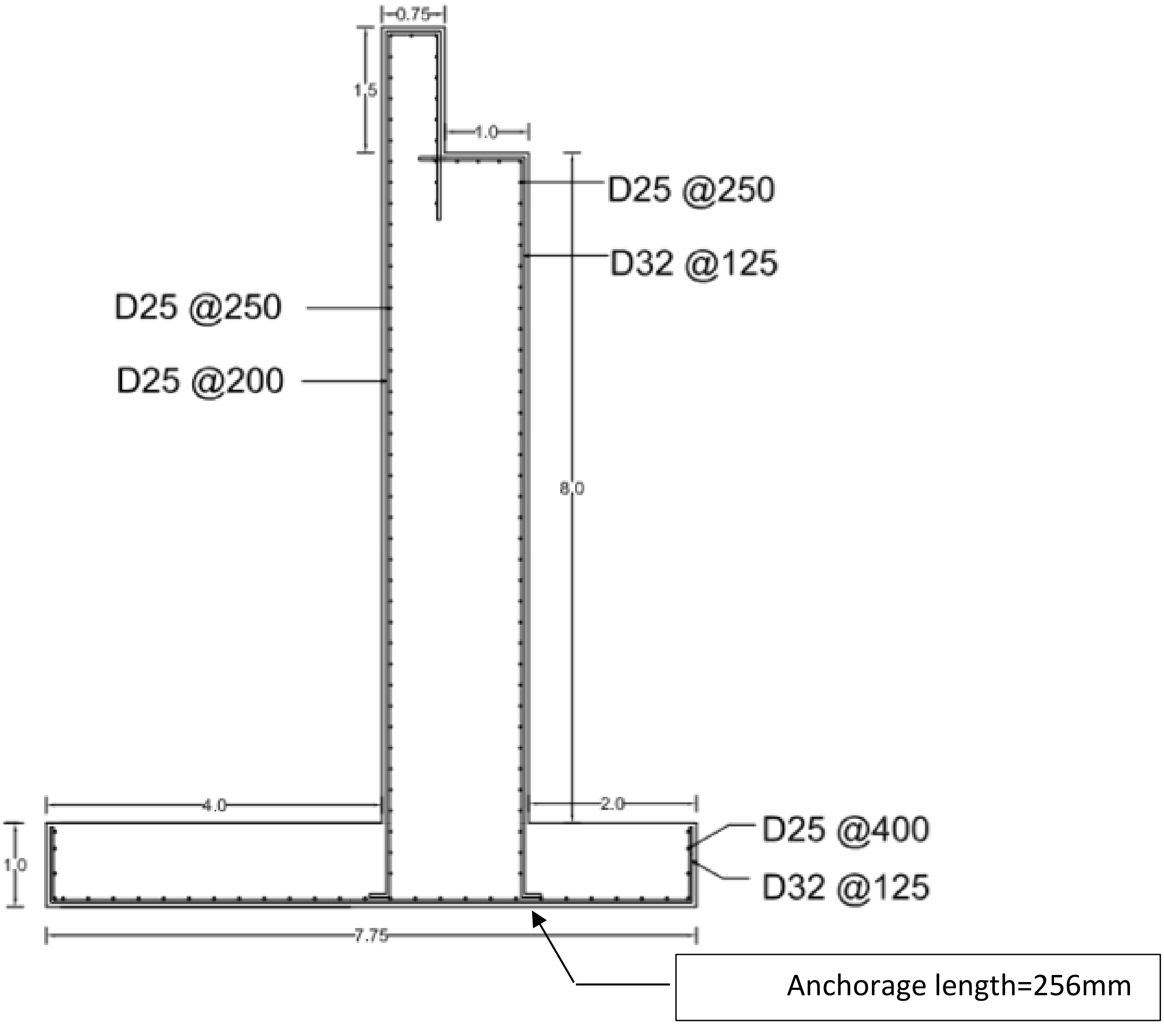

Figure 5.34 Reinforcement details for the left abutment

5.4.3 RIGHT ABUTMENT DESIGN

The same calculation procedure used in the left abutment design was used for the design of the right abutment. The height of the abutment was found as 14 m according to the longitudinal profile and width was taken as 10m.Dimensions of the right abutment are given in figure 5.35 and Reinforcement details for the right abutment is given in figure 5.36.

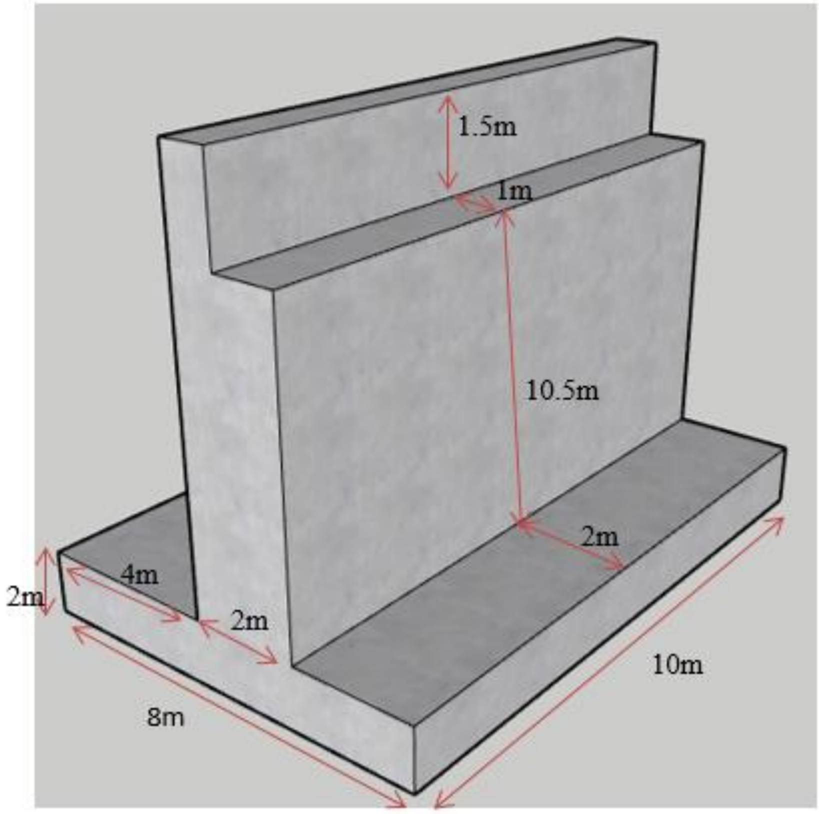

Figure 5.35 Dimensions of the right abutment

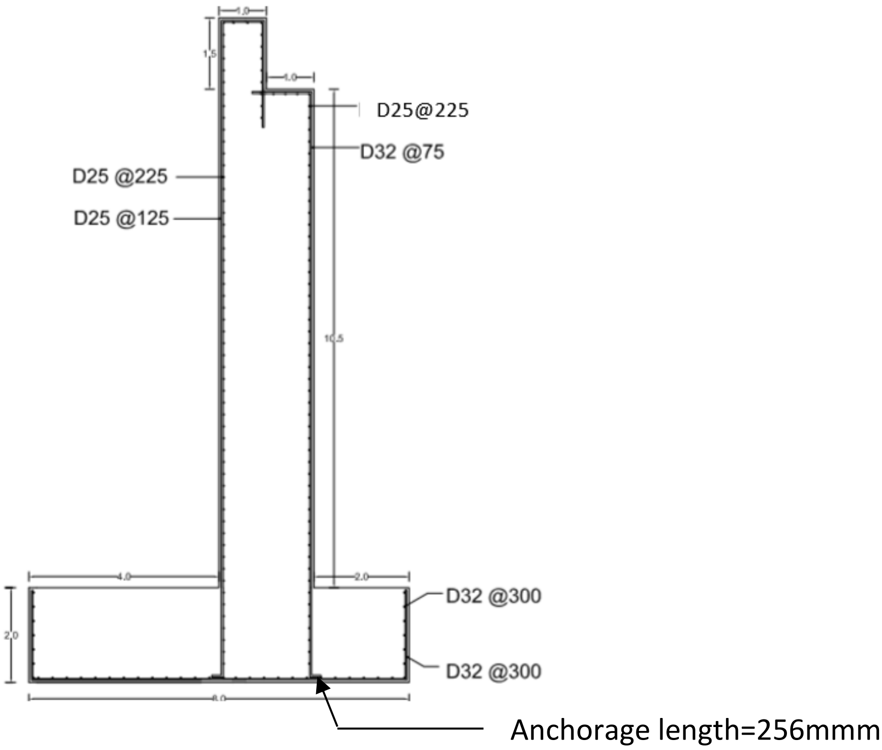

Figure 5.36 Reinforcement details for the right abutment

5.4.4 WINGWALL DESIGN FOR ABUTMENTS

There are three arrangements for the wing wall referring to AASHTO, 2012.

- Wingwall parallel to abutment:

Easy to build. Construction can be done in a short time. This type of wing wall will not disturb the existing embankment and utilities but it is not the most economical arrangement.

- Wingwall at an angle with abutments

This type is the most economical of the three arrangements.

- Wingwall perpendicular to abutment

In this arrangement. Wingwall provides a continuous alignment with bridge deck which can be used to support parapets.

The third type is selected as shown in figure 5.37. The wing wall toe is on top of the abutment toe to transfer the load on the wing wall to abutment footing.

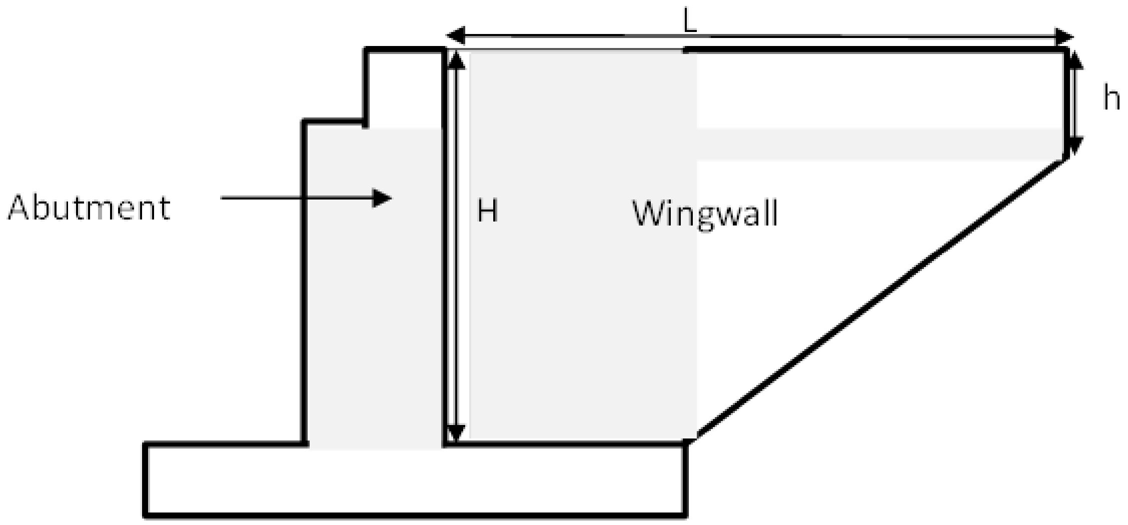

Figure 5.37 Abutment type

Reinforcement details of the wingwalls for right and left abutment are shown in figure 5.38 and 5.39 respectively.

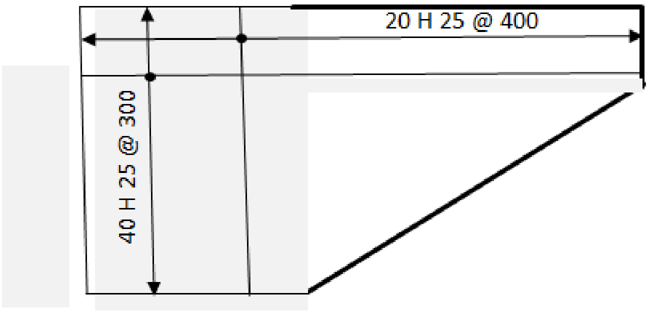

Figure 5.38 Reinforcement details of the wing wall for the right abutment

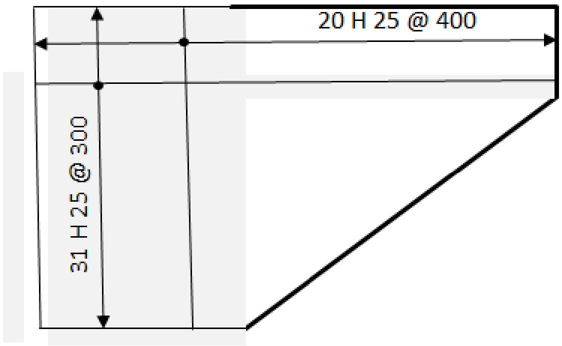

Figure 5.39 Reinforcement details of the wing wall for the left abutment

5.4.5 PILE DESIGN

The rock type was selected as weak jointed cemented mudstone.

For the reinforcement design of both of piles in left and right abutment,

Longitudinal bar size = 25 mm

Outer rings = 10 mm

Cover = 75 mm

Using 8 H25 bars,

Spacing = 150 mm < 200 mm -OK

Therefore, use 8 H25 @ 150

Plan view of left abutment pile cap and right abutment pile cap are shown in figure 5.40 and 5.41 respectively.

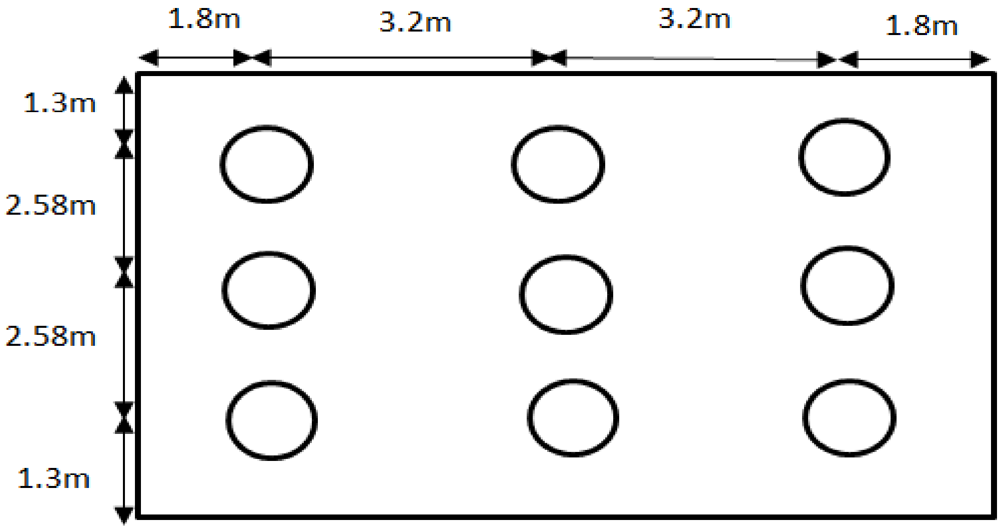

Figure 5.40 Plan view of left abutment pile cap

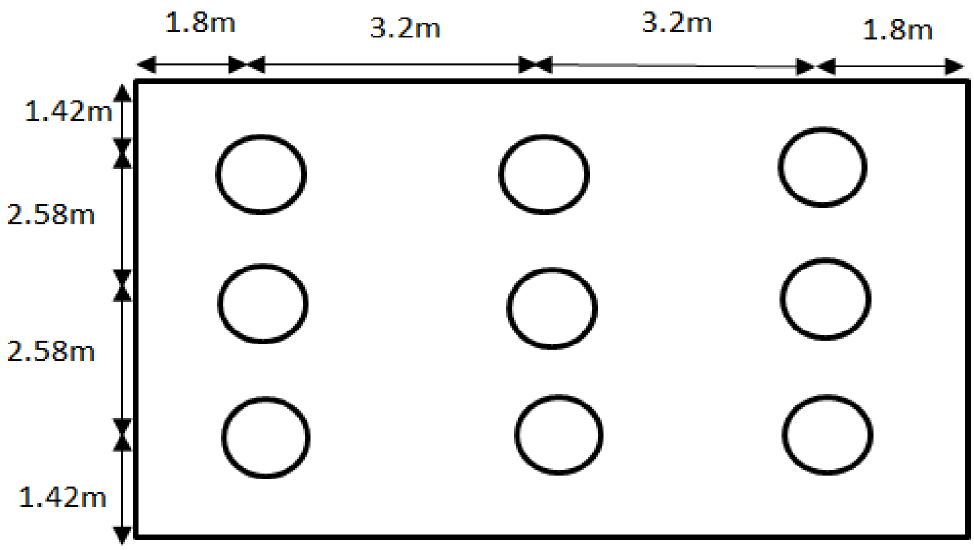

Figure 5.41 Plan view of right abutment pile cap

5.4.6 ABUTMENT SCOUR

Scour is the erosion caused by water of the soil surrounding a bridge foundation. There are three types of scour in a river

1. Long-term degradation of the river bed
2. Contraction scour at the bridge
3. Local scour at the piers or abutments

These three scour components are added to obtain the total scour at a pier or abutment. It was assumed that each component occur independently of the other.

The following methods can be used to estimate the scour at abutments:

- Froehlich's Abutment Scour Equation (when L/y_a < 25)
- HIRE Abutment Scour Equation(when L/y_a > 25)
- NCHRP 24-20 Abutment Scour Approach

NCHRP 24-20 Abutment Scour Approach is selected to estimate the scour in this design. The advantages of using the NCHRP abutment scour equations include ,

1. Not using the effective embankment length, L', which is difficult to determine in many situations.
2. The equations are more physically representative of the abutment scour process.
3. The equations predict total scour at the abutment rather than the abutment scour component that is then added to contraction scour.

$Y_{max} = \alpha_A Y_C$ (live- bed)---(a) $Y_{max} = \alpha_B Y_C$ (clear water)---(b)

$$Y_S = Y_{max} - Y_0$$

Y_{max} = Maximum flow depth resulting from abutment scour, ft (m)

Y_C = Flow depth including live-bed or clear-water contraction scour, ft (m)

α_A = Amplification factor for live-bed conditions

α_B = Amplification factor for clear-water conditions

Y_S = Abutment scour depth, ft (m)

Y_0 =Flow depth prior to scour, ft (m)

The scour calculations are given in appendix E. The rip rap was designed accordingly.

Sizing rock rip rap for abutment protection

Using Bridge Scour Manual, 2019 rip rap was designed for the abutments. Figure 5.42 shows the designed rock rip rap.

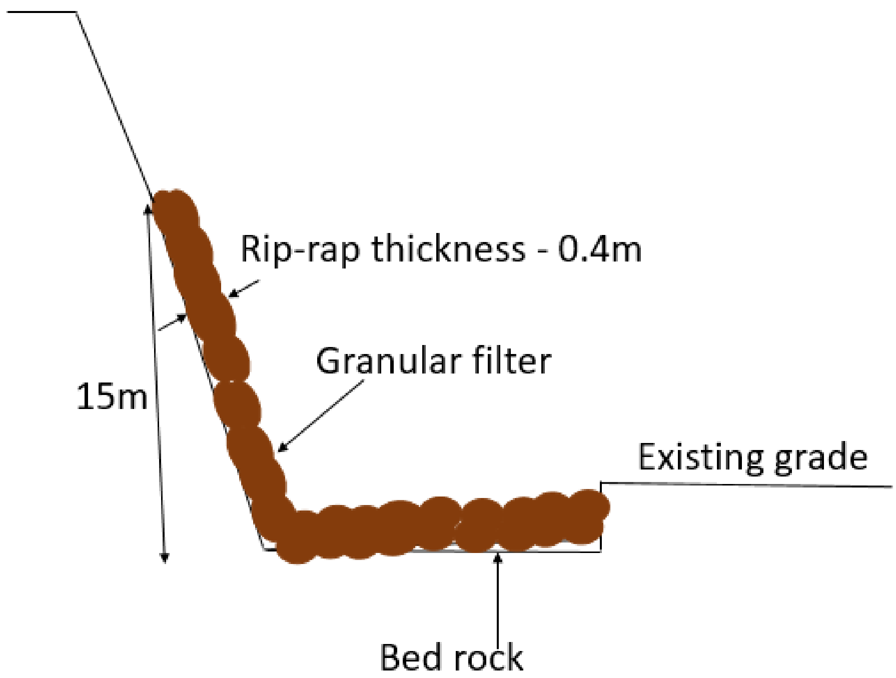

Figure 5.42 Rock rip rap

Thickness of rip rap	=	1.5 d_{50}
	=	0.42m

5.5 LONGITUDINAL PROFILE

All the designs were done following the longitudinal profile along the bridge. Figure 5.43 shows the longitudinal profile across the proposed bridge with heights of bridge components, proposed bridge elevation, soil layer and bed rock layer. Table 5.3 shows the distance to abutments and piers of the proposed bridge from the center line of Colombo – Gampola (A1 – A5) road. Thus, using that details and contour plan for the area the longitudinal profile with the proposed bridge was created as shown in figure 5.43.Then, using figure 5.43, required pier and abutment heights were obtained according to the soil profile. After doing pier and abutment design, required pile heights were identified.

Table 5.3 Locations of the key components of the bridge

Key Points with bridge deck Elevation (m)		Distance from centre of A1 – A5 Road (m)
left abutment	480	32
left pier	480	57
right pier	480	82
right abutment	480	107

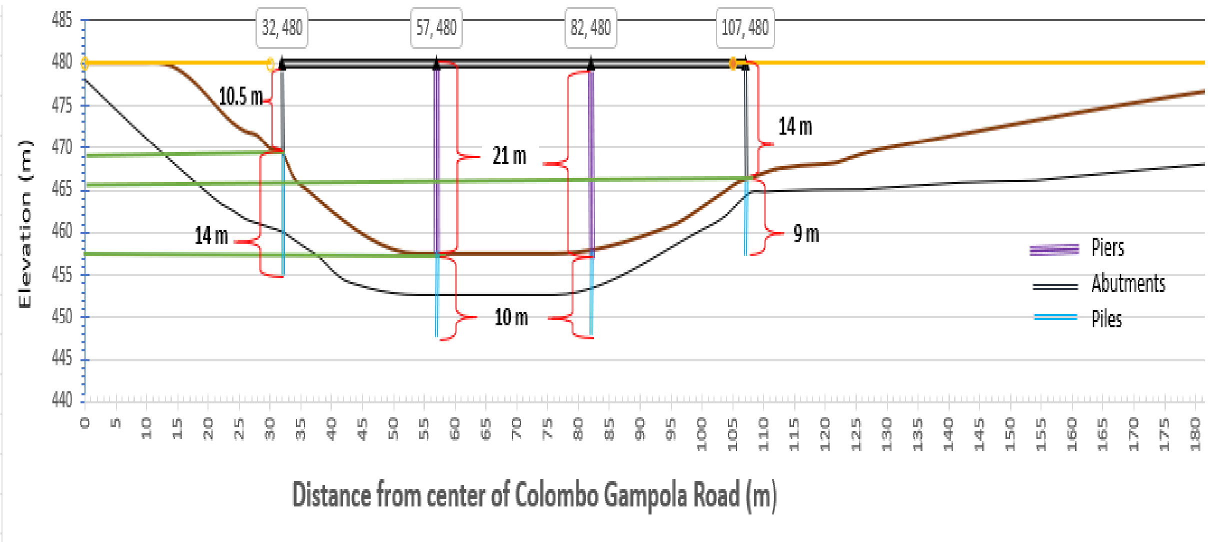

Figure 5.43 Longitudinal profile of the proposed bridge with heights of piers, piles and abutments

5.6 BEARING DESIGN

5.6.1 INTRODUCTION

Bridge bearings are used to transfer the vehicular and other environmentally imposed loads from the superstructure down to the substructure and finally to the ground. Service movements and extraordinary movements caused by extreme load cases are resisted by these bearings.

There are many types of bearings used in bridge constructions. They are steel reinforced elastomeric bearings, fabric pad sliding bearings, steel pin bearings, rocker bearings, roller bearings, steel pin bearings, pot bearings, disc bearings, spherical bearings, and seismic isolation bearings. Each type of bearings has different characteristics and understanding the characteristics is essential for economical bearing selection and design.

Steel reinforced elastomeric bearing type is proposed to this bridge construction. Because it is simplest and economical of all modern bridge bearings. Therefore, this bearings are commonly used with pre-stressed concrete girder bridge and may be used with other bridge types. There are four types steel elastomeric bearings. They are plain elastomeric pads, fiberglass reinforced elastomeric pads, steel reinforced elastomeric pads, and cotton duck reinforced elastomeric pads. Of these four types, steel reinforced elastomeric pads are used most extensively for bridge construction applications. Therefore steel reinforced elastomeric pads are proposed in this design. A typical bridge bearing is shown in figure 5.44.

Figure 5.44 A typical bridge bearing

5.6.2 DESIGN METHODS

Two design methods are allowed by the AASHTO – LRFD specifications. They are Method A and Method B. Method A, is applicable to plain, steel reinforced and fiber glass reinforced elastomeric pads as well as cotton duck pads. Method B, specified is applicable to steel reinforced elastomeric bearings. Method B is used in this design. The final bearing size is given below.

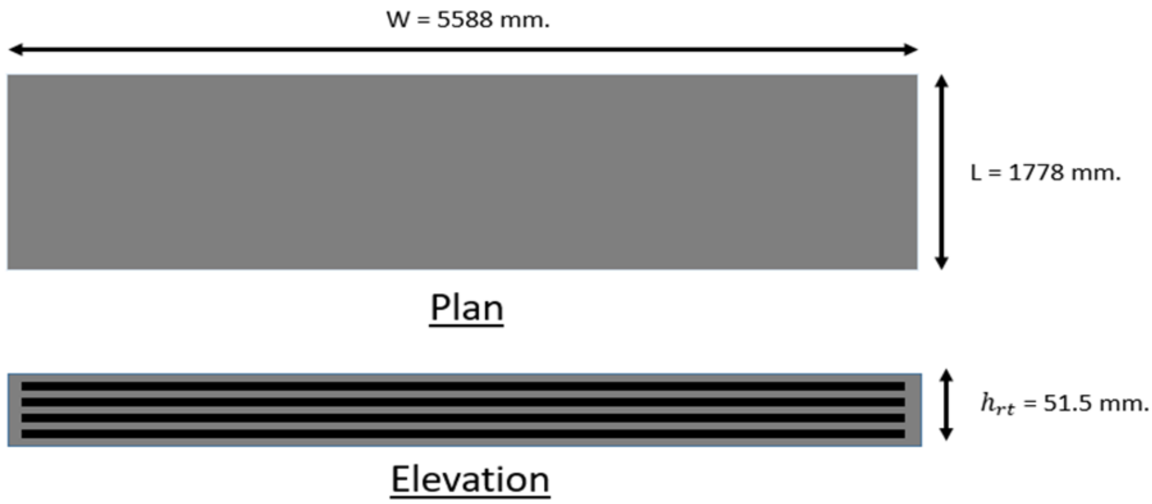

Figure 5.45 Dimensions of design bearing

5.7 EXPANSION JOINTS

Expansion joint systems are integral, yet often overlooked, components designed to accommodate cyclic movements. Properly functioning bridge expansion joint systems accommodate these movements without imposing significant secondary stresses on the superstructure.

5.7.1 DESIGN OF EXPANSION JOINTS

Expansion joints must accommodate movements produced by concrete shrinkage and creep, post-tensioning shortening, thermal variations, dead and live loads, wind and seismic loads, and structure settlements. Concrete shrinkage, post-tensioning shortening, and thermal variations are generally taken into account explicitly in design calculations. Because of uncertainties in predicting, and the increased costs associated with accommodating large displacements, seismic movements are usually not explicitly included in calculations.

Expansion joints should be designed to accommodate all shrinkage occurring after their installation. For unrestrained concrete, ultimate shrinkage strain after installation, β, may be estimated as 0.0002. More-detailed estimations can be used which include the effect of ambient relative humidity and volume-to-surface ratios (AASHTO 2012),. Shrinkage shortening of the bridge deck, Δshrink, in mm, is calculated as

$$\Delta shrink = (\beta)\cdot(\mu)\cdot(L_{trib})\cdot(1000\ mm/m)$$

Where,

L_{trib} =tributary length of structure subject to shrinkage; m

β = ultimate shrinkage strain after expansion joint installation; estimated as 0.0002 in lieu of more-refined calculations

μ = factor accounting for restraining effect imposed by structural elements installed before slab is cast = 0.0 for steel girders, 0.5 for precast Prestressed concrete girders, 0.8 for concrete box girders and T-beams, 1.0 for flat slab

Shrinkage: Δshrink = (0.0002 m/m) (0.5) (½) (75m)(1000 mm/m)

= 3.75 mm

Thermal displacements were calculated using the maximum and minimum anticipated bridge deck temperatures. These extreme values are functions of the geographic location of the structure and the bridge type(AASHTO 2012),. Thermal movement, in mm, is calculated as,

$$\Delta temp = (\alpha)\cdot(Ltrib)\cdot(\delta T)\cdot(1000\ mm/m)$$

Where,

α = coefficient of thermal expansion; 0.000011 m/m/°C for concrete and 0.000012 m/m/°C for steel

L_{trib} = tributary length of structure subject to thermal variation; m

δT = temperature variation; °C

In Kandy the temperature is expected to range between 18^0C and 35^0C during the life span of the structure. (National Centers for Environmental Information)

Temperature: $\Delta temp = (0.000011\ m/m/°C)\ (½)\ (75\ m)\ (17°C)\ (1000\ mm/m)$

$= \underline{\underline{7.02\ mm}}$

Therefore, total deck movement at the joint: $\underline{\underline{10.76mm}}$

Asphaltic plug joints are provided as expansion joints for the superstructure.

5.7.2 ASPHALTIC PLUG JOINTS

Asphaltic plug joint (also referred to as TST bridge joint) for movements up to 50 mm is a simple bridge expansion joint filled with asphalt. TST (crushed stone) elastic material is a special type of elastic-plastic material with high viscosity. Being heated sufficiently to melt, it can be poured into gravel and will be shaped into asphalt concrete form after molding to bear the vehicle load.

Asphaltic plug joints comprise liquid polymer binder and graded aggregates compacted in preformed block outs. The compacted composite material is referred to as polymer modified asphalt (PMA). These joints have been used to accommodate movement ranges up to 50 mm. This expansion joint system was developed in Europe and can be adapted for use with concrete or asphalt bridge deck surfaces. The PMA is installed continuously within a block out centered over the expansion joint opening with the top of the PMA flush with the roadway surface. A steel plate retains the PMA at the bottom of the block out during installation. The polymer binder material is generally installed in heated form. Aggregate gradation, binder properties, and construction quality are critical to asphaltic plug joint performance.

The asphaltic plug joint is designed to provide a smooth, seamless roadway surface. It is relatively easy to repair, is not as susceptible to snowplow damage as other expansion joint systems, and can be cold-milled and/or built up for roadway resurfacing. The material properties of PMA vary with temperature. Asphaltic plug joints have demonstrated a proclivity to soften and creep at warmer temperatures, exhibiting wheel rutting and eventual migration of PMA out of the block outs. Figure 5.46 shows a cross section view of an asphaltic plug joint and table 5.4 shows the specification table of TST bridge joints.

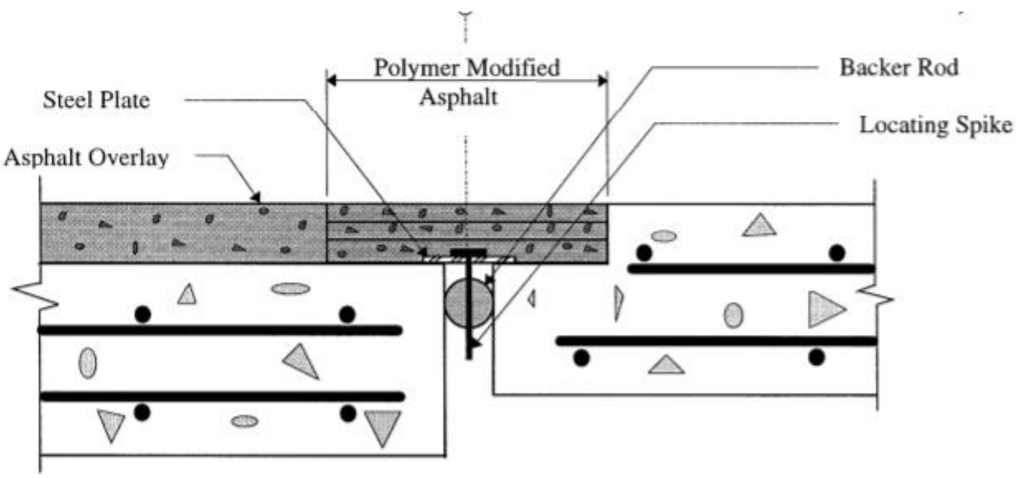

Figure 5.46 Asphaltic plug joint (cross section)

Table 5.4 Specification of TST Bridge Joints

Stretching (mm)	Slot Width (mm)	Slot Depth (mm)	Beam end clearance (mm)
10	80–100	≥ 20	15
20	160–200	≥ 50	20
30	240–300	≥ 75	25
40	320–400	≥ 100	30
50	400-500	≥ 120	35

Features

- High elasticity: TST bridge joints can adapt the load deformation and vehicle load well.
- Good performance: TST bridge joints have good low-temperature flexibility and high-temperature stability.
- Easy to construct: TST bridge joints can be installed easily and conveniently without blocking traffic.
- Open to traffic quickly: Bridge can be opened to traffic in two hours after TST bridge joints are installed. If cooling is accelerated, the bridge can be open to traffic in an hour.
- Shock-absorbing: TST bridge joints can absorb vibration of vehicle impact and make cars go smoothly.
- Long service life: Being strict accordance with the requirements of the production and installation of expansion joints, TST bridge joints generally have a longer service life.
- Low cost & high cost-effective.

Figure 5.47 Asphaltic plug joint used in connection of bridges

Figure 5.48 Asphaltic plug joint is used in end of bridge

Installation

Asphaltic plug joint can be installed with adhesive in the connection of bridges easily. And it can condense quickly and will not affect the normal traffic.

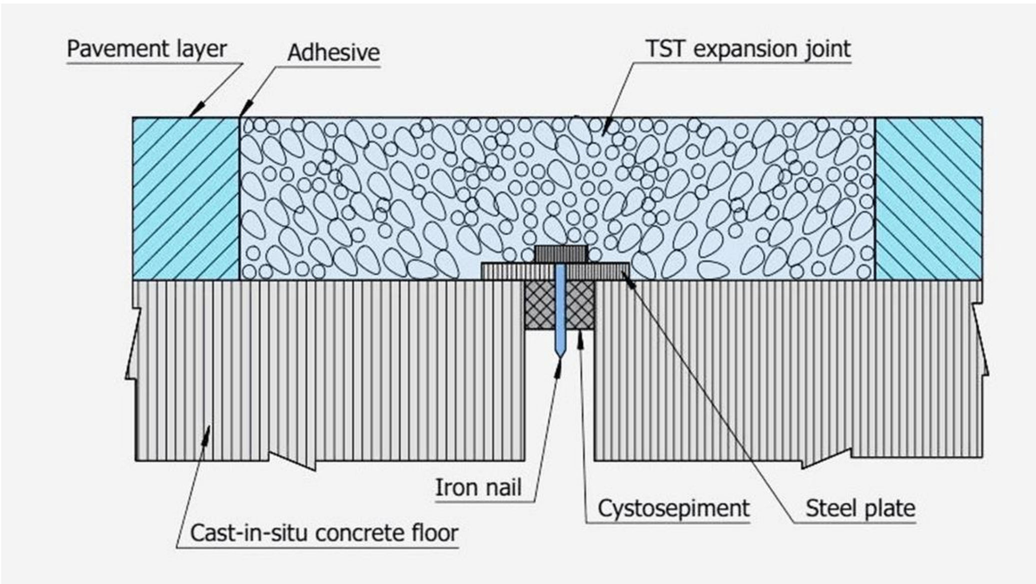

Figure 5.49 Installation of TST bridge joint

5.8 LAYERS ON THE BRIDGE DECK

Bridge deck pavements must satisfy a large number of requirements, such as: Resistance to permanent deformation, texture depth, skid resistance, rigidity, evenness, ageing resistance, etc. It must also protect and seal the underlying supporting structure as this determines to a large extent the life of the structure under the heavy load of traffic and weather conditions. They must absorb traffic loads, transfer them to the supporting structures and remain even and resistant to deformation and provide good anti-skid conditions for vehicles. Besides, they must protect the bridge structure from surface water which promotes corrosion. Because of the different requirements for the pavement structure on a bridge deck, these functions are generally not fulfilled or only partially fulfilled by one material, a functional division can be made for the layers constructing the surfacing of the deck, often called a “system”, consisting of several layers. In general, the asphalt bridge pavement system can be split into four different layers: a sealing/bonding layer (primer), a waterproofing layer, a protecting layer and surface layer (asphalt).

5.8.1 SEALING LAYER

Since asphalt layers cannot directly be bonded on a concrete or steel base, and nor is it 100% watertight, an intermediate sealing layer is necessary to establish a good bond to the waterproofing layer. On concrete bridges it also closes the voids in the concrete, thus minimizing the risk of formation of blisters or bulges between the concrete and the waterproofing sheets.

The most commonly used sealing method is applying a sealing layer with a kind of bituminous material as bitumen emulsion, PMB, epoxy resin, polyurethane, etc. Before applying the sealing layer, the bridge deck surface has to be prepared properly to be clean, dry, sound, and free of all bond-inhibiting substances. The concrete surface should have sufficient gradient in the longitudinal direction to ensure that drainage can occur in the drainage layer built-in and on the surface of the asphalt pavement. When the concrete has cured sufficiently, the concrete surface is shot or sandblasted in order to remove excess cement laitance and to create a surface texture that will ensure good bonding when the waterproofing is executed. After the shot or sandblasting, sealing of the concrete surface with a sealing layer is applied on the clean concrete which must be surface-dry.

5.8.2 SURFACE / ASPHALT LAYER

Good skid resistance, flat surface and low sound levels are needed for surface layer for a safe and comfortable drive. To ensure durability of the required characteristics of the surface layer, the surface layer needs to have:

- sufficient resistance against deterioration
- resistance against oil, water and minerals
- less susceptibility to weather conditions
- protection of the deck plate and the waterproofing layer
- high stability
- resistance to fatigue
- resistant to permanent deformation
- possibility to spread the loads

To assure sufficiently strong adhesion a tack coat is required. This tack coat has to provide the required strong adhesion. There are generally three types of tack coat layers, distinguished on the basis of bitumen (hot fluid bitumen), bitumen emulsion (cold fluid bitumen) and artificial resins. The resin tack coat layers consist of cold hardening epoxy resins scattered with grit. The surface layer is made of asphalt. Generally the asphalt mixture types used on the bridges are Dense Asphalt Concrete, Mastic Asphalt and Stone Mastic Asphalt (SMA).

In this design, following construction type is used.

- special priming: Primer layer; Solvent-less primer based on an epoxy resin system gritted with 300 - 500 g/m² quartz sand
- liquid plastic sealing thickness: 4 mm
- mastic asphalt protection layer (ZTV-ING Partl 7)
- Surface layer (TL Asphalt-StB ;EN 13108 for MA, SMA, AC or PA) An Asphalt pavement of 50mm thick and a cross fall of -2.5% is used.

5.9 BRIDGE DECK DRAINAGE SYSTEM

The bridge deck drainage system includes the bridge deck itself, bridge gutters, inlets, pipes, downspouts, and bridge end collectors

The bridge deck and gutters are surfaces that initially receive precipitation and debris. If grades, super-elevations, and cross-slopes are properly designed, water and debris are efficiently conveyed to the inlets or bridge end collectors.

From the deck and gutters, water and debris flow to the inlets, through pipes and downspouts, and to the outfall. Various grate and inlet box designs are available to discourage clogging. Collector pipes and downspouts with a minimum of T-connections and bends help prevent clogging mid-system. Collector pipes need sufficient slope to sustain self-cleansing velocities. Open chutes are not recommended for down drains because of difficulties in maintaining chutes and capturing, and then containing the flow.

Drainage collection devices placed at the ends of bridges are essential and have two basic purposes. First, they control the amount of upslope drainage that can run onto the bridge deck. Second, they intercept runoff from the bridge deck at the downslope end. An inlet should be provided just off the upslope end of the bridge in each gutter to intercept the drainage before it gets onto the deck. Collectors at the downslope end catch flow should not be intercepted by bridge inlets. If there are no bridge inlets, downslope inlets intercept most of the bridge drainage.

The existing bridge has an open deck drainage system with vertical penetration through the bridge as seen in figure 5.50. Thus, a similar design is proposed for the new bridge.

Figure 5.50 Drainage facility in the existing bridge

In this design open deck drainage with vertical penetration through the bridge deck is proposed as shown in figure 5.51. Since the river is below the bridge after a proper preliminary treatment the water can be discharged to the river.

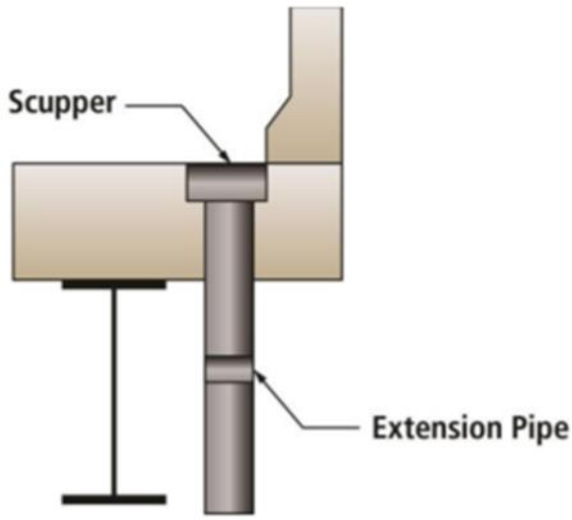

Figure 5.51 Schematic of an open deck drainage system with vertical penetration

5.9.1 DECK DRAINAGE DESIGN

Deck width between rails	= 9m/29.6ft (Right shoulder @1.5m/5ft, 2 lanes@ 3.5m/11.5ft, Left shoulder @ 0.5m /1.6 ft.).
Structure length	= 75m /246ft- between expansion joints.
Cross slope @ 2.5 %	= 0.025
Profile gradient is constant @ 1%	= 0.01
Pavement	= Asphalt, Manning's coefficient, n= 0.016

A gutter is defined, as the section of bridge deck next to the barrier which conveys water during a storm runoff event. It may include a portion or all of the shoulder. Gutter cross sections usually have a triangular shape with the barrier forming the near-vertical leg of the triangle. The gutter may have a straight cross slope or a cross slope composed of two straight lines.

Modification of the Manning equation is necessary to use in computing flow in triangular channels because the hydraulic radius in the equation does not adequately describe the gutter cross section, particularly where the top width of the water surface may be more than 40 times the depth at the curb. To compute gutter flow, the Manning equation is integrated for an increment of width across the section. (CALTRANS Manuals: Engineering Services - Bridge Manuals)

The resulting equation in terms of cross slope and spread on the pavement is:

$$Q = (K/n)\, S_X^{5/3}\, S^{1/2}\, T^{8/3}$$

Where,

$K = 0.56$;

Q = flow rate, ft^3/s;

T = width of flow (spread) ft;

S_x = cross slope, ft/ft;

S = longitudinal slope, ft/ft;

n = Manning's coefficient

For $n = 0.016$, $T = 3.28$ ft, $Sx = 0.025$ ft/ft, $S = 0.01$ft/ft

$$Q = (K/n)\, S_X^{5/3}\, S^{1/2}\, T^{8/3}$$

$$= (0.56/0.016)\,(0.025)^{5/3}\,(0.01)^{1/2}\,(3.28)^{8/3}$$

$$= 0.1776 ft^3/s\ (0.005\ m^3/s)$$

To check the capacity of the drain outlet,

The capacity of inlets can also be controlled by the orifice capacity of the drain outlet pipe. Following equation was used with, *d* equal to the depth of water above the center of the outlet pipe and *A* equal to the area of the pipe opening.

$Qi = 0.67\ A\ (2gd)^{0.5} = 5.37\ Ad^{0.5}$

Where,

Qi = rate of discharge into the grate opening, in cubic feet per second;

A = area of the pipe opening;

g = acceleration of gravity, 32.2 feet per second^2;

d = the depth of water above the center of the outlet pipe

Drain Properties

D-2	d= 0.522 + y feet	A= 0.129 ft^2
D-3	d= 0.5 + y feet	A= 0.194 ft^2
D-1	d= 0.961 + y feet	A= 0.194 ft^2

where y = flow depth at curb

D3 type was selected.

d = 0.5+ 0.1

= 0.6 ft

Q = 5.37 (0.194) (0.6) $^{0.5}$

= 0.8069 ft3 /s > 0.1776,

Therefore drain pipe can handle flow

Since, open deck drains were used in this design, the water directly discharged to the river. An 8 inch diameter pipes were used.

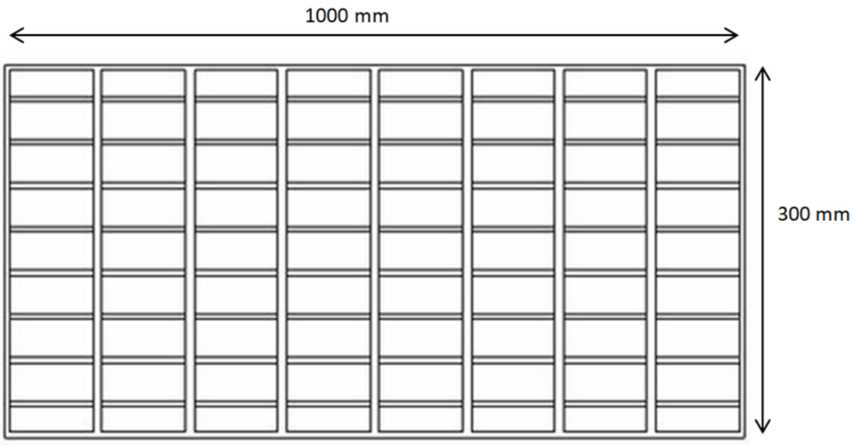

Figure 5.52 Dimensions of the grate inlet

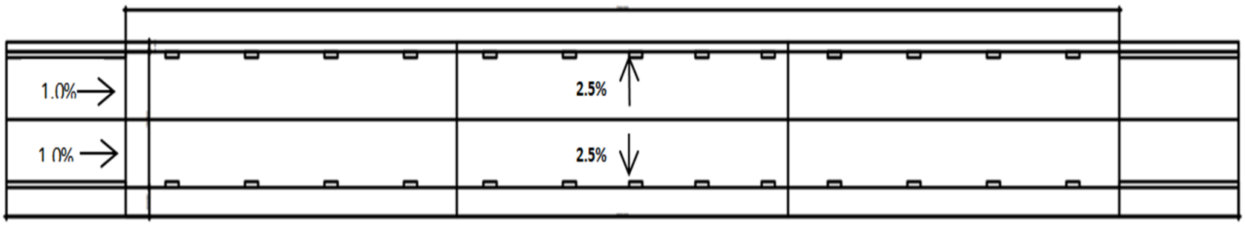

Figure 5.53 Inlet locations

To drain the water in the abutments Envriobridge curb drainages are used, the drain outlet is connected to bridge drainage grate to remove surface water efficiently. The elements are in accordance with EN1433 class D400. The units are equipped with drainage holes 3 per half meter element as per figure 5.54.

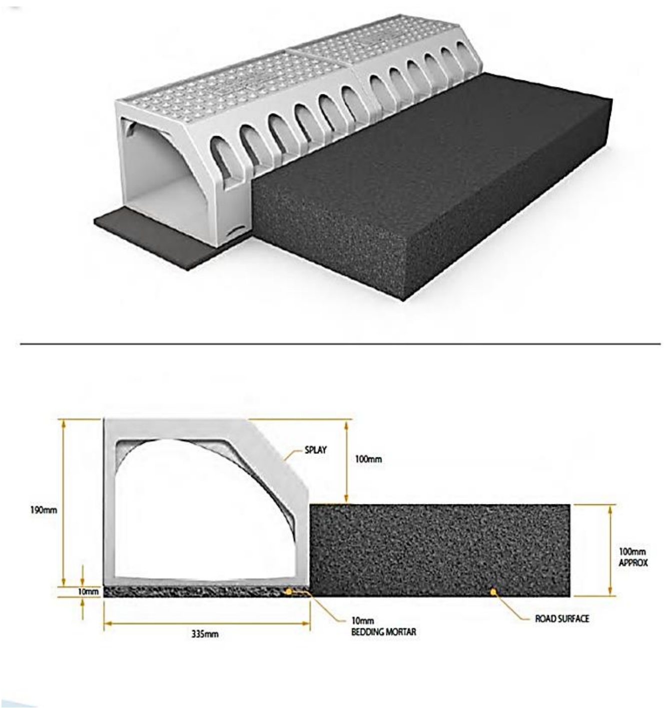

Figure 5.54 Envirobridge deck drainage curbs

5.9.2 CATCH BASIN INSERTS

Catch Basin Inserts (Drain Guards / Sediment Traps) protect our rivers and streams by capturing sediment, debris, oil and grease at storm water catch basins. Catch Basin Inserts are an economical and effective method to protect from costly clean-up work. The standard filter material is a non-woven geotextile with built-in overflow ports for cases of abnormally high water flow or over-filled filter bags. Catch Basin Inserts are available with a replaceable oil absorbent boom that floats to absorb any oil, gas or diesel entering a storm water catch basin. In this design catch basing inserts are provided to trap sediment to improve the quality of the discharged water as seen in figure 5.55.

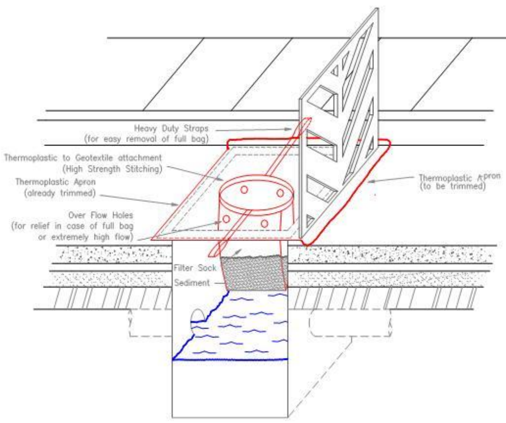

Figure 5.55 A catch basing insert

5.10 LAMP POST DESIGN

A lamp post is a raised source of light on the edge of a road or path. A proper lighting design is required to prevent of accidents and increase the safety on the bridge. Figure 5.56 shows the lamp post arrangement used in this design.

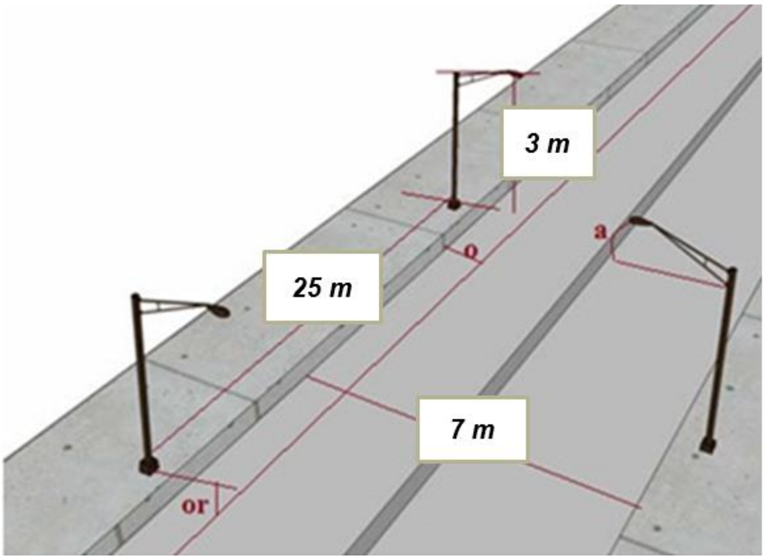

Figure 5.56 Lamp posts locations

Luminaries are properly selected and mounted on a location most feasible and effective with minimum cost. For a 230 volts system, a voltage drop of 5% is allowed although in extreme cases 15% voltage drop is sometimes tolerated.

Street illumination level in Lux,

E= (Al x (cu x mf)) / (w x d)

Where,

E = The illumination in Lux

w = Width of the roadway

d = Distance between luminaries

cu = Coefficient of utilization. Which is dependent on the type of fixture, mounting height, width of roadway and the length of mast arm of outreach.

Al = Average lumens,

Al = (E x w x d) / Cu x mf

The typical value of Al is

20500 lumens for 400 watts, 11500 lumens for 250 watts, 5400 lumens for 125 watts.

The value of Al varies depending upon the type of lamp specified.

mf : It is the maintenance factor (Normally 0.8 to 0.9)

5.10.1 LAMP WATT FOR STREET LIGHT POLE CALCULATION

Road width = 9 m,

Distance between two Pole = 25 m,

Maintenance factor = 0.9,

Coefficient of utilization factor =0.29,

Recommended of illumination (E) in Lux is 6.46 per sq. meter.

w = 9 m, d = 25 m, mf = 0.9, cu = 0.32

To decide Lamp Watt It is necessary to calculate Average Lumens of Lamp (Al).

Al = (6.46x9x25) / (0.32×0.9)

= 5056.875 Average lumen

Lamp lumen of a 125 watts lamp is 5400 lm which is the nearest value to 5057 lumen. Therefore, a 125 watts lamps are acceptable. The actual illumination E for 125 Watt Lamp,

E = (5400×0.29×0.9) / (9×25)

= 6.912 lumen / m^2.

Actual illumination (E) for 125 Watt is 6.91 lumen per sq meter which is higher than recommended illumination (E) 6.46.

5.11 HAND RAILS

Railings are provided along the edges of a bridge to protect vehicles, bicyclists, and pedestrians. Based on functionalities, bridge railings may be classified as: pedestrian railings, bicycle railings, traffic railings, and combination railings. Bicycles and pedestrians travel at low speeds. The impact loads are small and crash testing is not required for railings protecting them. In this design pedestrian railings were used as shown in Figure 5.57 (AASHTO. 2012. AASHTO LRFD Bridge Design Specifications, Customary U.S. Units 2012, American Association of State Highway and Transportation Officials, Washington, DC.)

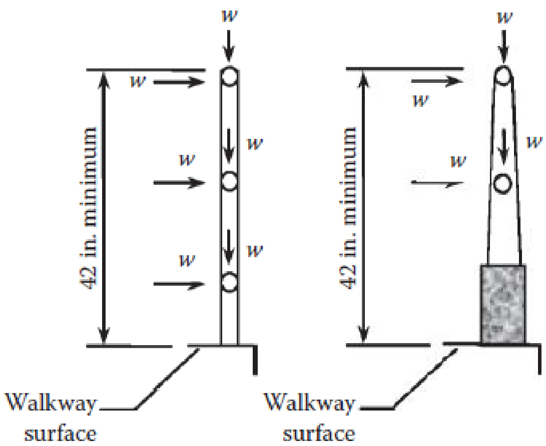

Figure 5.57 Pedestrian railings and loads.

5.11.1 DESIGN CONSIDERATION OF PEDESTRIAN RAILING TYPE PR3

A 2-ft-wide concrete posts spaced a maximum of 12 ft. apart. Between the concrete posts there are two steel pipe rails centered 42 inches and 13.5 inches from the sidewalk surface with vertical steel pickets connected to the steel pipe rails. A 6-inch-tall concrete curb is placed between the concrete posts. The PR3 railing is designed for pedestrian loads only. It has not been crash-tested, and it is not intended for exposure to traffic. If this railing is used on a bridge or culvert, it must be protected from vehicular impact by an approved bridge rail type (Bridge railing manual, Texas Department of Transportation, Septemner, 2019). PR3 Railing dimensions are given in table 5.5 and a sketch is shown in figure 5.58

Table 5.5 PR3 Railing dimensions

Nominal height	43.75 inches / 1.11 m
Minimum height after maintenance overlays	42 inches / 1m
Special notes	The PR3-HD is to be used with this railing when an ADA-compliant handrail is needed.

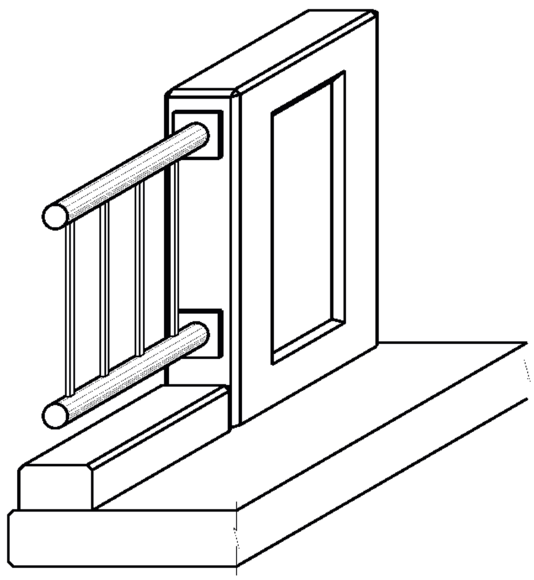

Figure 5.58 Sketch of PR3 railing

5.12 APPROACH ROAD DESIGN

An approach road was designed to divert the traffic to the new bridge from the existing A1 road.

5.12.1 PAVEMENT DESIGN

Traffic:

As per the IRC: 37 design traffic should be 0.1 to 2 msa (million standard axles). Weight of commercial vehicle (laden) is considered as 3 tons or more. For this design traffic the existing traffic and rate of growth was considered.

Design life: The number of years to be taken until the major reconstruction.

Design life depend upon the environmental conditions, materials used, maintenance etc. For rural roads design life of 10 years is considered. In low volume roads for the thin bituminous surfacing design life of 5 years is considered.

Computation of design traffic:

Pavement components

$$a = p\,(1 + r) \,\hat{}\, (n + x)$$

Where,

- a = no. of commercial vehicles/day for design
- p = no. of commercial vehicles/day at last count
- r = annual growth rate of commercial traffic
- n = no. of years between last count and year of completion of construction
- x = design life in years

Pavement components (RDA Highway Design manual)

- **Subgrade:**

- To provide support to the pavement as its foundation.
- Top 30 cm of the cutting or embankment at formation level in rural roads consider as subgrade.
- A minimum of 100% of standard proctor compaction should be attain in subgrade.
- For clayey soil 95% and moisture content of 2% in excess of optimum value.

- Soil below subgrade should be compacted to 97% of standard proctor compaction.

- **CBR:**

- Conduct on a sample which was remolded at OMC and dry density.
- Test should be done per km depending on the soil type.
- If CBR less than 2% for 100 mm thickness, then minimum CBR of 10% is to be provided to the sub-base for CBR of 2%.
- If CBR more than 15%, no need to provide sub-base.

- **Sub-base course:**

Selected materials placed on subgrade which is compacted to 98% of IS Heavy Compaction. Function of the sub-base is to distribute the stresses over a wide area of the subgrade imposed by the traffic. Materials used for the sub base course are,

- CBR of 15%
- Passing through 425 micron IS sieve
- L.L<25 and P.I<6

Waste materials such as Fly ash, Iron and steel slag Recycled concrete Municipal waste are also used. When subgrade is silty or clayey soil and an annual rainfall of area is more than 1000 mm, a drainage layer of 100 mm and formation width is provided.

- **Base course:**

To with stand high stress concentrations which develop due to traffic under the wearing surface. Different types of base course used are:

1. WBM
2. Crusher-run macadam
3. Dry lean concrete
4. Soft aggregate base course
5. Lime-fly ash concrete

➢ **Surface course:**

Thickness of the surface course depend upon the traffic volume and type of material used for it. For gravel roads extra thickness should be provided because of the lost in thickness due to the traffic action. Bituminous wearing courses must be made up of good quality aggregate with aggregate impact value not exceeding 30 % to reduce degradation of aggregates by crushing.

First, the total ESA value was calculated from the available traffic survey data at Peradeniya Junction. The detailed design is given in the Appendix G.

Total ESA cumulative is between 6.0 -10.0. The traffic class was taken as T6. Subgrade strength class was taken as S3 (5-7) as it is Organic silts, cross section of pavement was found. Figure 5.59 shows the cross section of the pavement and selected thicknesses of the materials are given in table 5.11.

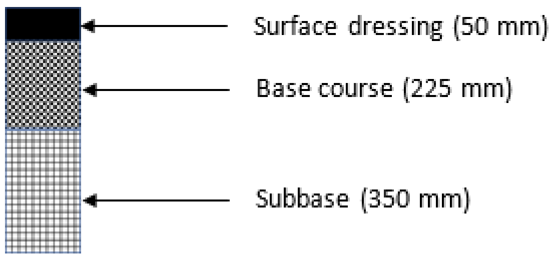

Figure 5.59 Pavement layer thickness

Table 5.6 Selected thickness and the materials

Layers	Material	Thickness (mm)
Surface dressing	Asphalt (AC14)	50
Base Course	Aggregate Base Course (ABC)	225
Subbase	Approved Gravel	350

5.12.2 GEOMETRIC DESIGN

Austroads 2016 edition and Overseas Road Note 6 were used as guidelines for the geometric designs of roads.

Shoulders

Shoulders are provided along the side of the road in order to provide lateral support to the pavement. It is also to serve as an emergency lane for vehicle and it act as a service lane for vehicles that have broken down. RDA road design guidelines recommended the minimum shoulder width is 2.5 m and it should have sufficient load bearing capacity even in wet weather. The surface of the shoulder should be rougher than the traffic lanes so that vehicles are discouraged to use the shoulder as a regular traffic.

The used shoulders for the roads – 0.85 m and made it as a treated shoulder.

Camber (Cross fall)

Camber is defined as the cross slope provided to raise the middle of the road surface in the transverse direction which is expressed in percentage in terms of elevation difference between the central crowns to the pavement edge. Main functions of this cross fall are drain off surface water quickly and segregate traffic lanes in two directions. RDA design manuals recommended for 2-3 % of cross fall for an asphalt pavement. Selected cross fall is 2.5%.

Kerb

The boundaries between pavement and shoulders or footpath are known as kerbs. It prevents encroachment of slow speed or parking vehicles to the footpath. But at emergency vehicle can climb over and parked on footpath or shoulder. Its height is **15 cm**.

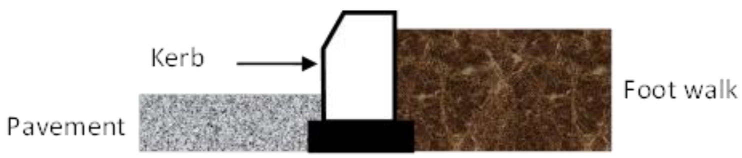

Figure 5.60 Details road side walk

Design speed was selected as 60km/h. Therefore according to the RDA highway specification stopping sight distance is 85m.

Horizontal alignment (Simple curve)

Figure 5.61 shows the selection criteria for a simple curve.

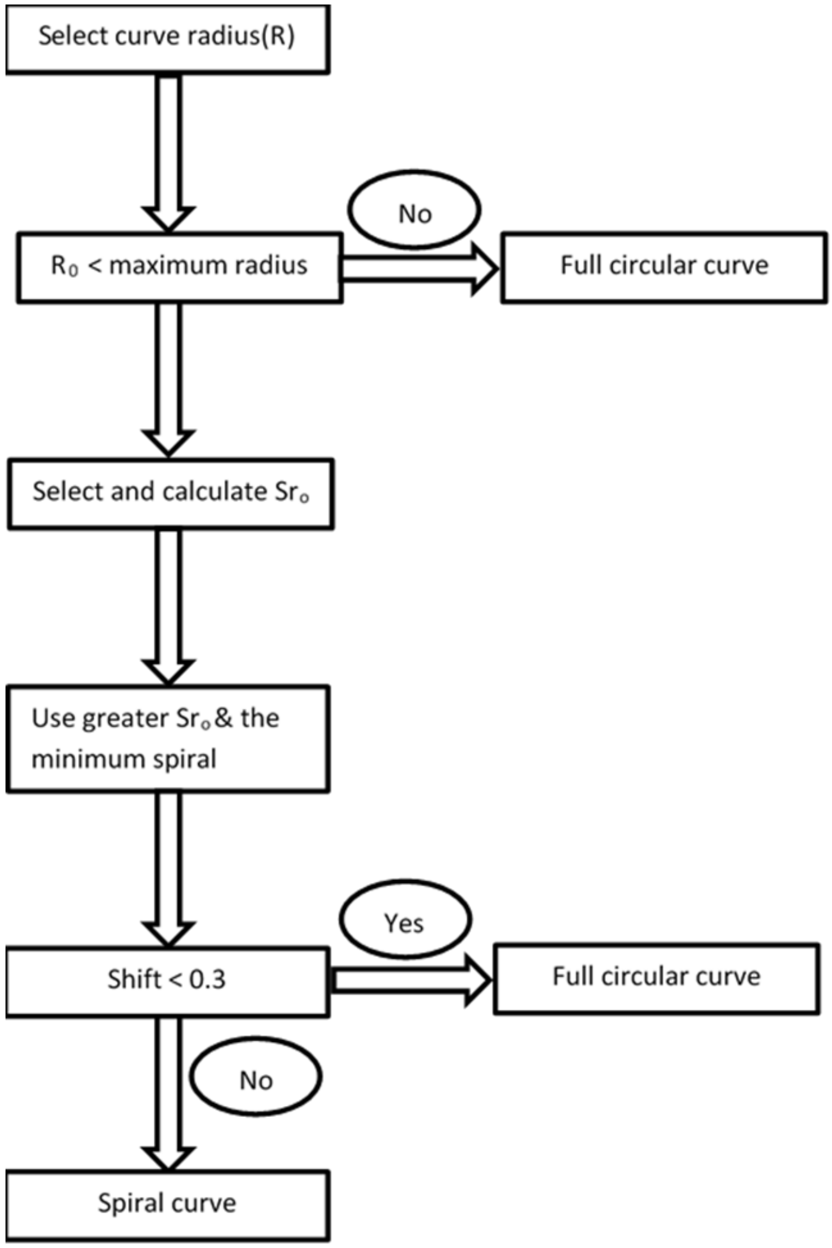

Figure 5.61 Simple curve flow chart

Superelevation

The superelevation is used in horizontal curves of a road for the safety, driver comfort and appearance. In this design the horizontal curve part has considerable radius superelevation design was incorporated,

Minimum superelevation

For drainage consideration it may be necessary to specify a minimum value for superelevation. Therefore it is consider that the minimum superelevation is equal to normal cross fall (-2.5%) even for larger radii that smaller superelevation is sufficient.

Length of superelevation Development (L_e)

The length of superelevation development is the transition of cross fall from a normal roadway on straight alignment to that of a fully superelevated crossfall on a circular curve.

L_e = Tangent Runout length (T_{ro}) +Superelevation runoff length (S_{ro})

Tangent Runout length (T_{ro})

The length of roadway requires to accomplish the change in crossfall from normal crown section to a flat crossfall.

$$T_{ro} = L_e - S_{ro}$$

Superelevation runoff length (S_{ro})

The length of roadway needed to accomplish the change in crossfall from flat crossfall to a fully superelevated crossfall

$$S_{ro} = L_e - L_e \left[\frac{n}{n+e}\right]$$

n = normal crossfall

e = full superelevation crossfall

There are two methods to calculate the length of superelevation development

- Relative gradient method
- Rate of pavement method

Relative gradient method

$$L_e = \frac{W(e+n)}{G_r}$$

Where

L_e = Length of superelevation Development (m)

W = Lane width (m)

e = Full superelevation

n = normal crossfall

G_r = Relative Gradient

Rate of pavement method

$$L_e = \frac{(e+n)*V}{3.6\,\beta}$$

Where

L_e = Length of superelevation Development (m)

e = Full superelevation

n = normal crossfall

β = Rate of pavement Rotation (rad/s)

V = Design speed (km/h)

Rate of pavement rotation is 0.035 rad/s for design speeds less than 80 km/h and 0.025 rad/s for design speed greater or equal to 80 km/h

Select a superelevation development length that satisfy both rate of rotation and relative grade cirteria

A 160 m radius was selected for the curve. Then, the Super elevation development length for the horizontal curve was calculated using the relative gradient method and rate of pavement rotation methods. The critical super elevation development length was calculated to be 29.94m from the Relative gradient method. Tangent runout length and the portion of runoff within the curve were also calculated. The super elevation curve is shown in figure 5.62.

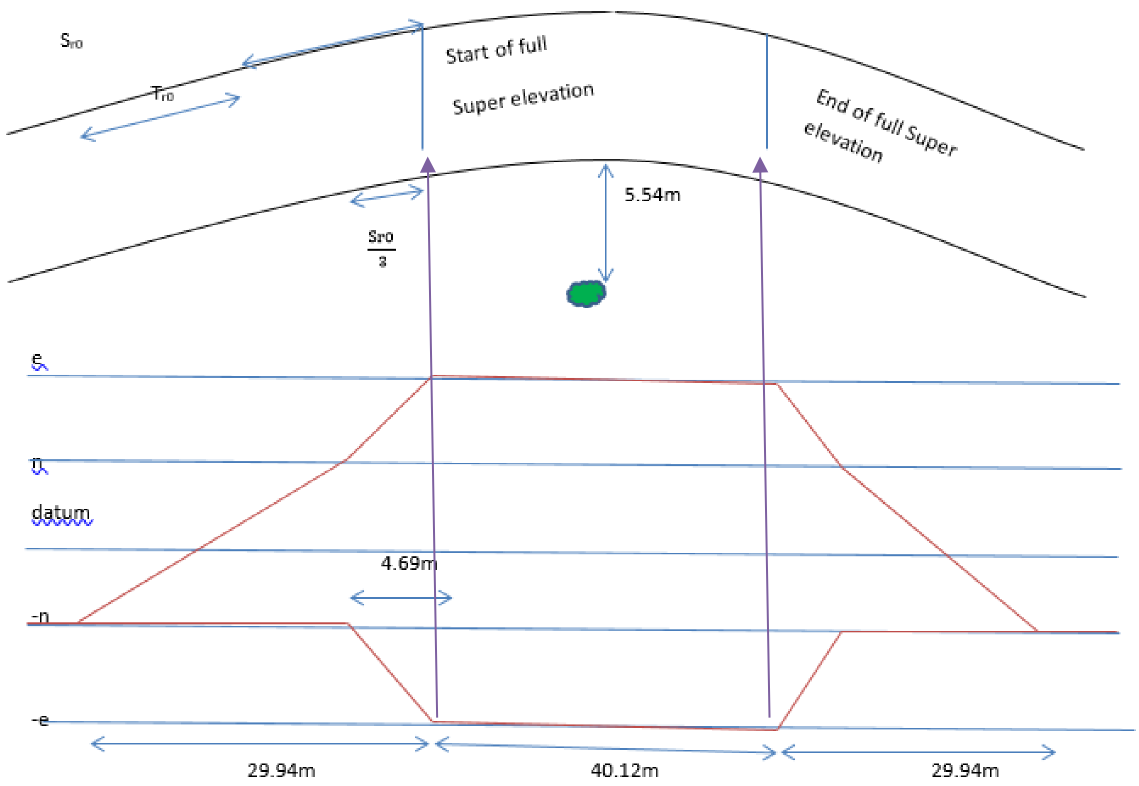

Figure 5.62 Super elevation detailing of approach road

Autodesk Civil 3D software was used to design the road. Following work flow was followed in designing the road.

- Create base map— Road design was started by creating an existing conditions surface and compiling a base map of existing conditions; information about the topography, parcels, utilities, and other potential impacts to the route design.
- Design alignment— an alignment defines the main horizontal route that typically represents the construction baseline of the roadway. Alignments were created using from existing CAD entities.
- Apply design criteria—the design intent and the constraints was determined that are to be placed on the alignment. This includes speed and superelevation parameters. Design criteria may be assigned at the onset of the alignment layout or at any time during the design process.
- Generate existing ground profile and design grades—existing ground surface data was inserted for the design alignment and create the finished grades. Finished grade profiles was created graphically using profile creation tools, or generated from a best fit analysis of existing entities or from information from an external file.
- Construct assemblies—Assemblies define the cross-sectional component of the design and was built by connecting individual subassembly objects, thereby helping to simulate the geometry and material makeup of the road as well as helping to define how it interacts with surrounding features along the route. The subassemblies were selected from the prebuilt libraries contained in the civil 3D Tool Palette
- Build the corridor—Corridors are the resulting dynamic 3D model representation built from the combination of horizontal, vertical and cross-sectional design elements.
- Analyze resulting model—Corridors may be used to calculate earthworks and quantity takeoffs, to perform sight and visual analysis, to generate surfaces, and to extract information for construction purposes.

5.12.3 THE CORRIDOR MODEL

Corridors combine surface, alignment, profile, and assembly information to create dynamic three-dimensional representations of route-type features, such as roads, railroads, channels, and bridges.

Corridors are the main design object of road modeling and simulation in Civil 3D. They rely on interaction with other model objects and they help to simulate behavior prescribed by assignable and customizable parameters, such as daylighting, lane widening, and superelevation schemes.

The corridor was created by applying an assembly along the horizontal and vertical path defined by the combined information of the alignment and profile. To complete the corridor, targets are specified to achieve daylighting.

The type of corridor was determined by the assembly configuration that was applied along the baseline at desired intervals. The assemblies that were used to create the corridor contained sophisticated behavior such as conditional targeting, widening, and superelevation.

The result is a 3D model that extrudes the specified assemblies along the desired path. Feature lines connect similar points from assembly to assembly. These feature lines establish the longitudinal edges of the 3D model. Individual points in the assembly may also be assigned behavior that automatically follows prescribed targets, such as curb return alignments or lane-widening feature lines.

The resulting corridor model is shown in figure 5.63.

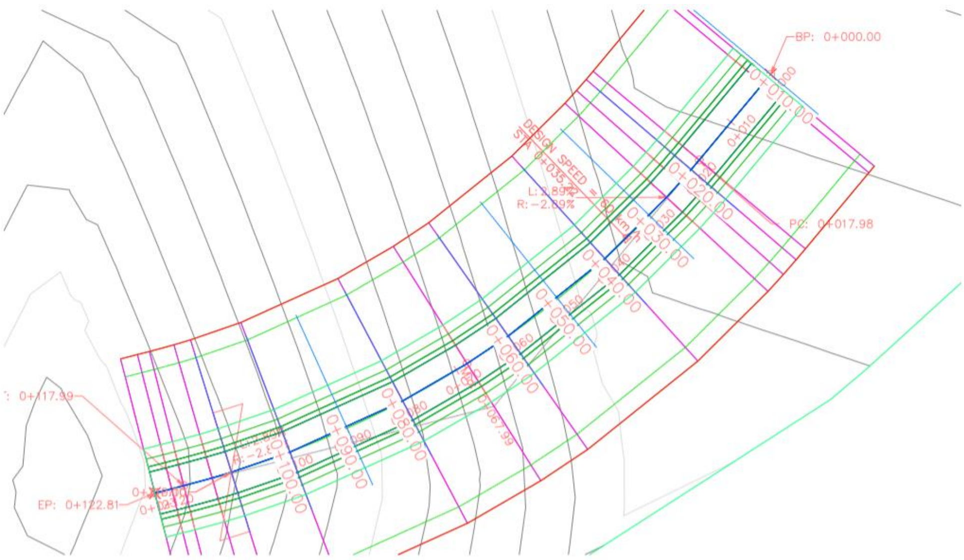

Figure 5.63 Corridor view

5.12.4 SUPERELEVATION

Superelevation is the banking of a curve to help counterbalance the centrifugal forces that a vehicle experiences while traveling through a curve. When superelevation parameters are applied to the alignment object, the resulting corridor model will rotate and warp the cross-sectional links and shapes to reflect the raising of one edge of the travel way above the other. The corridor relies on the behavior of subassemblies to adjust its final shape and position accordingly. Many out of-the-box Civil 3D subassemblies support superelevation.

Table 5.7 Super elevation details

Description	Start Station	End Station	Left side shoulder	Left side lane	Right side lane	Right side shoulder
Being alignment	0+000 m		-4.00%	-2.5%	-2.5%	-4.00%
End of Normal cross fall	0+017.91m		-4.00%	-2.5%	-2.5%	-4.00%
Transition In Region	0+017.91 m	0+047.85 m	-4.00%	0.0%	-2.5%	-4.00%
Tangent Run out	0+017.91 m	0+033.96 m	-4.00%	0.0%	-2.5%	-4.00%
Tangent Run Off	0+033.96 m	0+047.85 m	-4.00%	2.5%	-2.5%	-4.00%
Full super elevation	0+047.85 m	0+087.97 m	-4.00%	2.89%	-2.89%	-4.00%
Transition Out Region	0+087.97 m	0+117.91 m	-4.00%	2.5%	-2.5%	-4.00%
Tangent Run Off	0+087.97 m	0+104.02 m	-4.00%	2.5%	2.5%	-4.00%
Tangent Run Out	0+104.02m	0+117.91 m	-4.00%	0.0%	-2.5%	-4.00%
Being Normal cross fall	0+117.91 m		-4.00%	-2.5%	-2.5%	-4.00%
End of Alignment	0+123 m		-4.00%	-2.5%	-2.5%	-4.00%

5.12.5 ROAD FILL BETWEEN APPROACH ROAD AND EXISTING ROAD

Between the approach road and the existing A1 road a fill was designed. The top of the fill be covered with grass and trees will be planted to improve the aesthetic around the bridge.

The filling plan view and profile through road chainages are shown in figure 5.64. The filling area was divided in to 10 m intervals to calculate cumulative filling volume. The detailed calculations are given in Appendix G.

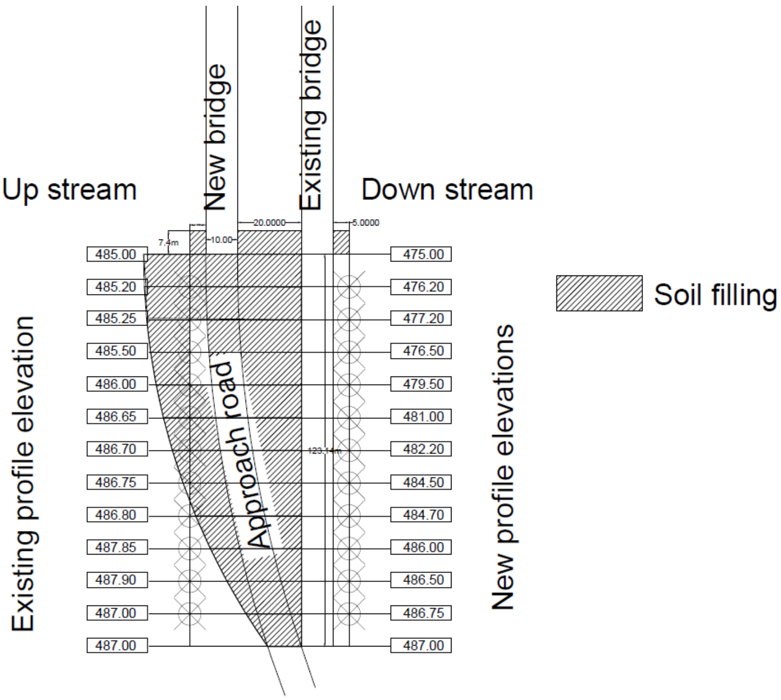

Figure 5.64 Plan view of the fill

There is a valley area between the existing bridge and the proposed bridge. It should be filled with a sandy loam soil. All fill material used must be free from roots, or any vegetable matter. In that valley area there is a water path. Therefore, a drainage path should be provided. New fill material must be fully keyed by means of benching. Each step should be compacted and filled with a suitable soil as shown in figure 5.65.

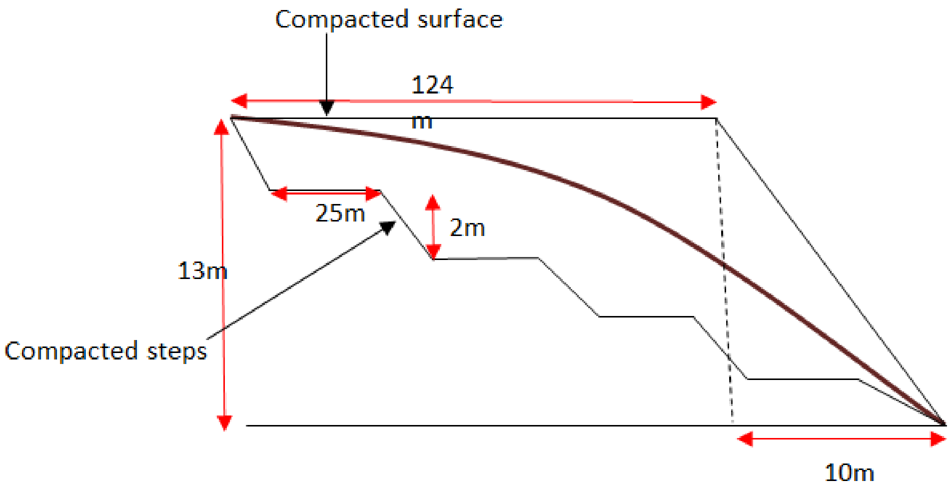

Figure 5.65 End view of steps

Then, a culvert was designed to provide drainage through the filling section as shown in figure 5.66. Scouring protection design was done to both pipe inlet and outlet. After that, a riprap protection was used along the filling section to provide protection against erosion on both sides of the new bridge.

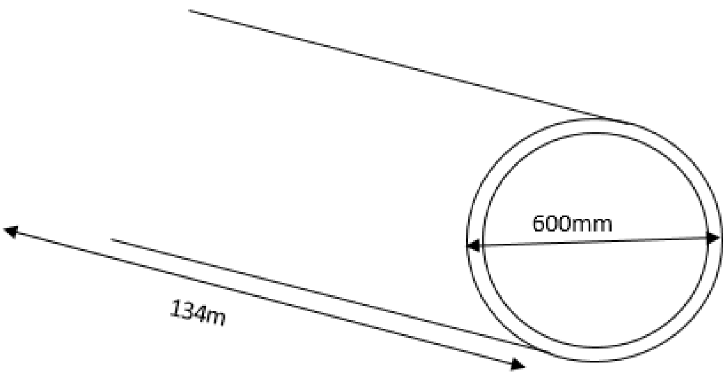

Figure 5.66 Diameter and the length of the culvert

For the pipe outlet, 2.6m length rip rap going to be provided.

5.13 ROAD WIDENING DESIGN

The A1 and A5 roads were proposed to wide as four lane roads in order to cater the additional traffic from the new bridge. The road will be widened from both sides of the road. The new L1 and L4 lanes are shown with the existing L2 and L3 lanes in figure 5.67.

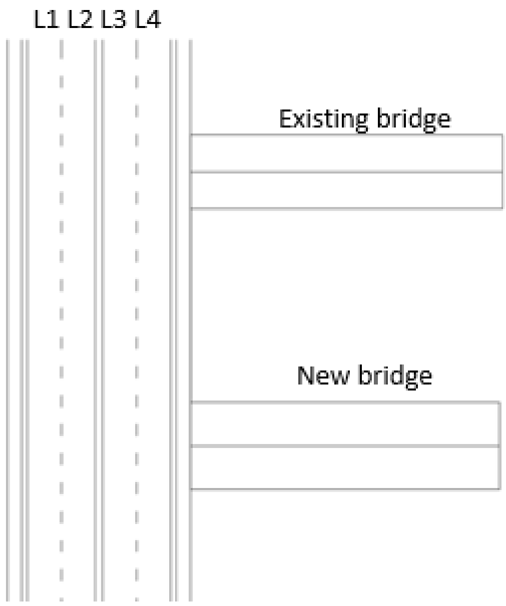

Figure 5.67 Plan view of the road

Design Period

The design life of the road is decided based on the type of construction as shown in table 5.8,

Table 5.8 Design life for various types of road works (Austroads, 2009)

Work type	Design life
Pavement rehabilitation	15-20
Widening	30
New road	30
New bridge	100
Future bridge widening	50

For road widening, the design life is considered to be 30 yrs.

Main elements of road cross-section

- Carriageway
- Camber
- Median

- Sidewalk
- Verge
- Batter
- Side slopes
- Drains

Carriageway (Traffic lane+ Shoulder)

- **Traffic Lane**

Traffic lane is the strip of the carriageway occupied by vehicles moving in a single stream along the road. Generally, it varies from 3.1m to 5.5m according to *Austroads, 2009.*

A 3.5m lane width was selected, relating to the road type (A class road), traffic volume and composition.

- **Shoulder**

The shoulder is the sealed edge of the road outside the traffic lanes.

Functions - Accommodation stop vehicles
Provide lateral support to the road pavement layers
- minimum 0.5m width

Provide space for driver use to avoid a collision and regain control
- Desirable width =3.0m
- Minimum width =2.4m
- Absolute minimum width =1.8m

Offers improved condition for cyclist

In this design shoulders are not provided to accommodate the vehicle parking as parking is not allowed in the project boundary. Therefore, shoulder-width was selected as 0.5m.

Camber

Camber is defined as the cross slope provided to raise the middle of the road surface in the transverse direction (Guide to Road Design Part 3). Recommended Pavement cross fall and shoulder cross fall values are given in Table 5.9 and 5.10

Table 5.9 Pavement cross fall

Type of surface of the pavement	Recommended cross fall (%)
Portland cement concrete	2.0
Asphalt Pavement	2.5
Surface seal	3.0
Unseal gravel	4.0

Table 5.10 Shoulder cross fall

Type of surface	Recommended cross fall (%)
Bitumen	3-4
Gravel	4-5

Carriageway cross fall is selected as 2.5% and shoulder cross fall is selected as 3.0%.

Median

Median is defined as the central raised or depressed strip within the roadway constructed to separate the traffic flow. For this design, a 0.8m median was provided to separate the traffic flow according to the table 5.11.

Table 5.11 Median Widths (Austroads, 2009)

Median function	Minimum width (m)
Separate traffic flows with a rigid (concrete) safety barrier (1) (no provision for shoulder or allowance for shy line effects) (2)	0.8
Shelter a small sign	1.2
Shelter signal pedestals or lighting poles	2.0
Shelter pedestrians (provision for Tactile Ground Surface Indicators) and traffic signals	2.5
Shelter turning vehicles and traffic signals	6.0
Shelter crossing vehicles	7.0
For planting and drainage	10.0
Recovery area	15.0

Sidewalk

Desirable minimum width of a footpath in an urban area is 1.2m according to the Austroads, 2009. In this design, a side walk is provided with a width of 1.5 m on both sides of the roads. The cross-section of the widened road is shown in figure 5.68

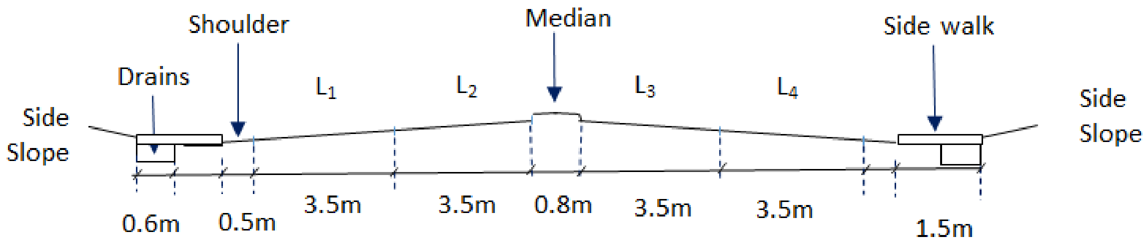

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Figure 5.68 The cross-section of the widened road

Moreover, the Mahaweli River and the botanical garden can be seen along the sidewalks. Therefore, the side walk can be used as a walking path with a sightseeing view.

Drainage

Issues associated with drainage of the road and surrounding land can significantly affect the geometry and cross-section of the road. Provision of drainage structures at watercourses affects the grading of the road. The choice of the drainage system can affect the cross-section or formation width, maintenance requirements and cost of the project.

Using rational formula for the runoff water drainage system,

$$Q = 0.028CIA$$

Q = Maximum runoff in m^3per sec

C = A constant depend upon nature of the surface range (0.31 – 0.93)

I = the critical intensity of storm in mm per hour

I = 198.9 mm from rainfall data (Irrigation department)

c = 0.7

Assuming the catchment area as, A = 0.072km2

Q = 0.28m3/s

$Q = AV$

Assuming,

$V = 1m/s$

$A = 0.28m2$

Assuming drain width as 0.6m (minimum width according to the standard drawing)

Height of the drain, $= A/W$

$= 0.45m$

With 150mm freeboard,

Height of the drain is 0.6m

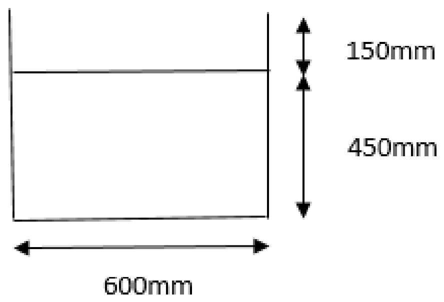


Figure 5.69 Cross section of the drainage

Minimum slope for the road drainage is 1% (Guide to Road Design Part 3) Therefore, proposed slope for the drainage is 1%. The drained water will be released to Mahaweli River from drain outlets located near the bridge.

5.14 SLOPE STABILIZATION

Slope stabilization is often required to retain soil in natural unstable slope or man-made excavation. There are several slope stabilization methods. According to the Geotechnical Design Manual, slope stabilization methods can be summarized as figure 5.71. Figure 5.70 shows the slope representation as Horizontal length / Vertical height (H/V).

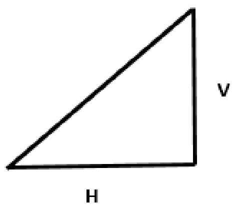


Figure 5.70 Slope representation

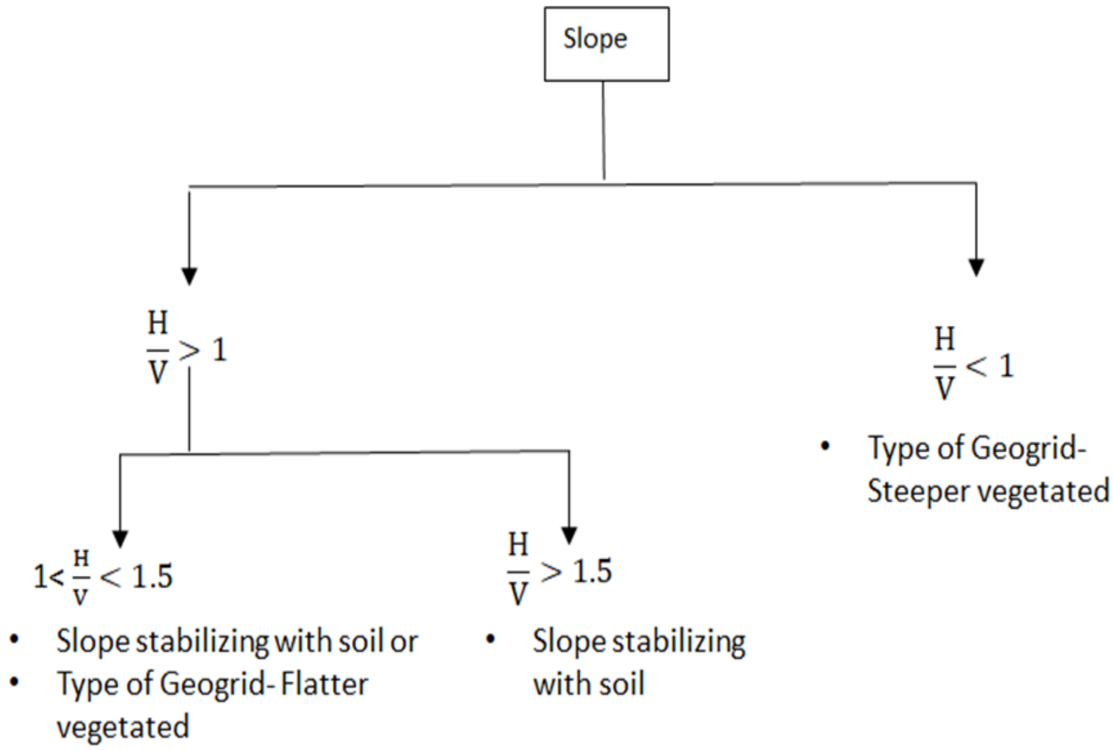


Figure 5.71 Slope stabilization method according to the slope

According to the soil type, stable slope ratios are can be shown in table 5.12

Table 5.12 Common stable slopes ratios

Soil/ rock condition	Common stable slope ratio (H: V)
Most rock	¼ : 1 to ½ :1
Very well cemented soils	¼ : 1 to ½ :1
Most in place soils	3/4: 1 to 1:1
Very fractured soil	1:1 to 1 ½ :1
Loose coarse granular soil	1 ½ to 1
Heavy clay soil	2:1 to 3:1
Soft clay-rich	2:1 to 3:1
Fills most soil	1 ½ :1 to 2:1
Fill with hard, angular rock	1 1/3 to 1
Low cuts and fills (< 2-3m)	2:1 or flatter

Plan view of the left river bank is shown in figure 5.72.

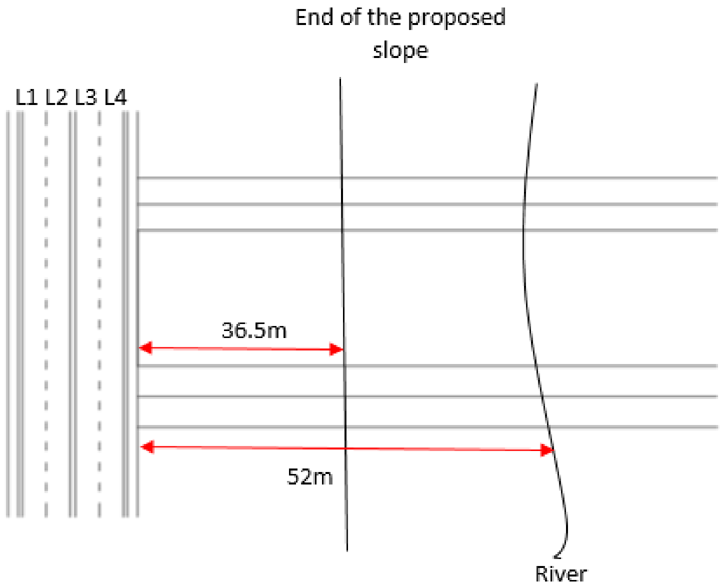


Figure 5.72 Plan view of the left river bank

Slope starts from the end of the road. Considering height as 21m and width as 36.5m, slope can be obtained as,

H:V = 1.73:1

5.14.1 SLOPE STABILITY CHECK USING SLOPE/W SOFTWARE

The proposed slope stability was checked using the GeoStudio software. The software required some soil parameters as inputs. Those parameters were obtained from the data available in the Geotechnical Laboratory, University of Peradeniya.

Input soil parameters

- Cohesion – 20kPa
- Friction angle - 28˚
- Unit weight – 18kN/m^3

Surcharge load on the road

- 10kN/m^2

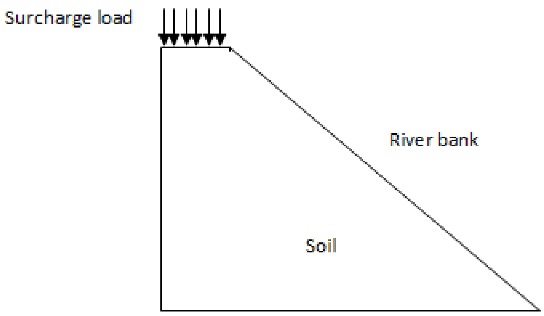


Figure 5.73 Input parameters to the software

The critical value of the factor of safety was found as 1.823 by the GeoStudio software. Therefore, this slope is stable as shown in figure 5.74.

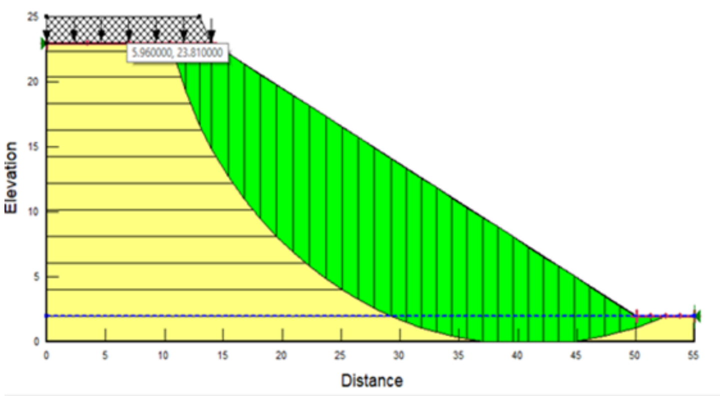


Figure 5.74 Slope analysis result from Geo studio software.

In this situation,

- Soil slope = 1.73:1 > 1.5: 1 (using design manual)
- FOS = 1.823 > 1 (using slope/W)
- Only one lane is going to be filled with the soil (using design manual)

Therefore, the slope was designed to fill with soil and covered with vegetation to reduce erosion. This is adequate for the left bank and the aesthetic view for the area can be improved.

Vegetation can improve the stability of slopes through

- Soil moisture depletion
- Root reinforcement
- Buttressing and arching
- Surface cover shading the soil.

Vegetation cover can reduce soil moisture content through

- Foliar interception and direct absorption and evaporation of rainwater, which reduces the amount infiltrating into the soil
- Extracting soil moisture via the transpiration stream, thus reducing pore water pressure and counteracting the reduction in soil strength that wetting causes.

Moreover, a rock cover is located at the toe of the slope to prevent any erosion due to floods. According to the proposal, slopes can be finalized as shown in figure 5.75.

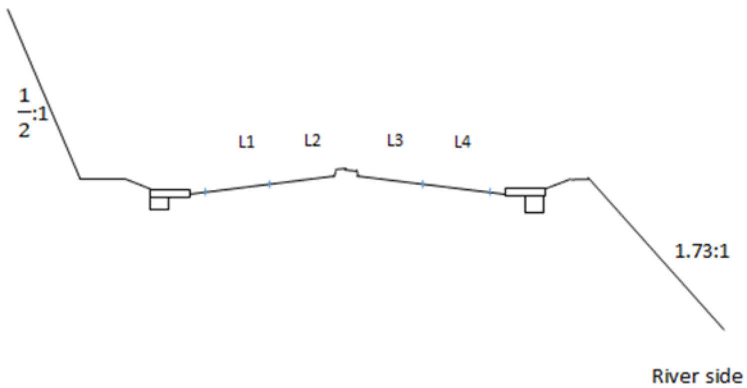


Figure 5.75 Section view of proposed slopes and four-lane road

5.14.2 MATERIAL CONSIDERATION

Selection of grass-type

When selecting grass type, there are some factors should be considered,

- Density
- Height
- Propagation rate
- lifetime
- Soil surface covering ability
- Relative climate condition

Seeding type grass

Function – Grass is sown directly on to the surface

Sites – Slope is greater than 45^0

Advantages – A very cheap and rapid method

Timing – During the monsoon period

Maintenance – It should be maintained annually

Therefore, seeding type grass was selected to cover the soil as shown in figure 5.76.

Figure 5.76 Greenfix grass

Selection of soil and rock

Soil -In the research paper "Application bioengineering to slope stabilization in Sri Lanka", sandy loam was selected for the 55^0-60^0 slopes.

Therefore, Sandy loam is selected as this design with a fill slope (1.73: 1 ≈ 60^0) as shown in figure 5.77.

Rock -4"-9" diameter rocks are selected as rock cover at the toe to prevent the erosion according to the geotechnical design manual.

Figure 5.77 Sandy loam soil

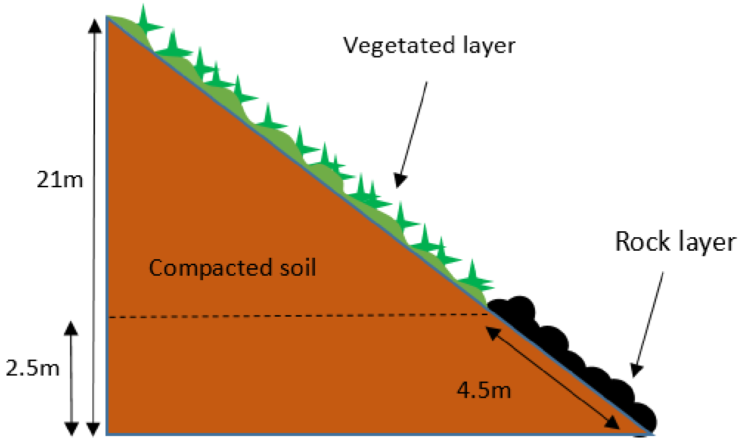


Figure 5.78 Sectional view of proposed fill slope with material

Facing area of the stabilized slope

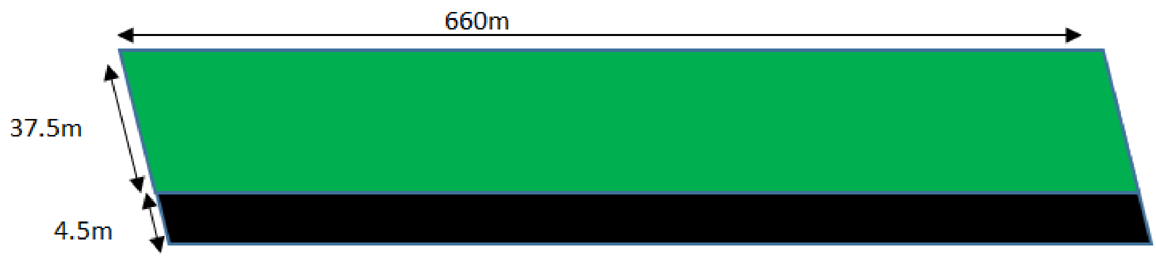


Figure 5.79 facing area of the stabilized slope

Vegetated area = 660m × 37.5m

= 24,750m^2

0.6m < Rock layer thickness < 1.2m

- 0.6 m is selected as the rock layer thickness
- Rock volume = 660m × 4.5m × 0.6m

  = 1782m^3

5.15 TRAFFIC SIGNAL SYSTEM DESIGN

Traffic Signal lights are assigned at intersections in this design. The conflicts arising from movements of traffic in different directions is solved by time sharing of the principle. The advantages of traffic signal include an orderly movement of traffic, an increased capacity of the intersection and requires only simple geometric design. However, traffic signals may result in delays. To minimize such delays, the phasing of traffic signals is essential.

Initially, the traffic survey results were analyzed and passenger car unit volumes were found for all the traffic movements. After that, the critical traffic flows were identified and the traffic signal design was done to the most critical directional flows as per table5.13. The detailed design of traffic signal system is given in Appendix H.

Table 5.13 Critical 3 direction flow in PCU/h

	TFlow	PCEF	RT LAF	PAF	LWAF	PCU	Rounded value
Colombo to Kandy	520	0.86	-	1	1	447	450
Gampola to Kandy(RT)	1050	0.8	1.05	1	1	882	885
Kandy to Colombo(RT)	520	0.93	1.05	1	1.1	559	560

The figure 5.80 shows the traffic flow directions in the Peradeniya junction and the blue color arrows show the critical flows.

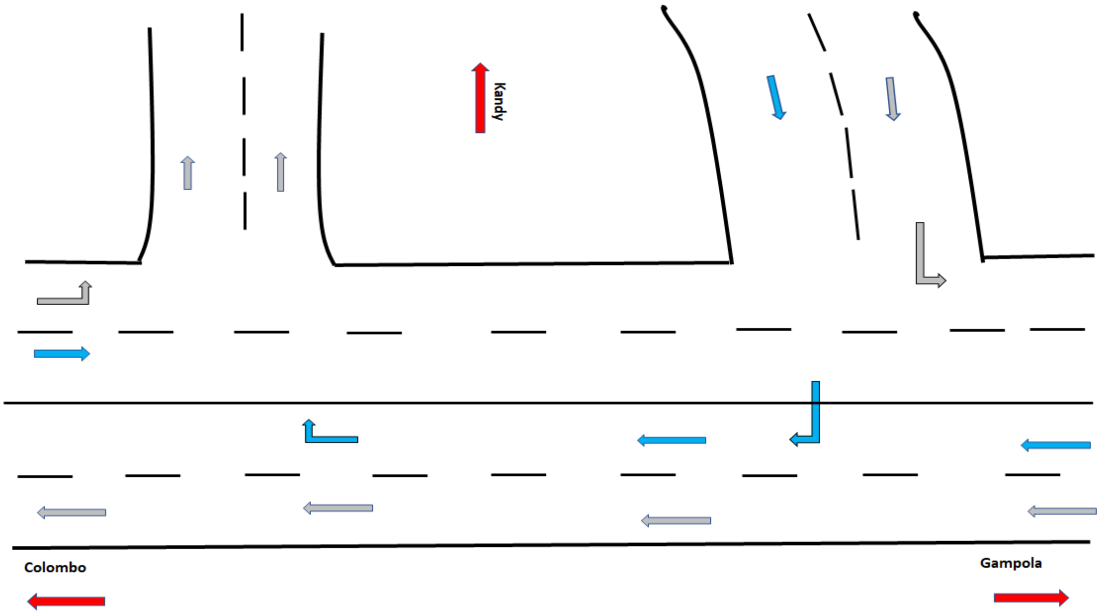


Figure 5.80 Traffic flow direction

After analyzing the critical flow volumes the signal design was done to the locations A, B and C in the figure 5.81.

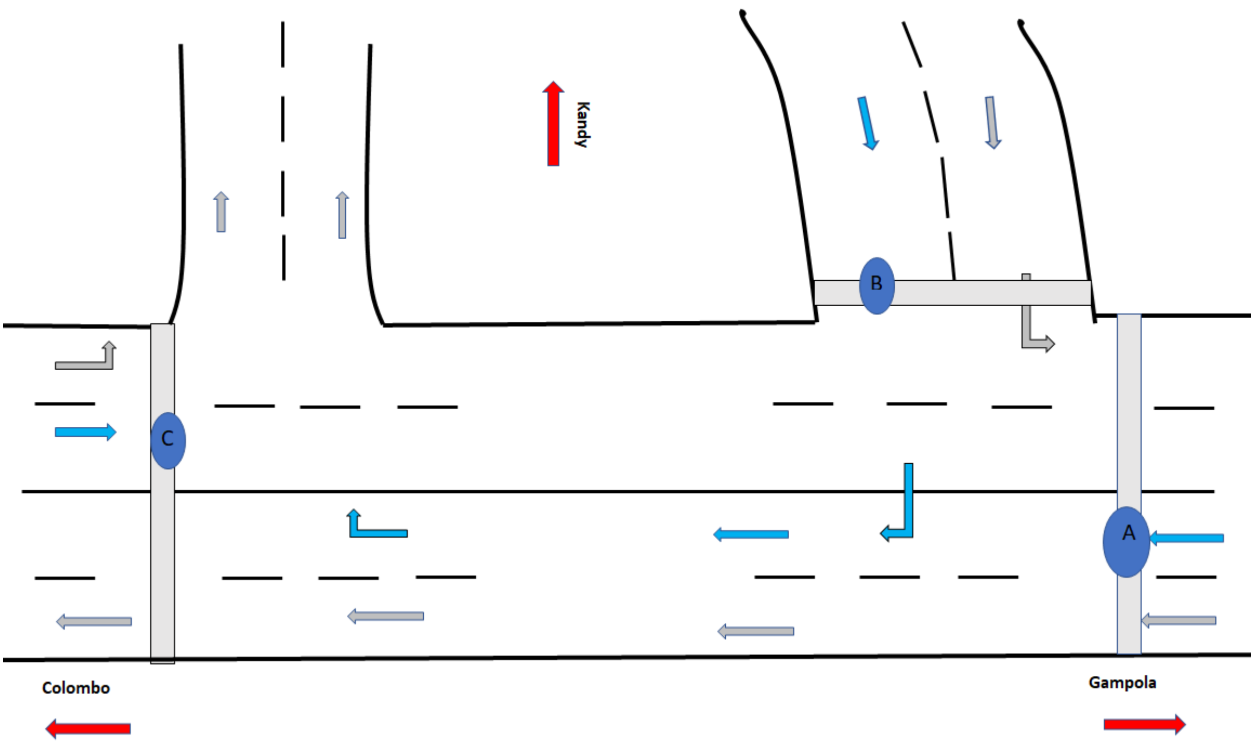


Figure 5.81 Placement of signals

A traffic signal sequence design was done to signal A, B and C using Webster and Cobbe formula. The red amber and green time was given for all the three signals in the figure 5.82.

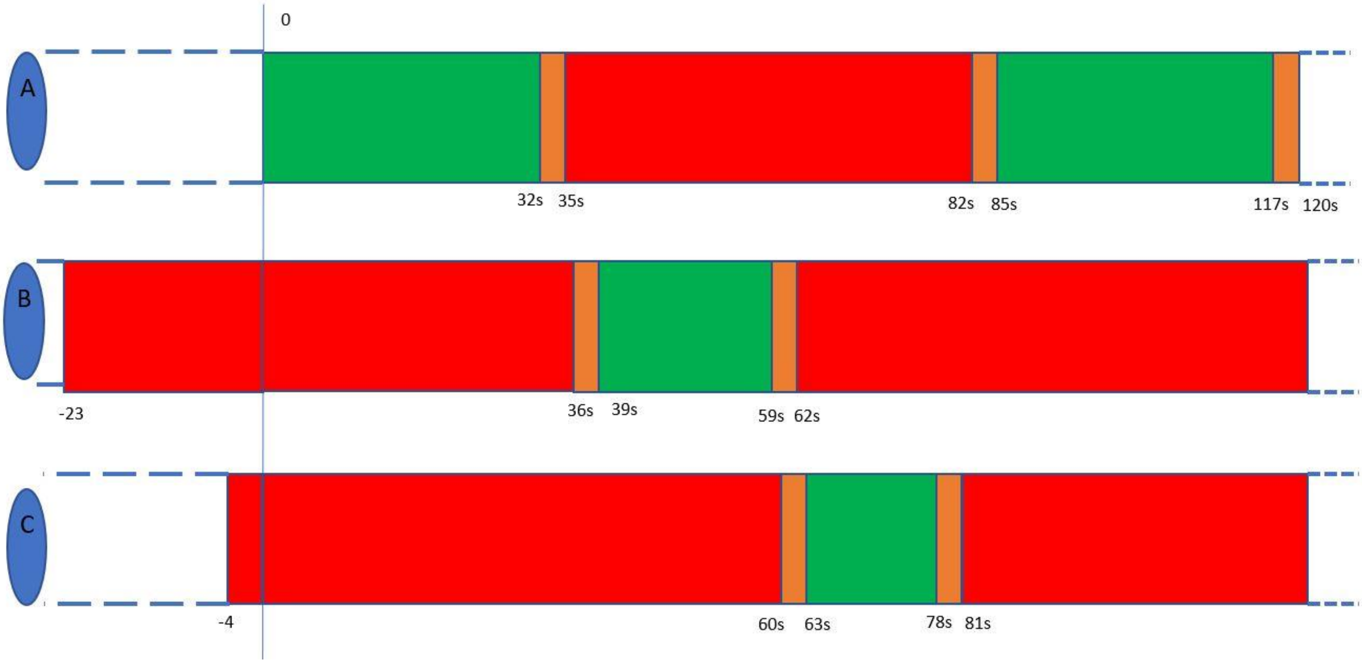


Figure 5.82 Traffic signal timing diagram

5.16 UNDERPASS DESIGN

As mentioned in chapter 2, it was observed that the Peradeniya junction does not have an adequate pedestrian crossings across the road. With the widening of the road a proper pedestrian crossing is an essential component in the design. Pedestrian overpasses and underpasses allow for an uninterrupted flow of pedestrian movement separate from vehicle traffic. Thus, an overpass is not suitable for the aesthetic view of Peradeniya. Therefore, an underpass system was designed as the pedestrian crossing for this design.

Box type single cell culvert was used as the underpass system in this design. The RCC culvert consists of two horizontal and two vertical slabs. For an underpass, the top slab is required to withstand dead loads, live loads from moving vehicles, earth pressure on side walls and pedestrian live loads, pressure on the bottom slab besides self-weight of the slab. A preliminary design study was conducted for the underpass system.

The cross section of the underpass was designed considering all the loads acting on the top of the underpass as per figure 5.83. The detailed design of underpass is given in Appendix I.

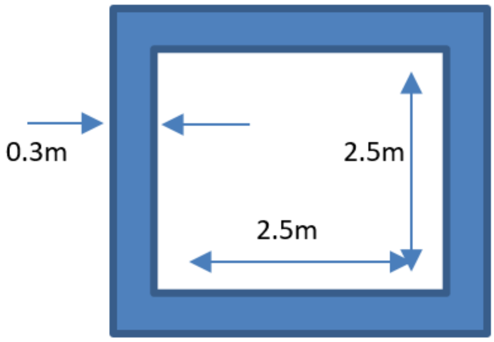


Figure 5.83 Cross section of culvert

Then, an approach slab (Figure 5.84) was provided to equalize settlements of embankment soil and provides the transition between road pavement and culvert.

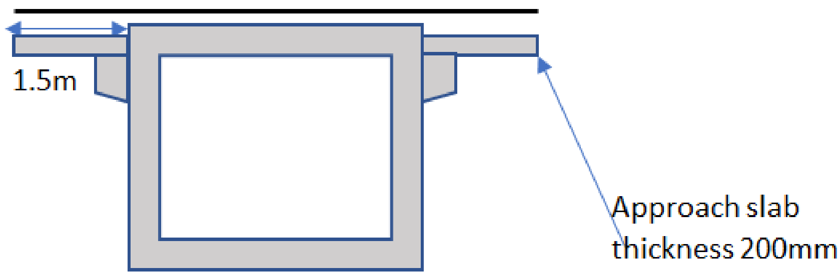


Figure 5.84 Placement of approach slab and dimensions

5.16.1 VENTILATION

For the ventilation, Ground type ventilation opening (Figure 5.85) was going to be used in the middle of the underpass. Ground type openings are normally installed above 0.3m height and 2m diameter. Roof will be placed over the opening to avoid the rain water go inside. This type is beneficial to take fresh air into ventilation opening. The exhausted air is also expected to have better dispersion. It will give aesthetic view at the junction.

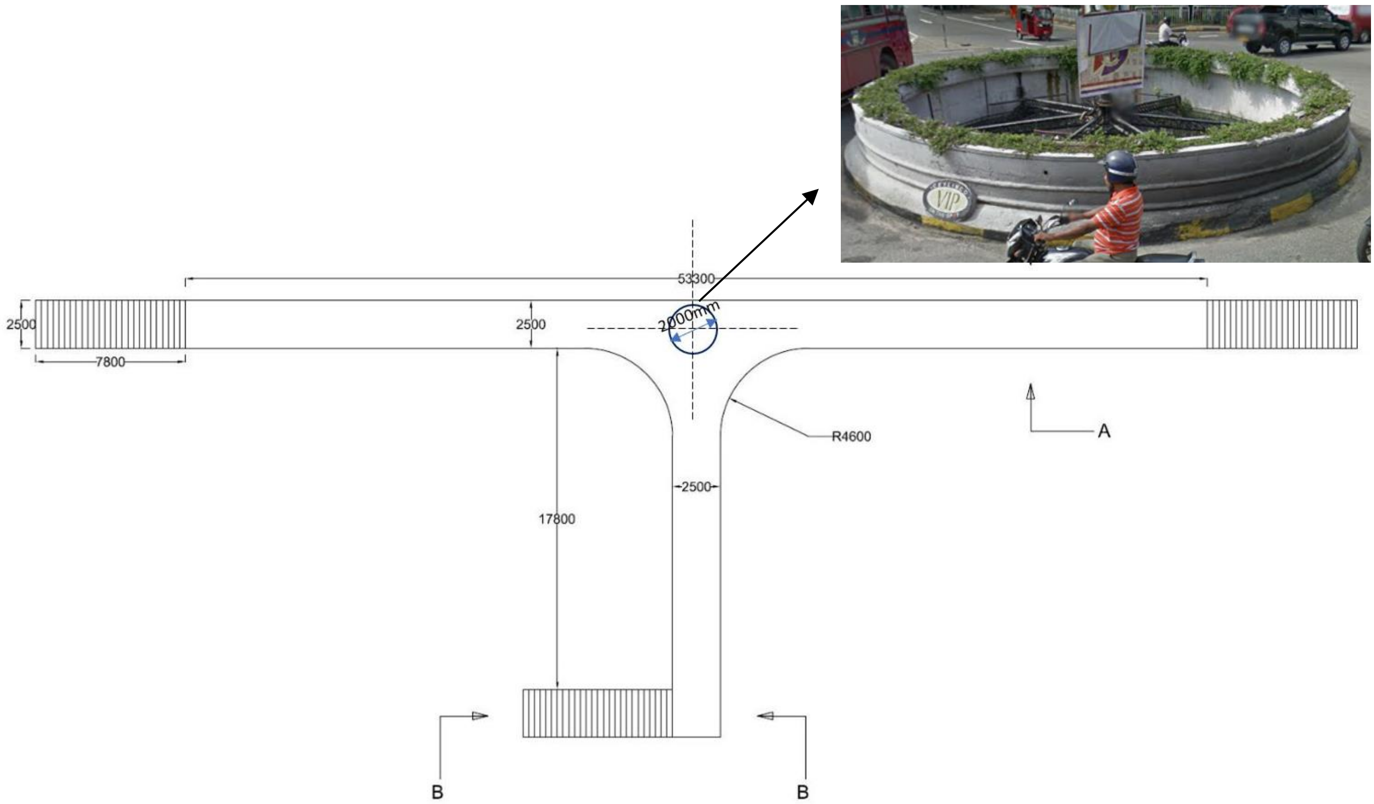


Figure 5.85 Proposed ventilation system

5.16.2 DRAINAGE

The drainage system should be large enough to deal with the water and detritus entering the underpass from the stairs. Drainage grills are provided at entrances to collect the water. Those Grills are connected with 315mm diameter pipes and drained water is discharged into the Mahaweli River. Drainage system for the underpass is shown in figure 5.86.

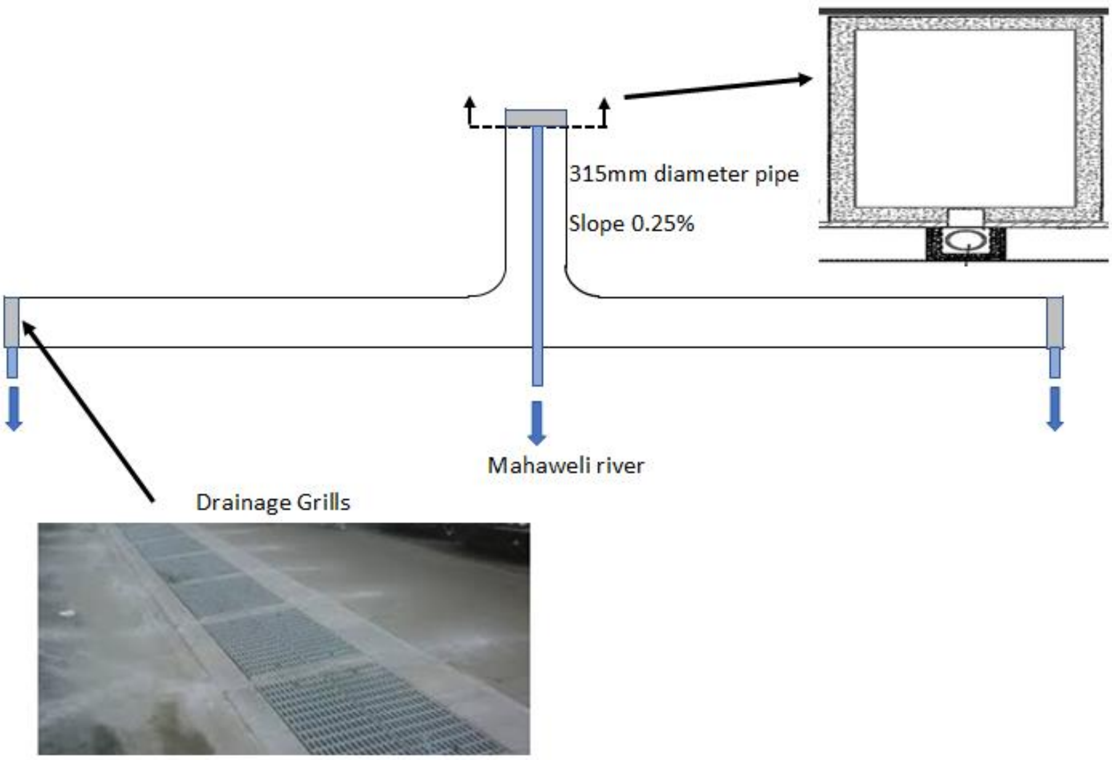


Figure 5.86 Proposed drainage system

5.16.3 ACCESS TO THE UNDER PASS

Pedestrian can use the stairs to enter the under pass. Dimension stair elements are given in

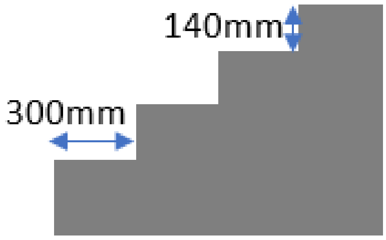


Figure 5.86 Stair elements

Required no of steps = 364/14

= 26

Total length of stair = 7.8m

Width = 2.5m

Stair pitch should be uniform for the underpass, with steps of equal rise. Nosing on the stairs should be rounded to a 6mm radius without overhang handrails is provided on both side of stairs. Details of hand railing are shown in below figure 5.88.

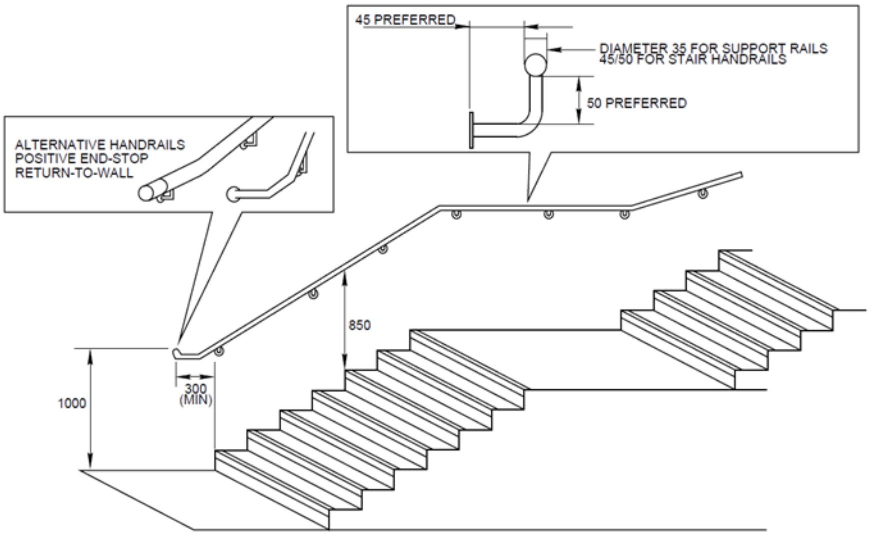


Figure 5.88 Hand railing details

5.16.4 LIGHTING

Artificial lighting is provided for use in the hour of darkness both inside the underpass and on the underpass approach. For the lightening purpose longitudinal mounted lamps system (figure 5.89) will be used. Level of illumination given in BS 5489 is recommended for the underpass which is shown in Table 5.14.

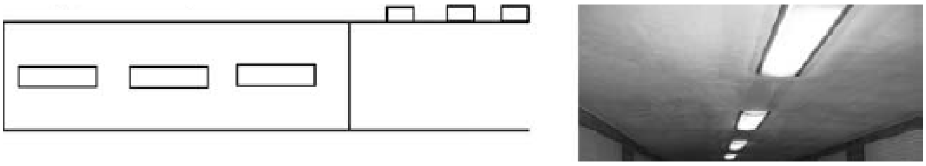


Figure 5.89 Longitudinal mounted lamps system

Table 5.14 Level of illumination

Type		**Day**		**Night**	
		E_H **(average)** lx	E_H **(minimum)** lx	E_H **(average)** lx	E_H **(minimum)** lx
Subways:	open	n/a	n/a	50	25
	enclosed	350	150	100	50
Footbridges:	open	n/a	n/a	30	15
	enclosed	350	150	100	50
Stairway/ramp:	open	n/a	n/a	30	15
	enclosed	350	150	100	50
NOTE 1 "Open" equates to major daylight penetration.					
NOTE 2 For "enclosed" areas emergency lighting needs to be considered. It is essential that it is installed if the area forms part of an escape route from a shopping centre, car park or transport interchange.					
NOTE 3 Where longer subways have poor daylight penetration, or where subway user confidence needs to be ensured, it may be necessary for the threshold illuminance value to be up to twice the value of the general daytime service level.					
Key					
E_H (average) is the maintained average horizontal illuminance (in lx);					
E_H (minimum) is the maintained minimum horizontal illuminance at any point (in lx);					
n/a not applicable.					

5.17 TRAFFIC CONDITION AFTER IMPLEMENTATION OF THE NEW BRIDGE

After completing all the detailed designs, level of serviceability of the new bridge and existing bridge were checked with the reduced traffic flows.

In section 2.3 table 2.2 and 2.3 shows the traffic volume from Kandy to Peradeniya junction and Peradeniya junction to Kandy through existing Peradeniya Bridge. After implementing the new bridge the traffic from Kandy to Peradeniya junction will be on the new bridge and traffic from Peradeniya junction to Kandy will be on the existing Peradeniya Bridge.

Initially, Passenger Car Unit values were calculated for both directions as table 2.5. Then, using that data, passenger car units per hour (PCU/h) was calculated as in table 5.15.

Table 5.15 Passenger car units per hour and vehicle per hour for both bridges

	New bridge		Existing bridge	
TIME (a.m.)	**Vehicles per hour**	**PCU per hour**	**Vehicles per hour**	**PCU per hour**
6.30 - 7.30	1132	1009.6	1645	1382.2
6.45 - 7.45	1273	1115.8	1651	1302.4
7.00 - 8.00	1312	1144.6	1685	1304.6
7.15 - 8.15	1272	1119.2	1897	1459
7.30 - 8.30	1205	1067.8	1997	1555.6
7.45 - 8.45	867	778.6	1522	1194.4

From the table 5.15, peak hours were found as 7.00 – 8.00 for the new bridge and 7.30 – 8.30 for the existing bridge.

<u>New bridge</u>

Peak hour 7.00 - 8.00 a. m.

Peak hour volume = 1144.6 PCU/h

Total vehicles in peak hour = 1312 (1300 -1400)

So, PHF = 0.95 (Transport Research Board, 1984, "Highway Capacity Manual")

<u>Existing bridge</u>

Peak hour 7.30 - 8.30 a. m.

Peak hour volume = 1555.6 PCU/h

Total vehicles in peak hour = 1997 (> 1900)

So, PHF = 0.96 (Transport Research Board, 1984, "Highway Capacity Manual")

Adjustment factor of directional distribution for the bridges

Both bridges are acting as one way bridges,

So, no directional split of vehicles on the bridge.

Therefore, adjustment factor for directional distribution (f_d) = 1.00

Level of Service (LOS) on the bridges

Bridge is located in level terrain and no passing zones is 0% due to one way two lane bridge.

New bridge

Lane width = 3.5 m

Shoulder width = 0 m

Therefore, adjustment factor for narrow lanes and restricted shoulders (f_w),

For LOS A – D, = 0.675

For LOS E = 0.850

Consider LOS A,

Volume / Capacity (v/c) ratio = 0.15

Service flow rate, SF = 2800 * f_w * f_d * v/c

= 2800 * 0.675 * 1.00 * 0.15

= 283.5 PCU /h

Likewise, for each LOS, Service flow rates (SF) were calculated as in table 5.16.

Table 5.16 Service flow rates for each LOS on existing bridge

LOS	Adjustment factor for narrow lanes and restricted shoulders(f_w)	Adjustment factor for directional distribution (f_d)	v/c ratio	SF (PCU/h)
A	0.675	1	0.15	283.5
B	0.675	1	0.27	510.3
C	0.675	1	0.43	812.7
D	0.675	1	0.64	1209.6
E	0.850	1	1	2380

Peak flow rate for the traffic = peak hour volume of new bridge / PHF

= 1144.6 / 0.95

= 1204.84 PCU /h (< 1209.6)

So, according to the service flow rates in table 5.166, New Bridge operate in **LOS C**.

Existing bridge

Lane width = 2.76 m

Shoulder width = 0 m

Therefore, adjustment factor for narrow lanes and restricted shoulders (f_w) (Table 7.2 – Highway Capacity Manual 1985)

For LOS A – D, = 0.49

For LOS E = 0.66

Consider LOS A,

Volume / Capacity (v/c) ratio = 0.15

Service flow rate, SF = 2800 * f_w * f_d * v/c

= 2800 * 0.49 * 1.00* 0.15

= 205.8 PCU per hour

Likewise, for each LOS, Service flow rates (SF) were calculated as in table 5.17.

Table 5.17 Service flow rates for each LOS on existing bridge

LOS	Adjustment factor for narrow lanes and restricted shoulders(f_w)	Adjustment factor for directional distribution (f_d)	v/c ratio	SF (PCU/h)
A	0.49	1.00	0.15	205.8
B	0.49	1.00	0.27	370.44
C	0.49	1.00	0.43	589.96
D	0.49	1.00	0.64	878.08
E	0.66	1.00	1	1848

Peak flow rate for the traffic =peak hour volume on existing bridge (from table 05)/ PHF

= 1555.6 / 0.96

=1620.42 PCU per hour (< 1848.00)

So, according to the service flow rates in above table 5.17, Existing Bridge operate in **LOS D**.

The two bridges have higher level of serviceability during peak hour than the existing bridge. Thus, it can be stated that with the implementation of the new bridge level of serviceability of bridges will be increased. Hence, traffic congestion in Peradeniya junction will be reduced.

CHAPTER 6

PRELIMINARY EIA AND COST ESTIMATE

6.1 PRELIMINARY ENVIRONMENTAL IMPACT ASSESSMENT

Environmental issues are receiving high priority in the development agenda at present as humans are now suffering from neglecting those in the early stages of development. Climate change and resource degradation are some of the major impacts, the world faces today. Learning from past, Environmental Impact Assessment (EIA) was established to manage the impacts on environment due to development projects and to enhance the environmental quality where possible.

Peradeniya town is situated 5 km away from Kandy town and lies at crossroads of several major road networks of Sri Lanka, namely A1 road (Colombo- Kandy) and A5 road (Peradeniya-badulla-Chenkalady). Therefore, Peradeniya Town acts as a gateway to the historic Kandy city and experiences a huge movement of traffic and passengers every day. The Peradeniya Bridge in the Colombo – Kandy main road is a very old bridge and due to the lack of capacity, the traffic will be congested during peak hours. Therefore it is essential to establish a proper road system to reduce the traffic congestions.

The major objective of this project is to mitigate traffic congestion in the existing Peradeniya junction. In the selected solution, a new two lane bridge parallel to the existing bridge was proposed along with the widening of A1 and A5 roads from Gannoruwa junction to Penideniya junction. Moreover, a new approach road was designed to connect A1 road with the new bridge. On the other hand, Signal light system will be introduced to ease the traffic congestion. Shops located along the roadside edge besides Mahaweli River will be removed and walkways will be provided for the pedestrians. And underpass for safe pedestrian crossing will be implemented in this project. Another objective is to enhance the aesthetic appearance in the Peradeniya town area. For that, a bedded grass slope protection technique is proposed instead of a retaining wall for left river bank. Subsequently, to improve all the facilities to road users, landslide mitigated area will be utilized to build a multi-story building which will provide shopping facilities, banking facilities, parking facility, sanitary facilities and relocate some of the existing shops into.

The objective of EIA is,

- To identify, predict and evaluate the economical, environmental and social impact of development activities.
- To provide on the economical consequences for decision making.
- To promote environmentally sound and sustainable development through the identification of appropriate alternatives and mitigation measures.

6.1.1 METHODOLOGIES AND TECHNOLOGIES ADOPTED FOR EIA

A discussion session was conducted among the group members, before preparing the EIA. Initially, the project was familiarized through site visits, discussions among group members, and referring relevant information. Thereafter, all possible impacts on the environment were identified and analyzed to recommend possible mitigation measures to reduce them.

The EIA report is a presentation of the potential impacts of the project on the physical and social environment, as well as a discussion on mitigation measures to minimize these adverse impacts. By monitoring basic conditions of critical environmental parameters, significant adverse changes due to project activities can be detected.

6.1.2 EXISTING ENVIRONMENT CONDITION

Topography

Peradeniya is in the hill country of Sri Lanka at an elevation range of 450-500 m above MSL. Mountainous topography is to be noted. The bridge is proposed to construct across the Mahaweli River. The proposed approach of road alignment follows mostly plain and steep terrain. So, the approach road is proposed to construct with the suitable filling. Proposed alignment mostly passes through the vacant land. The elevation profile of the approach road is shown in figure 6.1.

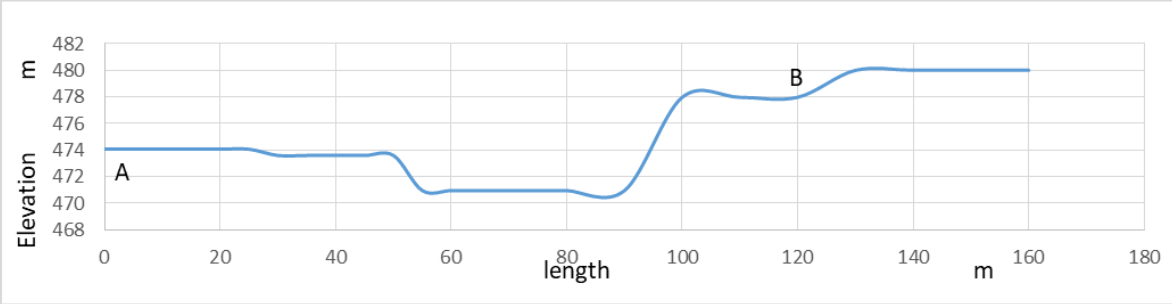


Figure 6.1 Existing elevation of profile of approach road

Weather and Climate

Peradeniya is in the wet zone of Sri Lanka and experiences relatively higher annual precipitation and milder temperatures than most areas of the country.

Soil

Peradeniya is in the Highland Complex of the Sri Lankan geology map. Highland Complex consists of Granulite facies rocks such as Charnokitic gneiss, Marble, Quartzite, and Quartzofeldspathic gneiss. Floodplains along Mahaweli River provides thick alluvial profiles.

Air quality

All roads within the project area are busy urban streets with heavy vehicle movements. This lead to high levels of air pollutant emissions, such as dust including particulate matter, smoke including CO, CO_2, NO_x , SO_x etc. Such air pollution episodes can aggravate during peak hours when traffic movements are impeded.

Noise quality

Noise is an important environmental attribute in the project because vehicular traffic is a major source of noise pollution. The main objective of noise monitoring is to establish the baseline noise levels, which was used to assess the impact of total noise generated by the proposed project activities.

Sensitive Areas

Peradeniya botanical garden is the largest botanical garden in Srilanka located near the project area. Moreover, a Buddhist shrine and Bodhi tree, Jumma Muslim Mosque, Peradeniya University and children’s hospital are sensitive areas in Peradeniya town.

6.1.3 ENVIRONMENTAL IMPACTS & MITIGATION PLAN

This section assesses the nature, type and magnitude of the potential impacts that are likely to occur on various relevant physical, biological and cultural environmental components along the project corridor and its suitable mitigation.

The impacts on the various environmental components were assessed considering following stages of the project planning and implementation:

- Planning and design stage
- Construction stage
- Operating stage

<u>Beneficial Impacts</u>

- **Beneficial Impacts during Construction Phase**

Employment Opportunities

During construction, the project will generate significant man-days of temporary employment opportunities that constitute unskilled person power and skilled person power including engineers, technicians and operators. People will be required for excavation, concrete work, road work, transportation, supervision and management during the construction of this project and local people based on their qualifications and skills will have additional income opportunities.

Enhancement of Technical Skills

This project not only provides employment opportunities to the poor and unskilled but also supports the transfer of skills and technical knowledge on how to work in similar construction works. These skills will encourage the locals in getting long term employment opportunities. Workers will acquire the additional knowledge through training in material handling, occupational safety, general environment, health and social precautionary measures.

- **Beneficial Impacts during Operation Phase**

Reduced Traffic congestion at existing bridge

Traffic congestion in Peradeniya Bridge has become a major issue due to high concentrations of vehicles on the bridge in the morning and evening peak hours. After the implementation of the project, the time and cost of travelling/transportation will also be reduced significantly. And also air quality will be improved.

Increment on Capacity of A1and A5 road sections

Existing A1 and A5 road have two lanes which will not be adequate with the implementation of the project. Thus, these roads are proposed to be widen to four lanes. So, A1 and A5 road capacity will be increase.

Reduction of Accidents and traffic conflicts

A traffic signal system will be installed at the adjacent junctions. It will reduce the traffic conflicts, accidents and increase smooth flow.

Safe movement for pedestrians

Pedestrian will be not allowed to cross on the bridges and the new implemented underpass system will be used for pedestrian crossing. This will provide safe movement for pedestrians. Traffic flow will not be interrupted by pedestrians. Disabled person can move easily with using of ramps.

Adequate walking space for pedestrians along the A1 road

1.5m width of walkway will be constructed on the both sides of A5 road. It will not adequate for the safe and comfortable movements of pedestrian.

Adverse Impacts

The adverse impacts during construction, subsequent operation and maintenance in terms of physical, socioeconomic, cultural and religious aspects due to project actions are evaluated as follows,

- **Pre-Construction Phase**

➢ Removal of Vegetation due to Site Clearance

The project requires clearing of trees and ground vegetation when the following activities are going on. Peradeniya is known for its greenery, however, not many vegetation is present in the Peradeniya town. Furthermore, there is no aquatic vegetation within the Peradeniya town alongside roads

- ➢ Construction of new road
- ➢ Construction of abutment and working site

Mitigation measures- There will be enough space (Figure 6.2) for planting new trees after the project completion.

Figure 6.2 Tree planting areas

➢ Demolition of the shops for widening of road

About 120 shops are established in Peradeniya town. No Hospitals or Schools are present within the project boundary. A Buddhist shrine and Bodhi tree and Jumma Muslim Mosque are located in Peradeniya town. So, buildings will be demolished in the area of 5m from both edges of A1and A5 road.

Mitigation measures- All the shops will be relocate in the new shopping complex (Figure 6.3) at mitigated landslide area. Completely demolished shops will be shifted in to the shopping complex. Others will get compensation for damage.

Figure 6.3 Proposed shopping complex

- **Slope stability**

Traffic in the widened road will affect the stability left river bank soil.

Mitigation measures- left river bank will be protected using a vegetated layer with suitable rip rap protection to reduce the erosion and gain the aesthetic view for the area.

- **Material transport and storage**

Emission of dust during stockpiling and transporting construction materials that are Inconvenience to road users.

Mitigation measures -All construction should be stored with proper cover. Carry out the transport and storage of materials during off traffic off peak hours or at night. Place sign boards at appropriate locations to keep the road users and pedestrians and motorists informed.

- **Construction Phase**

- **Sediment Load in Mahaweli River**

Excavation of sand and aggregates from the river flood plain can generate suspended sediment load in the river. This could affect the downstream channel.

Mitigation measures- Excavated material will be stockpiled for use in filling or reclamation of land during approach road construction. Foundation works will be avoided in rainy season. Blockage of river flow will be cleared immediately. Adoption of method of pumping mud slurry to avoid sediment load discharge in the river.

- **Construction of Bridge Superstructure**

Completion of the bridge superstructure would include piers, columns, bridge deck, roadway finishes, lighting, and the shared use path. Much of the material would be pre-fabricated at various locations and delivered to the project site. Working area may not adequate for handling the bridge elements.

Mitigation measure- temporary working platform will be built at the site. Bridge elements would be lifted into place by cranes operating on barges.

- **Traffic diversion**

Improper traffic handling during the construction (new road and underpass construction) may cause Delays in transportation of goods and passengers, Traffic congestion.

Mitigation measures

- New road construction- Traffic flow will be affected by the new road construction at the intersection between the new road and A1 road. Thus, two direction Traffic will be allowed to move through two lane road (A1). And all the constructions has to be taken place during night to minimize the disturbances.
- Under pass construction- One part of the under pass (along the Road) can construct at the end of the new bridge construction. Then, allowing the traffic to pass through new bridge and

remaining part can construct. Perpendicular (to the A5 road) under pass part can be constructed during the widening of the road section.

- Warn road users about traffic diversion by using signs and appoint labours to control the traffic movements.

6.1.4 IMPACT ON AIR QUALITY, WATER QUALITY AND NOISE LEVEL

- **Air Pollution:** Diesel generator, machinery equipment, excavators, drillers, dozers and transportation vehicles may introduce fugitive & combustion emissions into the atmosphere. Dust emission can be expected to be high along approach road and of A5 road (Widening). Smoke & Dust will also affect the road/bridge site, vegetation, local people and workers.

Mitigation measures- To mitigate the air pollution and its effect the following measures will be carried out:

- Construction equipment and vehicles will be regularly examined and maintained in proper condition.
- Water will be sprinkled along the access road at least two times a day to reduce the dust emission.
- Construction materials will be properly covered during conveyance.
- The speed limit will be enforced for service vehicles.
- Workers will be encouraged to use masks.
- Use Reclaimed asphalt pavement in road construction which are not reused by the industry such as polystyrene, poly ethylene and used auto motive oil can be used as an alternative to formulate bitumen. It will cause reduction in CO_2 emission.

- **Water pollution:** The water quality of the river is in a suitable range however construction and personal activities of the labors may introduce several water pollutants into the river. Probable water pollutants are turbidity, suspended & dissolved solids, fecal contamination, oil & grease from vehicles and equipment, etc. The inappropriate use, storage, processing & application of chemicals for the construction may cause soil & water pollution. Fluid and solid waste resulting from the construction camp may affect the nearby land and water body.

Mitigation measures- The proponent will adopt the following mitigation measures in order to minimize the impact on the water quality;

- Avoiding disposal of the soil, sludge and the other wastes directly into the water body.
- Prohibition of the open urination by workers and provide temporary sanitary facilities during construction.
- Safe storage and the careful use of the chemicals and hazardous substances.

➢ **Noise pollution:** The present noise level of the area is very ambient. During the construction period, operation of machines, excavators, power tiller, movement of transporting vehicles, trucks and construction equipment will increase the existing noise level.

Mitigation measures- The following mitigation measures will be adopted to minimize the noise pollution:

- Concrete mixer, including other construction equipment etc. will be maintained in proper condition by applying grease and lubricants.
- Earplug will be provided to the worker involved in high noise equipment operations.

- **Operation Phase**

The following mitigation measures will give good quality during the operation stage and avoid unwanted impacts.

➢ Speed Limit signs will be used adjacent to the approach Road and Bridge.
➢ Prohibition of construction of any kind of permanent structure within the Right of way.
➢ Road and bridge will be maintained frequently.
➢ Introduce legal requirements for proper maintenance of vehicles.
➢ Provide one lane as bus lane for peak hour to avoid delay of Public transportation users. So, private vehicle users will tend to use public transportation.

6.1.5 SUMMARY OF ENVIRONMENTAL IMPACTS AND SIGNIFICANCE OF THE IMPACTS

Table 6.1 Environmental impacts and significance of the impacts

Potential environmental impacts	**Yes**	**No**	**Significance of the impact**
Construction and operation of the Project involves actions which will cause physical changes in the local environment.	Yes		Town layout will be majorly altered. However, providing better designed buildings, pavements, improved roads and other infrastructure facilities will have a High positive impact.
Project involves use, storage, transport, handling or production of substances or materials which could be harmful to human health or the environment.	Yes		Transport of material and construction activities will emit small amounts of dust, and fugitive particles.
Project produces solid wastes during construction or operation.	Yes		Construction debris (broken concrete blocks, material packaging, some amounts of top soil etc.) which will need to be removed from the site.
Project may release pollutants or any hazardous, toxic or noxious substances to air	Yes		There will be bituminous material used during overlay and re-surfacing of roads, bituminous patching, crack sealing, carriageway edges and shoulder repairs. Paints and solvents used for road markings can emit toxic and noxious air-borne substances.

Project may cause noise and vibration or release of light, heat energy or electromagnetic radiation	Yes		There may be a high noise level increase during demolishing of buildings. However, once the project has been completed the noise level will reduce due to the easing of traffic congestion.
Project may lead to risks of contamination of land/water from releases of pollutants onto ground or into surface waters, groundwater or coastal waters.	Yes		Stock piling of material may lead to washing away of soil and may increase turbidity and TSS in road side drainage temporarily during the construction.
Project may cause localized flooding and poor drainage during construction.		No	Unlikely.
There are transport routes on or around the location which are susceptible to congestion which could be affected by the project	Yes		During the improvement of the road stretch, demolition of buildings in Peradeniya town a significant increase of traffic congestion may be experienced. However, once the project is over a huge positive impact could be expected.
Areas or features of high landscape or scenic value on or around the location may be affected by the project.		No	Landscape and scenic value will be improved after the project.
Project may cause the removal of trees in the locality.	Yes		Several large trees might have to be removed. However, trees will be planted along the river banks and filling area.

Areas or features of historic or cultural importance on or around the location may affected by the project.	Yes		Buddhist Shrine at Peradeniya and Jumma Muslim Mosque will be altered.
Existing land uses on or around the location e.g. homes, gardens, other private property, industry, commerce, recreation, public open space, community facilities, agriculture, which could be affected by the project.	Yes		Existing land use pattern will be disturbed during construction. However, it will be altered in a positive manner once the project is completed.
Any areas on or around the location which are densely populated or built-up may be affected by the project.	Yes		Movement of people may be slightly hindered during construction. However, it will be altered in a positive manner once the project is completed.
Major relocation of established shops, people due to the project.	Yes		Will have a significant impact on the lives of people already established in Peradeniya town. They will have to be compensated and/or relocated.
Environmentally sensitive areas (ex: Water courses, Wetlands) may be affected by the project.		No	Mahaweli river is an important environmentally sensitive area therefore the project will not be utilizing the river reservation.
Areas on or around the location which are used by protected, important or sensitive species of fauna or flora e.g. for breeding, nesting, foraging, resting, migration, may be affected by the project.		No	No protected one is found in the project area.

6.1.6 SUMMARY OF MITIGATION MEASURES OF THE ENVIRONMENTAL IMPACTS

Table 6.2 Mitigation measures of the environmental impacts

Project Activities	Potential Environmental impacts	Mitigation measures
Removal of pavements, debris, demolition waste	Physical: Excavations and trenching for construction activities will disrupt public and road users, cause inconvenience due to generation of noise, vibrations, dust, and temporarily blocking access to certain areas.	Physical: Sprinkling of water, use of hydraulically driven machines instead of pneumatically driven ones are necessary to be practiced whenever possible.
	Transportation of construction material during working hours may disrupt traffic, cause inconvenience to pedestrians and commuters.	Construction material transport should be restricted to only during non-peak hours or night time as practical as possible.
	Social: Inconvenience to pedestrians and motorists	Social: Carry out such removal of pavements during off traffic peak hours. Locate sign boards at appropriate locations to keep the pedestrians and motorists informed.
	During the period of construction, roadside parking may not be possible both due to ongoing construction activities.	Motorist must be informed well in advance of the non-availability of parking slots for the period intended for construction work. People must be informed of the activities so that they can take alternative routes.

Material transport and storage	Physical: Emission of dust due to stockpiling and transport. Social: Inconvenience to users of the road and pedestrians.	Physical: All construction materials (sand, soil gravel, aggregates, cement, and bituminous products) should be stored with proper cover. Social: Carry out the transport and storage of materials during off traffic peak hours or in the night. Locate sign boards at appropriate location to keep the road users and pedestrians and motorists informed of material piles.
Re-surfacing of the road network	Social: Creating traffic congestion, and thus inconvenience to the users of the park, pedestrians and motorists	Social: Carry out overlay during night time which will cause minimum disturbance. If day time work has to be carried out, prepare a good road de-tour system in consultation with the police and provide adequate sign boards at appropriate locations to keep both the motorists and pedestrians informed of the project activity.
Demolition and construction of buildings and Underpass	Physical: Elevated noise levels and emission of dust. Social: Inconvenience and safety issues due to falling debris to users of the road and pedestrians.	Physical: Care should be taken that the negative impacts are kept at a minimum. Social: Locate sign boards at appropriate location to keep the road users and pedestrians and motorists informed.

Relocation	Social: Major disturbance to the lifestyles of people established at Peradeniya Town.	Social: Should be adequately compensated and/or relocated to a satisfactory area.
Rearrangement of bus bays and parking bays	Social: Traffic congestion could be heightened during the activity.	Social: Carry out such activities during off traffic peak hours, preferably at night. Inform bus drivers, pedestrians and passengers of the new arrangement. Arrange sign boards at appropriate locations.

6.2 URBAN DEVELOPMENT GUIDELINES

Importance in the Road Network

The importance of bridges in the road network was evaluated based on road's functional classifications, high traffic volume, and no alternative route. Bridges on Class -A & -B roads were regarded as highly important. The new proposed bridge is important to connecting with A5 and A1 roads. Also, the presence of high traffic volume in the road implies, it's a significant importance to social and economic activities in the area. When such a road is closed for traffic, no alternative route for such a road can be found in the region.

Road Width

Due to the increment of private vehicle ownership, the demand for road space has been increased in the recent past causing more and more congestion on roads. Existing narrow bridges will be the bottlenecks in the road network, and may give adverse effects on social and economic activities nationwide. Both Class -A and -B roads are classified as national roads, and it is recommended the bridges meet the requirements of 3.5m wide traffic lane and required the number of lanes, depending on the PCU basis traffic volume.

Soundness on Existing Bridge

Visual inspection has to be conducted with careful attention to the damage to primary structural members, which may affect the overall structure. Structural conditions were evaluated with the following criteria.

Good: No defect / minor damage is found.

Fair: It may have a small loss of member section, deterioration, crack, scaling and scour. No effect on the overall structure is expected.

Poor: Progress of loss of member section, deterioration, crack, scaling, and scour is observed. Serious damage such as loss of member section, deterioration, crack, scaling, and scour, which may affect the overall structure, is observed. It needs urgent measures as repair or reinforcement works.

Critical: Progress of serious damage on the primary structural member is observed. Imminent failure is expected. It shall be closed to traffic immediately.

Inundated or Insufficient Water Opening

Where erosion of riverbank near the bridge or scouring around bridge piers is observed, bridge span/length shall be increased to provide sufficient water opening. The bridge is generally constructed at the narrow river segment in order to achieve the economic benefit for its short length. It will expedite the riverbank erosion and scouring, thereby resulting in bridge collapse.

Insufficient freeboard under the bridge deck soffit will also be a problem during floods. The collision of debris to the bridge deck will damage the bridge structures if no sufficient freeboard is provided.

Classification of Bridge Replacement / Reconstruction

Through the selection procedure, bridge replacement/reconstruction is categorized into two (2) types: full-width new bridge construction and half-width parallel bridge, due to the soundness of existing bridge structure and required curb-to-curb width/number of traffic lanes.

Where the existing bridge is structurally not sound and needs replacement, the “full-width” new bridge shall be constructed irrespective of the current width/number of traffic lanes. Structurally sound bridges with insufficient width/number of traffic lanes will need a “half-width” parallel bridge.

Adopted Standards

The Geometric Design Standard of Road 1998 has been applied for the geometric design of roads and the Euro code has been applied for the design of the bridge.

Consideration of Future Maintenance

The bridge has been designed in consideration of minimum and easy maintenance. Bridge drains, expansion joints, and shoes have been designed considering easy cleaning and durability. Concrete design strength, which should be durable against water, has been determined and designed for the quality of cement, aggregate, sand, and their design mix proportion.

Geological Features

Stone layers are generally considered as the bearing layer for the bridge. However, the bedrock layer is selected as the bearing layer for pile bents of pier foundation and cast-in-situ concrete bored piles for abutment foundation to support the large load of the bridge without a settlement.

Land Acquisition and Resettlement

Resettlement- the National Involuntary Resettlement Policy (NIRP) stipulates that a comprehensive RAP (Resettlement Action Plan) be prepared where 20 or more families are displaced. In a case where less than 20 families are displaced, the NIRP still requires an RP with a lesser level of detail. NIRP applies to all projects irrespective of the source of funding.

Land Acquisition- the Land Acquisition Act (LAA), 1950 makes provisions for acquisition of the Lands and Servitudes for public purposes and provides for matters connected with or incidental to such provision. It provides the payment of compensation at market rates for lands, structures, and crops.

Restrictions due to Environmental Considerations

Bridges are subjected to social and environmental act/ordinance/regulations for reconstruction, replacement, and widening, and need Initial Environmental Examination (IEE), Environmental Impact Assessment (EIA), and/or Resettlement Action Plan (RAP).

6.3 SAFETY CONCERN AND CONSTRUCTION CONSIDERATION

The purpose of Construction Health and Safety Concern is to assign responsibilities, establish project personnel and community protection standards and procedures, and to plan for contingencies that may arise during construction. This is intended to minimize health and safety risks associated with the known and potential hazards at the site to protect workers and the surrounding community. Moreover, the following benefits can be achieved due to the best safety plan.

- Decreases the chances for project delays
- Decreases the possibility of injuries
- Increases the potential for success
- Increases the confidence of team members

As part of risk management, safety planning is used to:

- Ensure worker's protection
- Anticipate possible dangerous situations and hazards
- Guide the evaluation of the safety conditions of the project environment
- Determine the minimum requirements, equipment or tools needed to perform specific activities
- Meet or exceed the legal obligation for safety and health conditions in the work environment.

By focusing on inputs, tools, techniques, and outputs, project managers can implement an efficient safety plan with team feedback as a critical element.

6.3.1 SAFETY PLAN TOOLS AND TECHNIQUES

Training

The project team must be alerted to the safety concerns related to the project and the working area that focus on injury/illness prevention training. Every time new members are appointed to the project, they must be trained in the safety and security issues related to their specific tasks.

Observation, Inspection, Interview, and Analysis

Before executing specific tasks, the team must be aware of certain circumstances or combinations of events that may lead to dangerous situations. In some cases, interviewing other people in the area could present historic information related to the area or task.

Safety Committees

Periodically, the project team must discuss relevant topics related to its safety and security to update the safety plan. A committee dedicated to those concerns ensures that the necessary communications take place. In addition, a technical expert in safety can enhance the safe execution of the project.

Hazard evaluation

The chemical hazards will be minimized by limiting exposure of personnel to hazardous conditions through air monitoring, the use of personal protective equipment (PPE), and application of mitigation controls if warranted. Dust suppression controls, such as water misting, will be used as necessary to limit exposure to airborne particulates. Mechanical venting equipment will be required to be on hand to vent excavations as warranted based on the real-time air monitoring results.

Following physical hazards present during the construction. They are slip, fall hazards, Noise hazard, Environmental stress, moving vehicles, and use of heavy equipment. The site will be kept neat and free of clutter to protect against trips and falls. Site personnel will be briefed at each safety meeting. The perimeter of the work zone shall be secured with construction barriers such that pedestrians and public traffic will have safe access around the zone, and thus, project workers will be protected from the moving vehicles. Hearing protection will be used always around loud equipment.

Health and safety officer

The Contractor will be required to designate a Site Safety Officer. The safety officer will be a competent person responsible for implementing this safety plan. The Safety officer will have a stop-work authorization, which he/she will execute on his/her determination of an imminent safety hazard, emergency, or other potentially dangerous situation. If the Safety Officer is absent from the site for any reason, he/she will designate a suitably qualified replacement that is familiar with the requirements of the safety plan.

The Safety Officer or their designees are responsible for the following:

- Monitoring to determine the degree of hazard.
- Establish site work zones.
- Ensure that all personnel in the work zone are wearing proper hearing protection.

- Determining the protection levels and equipment required to ensure the safety of personnel.
- Evaluating on-site conditions (i.e., weather and chemical hazard information) and recommending to the project manager and/or the field coordinator, modifications to the work plan, and personal protection level.
- Monitoring performance of all personnel to ensure compliance with the required safety procedures.
- Notifying emergency authorities (police, fire, and ambulance) of the team's presence, assignments, and emergency procedures (as required).
- Conducting daily briefings as necessary.
- Halting work if necessary.
- Ensuring strict adherence to the safety plan.
- Reviewing personnel medical monitoring participation and health and safety training.

General work practices

The following general safety rules will be followed to increase the level of safety at the site:

- Work boots, hard hats, and reflective orange vests must be worn at all times.
- Remove slipping, tripping, or falling hazards from paths.
- During work executions, pause every few minutes and assess the surrounding traffic conditions. The Safety officer will serve as a "spotter," to the maximum extent possible, keeping a lookout throughout field activities.
- Hearing protection will be used during the operation of loud machinery.
- When walking on right-of-ways or road-shoulders, keep a sharp lookout in both directions.
- Be sure that the appropriate roadway safety equipment is on-site including road flares, reflective traffic cones, flags, etc.
- Be cognizant of surroundings and ensure that equipment is properly secured.
- All personnel who participate in field activities will be required to attend a Health and Safety meeting before the commencement of field activities.
- Eating, drinking, and smoking in the work area are prohibited.
- Dermal contact with soil and groundwater should be avoided. This includes avoiding walking through puddles, pools, mud, sitting, or leaning on or against drums, equipment, or on the ground. Site personnel should wash their hands before eating, smoking, using the toilet, etc.

Site personnel should wash their hands, face, and shower (daily) as soon as possible after leaving the Site.

Storage hazards include lack of storage planning, haphazard storage, and unstable pre-cast elements due to inadequate supports, insufficient ground support strength, and exceeding stack stability height. The storage area should be reasonably level, hard-surfaced, and large enough for pre-cast components to be stored properly with adequate room for lifting equipment and maneuvering trucks, cranes, or relevant vehicles.

Lifting hazards: Lifting is considered as the most critical life-threatening daily activity in the site as the pre-cast elements need to be lifted several times during production using overhead cranes, mobile cranes, and tower cranes. Extensive care should be taken with regards to the following:

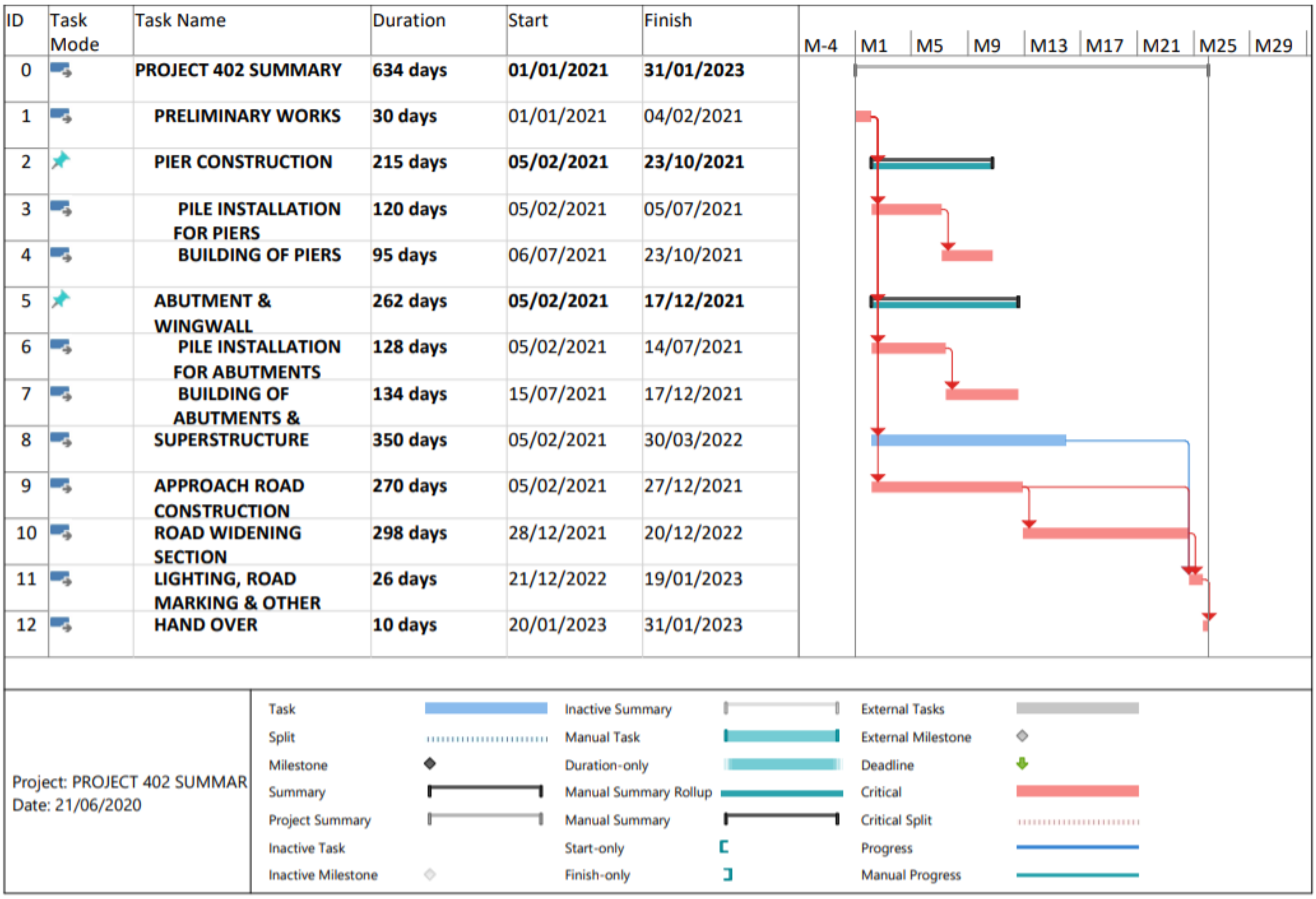
- Permit to Work should be valid on site and attached to it the approved method statement in addition to all rigging studies for the worst/different cases of lifting operations that will be carried on.
- Failure of lifting machines/defective lifting gears: Daily, all cranes/machines in addition to lifting gears should be inspected before starting any activity using an approved checklist.
- Avoid overloading of cranes or trailers. Always follow the Safe Working Load.
- Area Evacuation: Always make sure the area around the lifting/rotation radius is evacuated.
- Ensure the elements are being secured with safety tension belts on trailers before transportation to the site.

General Trenching and Excavation Rules:

- Keep heavy equipment away from trench edges.
- Keep surcharge loads at least 2 feet (0.6 meters) from trench edges.
- Know where underground utilities are located.
- Inspect trenches at the start of each shift.

6.4 CONSTRUCTION PLAN

The construction plan was created for the project using MS Project software. The total duration calculated assuming working 8 hour per day and 6 days per week using RDA standard rates. Considering delay due to rainy seasons the duration was adjusted. The total working days were found as 634 days and project duration was found as 25 months. The construction plan is given in Appendix J.



ID	Task Mode	Task Name	Duration	Start	Finish
0		**PROJECT 402 SUMMARY**	**634 days**	**01/01/2021**	**31/01/2023**
1		**PRELIMINARY WORKS**	**30 days**	01/01/2021	04/02/2021
2		**PIER CONSTRUCTION**	**215 days**	**05/02/2021**	**23/10/2021**
3		**PILE INSTALLATION FOR PIERS**	**120 days**	05/02/2021	05/07/2021
4		**BUILDING OF PIERS**	**95 days**	06/07/2021	23/10/2021
5		**ABUTMENT & WINGWALL**	**262 days**	**05/02/2021**	**17/12/2021**
6		**PILE INSTALLATION FOR ABUTMENTS**	**128 days**	05/02/2021	14/07/2021
7		**BUILDING OF ABUTMENTS &**	**134 days**	15/07/2021	17/12/2021
8		**SUPERSTRUCTURE**	**350 days**	05/02/2021	30/03/2022
9		**APPROACH ROAD CONSTRUCTION**	**270 days**	05/02/2021	27/12/2021
10		**ROAD WIDENING SECTION**	**298 days**	28/12/2021	20/12/2022
11		**LIGHTING, ROAD MARKING & OTHER**	**26 days**	21/12/2022	19/01/2023
12		**HAND OVER**	**10 days**	20/01/2023	31/01/2023

Figure 6.4 Construction plan for the project

6.5 SUMMARY OF BOQ

Table 6.3 Summary of BOQ

Project : New Bridge for Peradeniya

Contract sum analysis

Bill No	Description	Amount / Rs. Cts.
01	Preliminary works	15,589,200.00
02	Abutments & wingwalls	57,249,511.62
03	Piers	52,420,801.76
04	Superstructure	30,313,308.83
05	Road widening & Slope protection	115,863,435.94
06	New road section	19,213,979.70
SUB TOTAL		290,650,237.84
CONTRACT PRICE WITHOUT VAT		290,650,237.84
VAT (15%)		43,597,535.68
CONTRACT PRICE WITH VAT		**334,247,773.51**

The detailed BOQ is given in Appendix K

CHAPTER 7

DISCUSSION

The existing Peradeniya Bridge is one of the most important bridges on Kandy - Colombo, and Kandy – Gampola roads. It is a 68 m long, two Lane Bridge which is a key link for transportation in A1 and A5 roads. Current traffic congestion in Peradeniya Bridge has become a major issue due to high concentrations of vehicles on the bridge in the morning and evening peak hours. In addition, as further traffic congestion is expected from the new expressway connecting Kandy and Colombo, congestion reduction has become an urgent issue. Considering the above issues, a Basic Design Study on the Project for the Construction of New Peradeniya Bridge was conducted addressing the major issues arising with the new project.

In this multi-disciplinary design project, a new bridge parallel to the existing bridge was designed to convey half of the traffic volume of the existing bridge. The existing A1 and A5 roads were widened to four lane roads to minimize traffic congestion. An approach road was designed to transfer the traffic to the new bridge. A traffic signal system design was done to have a proper traffic control system in the Peradeniya junction. Moreover, the riverbank slopes were protected using a vegetated layer to carry the additional load from the widened road. On the other hand, the valley area between the existing road and the new approach roads were designed to filled providing adequate drainage facilities.

Thereafter, an EIA study was conducted to measure major impacts on the environment due to this project and according to the EIA some mitigation measures were suggested to follow, so that the negative impacts due to the project can be eliminated. Finally, the cost for the project was estimated and that was around 334 million rupees. Finally, a construction plan was created and the time for construction completion was calculated.

Consequently, for future improvement we would like to suggest replacing the traffic light system with a smart traffic lighting system which can adapt to ongoing traffic conditions, Construct a vertical car parking system next to the shopping complex and widening of A1 road from Gannoruwa junction Pilimatalawa town.

In conclusion, this design was created to reduce the traffic congestion and to improve the commuter’s comfort and transform Peradeniya to a more aesthetic place.

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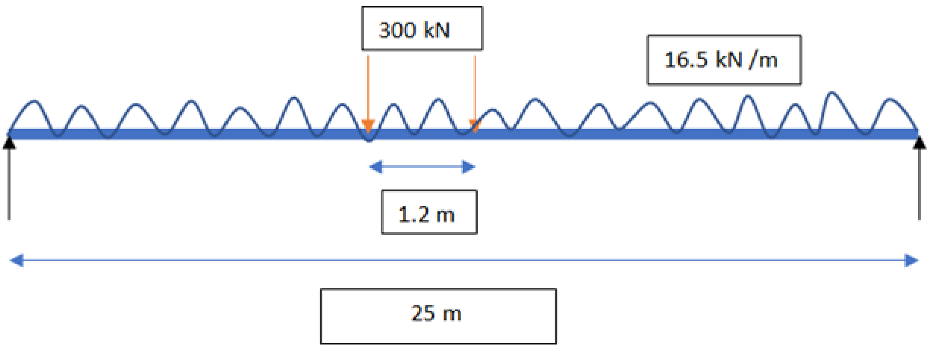
APPENDIX A WORK DISTRIBUTION

Reg. No.	Name	Component of work carried out
E/14/045	BRANAVAN K.	Traffic calculation, Pavement design, Road design, EIA of road construction, Drawings of road structure
E/14/082	DINELKA K.H.S.	Pier design with pile foundation, creation of longitudinal profile and complete BOQ for piers , construction plan , Complete BOQ
E/14/170	KALABAN P.	Signal design , Underpass, EIA , construction plan for road section
E/14/187	KUMARI R.D.N.D.	Slope design, Road widening, Abutment design
E/14/239	PATHIRANA A.P.U.M.	Sketches of alternate solutions, Site layout, DEM of the area , Traffic Simulation , Final Drawings , Final video modelling
E/14/261	PRIYASHAN H.M.M.	DEM and Contour map design, Abutment design
E/14/316	SENANAYAKE S.M.A.E	Preliminary calculations for the bridge, Superstructure Design, Components in the superstructure , BOQ for the superstructure, EIA summary
E/14/331	SOMASEKARA M.H.Y.S.	Slope design, Bearing design in substructure, BOQ of slope, Road widening and bearing design ,EIA
E/14/344	THANIKARUBAN T	Road design & Drawings for Road structure parts

APPENDIX B DESIGN OF BRIDGE LOADS

REFERENCE	CALCULATIONS	RESULTS
EN 1991-2:2003	**B.1 Vertical forces on the carriageway** **gr1a - Characteristic LM1 (TS and UDL)** Maximum loading occurs on notional lane 1, Tandem system (TS) $= \alpha_Q * Q_{1k}$ $= 1.0*300$ $=300$ kN Uniformly Distributed Load $= \alpha_q * q_{ik}*$notional lane width $= 0.61*9*3$ $= 16.5$ kN / m  **Figure B.1 gr1a loading**	LM1, TS = 300 kN UDL = 16.5 kN/m

Following results were obtained by SAP 2000 for gr1a load group,

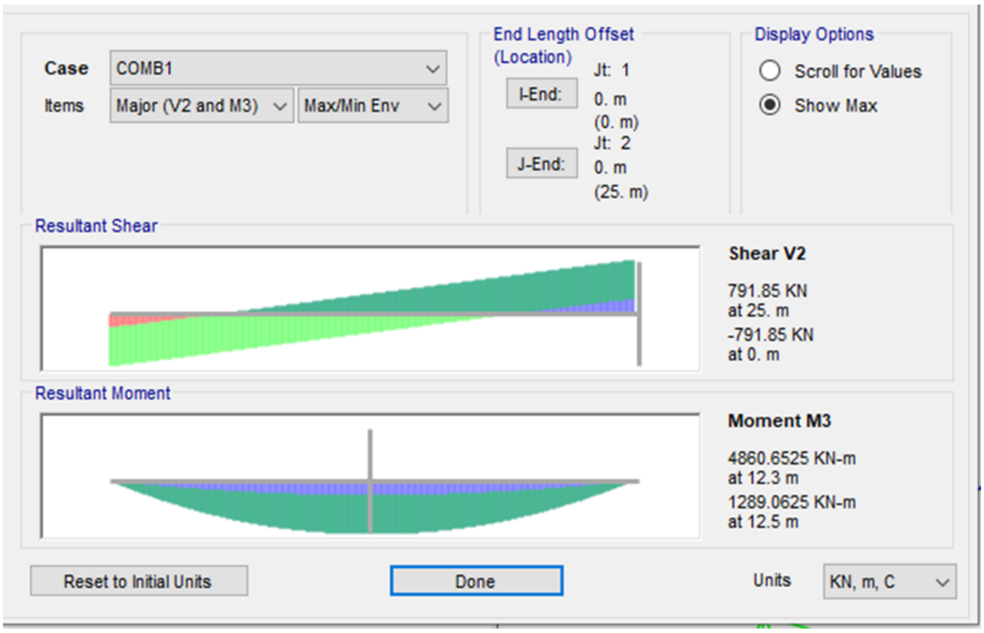


Figure B.2 Results for gr1a

Maximum Bending Moment

= 4860.65 kNm per 3m lane width occurring at 12.3 m from left end of the beam

Maximum shear force

= 791.85 kN per 3m lane width occurring at the both ends of the beam.

BM= 4860.65 kNm

SF= 791.85 kN

.

gr5 – Frequent LM1 (TS and UDL) + Characteristic LM3 (Special vehicles)

For a special vehicle, SV80 was selected. That means the weight of the maximum special vehicle is 80 tons for this bridge.

Frequent LM1,

Tandem system (TS)

= 0.75*αQ * Q1k

= 1.0*300*0.75

=225 kN

LM1,
TS = 225 kN

Uniformly Distributed Load

= 0.75* αq *qik *notional lane width

=0.75*0.61*9*3

= 12.375 kN / m

UDL = 12.372 kN/m

Characteristic LM3,

LM3,

Axle load	= 130 kN	
DAF	= 1.16	
Factored axle load	=1.16*130 = 150.8 Kn	

SV80 =150.8 kN

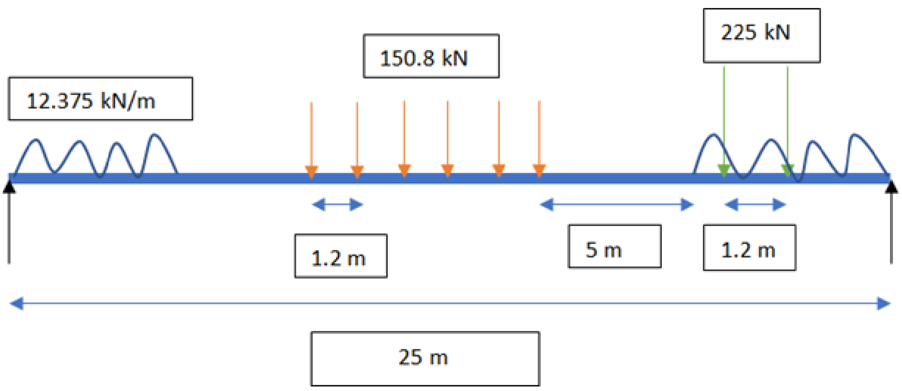


Figure B.3 gr5 loading

Following results were obtained using SAP 2000,

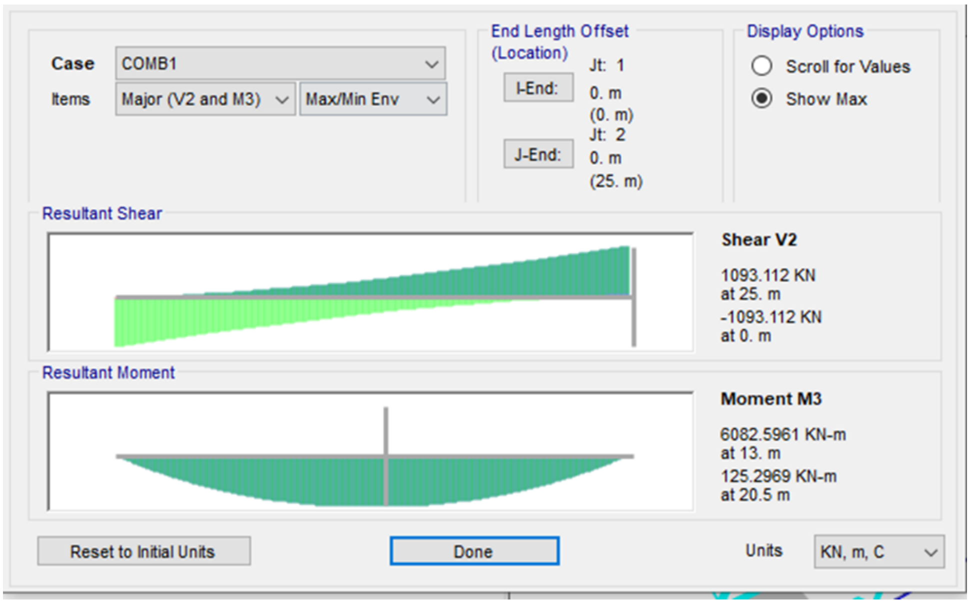


Figure B.4 Results of gr5

Max BM = 6082.59 kNm per 3m at 13.0 m

= 2027.53 kNm /m

Max shear = 1093.11 kN per 3m at 25 m

= 364.37 kN/m

Considering both load groups

Design BM = 2027.53 kNm /m

Design shear force = 364.37 kN/m

BM = 6082.59 kNm = 2027.53 kNm/m

SF= 1090.11 kN = 364.37 kN/m

B.2 Horizontal force of the carriageway calculation

gr2 – (LM1) Axial (Breaking + Acceleration) Forces

Longitudinal Breaking force $Q_{lk} = 0.6x\ \alpha_{Q1}*2Q_{1k} + 0.1*\alpha_{q1}*q_{1k}*w_1*L$

$= 0.6*1*2*300 + 0.1*1*9*3*25$ (180 kN<= Q_{lk} >=900 kN)

= 427.50 kN

Q_{lk} = 427.50 kN

Longitudinal Acceleration force = 427.50 kN (opposite direction of the Q_{lk})

Lateral forces on bridge deck

= 50% * Q_{lk}

= 0.5*427.50

= 213.75 kN

Acce. = 213.75 kN

gr6 – (LM3 -SV80) Axial (Breaking + Acceleration) Forces

Longitudinal Breaking force Q_{lk} = $Q_{lk,s}$* No: of axels*DAF

= δ * w*6*1.16

= 0.5*130*6*1.16

= 452.4 kN

Q_{lk}= 452.4 kN

Longitudinal Acceleration force = 10% * gross weight of the vehicle* No: of axels

= 0.1*80*9.81*6

= 470.88 kN

Acce. = 470.88 kN

Lateral forces on bridge deck = 50% * Q_{lk}

= 0.5* 452.4

= 226.2 kN

Lat. = 226.2

Considering both load groups,

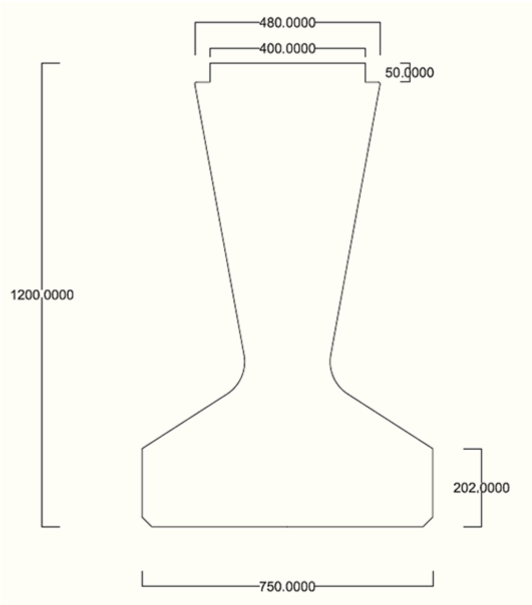
Since bridge has no curvature no centrifugal forces are acting on the bridge.

Design Longitudinal Breaking force Q_{lk} = 452.4 kN

Design Longitudinal Acceleration force = 470.88 kN

Design Lateral forces on bridge deck = 226.2 kN

APPENDIX C SUPERSTRUCTURE DESIGN

REFERENCE	CALCULATIONS	RESULTS
EN 1992-2	**SECTION PROPERTIES OF THE COMPOSITE BEAM SECTION** For the superstructure, 25m long Y6 composite prestressed beams were selected.  **Figure C.1 Superstructure design** A 200 mm in situ concrete slab was designed on top of the beams and final composite beam section is shown in figure C.2.	

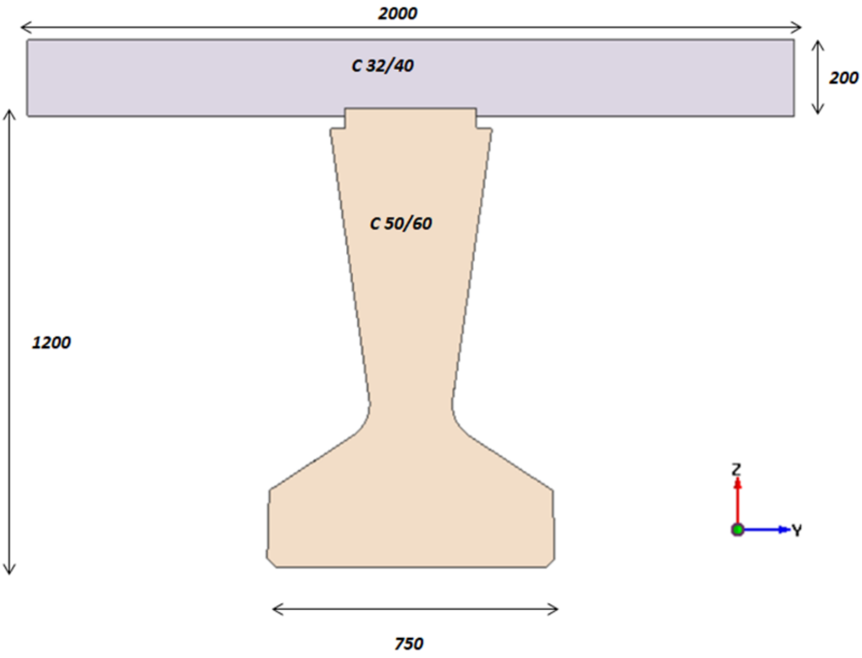


Figure C.2 Cross-section of the composite beam

The composite beam's section properties are given below. In this, element ref. 1 is the prestressed beam and element ref 2 is the slab section.

Overall dimensions	height	= 1.37 m
	width	= 2.0 m
Centroid coordinates	y	= 0.000 mm
	z	= 849.669 mm
Cross section area		= 881721.47 mm²
External surface area		= 7472.7825 mm²/mm
About global centroidal axes:		
Second moment of area	I_{yy}	= 1.9708E11 mm^4
	I_{zz}	= 1.4454E11 mm^4
Section modulus	W_{yt}	$= I_{yy}/(z_{max} - z)$
		= 3.78755E8 mm^3
	W_{yb}	$= I_{yy}/(z_{min} - z)$
		= -2.3195E8 mm^3

Table C.1 Properties of individual elements (about local axes)

Element	Z_{min} To Centroid (m)	Overall height (m)	I_{yy} (mm^4)	I_{zz} (mm^4)
1	0.51491	1.2	7.1097E10	1.1311E10
2	0.10223	0.2	1.25648E9	1.3323E11

Table C.2 Section properties about global axes (through y=0,z=0)

Element	Centroid coordinates Y (m)	Centroid coordinates z (m)	I_{yy} (mm^4)	I_{zz} (mm^4)
1	0	514.917	2.0154E11	1.1311E10
2	0	1272.24	6.3208E11	1.3323E11

Table C.3 Section Weights and Perimeters

Element	Section Area (mm^2)	Weight KN/m
1	491982.34	11.807576
2	389739.13	9.3537391

After selecting the section, tendons were inserted to the sections as per figure C.3. The eccentricities of the strands are given in table C.4.

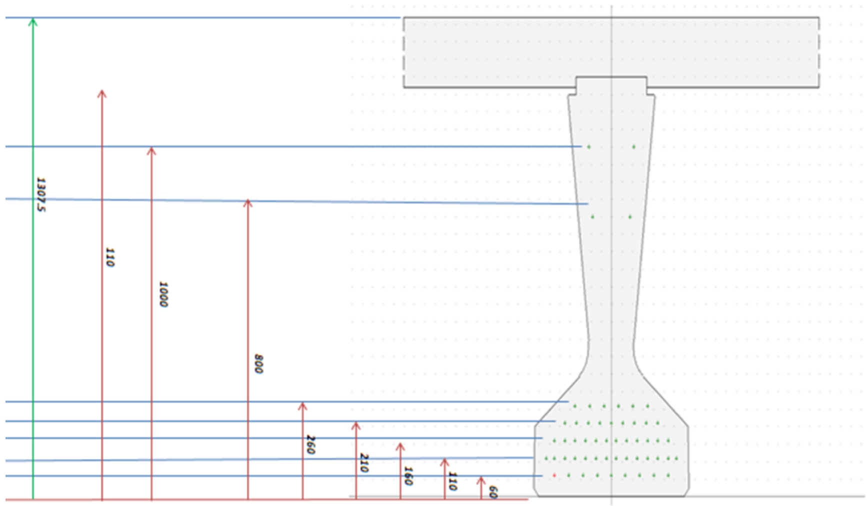


Figure C.3 Tendon profile of the composite beam

Table C.4 Eccentricities of Strands

e	no. of strands (n)	∑ e.n
60	9	540
110	14	1540
160	12	1920
210	9	1890
260	6	1560
800	2	1600
1000	2	2000
∑	54	11050

Overall eccentricity = 11050/54
= 205 mm

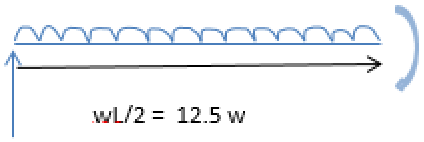
e = 205 mm

Tendon forces have been reduced from 204 kN to 201 kN to represent relaxation loss of 1.25% at transfer.

Table C.5 Parameters for cable profile calculations

Parameter	Value
I (mm^4)	7.11E+10
I' (mm^4)	1.97E+11
Yt (mm)	680
Yb (mm)	514
Y't (mm)	350
Y'b (mm)	850
Zt (mm^3)	1.05E+08
Zb (mm^3)	1.38E+08
Z't (mm^3)	5.63E+08
Z'b (mm^3)	2.32E+08
A1 (m^2)	0.4920
A2 (m^2)	0.3897
Wg1 (kN/m)	11.8076
Wg2 (kN/m)	9.3537
Wq (kN/m)	51.776
Length (m)	25
Mg1 (kNm)	922.4669
Mg2 (kNm)	730.7609
Mq (kNm)	4055.1
Initial force in a tendon (kN)	201
No of strands	54
Po (kN)	10854000
Initial Prestress (MPa)	999.6920
Zt/A (mm)	212.5256
Zb/A (mm)	281.1622
fck (MPa)	50
f'ck (MPa)	36
f'min (MPa)	-1
f'max (MPa)	21.6
fmin (MPa)	0
fmax (MPa)	30

Consider distance x from the edge of the beam,



At a distance x,

M = $wx^2/2 - 12.5\ wx$

= $0.5\ wx^2 - 12.5\ wx$

$$e \leq \frac{Z_t}{A} - \frac{f'_{min} Z_t}{P_0} + \frac{M_{g1}}{P_0}$$

$$e1 < 212.53 - \frac{(-1) \times 104.56 \times 10^6}{10.854 \times 10^6} + \frac{(12.5 \times 11.81x) - (0.5 \times 11.81 \times x^2)}{10.854 \times 10^6}$$

$$e1 < 222.16 + 13.60x - 0.54x^2$$

$$e \geq -\frac{Z_b}{A} + \frac{f_{min} Z_b}{KP_0} + \frac{M_{g1} + M_{g2} + (Z_b/Z'_b) M_q}{KP_0}$$

$$e4 < -281.16 - 0 + \frac{(12.5 \times 11.81x) - (0.5 \times 11.81 \times x^2)}{0.8 \times 10.854 \times 10^6} + \frac{(12.5 \times 9.36x) - (0.5 \times 9.36 \times x^2)}{0.8 \times 10.854 \times 10^6} + \frac{\frac{138.33}{231.76}((12.5 \times 51.776x) - (0.5 \times 51.776 \times x^2))}{0.8 \times 10.854 \times 10^6}$$

$$e4 \leq -281.16 + 74.95x - 3.00x^2$$

The values for e1 and e4 along the length of the beam are given in table C.6

Table C.6 e1 and e4 values variation with length

Length (m)	**e_1 (mm)**	**e_4 (mm)**
0	-222.16	281.162
1	-235.22	209.212
2	-247.2	143.262
3	-258.1	83.312
4	-267.92	29.362
5	-276.66	-18.588
6	-284.32	-60.538
7	-290.9	-96.488
8	-296.4	-126.438
9	-300.82	-150.388
10	-304.16	-168.338
11	-306.42	-180.288
12	-307.6	-186.238
12.5	-307.785	-186.963
13	-307.7	-186.188
14	-306.72	-180.138
15	-304.66	-168.088
16	-301.52	-150.038
17	-297.3	-125.988
18	-292	-95.938
19	-285.62	-59.888
20	-278.16	-17.838
21	-269.62	30.212
22	-260	84.262
23	-249.3	144.312
24	-237.52	210.362
25	-224.66	282.412

.

After plotting e1 and e3 values along the length of the beam the following, cable profile was achieved.

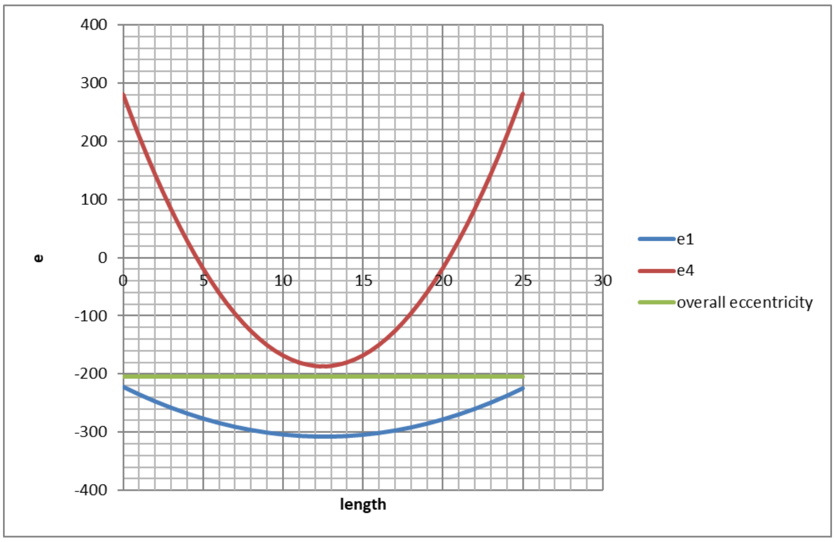


Figure C.4 Cable profile

Since the overall eccentricity is between the e1 and e4 graphs de-bonding is not required.

For the in-situ concrete slab, 25 mm diameter r/f bars are used with a cover of 50 mm from top and corners as per figure C.5.

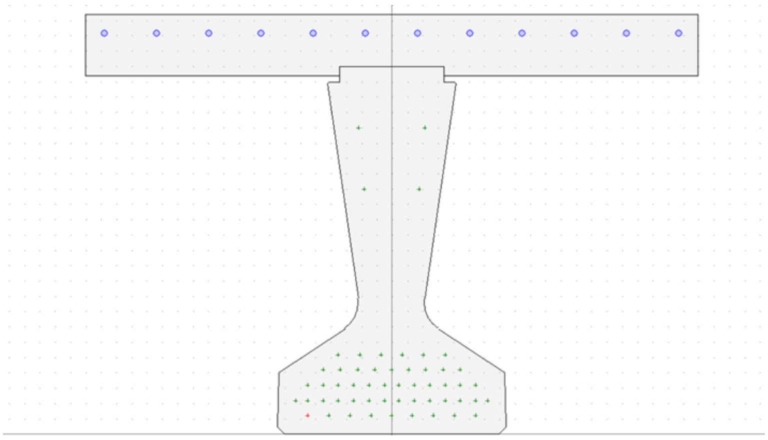


Figure C.5 Tendon and R/F in the composite section

COMPOSITE BEAM DESIGN

The complete beam was designed using Autodesk Structural Bridge design 2019 software.

The self-weight moment was calculated based upon a weight density of 24kN/m3 and applied in the load table. An SLS stress analysis was carried out assuming that the neutral axis remains horizontal, and the elastic modulus was set to the short term modulus. The stress results are the stresses in the concrete taking into account the losses in the tendons due to the elastic deformation of the concrete.

The cross-sectional area is 0.8812 m2 the weight density was assumed as 24kN/m^3 and the length of the beam is 25m, then an M_y bending moment of

M_y =0.8812 x 24 x 25x25/8

= 1652.25 KNm has to be applied.

M_y =1652.25 KNm

On the other hand a variable moment of,

M_v = 2027.53 kNm /m x 2m

= 4055.1 kNm has to be applied.

Mv = 4055.1 kNm

The composite beam is shown in figure C.6. The beam 25 m long and top width is 2m,

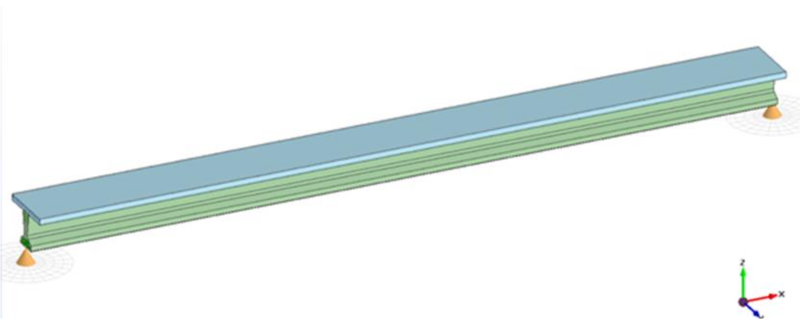


Figure C.6 Composite beam

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Erection loads

During construction, the beam is initially supported on temporary supports at 1m from the beam ends. There is also a temporary load of 1.4kN/m over the length of the beam which represents temporary construction loads and the water in wet concrete. This load and the temporary supports are removed once the concrete has hardened. The erection of beam loads which include two extra components was designed; one for the temp 1kN/m and the other for the support loads (upwards). Figure C.7 Shows the ultimate limit state and serviceability limit state bending moment and shear force diagrams for the beam erection loads.

Table C.7 bending moment and shear force values of Erection loads

Position along span	Moment (kN.m)	Moment (kN.m)	Shear (kN)	Shear (kN)
Dimension (m)	ULS	SLS	ULS	SLS
0	0	0	23.62498	17.49998
2.5	312.6	231.5556	184.944	136.9956
5	717.1651	531.2334	138.708	102.7467
7.5	1006.14	745.289	92.47202	68.49779
10	1179.525	873.7224	46.23601	34.2489
12.5	1237.32	916.5335	0	0
15	1179.525	873.7224	46.23601	34.2489
17.5	1006.14	745.289	92.47202	68.49779
20	717.1651	531.2334	138.708	102.7467
22.5	312.6	231.5556	184.944	136.9956
25	0	0	23.62498	17.49998

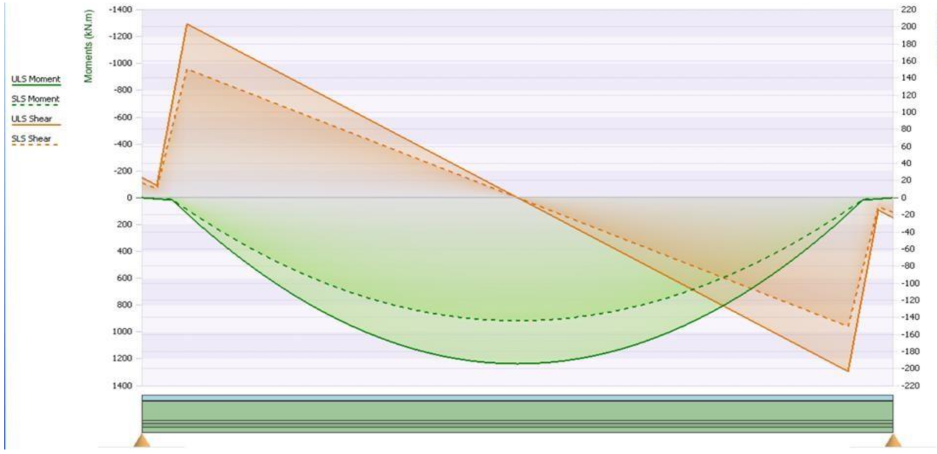


Figure C.7 Beam erection loads

Construction stage 1 Loads

Construction stage loadings are cumulative, and will be applied as follows

Stage 1 - to the pre-cast beam only,

Stage 2 - to the pre-cast and stage 1 composite section

Stage 3 - to the pre-cast, stage 1 and stage 2 composite section

Then, the load effects for construction stage 1 was defined. The UDL intensity of 9.743 kN/m for the self-weight of the slab has to be added as construction loads. The ultimate limit state and serviceability limit state bending moment and shear force values are given in table below. Figure C.8 Shows the ultimate limit state and serviceability limit state bending moment and shear force diagrams for the construction loads applied to the beam.

Table C.8 Temporary Loads and Supports

Position along span	Moment (kN.m)	Moment (kN.m)	Shear (kN)	Shear (kN)
Dimension (m)	ULS	SLS	ULS	SLS
0	0	0	164.4212	121.7935
2.5	369.9477	274.0353	131.537	97.43478
5	657.6848	487.1739	98.65272	73.07609
7.5	863.2113	639.4158	65.76848	48.71739
10	986.5272	730.7609	32.88424	24.3587
12.5	1027.632	761.2092	0	0
15	986.5272	730.7609	32.88424	24.3587
17.5	863.2113	639.4158	65.76848	48.71739
20	657.6848	487.1739	98.65272	73.07609
22.5	369.9477	274.0353	131.537	97.43478
25	0	0	164.4212	121.7935

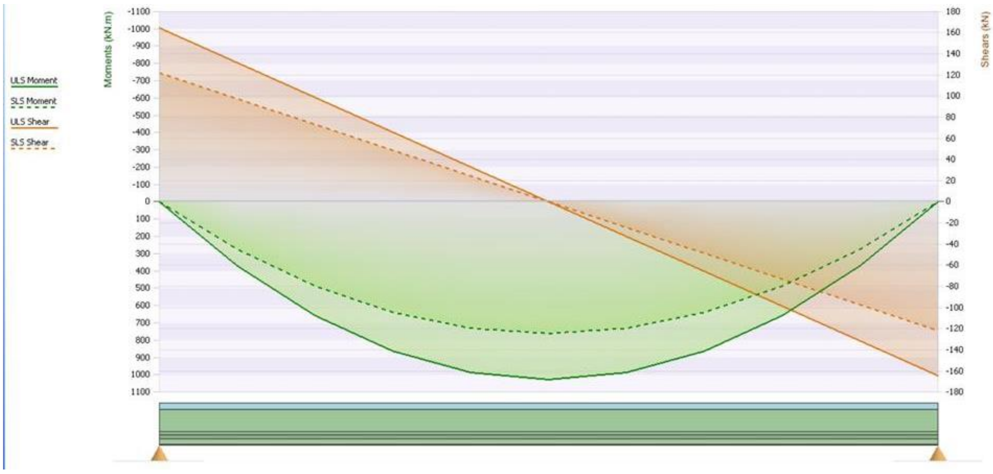


Figure C.8 Construction loads

Temporary Loads and Supports removal

Then, a load case to remove the effects of temporary loads and supports was defined. The ultimate limit state and serviceability limit state bending moment and shear force values are given in table below.

Table C.9 Bending moment and shear force values of Temporary Loads and Supports

Position along span	Moment (kN.m)	Moment (kN.m)	Shear (kN)	Shear (kN)
Dimension (m)	ULS	SLS	ULS	SLS
0	0	0	37.5	31.25
2.5	84.375	70.3125	30	25
5	150	125	22.5	18.75
7.5	196.875	164.0625	15	12.5
10	225	187.5	7.5	6.25
12.5	234.375	195.3125	0	0
15	225	187.5	7.5	6.25
17.5	196.875	164.0625	15	12.5
20	150	125	22.5	18.75
22.5	84.375	70.3125	30	25
25	0	0	37.5	31.25

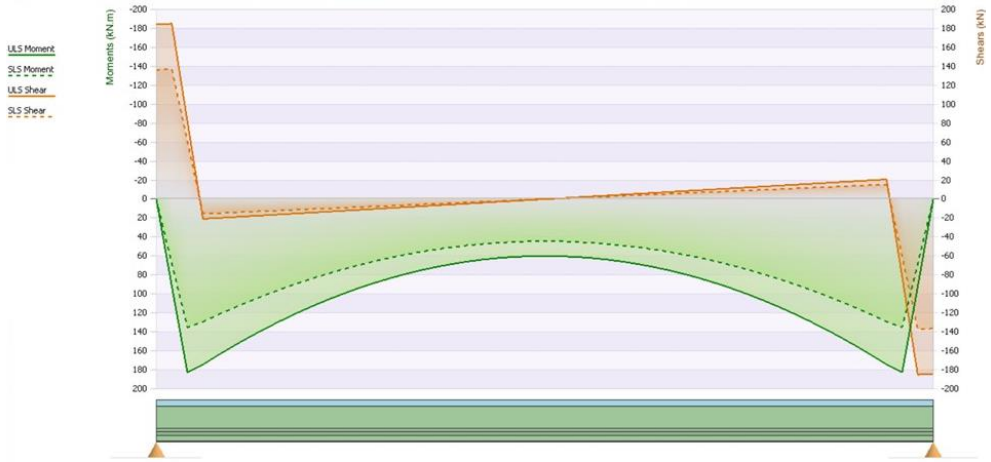


Figure C.9 Removed temp construction loads

Surfacing Loads

Subsequently, the super imposed dead load of 2.5 kN/m is applied to the beam. The ultimate limit state and serviceability limit state bending moment and shear force values were generated and values are given in table below.

Table C.10 Bending moment and shear force values of Surfacing loads

Position along span	Moment (kN.m)	Moment (kN.m)	Shear (kN)	Shear (kN)
Dimension (m)	ULS	SLS	ULS	SLS
0	0	0	37.5	31.25
2.5	84.375	70.3125	30	25
5	150	125	22.5	18.75
7.5	196.875	164.0625	15	12.5
10	225	187.5	7.5	6.25
12.5	234.375	195.3125	0	0
15	225	187.5	7.5	6.25
17.5	196.875	164.0625	15	12.5
20	150	125	22.5	18.75
22.5	84.375	70.3125	30	25
25	0	0	37.5	31.25

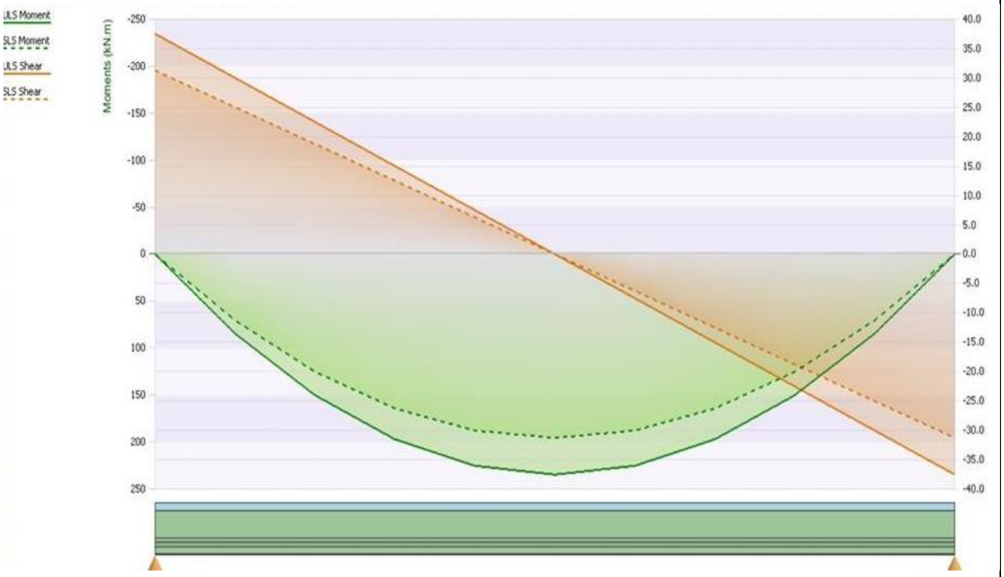


Figure C.10 Super imposed loads

Live loads

gr5 was found as the critical load combination on previous live load analysis therefore, *gr5* load group is used in this beam design. Following table shows the bending moment and shear force values from live loads, applied to the beam.

Table C.11 Bending moment and shear force values of live loads

DISTANCE	BENDING MOMENT / kNm	SHEAR FORCE/ kN	ABSOLUTE MAX SHEAR FORCE/ kN
0	0	-37.1253	728.7413
2.5	1570.271	6.3	617.796
5	2734.539	52.8	510.976
7.5	3526.835	108.576	420.656
10	3963.659	168.896	330.336
12.5	4045.011	240.016	240.016
15	3963.659	330.336	330.336
17.5	3526.835	420.656	420.656
20	2734.539	510.976	510.976

22.5	1570.271	617.796	617.796
25	0	728.7413	728.7413

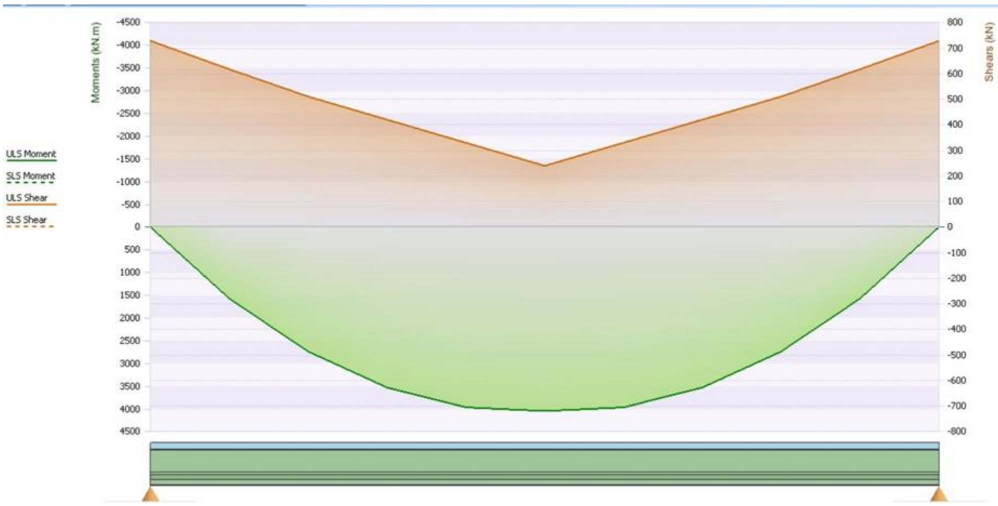


Figure C.11 Live loads according to GR5

Differential Temperature Analysis

A differential temperature profile needs to be defined and values for the shrinkage and shear parameters need to be specified. These are entered from the analysis form.

Section details:

Ref 1 "Section 1"

at 0.0 x span = 0.0 x 25 = 0m from left end of beam

Analysis:

Differential Temperature Primary Stresses

At time considered, $t = \infty$

Profile:
EN 1991-1-5:2003
Figure 6.2 Non-linear

DIFFERENTIAL TEMPERATURE

Profile: EN 1991-1-5:2003 Figure 6.2 Non-linear

Figure 6.2c: Type 3b. Concrete Beams

Surfacing : surfaced

Surfacing thickness: 0.1 m

Top warmer than bottom height m	Temperature °C	Bottom warmer than top height m	Temperature °C
0.0	13.5	0.0	-8.296
0.15	3.0	0.25	-0.76
0.4	0.0	0.45	0.0
1.17	0.0	0.92	0.0
1.37	2.5	1.12	-1.13
		1.37	-6.448

Relaxing Forces

-

	Moment kN.m	Axial kN
Heating Temperature difference	-387.898	-1019.826
Cooling Temperature difference	130.4534	1003.8252

Self-Equilibrating Stresses

Y6 Beam

Distance to top of section - m	Stress - MPa Heating	Cooling
0.17	-0.945849	0.2682988
0.25		1.1120964
0.4	-1.503124	
0.45		1.2574939
0.92		0.9333952
1.12		0.3742409
1.17	0.0756946	
1.37	1.4177241	-1.780589

Slab

Distance to top of section - m	Stress - MPa Heating	Cooling
0.0	2.4211771	-1.362657
0.15	-0.801980	
0.2	-0.910246	0.5225549

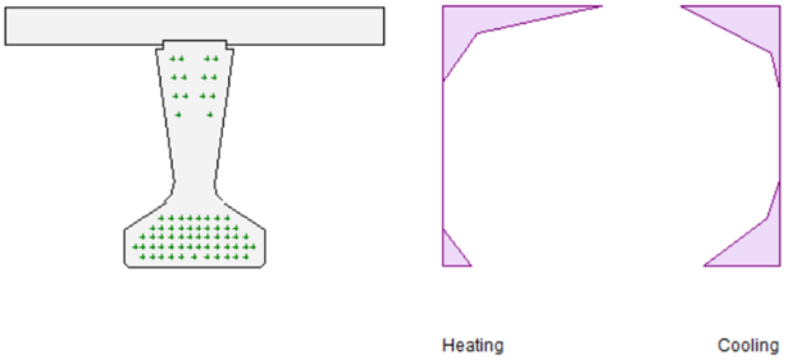


Figure C.12 Differential temperature analysis

Shrinkage and Creep Parameters

The Shrinkage strain was taken as 0.00025, the Long term losses occurring before composite was taken as 20%, the Differential shrinkage was taken as 0.0001 and the Creep coefficient, 1.5.Exposure class , XC4.

Figure C.13 shows the stresses of the beam at transfer.

Figure C.13 Stresses at transfer when curing at 28 celsius

Section details:

Ref 1 "Section 1"

at 0.5 x span = 0.5 x 25 = 12.5m from left end of beam

Analysis:

Stresses at Transfer

Serviceability Limit State: Characteristic - EN 1990 Equation 6.14

EN 1990 Equation 6.14

ACTUAL STRESSES IN PRECAST BEAM

No. of tendons fully bonded at this section: 54

EN1992-1-1

Maximum Stressing Force - EN 1992-1-1 Clause 5.10.2.1(1)P

Clause 5.10.2.1(1)P

For tendon property Grade 1600 Ep 195.0 INC

$k_1 . f_{pk}$ = 0.8*1860.0 = 1488.0 MPa

$k_2 . f_{p0,1k}$ = 0.9*1600.0 = 1440.0 MPa

Wedge draw-in loss Clause 5.10.4(1)(i)

Clause 5.10.4(1)(i)

draw-in strain = 0.006/100.0

= 0.00006

Loss = E_p . strain

	$= 195.0*0.00006$ $= 11.7$ MPa	
Curing Clause 5.10.4(1)(ii	Heat Curing Clause 5.10.4(1)(ii)(Note) Ambient temperature, T_0 = 25.0°C Maximum Curing temperature, T_{max} = 25.0°C	
	Immediate Losses	
EN 1992-1-1 Clause 5.10.4 .	(see table below)	

height mm	No of tendons	f_p MPa	k_1 / k_2	draw-in MPa	heat cure MPa	area mm^2	initial force kN
60.0	9	1600.0	0.9	11.7	4.875	150.0	1921.6237
110.0	14	1600.0	0.9	11.7	4.875	150.0	2989.1925
160.0	12	1600.0	0.9	11.7	4.875	150.0	2562.165
210.0	9	1600.0	0.9	11.7	4.875	150.0	1921.6237
260.0	6	1600.0	0.9	11.7	4.875	150.0	1281.0825
800.0	2	1600.0	0.9	11.7	4.875	150.0	427.0275
1000.0	2	1600.0	0.9	11.7	4.875	150.0	427.0275
TOTAL	54						11569.23

In accordance with clause 5.10.9(1), for SLS, the Characteristic value must be used.

With r_{inf} = 0.95, $P_{k,inf}$ = 10990.769 kN

Clause 5.10.4(1)(i)

Transverse Eccentricity

Tendon layout is symmetrical about the vertical centroidal axis.

Friction Clause

All tendons are straight in this beam.

Clause 5.10.4(1)(ii)

Initial Relaxation

Loss was calculated from clause 3.3.2(7)

For tendon property Grade 1600 Ep 195.0 INC

clause 3.3.2(7)

relaxation loss at 1000 hours,

ρ_{1000} = 2.5 %

$$\mu = \sigma_{pi} / f_{pk}$$

= (1440.0-11.7-4.875)*0.95/1860.0

= 0.72701

time after tensioning = 96.0 hours

A time adjustment for heat curing is required - clause 10.3.2.1(2)

from expression (10.2) ,

clause 10.3.2.1(2) from expression (10.2)

$$t_{eq} = \frac{1.14^{(25.0\ -20)}}{(25.0-20)} * 10.3874 = 4.0 \text{ hours}$$

for Class 2 relaxation, use Expression (3.29)

$0.66 \,.\, \rho_{1000} \,.\, e^{9.1\,\mu} \,.\, [t/1000]^{0.75(1-\ \mu)} \,.\, 10^{-5}$

$= 0.66 * 2.5 * 746.851 * 0.32289 * 10^{-5}$

Expression (3.29)

= 0.00398

height mm	No of tendons	area x σ_{pi}	relaxation %	loss kN	After relaxation force kN	moment kN.m
60.0	9	1831.79	0.41	7.5329876	1824.2618	109.45571
110.0	14	2849.46	0.41	11.717981	2837.7405	312.15146
160.0	12	2442.39	0.41	10.043983	2432.349	389.17584
210.0	9	1831.79	0.41	7.5329876	1824.2618	383.09497
260.0	6	1221.2	0.41	5.0219917	1216.1745	316.20537
800.0	2	407.065	0.41	1.6739972	405.3915	324.3132
1000.0	2	407.065	0.41	1.6739972	405.3915	405.3915
TOTAL	54				10945.571	2239.7881

Moment about the centroid of the precast beam:

M_r = 2239.7881-(10945.571*0.5149171)

= -3396.274 kN.m

Corresponding stresses:

top stress = 10945.571/491982.34+-3396.274/1.0378E8

= 22.247893+-32.72609

= -10.4782 MPa

bottom stress = 10945.571/491982.34+-3396.274/-1.381E8

= 22.247893+24.597352

= 46.845246 MPa

Self-weight moment:

c.s.a. = 4.92E5 mm²

EN 1991-1-1 Table A.1 Notes (1) and (2)	density = 24.0 kN/m³ + 1.0 kN/m³ + 1.0 kN/m³ = 26.0 kN/m³ (Refer to EN 1991-1-1 Table A.1 Notes (1) and (2)) self-weight = 4.92E5*26.0 = 12.7915 kN/m beam length = 25.0 m distance = 12.5 m M_{sw} = 0.5*12.7915*12.5*(25.0-12.5) = 999.339 kNm Corresponding stresses: top stress = 999.339/1.0378E8 = 9.62951 MPa bottom stress = 999.339/-1.381E8 = -7.2377 MPa	
.		
Clause 5.10.4(1)(iii)	**Elastic Deformation** stress at top of precast beam = -0.8486 MPa stress at bottom of precast beam = 39.6076 MPa depth of precast beam = 1200.0 mm elastic modulus of concrete at transfer = 33.6588 GPa	

height mm	No of tendons	conc stress MPa	conc strain	tendon force kN	tendon moment kN.m
60.0	9	37.58477	0.001117	293.95526	17.637316
110.0	14	35.89909	0.001067	436.75545	48.043099
160.0	12	34.21341	0.001016	356.78328	57.085324
210.0	9	32.52773	9.664E-4	254.40356	53.424747
260.0	6	30.84206	9.163E-4	160.8131	41.811407
800.0	2	12.63674	3.754E-4	21.963007	17.570406
1000.0	2	5.894026	1.751E-4	10.243985	10.243985
TOTAL	54			1534.9176	245.81628

.

Moment about the centroid of the precast beam:

M_{ed} = 245.81628-(1534.9176*0.5149171)

= -544.5391 kN.m

hence,

top stress = -0.848-1534.9176/491.98234--544.5391/1.0378E8

= -0.848-3.1198633--5.247114

= 1.2785665 MPa

bottom stress = 39.608-1534.9176/491.98234--544.5391/-1.381E8

= 39.608-3.1198633-3.9437988

= 32.543917 MPa

After a further 4 iterations of the above process, the top and bottom stresses were obtained follows:

top stress = 0.94095783 MPa

bottom stress = 33.5810642 MPa

EN 1992-1-1 Clause 5.10.3.(2)

Max Prestress Force after transfer

For tendon property Grade 1600 Ep 195.0 INC

$k_7 . f_{pk}$ = 0.75*1860.0 = 1395.0 MPa

$k_8 . f_{p0,1k}$ = 0.85*1600.0 = 1360.0 MPa

Maximum tendon stress after transfer = 1314.34 MPa

Which is not greater than 1360.0 and therefore OK.

TOTAL LOSS OF PRESTRESS SUMMARY

Initial stressing force = 10990.8 kN

Prestress after all transfer losses = 9630.69 kN

Corresponding loss = 12.4 %

Total loss of prestress = 12.4 %

LIMITING STRESSES IN PRECAST BEAM

Compression

EN 1992-1-1 Clause 3.1.2(5) 3.1.2(6)

.

For transfer at t = 4.0 days

Age adjusted for heat curing - clause 10.3.1.1(3)

from expression (B.10) adjusted t0 = Σ = 5.02023 days

	fck (t) = fcm (t) - 8.0 fcm (t) = βcc (t).fcm *Equation 3.1* βcc (t) = exp{s[1-√(28/t)]} *Equation 3.2* for Class N cement, s = 0.25	
EN 1992-1-1 Clause 5.10.2.2(5)	hence βcc (t) = exp{0.25[1.0-√28/5.02023)]} = 0.71147 fcm = fck + 8.0 = 58.0 MPa fcm (t) = 0.71147*58.0 = 41.2656 and fck (t) = 41.2656 - 8.0 MPa = 33.2656 MPa	
	σc <= 0.6*fck (t) *Equation 5.42* = 0.6 * 33.2656 = 19.9593 MPa hence limiting compression stress = 19.959334 MPa This may be increased if justified to: σc <= k6 .fck (t) = 0.7 * 33.2656 = 23.2859 MPa	.
	Tension	
EN 1992-1-1 Clause 7.3.2(4)	Tension is governed by crack width considerations, and reinforcement provided for crack width control. No reinforcement is required for tensile stress less than $\sigma_{ct,p}$ where: $\sigma_{ct,p} = f_{ct,eff}$ from clause 7.3.2(2), $f_{ct,eff} = f_{ctm}(t)$	

.	f_{ctm} = 0.3*f_{ck} $^{(2/3)}$ *(from Table 3.1)* = -4.0716 MPa f_{ctm} (t) = β_{cc} (t).f_{ctm} *(clause 3.1.2(9))* = 0.71147*-4.0716 = -2.8969 MPa hence limiting tension stress = -2.896861 MPa	
EN 1992-1-1 Clause 8.10.2.2(1)	**TRANSMISSION LENGTH** **Bond stress at release, EN 1992-1-1 Clause 8.10.2.2(1)** f_{bpt} = η_{p1} .η_1 .f_{ctd} (t) Expression (8.15) where f_{ctd} (t) = α_{ct} .0.7f_{ctm} (t)/γ_c f_{ctm} (t) = -2.8969 MPa (For the derivation of this value refer to the limiting stress calculations for transfer) α_{ct} = 1.0 - from EN 1992-1-1/3.1.6(2) tendon type coefficient, η_{p1} = 3.2 bond condition coefficient, η_1 = 1.0 hence f_{ctd} (t) = 1.0*0.7*-2.8969/1.5 = -1.3519 MPa and f_{bpt} = 3.2*1.0*-1.3519 = -4.326 MPa	.
EN 1992-1-1 Clause 8.10.2.2(2)	**Basic transmission length, EN 1992-1-1 Clause 8.10.2.2(2)** l_{pt} = α_1 .α_2 .ϕ.σ_{pm0} /f_{bpt} Expression (8.16) where speed of release coefficient, α_1 = 1.0 tendon surface coefficient, α_2 = 0.19	.

	nominal diameter of tendon, ϕ = 15.7 mm tendon stress after release, σ_{pm0} = 1360.0 MPa hence l_{pt} = 1.0*0.19*15.7*1360.0 / 4.32598 = 0.93779 m	
EN 1992-1-1 Clause 8.10.2.2(3)	**Design value of transmission length, EN 1992-1-1 Clause 8.10.2.2(3)** l_{pt1} = 0.8*l_{pt} = 0.8*0.93779 = 0.75023 m	

SLS STRESS SUMMARY TABLE

	force kN	moment kN.m	Concrete Stresses (MPa) In situ top	In situ bottom	Precast top	Precast bottom
CHARACTERISTIC PERMANENT ACTIONS AND PRESTRESS						
Prestress[3]	10945.6	-3396.3			-10.478	46.8452
Self Weight		999.339			9.62951	-7.2377
Prestress + Self Weight					-0.8486	39.6076
Elastic Def	-1314.9	463.088			1.78964	-6.0265
TRANSFER	9630.69	-1933.8			0.94095	33.5811

SLS FLEXURE

EN 1992-1-1 Clause 7.4.3	**EN 1992-1-1 Clause 7.4.3 calculated for un-cracked sections** The deflections are calculated from integration of the curvatures along the beam, using the parameters detailed below: Elastic Modulus at Transfer, E_T = 33.6588 GPa [EN1992-1-1 Clause 3.1.3(3) and 3.1.2(6), for age adjusted for heat curing, t_0 = 5.02023 days]	.

Precast section height = 1200.0 mm

		Precast Stress (MPa)	E	Strain ($x10^{-6}$)	Curvature ($x10^{-6}$) (rad/m)	Deflection[4] (mm) Here	Max.
At Transfer	T	0.94095	E_T	27.9558	-446.45	37.1099	37.1099
	B	33.5811		997.69			

DESIGN FOR SHEAR

The beam was checked for shear, for both at SLS for web shear cracking and at ULS for direct vertical shear.

The results are displayed graphically in figure C.14 for SLS frequent analysis for shear. Generally the blue lines are resistance, or limiting effects, and the green lines are the actual action effects (unless these are anywhere greater than the limiting, in which case they are plotted in red).

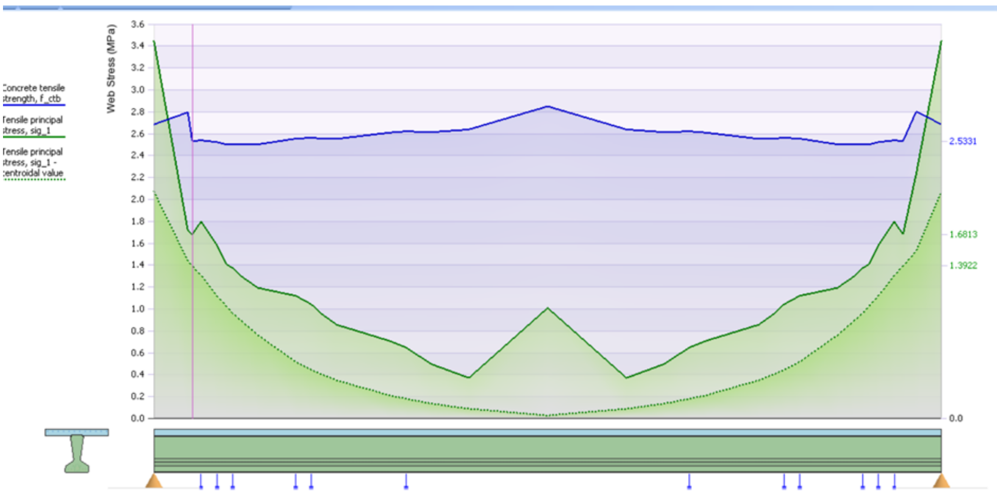


Figure C.14 SLS frequent analysis for shear

This analysis was done assuming that all tension steel (As) is adequately anchored to resist the required tensile forces. (Refer to clause 6.2.1(7), Figure 6.3, and clause 6.2.3(7) of EN1992-1-1).

EN1992-1-1 clause 6.2.1(7), Figure 6.3 clause 6.2.3(7) .	**SPECIMEN CALCULATION** Ref 1 "Section 1" at 0.049 x span = 0.049 x 25 = 1.22176m from left end of beam **Analysis:** Traffic Actions: Shear for gr5, loading I.D. 1 At time considered, $t = \infty$ Serviceability Limit State: Frequent - EN 1990 Equation 6.15	.

SUMMARY OF ACTIONS

PERMANENT ACTIONS

ACTION TYPE		SHEAR kN	MOMENT kN.m	AXIAL kN
Beam erection before composite	=	111.7297	45.898016	0.0
Construction stage 1A	=	109.88924	133.92268	0.0
Surfacing	=	28.195588	34.362132	0.0
Other permanent action	=	40.648323	132.38228	0.0
TOTAL PERMANENT ACTIONS,	G_k	290.46285	346.5651	0.0

VARIABLE ACTIONS

ACTION TYPE		SHEAR kN	MOMENT kN.m	AXIAL kN	ψ_0	ψ_1	ψ
Traffic,gr5 - for Shear design	=	674.522	767.401		0.0	0.0	0.75

TOTAL VARIABLE ACTIONS, $\psi_{1,1}$ x $Q_{k,1}$ "+" $\Sigma\psi_{2,i}$ x $Q_{k,i}$

		SHEAR	MOMENT	AXIAL
Traffic leading:	ψ_1 x Traffic	505.89127	575.5506	0.0
	ψ_2 x Other	0.0	0.0	0.0
	Total	505.89127	575.5506	0.0

Critical case is with traffic leading

TOTAL COMBINATION

	SHEAR	MOMENT	AXIAL
	796.35412	**922.11571**	**0.0**

.

WEB SHEAR CRACKING

EN 1992-2 Annex QQ

Characteristic strength of concrete in web, f_{ck} = 50.0 MPa

Characteristic tensile strength 5% fractile, $f_{ctk;0,05}$ = -2.8501 MPa

Design shear on precast section before composite, $V_{Ed,1}$ = 221.61894 kN

Total design shear on precast section, V_{Ed} = 796.35412 kN

Design shear on precast section after composite, $V_{Ed,2}$ = 574.73518 kN

.

Stress in precast from Prestress P and bending M_{Ed} :

at the top of the precast section, σ_a = -0.3390 MPa

at the bottom of the precast section, σ_b = 8.56473 MPa

.

Height of precast section, h = 1200.0 mm

Principal tensile stress is checked at the level of the centroid, and in addition at 100 points through the depth of the section to find the critical level.

At the composite section centroid:

At the composite section centroid, $z_{f,max}$ = 843.37825 mm

.

At this height:

width of precast section, b = 334.57627 mm

direct stress, $\sigma_{c1} = \sigma_b + z_{f,max} / h^*(\sigma_a - \sigma_b)$

= 8.56473 + 843.378/1200.0*(-0.3390-8.56473)

= 2.307 MPa

For precast section:

.

area beyond level $z_{f,max}$ A_1 = 1.333E5 mm²

first moment of area $(A.z)_1$ = 6.796E7 mm³

second moment of area $I_{yy,1}$ = 7.1097E10 mm^4

Shear stress at height $z_{f,max}$ $\tau_{yz,Ed,1} = V_{Ed,1} * (A.z)_1 / (I_{yy,1} .b)$

= 221.619 * 6.796E7 / (7.1097E10*334.576)

= 0.63313 MPa

.

For composite section:

area beyond level $z_{f,max}$ (transformed) A_2 = 3.587E5 mm² first moment of area $(A.z)_2$ = 1.858E8 mm³

second moment of area $I_{yy,2}$ = 1.9503E11 mm⁴

Shear stress at level $z_{f,max}$ $\tau_{yz,Ed,2} = V_{Ed,2} * (A.z)_2 /(I_{yy,2} .b)$

. = 574.735 * 1.858E8 / (1.9503E11*334.576)

= 1.63625 MPa

total shear stress, $\tau_{yz,Ed}$ = 0.63313 + 1.63625

= 2.26938

From Mohr's circle:

center, σ_{c0} = 0.5*(2.307+0.0) = 1.1535 MPa

radius, $\sigma_r = \sqrt{[\tau_{yz,Ed}{}^2 + (\sigma_{c1} - \sigma_{c0})^2]}$ = 2.54571 MPa

.

σ_3 = 1.1535 + 2.54571 = 3.69921 MPa

σ_1 = 1.1535 - 2.54571 = -1.3922 MPa

From Expression QQ.101, and using Annex QQ sign convention,

σ_1 = 1.39221 MPa

$f_{ctb} = [1 - 0,8.(\sigma_3 / f_{ck})].f_{ctk;0,05}$

= 0.94081 * 2.85014

= 2.68145 MPa

.

Through the depth of the section:

Level at which critical tension stress occurs, $z_{f,max}$ = 444.0 mm

At this height:

width of precast section, b = 218.77988 mm

direct stress, $\sigma_{c1} = \sigma_b + z_{f,max} / h*(\sigma_a - \sigma_b)$

= 8.56473 + 444.0/1200.0*(-0.3390-8.56473)

= 5.27032 MPa

For precast section:

area beyond level $z_{f,max}$ A_1 = 2.482E5 mm²

.

first moment of area $(A.z)_1$ = 8.373E7 mm³

second moment of area $I_{yy,1}$ = 7.1097E10 mm^4

Shear stress at height $z_{f,max}$

$\tau_{yz,Ed,1} = V_{Ed,1} * (A.z)_1 / (I_{yy,1} .b)$

= 221.619 * 8.373E7 / (7.1097E10*218.78)

= 1.19291 MPa

.

For composite section:

area beyond level $z_{f,max}$ (transformed) A_2 = 2.482E5 mm²

first moment of area $(A.z)_2$ = 1.652E8 mm³

second moment of area $I_{yy,2}$ = 1.9503E11 mm^4

Shear stress at level $z_{f,max}$

$\tau_{yz,Ed,2} = V_{Ed,2} * (A.z)_2 / (I_{yy,2} .b)$

= 574.735 * 1.652E8 / (1.9503E11*218.78)

= 2.2258 MPa

. total shear stress, $\tau_{yz,Ed}$ = 1.19291 + 2.2258

= 3.41871

From Mohr's circle:

centre, σ_{c0} = 0.5*(5.27032+0.0) = 2.63516 MPa

radius, $\sigma_r = \sqrt{[\tau_{yz,Ed}{}^2 + (\sigma_{c1} - \sigma_{c0})^2]}$ = 4.31644 MPa

σ_3 = 2.63516 + 4.31644 = 6.9516 MPa

σ_1 = 2.63516 - 4.31644 = -1.6813 MPa

From Expression QQ.101, and using Annex QQ sign convention,

σ_1 = 1.68128 MPa

. $f_{ctb} = [1 - 0{,}8.(\sigma_3 / f_{ck})].f_{ctk;0,05}$

= 0.88877 * 2.85014

= 2.53313 MPa

Which is greater than σ_1, so minimum reinforcement in accordance with 7.3.2 should be provided.

After that, the beam was analyzed for ULS persistent/ transient shear. The results are displayed graphically in figure C.15.

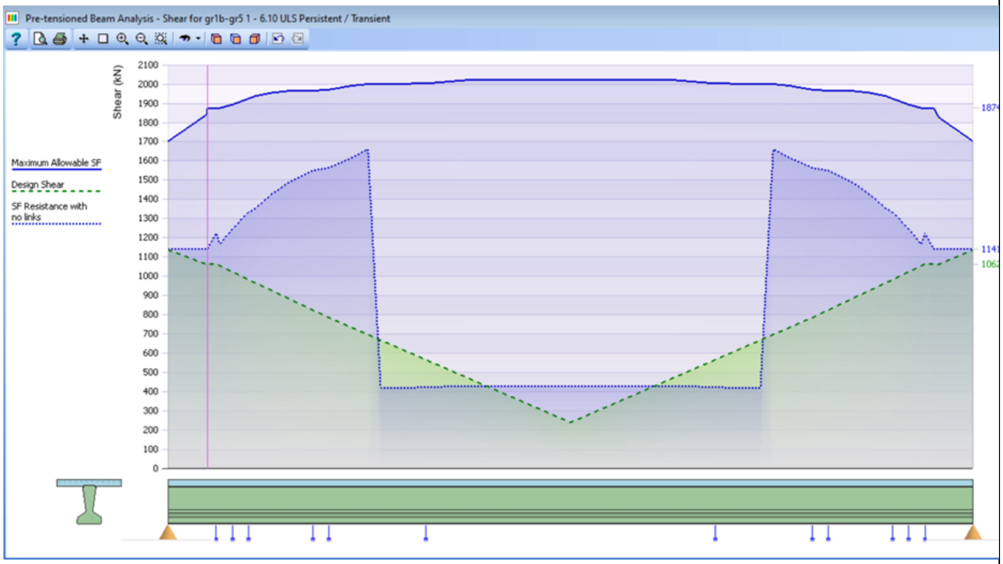


Figure C.15 ULS persistent/ transient analysis for shear

This analysis assumes that all tension steel (As) is adequately anchored to resist the required tensile forces.(Refer to clause 6.2.1(7), and clause 6.2.3(7) of EN1992-1-1).

.

SPECIMEN CALCULATION

Ref 1 "Section 1"

at 0.049 x span = 0.049 x 25 = 1.22176m from left end of beam

Analysis:

Traffic Actions: Shear for gr5, loading I.D. 1

At time considered, $t = \infty$

Ultimate Limit State: Persistent / Transient - EN 1990 Equation 6.10

EN1992-1-1

SUMMARY OF ACTIONS

PERMANENT ACTIONS

ACTION TYPE		SHEAR kN	MOMENT kN.m	AXIAL kN
Beam erection before composite	=	150.83509	61.962322	0.0
Construction stage 1A	=	148.35047	180.79561	0.0
Surfacing	=	33.834706	41.234559	0.0
Other permanent action	=	54.875236	178.71607	0.0
TOTAL PERMANENT ACTIONS,	$\gamma_G \times G_k$	387.89551	462.70857	0.0

VARIABLE ACTIONS

ACTION TYPE[1]		SHEAR kN	MOMENT kN.m	AXIAL kN	ψ_0	ψ_1	ψ_2
Traffic gr1b-gr5 - for Shear design	=	674.522	767.401	0.0	0	0.75	0

TOTAL VARIABLE ACTIONS, $\gamma_{Q,1} \times Q_{k,1}$ "+" $\Sigma\gamma_{Q,i} \times \psi_{0,i} \times Q_{k,i}$

		SHEAR	MOMENT	AXIAL
Traffic leading:	Traffic	674.5217	767.40081	0.0
	ψ_0 x Other	0.0	0.0	0.0
	Total	674.5217	767.40081	0.0

TOTAL COMBINATION

SHEAR	MOMENT	AXIAL
1062.4172	**1230.1094**	**0.0**

The check for the requirements for design shear reinforcement, first if the section is cracked or un-cracked under the design actions is needed to be checked.

	Axial	Moment	σ_t	σ_b
	kN	**kN.m**	**MPa**	**MPa**
Prestress	3001.39	-1099.0	-4.4893	14.0601

Precast only

A_c = 4.92E5 mm²

I_{yy} = 7.11E10 mm⁴

z_{na} = 514.917 mm

W_t = 1.038E8 mm³

W_b = -1.38E8 mm³

	242.758	2.33919	-1.7582

Composite

A_c = 8.666E5 mm²

I_{yy} = 1.95E11 mm⁴

z_{na} = 843.378 mm

W_t = 5.469E8 mm³

W_b = -2.31E8 mm³

	987.351	1.80539	-4.2696
	TOTAL STRESS	**-0.3447**	**8.03236**

tension stress limit for cracking from clause 6.2.2(2):

$f_{ctk,0.05}$ = 0.7 * f_{ctm}

f_{ctm} = $0.3 * f_{ck}^{(2/3)}$ *(from Table 3.1)*

= -4.0716 MPa

$f_{ctk,0.05}$ = 0.7 * -4.0716

= -2.8501 MPa

clause 6.2.2(2) . Clause 6.2.2(2) . . .	$f_{ctk,0,05}$ $/\gamma_c$ = -2.8501/1.5 = -1.9001 MPa Therefore section is un-cracked. **Shear Resistance with no design shear reinforcement** **Prestressed section un-cracked in bending** At the centroidal axis of the section the shear resistance limited by the tensile strength of the concrete is given by: $V_{Rd,c} = I.b_w/S \,.\, \sqrt{[f_{ctd}{}^2 + \alpha_l\,.\sigma_{cp}\,.f_{ctd}\,]}$ *Expression (6.4)* For the precast section, $V_{c1} = I.b_w/S \,.\, \tau_s$ (where I, b_w and S relate to the precast section) Setting V_{c1} to V_{Gd} and rearranging, $\tau_s = V_{Gd}\ .\ (S/I)\ /\ b_w$ For the composite section, $V_{c2} = I.b_w/S \,.\, [\,\sqrt{(f_{ctd}{}^2 + \alpha_l\,.\sigma_{cp}\,.f_{ctd}\,)} - \tau_s\,]$ and, $V_{Rd,c} = V_{c1} + V_{c2}$ For the evaluation of τ_s : Second moment of area of precast section, I = 7.11E10 mm^4 At centroidal axis of section (z = 0.84337 m): Width of section, b_w = 334.576 mm First moment of area of precast section, S = 2.418E7 mm^3 Hence, τ_s = 299.186 * (2.418E7/7.11E10) / 334.576 = 0.30408 MPa For the composite section:	

Second moment of area of section, $I = 1.95E11\ mm^4$

At centroidal axis of section ($z = 0.84337$ m):

Width of section, $b_w = 334.576$ mm

First moment of area, $S = 1.913E8\ mm^3$

Concrete compressive stress, $\sigma_{cp} = 2.14484$ MPa

Transmission length upper bound value, $l_{pt2} = 1.20819$ m

At current section, considering all tendons, $\alpha_l = 1.0$

Characteristic strength of concrete $f_{ck} = 50.0$ MPa

Partial safety factor $\gamma_c = 1.5$

$f_{ctd} = \alpha_{ct} . f_{ctk,0.05} / \gamma_c$ *Expression (3.16)*

$\alpha_{ct} = 1.0$

$f_{ctk,0.05} = -2.8501$ MPa (from above)

hence,

$f_{ctd} = 1.0*-2.8501/1.5$

$= -1.9001$ MPa

and,

$V_{c2} = I.b_w / S . [\sqrt{(f_{ctd}^2 + \alpha_l . \sigma_{cp} . f_{ctd})} - \tau_s]$

$I.b_w / S = 1.95E11*334.576/1.913E8$

$= 3.411E5\ mm^2$

$V_{c2} = 3.411E5*[\sqrt{(1.90009^2 + 1.0*2.14484*1.90009)} - 0.30408]$

$= 3.411E5*2.46824$

$= 841.84$ kN

$V_{Rd,c} = V_{c1} + V_{c2}$

$= 299.186 + 841.84$

$= 1141.03$ kN

This calculation was repeated for other axes not through the centroid, and the minimum shear resistance found.

.

This occurs at the section height $z = 0.84337$ m :

width of section, $b_w = 334.576$ mm

first moment of area of precast section, $S = 2.418E7$ mm³

first moment of area, $S = 1.913E8$ mm³

concrete compressive stress, $\sigma_{cp} = 2.14484$ MPa

.

$\tau_s = 299.186 * (2.418E7/7.11E10) / 334.576$

$= 0.30408$ MPa

$V_{c2} = I.b_w /S . [\sqrt{(f_{ctd}{}^2 + \alpha_l .\sigma_{cp} .f_{ctd})} - \tau_s]$

$I.b_w /S = 1.95E11*334.576/1.913E8$

$= 3.411E5$ mm²

$V_{c2} = 3.411E5*[\sqrt{(1.90009^2 + 1.0*2.14484*1.90009)} - 0.30408]$

.

$= 3.411E5*2.46824$

$= 841.84$ kN

$V_{Rd,c} = V_{c1} + V_{c2}$

$= 299.186 + 841.84$

$= 1141.03$ kN

Since $V_{Ed} < V_{Rd,c}$ minimum shear reinforcement only may be provided at this section, to comply with clause 6.2.1(4).

Compression shear component, $V_{ccd} = 0.0$ kN

Tension shear component, $V_{td} = 0.0$ kN

.

for clause 6.2.1(6),

$V_{Ed} - V_{ccd} - V_{td} = 1062.42-0.0-0.0 = 1062.42$ kN

Shear Resistance with design shear reinforcement

Characteristic strength of shear rft, f_{ywk} = 500.0 MPa

Material partial factor γ_s = 1.15

Design strength of shear rft f_{ywk}/γ_s, f_{ywd} = 434.783 MPa

Characteristic strength of concrete f_{ck} = 50.0 MPa

Material partial factor γ_c = 1.5

ULS concrete strength coefficient, α_{cc} = 0.85

Design strength of concrete $f_{ck}/\gamma_c * \alpha_{cc}$, f_{cd} = 28.3333 MPa

Angle between compression strut & beam axis, θ = 45.0 °

$\cot\theta$ = 1.0

Angle between shear rft and beam axis, $\alpha = \pi/2$ rad

Axial force in cross section, N_{Ed} = 3001.39 kN

Area of concrete cross section, A_c = 8.666E5 mm²

Concrete compressive stress N_{Ed}/A_c, σ_{cp} = 3001.39/8.666E5

= 3.46332 MPa

Effective depth, d = 1221.76 mm

Maximum Shear Force Value

The maximum value of shear resistance is given by:

$V_{Rd,max} = \alpha_{cw}.b_w.z.\upsilon_1.f_{cd}(\cot\theta + \cot\alpha)/(1 + \cot^2\theta)$ *Expression (6.14)*

Compression chord stress coefficient, α_{cw}:

σ_{cp}/f_{cd} = 3.46332 / 28.3333

= 0.12223

Hence, from Expression (6.11aN) α_{cw} = 1.12223

Minimum width between tension and compression chords,

b_w = 216.0 mm

Inner lever arm, z = 1156.3 mm[2]

Strength reduction factor, υ_1:

$\upsilon_1 = 0.6[1.0 - f_{ck}/250]$ *Expression (6.6N)*

$= 0.6*(1.0-50.0/250.0)$

$= 0.48$

$\alpha_{cw}.b_w.z.\upsilon_1.f_{cd} = 1.12223*216.0*1156.3*0.48*28.3333$

$= 3811.96$ kN

Since, $\alpha = \pi/2$, the expression (6.14) reduces to:

$V_{Rd,max} = \alpha_{cw}.b_w.z.\upsilon_1.f_{cd}/(\cot\theta + \tan\theta)$ Expression (6.9)

$= 3811.96/(1.0 + 1.0)$

$= 1874.84$ kN

Which is greater than $V_{Ed} - V_{ccd} - V_{td}$ (1062.42 kN) and therefore OK.

Minimum Shear Reinforcement

The minimum shear reinforcement ratio is given by Expression (9.5N) as

$\rho_{w,min} = (0.08\sqrt{f_{ck}})/f_{ywk}$

$= (0.08\sqrt{31.875})/500.0$

$= 9.03E-4$

From Expression (9.4)

$\rho_w = A_{sw}/(s.b_w.\sin\alpha)$

Hence,

$(A_{sw}/s)_{,min} = \rho_{w,min}.b_w.\sin\alpha$

$= 9.03E-4*216.0*1.0$

$= 0.19511$ mm²/mm

Therefore, $A_{sw}/s = 0.24437$ mm²/mm

Table C.12 Link arrangement:

Diameter	Maximum spacing for 2 legs (mm)	Maximum spacing for 4 legs (mm)
6.0	231.4	462.8
8.0	411.378	822.756
10.0	642.778	1285.56
12.0	925.601	1851.2
16.0	1645.51	3291.02

Maximum longitudinal spacing between shear assemblies

Clause 9.2.2(6)

$s_{l,max}$ = 0.75*d(1+cotα) *Expression (9.6N)*

= 0.75*1221.76*1.0

= 916.324 mm

Clause 9.2.2(8)

Maximum transverse spacing of legs in shear links

$s_{t,max}$ = 0.75*d *Expression (9.8N)*

= 0.75*1221.76

= 916.324 mm

but $s_{t,max}$ <= 600.0 mm

so $s_{t,max}$ = 600.0 mm

H10 2 legs, links are used as shear links.

As = 157 mm2

0.75d = 0.75 x 1.22176

= 0.92mm

S = 157 mm2 / 0.2443

= 642.778 mm

= 600 mm

Clause 6.2.3(107)

Longitudinal Reinforcement for Shear

No design shear reinforcement is required at this location, and additional tensile force in longitudinal reinforcement therefore need not be considered.

Table C.13 Summary of link requirements along the beam

Distance from left beam end (m)	Asw / s mm^2/mm	Spacing in mm	S
0	0.24437	642.778	600
1.22176	0.24437	642.778	600
2.5	0.24437	642.778	600
5	0.24437	642.778	600
7.5	1.20059	130.835	125
10	0.24437	642.778	600
12.5	0.24437	642.778	600
15	0.24437	642.778	600
17.5	1.20059	130.835	125
20	0.24437	642.778	600
22.5	0.24437	642.778	600
25	0.24437	642.778	600

After that, the beam was analyzed for Interface shear. The results are displayed graphically in figure C.16.

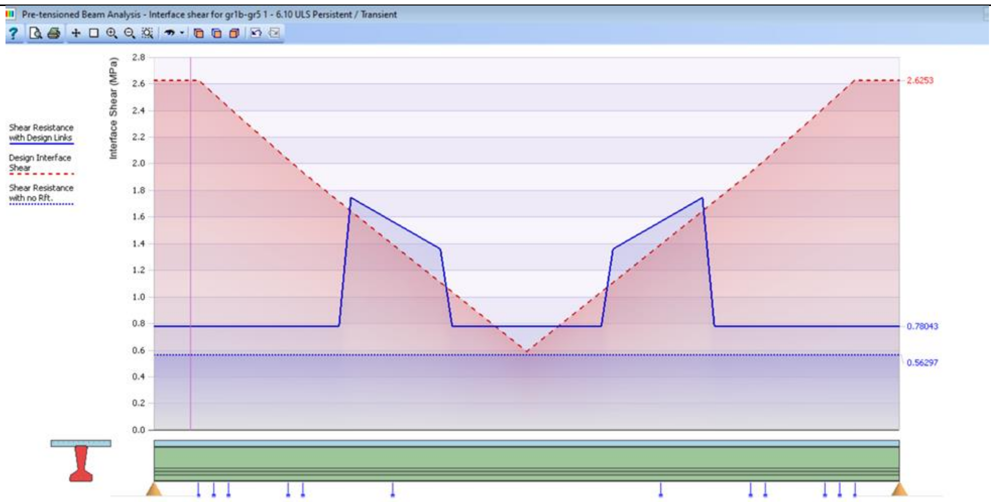


Figure C.16 Interface shear analysis for the beam

The interface shear requirement is the red dashed line and the interface shear resistance supplied by the links defined for direct shear is the solid blue line, so it can be seen that additional links are required in the areas where the red line is above the blue line.

SPECIMEN CALCULATION

Ref 1 "Section 1"

at 0.049 x span = 0.049 x 25 = 1.22176m from left end of beam

Analysis:

Interface and web/flange shear for gr5, loading I.D. 1

At time considered, $t = \infty$

Ultimate Limit State: Persistent / Transient - EN 1990 Equation 6.10

SUMMARY OF ACTIONS

PERMANENT ACTIONS

ACTION TYPE	SHEAR kN
Beam erection before composite	= 150.83509
Construction stage 1A	= 148.35047
Surfacing	= 33.834706

	Other permanent action = 54.875236 —————————— TOTAL PERMANENT ACTIONS γ_G x G_k 387.89551 **VARIABLE ACTIONS** ACTION TYPE SHEAR kN ψ_0 ψ_1 ψ_2 Traffic gr1b-gr5 - for Shear desigN = 674.5217 0.0 0.75 0.0 **TOTAL VARIABLE ACTIONS, $\gamma_{Q,1}$ x $Q_{k,1}$ "+" $\Sigma\gamma_{Q,i}$ x ψ_0 x $Q_{k,i}$** Traffic leading: Traffic 674.5217 ψ_0 x Other 0.0 —————————— Total 674.5217 **TOTAL COMBINATION** —————————— 1062.4172 **INTERFACE SHEAR CALCULATIONS** The calculations below are for the precast / in situ interface. They assume the presence of a horizontal top surface to the precast beam.	
Clause 6.2.5 .	Transverse shear force design value, V_{Ed} = 1062.42 kN Lever arm of the composite section, z = 1156.3 mm Width of the interface, b_i = 342.029 mm Reduction factor, β = 0.97729 For interface surface classification, Cohesion factor, c = 0.4	

Friction factor, $\mu = 0.7$

For the weaker concrete,

Compressive strength design value, $f_{cd} = 31.875/1.5$

$= 21.25$ MPa

Tensile strength design value, $f_{ctd} = 1.40743$ MPa

Normal stress across the interface, $\sigma_n = 0.0$ MPa

Area of rft crossing the interface, A_s is taken as $A_{sw}/s \,.\, s$

Area of interface, $A_i = b_i \,.\, s$

Hence, reinforcement ratio, $\rho = A_s/A_i$

$= 0.24437/342.029$

$= 7.14E-4$

Design strength of rft. across interface, $f_{yd} = 500.0/1.15$

$= 434.783$ MPa

Angle of reinforcement across interface, $\alpha = 90.0°$

$\sin\alpha = 1.0$

$\cos\alpha = 0$

Strength reduction factor (6.2.2(6)) $\upsilon = 0.5235$

Interface design shear stress v_{Edi} is given by Expression (6.24):

$v_{Edi} = \beta \,.\, V_{Ed} \,/\, (z \,.\, b_i)$

$= 0.977*1062.42/(1156.3*342.029)$

$= 2.62534$ MPa

interface design shear resistance v_{Rdi} is given by Expression (6.25):

$v_{Rdi} = c.f_{ctd} + \mu.\sigma_n + \rho.f_{yd}.(\mu.\sin\alpha+\cos\alpha) <= 0.5*\upsilon.f_{cd}$

$= 0.4*1.40743 + 0.7*0.0 + 7.14E-4*434.783*0.7$

$= 0.78042$ MPa

$0.5*\upsilon.f_{cd} = 0.5*0.5235*21.25$

$= 5.56219$ MPa

hence,

$v_{Rdi} = 0.78042$ MPa

	Since $v_{Edi} > v_{Rdi}$, additional resistance must be provided. The following table summarizes the options:	

surface roughness 6.2.5(2)	v_{Rdi} MPa	$*A_s$ required mm^2/mm	$A_s - A_{sw}$ **
Very smooth	0.15532	4.13054	3.88616
Smooth	0.18638	3.44211	3.19774
Rough	0.21745	2.95038	2.70601
Indented	0.27958	2.29474	2.05037
As defined	0.78042	2.31771	2.07333

* calculated by re-arranging Expression (6.25)

** transverse shear design links are adequate

Shear between web and flanges

In accordance with clause 6.2.4(103) the shear flow between web and flange may be conservatively taken as V_{Ed}/z per unit length of flange. These calculations assume:

Clause 6.2.4

b = width of stage 1 in-situ concrete

$b_{eff} = b$

$b_{eff,1} = b_{eff,2}$

h_f = slab thickness at precast centerline

$0.4h_f$ = depth in compression for transverse flexure

Design shear stress v_{Ed} on each side of the web is given by:

$v_{Ed} = 0.5 * V_{Ed} / z . [1.0 - b_w / b_{eff}] / h_f$

$= 0.5 * 1062.42/1156.3*[1.0-216.0/2000.0]/170.0$

$= 2.41051$ MPa

$k.f_{ctd} = 0.4*1.40743$

$= 0.56297$

$v_{Ed} > k.f_{ctd}$, therefore provide additional transverse reinforcement from Expression (6.21)

angle of compression strut, $\theta_f = 38.6598°$

depth of flange at junction, h_f = 170.0 mm

h_f reduced by compression depth per 6.2.4(105) = 102.0 mm

$A_{sf}/s_f = v_{Ed}.h_f/(f_{yd}.\cot\theta_f)$

= 2.41051*170.0/(434.783*1.25)

= 0.75400 mm²/mm

check for crushing in the flange struts from Expression (6.22):

$v_{Ed} <= \upsilon.f_{cd}.\sin\theta_f.\cos\theta_f$

= 0.5235*21.25*0.48780

= 5.42652 MPa

Which is satisfied.

Table C.14 Summary of link requirements along the beam

dimension from left beam end m	Flexural Shear A_{sw} / s mm²/mm	Interface Shear A_{sw} / s mm²/mm	6.2.5(3) suggestion	Spacing in mm for link diameter 10.0mm[2]
0.0	0.24437	2.31771	2.04949	76.6432
1.20819	0.24437	2.31771	2.04949	76.6432
1.22176	0.24437	2.31771	2.04949	76.6432
1.5	0.24437	2.32233	2.04949	76.6432
1.60689	0.24437	2.29831	0.42796	367.037
2.0	0.24437	2.20855	0.42796	367.037
2.5	0.24437	2.09378	0.42796	367.037
2.70819	0.24437	2.04693	0.42796	367.037
3.10689	0.24437	1.95809	0.42796	367.037
3.20819	0.24437	1.93554	0.42796	367.037
3.60689	0.24437	1.84737	0.42796	367.037
3.70819	0.24437	1.82498	0.42796	367.037
4.10689	0.24437	1.7374	0.42796	367.037
4.5	0.24437	1.65105	0.42796	367.037
5.0	0.24437	1.54097	0.42796	367.037
5.70819	0.24437	1.39749	0.42796	367.037
6.10689	0.24437	1.31707	0.42796	367.037
6.20819	0.24437	1.29665	0.42796	367.037
6.60689	1.32972	1.21644	1.32972	118.13
7.5	1.20059	1.03679	1.20059	130.835
8.0	1.1283	0.93623	1.1283	139.219
9.20819	0.95413	0.69325	0.95413	164.631
9.60689	0.89644	0.61308	0.89644	175.224
10.0	0.24437	0.53403	0.42796	367.037
12.5	0.24437	0.03138	0.42796	367.037
15.0	0.24437	0.53403	0.57271	274.271
15.3931	0.89644	0.61308	0.89644	175.224
15.7918	0.95413	0.69325	0.95413	164.631
17.0	1.1283	0.93623	1.1283	139.219
17.5	1.20059	1.03679	1.21293	129.505
18.3931	1.32972	1.21644	1.32972	118.13
18.7918	0.24437	1.29665	1.21293	129.505
18.8931	0.24437	1.31707	1.21293	129.505
19.2918	0.24437	1.39749	1.60156	98.0791
20.0	0.24437	1.54097	1.60156	98.0791
20.5	0.24437	1.65105	1.60156	98.0791
20.8931	0.24437	1.7374	1.60156	98.0791
21.2918	0.24437	1.82498	2.04949	76.6432
21.3931	0.24437	1.84737	2.04949	76.6432
21.7918	0.24437	1.93554	2.04949	76.6432
21.8931	0.24437	1.95809	2.04949	76.6432
22.2918	0.24437	2.04693	2.04949	76.6432
22.5	0.24437	2.09378	2.04949	76.6432
23.0	0.24437	2.20855	2.04949	76.6432
23.3931	0.24437	2.29831	2.04949	76.6432
23.5	0.24437	2.32233	2.04949	76.6432
23.7782	0.24437	2.31771	2.04949	76.6432
23.92	0.24437	2.31771	2.04949	76.6432
25.0	0.24437	2.31771	2.04949	76.6432

Displacements and rotations along the beam

One beam member has divided into 20 parts to analyze the structure. Following results were obtained.

Figure C.17 Main reference joint locations

Table C.15 Displacements and rotations along the beam

Reference point	Displacement			Rotation		
	Dx(mm)	Dy(mm)	Dz(mm)	Rx (deg)	Ry (deg)	Rz (deg)
1	0	0	-0.204	0	0.238961	0
2	0	0	-5.560	0	0.232018	0
3	0	0	-10.623	0	0.213678	0
4	0	0	-15.183	0	0.187379	0
5	0	0	-19.089	0	0.155239	0
6	0	0	-22.234	0	0.11938	0
7	0	0	-24.558	0	0.08192	0
8	0	0	-26.046	0	0.044973	0
9	0	0	-26.725	0	0.009824	0
10	0	0	-26.638	0	-0.02296	0
11	0	0	-25.839	0	-0.05281	0
12	0	0	-24.398	0	-0.07916	0
13	0	0	-22.395	0	-0.10144	0
14	0	0	-19.922	0	-0.11907	0
15	0	0	-17.086	0	-0.1315	0
16	0	0	-14.003	0	-0.13814	0
17	0	0	-10.803	0	-0.13844	0
18	0	0	-7.631	0	-0.13181	0
19	0	0	-4.639	0	-0.11771	0
20	0	0	-1.996	0	-0.09554	0
21	0	0	0.120	0	-0.06476	0
22	0	0	1.581	0	-0.03318	0
23	0	0	2.469	0	-0.00865	0
24	0	0	2.904	0	0.009408	0
25	0	0	2.999	0	0.021652	0
26	0	0	2.849	0	0.028797	0
27	0	0	2.542	0	0.03156	0
28	0	0	2.150	0	0.030661	0
29	0	0	1.733	0	0.026817	0
30	0	0	1.339	0	0.020746	0
31	0	0	1.006	0	0.013168	0
32	0	0	0.755	0	0.004799	0
33	0	0	0.599	0	-0.00364	0
34	0	0	0.535	0	-0.01144	0
35	0	0	0.550	0	-0.01787	0
36	0	0	0.618	0	-0.02221	0
37	0	0	0.700	0	-0.02376	0
38	0	0	0.745	0	-0.02179	0
39	0	0	0.691	0	-0.01558	0
40	0	0	0.460	0	-0.00448	0
41	0	0	-0.035	0	0.012085	0

42	0	0	-0.858	0	0.028916	0
43	0	0	-1.937	0	0.04082	0
44	0	0	-3.173	0	0.04836	0
45	0	0	-4.479	0	0.052018	0
46	0	0	-5.779	0	0.052213	0
47	0	0	-7.004	0	0.049365	0
48	0	0	-8.096	0	0.043891	0
49	0	0	-9.005	0	0.03621	0
50	0	0	-9.690	0	0.026738	0
51	0	0	-10.119	0	0.015894	0
52	0	0	-10.272	0	0.004095	0
53	0	0	-10.133	0	-0.00824	0
54	0	0	-9.699	0	-0.02069	0
55	0	0	-8.976	0	-0.03283	0
56	0	0	-7.977	0	-0.04426	0
57	0	0	-6.725	0	-0.05454	0
58	0	0	-5.254	0	-0.06327	0
59	0	0	-3.604	0	-0.07002	0
60	0	0	-1.827	0	-0.07438	0
61	0	0	0.000	0	-0.07592	0

Allowable max deflection = span / 250

= 25 000/250

= 100 mm

All values are less than that.

Hence ok.

APPENDIX D PIER DESIGN

REFERENCE	CALCULATIONS	RESULTS
EC2	C 32/40 in situ concrete and yield steel were used for the pier design. Concrete density = 2400 kg/m^3 Concrete strength = 32 Mpa Elastic modulus of concrete = 33.314 Gpa Steel yield strength = 500 Mpa Design life = 100 yrs	

Table D.1 Partial factors in Eurocode 2

In Eurocode 2	
Actions	Factor
superstructure dead loads	1.35
vehicle UDL (live)	1.35
vehicle TS (live)	1.35
Wind load	1.5
Stream pressure	1.5
breaking force	1.35

PIER HEAD DESIGN

Hammerhead type pier cap (since STM) was selected. For the final loads self-weight of pier cap should be added. Final loads are shown in table D.2.

Table D.2 Loads affecting on Pier cap

	all maximum bearing loads on the pier (KN)				
Action	Load 1	Load 2	Load 3	Load 4	Load 5
superstructure dead loads	272.5	302.5	302.5	302.5	272.5
vehicle live TS	274.77	610.59	915.88	610.59	274.77
vehicle live UDL	50.55	197.05	206.33	197.05	50.55
pier cap dead load	79.11	185.42	198.66	185.42	79.11
factored total bearing loads ULS	**886.17**	**1684.11**	**2122.11**	**1684.11**	**886.17**

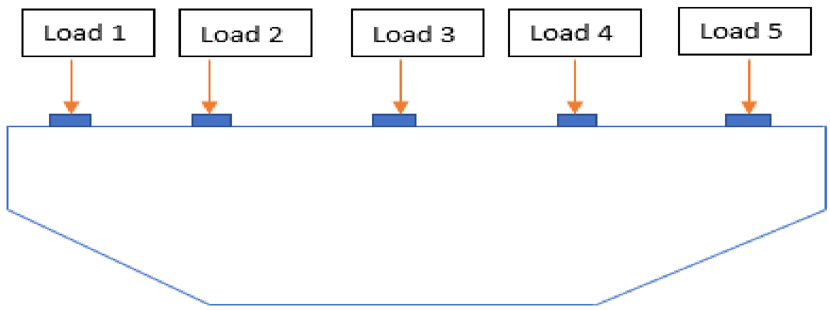


Figure D.1 Load distribution points of pier cap

Design calculation were shown in below according to Eurocode 2 following with STM. STM was developed using 16 nodes and 29 struts and ties with 2 supports. Dimensions of the Pier head are mentioned in above respective section.

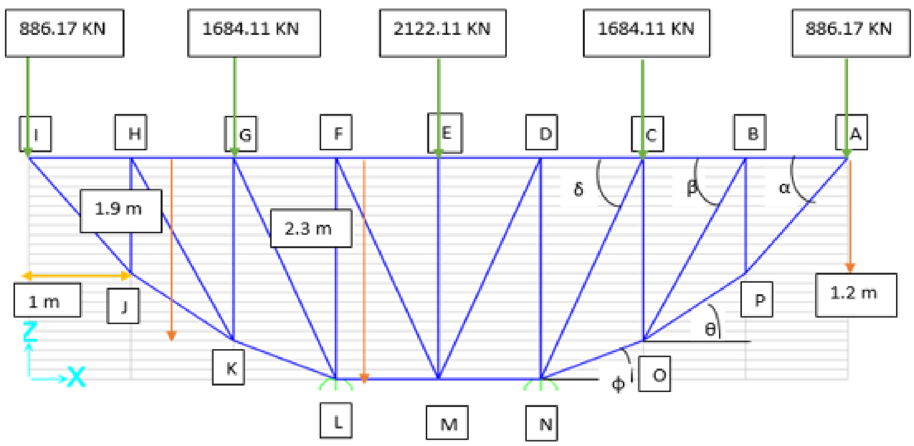


Figure D.2 Strut and tie model structure for pier cap

Figure D.2 shows the STM with loads that are affecting on the piers. Then forces were calculated in every member.

α = 50.19°, β = 62.24°, δ = 66.50°, θ = 34.99° and ϕ = 21.80°

Vertical Eqm of node A,

886.17 + C1*sin50.19 = 0, C1 = -1153.61 KN = FAP

Horizontal eqm, of node A,

T1 + C1*cos50.19 = 0, T1 = 738.59 KN = FAB

Eqm of ABP part,

Moment around B,

1.2*C3*cos34.99 + 886.17*1 = 0, C3 = -901.40 KN = FPO

Vertical forces,

886.17+ C2*sin62.24 + C3*sin34.99 =0, C2 = -417.31 KN = FBO

Horizontal forces,

C2*cos62.24 + C3*cos34.99 + T2 =0, T2 = 932.84 KN = FBC

Vertical Eqm of node P,

T3 + C1*sin50.19 – C3*sin34.99 =0, T3 = 369.28 KN = FBP

Eqm of ABPCO part,

Moment around C,

88617*2 + C5*cos21.8*1.9 = 0, C5 = -1004.66 KN = FON

Vertical forces,

886.17 + 1684.11 + C5*sin21.8 + C4*sin66.50 = 0, C4 = -2395.89 KN = FCN

Horizontal forces,

T4 + C4*cos66.50 + C5*cos21.8 =0, T4 = 1888.17 KN = FCD

Vertical Eqm of node O,

T5 + C5*sin21.8 + C3*sin34.99 =0, T5 = 143.79 KN = FCO

Eqm of ABPCODN part,

Moment around D,

2.3*C7 = 886.17*3 + 1684.11*1, C7 = -1888.09 KN = FNM

Vertical forces,

886.17 + 1684.11 + T6*sin66.50 = 3631.28, T6 = 1156.96 KN = FDM

Horizontal forces,

T7 + T6*cos66.5 + C7 = 0, T7 = 1426.75 KN = FDE

Vertical Eqm of node D,

C8 + C6*sin66.5 = 0, C8 = -1061.01 KN = FDN

Vertical Eqm of node E,

2122.01 + C9 = 0, C9 = -2122.01 KN = FEM

Since, STM is symmetric each opposite members have same force.

Table D.3 Forces in tension members

Member	Tension force KN
AB = HI	738.59
BC = HG	932.84
CD = GF	1888.17
DM = FM	1156.96
DE = FE	1426.75
BP = HJ	369.28
CO = GK	143.79

Table D.4 Forces in compression members

Member	Compression force KN
AP = IJ	1153.61
PO = JK	901.4
ON = KL	1004.66
NM = LM	1888.09
DN = FL	1061.01
EM	2122.01
BO = HK	417.31
CN = GL	2395.89

Reinforcement for Pier head

Summary of reinforcement layout provided in the main body.

EC2 4.1 to 4.6 page 9-11

Main reinforcement bar sizes = 25 mm for Top and Bottom

The vertical ties represent the centroid of stirrups that will be spaced across a "stirrup band". For this H10 4-legged stirrup band were used.

EC 2 page 44

Stirrups size = 10 mm was selected. (Exposure class XC 2)

Cover is 50 mm.

Ties

AB = HI members

f,ck = 32 MPa

f,yd = 500/1.15 = 435 MPa

tie force = 738.59 KN

As >= 738.59*1000/435 =1697.91 mm^2

	Required 25 mm bars = 1697.91/4*π*12.5^2 = 3.45 bars Use 4H25 = 4*π*12.5^2 = 1963.49 mm^2 > 1697.91 mm^2 - OK Likewise, found the required reinforcement for all the top tension ties. Then, required longitudinal reinforcement was calculated by sizing the ties as in table D.5. **Table D.5 Required reinforcement for top tension members**	

longitudinal steel			
member	As required (mm^2)	25 mm bars required	provided 25 mm bars
AB=HI	1697.11	3.45	4
BC=HG	2144.45	4.37	5
CD=GF	4340.62	8.84	9
DE=FE	3279.88	6.68	7

Maximum of 9 bars with 80 mm space are enough for top layer. For the longitudinal steel, anchorage will be provided by 90° (70 mm) hooks.

EC2 11.4 page 62	Hook length = 7*25 = 175 mm >= 70 mm -OK	
	Horizontal spacing for longitudinal spacing, = (1000 - 2*50 – 4*10 - 9*25)/8 = 79.375 mm = 80 mm (within 45-400 mm) - OK	Spacing = 80 mm
	No need of bottom reinforcement since there are no any bottom tension ties. Then required stirrups were calculated. <u>BP = HJ members</u> As max = 0.04*1000*1000 = 4*10^4 mm^2 As min = max {0.05*As max = 2000 or (0.1*369.28*1000/435) = 84.89} = 2000 mm^2	

	No of stirrups bands = 2000 / 4*π*5^2 = 6.36 Max Stirrup spacing = 1000/6.36 = 157.23 mm Use as Spacing = 150 mm Provided As = 1000*4*π*5^2/150 = 2094.39 mm^2 -OK Likewise, found the required reinforcement for rest of the tension ties	Spacing = 150 mm

Table D.6 Stirrups details of tension members

Member	As required (mm^2)	No of bands	spacing max (mm)	used spacing (mm)	provided As (mm^2)
BP=HJ	2000	6.36	157.23	150	2094.39
CO=GK	2000	6.36	157.23	150	2094.39
DM=FM	2000	6.36	157.23	150	2094.39

EC2 11.4 page 62

135° hooks (50 mm) are recommended for the shear stirrups.

Hook length = 5*10 = 50 mm >= 50 mm -OK

<u>Adequacy of Struts</u>

<u>AP = IJ Members</u>

EC 2 page 44

Strut width = 220 mm

Strut force = 1153.61 KN

Maximum allowable stress in a strut (σ_{RD} max) = k2*ύ*f_{Ecd} (CCT node)

k 2= 0.85, ύ = [1-(f_{ck}/250)] and f_{Ecd} = 0.85*f_{ck}/1.5

So, σ_{RD} max = 0.85*[1-(32/250)] *0.85*32/1.5 = 13.44 MPa

Stress in the strut = 1153.61*1000/ 220*1000

= 5.24 MPa > 13.44 MPa -OK

Likewise, all struts were checked in each compression members

Table D.7 Strut analysis for each compression member

Member	Compression force (KN)	stress in the strut (Mpa)	σ_{RD} max (Mpa)
AP = IJ	1153.61	5.24	13.44
PO = JK	901.4	4.1	13.44
ON = KL	1004.66	4.57	13.44
NM = LM	1888.09	8.58	15.81
DN = FL	1061.01	4.82	11.86
EM	2122.01	9.65	11.86
BO = HK	417.31	1.9	11.86
CN = GL	2395.89	10.89	11.86

After that struts were checked whether their stresses are within the allowable range. So, all the compression members are satisfactory as in table D.7. Then crack control check was carried out.

EC 2 page 57,58

Crack control check was carried out and required reinforcement was assigned like below. H10 4-legged stirrups were provided as required.

Vertical crack control,

Provided As = 2094.39 mm^2

Required min, As = 2000 mm^2 so, it should be used to entire cap with having space of 150 mm

Spacing = 150 mm

Horizontal crack,

Required min, As = 2000 mm^2

using H10 stirrups 4-legged with space of 300 mm

provided As = 3000*4*π*8^2/300 = 3141.59 mm^2 -OK

Spacing = 300 mm

	PIER STEM (COLUMN) DESIGN	
	Design calculation were shown in below according to the Eurocode 2. Height, width and depth of the pier stem are 16.2 m, 4 m and 1 m respectively.	
	Self weight of the column = 1.35*25*4*1*16.2 = 1620 KN	
	N ed = 8882.57 KN	
	Mx max = 9656.6 KNm	
	My max = 2701.01 KNm	
	Pier column acting as a unbrased compression member.	
	Nominal cover = 50 mm	
	Geometric imperfections,	
	$\theta_i = \theta_0\ \alpha_n\ \alpha_m$	
	$\theta_0 = 1/200$	
EC2 5.5	$\alpha_n = 1$ (isolated column)	
page 25	$\alpha_m = (0.5\ [1+1/m])^{0.5}$; m = 1 So, $\alpha_m = 1$	
	$\theta_i = 1/200$	
	λ lim = 20*ABC/ n	
	A = 0.7, B = 1.1	
	$C = 1.7 – M_{01} / M_{02}$,($\lvert M_{01}\rvert > \lvert M_{02}\rvert$)	
EC2	C = 1.7 – (8494.4/ 9655.4) = 0.82	
5.6.1.4		
page 28-30	$n = N_{cd} / (A_{CD} * f_{cd})$ = 8882.57 / (4*1*32*0.85/1.5)	
	n = 0.122	
EC2	λ lim = 20*0.7*1.1*0.82 / $0.122^{0.5}$ = 36.15	λ lim = 36.15
5.6.2.2		
page 31	k = θ*EI / ML	
	k1 = 0.33*33.314*10^6 / (200*8494.4*16.2) = 0.40	
	k2 = 0.33*33.314*10^6 / (200*9655.6*16.2) = 0.35	

	effective length factor = max of {$[1 + (10*k1*k2/ k1+k2)]^{0.5}$;	
	$[(1+ k1/1+k1)*(1+ k2/1+k2)]$}	
	= max of {1.69 ; 1.62} = 1.69	
	effective length = l_0 = 1.69*16.2 = 27.378 m	
	radius of gyration = i = h/ $12^{0.5}$ = 1/$12^{0.5}$ = 0.289 m	
	$\lambda = l_0 / i$ = 27.378 / 0.289 = 94.73 > λ lim (so, slender column)	
EC2		94.73 = λ
5.6.1.3	<u>Design moments calculation</u>	
page 27	Consider Mx,	
	Mo2x = Max M (top or bottom)+ ei*N_{Ed} ; ei = $\theta_i\, l_0 /2$	
EC2	Mo2 = 9655.6 + 8882.57*1*27.378/400 = 10264.57 KN/m	
5.6.2.2	Mo1x = 8494.4 KN/m	
page 30		
	MoEdx = 0.6*Mo2+ 0.4*Mo1 ≥ 0.4*Mo2	
	= 0.6*10264.57 + 0.4*(8494.4) ≥ 0.4*10264.57	
	= 9556.502 ≥ 4105.83	
	M2 = Ned*e2 = 8882.57*0.292 = 2596.61 KN/m	
	MEdx = Max {Mo2, MoEd +M2, Mo1 + 0.5M2}	
	= max {10264.57, 12153.112, 9792.71}	
	= 12153.11 KN/m	
	Consider My,	
	MEdy = M_{o2}	
	Mo2 = Max M (top or bottom)+ ei*N_{Ed} ; ei = $\theta_i\, l_0 /2$	
EC2	Mo2 = 2701.01 + 8882.57*1*27.378/400 = 3308.98 KN/m	
5.6.2.2	Mo1y = 2701.01 KN/m	
page 30	MoEdy = 0.6*Mo2+ 0.4*Mo1 ≥ 0.4*Mo2	
	= 3308.98*0.6 +0.4*2701.01 ≥ 0.4*3308.98	

	= 3065.79 ≥ 1323.59 M2 = Ned*e2 = 8882.57*0.292 = 2596.61 KN/m MEdy = Max {Mo2, MoEd +M2, Mo1 + 0.5M2} = max {3308.98, 5662.4, 3999.32} = 5662.4 KN/m So, the critical moment is M_{Edx} = 12153.11 KN/m	M = 12153.11 KN/m
EC2 15.9.6 page 96	<u>Reinforcement in Pier stem</u> Main reinforcement – 32mm bars Shear links – 4-legged 10mm bars used. d_2 = cover + shear link diameter + main rf diameter/2 = 50 +10 +32/2 =76 mm d_2 / h = 76/1000 = 0.076 N_{Ed} / bhf_{ck} = 8882.57 *1000/32*4000*1000 = 0.069 M_{Edx} / bh^2f_{ck} = 12153.11*10^6 /32*4000*1000^2 = 0.095 Note: d2/h = 0.10 chart is referred to find the As, reinforcement area, but it is more conservative. Interpolation between charts d2/h = 0.10 and d2/h = 0.05 can be used to find more accurate answer. As*f_{yk} / b*h*f_{ck} = 0.18 As = 0.18*4000*1000*32/500 = 46080 mm^2 Required main bars = 46080/π*16^2 = 57.29 So, 60 H32 bars needed. As provided = 48254.86 mm^2 Check for Biaxial Bending, MEdx =12153.11 kNm MEdy = 5662.4 kNm	

EC2 5.6.3 page 32	ey/heq = (MEdx / NEd) / heq=12153.11/8882.57*1 =1.37 ex/beq = (MEdy / NEd) / beq = 5662.4/8882.57*4 = 0.16 (ey/heq)/(ez/beq) = 1.37/0.1 = 8.56 (>0.2 and >5) Therefore, biaxial check is required. $(MEdz / MRdz)^a + (MEdy / MRdy)^a \leq 1$ MRdz and MRdy are the moment resistance in respective direction, corresponding to an axial load NEd. For symmetric reinforcement section MRdz = MRdy As Provided = 48254.86 mm^2 As*fyk / b*h*fck = 48254.86*500/ (4000*1000*32) = 0.19	
EC2 15.9.6 page 96	NEd / (b*h*fck) = 0.069 From the chart d2/h =0.10 MEd / [b*(h^2) *fck] = 0.099 MRd = 0.099*4*1*1000*32 = 12672 kNm NRd = Ac*fcd + As*fyd NRd = 4*1*(0.85*32/1.5) + 48254.86*(500/1.15) = 94.47 MN NEd / NRd = 8882.57/ 94.47*1000 = 0.094 By interpolating a = 1.00	

	$(MEdz / MRdz)^a + (MEdy / MRdy)^a = (12153.11/12672) + (5662.4/12672)$ = 1.44 > 1 Not ok. So, need to again using trial and error. MEd / [b*(h^2) *fck] = 0.14 and NEd / (b*h*fck) = 0.069 As*f_{yk} / b*h*f_{ck} = 0.32 As = 81920 mm^2 Required bars = 101.86 No of bars should be symmetric. Bars = 108 So, 4 layers with having 27 bars were assigned. Provided As = 86858.75 mm^2 As min = maximum (0.1*N*Ed/ *f*yd; 0.002*A*c) = max {2041.97, 8000} = 8000 mm^2 As max = 0.04*1000*4000 = 160000 mm^2	
EC2 12.5 page 74	So, As =86858.75 mm^2 is satisfactory.	
	Spacing = 4000- 50*2 -4*10 -27*32 / 26 =115.23 mm = 115 mm.	Spacing = 115 mm
	Transverse rf links max spacing = min of {12*32=384, 1000*0.6=600, 240} = 240 mm links spacing selected as 225 mm.	Spacing = 225 mm
	PILE CAP DESIGN Design calculation were shown in below according to EC2 following with STM. In X direction, STM was designed with 7 strut and ties with 3 supports and in Z direction STM was designed with 7 strut and ties with 3 supports. Figure D.3 –D.5 show the STM with loads affecting on pile cap. Self-weight of the pile cap = 1.35*1.8*6.1*3.05 = 837.225 KN Total vertical load ULS = 10012.82 KN	

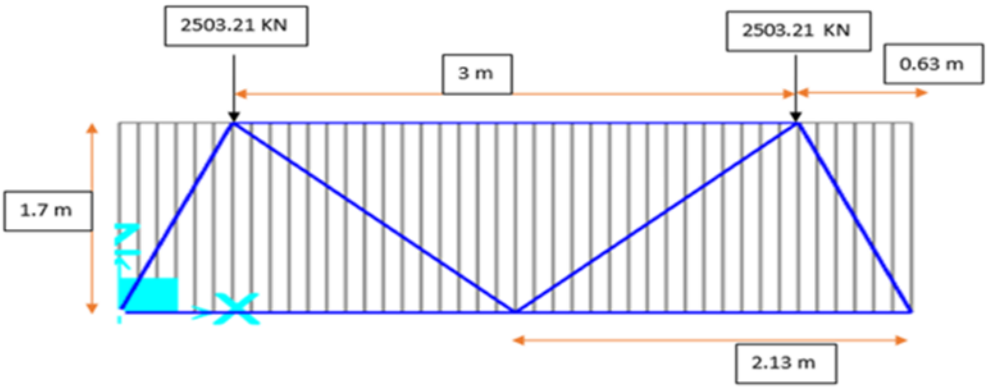


Figure D.3 Strut and tie model in X direction

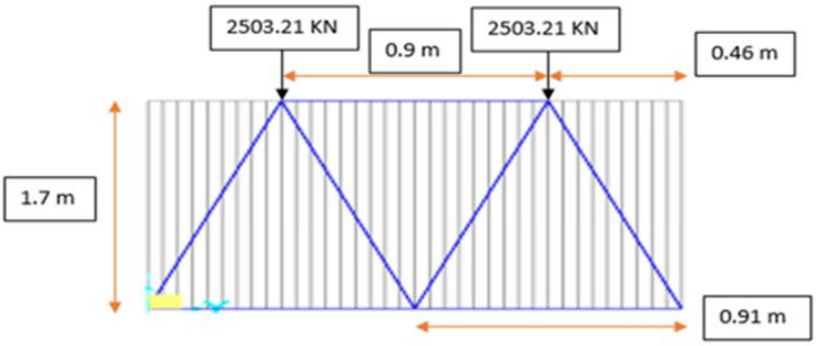


Figure D.4 Strut and tie model in Z direction

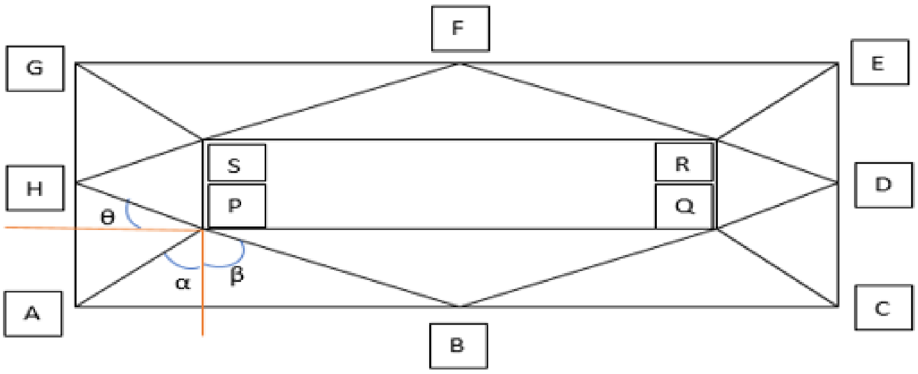


Figure D.5 Plan view of the STM

tanα = 0.63/0.46, tanβ =1.5/0.46, tanθ =0.45/0.63

Angle between AP member and Z direction is δ1, tanδ1 =0.78/1.7

Angle between BP member and Z direction is δ2, tanδ2 =1.57/1.7

Angle between HP member and Z direction is δ3, tanδ3 =0.77/1.7

Forces were calculated in every member and shown in table D.8.

Eqm of node A,

Z direction, 1251.61 + C1*1.7/1.87 = 0,   C1 = -1376.77 KN

X direction, T1 + C1*0.78/1.87*0.63/0.78 =0,   T1 = 463.83 KN

Y direction, T2 + C1*0.78/1.87*0.46/0.78 =0,   T2 = 338.67 KN

Eqm of node B,

Z direction, 1251.61 + C2*2*1.7/2.31 =0,   C2 = -850.36 KN

Eqm of node H,

Z direction, 1251.61 + C3*2*1.7/1.87 =0,   C3 = -688.39 KN

Eqm of node P,

X direction, C4 + C2*1.57/2.31*1.7/1.57 - C1*0.78/1.87*0.63/0.78 – C3*0.77/1.87*0.63/0.77 =0,

C4 = -69.92 KN

Y direction, C5+ C3*0.77/1.87*0.45/0.77 - C1*0.78/1.87*0.46/0.78 - C2*1.57/2.31*0.46/1.57 =0,

C5 = -342.24 KN

Table D.8 Forces in each member of the STM

Member	Compression Force (KN)	Tension Force (KN)
AB = BC = EF = FG		463.83
CD =DE = GH = HA		338.67
AP = CQ = ER = GS	1376.77	
BP = BQ = FR = FS	850.36	
DQ = DR = HS = HP	688.39	
PQ = RS	69.92	
QR = SP	342.24	

	<u>Reinforcement in pile cap</u> Main reinforcement bar sizes = 25 mm Cover is 50 mm. f,ck = 32 MPa f,yd = 500/1.15 = 435 MPa **<u>Ties</u>** <u>AB = BC = EF = FG members</u> tie force = 463.83 KN As >= 463.83*1000/435 = 1066.28 mm^2 Required 25 mm bars = 1066.28/π*12.5^2 = 2.17 bars are enough. But, required minimum RF area in X direction = 0.0015*b*h = 0.0015*1800*3050 = 8235 mm^2 So, required minimum bars = 16.77 Use 17 H25 = 17*π*12.5^2 =As prov = 8344.86 mm^2 > 1066.28, 8235 mm^2 - OK Spacing = 3050- 50*2 / 17 = 173.53 mm = 175 mm. <u>CD = DE = GH = HA members</u> tie force = 338.67 KN As >= 338.67*1000/435 = 778.55 mm^2 Required 25 mm bars = 778.55/π*12.5^2 = 1.58 bars are enough. But, required minimum RF area in X direction=0.0015*b*h = 0.0015*1800*3050 = 16470 mm^2 So, required minimum bars = 33.55 Use 34 H25 = 34*π*12.5^2 =As prov = 16689.71 mm^2 > 778.55, 16470 mm^2 - OK Spacing = 6100 – 50*2 / 34 = 176.5 mm = 175 mm.	Spacing = 175 mm Spacing = 175 mm

	Top reinforcement bars are not need for both X, Y directions since there are no top tension members in those directions. So, according to the calculations 17 and 34 bars are required for Bottom reinforcement of X and Z directions respectively with spacing of 175 mm.	

<u>Adequacy of Struts</u>

EC2 page 44

<u>AP = CQ = ER = GS members</u>

Strut force = 1336.77 KN

Maximum allowable stress in a strut (σ_{RD} max) = k2*ύ*f_{Ecd} (CTT node)

k 3= 0.75, ύ = [1-(f_{ck}/250)] and f_{Ecd} = 0.85*f_{ck}/1.5

So, σ_{RD} max = 0.75*[1-(32/250)] *0.85*32/1.5 = 11.86 MPa

Stress in the strut = 1336.77*1000/ π*300^2 = 4.73 MPa > 11.86 MPa - OK

Likewise, all struts were checked. So, all the compression members are satisfactory.

Table D.9 Strut analysis for each compression member

Member	Compression force (KN)	stress in the strut (Mpa)	σ_{RD} max (Mpa)
AP=CQ=ER = GS	1376.77	4.73	11.86
BP=BQ =FR = FS	850.36	3.01	11.86
DQ=DR=HS=HP	688.39	2.43	11.86
PQ = RS	69.92	0.07	15.81
QR = SP	342.24	0.34	15.81

	PILE DESIGN Design Method based on EC7 with PDCPB (Pile Design and Construction Practice book fifth edition, 2008)	
EC2 page 44	Since, the exact rock type and its properties and details are not available, here assumed worst case of having weak rock in that area. According to the longitudinal profile Soil layer is 5m. Assumptions • Rock type is weak jointed cemented mudstone. • Soil type is loose sand and organic soils since river bed. • Rock Quality Designation (RQD) values increased from an average of 15% to 35%. • Average unconfined compression strength, q_{uc} is 4.5 MPa and modulus ratio of a cemented mudstone is 150 MPa Design problem, • No of piles = 9 • Design pile length = 10 m • Max. factored reaction on the pile = 2503.21 k N • So, required pile reaction = 2600 k N (design working load) • Pile type is Bored and cast in-situ pile • Concrete grade C32/40 Concrete cube crushing strength is 40 MPa. So, allowable working stress of the concrete = 25% * 40 MPa (BS 8004) = 10 MPa Required pile diameter = $\{(2600*1000*4)/(\pi*10)\}^{0.5}$= 575.36 mm So, selected diameter = 600 mm The stress on the shaft of a 0.6 m pile = $2.6/(\pi*0.6^2/4)$ = 9.19 MPa < 10 MPa – ok Load carried in shaft friction in the loose sand taken as negligible.	Diameter = 600 mm

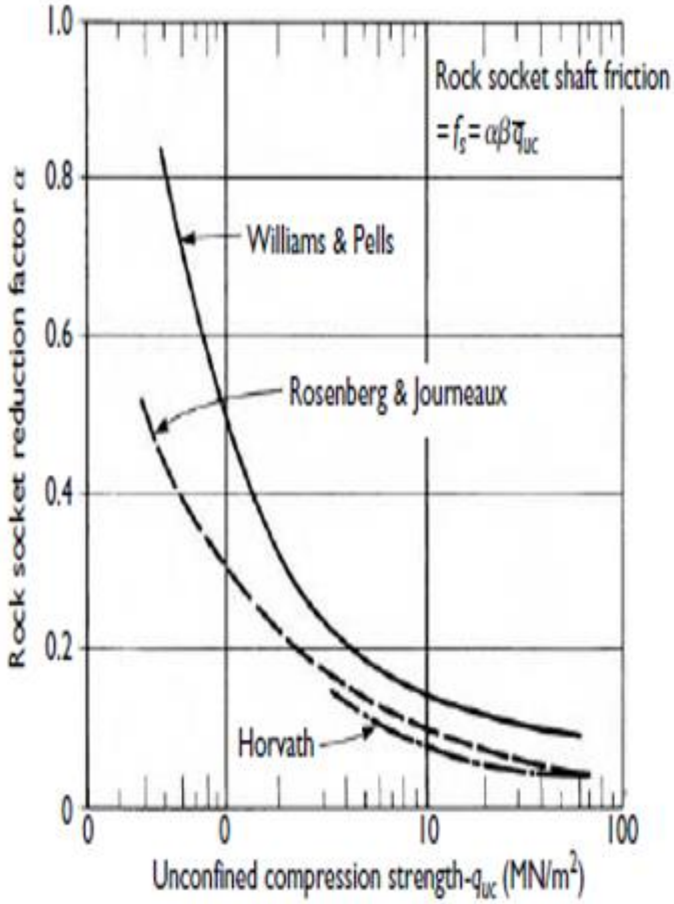


Figure D.6 Reduction factors for rock socket shaft friction

PDCPB page 230	Figure D.6 was taken from 'Pile Design and Construction Practice book, fifth edition', by M. Tomlinson and J. Woodward, 2008, New York, p. 207.

From, figure D.6 and q_{uc} = 4.5 MPa

so, rock socket reduction factor, α = 0.2

since RQD 15% - 35%, the mass factor, j = 0.2

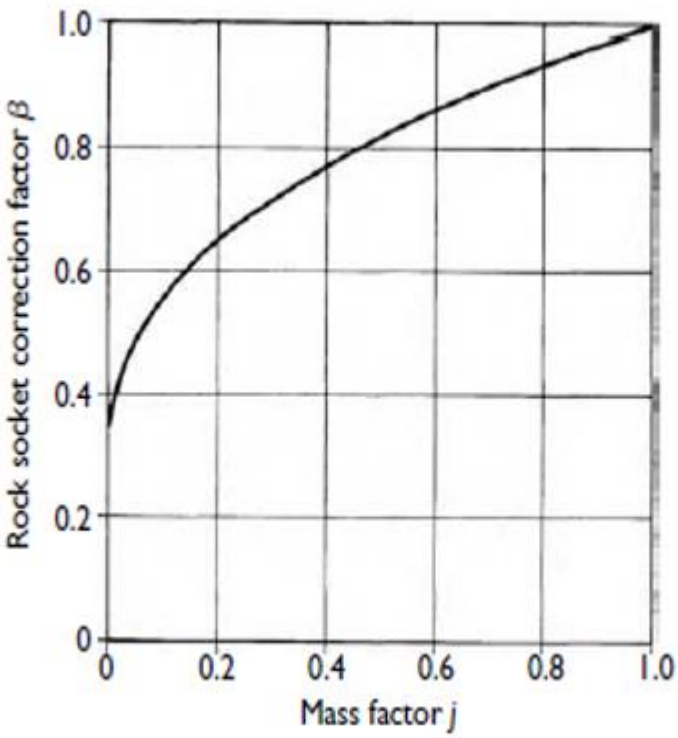


Figure D.7 Reduction factors for discontinuities in rock mass

Figure D.7 was taken from 'Pile Design and Construction Practice book, fifth edition', by M. Tomlinson and J. Woodward, 2008, New York, p. 207.

Therefore, from Figure 7,

rock socket correction factor, $\beta = 0.65$

rock shaft friction, $f_s = \alpha\beta q_{uc}$

$f_s = 0.2*0.65*4.5 = 0.585$ MPa

Assume, Socket length = 5 m

Ultimate shaft friction of the pile, = 585*π*0.6*5 = 5513.495 k N

Therefore, factor of safety on shaft friction = 5513.495/2600 = 2.12 >1

FOS is satisfactory.

Total base resistance = $\pi*0.6^2*4500/4$ = 1272.35 k N

So, total pile resistance = 5513.49 + 1272.35 = 6785.84 k N

Total factor of safety of the pile = 6785.84/2600 = 2.61 > 1

FOS is satisfactory.

Deformation modulus of the rock mass = 0.2*150*4.5 = 135 MPa

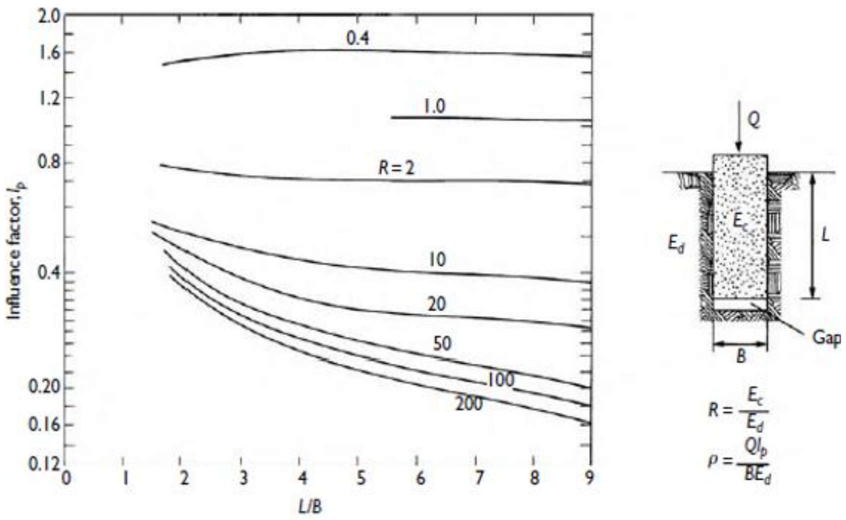
R= E_C / E_D = 33314 / 135 = 246.77

L / B = 5 m / 0.6 m = 8.33

So, from figure 16, influence factor = 0.18

According to the EC 7, up to the base diameter 600 mm settlement should not exceed 10 mm.

FOS = 2.12



PDCPB 4.7.3 page 207

FOS = 2.61

Figure D.8 Elastic settlement influence factors for rock-socket shaft friction on piles

Figure D.8 was taken from ‘Pile Design and Construction Practice book, fifth edition’, by M. Tomlinson and J. Woodward, 2008, New York, p. 213

Settlement of the pile $\rho = Q\,I_P / B\,E_d$

ρ= 2600*0.18/ 0.6*135

ρ= 5.78 mm < 10 mm

PDCPB 4.7.4 page 211

ρ= 5.78 mm

Which is satisfactory.

So, 0.6 m diameter bored cast in-situ concrete pile having length of 10 m is satisfactory for the design pile foundation.

EC2 table 12.2 page 76	**Reinforcement for the piles** Bored piles not exceeding 600 mm in diameter should have the minimum reinforcement. A minimum of six longitudinal bars with diameter of at least 16 mm should be provided with a maximum and minimum spacing of 200 mm and 100 mm around the periphery of the pile. Longitudinal bar = 25 mm Outer rings = 10 mm > 6 mm -OK Cover = 75 mm Cross section area of the pile Ac = $\pi*300^2 = 0.283\ m^2 < 0.5\ m^2$ So required minimum longitudinal reinforcement area As = 0.005*Ac = 1413.72 mm^2 Required minimum longitudinal bars = 2.88 (should at least of 6 bars) So, use 8 H25 bars, As provided = 3926.99 mm^2 Spacing of the longitudinal bars = $2*\pi*(300-75-25-10)/8$ = 149.23 = 150 mm < 200 mm -OK Spacing of rings = 150 mm **Scour depth of the Pier** Melville and Sutherland (1988) suggested an equation to find depth of scour is shown below. $$ds = Kl * Kd * Ky* K\alpha L * Ks *b$$ Where ds is the scour depth, Kl is the coefficient of velocity, Kd is the coefficient of sediment size, Ky is the coefficient for flow depth, Ks is correction factor for pier nose shape, KαL is correction factor for the angle of attack flow, b is the pier width and Fr1 is the Froude number at upstream of the pier. L is the pier length.	Spacing = 150 mm

L =4 m, b = 1 m, y = 4 m, Vo = 5 m/s

Assume d50 = 1 mm, dmax = 2 mm

$Fr1 = v/(gy)^{0.5} = 0.79$

Ks = 1.1 (rectangular pier shape)

L/b = 4, so KαL max = 2.5 (when angle is 90)

b/ d50 = 1000/1 = 1000 > 25, So, Kd = 1.0

y/ b = 4/ 1 = 4 > 2.6, So, Ky = 1.0

X = [Vo – (Va – Vc)]/ Vc

Here Vo is flow velocity, Va is critical velocity of the armour layer and Vc is the critical velocity. Vca is the critical velocity of the armour layer. U*c is the critical shear velocity. y is flow depth at the upstream of the pier, d50 is median grain size of bed material, dmax is the maximum grain size of bed material.

Vc = 5.75*(U*c) * log (5.53*y/d50) = 0.625 m/s

Vca = 5.75*(U*c) * log (5.53*y/d50a)

d 50a = dmax / 1.8

Va = 0.8 Vca, if Va > Vc (otherwise Va= Vc)

Va = 0.529 m/s

If X > 1, Kl = 2.4

If X < 1, Kl = 2.4 |X|

Kl = 2.4 ([Vo – (Va – Vc)]/ Vc > 1) in here Vc =0.625 m/s, Va = 0.529 m/s

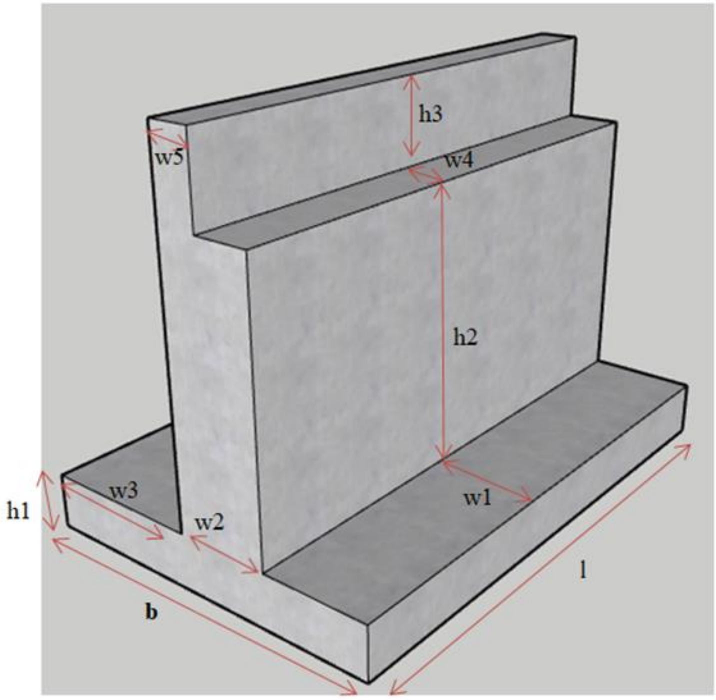
So from equation 1, ds = 2.4*1*1*2.5*1.1*1 = 6.6 m

ds /y = 6.6/4 =1.65

It is recommended that the limiting value of ds/y is 2.4 for Fr1≤ 0.8 and 3.0 for Fr1 >0.8.

Fr1 <0.8 and ds/y < 2.4, Hence scour depth with pier shape is ok.

APPENDIX E ABUTMENT DESIGN

REFERENCES	CALCULATIONS	RESULTS
	E.1 Left abutment design Height of the abutment should be 10.5m according to longitudinal profile of the site. Thus, the abutment dimensions were assumed as figure E.1.  **Figure E.1 Dimensions of the left abutment** Dimensions, h1 = 1m h2 = 8m h3 = 1.5m W1 = 3m W2 = 1.75m W3 = 5m W4 = 1m W5 = 0.75m	

Material properties,

Cohesion(C) = 20KPa

Friction angle(Ø) = 28˚

Specific gravity of soil(ϒsoil) = 20KN/m³

Specific gravity of concrete(ϒcon) = 24KN/m³

Self weight of the abutment,

W1 = 180 KN

W2 = 3360 KN

W3 = 2340 KN

Load from the soil,

Wsoil = 9500 KN

Pa1 = 5625*Ka KN

Pa2 = 4500*Ka KN

Pa3 = 458.55*Ka KN

Surcharge,

Pq = 26.25×Ka KN

Up thrust,

Pu = 1434.71 KN

Water pressure,

Pw1 = 441.45 KN

Pw2 = 441.45 KN

Load from the deck,

Self weight of the deck(Fg) = 314.345 KN

Super imposed load(Fq) = 31.25 KN

Traffic load(Ft) = 530.48 KN

Acceleration force(Fax) = 470.88 KN

Breaking force(Fbx) = 452.4 KN

Consider the 3 combinations,

1. Design Approach 01/combination 01(A1+M1+R1)
2. Design Approach 01/combination 02(A2+M2+R1)

3. Design Approach 02/combination 01(A1+M1+R2)

1. Design Approach 01/combination 01(A1+M1+R1)

ϒG,dst = 1.35

ϒG,stb = 1

ϒQ,dt = 1.5

ϒQ' = 1

ϒc' = 1

ϒr = 1

ϒRv = 1

ϒRh = 1

Design material properties,

Cd$(C/\gamma c)$ = 20 Kpa

Ød$(\tan^{-1}(\tan \emptyset/\gamma\emptyset))$ = 28˚

ϒsoil,d (ϒsoil/ϒϒ) = 18 KN/m³

Kad $\left(\tan(\frac{\pi}{4} - \emptyset d/2)^2\right)$ = 0.361

Table E.1 Design of actions and bending moments for DA1/COM1,

Action	Value (KN)	Lever arm (m)	Moment (KNm)	Remark
W1d (=W1×ϒG,stb)	180	4.375	787.5	Stabilizing
W2d (=W2×ϒG,stb)	3360	3.875	13020	Stabilizing
W3d (=W3×ϒG,stb)	2340	4.875	11407.5	Stabilizing
Wsoil,d (=Wsoil×ϒG,stb)	9500	7.25	68875	Stabilizing
Pa1,d (=Pa1×ϒG,dst)	2741.344	5.5	15077.39	Destabilizing

Pa2,d (=Pa2×ϒG,dst)	2193.075	1.5	3289.61	Destabilizing
Pa3,d (=Pa1×ϒG,dst)	223.4743	1	223.47	Destabilizing
Pqd (=Pq×ϒQ,dst)	14.214	5.25	74.625	Destabilizing
Pw1,d (=Pw1×ϒG,dst)	595.9575	1	595.9575	Destabilizing
Pw2,d (=Pw2×ϒG,dst)	595.9575	1	595.9575	Stabilizing
Pu,d (=Pu×ϒG,dst)	1936.862	6.5	12589.602	Destabilizing
Fg,d (=Fg×ϒG,stb)	314.34	3.5	1100.2075	Stabilizing
Fq,d (=Fq×ϒG,stb)	31.25	3.5	109.375	Stabilizing
Ft,d (=Ft×ϒG,stb)	530.48	3.5	1856.68	Stabilizing
Fax,d (=Fax×ϒG,dst)	635.688	9	5721.192	Destabilizing
Fbx,d (=Fbx×ϒG,dst)	610.74	9	5496.66	Destabilizing

Stabilizing moment(Mstb) = 97752.22 KNm

Destabilizing moment(Mdst) = 43068.51 KNm

Over design factor(ꞁ=Mstb/Mdst) = 2.26 **~ OK**

ꞁ > 1

Bearing check

M1(=Mstb× ϒG,dst) = 131965.5 KNm

M2(=(Mdst- M,Pqd)× ϒG,dst+ M,Pqd× ϒQ,dst) = 58153.69 KNm

Vertical force(Rvd) = 19330.94 KN

	Horizontal force(Hvd) = 8645.833 KN x′ (= $(M1 - M2)/Rvd$) = 3.818325 m e $(= \frac{b}{2} - x')$ = 1.056675 m b/6 = 1.625 m $$e < \frac{b}{6} \sim \text{middle third rule is ok.}$$	e < b/6
	B´ $(= B - 2e)$ = 7.636651 m L´$(= L - 2e)$ = 10 m A´(=$B' \times L'$) = 76.36651 m² q´ = 210 KN/m²	
	Terzaghi's BC equations $$\frac{R}{A} = C'Nc \times bc \times Sc \times ic + q' \times Nq \times bq \times Sq \times iq + 0.5\Upsilon' B' \times Nr \times br \times Sr \times ir$$	
	Nc = 25.803 Nq = 14.72 Nr = 14.59 Bc = 1 Bq = 1 Br = 1 Sc = 1.352 Sq = 1.328 Sr = 0.79024 Ic = 0.5295 Iq = 0.5615 Ir = 0.3904 R/A 3018.233 KN	

	R 230491.9 KN Rd 230491.9 KN $\frac{\text{Rd}}{\text{Rvd}}$ = 11.9235 > 1 ~ Bearing check is ok. **Sliding check** Rhd = 8645.833 Rvd = 19330.94 $\sigma = \emptyset$ = 28˚ $a = C'd \times b \times l$ = 1950 KN Design resistance = $a + Rvd \times \tan\alpha$ = 12228.4 KN Over design factor,ꓶ = $\frac{design\ resistance}{Rhd}$ = 1.4143 > 1 ~ Sliding is ok. **2. <u>Design Approach 01/combination 02(A2+M2+R1)</u>** ϒG,dst = 1 ϒG,stb = 1 ϒQ,dt = 1.3 ϒQ' = 1.5 ϒc' = 1.25 ϒr = 1 ϒRv = 1 ϒRh = 1 Design material properties, Cd$(C/\gamma c)$ = 16 Kpa Ød$(\tan^{-1}(\tan\emptyset/\gamma\emptyset))$ = 18˚ ϒsoil,d (ϒsoil/ϒϒ) = 20 KN/m³ Kad $\left(\tan(\frac{\pi}{4} - \emptyset d/2)^2\right)$ = 0.515	ꓶ > 1

Table E.2 Design of actions and bending moments for DA1/COM2,

Action	Value (KN)	Lever arm (m)	Moment (KNm)	Remark
W1d (=W1×ϒG,stb)	180	4.375	787.5	Stabilizing
W2d (=W2×ϒG,stb)	3360	3.875	13020	Stabilizing
W3d (=W3×ϒG,stb)	2340	4.875	11407.5	Stabilizing
Wsoil,d (=Wsoil×ϒG,stb)	9500	7.25	68875	Stabilizing
Pa1,d (=Pa1×ϒG,dst)	2896.9	5.5	15932.81	Destabilizing
Pa2,d (=Pa2×ϒG,dst)	2317.5	1.5	3476.25	Destabilizing
Pa3,d (=Pa1×ϒG,dst)	236.15	1	236.153	Destabilizing
Pqd (=Pq×ϒQ,dst)	17.574	5.25	92.265	Destabilizing
Pw1,d (=Pw1×ϒG,dst)	441.45	1	441.45	Destabilizing
Pw2,d (=Pw2×ϒG,dst)	441.45	1	441.45	Stabilizing
Pu,d (=Pu×ϒG,dst)	1434.71	6.5	9325.63	Destabilizing
Fg,d (=Fg×ϒG,stb)	314.34	3.5	1100.21	Stabilizing
Fq,d (=Fq×ϒG,stb)	31.25	3.5	109.38	Stabilizing

Ft,d (=Ft×ϒG,stb)	530.48	3.5	1856.68	Stabilizing
Fax,d (=Fax×ϒG,dst)	470.88	9	4237.92	Destabilizing
Fbx,d (=Fbx×ϒG,dst)	452.4	9	4071.6	Destabilizing

Stabilizing moment(Mstb) = 97597.7125 KNm

Destabilizing moment(Mdst) = 37814.082 KNm

Over design factor(ℸ=Mstb/Mdst) = 2.58 **~ OK** $ℸ > 1$

Bearing check

M1(=Mstb× ϒG,dst) = 97597.7 KNm

M2(=(Mdst- M,Pqd)× ϒG,dst+ M,Pqd× ϒQ,dst) = 37841.8 KNm

Vertical force(Rvd) = 14821.4 KN

Horizontal force(Hvd) = 6373.81 KN

x′ (= $(M1 - M2)/Rvd$) = 4.0317 m

e $(= \frac{b}{2} - x')$ = 0.8432 m

b/6 = 1.625 m

$$e < \frac{b}{6} \sim \text{middle third rule is ok.}$$ $e < b/6$

B′ $(= B - 2e)$ = 8.06349 m

L′$(= L - 2e)$ = 10 m

A′(=$B' \times L'$) = 80.6349 m²

q′ = 210 KN/m²

Terzaghi's BC equations

$$\frac{R}{A} = C'Nc \times bc \times Sc \times ic + q' \times Nq \times bq \times Sq \times iq + 0.5ϒ'B' \times Nr \times br \times Sr \times ir$$

Nc = 13.64

Nq = 5.61

	Nr = 3.115 Bc = 1 Bq = 1 Br = 1 Sc = 1.28 Sq = 1.23 Sr = 0.781 Ic = 0.522 Iq = 0.6074 Ir = 0.442 R/A = 1112.74 KN/m² R = 89725.9 KN Rd = 89725.9 KN $\frac{\mathrm{Rd}}{\mathrm{Rvd}}$ = 6.05381 > 1 ~ Bearing check is ok. **Sliding check** Rhd = 6373.81 KN Rvd = 14821.4 KN $\sigma = \emptyset$ = 18.67° $a = C'd \times b \times l$ = 1560 KN Design resistance = $a + Rvd \times \tan\alpha$ = 6567.15 KN Over design factor,˥ = $\frac{design\ resistance}{Rhd}$ = 1.03033 > 1	˥ > 1 ˥ > 1

3. **Design Approach 02/combination 01(A1+M1+R2)**

Design factors,

ϒG,stb = 1

ϒQ,ds

t = 1.5

ϒQ' = 1

ϒc' = 1

ϒr = 1

ϒRv = 1.4

ϒRh = 1.1

Design material properties,

Cd$(C/\gamma c)$ = 20 Kpa

Ød$(\tan^{-1}(\tan \emptyset/\gamma\emptyset))$ = 28°

ϒsoil,d (ϒsoil/ϒϒ) = 20 KN/m³

Kad $\left(\tan(\frac{\pi}{4} - \emptyset d/2)^2\right)$ = 0.361

Table E.3 Design of actions and bending moments for DA2/COM1

Action	Value (KN)	Lever arm (m)	Moment (KNm)	Remark
W1d (=W1×ϒG,stb)	180	4.375	787.5	Stabilizing
W2d (=W2×ϒG,stb)	3360	3.875	13020	Stabilizing
W3d (=W3×ϒG,stb)	2340	4.875	11407.5	Stabilizing
Wsoil,d (=Wsoil×ϒG,stb)	9500	7.25	68875	Stabilizing
Pa1,d (=Pa1×ϒG,dst)	2741.34	5.5	15077.39	Destabilizing

Pa2,d (=Pa2×ϒG,dst)	2193.08	1.5	3289.61	Destabilizing
Pa3,d (=Pa1×ϒG,dst)	223.474	1	223.47	Destabilizing
Pqd (=Pq×ϒQ,dst)	14.2144	5.25	74.625	Destabilizing
Pw1,d (=Pw1×ϒG,dst)	595.958	1	595.9575	Destabilizing
Pw2,d (=Pw2×ϒG,dst)	595.958	1	595.9575	Stabilizing
Pu,d (=Pu×ϒG,dst)	1936.86	6.5	12589.60	Destabilizing
Fg,d (=Fg×ϒG,stb)	314.345	3.5	1100.21	Stabilizing
Fq,d (=Fq×ϒG,stb)	31.25	3.5	109.38	Stabilizing
Ft,d (=Ft×ϒG,stb)	530.48	3.5	1856.68	Stabilizing
Fax,d (=Fax×ϒG,dst)	635.688	9	5721.192	Destabilizing
Fbx,d (=Fbx×ϒG,dst)	610.74	9	5496.66	Destabilizing

Stabilizing moment(Mstb) = 97752.22 KNm

Destabilizing moment(Mdst) = 43068.514KNm

Over design factor(˥=Mstb/Mdst) = 2.2697~ **OK** ˥ > 1

Bearing check

M1(=Mstb× ϒG,dst) = 131965 KNm

M2(=(Mdst- M,Pqd)× ϒG,dst+ M,Pqd× ϒQ,dst) = 58153.7 KNm

Vertical force(Rvd) = 19330.9 KN

Horizontal force(Hvd) = 8645.83 KN

x′ (= $(M1 - M2)/Rvd$) = 3.81833 m

e ($= \frac{b}{2} - x'$) = 1.05667m

b/6 = 1.625 m

e < b/6

$$e < \frac{b}{6} \sim \text{middle third rule is ok.}$$

B´ ($= B - 2e$) = 7.63665 m

L´($= L - 2e$) = 10 m

A´(=$B' \times L'$) = 76.3665 m²

q´ = 210KN/m²

Terzaghi's BC equations

$$\frac{R}{A} = C'Nc \times bc \times Sc \times ic + q' \times Nq \times bq \times Sq \times iq + 0.5\Upsilon'B' \times Nr \times br \times Sr \times ir$$

Nc	= 25.803
Nq	= 14.72
Nr	= 14.59
Bc	= 1
Bq	= 1
Br	= 1
Sc	= 1.352
Sq	= 1.328
Sr	= 0.79024
Ic	= 0.5295
Iq	= 0.5615
Ir	= 0.3904

R/A	= 3018.23 KN/m²
R	= 230492 KN
Rd	= 164637 KN

	$\frac{\text{Rd}}{\text{Rvd}}$ = 8.51677 > 1 ~ Bearing check is ok. **Sliding check** Rhd = 7859.85 KN Rvd = 19330.9 KN $\sigma = \emptyset$ = 28˚ $a = C'd \times b \times l$ = 1950 KN Design resistance = $a + Rvd \times \tan\alpha$ = 12228.4KN Over design factor,Ꞁ = $\frac{design\ resistance}{Rhd}$ = 1.55581 > 1 ~ Sliding is ok. . **Settlement (SLS)** Settlement is the movement of the foundation in downward direction due to the load of entire structure over it. Settlements are usually checked under the vertical load (Q) obtained with quasi-permanent SLS combinations. (EUR 25193 EN - 2012) **S** **=** $(\boldsymbol{q} - \sigma_{vo}) * [\frac{\boldsymbol{2B_0}}{\boldsymbol{9E_d}} * \left(\frac{\boldsymbol{\lambda_d B}}{\boldsymbol{B_0}}\right)^{\boldsymbol{a}} + \frac{\alpha\boldsymbol{\lambda_c}}{\boldsymbol{9E_c}}]$ Bo is a reference width of 0.6 m B is the width of the foundation λ_d , λ_c are shape factors α is a rheological factor Ed is the weighted value of EM immediately below the foundation Ed is the harmonic mean of EM in all layers up to 8 x B below the foundation σ_{vo} is the total (initial) vertical stress at the level of the foundation base q is the design normal pressure applied on the foundation	Ꞁ > 1

Gk,1 - Self weight of the half of the deck

Gk,2 - Non-structural load

Gk,1- 314.375kN Gk,2- 31.25kN B=12.25m L=10m

Q = (Gk,1 + Gk,2) *2

= 691.25kN

Finding unknown parameters

q = Q/BL

= 5.64kPa

For preliminary rough estimation can be done by assuming a homogeneous soil with Ec=Ed and σ_{vo} = 0. Considering worst case density index was selected as “Very loose” soil. For very loose soil, drained young’s modules (E) should be less than 10Mpa (EN 1997-2:2007, Table D.1). Therefore, it was assumed as 6MPa.

And there are 30.4% of clay, 56.7% of slit in that soil according to data from the geo laboratory. Therefore, soil type was found using the soil textural triangle.

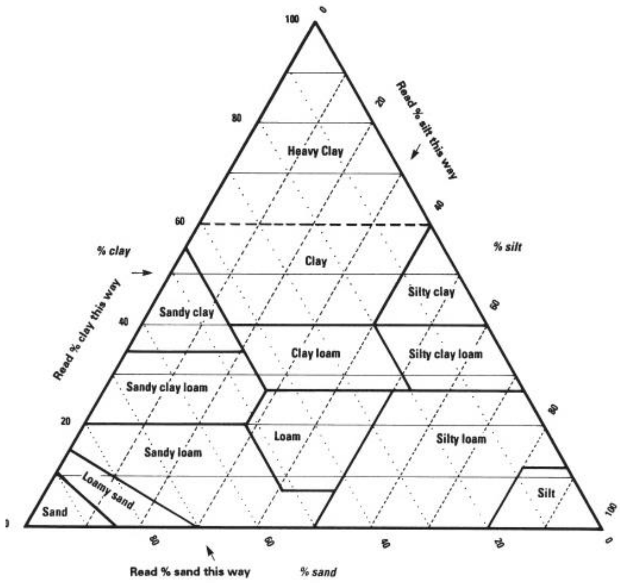


Figure E.2 Soil textural triangle

From soil textural triangle, soil type is identified as silty clay loam soil. Using the soil type and assuming this is a normally consolidation soil, rheological factor (α) can be identified as 0.5 (En1997-2, Annex E).

There are some shape coefficients, for settlement of spread foundations λ_d, λ_c .(En1997-2,Annex E)

Table E.4 Shape coefficients

L/B	Circle	Square	2	3	5	20
λ_d	1	1.12	1.53	1.78	2.14	2.65
λ_c	1	1.1	1.2	1.3	1.4	1.5

L/B = 1.225

Therefore, $\lambda_d = 1.212$

$\lambda_c = 1.045$

Settlement of the abutment footing (according to the eq.1) = 0.068cm <3cm (Allowable settlement)

All combinations are satisfied for bearing and sliding check for abutment. Since, this footing is existing on the soil and bed rock is very near to the footing, a pile foundation was selected as the foundation type.

1.1.2 Reinforcement design for Abutment (@left side)

Load cases,

1.Deck force + lateral soil pressure + surcharge

2.Lateral soil pressure + surcharge

3.Deck load + surcharge

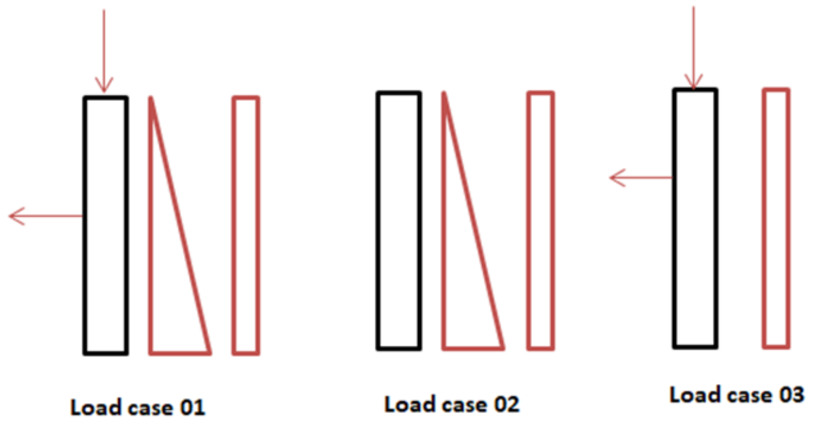


Figure E.3 Load cases acting on the left abutment wall

Using bending moment diagram,

Table E.5 values of bending moments for load cases for left abutment

Load cases	Combination 01 (KNm)		Combination 02 (KNm)	
	permanent	variable	permanent	variable
Case 01	4469.49	16.92	3310.77	14.67
Case 02	3459.58	16.96	2572.13	14.67
Case 03	997.12	16.92	738.64	14.67

SLS bending moment and shear,

Permanent = 3310.77 KNm

Variable = 11.28 KNm

Shear force = 906.95 KN

EN 1992-1-1 clause 5.8.3.1

Check the slenderness of abutment wall

Consider the rear face for the design.

Λ $= l°/i \leq 20ABC/\sqrt{n}$

A = 0.7

B = 1.1

C = 0.7

N $= \frac{Ned}{Ac*fcd}$

Fcd $= \propto cc * \frac{fck}{\gamma c}$

$= 0.85 * \frac{30}{1.5}$

$= 17\ N/mm^2$

Ac $= 1.75*10^3*10$

$=1.75*10^6$

∴n $= \frac{87.6075*10^3}{1.75*10^6*17}$

$= 2.944*10^{-3}$

	Λlim $= \frac{20*0.7*1.1*0.7}{\sqrt{(2.944*10^{-3}}}$ = 198.678 $l°$ = 2*l = 2*8 = 16m I $= \frac{\sqrt{1}}{12}$ = 0.289m Λ = 16/0.289 = 55.363 < λlim ~ OK. Second order effect need not be considered. Now design reinforcement concrete for the ULS and check for serviceability condition. MULS = 4486.41KNm MSLS = 3322.05 KNm Med = 4486.41 KNm D = 1750-50-32/2 = 1684 mm K $= \frac{Med}{bd^2*fck}$ $= \frac{4486.41}{1*1684^2*30}$ = 0.0527 Z $= d\left[0.5 + \sqrt{0.25 - 0.88k}\right]$ $= 1684\left[0.5 + \sqrt{0.25 - 0.88*0.0527}\right]$ = 1601.9 mm As $= \frac{Med}{0.87*fyk*z}$ $= \frac{4486.41}{0.87*500*1601.9}$ = 6438.346 mm²	
EN 1992-1-1 & EN 1992-2		

No of bars $= \frac{6438.346}{\frac{\pi}{4}*32^2}$

= 8.005

Spacing = 1000/8 mm

= 125 mm

Use the 32mm bar @125mm for main bar for the wall.

Check serviceability limit state

Characteristic combination SLS design moment = 3322.05 KNm

Check stresses in the concrete and reinforcement at,

i. Early age
ii. Long term

1.Early age (before creep has occurred)

1992-1-1 table 3.1

Ecm = 33 KN/mm^2

Ec,eff = 33 KN/mm^2

Module ratio,m $= \frac{Ec}{Em}$

= 200/33

= 6.06

Dc=depth to neutral axis then equating strain for cracked section.

Ξs $= \frac{\xi c(d-dc)}{dc}$

Dc $= \frac{\left[-As*Es+\left\{(As*Es)^2+2b*As*Es*Ec,eff*d\right\}^2\right]}{b*Ec,eff}$

= 323.494mm

Cracked second moment area $= As(d-dc)^2 + \frac{Ec,eff*b*dc^3}{3Ec}$

$=6433.98(1684-323.494)^2 + \frac{33*10^3*323.494^3}{3*200}$

= $13.77*10^9$ mm^4

Approximate concrete stress,

Σc $= \frac{M}{Zc} + \frac{N}{Ac}$

M = 3322.05 KNm

N = 87.61 KN

Ac = dc*b

= 323.494*10³ mm²

Σc $= \frac{3322.05*10^6*323.494}{13.77*10^9*6.06} + \frac{87.61}{323.494}$

= 13.149 N/mm²

Limiting concrete stress = K1*fck

= 0.6*30

= 18 N/mm² > σc ~ OK.

2.After all creep has taken place,

The cracked section properties are based on the long term and short term modulus for various action.

Short term modulus $= E_{cm}$

Long term modulus $= \frac{Ecm}{1+\vartheta}$

Table 3.1 EN 1992-

Effective modulus $= \frac{(Mqp+Mst)Ecm}{Mst+(1+\vartheta)Mqp}$

f_{cm} = 38 N/mm²

relative humidity of Kandy area = 80%

age of concrete at initial loading = 7 days

$h_\circ$ $= \frac{2Ac}{U}$

U = perimeter of the number in contact with the atmosphere.

$h_\circ$ $= \frac{2*8000*1000}{2*(8000+1000)}$

=888.89 mm

α1 $= \left(\frac{35}{fcm}\right)^{0.7}$

$= \left(\frac{35}{38}\right)^{0.7}$

$= 0.944$

α2 $= \left(\frac{35}{fcm}\right)^{0.2}$

$= \left(\frac{35}{38}\right)^{0.2}$

$= 0.983$

α3 $= \left(\frac{35}{fcm}\right)^{0.5}$

$= \left(\frac{35}{38}\right)^{0.5}$

$= 0.959$

~fcm > 35 Mpa

ϑrh $= \left[1 + \frac{\alpha 1\left((1-\frac{RH}{100}\right)}{0.1 * h_{\circ}^{\frac{1}{3}}}\right] * \alpha 2$

$= \left[1 + \frac{0.944\left((1-\frac{80}{100}\right)}{0.1 * 888.89^{\frac{1}{3}}}\right] * 0.983$

$= 1.176$

β(fm) $= \frac{16.8}{\sqrt{fcm}}$

$= \frac{16.8}{\sqrt{38}}$

$= 2.725$

$\beta(t_{\circ})$ $= \frac{1}{0.1+t_{\circ}^{0.2}}$

$= \frac{1}{0.1+7^{0.2}}$=0.634

$\vartheta_{\circ}$ $= \vartheta rh * \beta(fcm) * \beta(t_{\circ})$

$= 1.176 * 2.725 * 0.6346$

$= 2.0336$

Moment due to long term action

Mqp = 3310.77 KNm (from dead load)

Moment due to short term action,

Mst = 11.28 KNm (from live load)

	Effective modulus,Ec,eff = $\frac{(Mqp+Mst)*Ecm}{Mst+(1+\vartheta_{\circ})*Mqp}$ = $\frac{(3310.77+11.28)*33}{11.28+(1+2.03)*3310.77}$ = 10.915 KN/mm² Modular ratio = $\frac{Es}{Ec,eff}$ = 200/10.914 = 18.325 dc=depth to neutral axis ϑ = $\frac{\varepsilon c(d-dc)}{dc}$ Dc = $\frac{\left(-AsEs+\left\{(AsEs)^2+2b*As*Es*Ec,eff*d\right\}^{0.5}\right)}{b*Ec,eff}$ = 523.169 mm Cracked second moment of area, = $As(d-dc)^2+\frac{Ec,eff*b*dc^3}{3*Es}$ = $6433.98(1684-523.169)^2+\frac{10.915*10^3*523.169^3}{3*200}$ = $11.275*10^9$ mm⁴	
cl.7.3.4. EN 1992-1-1	Concrete stress, Σc = $\frac{M}{Zc}+\frac{N}{Ac}$ M = 3322.05KNm N = 87.6075 KN Ac = dc*b = $523.169*10^3$ mm² Σc = $\frac{3322.05*10^6*523.169}{11.275*10^9*18.325}+\frac{87.601}{523.169}$ = 8.579 KN/mm² = K1*fck Limiting concrete stress = 0.6*30 = 18 N/mm² > σc ~ OK.	

Limiting steel stress = $k3 * fyk$

= 0.8*500

= 400 N/mm²

Available steel stress = $\frac{M}{zs}$

= $\frac{1382.34*10^6*(1687.5-395.912)}{6.573*10^9}$

= 271.63 N/mm² < 400

Cl 7.3.4. EN 1992-1-1

Crack control

Consider worst condition before creep has occurred.

Crack width,Wk = $Sr,\max(\varepsilon sm - \varepsilon cm)$

Spacing limit = $5(c + \frac{\emptyset}{2})$

Ac,eff = heff*b = 165*10³ mm²

$\rho p,eff = \frac{As}{Ac,eff}$ = 0.0389

Sr,max = $K3 * C + \frac{K1*K2*K4*\emptyset}{\rho p,eff}$

K4=0.425 (recommended)

Heff=$min\left\{2.5(h-d); \frac{h-x}{3}; \frac{h}{2}\right\}$

=$min\left\{2.5(1750-1684); \frac{1750-323.494}{3}; \frac{1750}{2}\right\}$

=$min\{165; 475.502; 875\}$

= 165 mm

∴Ac,eff= heff*b= 165*10³ mm²

$\rho p,eff = \frac{As}{Ac,eff} = \frac{6433.98}{165*10^3}$= 0.0389

Sr,max=$3.4 * 50 + \frac{0.8*0.5*0.425*32}{0.0389}$

ξsm- ξcm=$\frac{\left(\sigma s - \left\{Kt*fct,\frac{eff(1+\alpha e*\rho p,eff)}{\rho p,ef}\right\}\right)}{Es} \geq 0.6 * \frac{\sigma s}{Es}$

Kt== 0.4; for permanent load

Ae== $\frac{Es}{Ecm} = \frac{200}{33}$=6.06

Table NA2 EN 1992-2-1

σs = $\frac{M(d-dc)}{I} = \frac{3322.05(1684-323.494)}{13.77*10^9}$=328.226N/mm²

fct,eff (=fctm) = 2.9 N/mm²

ξsm- ξcm=$\frac{\left[328.226-\left\{0.4*\frac{2.9(1+6.06*0.0475)}{0.0478}\right\}\right]}{2*10^5}$= 1.356*10⁻³

cl.7.3.4. EN 1992-1-1	$0.6 * \frac{\sigma s}{Es} = 0.6 * \frac{328.226}{2*10^5}$=0.984*$10^{-3}$ < 1.456*10^{-3} ~ OK Crack width, Wk= $Sr, \max(\varepsilon sm - \varepsilon cm)$= 209.845*1.356*$10^{-3}$ Crack width, Wk =0.284 mm Recommended value of Wmax= 0.3 mm ~ OK.	
Table NA2 EN 1992-2-1	Hence 32mm bars @125mm are ok for the rear face of the wall. ∴As,prov = 6433.98 mm² As,min = 0.0013*Ac = 0.0013*1750*10³ = 2275 mm² < As,prov Hence 32mm bars @125mm are ok for the rear face of the wall. ∴As,prov = 6433.98 mm² As,min = 0.0013*Ac = 0.0013*1750*10³ = 2275 mm² < As,prov **Design of vertical bar for front face of the wall,** ∴As = As,min Provide the 25mm bar No of bars $= \frac{As,\min}{\frac{\pi}{4}*25^2}$ $= \frac{2275}{\frac{\pi}{4}*25^2}$ = 4.63 Spacing = $1000/4.63$ = 215.98 mm ≈ 200 mm Hence use the 25mm bars @200mm for the front face of the wall. **Design of horizontal bar for both faces,** ∴As $= max\{0.25 * As, min; 0.001Ac\}$	

$= max\{0.25 * 2275; 0.001 *$

$1750 * 10^3\}$

$= max\{568.75; 1750\}$

= 1750 mm²

Use 25mm bars,

No of bars $= \frac{As}{\frac{\pi}{4} * \emptyset^2}$

$= \frac{1750}{\frac{\pi}{4} * 25^2}$

= 3.565

spacing $= \frac{1000}{3.656}$

= 280.504 mm

≈ 250 mm

Use the 25mm bars @250mm as the horizontal reinforcement bars for both faces

Reinforce design for pile cap,

A pile foundation was designed to the abutment. Thus, footing size has to be changed now,

W1 = 2m

W2 = 2m

W3 = 4m

Consider the footing heel and toe separately

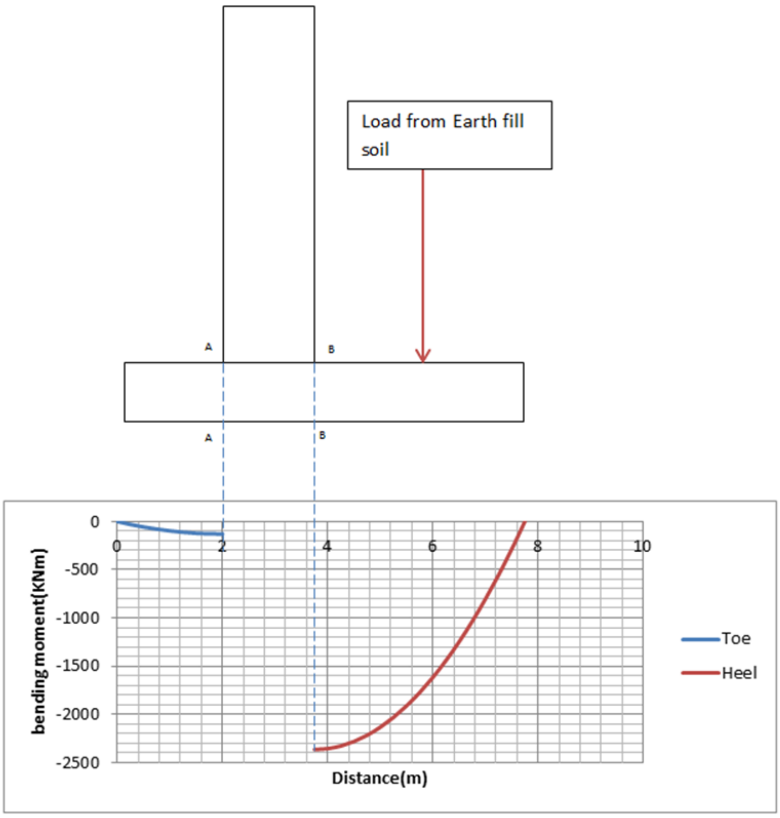


Figure E.4 Bending moment diagram for toe and heel under combination 01

Load cases, (considering 1m length)

combination 01

Backfill weight= (4*9.5*1)*18*1.35= 923.4 KN/m

ULS shear at A-A= 2*2*24*1.35= 129.6KN/m

ULS moment at A-A= 2*2*24*1.35*2/2 =129.6 KNm

~ tension at the bottom of the footing

ULS shear at B-B= 4*2*24*1.35+923.4= 1182.6 KN

ULS moment at B-B= 2*4*24*1.35*4/2 + 923.4*4/2

= 2365.2 KNm ~ tension at the bottom of the footing

Combination 02

Backfill weight= (4*9.5*1) *18*1= 484 KN/m

ULS shear at A-A= 2*2*24*1= 96 KN/m

ULS moment at A-A= 2*2*24*2/2= 96 KNm ~ tension at the bottom of the footing

ULS shear at B-B= 2*4*24*1 + 684= 876 KN

ULS moment at B-B=2*4*24*1*4/2 + 684*4/2= 1752 KNm

~ tension at the bottom of the footing

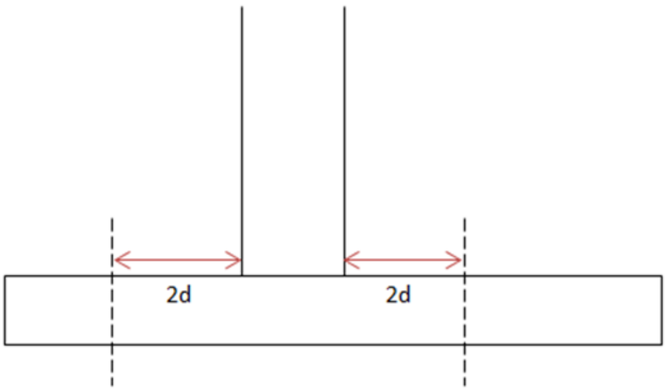
Reinforcement design of pile cap,

bottom reinforcement design (tension has occurred on bottom of the pile cap)

Med	= 2365.2 KNm
B	= 1000 mm
Cover	= 50 mm
Bar size	= 32 mm
Effective depth,d	= 1000-50-32/2
	= 934 mm
K	$= \frac{Med}{fck*b*d^2}$
	$= \frac{2365.2*10^3}{30*1*934^2}$
	= 0.09< 0.167

∴.no need compression reinforcement.

Z	$= d\left[0.5 + \sqrt{0.25 - 0.882 * K}\right]$
	$=d\left[0.5 + \sqrt{0.25 - 0.882 * 0.09}\right]$
	= 852.799 mm
As1	$= \frac{Med}{0.87*fyk*Z}$
	$= \frac{2365.2*10^6}{0.87*500*852.799}$
	= 6375.75 mm² > As,min = 2514.2 mm²
No or r/f bars	$= \frac{As}{\frac{\pi}{4}*\emptyset^2}$
	$= \frac{6375.75}{\frac{\pi}{4}*32^2}$
	= 7.92

	Spacing = 1000/7.92 = 126.26 mm ≈ 125 mm As,prov = 6433.98 mm² As,min = 0.0013*b*d = 0.0013*1000*934 = 1214.2 mm² < As,prov ∴provide 32mm bars @125mm for bottom of the pile cap **Distribution reinforcement design at heel,** As = As,min = 1214.2 mm² Provide 25mm bar, No of bars $= \frac{1214.2}{\frac{\pi}{4}*25^2}$ = 2.47 Spacing = 1000/2.47 = 404.858 mm ≈ 400 mm Provide 25mm bar @400mm for distribution bars **Check for the punching shear,**  **Figure E.5 Control area of the footing** Control perimeter, u $= (2d + 1750 + 2d + 10000)2$	

		= 2(4*937.5+1750+10000) mm	
		= 31000 mm	
	Control area	$= (2d + 1750 + 2d) * 10000$	
		= (4*934+1750)*10000 mm²	
		= $54.86*10^6$ mm²	
	Ved,red	= Ved - ΔVed	
	Ved	= 15628.3-1*7.75*10*24*1.35	
		= 13117.3 KN	
	Ved,red	= 13117.3 – 0	
		= 13117.3 KN	
	ved,red	= Ved,red/u*d	
		$= \frac{13117.3}{31000*934}$	
		= 0.453 N/mm²	
	Shear resistance section,		
	VRd,c	$= \frac{\left[CRd,c*K*(100\rho 1*fck)^{\frac{1}{3}}\right]2a}{d} \geq \frac{Vmin*2d}{a}$	
	K	$= 1 + \sqrt{\frac{200}{d}}$	
		$= 1 + \sqrt{\frac{200}{934}}$	
		= 1.462 < 2	
	Vmin	$= 0.035 * K^{\frac{3}{2}} * \sqrt{fck}$	
		$= 0.035 * 1.462^{\frac{3}{2}} * \sqrt{30}$	
		= 0.3388	
	Vmin*2d/a	$= 0.3388 * \frac{2d}{2d}$	
		= 0.3388	

	ρx(in transverse direction) $= \frac{As}{bd}$ $= \frac{\frac{\pi}{4}*25^2*\frac{7750}{400}}{7750*934}$ $= 1.314*10^{-3}$ ρy(in longitudinal direction) $= \frac{As}{bd}$ $= \frac{\frac{\pi}{4}*32^2*\frac{10000}{125}}{10000*934}$ $= 27.5*10^{-3}$ $\rho 1$ $= \sqrt{\rho x * \rho y}$ $= \sqrt{1.314 * 10^{-3} * 27.5 * 10^{-3}}$ $= 6.011*10^{-3} < 0.02$ VRd,c $= 0.12 * 1.462(100 * 6.011 * 10^{-3} * 30)^{\frac{1}{3}}) * 2 * 2d/2d$ $= 0.92$ KN/mm² vEd,red < VRd,c ~ this section is ok for punching shear.	

Right abutment design

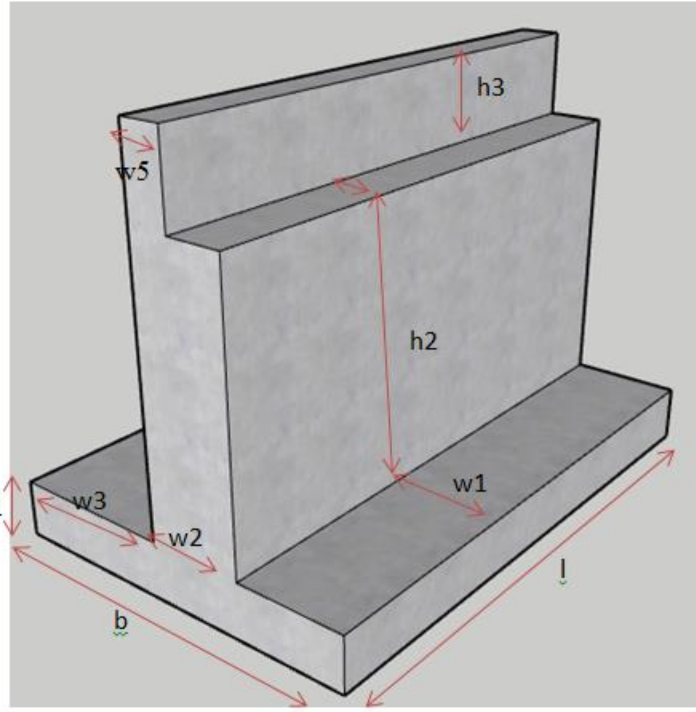
<table>
<tr><th>References</th><th>Calculations</th><th>results</th></tr>
<tr><td></td><td>After considering longitudinal profile of the site and super structure, the right abutment was designed.


Figure E.6 Dimensions of abutment

Dimensions,
h1 = 2m
h2 = 10.5m
h3 = 1.5m
W1 = 2m
W2 = 2m
W3 = 4m
W4 = 1m
W5 = 1m
Material properties,
Cohesion(C) = 20KPa
Friction angle(Ø) = 28˚
Specific gravity of soil(ϒsoil) = 20KN/m³
Specific gravity of concrete(ϒcon) = 24KN/m³</td><td></td></tr>
</table>

Actions,

Self weight of the abutment,

$W1 = 360$ KN

$W2 = 4410$ KN

$W3 = 5880$ KN

Load from the soil,

$W_{soil} = 15600$ KN

$Pa1 = 5625*Ka$ KN

$Pa2 = 9750*Ka$ KN

$Pa3 = 2152.64*Ka$ KN

Surcharge,

$Pq = 35 \times Ka$ KN

Up thrust,

$Pu = 3905.61$ KN

Water pressure,

$Pw1 = 2072.36$ KN

$Pw2 = 2072.36$ KN

Load from the deck,

Self weight of the deck(Fg) = 314.345 KN

Super imposed load(Fq) = 31.25 KN

Traffic load(Ft) = 530.48 KN

Acceleration force(Fax) = 470.88 KN

Breaking force(Fbx) = 452.4 KN

1.2.1 Reinforcement design for Abutment

Load cases,

1.Deck force + lateral soil pressure + surcharge

2.Lateral soil pressure + surcharge

3.Deck load + surcharge

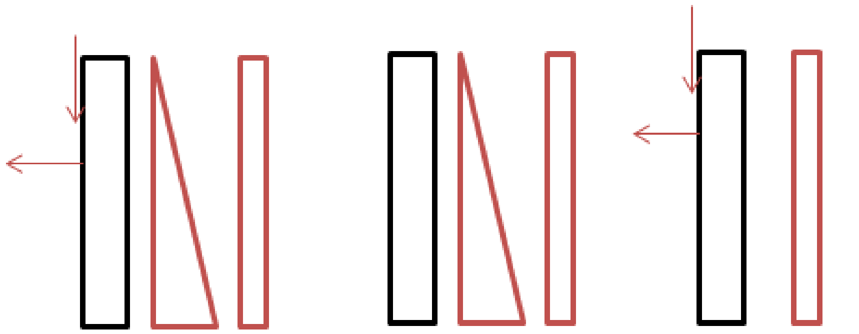


Figure E.7 Load cases for right abutment wall

Using bending moment diagram,

Table E.6 Values of bending moments for right abutment

Load cases	Combination 01 (KNm)		Combination 02 (KNm)	
	permanent	variable	permanent	variable
Case 01	8307.1	27	6153.5	23.4
Case 02	6998.4	27	5184	23.4
Case 03	1308.72	27	969.47	23.4

SLS bending moment = 6171.47 KNm

shear force = 1391.33 KN

Check the slenderness of abutment wall

Consider the rear face for the design,

EN 1992-1-1 clause 5.8.3.1

$\Lambda \quad = l°/i \leq 20ABC/\sqrt{n}$

A = 0.7

B = 1.1

C = 0.7

N $= \frac{Ned}{Ac*fcd}$

Fcd $= \propto cc * \frac{fck}{\gamma c}$

		$= 0.85 * \frac{30}{1.5}$	
		= 17 N/mm²	
	Ac	= $1.75*10^3*10$	
		=$1.75*10^6$	
	∴n	$= \frac{87.6075*10^3}{1.75*10^6*17}$	
		= $2.944*10^{-3}$	
	λlim	$= \frac{20*0.7*1.1*0.7}{\sqrt{(2.944*10^{-3}}}$	
		= 198.678	
EN 1992-1-1 & EN 1992-2		= 2*l	
	$l°$	= 2*12	
		= 24m	
	I	$= \frac{\sqrt{1}}{12}$	
		= 0.289m	
		= 24/0.289	
	Λ	= 83.045 < λlim	λ< λlim
		~ OK. second order effect need not be considered.	

Now design reinforcement concrete for the ULS and check for serviceability condition.

	MULS	= 8334.1 KNm	
	MSLS	= 6171.47 KNm	
	Med	= 8334.1 KNm	
	D	= 2000-50-32/2	
		= 1934 mm	
	K	$= \frac{Med}{bd^2*fck}$	
		$= \frac{8334.1}{1*1934^2*30}$	
		= 0.0743	
	Z	$=d\left[0.5\sqrt{0.25-0.88k}\right]$	

$=$

$$1934\left[0.5\sqrt{0.25 - 0.88 * 0.0743}\right.$$

$= 1797.98$ mm

As $= \frac{Med}{0.87 * fyk * z}$

$= \frac{8334.1}{0.87 * 500 * 1797.98}$

$= 10655.764$ mm²

No of bars $= \frac{10655.764}{\frac{\pi}{4} * 32^2}$

$= 13.24$

Spacing $= 1000/13.24$ mm

$= 75.53$ mm

≈ 75 mm

Use the 32mm bar @75mm for main bar for the wall.

Check serviceability limit state

Characteristic combination SLS design moment = 6171.47 KNm

Check stresses in the concrete and reinforcement at,

iii. Early age

iv. Long term

1.Early age(before creep has occurred)

1992-1-1 table 3.1

Ecm $= 33$ KN/mm²

Ec,eff $= 33$ KN/mm²

Modul e ratio,m $= \frac{Ec}{Em}$

$= 200/33$

$= 6.06$

Dc=depth to neutral axis then equating strain for cracked section.

Ξs $= \frac{\xi c(d - dc)}{dc}$

Dc=

$$\frac{\left[-As * Es + \left\{(As * Es)^2 + 2b * As * Es * Ec,eff * d\right\}^2\right]}{b * Ec,eff}$$

	= 439.37 mm Cracked second moment area $= As(d-dc)^2 + \frac{Ec,eff*b*dc^3}{3Ec}$ $= 28.47*10^9$ mm^4 Approximate concrete stress, Σc $= \frac{M}{Zc} + \frac{N}{Ac}$ M = 6171.47 KNm N = 87.61 KN Ac = dc*b $= 439.37*10^3$ mm^2 Σc $= \frac{6171.47*10^6*439.37}{28.47*10^9*6.06} + \frac{87.61}{439.37}$ = 15.91 N/mm^2 Limiting concrete stress = K1*fck = 0.6*30 = 18 N/mm^2 > σc ~ OK. **2.After all creep has taken place,** The cracked section properties will be based on the long term and short term modulus for various action. Short term modulus = Ecm Long term modulus $= \frac{Ecm}{1+\vartheta}$ Effective modulus $= \frac{(Mqp+Mst)Ecm}{Mst+(1+\vartheta)Mqp}$	σc= 15.91 N/mm^2
Table 3.1 EN 1992-1-1	fcm = 38 N/mm^2 relative humidity of ambient environment at kandy area = 80% age of concrete at initial loading = 7 days h。 $= \frac{2Ac}{U}$	

	U = perimeter of the number in contact with the atmosphere. $h_\circ$ $= \frac{2*10500*1000}{2*(10500+1000)}$ = 913.043 mm α1 $=\left(\frac{35}{fcm}\right)^{0.7}$ $= \left(\frac{35}{38}\right)^{0.7}$ = 0.944 α2 $= \left(\frac{35}{fcm}\right)^{0.2}$ $= \left(\frac{35}{38}\right)^{0.2}$ = 0.983 α3 $= \left(\frac{35}{fcm}\right)^{0.5}$ $= \left(\frac{35}{38}\right)^{0.5}$ = 0.959 ~fcm > 35 Mpa ϑrh $= \left[1 + \frac{\alpha 1\left((1-\frac{RH}{100}\right)}{0.1*h_\circ^{\frac{1}{3}}}\right] * \alpha 2$ $= \left[1 + \frac{0.944\left((1-\frac{80}{100}\right)}{0.1*913.043^{\frac{1}{3}}}\right] * 0.983$ = 1.174 β(fm) $= \frac{16.8}{\sqrt{fcm}}$ $= \frac{16.8}{\sqrt{38}}$ = 2.725 $\beta(t_\circ)$ $= \frac{1}{0.1+t_\circ{}^{0.2}}$ $= \frac{1}{0.1+7^{0.2}}$=0.634 $\vartheta_\circ$ $= \vartheta rh * \beta(fcm) * \beta(t_\circ)$ $=1.174 * 2.7250.6346$ = 2.03 Moment due to long term action	

	Mqp = 5184 KNm (from dead load) Moment due to short term action, Mst = 23.4 KNm (from live load) Effective modulus,Ec,eff $= \frac{(Mqp+Mst)*Ecm}{Mst+(1+\vartheta_{o})*Mqp}$ $= \frac{(5184+23.4)*33}{23.4+(1+2.03)*5184}$ = 10.92 KN/mm² Modular ratio $= \frac{Es}{Ec,eff}$ = 200/10.92 = 18.33 dc=depth to neutral axis $\vartheta = \frac{\varepsilon c(d-dc)}{dc}$ Dc = $\frac{\left(-AsEs+\{(AsEs)^2+2b*As*Es*Ec,eff*d\}^{0.5}\right)}{b*Ec,eff}$ = 362.016 mm Cracked second moment of area, $= As(d-dc)^2 + \frac{Ec,eff*b*dc^3}{3*Es}$ $= 10655.76(1934 - 362.016)^2 + \frac{10.914*10^3*362.016^3}{3*200}$ = 27.19*10⁹ mm⁴ Concrete stress, Σc $= \frac{M}{Zc} + \frac{N}{Ac}$ M = 6171.47 KNm N = 87.6075 KN Ac = dc*b = 362.016*10³ mm² Σc = $\frac{6171.47*10^6*705.763}{27.19*10^9*18.33}\frac{87.601}{362.016}$ = 4.7 N/mm²	σc=4.7 N/mm²

Limiting steel stress $= k3 * fyk$

$= 0.8*500$

$= 400$ N/mm²

Available steel stress $= \frac{M}{zs}$

$= \frac{6747.47*10^6*(1680-705.763)}{19.63*10^9}$

$= 334.87$ N/mm² < 400

~ OK

Limiting concrete stress = K1*fck= 0.6*30

= 18 N/mm² > σc ~ OK.

Crack control

Consider worst condition before creep has occurred.

Crack width,Wk= $Sr, \max(\varepsilon sm - \varepsilon cm)$

Spacing limit= $5(c + \frac{\emptyset}{2})$ = $5(50 + \frac{32}{2})$ = 330 mm > 75 mm ~ OK

Sr,max= $K3 * C + \frac{K1*K2*K4*\emptyset}{\rho p,eff}$

K1= 0.8 (high bond bars)

K2= 0.5 (for bending)

K4= 0.425 (recommended value)

Heff= $min\left\{2.5(h-d); \frac{h-x}{3}; \frac{h}{2}\right\}$= $min\left\{2.5(2000-1934); \frac{2000-439.37}{3}; \frac{2000}{2}\right\}$

Heff= $min\{165; 520.21; 1000\}$=165mm

∴Ac,eff== heff*b= 165*10³ mm²

$\rho p, eff$= $\frac{As}{Ac,eff}$= $\frac{10723.3}{165*10^3}$=0.065

Sr,max= $K3 * C + \frac{1*K2*K4*\emptyset}{\rho p,eff}$ =108.69 mm

ξsm- ξcm=$\frac{\left(\sigma s-\left\{Kt*fct,\frac{eff(1+\alpha e*\rho p,eff)}{\rho p,eff}\right\}\right)}{Es} \geq 0.6 * \frac{\sigma s}{Es}$

Kt= 0.4; for permanent load

Ae= $\frac{Es}{Ecm}$=$\frac{200}{33}$= 6.06

Σs= $\frac{M(d-dc)}{I}$=$\frac{6171.47(1934-439.37)}{28.47*10^9}$= 323.992 N/mm²

fct,eff (=fctm) = 2.9 N/mm²

ξsm- ξcm=$\frac{\left[323.992-\left\{0.4*\frac{2.9(1+6.06*0.0796)}{0.0796}\right\}\right]}{2*10^5}$= 1.495*10⁻³

$0.6 * \frac{\sigma s}{Es}$=$0.6 * \frac{323.992}{2*10^5}$=0.971*10⁻³ < 1.495*10⁻³ ~ OK

Crack width, Wk= $Sr, \max(\varepsilon sm - \varepsilon cm)$ = 108.69*1.495*10⁻³

Crack width, Wk= 0.162 mm

Recommended value of Wmax=0.3 mm ~ OK.

Hence 32mm bars @75mm are ok for the rear face of the wall.

∴As,prov	= 10723.3 mm²
As,min	= 0.0013*Ac
	= 0.0013*2000*10³
	= 2600 mm² < As,prov

Design of vertical bar for front face of the wall,

∴As	= As,min

Provide the 25mm bar

No of bars	=$\frac{As,\min}{\frac{\pi}{4}*25^2}$
	= $\frac{2600}{\frac{\pi}{4}*25^2}$
	= 5.296
Spacing	= 1000/5.296
	= 188.821 mm
	≈ 175 mm

Hence use the 25mm bars @175mm for the front face of the wall

Design of horizontal bar for both faces,

$\therefore As = max\{0.25 * As, min; 0.001Ac\}$

$= max\{0.25 * 2600; 0.001 * 2000 * 10^3\}$

$= max\{650; 2000\}$

= 2000 mm²

Use 25mm bars,

No of bars $= \frac{As}{\frac{\pi}{4} * \emptyset^2}$

$= \frac{2000}{\frac{\pi}{4} * 25^2}$

= 4.07

Spacing $= \frac{1000}{4.07}$

= 245.7 mm

≈ 225 mm

Use the 25mm bars @225mm as the horizontal reinforcement bars for both faces.

From above geological design

Reinforce design for footing,

Consider the footing heel and toe separately

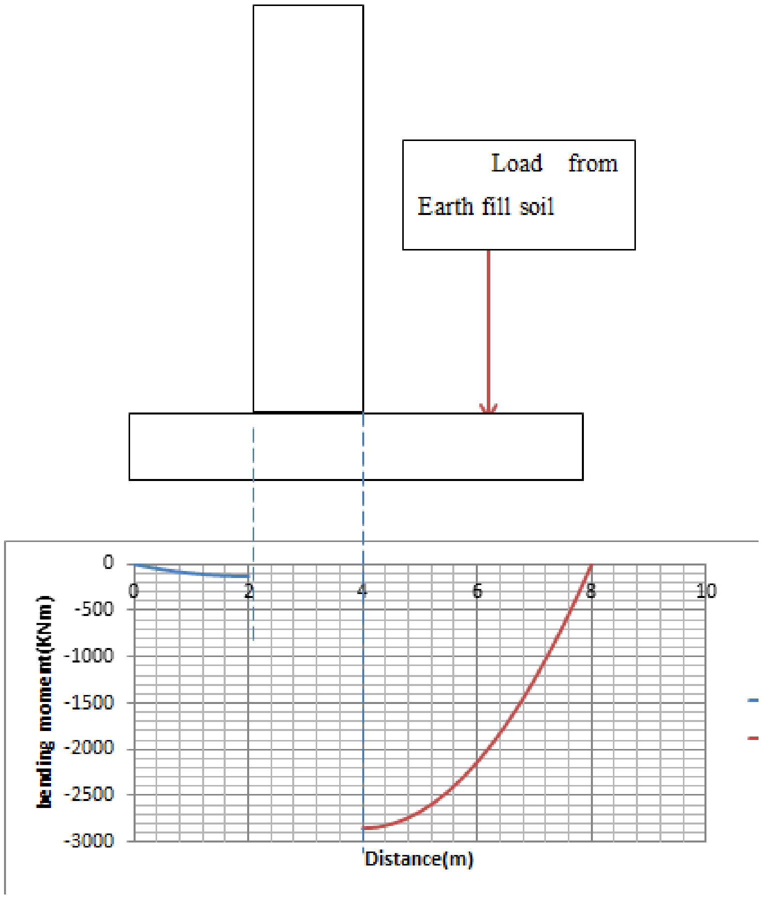


Figure E.8 Bending moment diagram for toe and heel under combination 01

Load cases,(considering 1m length)

combination 01

Backfill weight= (4*12*1)*18*1.35= 1166.4 KN/m

ULS shear at A-A= 2*2*24*1.35= 129.6KN/m

ULS moment at A-A= 2*2*24*1.35*2/2= 129.6 KNm

~ tension at the bottom of the footing

ULS shear at B-B= 4*2*24*1.35+1166.4= 1425.6 KN

ULS moment at B-B= 2*4*24*1.35*4/2 + 1166.4*4/2= 2851.2 KNm

~ tension at the bottom of the footing

Combination 02

Backfill weight= (4*12*1)*18*1= 864 KN/m

ULS shear at A-A= 2*2*24*1=96 KN/m

ULS moment at A-A= 2*2*24*2/2= 96 KNm

~ tension at the bottom of the footing.

ULS shear at B-B= 2*4*24*1*4/2 + 864*4/2=1152KN

ULS moment at B-B= 288*4*4/2= 2112 KNm

~ tension at the bottom of the footing.

Reinforcement design of pile cap,

bottom reinforcement design(tension has occurred on bottom of the footing)

Med	= 2851.2KNm
B	= 1000 mm
Cover	= 50 mm
Bar size	= 32 mm
Effective depth,d	= 2000-50-32/2
	= 1934 mm
K	$= \frac{Med}{fck*b*d^2}$
	$= \frac{2851.2*10^3}{30*1*1934^2}$
	= 0.0254 < 0.167

∴.no need compression reinforcement.

Z	$= d\left[0.5 + \sqrt{0.25 - 0.882 * K}\right]$
	$= d\left[0.5 + \sqrt{0.25 - 0.882 * 0.0254}\right]$
	= 1889.656 mm
As1	$= \frac{Med}{0.87*fyk*Z}$
	$= \frac{2851.2*10^6}{0.87*500*1889.656}$
	= 3468.61mm² > As,min = 2514.2 mm²
No or r/f bars	$= \frac{As}{\frac{\pi}{4}*\emptyset^2}$
	$= \frac{2514.21}{\frac{\pi}{4}*32^2}$

$= 3.12$

Spacing $= 1000/3.12$

$= 320.51mm$

≈ 300 mm

As,prov $= 2680.825\ mm^2$

As,min $= 0.0013*b*d$

$= 0.0013*1000*1934$

$= 2514.2\ mm^2 <$ As,prov

∴provide 32mm bars @300mm for bottom of the pile cap

Distribution reinforcement design at heel,

As = As,min

$= 2518.75\ mm^2$

Provide 25mm bar,

No of bars $= \frac{2518.75}{\frac{\pi}{4}*32^2}$

$= 3.13$

Spacing $= 1000/3.13$

$= 194.93$ mm

≈ 319.48 mm

Provide 32mm bar @300mm for distribution bars.

Check for the punching shear,

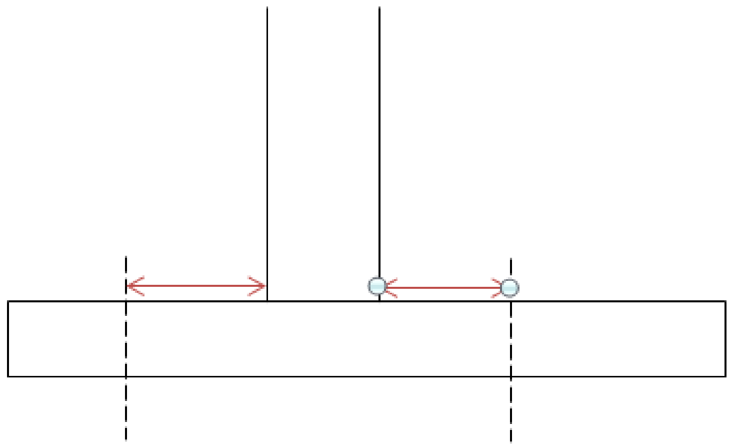


Figure E.9 Control area of the footing of the right abutment

Control perimeter, u = $(2d + 2000 + 2d + 10000)2$

= 2(4*1934+2000+10000) mm

= 39472 mm

Control area = $(2d + 2000 + 2d) * 10000$

= (4*1934+1750)*10000 mm²

= 94.86*10⁶ mm²

Ved,red = Ved - ΔVed

Ved = 20834.2-2*8*10*24*1.35

= 15650.2 KN

Ved,red = 15650.2 – 0

= 15650.2 KN

ved,red = Ved,red/u*d

= $\frac{15650.2}{39472*1934}$

= 0.205 N/mm²

Shear resistance section,

VRd,c = $\frac{\left[CRd,c*K*(100\rho 1*fck)^{\frac{1}{3}}\right]2a}{d} \geq \frac{Vmin*2d}{a}$

K $= 1 + \sqrt{\frac{200}{d}}$

$= 1 + \sqrt{\frac{200}{1934}}$

= 1.321 < 2

Vmin $= 0.035 * K^{\frac{3}{2}} * \sqrt{fck}$

$= 0.035 * 1.321^{\frac{3}{2}} * \sqrt{30}$

= 0.291

Vmin*2d/a $= 0.291 * \frac{2d}{2d}$

= 0.291

ρx(in transverse direction) $= \frac{As}{bd}$

$= \frac{\frac{\pi}{4}*32*\frac{7750}{300}}{7750*1934}$

= 1.386*10^{-3}

ρy(in longitudinal direction) $= \frac{As}{bd}$

$= \frac{\frac{\pi}{4}*32^2*\frac{10000}{300}}{10000*1934}$

= 1.386*10^{-3}

$\rho 1$ $= \sqrt{\rho x * \rho y}$

$= \sqrt{1.386 * 10^{-3} * 1.386 * 10^{-3}}$

= 1.386*10^{-3} < 0.02

VRd,c $= 0.12 * 1.321(100 * 1.386 * 10^{-3} * 30)^{\frac{1}{3}}) * 2 * 2d/2d$

= 0.509 KN/mm²

vEd,red < VRd,c ~ this section is ok for punching shear. So, no need to provide punching shear reinforcement.

wing wall design for the right abutment,

Surcharge, S = 1Ft.

End height, h = 0.6m(2Ft.)

Section height, H = 12m (40Ft.)

Length, L = 8m (26Ft.)

Reinforcement yield strength, F_y = 60000psi

oncrete strength, $F_c^{/}$ = 4000psi

β_1 = 0.85

Ø = 0.9

$$M_u = \left\{\frac{0.124 \times 26^2}{24}[3 \times 2^2 + (40+4)(40+2\times 2)]\right\} \times 1.35$$

= 9185.01 kips.ft

= 12,454.87 KN.m

$$V_u = \left\{\frac{0.124 \times 26}{6}[40^2 + (2+40)(40+2\times 2)]\right\} \times 1.35$$

= 177.25kips

= 788.45KN

Assuming 25mm bars @ 300mm spacing,

$$C = \frac{A_s F_y}{\beta_1 \O F_c^{/} b_w}$$

$$= \frac{0.76\times40\times60}{0.9\times0.85\times4\times12\times40}$$

= 1.24

a = βC

= 0.85×1.24

= 1.06

d = 12-2.5-0.5

= 9.5in

M_r = $ØM_n$

M_n = $A_s F_y \left(d - \frac{a}{2}\right)$

M_r = $0.9 \times 0.76 \times 40 \times 60 \left(9.5 - \frac{1.06}{2}\right)$

= 14725.15 kips.ft

$1.33M_u$ = 12,216.06 < M_r

Therefore, No need check M_{cr}

<u>Check for shear</u>

V_c = $2\sqrt{F_c^{/}}b_w d$

= $2\times\sqrt{4000} \times 40 \times 12 \times 9.5$

= 576.799 kips

= 2565 KN

$ØV_c$ = 0.9×576.799

= 519.12 kips

= 2309.16 KN > V_u

Therefore, no need reinforcement.

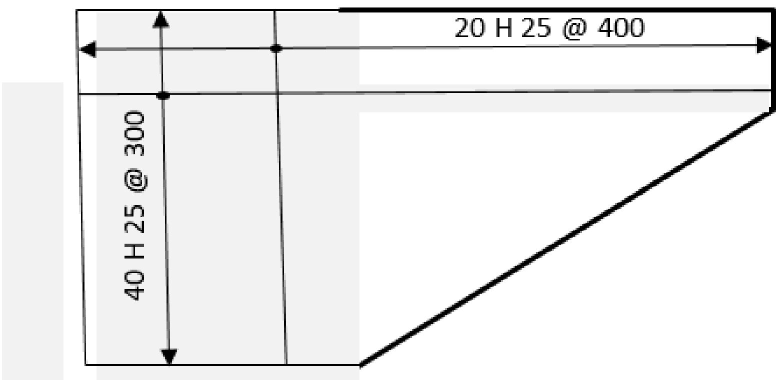


Figure E.10 Wing wall reinforcing for the right abutment

1.3.2 For the left abutment,

Surcharge, S = 1Ft.

End height, h = 0.6m(2Ft.)

Section height, H = 9.5m (31Ft.)

Length, L = 8m (26Ft.)

$$\mathrm{M_u} = \left\{\frac{0.124 \times 26^2}{24}[3 \times 2^2 + (31+4)(31 + 2 \times 2)]\right\} \times 1.35$$

= 5832.58 kips.ft

= 7908.98 kN.m

$$V_u = \left\{\frac{0.124 \times 26}{6}[31^2 + (2+31)(31 + 2 \times 2)]\right\} \times 1.35$$

= 153.50 kips

= 682.80 Kn

Assuming 25mm bars @ 300mm spacing,

$$C = \frac{A_s F_y}{\beta_1 \O F_c^{/} b_w}$$

$$= \frac{0.76 \times 40 \times 60}{0.9 \times 0.85 \times 4 \times 12 \times 31}$$

= 1.60

a = βC

= 0.85×1.60

= 1.36

d = 12-2.5-0.5

= 9.5in

$$M_r = \O M_n$$

$$M_n = A_s F_y \left(d - \frac{a}{2}\right)$$

$$M_r = 0.9 \times 0.76 \times 31 \times 60 \left(9.5 - \frac{1.36}{2}\right)$$

= 11221.16 kips.ft

$1.33\mathrm{M_u}$ = 10518.94 < M_r

Therefore, No need check M_{cr}

<u>Check for shear</u>

V_c = $2\sqrt{F_c^{/}} b_w d$

= $2 \times \sqrt{4000} \times 31 \times 12 \times 9.5$

= 447.02 kips

= 1988.42Kn

$\emptyset V_c$ = 0.9×447.02

= 402.318 kips

= 1789.59 KN > V_u

Therefore, no need reinforcemnet.

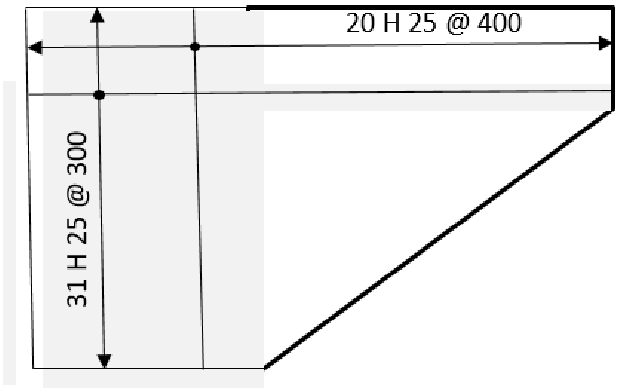


Figure E.11 Wing wall reinforcement for the left abutment

1.4 Pile design

Considering worst case of the rock, rock type is selected as weak jointed cemented mudstone.

For that rock type,

Average unconfined compression strength, (q_{uc}) = 4.5 MPa

Modulus ratio = 150 MPa

1.4.1 For the right abutment

Design parameters

- No of piles - 9
- Design pile length - 9 m
- Max. factored reaction on the pile - 2314.91 k N
- Required pile reaction - 2500 k N
- Pile type - Bored and cast in-situ pile
- Concrete grade - C32/40

Allowable working stress of the concrete = 25% of the concrete strength

= 40 × 25%

= 10 Mpa

Required pile diameter = $(\frac{2500*1000*4}{\pi*9})^{0.5}$

= 594.71 mm

≈ 0.6m

Stress on the shaft = $2.5/(\pi*0.6^2/4)$

= 8.84 Mpa < 10 Mpa

From figure E.12 (Using Rosenberg & Journeaux graph),

rock socket reduction (α) factor = 0.2

Rock Quality Designation (RQD) values increased from an average of 15% to 35%.

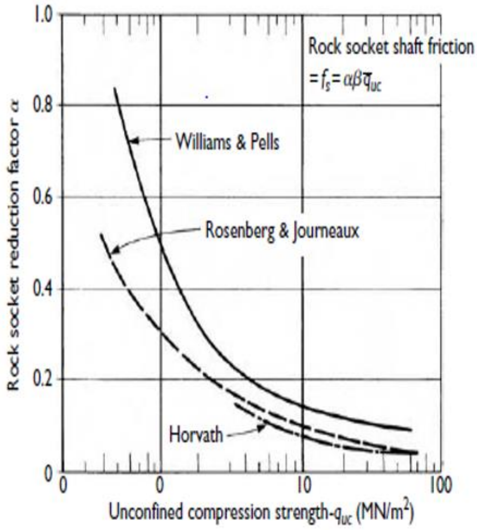


Figure E.12 Friction reduction factors for rock socket shaft friction(α)

Table E.7: Mass factors

RQD (%)	*Fracture frequency per metre*	*Mass factor j*
0–25	15	0.2
25–50	15–8	0.2
50–75	8–5	0.2–0.5
75–90	5–1	0.5–0.8
90–100	1	0.8–1

Therefore, mass factor (j) = 0.2

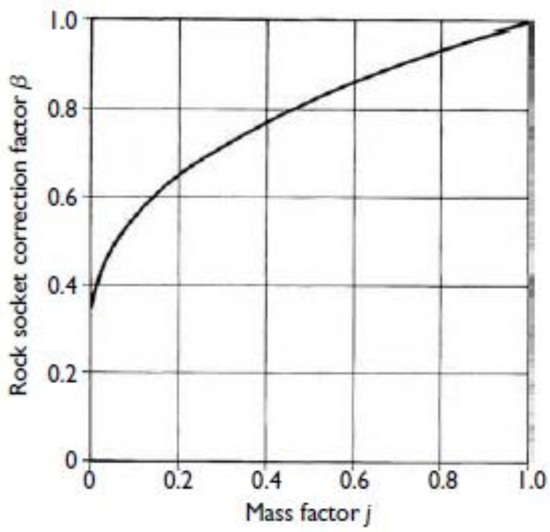


Figure E.13 Reduction factors for discontinuous in rock mass

From figure E.13,

Rock socket correction factor, β = 0.65

Rock shaft friction, f_s = $\alpha\beta q_{uc}$

= 0.2*0.65*4.5

= 0.585 MPa

Socket length of pile = 6m

Ultimate shaft friction of the pile, = 585*π*0.6*6

= 6616.19 kN

Therefore, factor of safety on shaft friction = 6616.19 /2500

= 2.64 >1 - OK

Total base resistance = $\pi*0.6^2*4500/4$

= 1272.35kN

So, total pile resistance = Shaft friction + Base resistant

= 6616.19 kN +1272.35 kN

= 7888.54kN

Total factor of safety of the pile = 7888.54 /2500

= 3.16 > 1 - OK

Deformation modulus of the rock mass = 0.2*150*4.5

= 135 MPa

R = E_C / E_D

= 33314 / 135

= 246.77

L / B = 6 m / 0.7 m

= 8.57

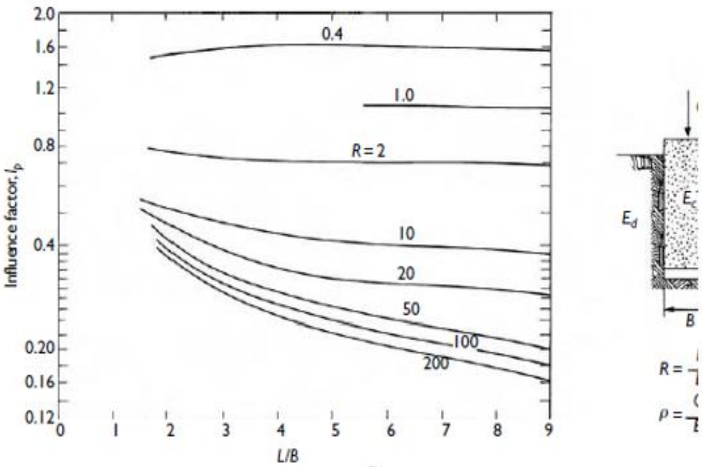


Figure E.14 Elastic settlement influence factors for rock-socket friction on piles

From figure E.14 ,

Influence factor = 0.18

According to the Eurocode, up to the base diameter 600 mm settlement should not exceed 10 mm.

Settlement of the pile ρ = $Q I_P / B E_d$

= 2500*0.18/ 0.6*135

= 5.56 mm < 10 mm

Nine bored piles which are having 0.6m diameter and 9m length , ok for the right abutment

1.4.2 For the left abutment

Design parameters

- No of piles - 9
- Design pile length - 15 m
- Reaction on the pile – 1736.48 k N
- So, required pile reaction - 2000 k N
- Pile type - Bored and cast in-situ pile
- Concrete grade - C32/40

Required pile diameter = $(\frac{2000*1000*4}{\pi*15})^{0.5}$

= 412.03mm

≈ 0.6 m

Stress on the shaft = $2.0/(\pi*0.6^2/4)$

= 7.07 Mpa < 10 Mpa

From figure 1 (Using Rosenberg & Journeaux graph),

rock socket reduction (α) factor = 0.2

From figure 2,

Rock socket correction factor, β = 0.65

Rock shaft friction, f_s = $\alpha\beta q_{uc}$

= 0.2*0.65*4.5

= 0.585 MPa

Socket length of pile = 5m

Ultimate shaft friction of the pile, = 585*π*0.6*5

= 5513.50 kN

Therefore, factor of safety on shaft friction = 5513.50/2000

= 2.76 >1 - OK

Total base resistance = $\pi*0.6^2*4500/4$ =1272.35 kN

So, total pile resistance = 5513.50 kN +1272.35 kN

= 6785.85 k N

Total factor of safety of the pile = 6785.85/2000

= 3.39 > 1 - OK

Deformation modulus of the rock mass = 0.2*150*4.5

= 135 MPa

R = E_C / E_D

= 33314 / 135

= 246.77

L / B = 5 m / 0.6 m

= 8.33

From figure 3,

Influence factor = 0.18

Settlement of the pile, ρ = $Q I_P / B E_d$

= 2000*0.18/ 0.6*135

= 4.44 mm < 10 mm

Nine bored piles which are having 0.6m diameter and 14m length , ok for the left abutment

1.4.3 Reinforced design for the pile (for both left and right abutments)

Referring concise Eurocode 2,

Minimum longitudinal bars - 6

Minimum diameter - 16mm

Maximum spacing around the periphery of the pile - 200mm

Minimum spacing around the periphery of the pile - 100mm

For the designing,

Longitudinal bar size = 25 mm

Outer rings = 10 mm

Cover = 75 mm

Cross section area of the pile, Ac = $\pi*300^2$

= 0.283 m^2 < 0.5 m^2

required minimum longitudinal reinforcement area = 0.005*Ac

= 1413.72 mm^2

Using 8 H25 bars,

As provided = 3926.99 mm^2> As,min

Spacing of the longitudinal bars =2*π*(300-75-25-10)/8

=149.23

= 150 mm < 200 mm~OK

Therefore, use 8 H25 @ 150

1.5 Abutment scour

From Mahaweli river flow data,

Q = 820m^3/s

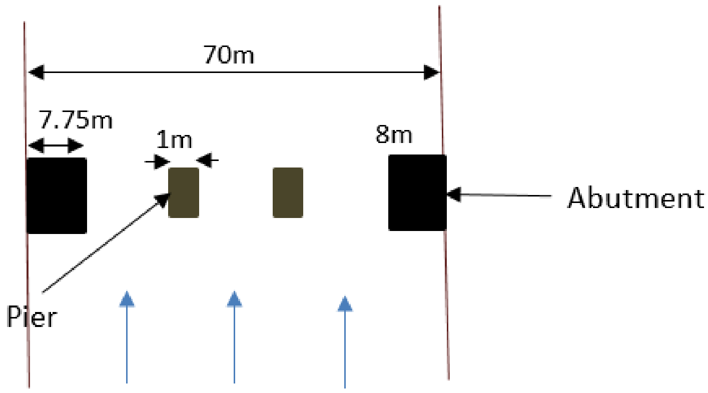


Figure E.15 Plan view of Mahaweli river

$$q_{2f} = \frac{820}{(70-(7.75+8)-1\times 2)}$$

$$= 15.62 m^2/s$$

Assuming, D_{50} = 20mm

$$Y_C = \left(\frac{15.62}{6.19\times 0.02^{1/3}}\right)^{6/7}$$

$$= 6.75 \text{ m}$$

$$q_f = \frac{820}{70}$$

$$= 11.71 m^2/s$$

$$q_{2f}/q_f = 1.3$$

Using Hydraulic engineering circular No. 18,

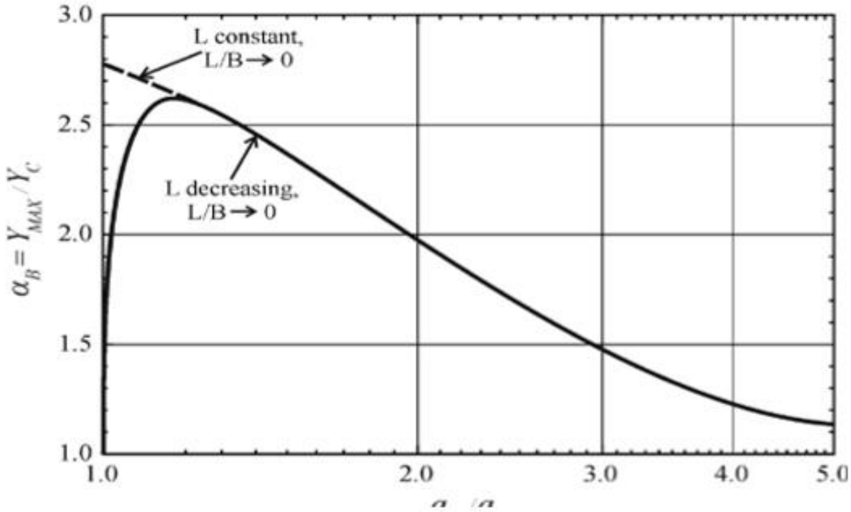


Figure E.16 Scour amplification factor for clear-water condition

Scour amplification factor(α_B) = 2.4

Using eq (b),

Y_{max} = 2.4 × 6.75

= 16.21m

Y_S = 16.21m – 15m

Abutment scour depth, Y_S = 1.21m

Sizing rock rip rap for abutment protection

Using Bridge Scour Manual, Transport and Main Roads, 2019.

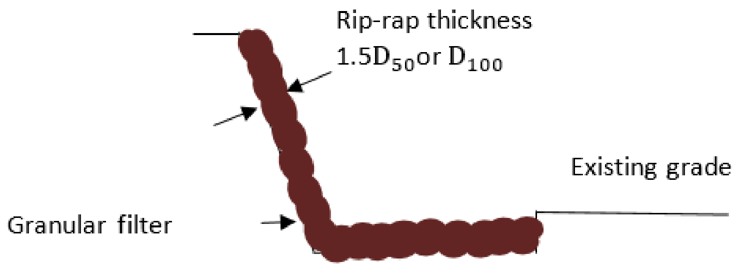


Figure E.17 Rock rip rap

$$\frac{d_{50}}{y} = \frac{1.026}{(S_s - 1)} Fr^2$$

	y = Water depth at abutment Ss = Specific gravity of rock Y = 15m , Ss=2.65 From Mahaweli ganga flow data, Maximum discharge -1660m^3/s $V_{avg} = \frac{Q}{A}$ $= \frac{1660}{15\times70}$ = 1.6m/s Bridge scour manual,2019, v = 1.33 V_{avg} = 1.33 × 1.6 = 2.1m/s $\mathrm{Fr}^2 = \frac{V^2}{gy}$ $= \frac{2.1^2}{9.81 \times 15}$ = 0.03 $d_{50} = \frac{1.026}{(S_s-1)} Fr^2 \times y$ = 0.28m Thickness of rip rap = 1.5 d_{50} = 0.42m Considering the flood level, Height of the rip rap = 15m	

APPENDIX F BEARING DESIGN

<table>
<tr><th>REFERENCE</th><th>CALCULATIONS</th><th>RESULTS</th></tr>
<tr><td>AASHTO – LRFD specification</td><td>

Load effects on each bearing

$P_{DL\ girder} + P_{DL\ slab} = 302.5\ kN = 68.0047\ kip$

$P_{LL\ lane} = 275\ kN = 61.8225\ kip$

$P_{LL\ truck} = 439.2\ kN = 98.7361\ kip$

Where,

$P_{DL\ girder}$ - Dead load of the girder.

$P_{DL\ slab}$ - Dead load of the slab.

$P_{LL\ lane}$ - Live load of the lane.

$P_{LL\ truck}$ -Live load of the truck.

Commonly used elastomers have a shear modulus between 0.080 and 0.175 ksi and a nominal hardness between 50 and 60 on the Shore A scale. A typical elastomer with hardness 60 Shore A Durometer and a shear modulus of 0.150 ksi is assumed. Shear modulus of the elastomer at 73˚F is used as the basis for design.

Design steps

Minimum bearing area was determined.

The maximum compressive stress limit under service limit state for bearings fixed against shear deformations.

$$\sigma_S \leq 2.00\ GS \leq 1.75\ ksi$$

$$\sigma_L \leq 1.00\ GS$$

</td><td></td></tr>
</table>

Where,

σ_S – Service average compressive stress due to the total load. (ksi)

σ_L – Service average compressive stress due to the live load. (ksi)

G – Shear modulus of elastomer. (ksi)

S – Shape factor of the thickest layer of the bearing.

To satisfy the 1.75 ksi limit, the determine the minimum bearing area, A_{req}

$$A_{req} = \frac{P_{TL}}{1.75}$$

Where,

A_{req} – Requirement area of bearing

P_{TL} – Total load

$A_{req} \geq \frac{228.5633}{1.75}$

$A_{req} \geq 130.61\ in^2$

$A_{req} \geq\ 0.0843\ m^2$

Choose a 558.8mm. width and 177.8mm. length,

Area = $0.5588m \times 0.1778m$

Area = 0.0994 m^2 $>$ $0.0843\ m^2$ OK

The shape factor of a layer of an elastomeric bearing (S_i) is taken as,

$$Shape\ factor\ = \frac{plan\ area\ of\ the\ layer}{area\ of\ perimeter\ free\ to\ bulge}$$

For rectangular bearings without holes, the shape factor of the layer may be taken as,

$$S_i = \frac{LW}{2\, h_{ri}\, (L + W)}$$

$$h_{ri} = \frac{LW}{2\, S_i\, (L + W)}$$

Where,

L - Length of a rectangular elastomeric bearing (parallel to the longitudinal bridge axis) (in.)

W- Width of the bearing in the transverse direction (in.)

h_{ri} - Thickness of i^{th} elastomeric layer in elastomeric bearing (in.)

Design Requirements

First, solve for the shape factor under total load (S_{TL})

$$S_{TL} \geq \frac{\sigma_S}{2.00\, G}$$

$$\sigma_S = \frac{P_{TL}}{LW}$$

$$\sigma_S = \frac{228.8633}{7 \times 22}$$

$$\sigma_S = 1.485\ ksi$$

$$S_{TL} \geq \frac{1.485}{2.00 \times 0.15}$$

$$S_{TL} \geq 4.95$$

$(S_L)_{minimum}$ = 4.95

Next, solve for the shape factor under live load (S_{LL})

$$S_{LL} \geq \frac{\sigma_L}{1.00\, G}$$

	$$\sigma_L = \frac{P_{LL}}{LW}$$ P_{LL} – Total live load $$\sigma_L = \frac{160.56}{7 \times 22}$$ $$\sigma_L = 1.043\ ksi$$ $$S_{LL} \geq \frac{1.043}{1.00 \times 0.15}$$ $$S_{LL} \geq 6.96$$ $(S_{LL})_{minimum}$ = 6.96 Using the shape factors, determine the elastomer thickness. $$h_{ri\,(TL)} < \frac{LW}{2\,S_{TL}\,(L+W)}$$ $$h_{ri\,(TL)} < \frac{7 \times 22}{2 \times 4.95 \times (7+22)}$$ $$h_{ri\,(TL)} < 0.537\ in.$$ $$h_{ri\,(LL)} < \frac{LW}{2\,S_{LL}\,(L+W)}$$ $$h_{ri\,(LL)} < \frac{7 \times 22}{2 \times 6.96 \times (7+22)}$$ $$h_{ri\,(LL)} < 0.382\ in.$$ $$h_{ri\,(LL)} < 9.7mm$$ Therefore, use an interior elastomer layer thickness is 0.35in (8.9 mm.) The shape factor is, $$S = \frac{LW}{2\,h_{ri}\,(L+W)}$$	

$$S = \frac{7 \times 22}{2 \times 0.35 \times (7 + 22)}$$

$$S = 7.59$$

Combined compression and rotation

Rectangular bearings are assumed to satisfy uplift requirements if they satisfy

$$\sigma_S > 1.0\, GS \left(\frac{\theta_S}{n}\right) \left(\frac{B}{h_{ri}}\right)^2$$

Where,

n - Number of interior layers of elastomer, where interior layers are defined as those layers which are bonded on each face. Exterior layers are defined as those layers which are bonded only on one face. When the thickness of the exterior layer of elastomer is more than one-half the thickness of an interior layer, the parameter, n, may be increased by one-half for each such exterior layer.

θ_S – Maximum service rotation due to the total load (rad). It is assume 0.005 rad

Determine the number of interior layers of elastomer (n_u),

$$n_u > \frac{1.0\, GS\, (\theta_S) \left(\frac{B}{h_{ri}}\right)^2}{\sigma_S}$$

$$n_u > \frac{1.0 \times 0.15 \times 7.59 \times 0.005 \times \left(\frac{7}{0.35}\right)^2}{1.485}$$

$$n_u > 1.53$$

To prevent excessive stress on the edges of the elastomer, rectangular bearings fixed against shear deformation must also satisfy,

$$\sigma_S < 2.25\,GS\,[1 - 0.167\left(\frac{\theta_S}{n}\right)\left(\frac{B}{h_{ri}}\right)^2]$$

Determine the number of interior layers of elastomer (n_u), required to limit compression along the edges,

$$n_c > \frac{-0.167\,(\theta_S)\left(\frac{B}{h_{ri}}\right)^2}{\left(\frac{\sigma_S}{2.25\,GS} - 1\right)}$$

$$n_c > \frac{-0.167\,(0.005)\left(\frac{7}{0.35}\right)^2}{\left(\frac{1.485}{2.25\,\times 0.15\,\times 7.59} - 1\right)}$$

$$n_c > 0.79$$

Use 3 interior layers of 8.9mm thickness each. Use exterior layers of 6.4mm thickness.

Constituent elastomeric layers and steel shims shall be fabricated in standard thicknesses. For overall bearing heights less than about 127mm, a minimum of 6.4mm. of horizontal cover is recommended over steel shim edges. For overall bearing heights greater than 127mm, a minimum of 12.7mm. of horizontal cover is recommended.

For bearings with more than two elastomer layers, the top and bottom cover layers should be no thicker than 70% of the internal layers.

Stability of elastomeric bearings

Bearings satisfying following inequality considered stable,

$$2A < B$$

For which,

$$A = \frac{1.92 \times \frac{h_{rt}}{L}}{\sqrt{1 + \frac{2.0\,L}{W}}}$$

$$B = \frac{2.67}{(S + 2.0)\left(1 + \frac{L}{4.0\,W}\right)}$$

Where,

h_{rt} - Total thickness of the elastomer in the bearing (in.)

$$h_{rt} = 2\,(0.25) + 3\,(0.35)$$

$$h_{rt} = 1.2\,in$$

$$h_{rt} = 30.5mm$$

$$A = \frac{1.92 \times \frac{1.55}{7}}{\sqrt{1 + \frac{2.0 \times 7}{22}}}$$

$$A = 0.332$$

$$B = \frac{2.67}{(7.59 + 2.0)\left(1 + \frac{7}{4.0 \times 22}\right)}$$

$$B = 0.258$$

Check $2A < B$

$2 \times 0.332 > 0.258$ Therefore, the bearing is not stable.

	For bridge decks fixed against translation, the following equation needs to be satisfied to ensure stability. $$\sigma_S \leq \frac{GS}{A-B}$$ $$1.485 \leq \frac{0.15 \times 7.59}{0.332 - 0.258}$$ $1.485 \leq 17.789$ Therefore the bearing is stable. Reinforcement At the service limit state, $$h_S \geq 3\, h_{max} \frac{\sigma_S}{F_y}$$ Where, h_{max} - Thickness of thickest elastomeric layer in elastomeric bearing (in.) F_y - Yield strength of steel reinforcement (ksi) [F_y =36 ksi] $$h_S \geq 3 \times 0.35 \times \frac{1.485}{36}$$ $$h_S \geq 0.043\, in$$ $$h_S \geq 1.1 mm$$ At the fatigue limit state, $$h_S \geq 2.0\, h_{max} \frac{\sigma_L}{\Delta F_{TH}}$$ Where, ΔF_{TH} - Constant amplitude fatigue threshold for Category A, [$\Delta F_{TH} = 24 ksi$]	

	$$h_{S\,(LL)} \geq 2.0 \times 0.35 \times \frac{1.043}{24}$$ $$h_{S\,(LL)} \geq 0.8mm$$ Use $h_S = 3.0mm$ thick steel reinforced plates The total height of the bearing (h_{rt}), h_{rt} = Cover layers + Elastomer layers + Shim thicknesses $$h_{rt} = 2(6.4) + 3(8.9) + 4(3.0)$$ $$h_{rt} = 51.5mm$$	

APPENDIX G APPROACH ROAD DESIGN

REFERENCE	CALCULATIONS	RESULTS

Traffic volume data

At Peradeniya Bridge we did the traffic survey for our pavement designing purpose. Table G.1 shows the traffic data of Peradeniya Bridge.

Table G.1 Traffic volume data

Axle Configuration	24 hr volume	Annual Growth Rate (%)	Axle Loads		
1.1	83	4	40	70	
1.2	102	4	50	90	
1.2	68	2	55	99	
1.2	47	2	65	115	
1.22	33	2	60	145	
1.22	12	1	70	155	
1.22	6	1	80	185	
1.2-22	3	1	80	125	190

Standard axle loads for axle group

AASHTO Design Guide, Part III, Chapter 5

According to HDMI 2.1 VM-1876-MD ASSHTO design axle group and similar loads were taken as shown in Table G.2

Table G.2 Standard axle loads for axle group

Axle Group	Load (kN)
Single axle with single tyres (SAST)	53
Single axle with dual tyres (SADT)	80
Tandem axle with single tyres (TAST)	90
Tandem axle with dual tyres (TADT)	135
Triaxle with dual tyres (TRDT)	181
Quad-axle with dual tyres (QADT)	221

Calculation

Equivalent factor (EF) $= \left(\frac{40}{53}\right)^{4.5} + \left(\frac{70}{53}\right)^{4.5}$

= 3.779

Equivalent standard axle (ESA) $_{\text{base year}}$ = 3.779×83×365

= 114484.805

Growth factor (GF) $= \frac{(1+r)^n - 1}{r}$ (where r is growth rate and n is design life)

$= \frac{(1+0.04)^{15} - 1}{0.04}$

= 23.276

Equivalent standard axle (ESA)$_{\text{cumulative}}$ = ESA $_{\text{base year}}$ × GF

= 114484.805 × 23.276

=2664748.321

According to above calculation we can get the total value of ESA cumulative values. (Table G.3).

Table G.3 ESA cumulative values

EF	ESA base year	Growth Factor	ESA cumulative
3.779	114481.476	20.024	2292329.867
2.468	91895.624	20.024	1840080.075
3.790	94074.226	17.293	1626864.812
7.625	130805.906	17.293	2262081.067
3.127	37663.194	17.293	651325.325
5.359	23472.773	16.097	377838.767
10.506	23008.113	16.097	370359.193
18.483	20238.826	16.097	325782.268
Total ESA cumulative			9.747×10^6

RDA Road design manual	**Horizontal alignment (Simple curve)** Maximum/ Full Superelevation = $\frac{V^2}{127(e+f)}$ For 60km/h design speed, f value = 0.19 Selected value of R (min of R = 150m) = 160m Selected value of n = 2.5% Therefore, e_{max} = 2.89% 1. **Relative gradient method** Super elevation development length L_e = $\frac{w(e+n)}{Gr}$ According to RDA manual G_r = 0.63 Super elevation development length = 29.94m 2. **Rate of pavement method** Super elevation development length Le = ((e+n)v)/β β value = 0.35 Super elevation development length Le = 25.66m Based on above two methods, relative gradient method give the highest value. Therefore Super elevation development length is 29.94m. Super elevation runoff length (Sr_o) = $\mathrm{Le} - \mathrm{Le}\ \frac{n}{(n+e)}$ Super elevation runoff length = 13.89m Tangent Runout (Tr_0) = $L_e - Sr_0$ Tangent Runout (Tr_0) = 16.05m The portion of runoff located within the curve = $Sr_0/3$ = 4.69m	

Sight distances

Sight distance is the length of roadway visible to a driver. The three types of sight distance common in roadway design are intersection sight distance, stopping sight distance, and passing sight distance.

Object Height

Approaching vehicle – 1.15m

Driver Height

Passenger Car - 1.05 m

Commercial vehicle -1.8 m

Driver Perception – Reaction time (t_r)

t_r - 2.5 sec

SSD = $\frac{V\mathrm{tr}}{3.6} + \frac{V^2}{254\mu}$

Where

V- Design speed (km/h)

t_r – Total reaction time (sec)

μ – Coefficient of Longitudinal Friction

SSD = $\frac{V\mathrm{tr}}{3.6} + \frac{V^2}{254\mu}$

$= \frac{60\frac{km}{h}*2.5\ s}{3.6} + \frac{60^2}{254*0.33}$

= 84.6 m

Table G.4 Sight distance details

Station	Actual sight distance	Minimum sight distance	Obstruction point
0+000.00	84.6	85	NO
0+020.00	84.6	85	NO
0+040.00	83.176	85	NO
0+060.00	63.176	85	NO
0+080.00	43.176	85	NO
0+100.00	23.176	85	NO
0+120.00	3.176	85	NO
0+123.18	0	85	NO

Drainage Design

Catchment area =0.022km^2

Determination Runoff for upstream of the bridge

The following formula known as the rational formula is used for calculation of runoff water for drainage system

$$Q=0.028CIA$$

Where

Q = maximum runoff in m^{3}per se

C = a constant depend upon nature of the surface range (0.31 – 0.93)

I = the critical intensity of storm in mm per hour occurring during the time concentration

A = the catchment area in km^2

I = 198.9 mm from rainfall data

C = 0.7

A = 0.22km^2

$$Q = 0.028CIA$$

$$= \frac{0.028\times198.9\times0.022\times10^6}{3600\times1000}$$

$$= 0.034 \text{ m}^3/\text{s}$$

Hydraulic design

Once the design runoff Q is determined, the next is the hydraulic design of drains. The side drainage is designed based on the principles of flow through open channels.

The following formula is used to design the cross section area of the drainage

$$Q = AV$$

Where

Q = maximum run off m^3/sec

A = cross section area of the channel

V= allowable velocity of the flow m/s (more than 1 m/s)

$Q = 0.034\ m^3/s$

Take $V = 1.2\ m/s$

$Q = AV$

$0.034\ m^3/s = A \times 1.2\ m/s$

$A = 0.0283\ m^2$

Consider the formula of rectangular section

Area (A) = Height (H) × width (B)

$0.0283\ m^2 = H \times B$

Assume width (B) = 0.5 m

H = 0.06 m

Free board = 150 mm

Total height = 0.21 m

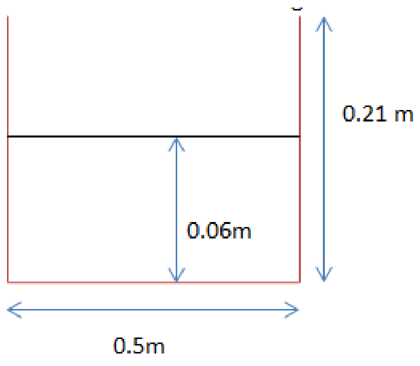


Figure G.1 Cross section of a drain

Longitudinal slope

Using the manning's formula

$$V = 1/n*R^{2/3}*S^{1/2}$$

Where

V = average flow velocity in m/s

n = manning's coefficient (0.02)

R = hydraulic radius

S = longitudinal slope

$$R = \frac{cross\ sectional\ area}{wetted\ perimeter}$$

Wetted perimeter = 0.5+ 0.06*2 m

= 0.62 m

$$R = \frac{0.0283}{0.62}$$

=0.0456

n = 0.02

V = 1.2 m/s

R = 0.0456

$$1.2 = \frac{1}{0.02} \times 0.0456^{2/3} \times S^{1/2}$$

$S = 0.035$

There for proposed slope for the side drainage is 0.035

Determination Runoff for downstream of the bridge

$Q = 0.005\ m^3/s$

Take $V = 1.2\ m/s$

$Q = AV$

$0.005\ m^3/s = A \times 1.2\ m/s$

$A = 0.0042\ m^2$

Consider the formula of rectangular section

Area (A) = Height (H) $\times$ width (B)

$0.0042\ m^2 = H \times B$

Assume width (B) = 0.4 m

H = 0.012 m

Free board = 150 mm

Total height = 0.17 m

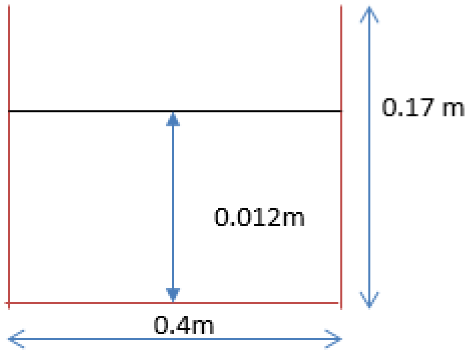


Figure G.2 Cross section of a drain

Longitudinal slope

Using the manning's formula

$$V = 1/n*R^{2/3}*S^{1/2}$$

Wetted perimeter P= 0.4+ 0.012*2 m

= 0.424 m

$R = \frac{0.0042}{0.424}$

=0.01

n = 0.012

V = 1.2 m/s

R = 0.01

$1.2 = \frac{1}{0.012} \times 0.01^{2/3} \times S^{1/2}$

S = 0.0962

There for proposed slope for the side drainage is 0.0962

Road Fill Between Approach road and Existing road

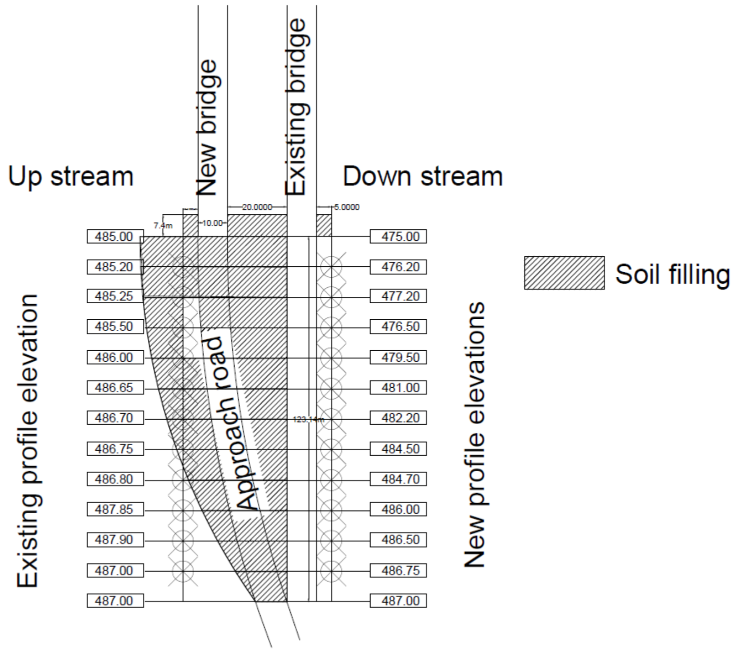


Figure G.3 Plan view of the fill

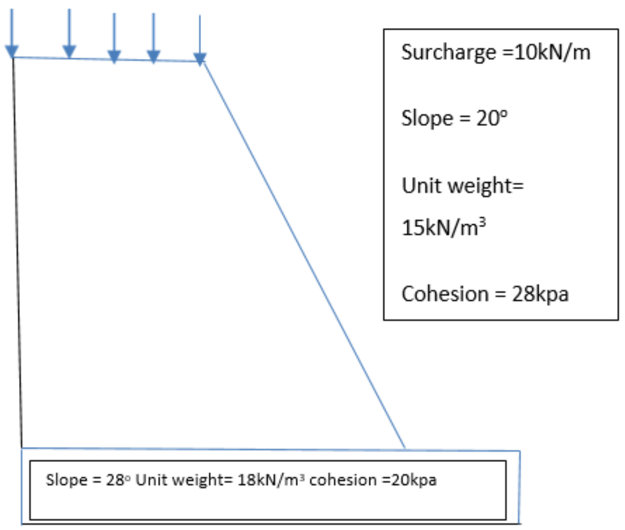


Figure G.4 Plan view of the fill

Since the filling volume is varying along the length of the road area of each 10m section was calculated using AutoCAD Civil 3D. The existing and new profile elevations are shown in figure G.3.

Specimen calculation for 0+010 to 0+020

Area of the filling area = $258.53m^2$

Average depth of the filling = 0.825m

Filling volume = $213.29m^3$

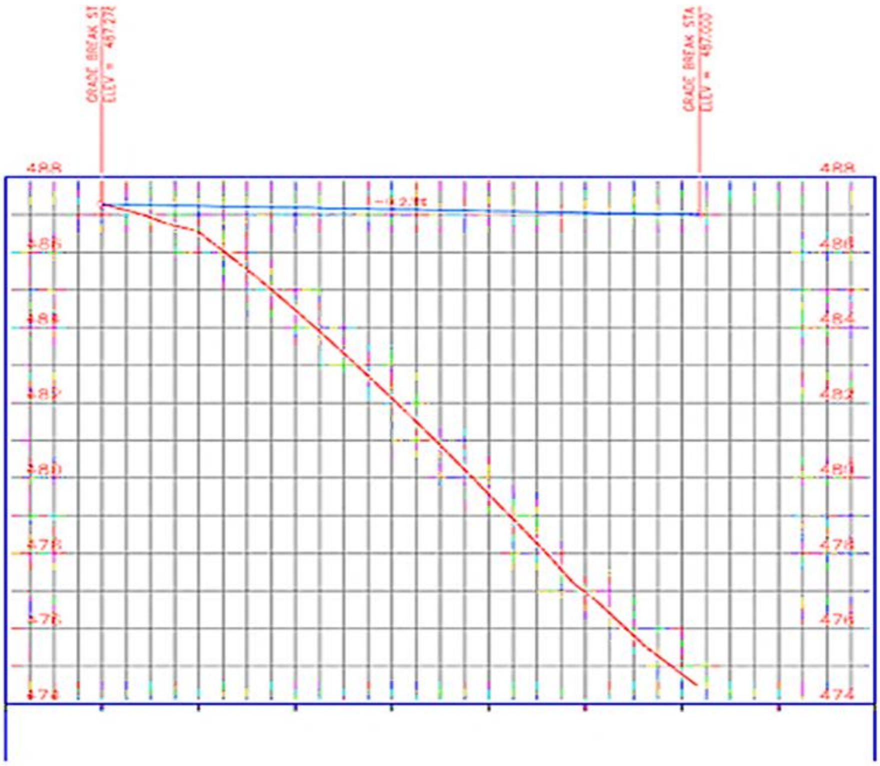


Figure G.5 New road and existing road elevation profile

Table G.5 Total volume of filling

Chainage	cross section	Average depth	Filling volume
0+010	193.06	0.125	24.13
0+020	258.53	0.865	223.62
0+030	316.14	1.625	513.72
0+040	261.27	1.975	516.00
0+050	300.00	2.175	652.50
0+060	343.77	3.375	1160.22
0+070	375.88	5.075	1907.59
0+080	402.77	6.075	2446.82
0+090	486.31	7.75	3768.90
0+100	400.13	8.525	3411.10
0+110	535.95	8.53	4571.65
0+120	543.63	9.5	5164.48
TOTAL FILLING VOLUME			24360.78

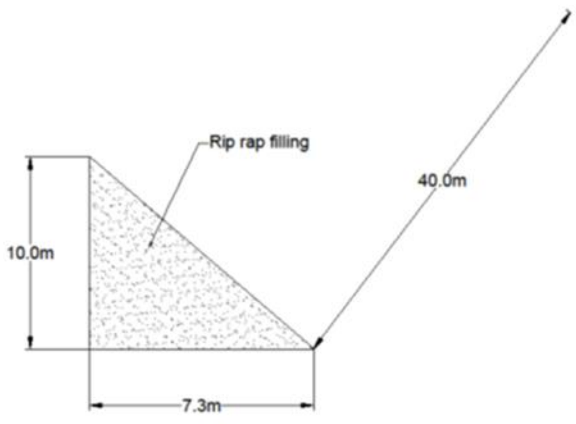


Figure G.6 Cross section of the rip rap filling

Total rip rap filling = ½*7.3*10*40

= 1460m3

Total soil filling = 24361+ 1460
= 25821m3

Filling design

New fill material must be fully keyed by means of benching. Each step should be compacted and filled with a suitable soil as shown in figure G.5.

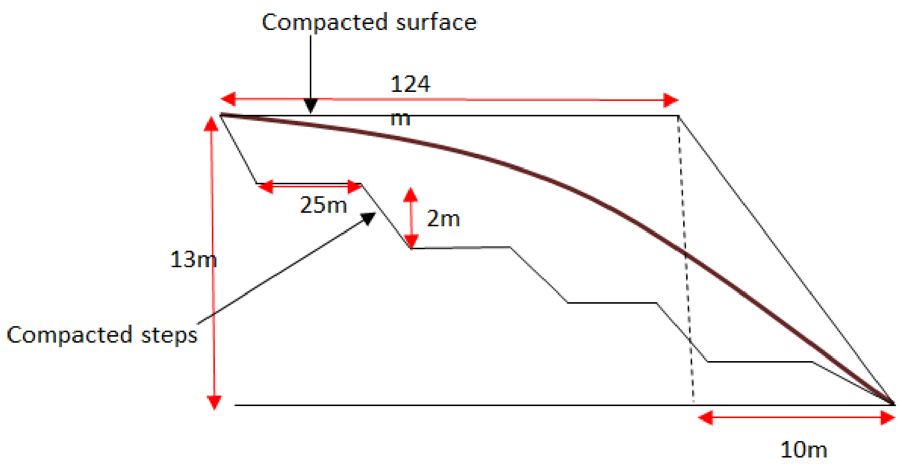


Figure G.7 End view of steps

<u>Scouring effect on facing</u> (Riprap design and construction guide, March 2000)

There are no published guidelines existing for bedding.

Rip rap sizing -

$$D_{50} = \frac{V^2}{2 \times g \times c^2(s-1)}$$

D_{50}-Median diameter of rock

C- Isbash constant (1.2)

S- Specific gravity of rock

V= 5m/s S= 2.65

$$D_{50} = \frac{5^2}{2 \times 9.81 \times 1.2^2 (2.65-1)}$$

= 0.53m

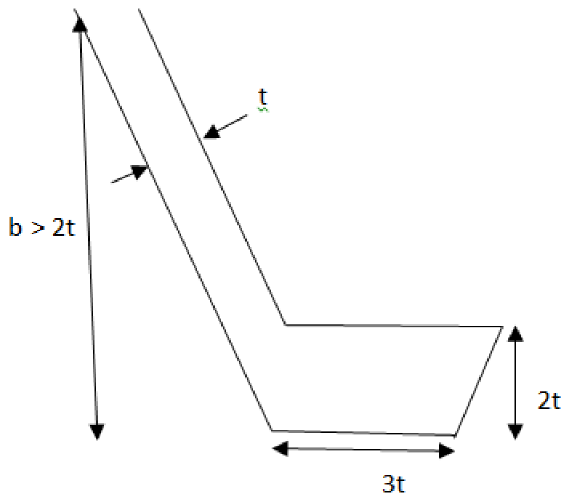


Figure G.8 Dimension of rip rap for soil

t = $1.5D_{50}$
= 0.8m
2t = 1.6m 3t = 2.4m

- There no maximum limit for height of the rip rap. Therefore, b=13m

<u>Culvert design</u> (Riprap design and construction guide, March 2000)

Scour protection for pipe inlets

Scour can be controlled by a reinforced concrete, grouted or loose riprap apron at the inlet. Grouted riprap further than one pipe diameter from the inlet edge of the pipe may have a mean stone diameter of not less than 0.4 times D50.

$$D_{50} = \frac{0.05Q}{d_0^{3/2}} - 0.075d_0$$

D_{50}- Mean diameter of rock, ft
$\mathrm{d}_0 -$ Diameter of pipe, ft

- Length of the culvert = 125m
- Minimum slope required = 2%

- Assume culvert diameter,

$$d_0 = 600\text{mm}$$

Discharge through the culvert is 2.5m3/s (88.288 cfs)

$$D_{50} = \frac{0.05 \times 88.288}{1.969_0^{\frac{3}{2}}} - 0.075 \times 1.969$$

= 1.45 ft

= 0.442m

$0.4D_{50}$ = 0.4× 1.45

= 0.58 ft < d_0= 1.969 ft -

OK

Scour protection for pipe outlet

In these cases, it is important to be certain that the outlet protection is adequate the outlet as shown in figure G.7.

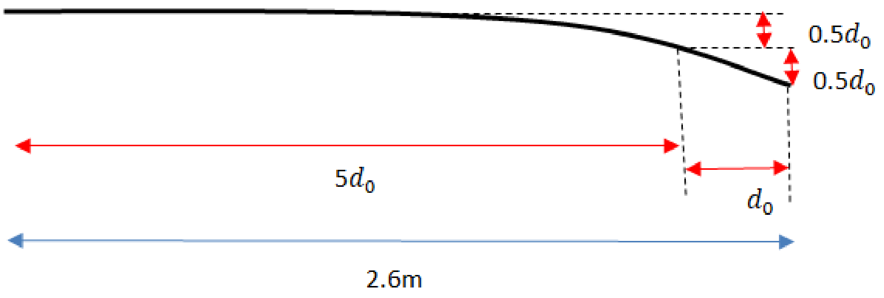


Figure G.9 Culvert outlet erosion protection

d_0- Diameter of the pipe is 600mm
Therefore, Total protection length is 2.6m

With including the all above details, side elevation of filling area can be as in figure G.8

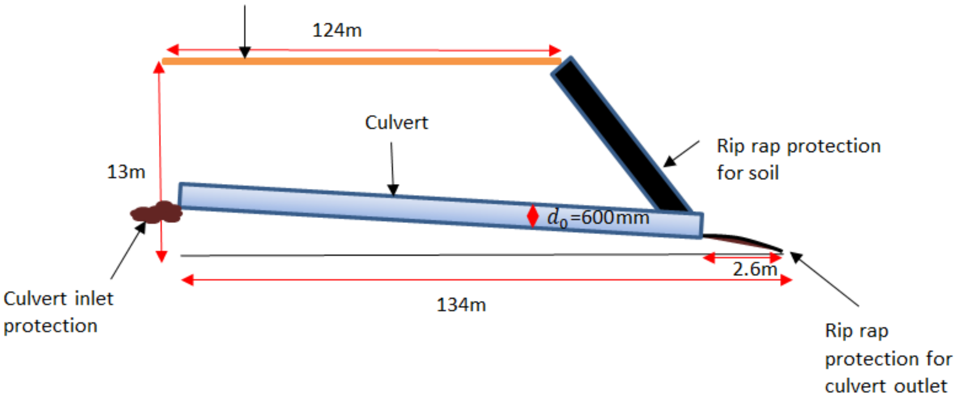


Figure G.10 Side elevation of filling area

- Therefore, 5m long protection will be implemented along the upstream and downstream from the New Bridge.

Slope stability check of upstream side filling

The slope stability check was done using GeoStudio software. The factor of safety value was found as 1.983, which is greater than 1. Therefore, slope is stable. Figure G.11 shows the slope stability check results.

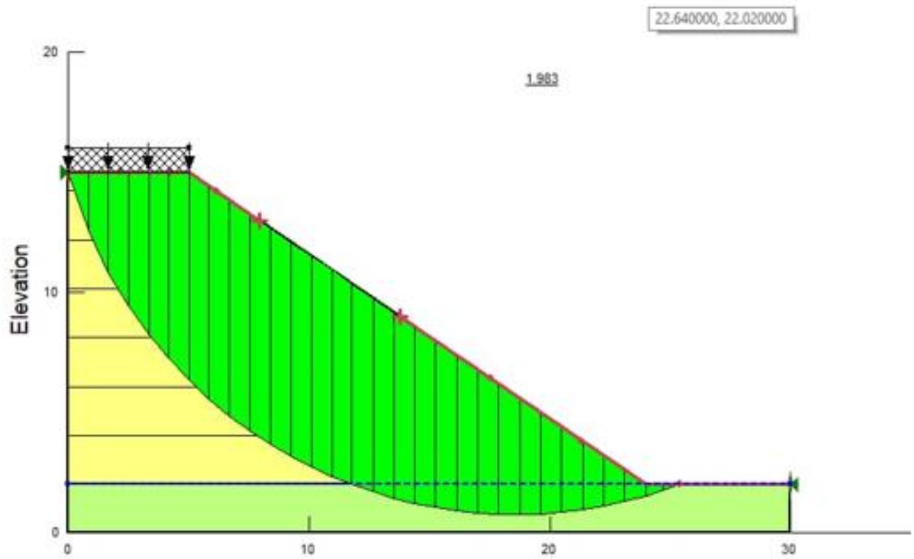


Figure G.11 Slope stability check results

Material calculation

Asphalt

Depth of the asphalt	=	50mm
Width of asphalt paving	=	7m
Length of asphalt paving	=	123.74m
Total volume	=	$43.309m^3$

ABC

Depth of the ABC	=	225mm
Width of ABC layer	=	7m
Length of ABC layer	=	123.74m
Total volume	=	$194.89m^3$

Gravel

This section material can't be calculated using manual calculation because of its laying area is complicated. we have used AutoCAD to calculate this value through inputting cross section drawings.

Total volume of gravel	=	$366m^3$

TACK COAT

Spraying width of tack coat	=	7m
Spraying length of tack coat	=	123.74m
Total area	=	$866.18m^2$

PRIME COAT

Spraying width of prime coat	=	7m
Spraying length of prime coat	=	123.74m
Total area	=	$866.18m^2$

	CURB CONCRETE Downstream curb's cross section of = $0.4916m^2$ Upstream curb's cross section = $0.1639m^2$ Length of curb section = 123.74m Total volume of curb concrete = (0.4316+0.1639)*123.74 = $74.35m^3$	

APPENDIX H TRAFFIC SIGNAL DESIGN

REFERENCE	CALCULATIONS	RESULTS

A traffic survey was carried out at the Peradeniya junction on 26th February 2020 from 6.30 AM to 8.30 AM. Traffic volumes were taken in 15 minutes interval. The traffic survey results are given in following tables.

MC – Motorcycles

3W – three-wheelers

C/V/J – Car /Van /Jeep

Table H.1 Gampola to Kandy Direction traffic volumes

TIME	MC	3W	C/V/J	BUS	LORRY	TOTAL
6.30-6.45	88	80	93	36	7	304
6.45-7.00	105	89	49	17	4	264
7.00-7.15	77	50	42	6	4	179
7.15-7.30	96	63	54	9	2	224
7.30-7.45	132	67	62	14	3	278
7.45-8.00	110	78	57	13	4	262
8.00-8.15	129	85	50	15	6	285
8.15-8.30	134	79	67	17	8	305

Table H.2 Gampola to Colombo Direction traffic volumes

TIME	MC	3W	C/V/J	BUS	LORRY	TOTAL
6.30-6.45	30	19	62	5	2	118
6.45-7.00	36	32	66	2	5	141
7.00-7.15	22	12	22	3	3	62
7.15-7.30	45	25	44	2	1	117
7.30-7.45	60	38	56	2	5	161
7.45-8.00	49	30	51	1	2	133
8.00-8.15	56	21	33	0	9	119
8.15-8.30	50	35	44	2	13	144

Table H.3 Kandy to Colombo Direction traffic volumes

TIME	MC	3W	C/V/J	BUS	LORRY	TOTAL
6.30-6.45	24	13	26	7	2	72
6.45-7.00	37	48	26	20	1	132
7.00-7.15	45	50	48	18	2	163
7.15-7.30	39	53	32	17	2	143

7.30-7.45	47	60	19	15	4	145
7.45-8.00	44	58	25	17	3	147
8.00-8.15	32	46	21	22	5	126
8.15-8.30	34	33	14	20	1	102

Table H.4 Kandy to Gampola Direction traffic volumes

TIME	MC	3W	C/V/J	BUS	LORRY	TOTAL
6.30-6.45	42	32	34	8	9	125
6.45-7.00	51	49	29	13	8	150
7.00-7.15	48	52	42	12	5	159
7.15-7.30	55	62	55	10	6	188
7.30-7.45	63	61	52	11	6	193
7.45-8.00	52	60	43	12	7	174
8.00-8.15	49	53	40	9	5	156
8.15-8.30	47	45	54	10	6	162

Table H.5 Colombo to Gampola Direction traffic volumes

TIME	MC	3W	C/V/J	BUS	LORRY	TOTAL
6.30-6.45	13	7	15	0	10	45
6.45-7.00	22	23	23	2	7	77
7.00-7.15	26	14	42	0	9	91
7.15-7.30	32	21	54	1	8	116
7.30-7.45	48	29	62	2	9	150
7.45-8.00	44	78	57	2	8	189
8.00-8.15	40	85	50	4	8	187
8.15-8.30	33	79	67	0	3	182

Table H.6 Colombo to Kandy Direction traffic volumes

TIME	MC	3W	C/V/J	BUS	LORRY	TOTAL
6.30-6.45	54	40	32	33	6	165
6.45-7.00	67	50	33	30	3	183
7.00-7.15	44	47	24	16	3	134
7.15-7.30	75	51	45	19	2	192
7.30-7.45	82	54	40	17	4	197
7.45-8.00	92	56	51	18	2	219
8.00-8.15	94	58	64	23	1	240
8.15-8.30	74	55	54	27	1	211

Passenger car unit volume for lanes

From traffic data, percentage of all type of vehicle was calculated. After that PCVs was calculated which are shown in following tables,

Table H.7 PCVs for Gampola to Kandy direction

vehicles	percentage	equivalent passenger car unit	PCVs
2W	0.41	0.5	0.21
3W	0.28	0.67	0.19
4W(C,V,J)	0.23	1	0.23
bus	0.06	2.25	0.14
lorry	0.02	2.25	0.05
	1		0.80

Table H.8 PCVs for Kandy to Gampola direction

vehicles	percentage	equivalent passenger car unit	PCVs
2W	0.31	0.5	0.16
3W	0.32	0.67	0.21
4W(C,V,J)	0.27	1	0.27
bus	0.06	2.25	0.14
lorry	0.04	2.25	0.09
	1		0.86

Table H.9 PCVs for Gampola to Colombo direction

vehicles	percentage	equivalent passenger car unit	PCVs
2W	0.35	0.5	0.18
3W	0.21	0.67	0.14
4W(C,V,J)	0.38	1	0.38
bus	0.02	2.25	0.05
lorry	0.04	2.25	0.09
	1		0.83

Table H.10 PCVs for Colombo to Gampola direction

vehicles	percentage	equivalent passenger car unit	PCVs
2W	0.25	0.5	0.13
3W	0.32	0.67	0.21
4W(C,V,J)	0.36	1	0.36
bus	0.01	2.25	0.02
lorry	0.06	2.25	0.14
	1		0.86

Table H.11 PCVs for Kandy to Colombo direction

vehicles	percentage	equivalent passenger car unit	PCVs
2W	0.29	0.5	0.15
3W	0.35	0.67	0.23
4W(C,V,J)	0.21	1	0.21
bus	0.13	2.25	0.29
lorry	0.02	2.25	0.05
	1		0.93

Table H.12 PCVs for Colombo to Kandy direction

vehicles	percentage	equivalent passenger car unit	PCVs
2W	0.38	0.5	0.19
3W	0.27	0.67	0.18
4W(C,V,J)	0.22	1	0.22
bus	0.12	2.25	0.27
lorry	0.01	2.25	0.02
	1		0.88

Table H.13 Critical 3 direction flow in PCU/h

	Flow	PCV	RT LAF	PAF	LAF	PCV	rounded value
Colombo to Kandy	520	0.86	-	1	1	447	450
Gampola to Kandy(RT)	1050	0.8	1.05	1	1	882	885
Kandy to Colombo(RT)	520	0.93	1.05	1	1.1	559	560

Factors are based on TRB 1980.

TRAFFIC SIGNAL SEQUENCES

Phase 1

Signal C: Red

Signal B: Red

Signal A: Green

Phase 2

Signal C: Red

Signal B: Green

Signal A: Red

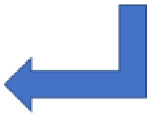
Phase 3

Signal C: Green

Signal B: Red

Signal A: Red

I – Gampola to Kandy



II- Kandy to Colombo

III- Colombo to Gampola

Critical lane volume

CL I (signal A) = 885 PCU/h

CL II (signal B) = 560 PCU/h

CL III (signal C) = 450 PCU/h

Sum of critical volume = 885+560+450

= 1895 PCU/h

Saturation Flow = 2800 PCU/h

Webster and Cobbe formula

Optimum cycle time (C_o) = $\frac{1.5L+5}{1-\varepsilon(\frac{q}{S})}$

Loss time (L) = loss time due to [(acceleration +deacceleration) +

amber time] *no of phase

standard amber time (a) : 3s-5s

acceleration time + deacceleration time

: 2s

L = 3*(2+3)

= 15s

C_o = $\frac{1.5*15+5}{1-(\frac{1895}{2800})}$

= 85s

	Effective green time(g) = $(C\text{-}L) * \frac{q}{\varepsilon(q)}$ g_I = $(85\text{-}15) * \frac{885}{1895}$ = 33s g_{II} = 21s g_{III} = 16s actual green time(G) G = g_i -a+l G_I = 33-3+2 = 32s G_{II} = 21-3+2 = 20s G_{III} = 16-3+2 = 15s Red time(R) R_i = $C\text{-}[G_i+a_i+(a_i/R_i)]$ R_I = 85-[32+3+3] = 47s R_{II} = 59s R_{III} = 64s Amber time, red amber time: 3s	

APPENDIX I UNDERPASS DESIGN

REFERENCE	CALCULATIONS	RESULTS

DESIGN LOADS

Permanent loads

- Dead load
- Super imposed load
- Horizontal earth pressure

Vertical live loads

- Traffic loads
- Pedestrian loads

Horizontal live loads

- Live load surcharge
- Traction

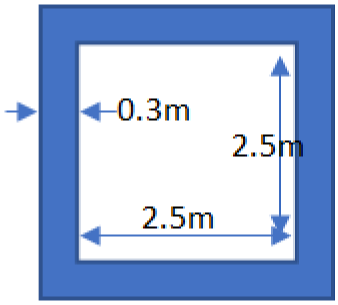


Figure I.1 Cross section of culvert

Cross section of underpass is shown in figure I.1

Width of culvert = 2.5m

Height =2.5m

Thickness of all elements = 0.3m

Material property:

Angle of friction of fill soil = 28°

Unit weight of backfill soil =18KN/m^3

Unit weight of concrete = 24KN/m^3

F_{ck} = 30Mpa

F_{yk} = 500Mpa

Cover = 50mm

.

Permanent actions

Self-weight of top, bottom slab = 0.3 × 25

= 7.5 KN/m^2

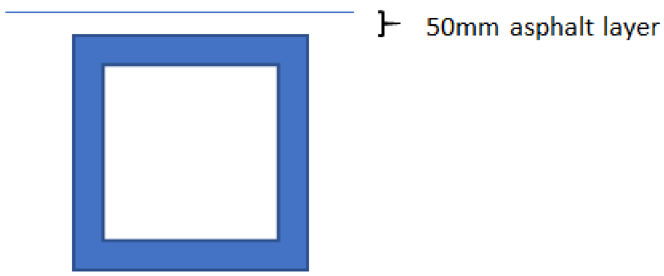


Figure I.2 Asphalt layer on the top slab

Permanent action from asphalt layer = 0.05 × 22

=1.1KN/m^2

Earth pressure

At rest earth pressure coefficient K_0 = 1-sin Ø

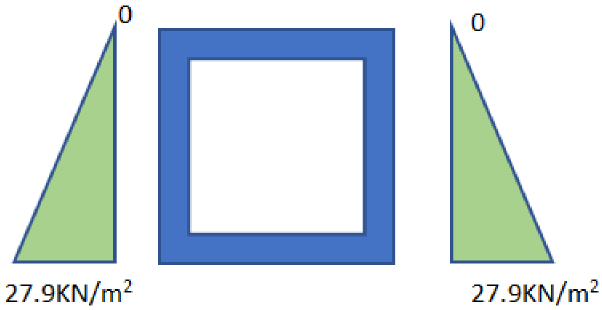
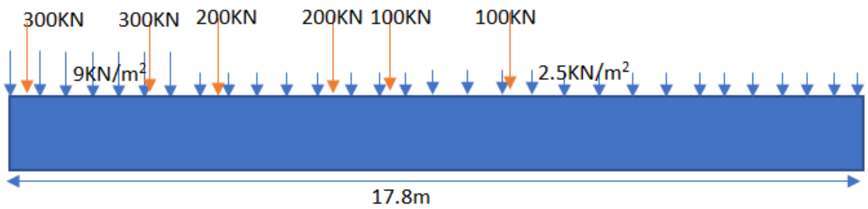
=1-sin(28)

=0.5

Maximum earth pressure on the side wall (P) =K_0PH

=0.5×18×3.1

=27.9KN/m^2

	 **Figure I.3 Earth pressure distribution on side walls**	
EN 1992-2 LM1	<u>live loads</u> Traffic load From EN1992-2 LM1, Traffic load distribution across the carriageway and national lane 1 are shown in figure I.4 & figure I.5.  **Figure I.4 Across the carriageway**	

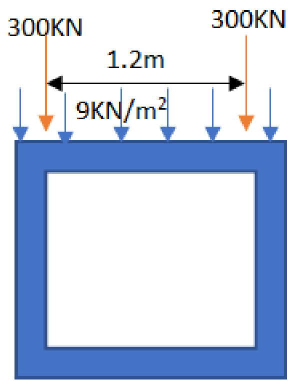
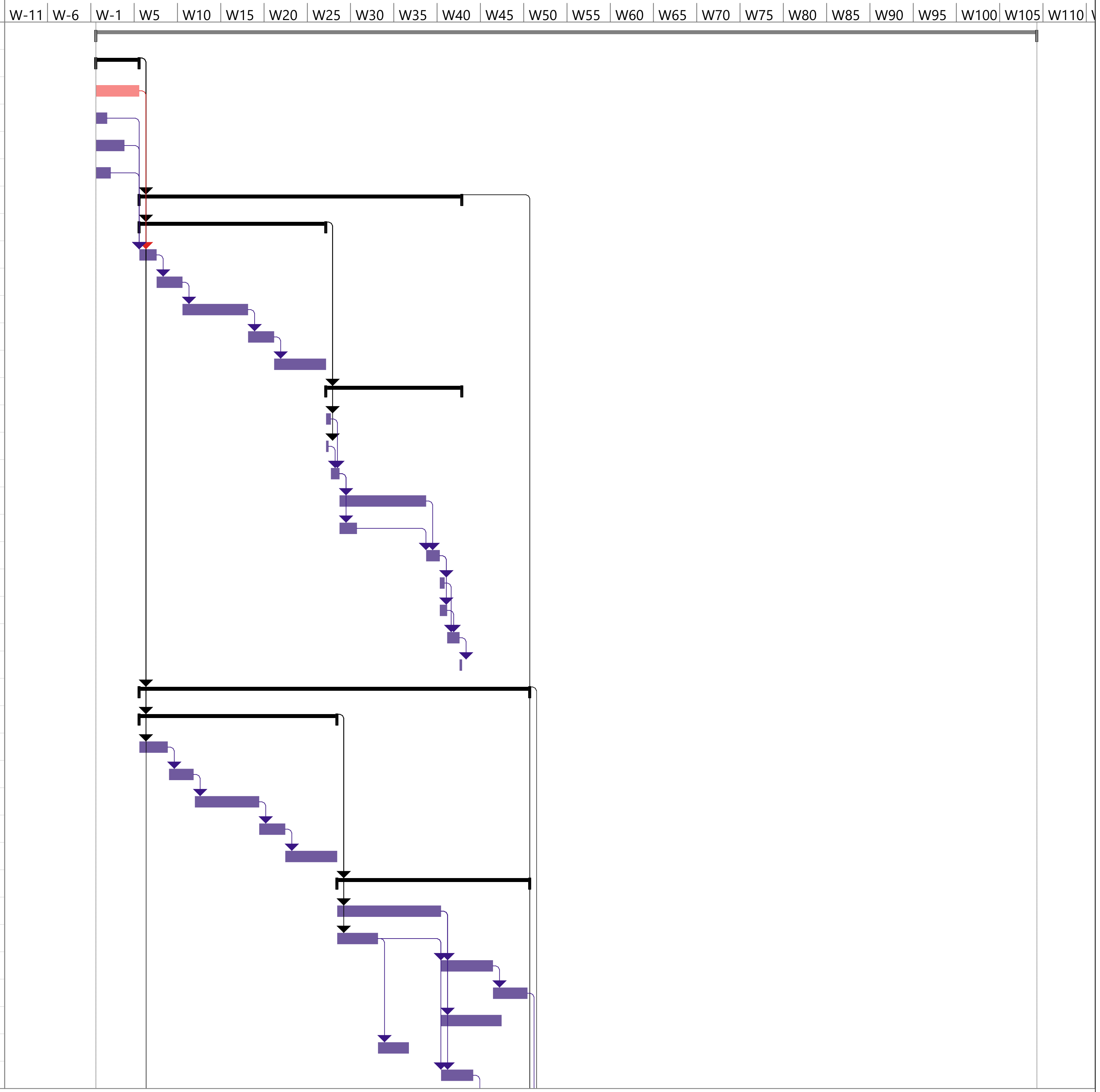


Figure I.5 Across the national lane 1

Pedestrian load on bottom slab	= $3KN/m^2$
Live load surcharge	
Assume live load surcharge q	$=10KN/m^2$
So, horizontal surcharge pressure	$=K_0q$
	$=0.5\times10$
	$=5KN/m^2$

APPENDIX J CONSTRUCTION PLAN

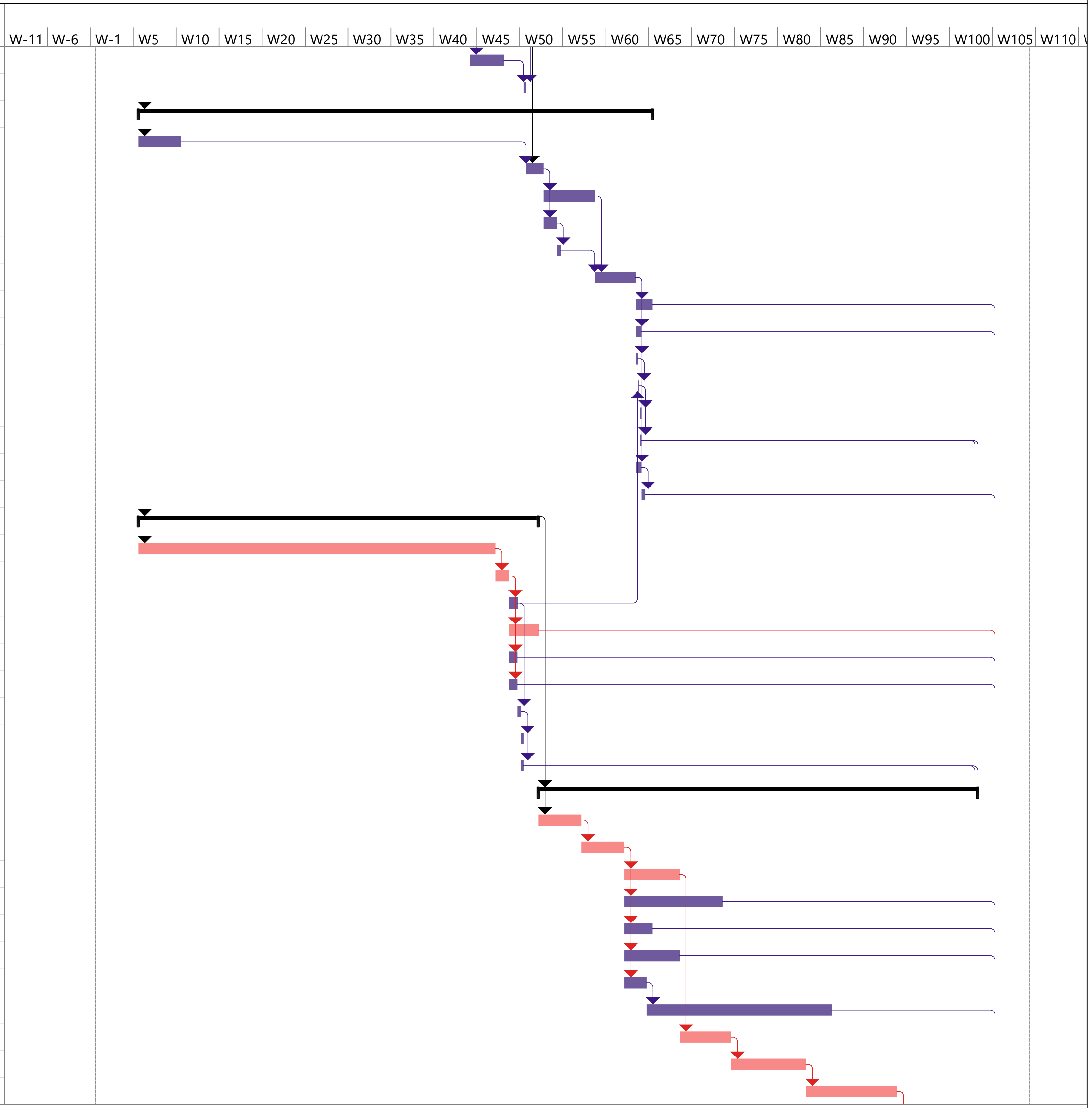
ID	Task Mode	Task Name	Duration	Start	Finish
0		**Project402**	**634 days**	**01/01/2021**	**31/01/2023**
1		**PRELIMINARY WORKS**	**30 days**	**01/01/2021**	**04/02/2021**
2		All the clearing and grubbing works	30 days	01/01/2021	04/02/2021
3		Removing trees	8 days	01/01/2021	09/01/2021
4		Other preliminary works	20 days	01/01/2021	23/01/2021
5		Required machines and equipment	10 days	01/01/2021	12/01/2021
6		**PIER CONSTRUCTION**	**215 days**	**05/02/2021**	**23/10/2021**
7		**PILE INSTALLATION FOR PIERS**	**120 days**	**05/02/2021**	**05/07/2021**
8		Coffer dam construction	12 days	05/02/2021	18/02/2021
9		pile boring work	18 days	19/02/2021	11/03/2021
10		supply and fixing reinforcement and	36 days	12/03/2021	03/05/2021
11		concreting piles	18 days	04/05/2021	24/05/2021
12		pile testing & other works	36 days	25/05/2021	05/07/2021
13		**BUILDING OF PIERS**	**95 days**	**06/07/2021**	**23/10/2021**
14		reinforcement for pile cap	4 days	06/07/2021	09/07/2021
15		formwork for pile cap	2 days	06/07/2021	07/07/2021
16		concreting pile cap	6 days	10/07/2021	16/07/2021
17		reinforcement for pier stem	60 days	17/07/2021	24/09/2021
18		formwork for pier stem	12 days	17/07/2021	30/07/2021
19		concreting pier stem	9 days	25/09/2021	05/10/2021
20		reinforcement for pier head	4 days	06/10/2021	09/10/2021
21		formwork for pier head	5 days	06/10/2021	11/10/2021
22		concreting pier head	9 days	12/10/2021	21/10/2021
23		supply and fixing bearing pads	2 days	22/10/2021	23/10/2021
24		**ABUTMENT & WINGWALL CONSTRUCTION**	**262 days**	**05/02/2021**	**17/12/2021**
25		**PILE INSTALLATION FOR ABUTMENTS**	**128 days**	**05/02/2021**	**14/07/2021**
26		Excavation work for abutments &	20 days	05/02/2021	27/02/2021
27		pile boring work	18 days	01/03/2021	20/03/2021
28		supply and fixing reinforcement and	36 days	22/03/2021	12/05/2021
29		concreting piles	18 days	13/05/2021	02/06/2021
30		pile testing & other works	36 days	03/06/2021	14/07/2021
31		**BUILDING OF ABUTMENTS & WINGWALLS**	**134 days**	**15/07/2021**	**17/12/2021**
32		reinforcement for right abutment & wing	72 days	15/07/2021	06/10/2021
33		formwork for right abutment & wing	28 days	15/07/2021	16/08/2021
34		concreting right abutment & wing walls	36 days	07/10/2021	17/11/2021
35		Backfilling work for right abutment &	24 days	18/11/2021	15/12/2021
36		reinforcement for left abutment & wing	42 days	07/10/2021	24/11/2021
37		formwork for left abutment & wing walls	22 days	17/08/2021	10/09/2021
38		concreting left abutment & wing walls	22 days	07/10/2021	01/11/2021



Project: Project402

Task; Split; Milestone; Summary; Project Summary; Inactive Task; Inactive Milestone; Inactive Summary; Manual Task; Duration-only; Manual Summary Rollup; Manual Summary; Start-only; Finish-only; External Tasks; External Milestone; Deadline; Critical; Critical Split; Progress; Manual Progress

ID	Task Mode	Task Name	Duration	Start	Finish
39		Backfilling work for left abutment & wing	24 days	02/11/2021	29/11/2021
40		supply and fixing bearing pads	2 days	16/12/2021	17/12/2021
41		**SUPERSTRUCTURE**	**350 days**	**05/02/2021**	**30/03/2022**
42		Casting precast beams and supply	30 days	05/02/2021	11/03/2021
43		Fixing precast beams	12 days	18/12/2021	31/12/2021
44		reinforcement for bridge deck	36 days	01/01/2022	11/02/2022
45		formwork for bridge deck	9 days	01/01/2022	11/01/2022
46		fixing drainage pipes for the bridge	3 days	12/01/2022	14/01/2022
47		concreting bridge deck	28 days	12/02/2022	16/03/2022
48		concreting footwalks and footwalk ramps	12 days	17/03/2022	30/03/2022
49		Supplying and constructing Asphaltic Plug	4 days	17/03/2022	21/03/2022
50		supply and fixing precast kerbs	2 days	17/03/2022	18/03/2022
51		Bitumen surfacing , 1st coat tarring (prime	1 day	19/03/2022	19/03/2022
52		Bitumen surfacing , subsequence 2nd coat	1 day	21/03/2022	21/03/2022
53		asphalt laying(50mm) and levelling	1 day	21/03/2022	21/03/2022
54		constructing end pilasters	4 days	17/03/2022	21/03/2022
55		supply and fixing precast railings	3 days	22/03/2022	24/03/2022
56		**APPROACH ROAD CONSTRUCTION**	**270 days**	**05/02/2021**	**27/12/2021**
57		soil filling with levelling to propper gradient	240 days	05/02/2021	22/11/2021
58		spreading type (I) gravel with watering and	10 days	23/11/2021	03/12/2021
59		ABC laying with compacting	6 days	04/12/2021	10/12/2021
60		drainage construction	20 days	04/12/2021	27/12/2021
61		Median constrution	6 days	04/12/2021	10/12/2021
62		Walking path construction	6 days	04/12/2021	10/12/2021
63		Bitumen surfacing , 1st coat tarring (prime	2 days	11/12/2021	13/12/2021
64		Bitumen surfacing , subsequence 2nd coat	2 days	14/12/2021	15/12/2021
65		asphalt laying(50mm) and levelling	2 days	14/12/2021	15/12/2021
66		**ROAD WIDENING SECTION**	**298 days**	**28/12/2021**	**20/12/2022**
67		removal of structures	30 days	28/12/2021	31/01/2022
68		excavation for roadway, underpass,	30 days	01/02/2022	07/03/2022
69		Underpass construction (Structural works)	30 days	08/03/2022	21/04/2022
70		side drainage construction	60 days	08/03/2022	26/05/2022
71		Median construction	20 days	08/03/2022	30/03/2022
72		Walking path construction	30 days	08/03/2022	21/04/2022
73		Rock filling for embankment construction	16 days	08/03/2022	25/03/2022
74		Furnish, spread and compact top soil with	120 days	26/03/2022	23/08/2022
75		subgrade,compaction	36 days	22/04/2022	02/06/2022
76		subbase laying and compacting	52 days	03/06/2022	02/08/2022
77		ABC laying and compacting	64 days	03/08/2022	15/10/2022



Project: Project402

Task	Inactive Task	Manual Summary Rollup	External Milestone	Manual Progress
Split	Inactive Milestone	Manual Summary	Deadline	
Milestone	Inactive Summary	Start-only	Critical	
Summary	Manual Task	Finish-only	Critical Split	
Project Summary	Duration-only	External Tasks	Progress	

ID		Task Mode	Task Name	Duration	Start	Finish
78			Bitumen surfacing , 1st coat tarring (prime	20 days	17/10/2022	08/11/2022
79			Bitumen surfacing , subsequence 2nd coat	4 days	09/11/2022	12/11/2022
80			asphalt laying(50mm) and levelling	36 days	09/11/2022	20/12/2022
81			Finishing work of Underpass (painting,	52 days	22/04/2022	21/06/2022
82			**LIGHTING, ROAD MARKING & OTHER WORKS**	**26 days**	**21/12/2022**	**19/01/2023**
83			supply and fixing street lamps and other	12 days	21/12/2022	03/01/2023
84			road marking and fixing road signs	2 days	21/12/2022	22/12/2022
85			other required works	14 days	04/01/2023	19/01/2023
86			**HAND OVER**	**10 days**	**20/01/2023**	**31/01/2023**

Project: Project402

Task | Split | Milestone | Summary | Project Summary | Inactive Task | Inactive Milestone | Inactive Summary | Manual Task | Duration-only | Manual Summary Rollup | Manual Summary | Start-only | Finish-only | External Tasks | External Milestone | Deadline | Critical | Critical Split | Progress | Manual Progress

APPENDIX K BILL OF QUANTITIES

Project : New Bridge for Peradeniya

BILL 01 : Estimate for Preliminary works

ITEM	DESCRIPTION	UNIT	QTY.	RATE Rs. Cts	Amount Rs. Cts.	Sub Total Rs. Cts.
A	**PRELIMINARIES**					
A.1	Allow for erection of temporary sheds for protection & storage of materials & maintain same for the duration of contract	Item	1.00	51,750.00	51,750.00	
A.2	Allow for erection of temporary sanitary accomodation at site & maintain same for the duration of the construction period.	Item	1.00	42,750.00	42,750.00	
A.3	Allow for erection of temporary site offices to accommodate contrator's staff & maintain same for the duration of the construction period.	Month	14.00	75,000.00	1,050,000.00	
A.4	Allow lump sum for providing & maintaining accomodation for workers & staff with necessary sanitary facilities and other relevant services required to maintain health & safety of workers and retained at the completion.	Item	1.00	32,250.00	32,250.00	
A.5	Allow for management ,safety & control of traffic in existing road including barricading lighting (according to traffic management plan with bollards & stackable safety barriers & cones), watching, traffic controlling (with provision for vehicles, pedestrians & cyclist), providing temporary notice/sign boards & periodic maintenances of existing road (priming, watering trimming, clearing of road sides with grading drainage ,etc.), etc. as instructed & approved by the engineer.	Month	14.00	25,000.00	350,000.00	
A.6	Allow for the provision of bonds and guarantees (Advance bond & performance bond).	Item	1.00	57,750.00	57,750.00	
A.7	Allow lump sum for all costs in connection with supplying, specimens, preparing samples for testing, making arrangements for testing materials, goods etc, obtaining test reports and submitting the same for the approval of Engineer.	Item	1.00	105,450.00	105,450.00	
A.8	Allow lump sum for suppying temporary electricity for the works including temporary connection & internal distribution arrangements.	Item	1.00	856,500.00	856,500.00	

A.9	Allow for temporary water connection & supply of water to site or an alternate means for the supply of water for the works during construction.	Item	1.00	745,000.00	745,000.00	
A.10	Allow for providing necessary security and security lighting system thruoghout the construction period.	Item	1.00	147,750.00	147,750.00	
A.11	Allow for mobilization & demobilization with site plan & necessary fencing & gates of work site including testing laboratory, curing area, setting out & Survey.	Item	1.00	12,000,000.00	12,000,000.00	
A.12	Provision of Project Name Boards as Directed by the Engineer	Nos	2.00	75,000.00	150,000.00	
						15,589,200.00
TOTAL PRELIMINARY WORKS CARRIED TO SUMMARY						**15,589,200.00**

Project : New Bridge for Peradeniya

BILL 01 : Estimate for Abutments & Wingwalls

ITEM	DESCRIPTION	UNIT	QTY.	RATE Rs. Cts	Amount Rs. Cts.	Sub Total Rs. Cts.
A	**CLEARING & GRUBBING**					
A.1	clearing & Grubbing	m^2	600.00	70.00	42,000.00	
						42,000.00
B	**Excavation & Backfill**					
B.1	excavation in unclassified soil and backfill for abutments and wingwalls	m^3	183.75	513.20	94,300.50	
B.2	Backfilling process	m^3	955.75	1,436.00	1,372,457.00	
						1,466,757.50
C	**PILE FOUNDATION (18 piles with 600 mm)**					
C.1	boring thruogh soil layer and all other works	m	117.00	14,000.00	1,638,000.00	
C.2	rock socketing works and all other works	m	99.00	78,510.00	7,772,490.00	
C.3	concreting process of piles with grade C32/40	m^3	244.29	40,000.00	9,771,600.00	
C.4	high steel reinforcement for piles	MT	9.33	192,500.00	1,796,025.00	
C.5	Cross - hole sonic logging test for pile integrity of bored pile.	Nos	18.00	45,600.00	820,800.00	
C.6	Load test on bored piles	Item	P. S	2,500,000.00	2,500,000.00	
						24,298,915.00
D	**Formwork for abutments & wingwalls**					
D.1	Rough finish formwork.	m^2	477.80	1,277.65	610,461.17	
D.2	Smooth finish formwork.	m^2	850.37	2,717.55	2,310,922.99	
						2,921,384.16
E	**Reinforcement for abutments & wingwalls**					
E.1	High yield steel bars	M.T.	56.02	192,500.00	10,784,235.00	
E.2						10,784,235.00
F	**CONCTRETING ABUTMENTS & WINGWALLS**					
F.1	concreting with C30/37 without reinfocement and formwork	m^3	823.99	21,524.80	17,736,219.95	
						17,736,219.95
	TOTAL ABUTMENTS & WINGWALL WORKS CARRIED TO SUMMARY					**57,249,511.62**

Project : New Bridge for Peradeniya

BILL 02 : Estimate for Piers

ITEM	DESCRIPTION	UNIT	QTY.	RATE Rs. Cts	Amount Rs. Cts.	Sub Total Rs. Cts.
A	**CLEARING & GRUBBING**					
A.1	clearing and grubbing	m^2	72.00	70.00	5,040.00	
						5,040.00
B	**PILE FOUNDATION (18 piles with 600 mm)**					
B.1	Construction of necessary cofferdams cribs sheeting & required works	Item	L. S	14,000,000.00	14,000,000.00	
B.2	dewatering	Item	L. S	3,000,000.00	3,000,000.00	
B.3	boring thruogh soil layer and all other works	m	90.00	14,000.00	1,260,000.00	
B.4	rock socketing works and all other works	m	90.00	78,510.00	7,065,900.00	
B.5	concreting process of piles with grade C32/40	m^3	223.93	40,000.00	8,957,200.00	
B.6	high steel reinforcement for piles	MT	8.55	192,500.00	1,645,875.00	
B.7	Cross - hole sonic logging test for pile integrity of bored pile.	Nos	18.00	45,600.00	820,800.00	
B.8	Load test on bored piles	Item	P. S	2,500,000.00	2,500,000.00	
						39,249,775.00
C	**REINFORCEMENT**					
C.1	high steel reinforcement for pile cap	MT	1.66	192,500.00	319,091.85	
C.2	high steel reinforcement for pier stem	MT	30.96	192,500.00	5,959,030.00	
C.3	high steel reinforcement for pier head	MT	1.90	192,500.00	365,110.90	
						6,643,232.75
D	**FORMWORK**					
D.1	smooth finish formwork for pile cap	m^2	65.88	2,717.55	179,032.19	
D.2	smooth finish formwork for pier stem	m^2	324.00	2,717.55	880,486.20	
D.3	smooth finish formwork for pier head	m^2	104.80	2,717.55	284,799.24	
						1,344,317.63
E	**CONCTRETE FOR STRUCTURES**					
E.1	concreting pile cap with C32/40 without reinfocement and formwork	m^3	66.98	21,524.80	1,441,731.10	
E.2	concreting pier stem with C32/40 without reinfocement and formwork	m^3	129.60	21,524.80	2,789,614.08	
E.3	concreting pier head with C32/40 without reinfocement and formwork	m^3	44.00	21,524.80	947,091.20	
						5,178,436.38
	TOTAL PIER WORKS CARRIED TO SUMMARY					**52,420,801.76**

Project : New Bridge for Peradeniya

BILL 03 : Estimate for Superstructure

ITEM	DESCRIPTION	UNIT	QTY.	RATE Rs. Cts	Amount Rs. Cts.	Sub Total Rs. Cts.
A	**PRESTRESSED CONCRETE**					
A.1	Pre-tentioned precast Y6 beams length of 25m supplies as per drawing,(C50/60 , 16 mm dia Y186OS7 strands)	Nos.	15.00	1,000,000.00	15,000,000.00	
						15,000,000.00
B	**CONCRETE FOR STRUCTURES**					
B.1	Concrete of grade C32/40 for bridge deck	m^3	150.00	22,284.60	3,342,690.00	
B.2	Concrete of grade C25/30 for footwalks and footwalk ramps	m^3	46.00	19,311.60	888,333.60	
						4,231,023.60
C	**FORMWORK**					
C.1	Smoothfinish form work underside for deck concreting	m^2	675.00	2,717.55	1,834,346.25	
C.2	Smoothfinish form work on sides for deck and footwalk	m^2	115.00	2,717.55	312,518.25	
						2,146,864.50
D	**REINFORCEMENT**					
D.1	High Yield steel bars of 25 mm diameter for bridge deck (3.85kg/m)	M.T.	17.50	192,500.00	3,368,750.00	
						3,368,750.00
E	**RAILINGS**					
E.1	Precast reinforced concrete railing and uprights in Class A Grade C 20/25 concrete inclusive of light reinforcement.	m	190.00	7,873.20	1,495,908.00	
						1,495,908.00
F	**CONCRETE KERBS**					
F.1	Precast Kerb in Class B Grade 20/25 concrete with light reinforcement	m	190.00	2,469.40	469,186.00	
						469,186.00
G	**EXPANSION JOINTS**					
G.1	Supplying and constructing Asphaltic Plug Joints	kg	52.00	700.00	36,400.00	
						36,400.00
H	**MISCELLANEOUS**					

H.1	8 Inch Dia. 200mm long stainless stell dowels supplied fixed and grouted	Nos.	170.00	845.07	143,661.90	
H.2	Stainless steel metal bar grating plates (30 cm x100cm)	Nos.	26.00	4,895.23	127,275.98	
H.3	Enviro Bridgedeck	m	36.00	5,000.00	180,000.00	
						450,937.88
I	**BRIGDE BEARINGS**					
I.1	Bearing pads	m	16.77	11,505.00	192,938.85	
						192,938.85
J	**BRIDGE PAVEMENT**					
J.1	**Epoxy rasin Prime layer**, Bituminious emulsion of Grade CSS-1 applied at a rate of 0.9-1.5 liters per square meter over the concrete deck	m^2	715.00	132.50	94,737.50	
J.2	Mastic Asphalt Protection layer applied at a rate of 0.5 liters per square meter over the prime layer	m^2	715.00	62.50	44,687.50	
J.3	Asphaltic Concrete binder course compacted to a thickness of 50 mm in position	m^2	715.00	1,625.00	1,161,875.00	
						1,301,300.00
K	**LIGHTING**					
K.1	Steel LED lamp post with Single arm bracket, supplied & fixed as per drawing This includes installation of electric cables, electric cables connecting other lamp posts, fixing of lamps, foundation and other necessary with electrical meter connection.	Nos	6.00	270,000.00	1,620,000.00	
						1,620,000.00
TOTAL SUPERSTRUTURE WORKS CARRIED TO SUMMARY						**30,313,308.83**

Project : New Bridge for Peradeniya

BILL 04 : Estimate for Road Widening & Slope protection

ITEM	DESCRIPTION	UNIT	QTY.	RATE Rs. Cts	Amount Rs. Cts.	Sub Total Rs. Cts.
A	**CLEARING & GRUBBING**					
A.1	Clearing and grubbing	m^2	32340.00	70.00	2,263,800.00	
						2,263,800.00
B	**REMOVING TREES**					
B.1	Girth 600 to 1200mm	Nos	4.00	5,000.00	20,000.00	
B.2	Girth 1200 to 2000mm	Nos	3.00	7,500.00	22,500.00	
						42,500.00
C	**REMOVAL OF STRUCTURE &OBSTRUCTION**					
C.1	Dismantle & remove brick masonry structures (provinsional quantity)	m^3	2908.30	2,277.66	6,624,118.58	
C.2	Dismantle & remove random rubble masonry structures (provinsional quantity)	m^3	124.74	2,070.60	258,286.64	
C.3	Dismantle &remove concrete structures (provinsional quantity)	m^3	885.40	3,796.10	3,361,066.94	
C.4	Remove fencing (provinsional quantity)	m	350.00	828.24	289,884.00	
						10,533,356.16
D	**ROADWAY EXCAVATION**					
D.1	Roadway excavation, unsuitable soil	m^3	2083.66	1,294.00	2,696,256.04	
						2,696,256.04
E	**UNDERPASS CONSTRUCTION**					
E.1	Roadway excavation, unsuitable soil	m^3	716.80	1,294.00	927,539.20	
E.2	Roadway pavement excavation	m^3	44.44	869.65	38,647.25	
						966,186.45
F	**SIDE WALKING PATH EXCAVATION**					
F.1	Chanel excavation, unsuitable soil	m^3	166.49	952.65	158,601.94	
						158,601.94
G	**CHANNEL EXCAVATION**					
G.1	Chanel excavation, unsuitable soil	m^3	595.67	952.65	567,460.26	
						567,460.26
H	**STRUCTURE EXCAVATION AND BACKFILL**					

H.1	Excavation for structures in unsuitable soil and backfill with suitable soil	m^3	810.00	2,320.00	1,879,200.00	
						1,879,200.00
I	**GRANULAR PAVEMENT**					
I.1	Sub base as compacted in position	m^3	2194.46	2,528.80	5,549,342.86	
I.2	Dence graded aggregete base as compacted position (ABC)	m^3	2461.73	5,700.00	14,031,832.50	
						19,581,175.36
J	**SURFACE APPLICATION AND SURFACING**					
J.1	Prime coat with emulsion / cold bitumen (CSS-1) using 1ltr/sq.m including blinding with sand at the rate of 250 sq.m/cu.m and brashing clearing and moistening road surface (including cost of emulsion)	m^2	10941.00	132.50	1,449,682.50	
J.2	Tack coat using emulsion (CRS-1) @ the rate of 0.5ltr/sq.m inclusive of brushing, clearing road surface and cost of emulsion	m^2	10941.00	45.00	492,345.00	
J.3	Asphalt concrete in wearing course 50mm compacted thickness	m^2	10451.16	1,625.00	16,983,135.00	
						17,475,480.00
K	**CONCRETE**					
K.1	Concrete grade of C20 for drain	m^3	369.11	19,325.60	7,133,272.22	
K.2	Concrete grade of C25 for precast concrete cover slab	m^3	161.03	22,086.40	3,556,572.99	
						10,689,845.21
L	**REINFORCEMENT**					
L.1	Steel bars of 10mm diameter for drain	MT	11.00	192,500.00	2,116,762.73	
L.2	Steel bars of 10mm diameter for cover slab	MT	12.67	192,500.00	2,438,975.00	
						4,555,737.73
M	**FORMWORK**					
M.1	Smooth finish formwork for drain	m^2	3356.76	1,200.00	4,028,112.00	
M.2	Smooth finish formwork for cover slab	m^2	1158.04	1,200.00	1,389,648.00	
						5,417,760.00
N	**ROAD MARKING & ROAD SIGNS**					

N.1	Application of road markings in white (rumble strips, continuous, broken & hatch lines) in 5mm thick 150mm width as per RDA specification ; Engineer's instruction with reflectorized thermoplastic materials.	m^2	392.67	1,680.00	659,685.60	
N.2	Supply & erection of sign boards with reflective diamond type sticker as per specification & engineer's instruction.	Nos	6.00	18,000.00	108,000.00	
						767,685.60
O	**CONCRETE KERBS AND PAVING BLOCKS**					
O.1	Lay standard type road kerbs (125x900 mm) set on 15mm thick 1:3 cement mortar layer on 100x275 mm 1:2:4 concrete foundation including excavation and shuttering (cost of kerb stones) for road edges	L.m	1337.40	2,150.00	2,875,410.00	
O.2	Lay standard type road kerbs (125x900 mm) set on 15mm thick 1:3 cement mortar layer on 100x275 mm 1:2:4 concrete foundation including excavation and shuttering (cost of kerb stones) for median	L.m	1206.00	2,150.00	2,592,900.00	
O.3	Paving blocks, 50mm quarry dust compacted for road edges	m^2	679.81	2,415.00	1,641,741.15	
O.4	Paving blocks, 50mm quarry dust compacted for median	m^2	336.77	2,415.00	813,299.55	
						7,923,350.70
Q	**EMBANKMENT CONSTRUCTION**					
Q.1	Furnish, spread and compact top soil	m^2	24750.00	345.10	8,541,225.00	
Q.2	Boulder packing for soft ground treatment (embankment construction using rock fill)	m^3	1782.00	3,001.50	5,348,673.00	
Q.3	Grass sodding	m^2	24750.00	462.43	11,445,142.50	
						25,335,040.50
R	**LIGHTING**					
R.1	Steel LED lamp post with Single arm bracket, supplied & fixed as per drawing This includes installation of electric cables, electric cables connecting other lamp posts, fixing of lamps, foundation and other necessary with electrical meter connection.	Nos	13.00	270,000.00	3,510,000.00	
R.2	Signal light system post including supply, electric cables, electric cables connecting, fixing of lamps, foundation and other necessary with electrical meter connection.	Nos	3.00	500,000.00	1,500,000.00	

						5,010,000.00
TOTAL ROAD WIDENING & SLOPE PROTECTION WORKS CARRIED TO SUMMARY						**115,863,435.94**

Project : New Bridge for Peradeniya

BILL 05 : Estimate for New Road Section

ITEM	DESCRIPTION	UNIT	QTY.	RATE Rs. Cts	Amount Rs. Cts.	Sub Total Rs. Cts.
A	**REMOVING TREES**					
A.1	Girth 600 to 1200mm	Nos	15.00	5,000.00	75,000.00	
A.2	Girth 1200 to 2000mm	Nos	5.00	7,500.00	37,500.00	
						112,500.00
B	**CLEARING & GRUBBING**					
B.1	Clearing all Shrubs, Weeds with roots including cutting of over hanging branches of tress within the road reservation and transport away from the site and burnt, as directed.	m^2	1608.00	70.00	112,560.00	
						112,560.00
C	**FILLING WORKS**					
C.1	filling & levelling existing entire length of road platform to proper gradient by Motor grader 120 - 135 H.P. All levelling & raising road platform & consolidate it by 7 Ton vibrating roller. (Rate including hire charge of roller)	m^3	23901.00	565.37	13,512,908.37	
C.2	Supplying approved quality Gravel and pilled at site as no spreading will commencement before the piles measurement are taking over.	m^3	366.00	1,000.00	366,000.00	
C.3	Spreading Gravel by layer, then levelling ,watering and consolidating by 7 ton vibrating roller (Rate including hire charge of roller)	m^3	366.00	300.00	109,800.00	
						13,988,708.37
D	**GRANULAR PAVEMENT**					
D.1	Dence graded aggregete base as compacted position (ABC)	m^3	195.00	5,700.00	1,111,500.00	
						1,111,500.00
E	**CONCRETE KERBS AND PAVING BLOCKS**					

E.1	Lay standard type road kerbs (125x900 mm) set on 15mm thick 1:3 cement mortar layer on 100x275 mm 1:2:4 concrete foundation including excavation and shuttering (cost of kerb stones) for road edges	L.m	125.00	2,150.00	268,750.00	
E.2	Lay standard type road kerbs (125x900 mm) set on 15mm thick 1:3 cement mortar layer on 100x275 mm 1:2:4 concrete foundation including excavation and shuttering (cost of kerb stones) for median	L.m	125.00	2,150.00	268,750.00	
E.3	Paving blocks, 50mm quarry dust compacted for road edges	m^2	59.38	2,415.00	143,390.63	
E.4	Paving blocks, 50mm quarry dust compacted for median	m^2	68.75	2,415.00	166,031.25	
						846,921.88
F	**SURFACE APPLICATION AND SURFACING**					
F.1	Prime coat with emulsion / cold bitumen (CSS-1) using 1ltr/sq.m including blinding with sand at the rate of 250 sq.m/cu.m and brashing clearing and moistening road surface (including cost of emulsion)	m^2	866.18	132.50	114,768.85	
F.2	Tack coat using emulsion (CRS-1) @ the rate of 0.5ltr/sq.m inclusive of brushing, clearing road surface and cost of emulsion	m^2	866.18	45.00	38,978.10	
F.3	Asphalt concrete in wearing course 50mm compacted thickness	m^2	866.18	1,625.00	1,407,542.50	
						1,561,289.45
G	**ROAD MARKING & ROAD SIGNS**					
G.1	Application of road markings in white (rumble strips, continuous, broken & hatch lines) in 5mm thick 150mm width as per RDA specification ; Engineer's instruction with reflectorized thermoplastic materials.	m^2	56.25	1,680.00	94,500.00	
G.2	Supply & erection of sign boards with reflective diamond type sticker as per specification & engineer's instruction.	Nos	2.00	18,000.00	36,000.00	
						130,500.00
H	**LIGHTING**					

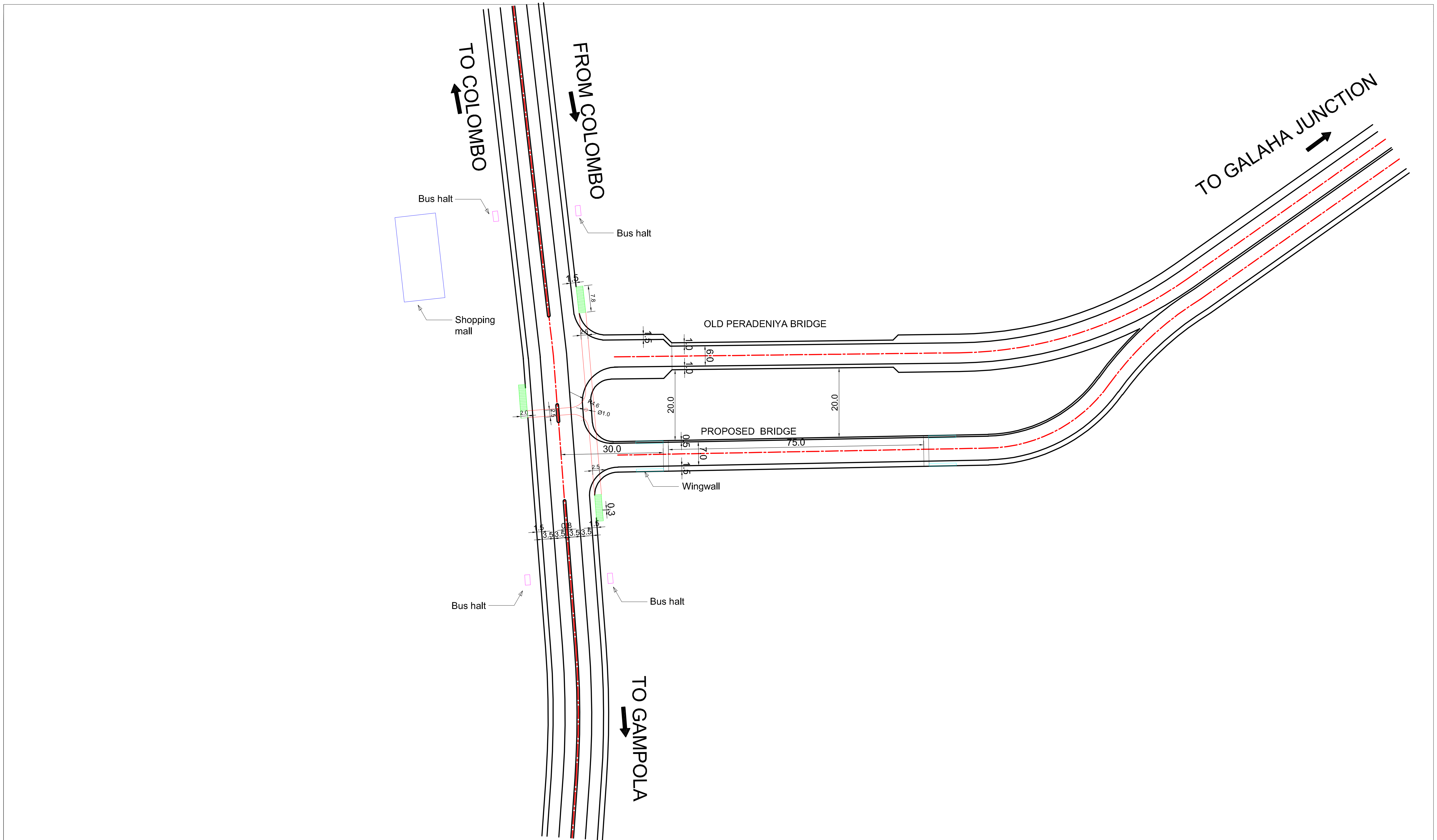
H.1	Steel LED lamp post with Single arm bracket, supplied & fixed as per drawing This includes installation of electric cables, electric cables connecting other lamp posts, fixing of lamps, foundation and other necessary with electrical meter connection.	Nos	5.00	270,000.00	1,350,000.00	
						1,350,000.00
TOTAL NEW ROAD SECTION WORKS CARRIED TO SUMMARY						**19,213,979.70**

Project : New Bridge for Peradeniya

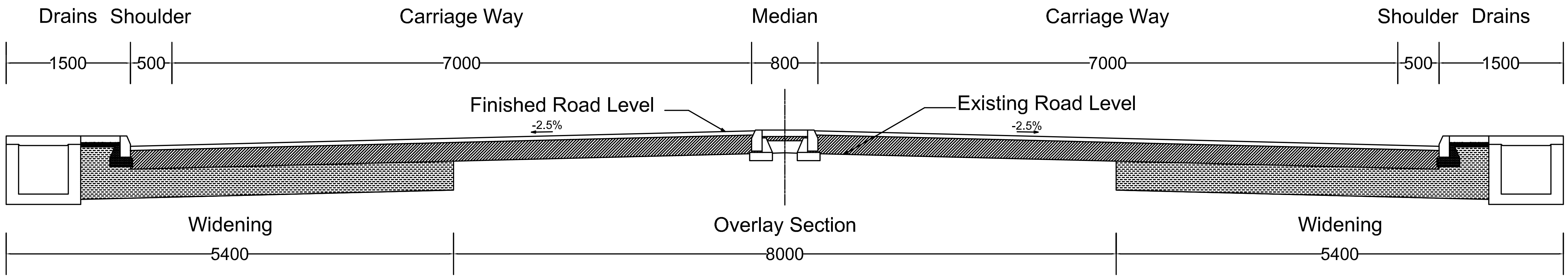
Contract sum analysis

Bill No	Description	Amount / Rs. Cts.
01	Preliminary works	15,589,200.00
02	Abutments & wingwalls	57,249,511.62
03	Piers	52,420,801.76
04	Superstructure	30,313,308.83
05	Road widening & Slope protection	115,863,435.94
06	New road section	19,213,979.70
SUB TOTAL		290,650,237.84
CONTRACT PRICE WITHOUT VAT		290,650,237.84
VAT (15%)		43,597,535.68
CONTRACT PRICE WITH VAT		**334,247,773.51**

APPENDIX L   DETAILED DRAWINGS



CLIENT : PANEL D CE 402	NEW BRIDGE FOR PERADENIYA	CONSULTANTS AND ARCHITECT : GROUP - D1	TITLE : SITE LAYOUT PLAN		
			DRAWN BY : PATHIRANA A.P.U.M.	DESIGN BY :	DATE : 05/06/2020 SCALE : 1:1000
			ALL DIMENSIONS ARE IN 'm'	SIGNATURE :	DRAWING NO. :



TYPICAL CROSS SECTION OF THE ROAD

CLIENT :

PANEL D
CE 402

NEW BRIDGE FOR PERADENIYA

CONSULTANTS AND ARCHITECT :
GROUP - D1

TITLE : CROSS SECTION OF THE ROAD

DRAWN BY :
PATHIRANA A.P.U.M.

DESIGN BY :
SOMASEKARA M.H.Y.S.

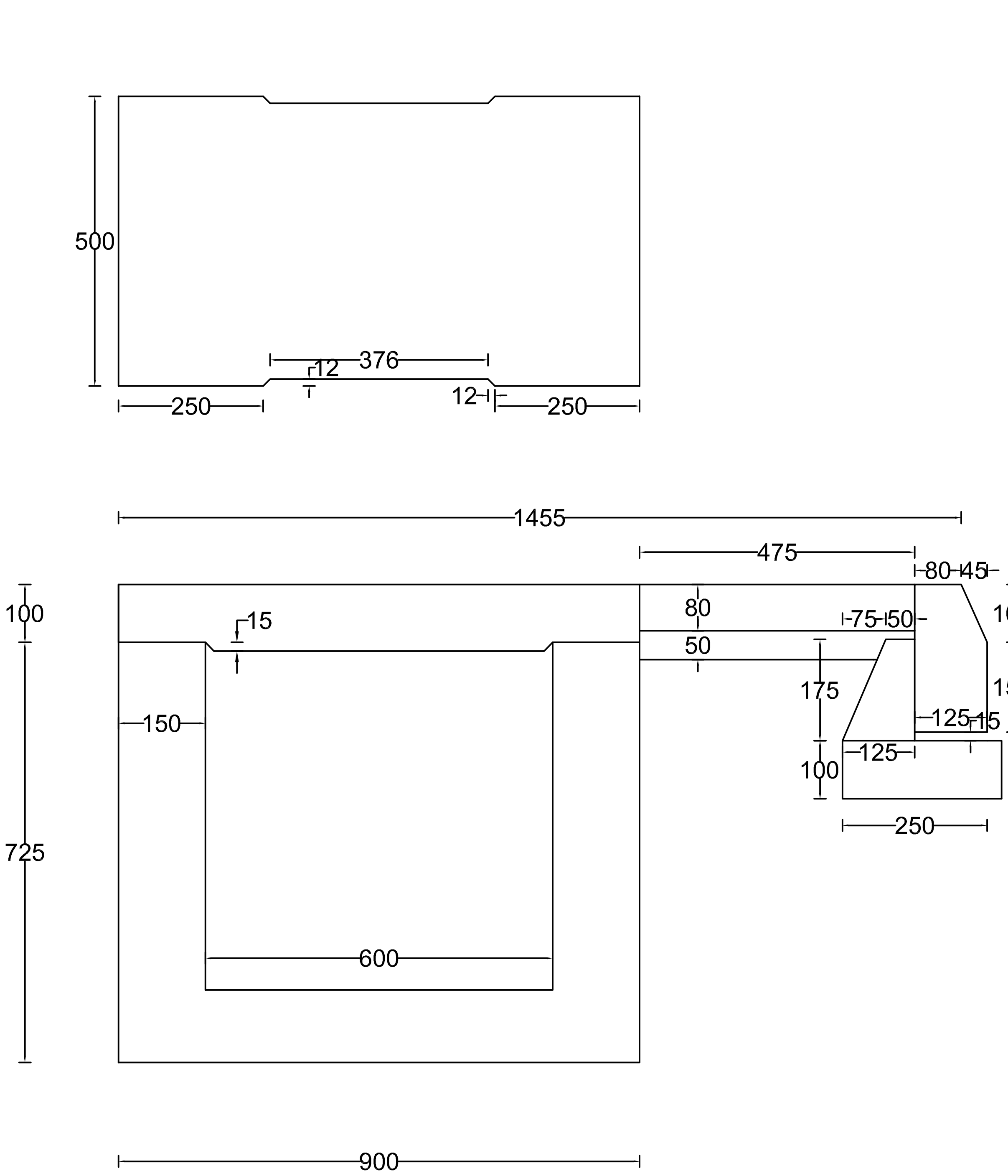
DATE :
05/06/2020

SCALE :
1:50

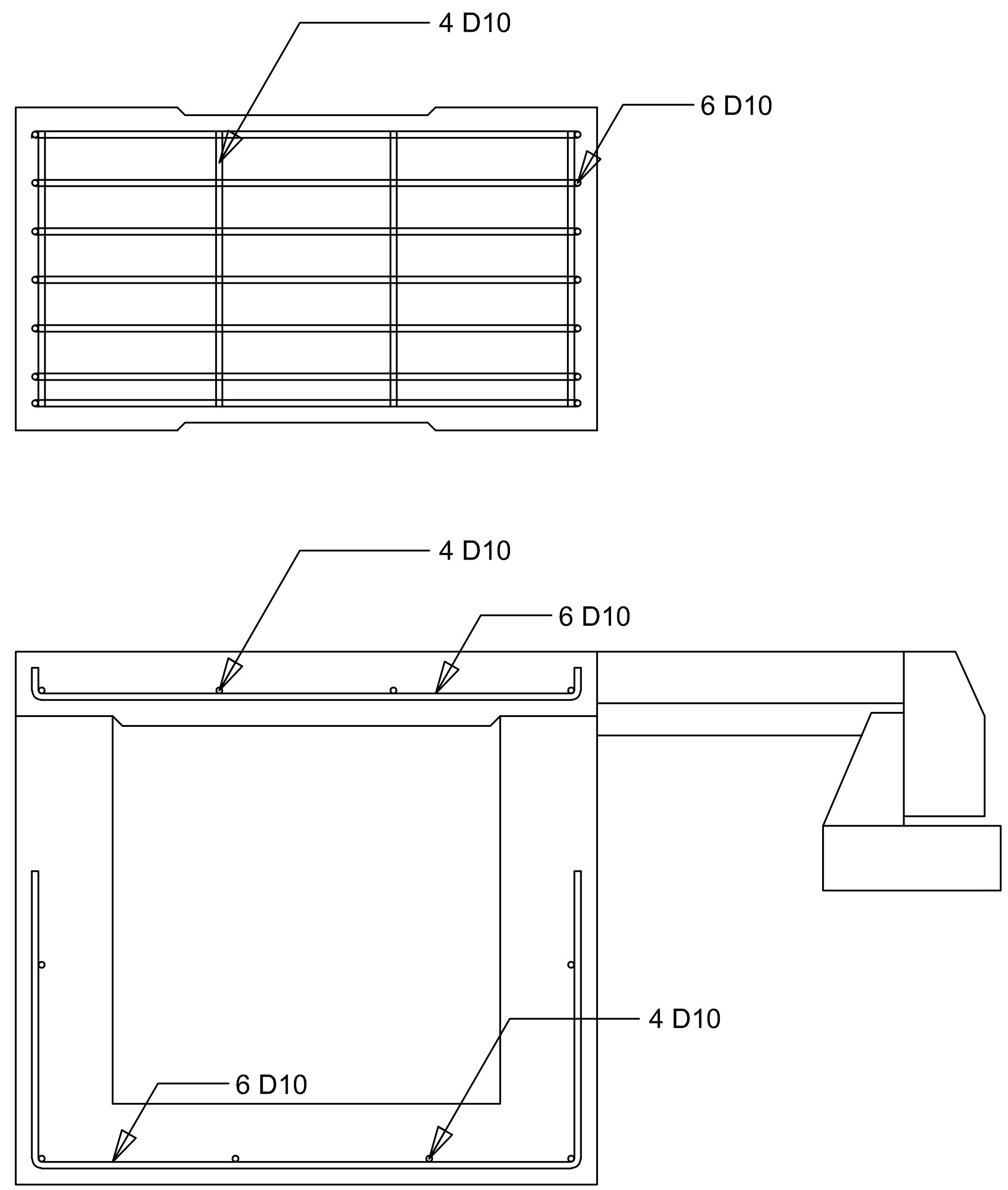
ALL DIMENSIONS ARE IN 'mm'

SIGNATURE :

DRAWING NO. :



DRAINAGE OF THE ROAD



REINFORCEMENT OF THE DRAINAGE

CLIENT :

PANEL D
CE 402

NEW BRIDGE
FOR
PERADENIYA

CONSULTANTS AND ARCHITECT :
GROUP - D1

TITLE : ROAD DRAINAGE

DRAWN BY :
PATHIRANA A.P.U.M.

DESIGN BY :
SOMASEKARA M.H.Y.S.

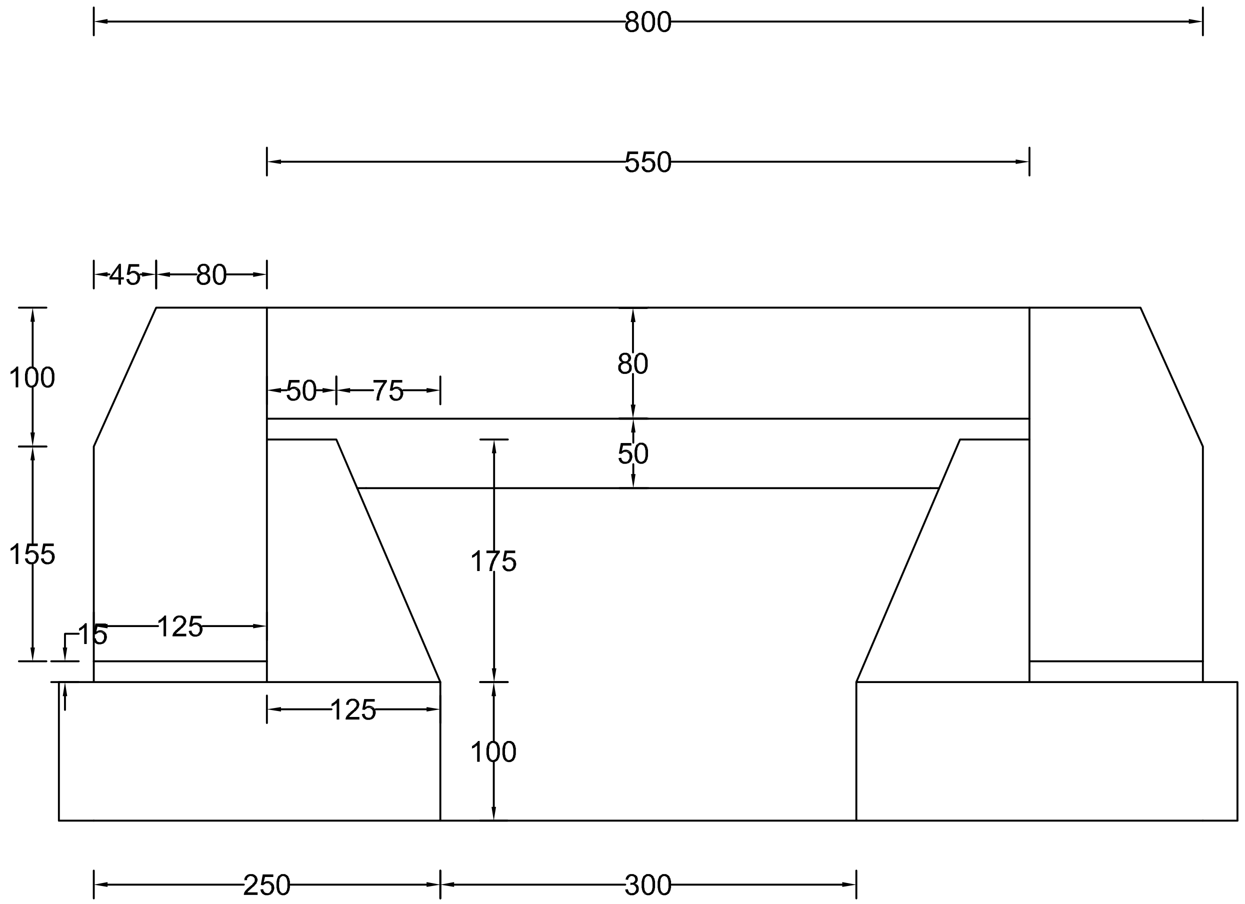
DATE :
05/06/2020

SCALE :
1:10

ALL DIMENSIONS ARE IN 'mm'

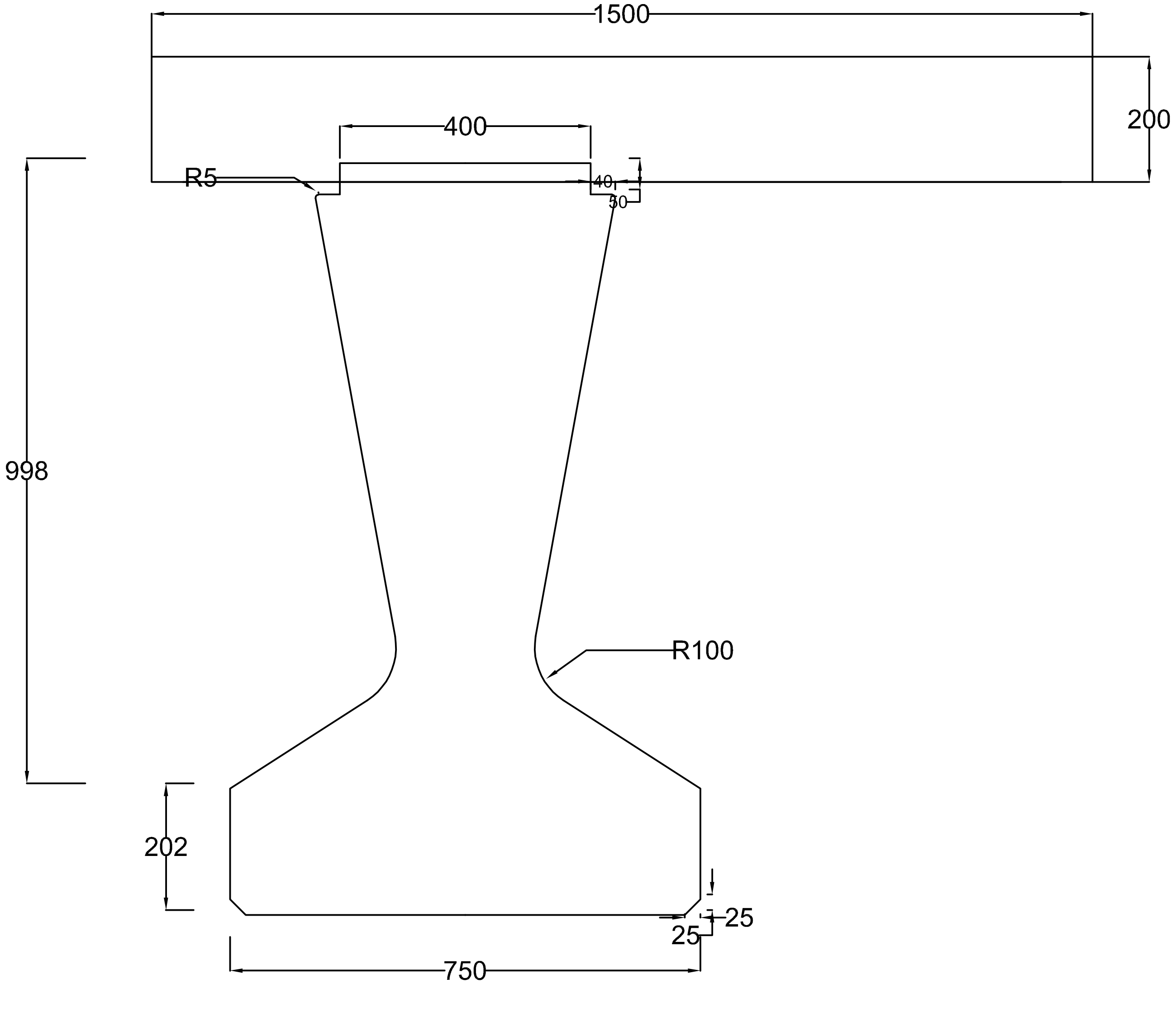
SIGNATURE :

DRAWING NO. :

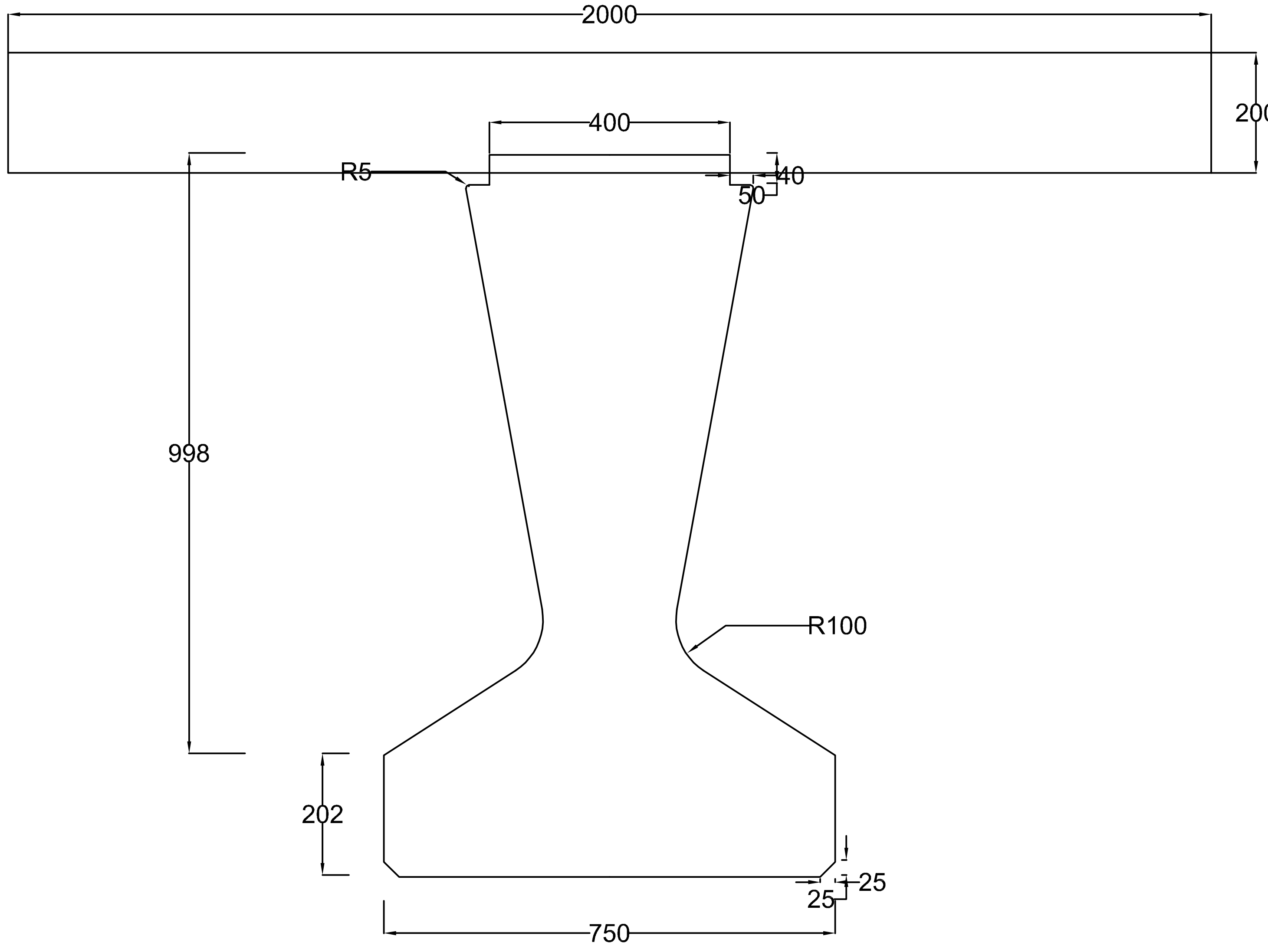


MEDIAN OF THE ROAD

CLIENT :	NEW BRIDGE FOR PERADENIYA	CONSULTANTS AND ARCHITECT : GROUP - D1	TITLE : MEDIAN OF THE ROAD		
Dr.(Ms) D. de S.UDAKARA			DRAWN BY : PATHIRANA A.P.U.M.	DESIGN BY : SOMASEKARA M.H.Y.S.	DATE : 05/06/2020
CE 402 GROUP D1			ALL DIMENSIONS ARE IN 'mm'	SIGNATURE :	SCALE : 1:5
					DRAWING NO. :



BEAM LAYOUT WITH SLAB(EDGE)



BEAM LAYOUT WITH SLAB(MIDDLE)

CLIENT :

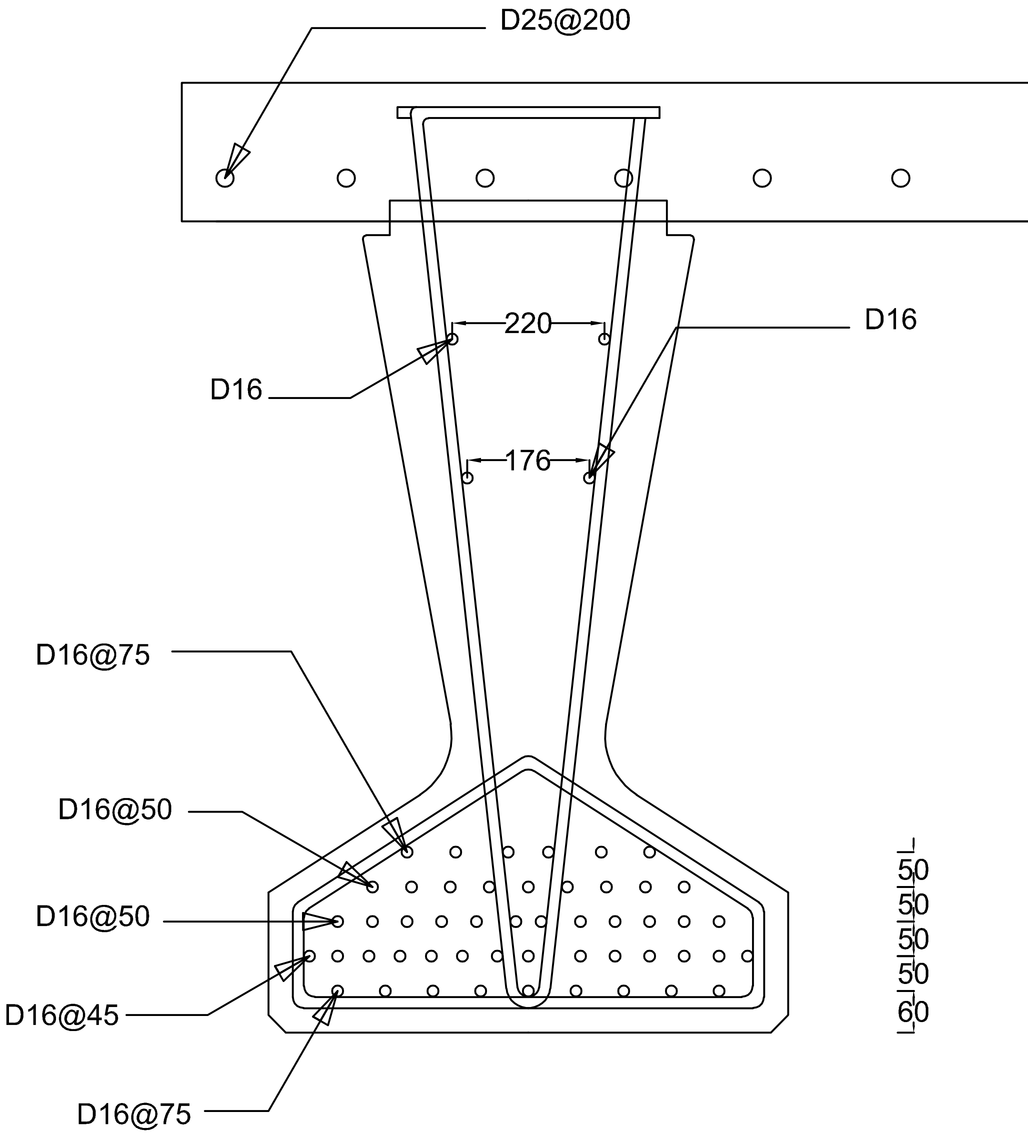
PANEL D
CE 402

NEW BRIDGE FOR PERADENIYA

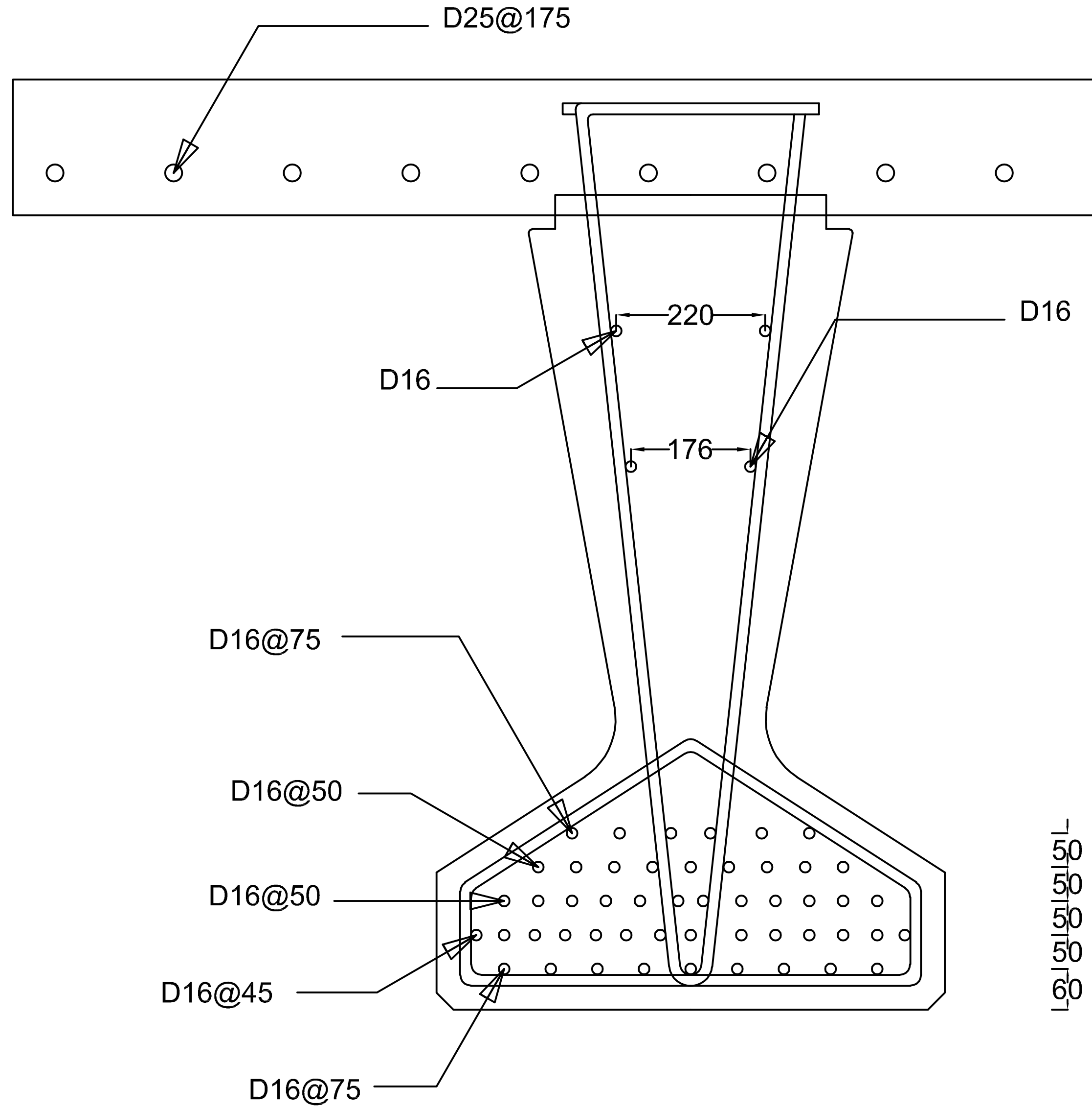
CONSULTANTS AND ARCHITECT :
GROUP - D1

TITLE : BEAM LAYOUT

DRAWN BY : PATHIRANA A.P.U.M.	DESIGN BY : SENANAYAKE S.M.A.E.	DATE : 05/06/2020
		SCALE : 1:10
ALL DIMENSIONS ARE IN 'mm'	SIGNATURE :	DRAWING NO. :



BEAM REINFORCEMENT WITH SLAB(EDGE)



BEAM REINFORCEMENT WITH SLAB(MIDDLE)

CLIENT :

PANEL D
CE 402

NEW BRIDGE FOR PERADENIYA

CONSULTANTS AND ARCHITECT :
GROUP - D1

TITLE : BEAM REINFORCEMENT LAYOUT

DRAWN BY :
PATHIRANA A.P.U.M.

DESIGN BY :
SENANAYAKE S.M.A.E.

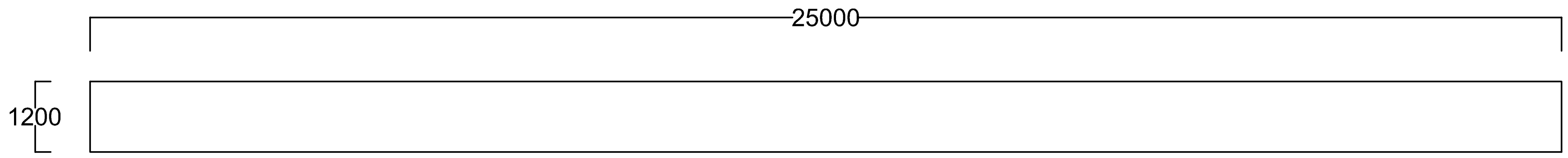
DATE :
05/06/2020

SCALE :
1:10

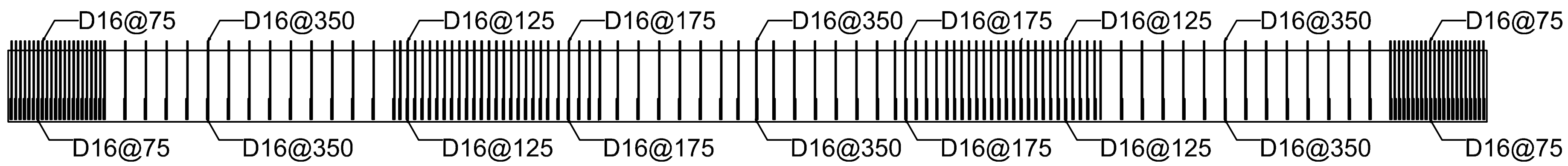
ALL DIMENSIONS ARE IN 'mm'

SIGNATURE :

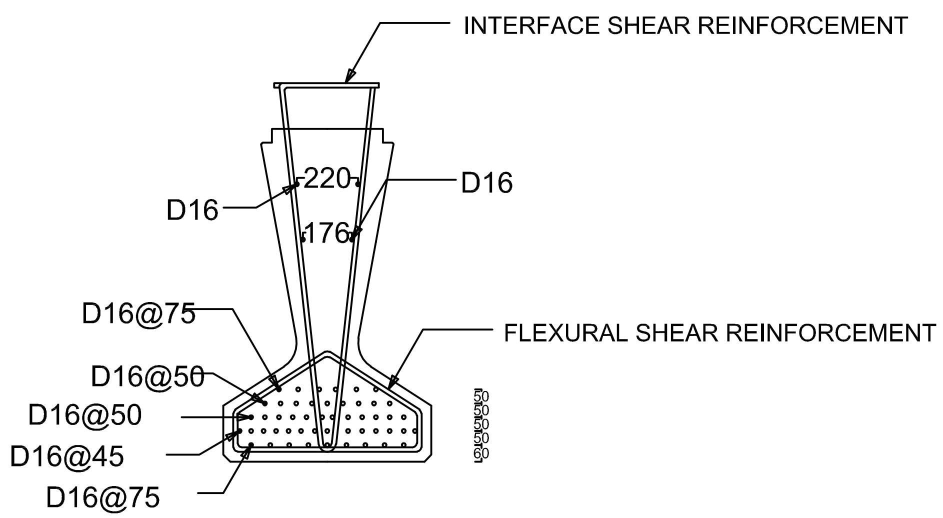
DRAWING NO. :



DIMENSIONS OF BEAM (SCALE 1:100)



SHEAR REINFORCEMENT OF BEAM (SCALE 1:100)



BEAM REINFORCEMENT (SCALE 1:25)

CLIENT :

PANEL D
CE 402

NEW BRIDGE
FOR
PERADENIYA

CONSULTANTS AND ARCHITECT :
GROUP - D1

TITLE : SHEAR REINFORCEMENTS OF THE BEAM

DRAWN BY :
PATHIRANA A.P.U.M.

DESIGN BY :
SENANAYAKE S.M.A.E.

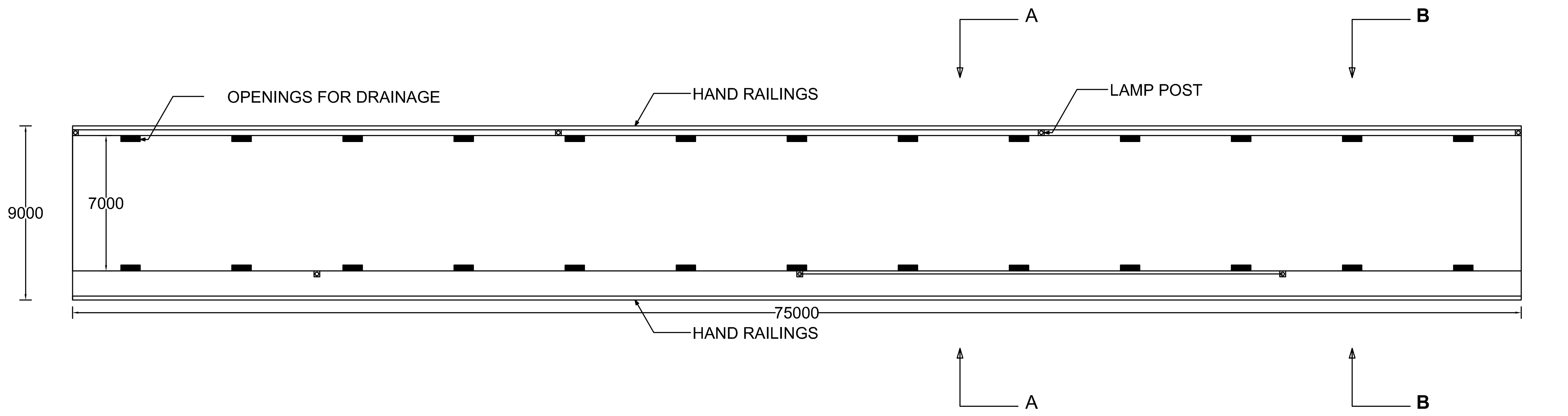
DATE :
05/06/2020

SCALE :
1:100,1:25

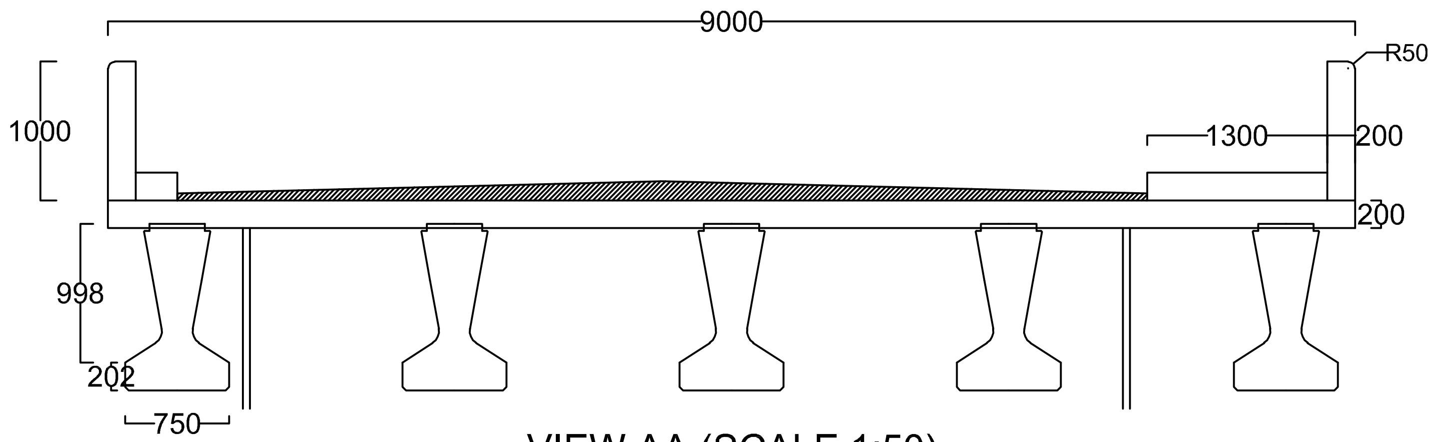
ALL DIMENSIONS ARE IN 'mm'

SIGNATURE :

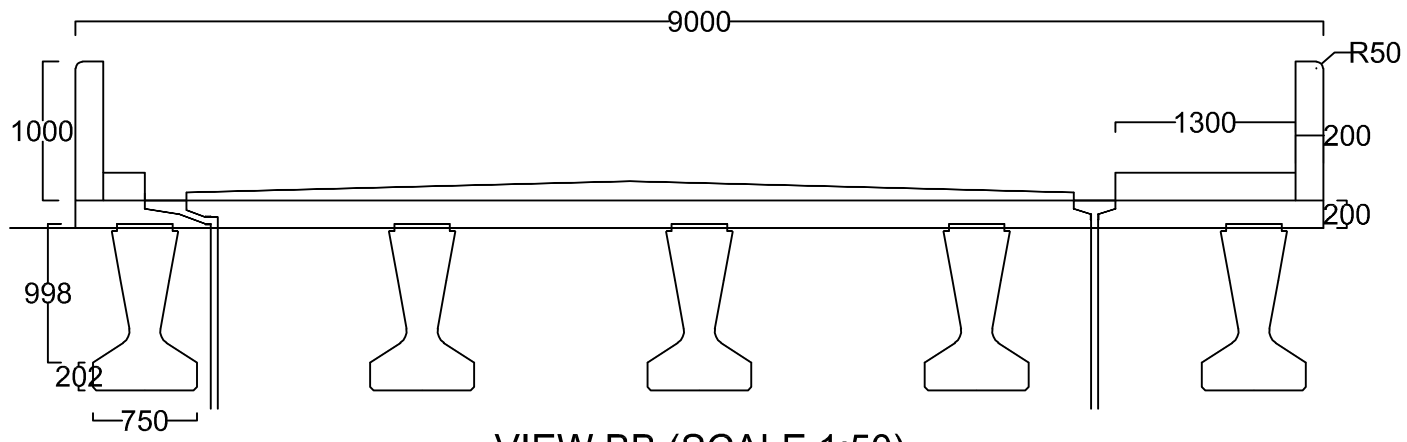
DRAWING NO. :



BRIDGE DECK PLAN VIEW (SCALE 1:200)

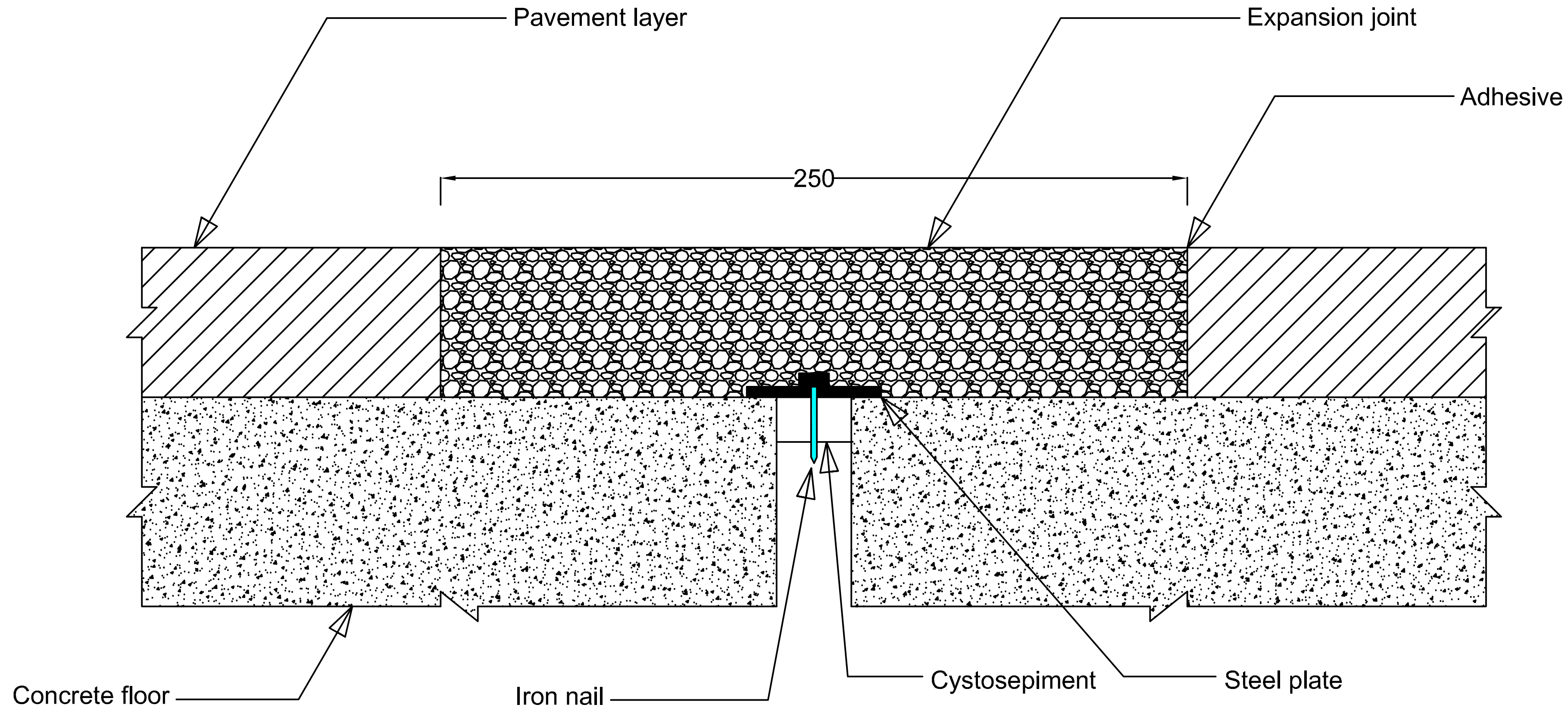


VIEW AA (SCALE 1:50)



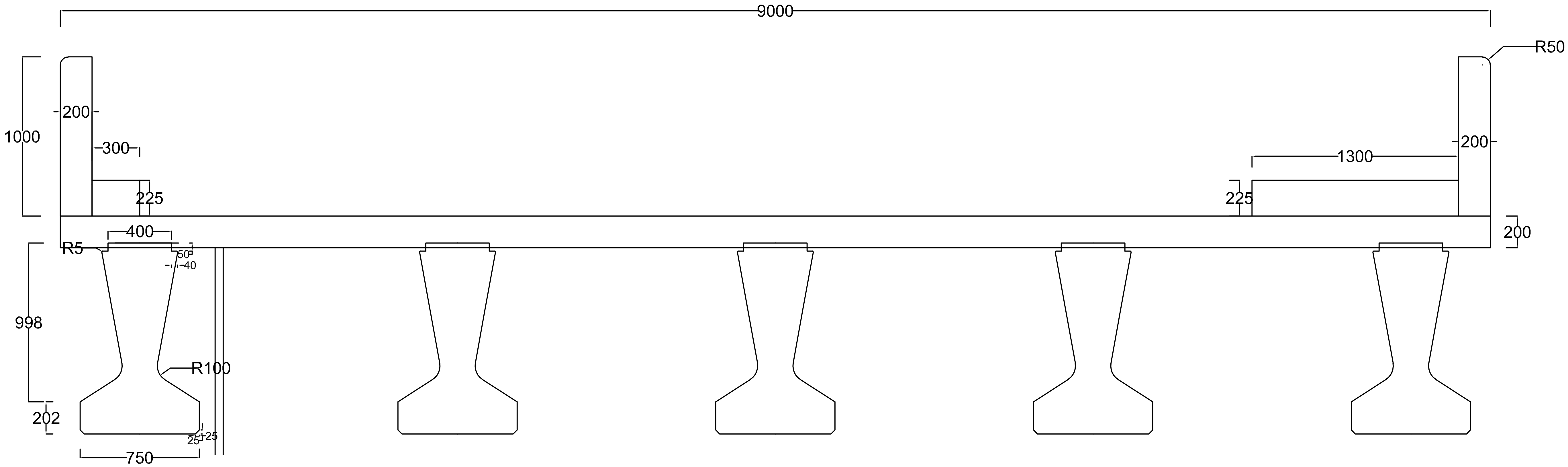
VIEW BB (SCALE 1:50)

CLIENT : PANEL D CE 402	NEW BRIDGE FOR PERADENIYA	CONSULTANTS AND ARCHITECT : GROUP - D1	TITLE : BRIDGE DECK PLAN VIEW		
			DRAWN BY : PATHIRANA A.P.U.M.	DESIGN BY : SENANAYAKE S.M.A.E.	DATE : 05/06/2020 SCALE : 1:200,1:50
			ALL DIMENSIONS ARE IN 'mm'	SIGNATURE :	DRAWING NO. :



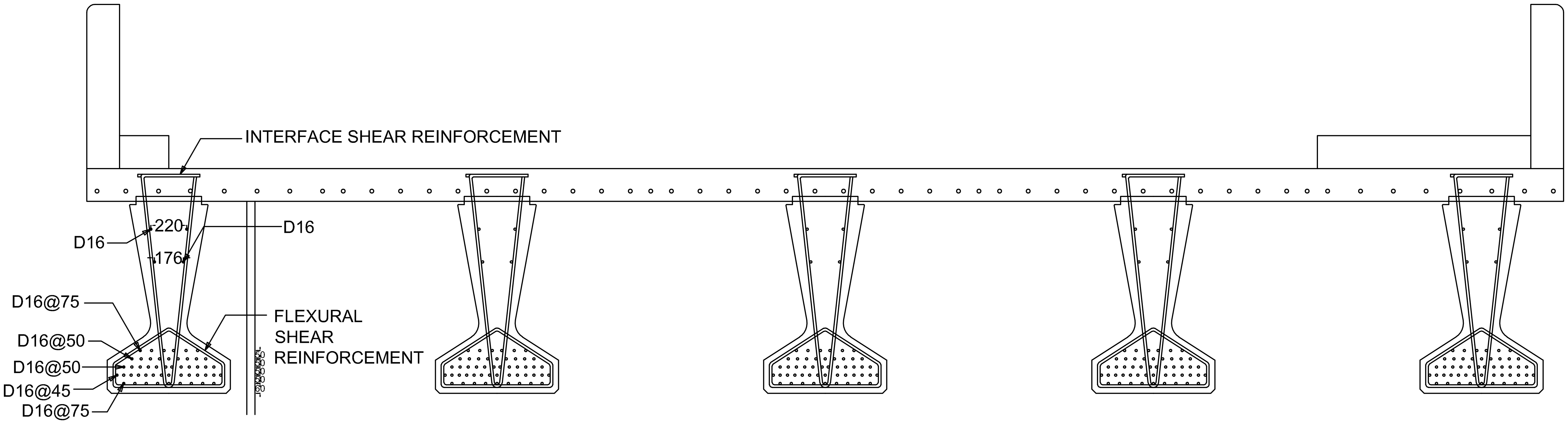
EXPANSION JOINT

CLIENT : PANEL D CE 402	NEW BRIDGE FOR PERADENIYA	CONSULTANTS AND ARCHITECT : GROUP - D1	TITLE : EXPANSION JOINT		
			DRAWN BY : PATHIRANA A.P.U.M.	DESIGN BY : SENANAYAKE S.M.A.E.	DATE : 05/06/2020
					SCALE : 1:2
			ALL DIMENSIONS ARE IN 'mm'	SIGNATURE :	DRAWING NO. :



DIMENSIONS OF BRIDGE DECK AND BEAMS

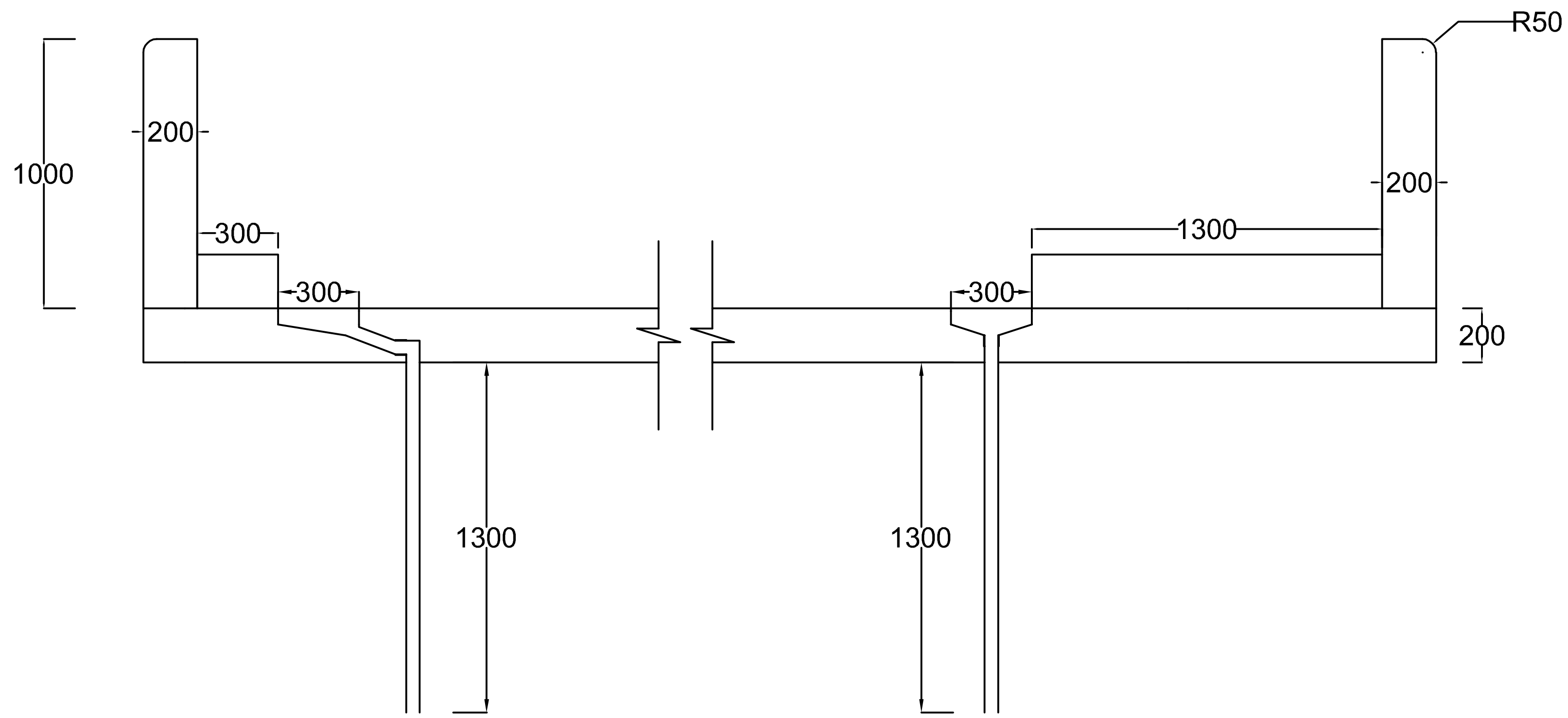
CLIENT : PANEL D CE 402	NEW BRIDGE FOR PERADENIYA	CONSULTANTS AND ARCHITECT : GROUP - D1	TITLE : DIMENSIONS OF BRIDGE DECK AND BEAMS		
			DRAWN BY : PATHIRANA A.P.U.M.	DESIGN BY : SENANAYAKE S.M.A.E.	DATE : 05/06/2020 SCALE : 1:25
			ALL DIMENSIONS ARE IN 'mm'	SIGNATURE :	DRAWING NO. :



REINFORCEMENTS OF BRIDGE DECK AND BEAMS

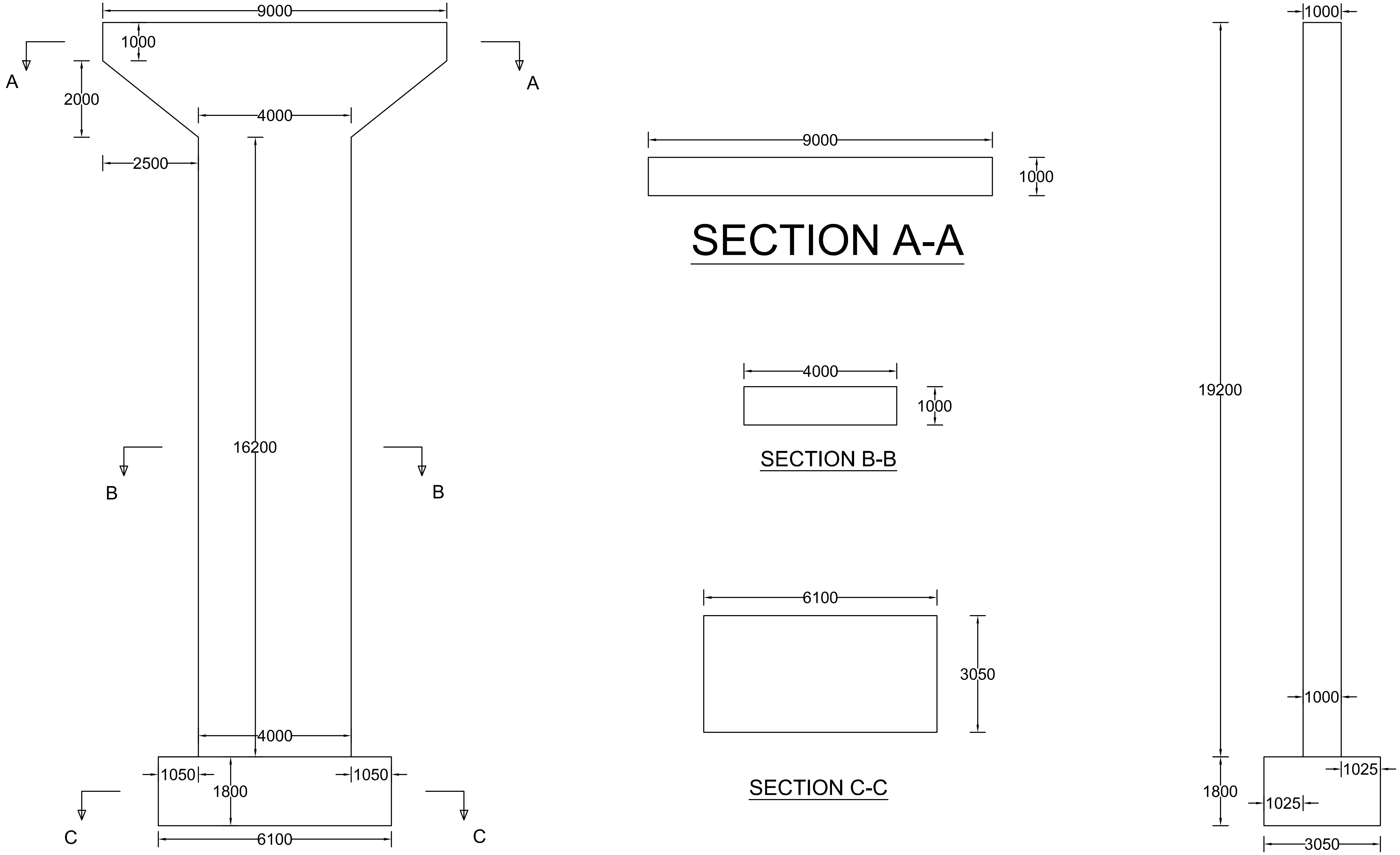
CLIENT : PANEL D CE 402	NEW BRIDGE FOR PERADENIYA	CONSULTANTS AND ARCHITECT : GROUP - D1	TITLE : REINFORCEMENTS OF BRIDGE DECK AND BEAMS DRAWN BY : PATHIRANA A.P.U.M. DESIGN BY : SENANAYAKE S.M.A.E. DATE : 05/06/2020 SCALE : 1:25 ALL DIMENSIONS ARE IN 'mm' SIGNATURE : DRAWING NO. :

OPENINGS FOR THE DRAINAGE



DRAINAGE OF THE BRIDGE

CLIENT : PANEL D CE 402	NEW BRIDGE FOR PERADENIYA	CONSULTANTS AND ARCHITECT : GROUP - D1	TITLE : DRAINAGE FOR THE BRIDGE		
			DRAWN BY : PATHIRANA A.P.U.M.	DESIGN BY : SENANAYAKE S.M.A.E.	DATE : 05/06/2020 SCALE : 1:25
			ALL DIMENSIONS ARE IN 'mm'	SIGNATURE :	DRAWING NO. :



PIER DIMENSIONS

PIER DIMENSIONS

CLIENT :

PANEL D
CE 402

NEW BRIDGE FOR PERADENIYA

CONSULTANTS AND ARCHITECT :
GROUP - D1

TITLE : PIER DIMENSIONS

DRAWN BY :
PATHIRANA A.P.U.M.

DESIGN BY :
DINELKSA K.H.S.

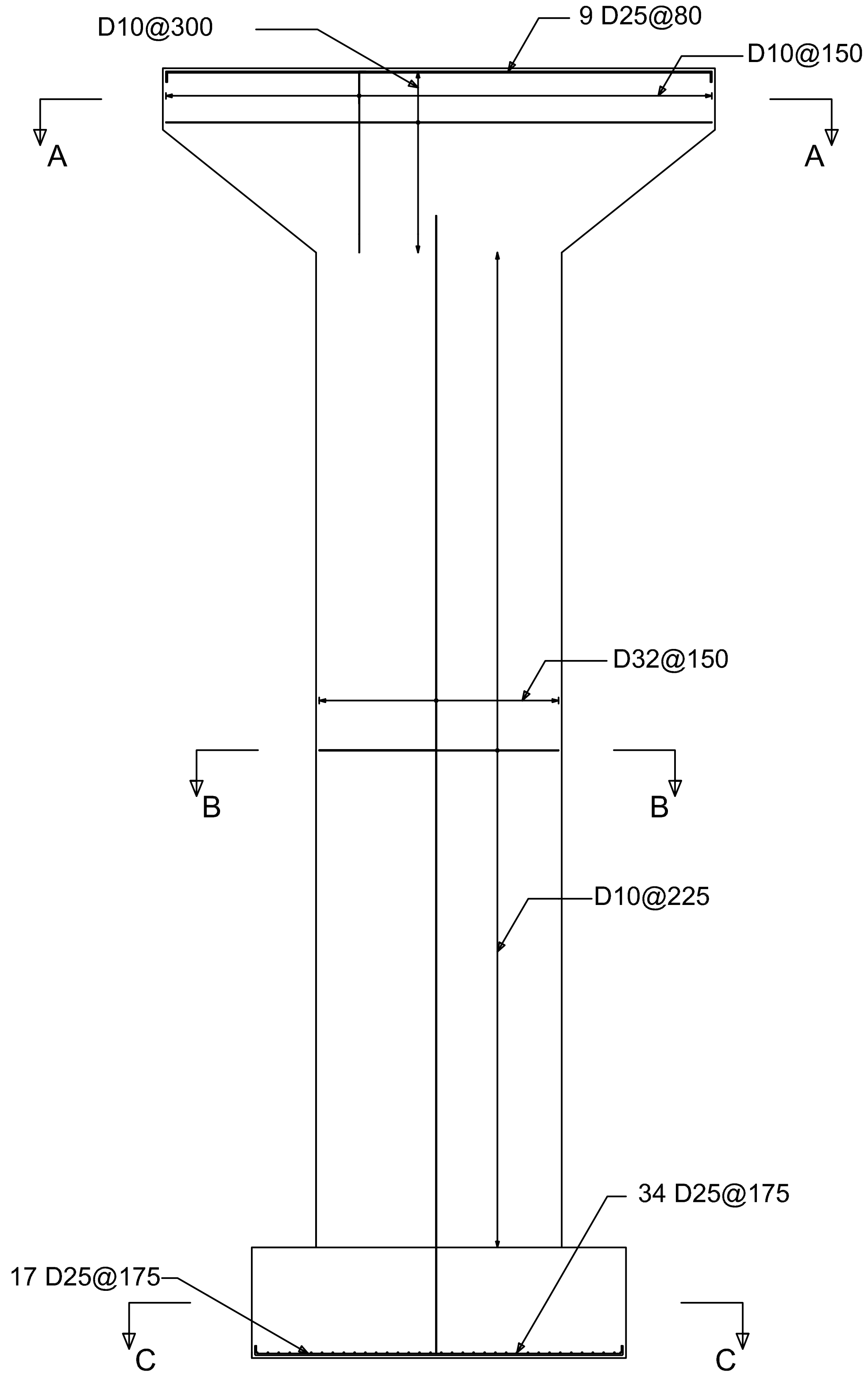
DATE :
05/06/2020

SCALE :
1:100

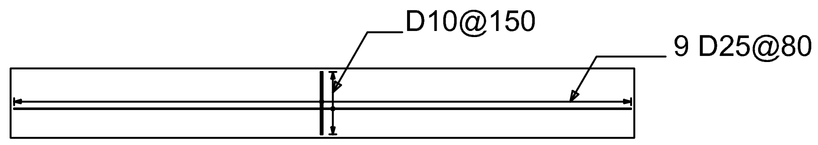
ALL DIMENSIONS ARE IN 'mm'

SIGNATURE :

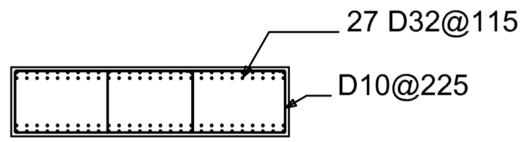
DRAWING NO. :



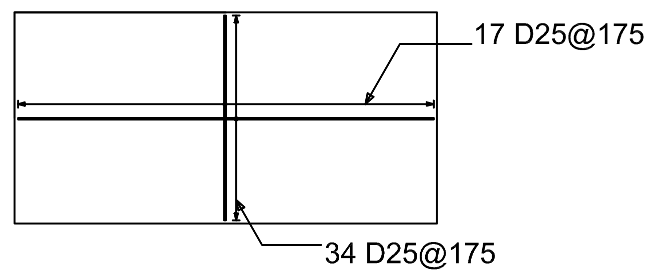
PIER REINFORCEMENT DETAILS



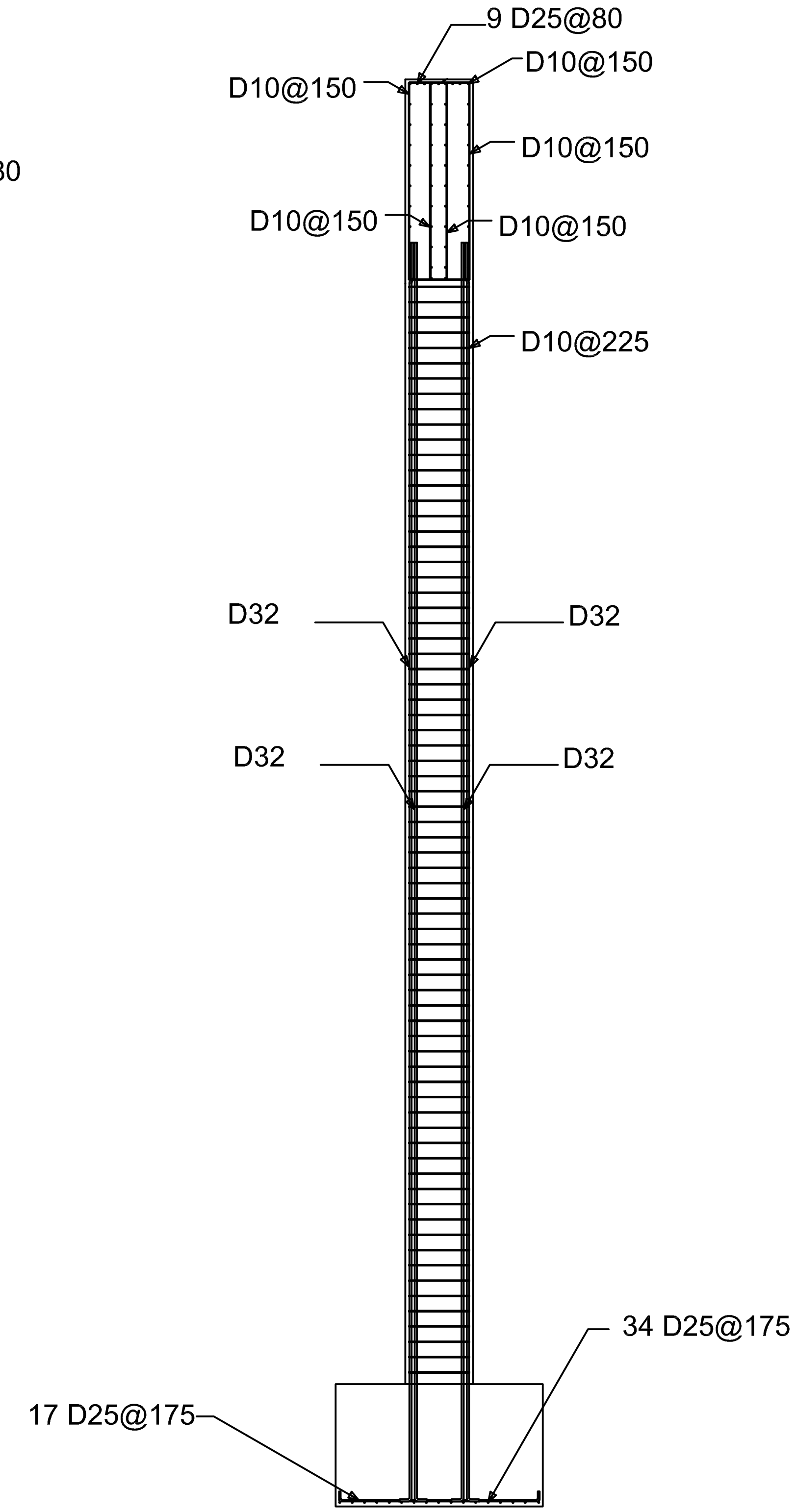
SECTION A-A



SECTION B-B

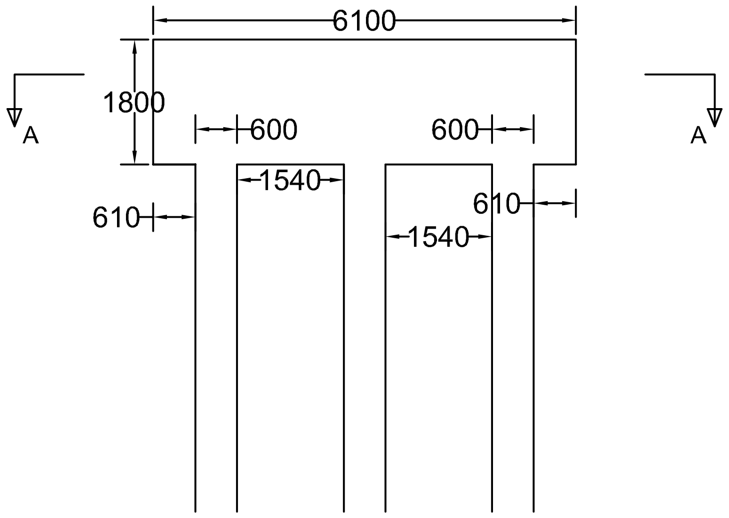


SECTION C-C

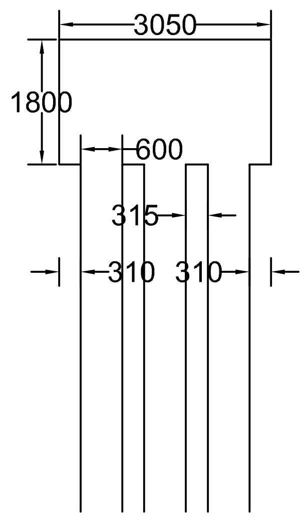


PIER REINFORCEMENT DETAILS

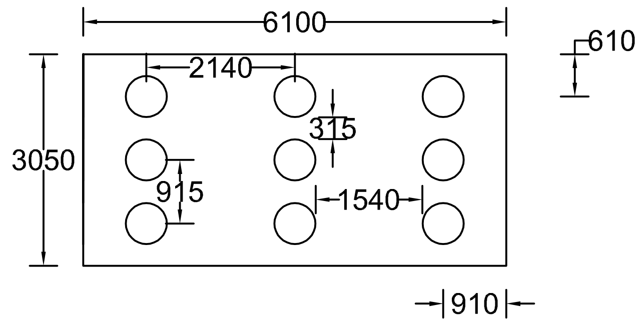
CLIENT : PANEL D CE 402	NEW BRIDGE FOR PERADENIYA	CONSULTANTS AND ARCHITECT : GROUP - D1	TITLE : PIER REINFORCEMENTS		
			DRAWN BY : PATHIRANA A.P.U.M.	DESIGN BY : DINELKSA K.H.S.	DATE : 05/06/2020
					SCALE : 1:100
			ALL DIMENSIONS ARE IN 'mm'	SIGNATURE :	DRAWING NO. :



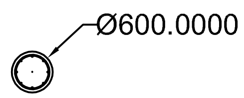
PILE DIMENSIONS



PILE DIMENSIONS



SECTION A-A



CLIENT :

PANEL D
CE 402

NEW BRIDGE FOR PERADENIYA

CONSULTANTS AND ARCHITECT :
GROUP - D1

TITLE : PILE DIMENSIONS

DRAWN BY :
PATHIRANA A.P.U.M.

DESIGN BY :
DINELKSA K.H.S.

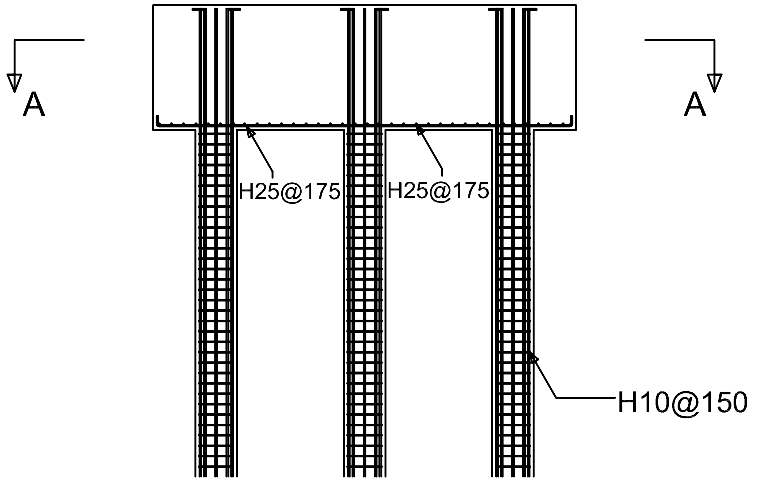
DATE :
05/06/2020

SCALE :
1:100

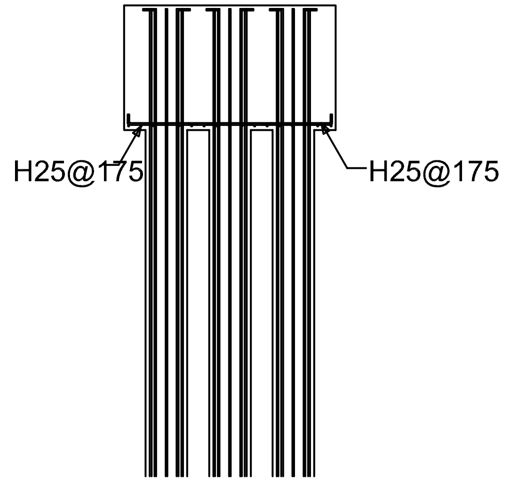
ALL DIMENSIONS ARE IN 'mm'

SIGNATURE :

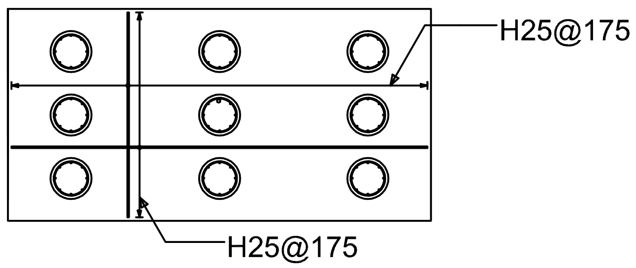
DRAWING NO. :



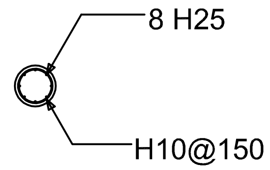
PILE REINFORCEMENT DETAILS



PILE REINFORCEMENT DETAILS



SECTION A-A



REINFORCEMENTS OF A PILE

CLIENT :

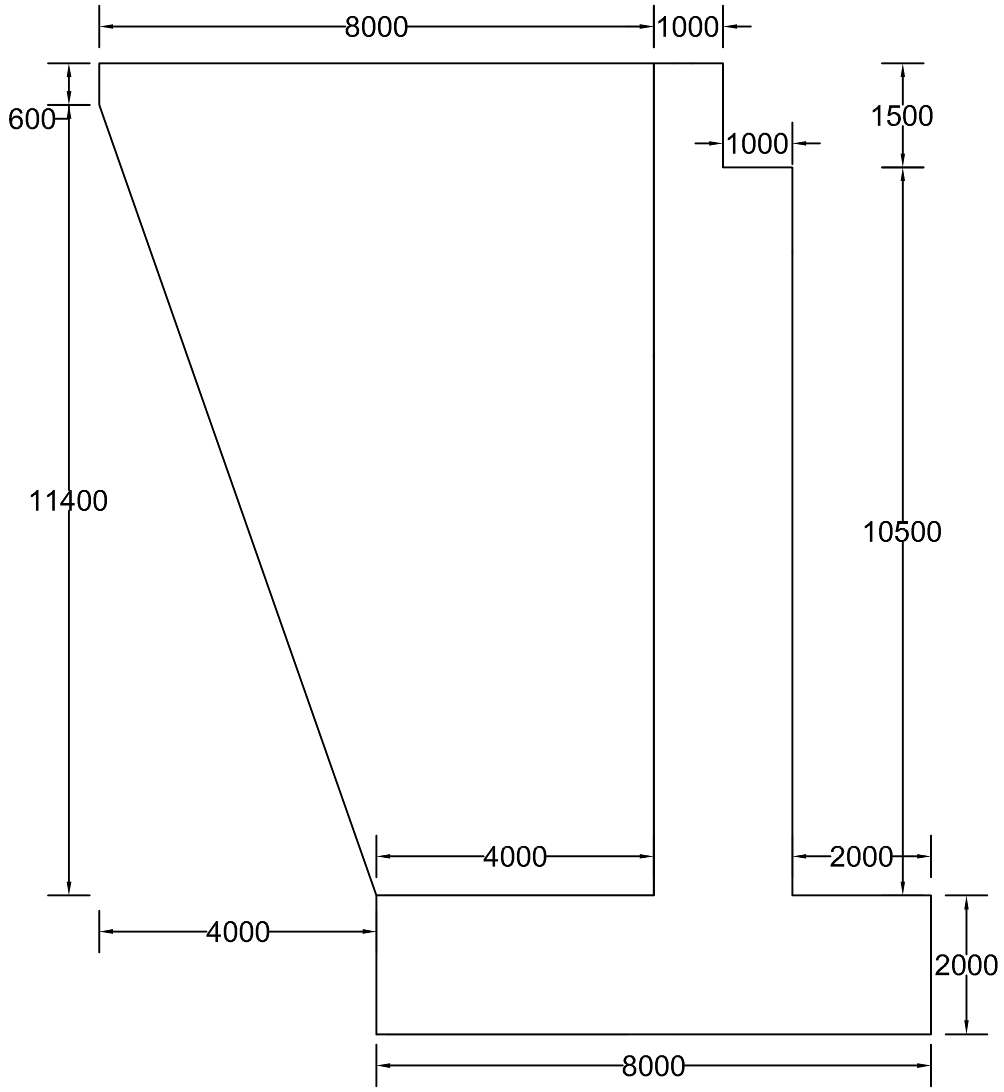
PANEL D
CE 402

NEW BRIDGE FOR PERADENIYA

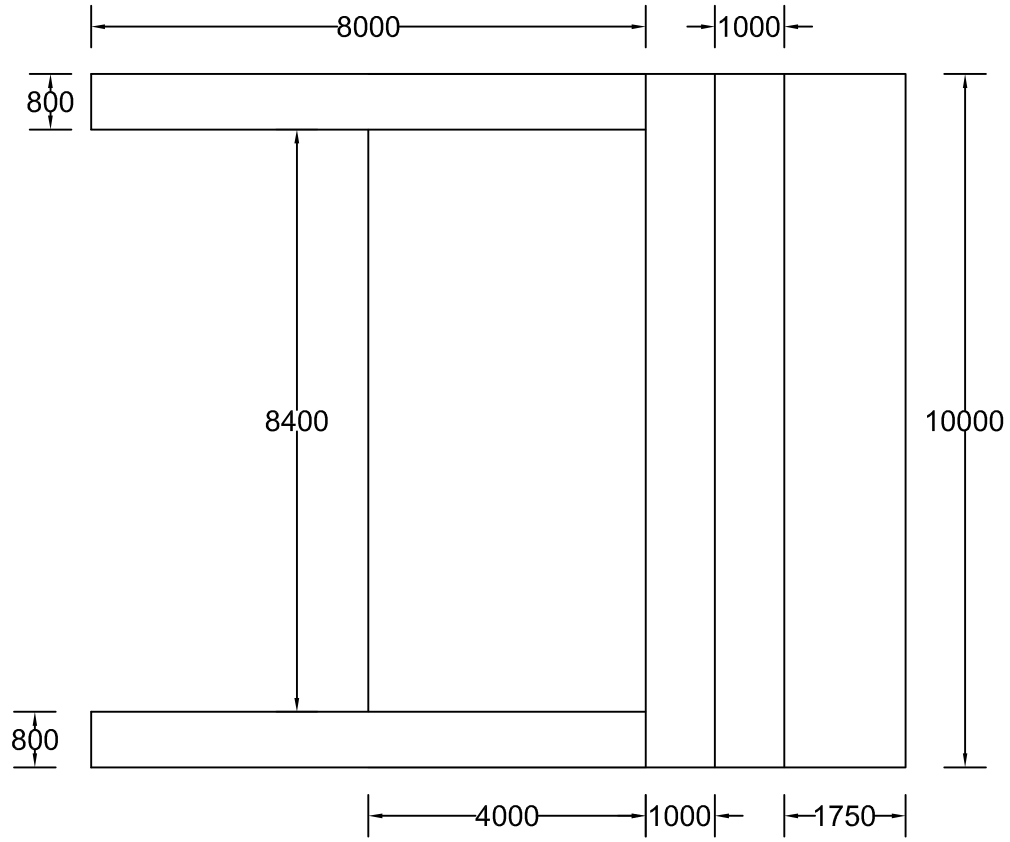
CONSULTANTS AND ARCHITECT :
GROUP - D1

TITLE : PILE REINFORCEMENTS

DRAWN BY :	DESIGN BY :	DATE :
PATHIRANA A.P.U.M.	DINELKSA K.H.S.	05/06/2020
		SCALE :
		1:100
ALL DIMENSIONS ARE IN 'mm'	SIGNATURE :	DRAWING NO. :



RIGHT ABUTMENT DIMENSIONS



RIGHT ABUTMENT TOP VIEW

CLIENT :

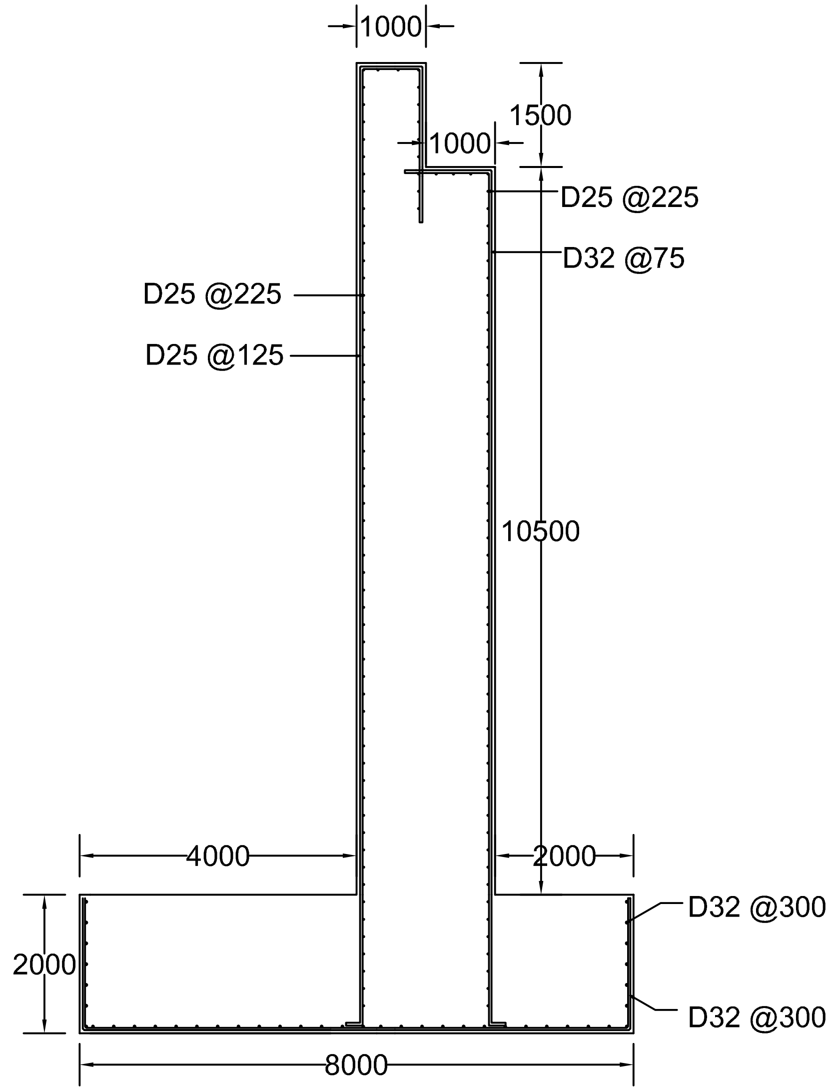
PANEL D
CE 402

NEW BRIDGE FOR PERADENIYA

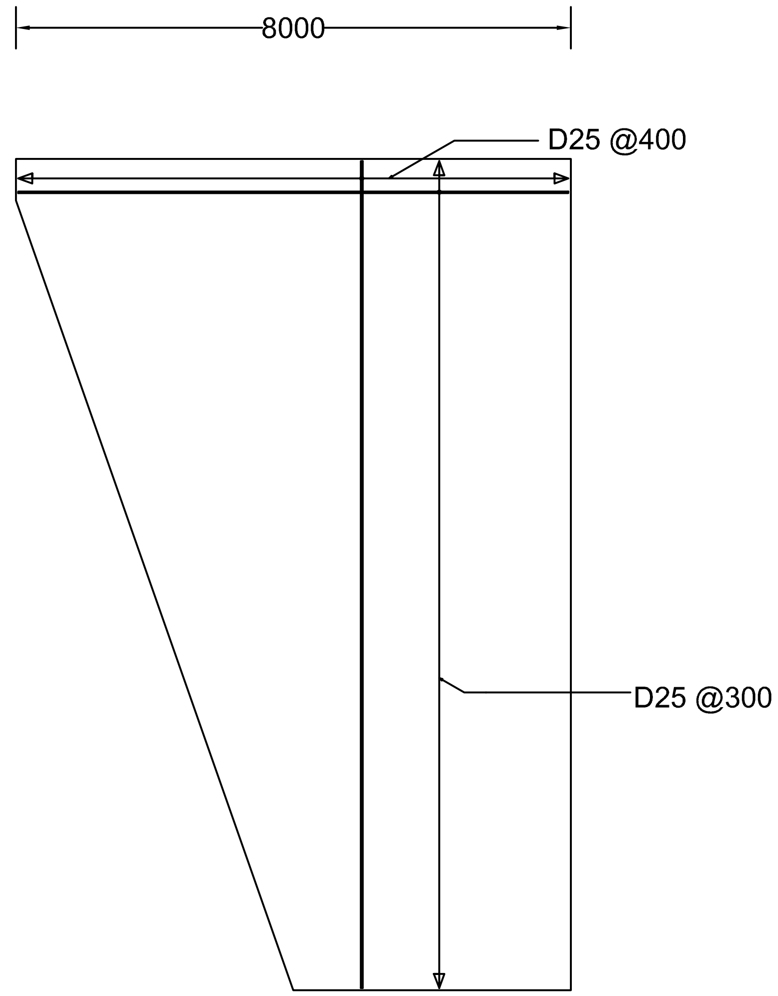
CONSULTANTS AND ARCHITECT :
GROUP - D1

TITLE : RIGHT ABUTMENT DIMENSIONS

DRAWN BY :	DESIGN BY :	DATE :
PATHIRANA A.P.U.M.	PRIYASHAN H.M.M.	05/06/2020
		SCALE : 1:100
ALL DIMENSIONS ARE IN 'mm'	SIGNATURE :	DRAWING NO. :



RIGHT ABUTMENT REINFORCEMENT DETAILS



WING WALL(RIGHT ABUTMENT) REINFORCEMENT DETAILS

CLIENT :

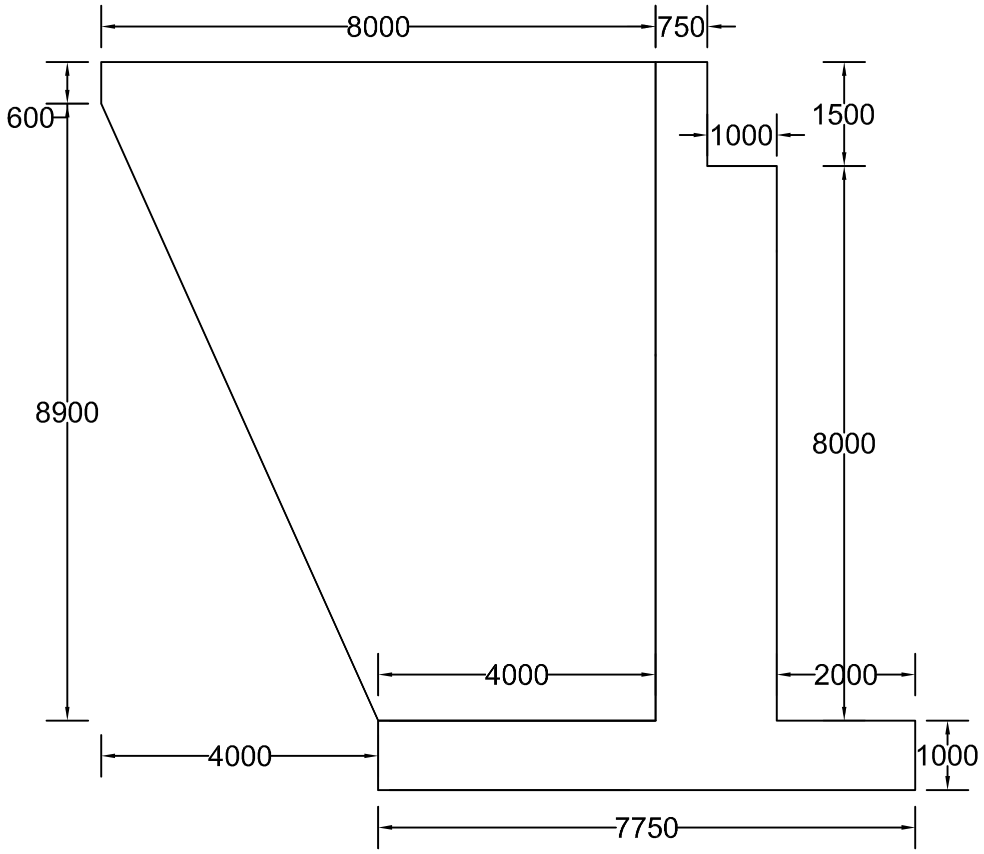
PANEL D
CE 402

NEW BRIDGE FOR PERADENIYA

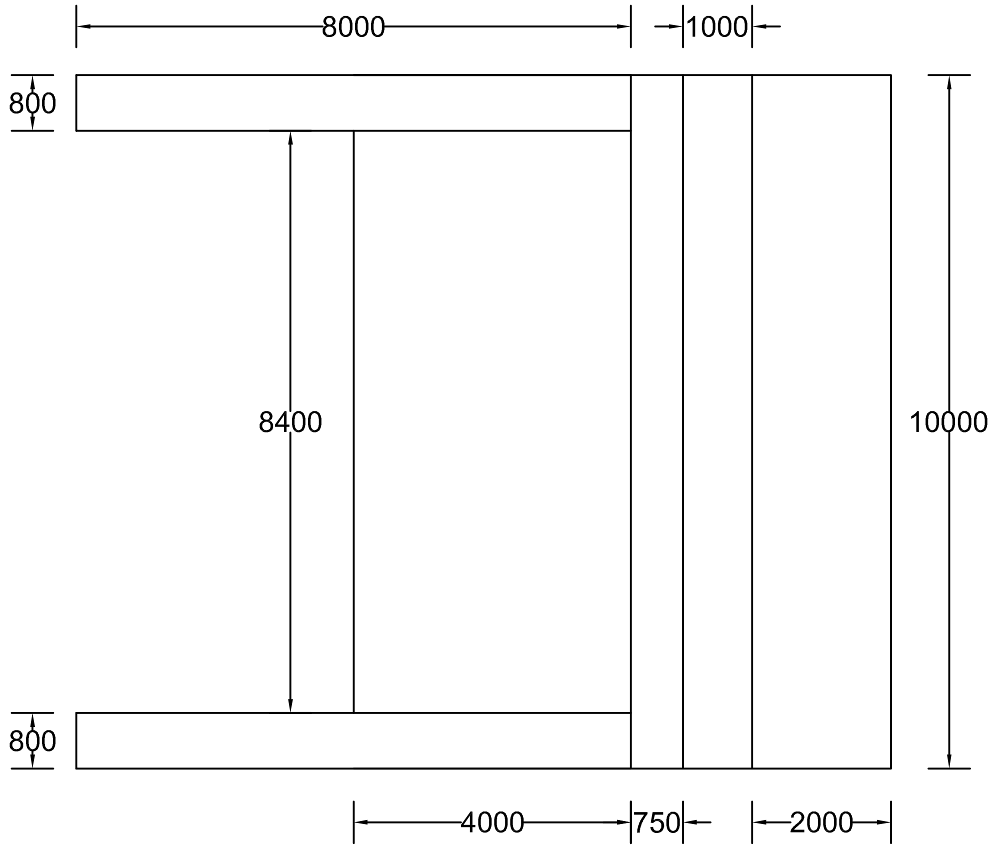
CONSULTANTS AND ARCHITECT :
GROUP - D1

TITLE : RIGHT ABUTMENT REINFORCEMENTS

DRAWN BY : PATHIRANA A.P.U.M.	DESIGN BY : PRIYASHAN H.M.M.	DATE : 05/06/2020
		SCALE : 1:100
ALL DIMENSIONS ARE IN 'mm'	SIGNATURE :	DRAWING NO. :

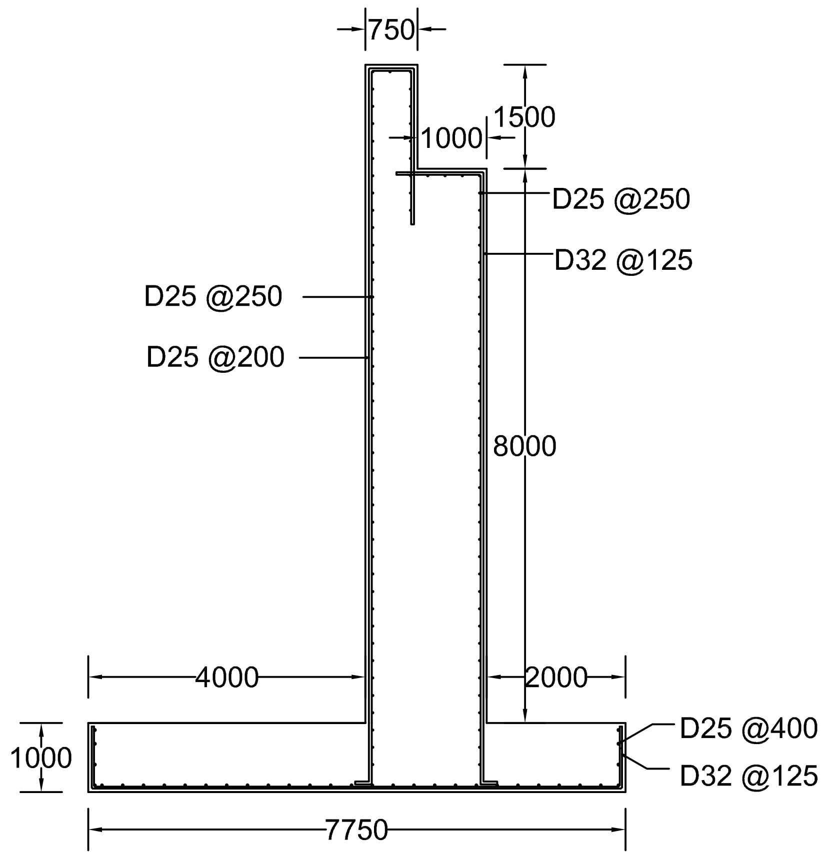


LEFT ABUTMENT DIMENSIONS

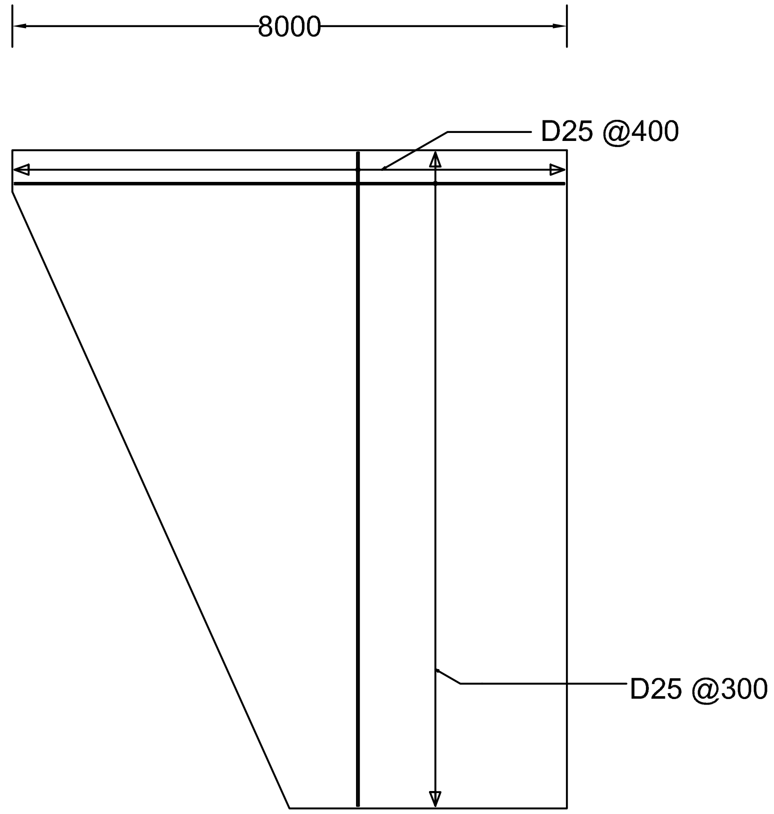


LEFT ABUTMENT TOP VIEW

CLIENT : PANEL D CE 402	NEW BRIDGE FOR PERADENIYA	CONSULTANTS AND ARCHITECT : GROUP - D1	TITLE : LEFT ABUTMENT DIMENSIONS		
			DRAWN BY : PATHIRANA A.P.U.M.	DESIGN BY : PRIYASHAN H.M.M.	DATE : 05/06/2020 SCALE : 1:100
			ALL DIMENSIONS ARE IN 'mm'	SIGNATURE :	DRAWING NO. :



LEFT ABUTMENT REINFORCEMENT DETAILS



WING WALL(LEFT ABUTMENT) REINFORCEMENT DETAILS

CLIENT :

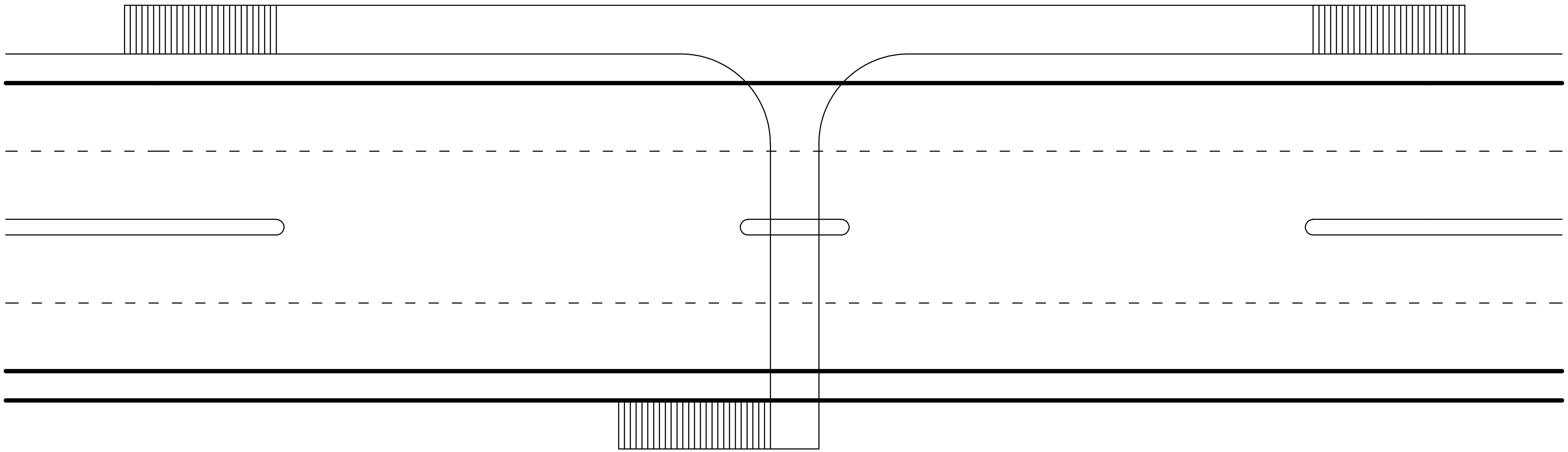
PANEL D
CE 402

NEW BRIDGE
FOR
PERADENIYA

CONSULTANTS AND ARCHITECT :
GROUP - D1

TITLE : LEFT ABUTMENT REINFORCEMENTS

DRAWN BY : PATHIRANA A.P.U.M.	DESIGN BY : PRIYASHAN H.M.M.	DATE : 05/06/2020
		SCALE : 1:100
ALL DIMENSIONS ARE IN 'mm'	SIGNATURE :	DRAWING NO. :



UNDERPASS LAYOUT

CLIENT :

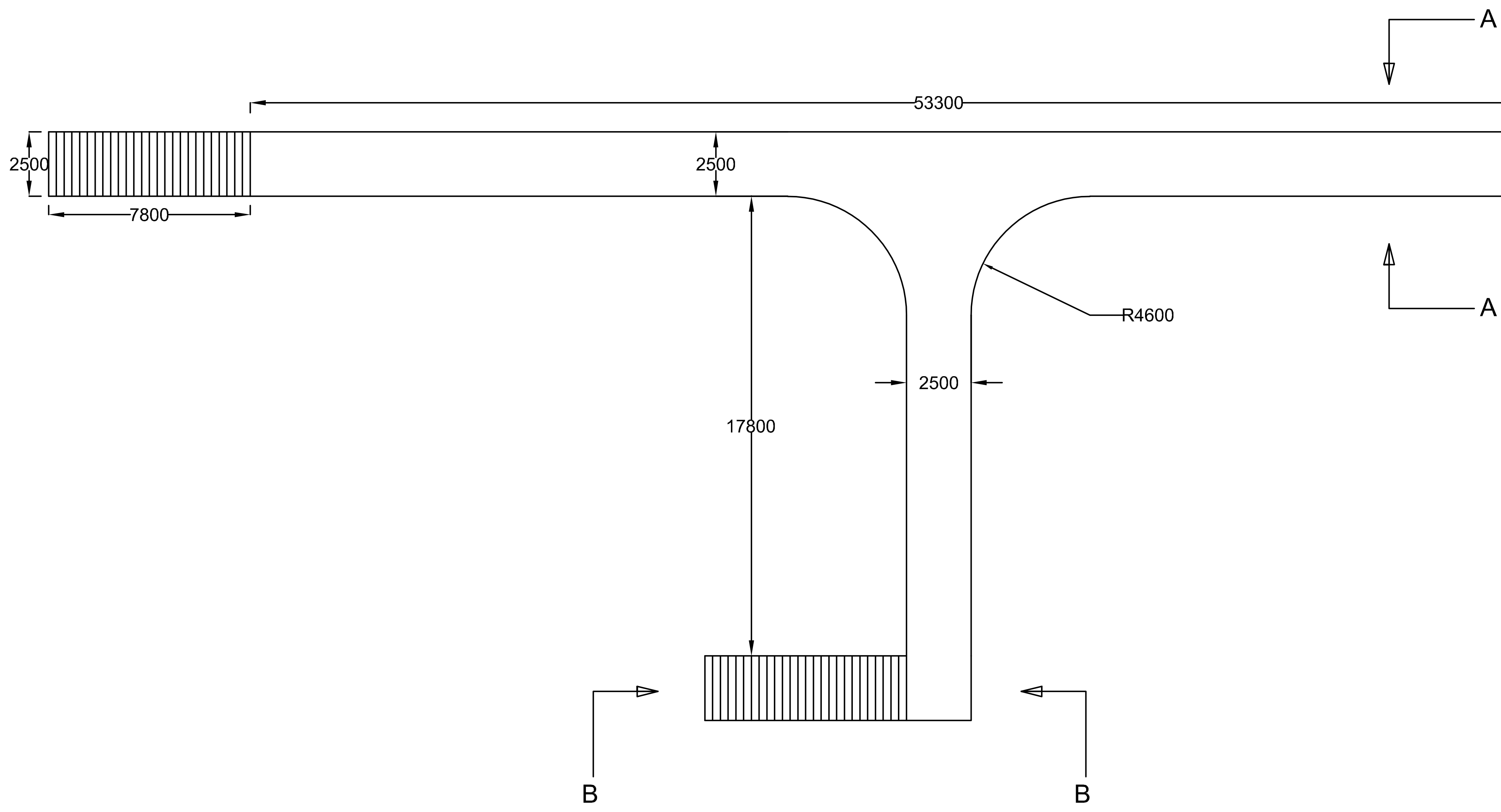
PANEL D
CE 402

NEW BRIDGE
FOR
PERADENIYA

CONSULTANTS AND ARCHITECT :
GROUP - D1

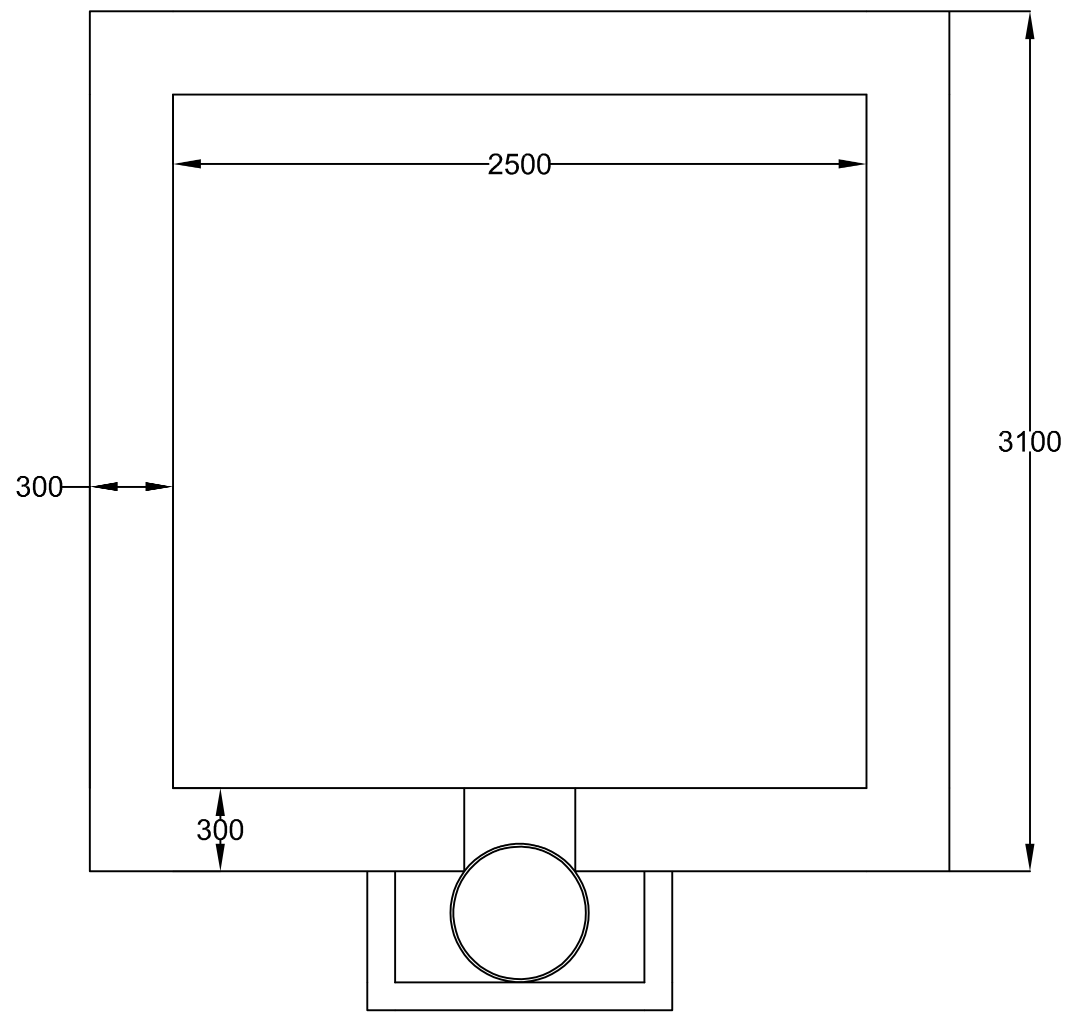
TITLE : UNDERPASS LAYOUT

DRAWN BY : PATHIRANA A.P.U.M.	DESIGN BY : KALABAN P.	DATE : 05/06/2020
		SCALE : 1:200
ALL DIMENSIONS ARE IN 'mm'	SIGNATURE :	DRAWING NO. :



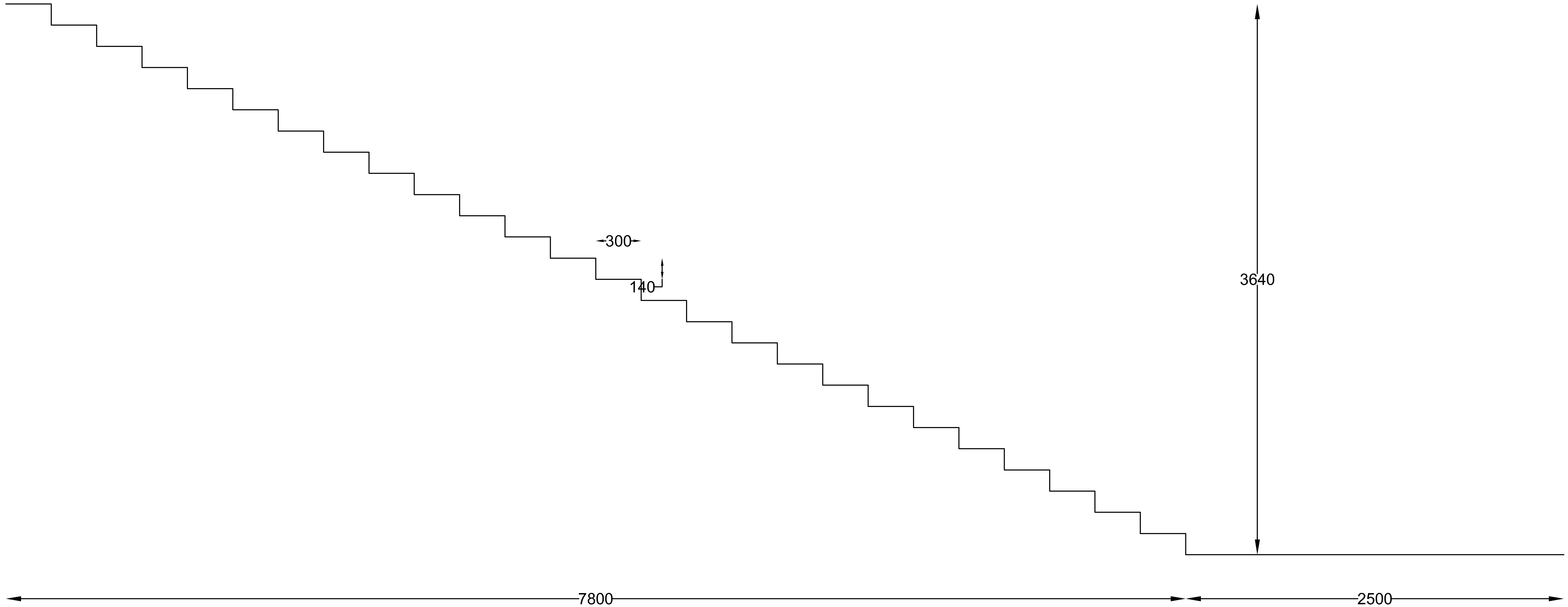
UNDERPASS DIMENSIONS

CLIENT : PANEL D CE 402	NEW BRIDGE FOR PERADENIYA	CONSULTANTS AND ARCHITECT : GROUP - D1	TITLE : UNDERPASS DIMENSIONS	
			DRAWN BY : PATHIRANA A.P.U.M.	DESIGN BY : KALABAN P.	DATE : 05/06/2020
			ALL DIMENSIONS ARE IN 'mm'	SIGNATURE :	SCALE : 1:200
					DRAWING NO. :



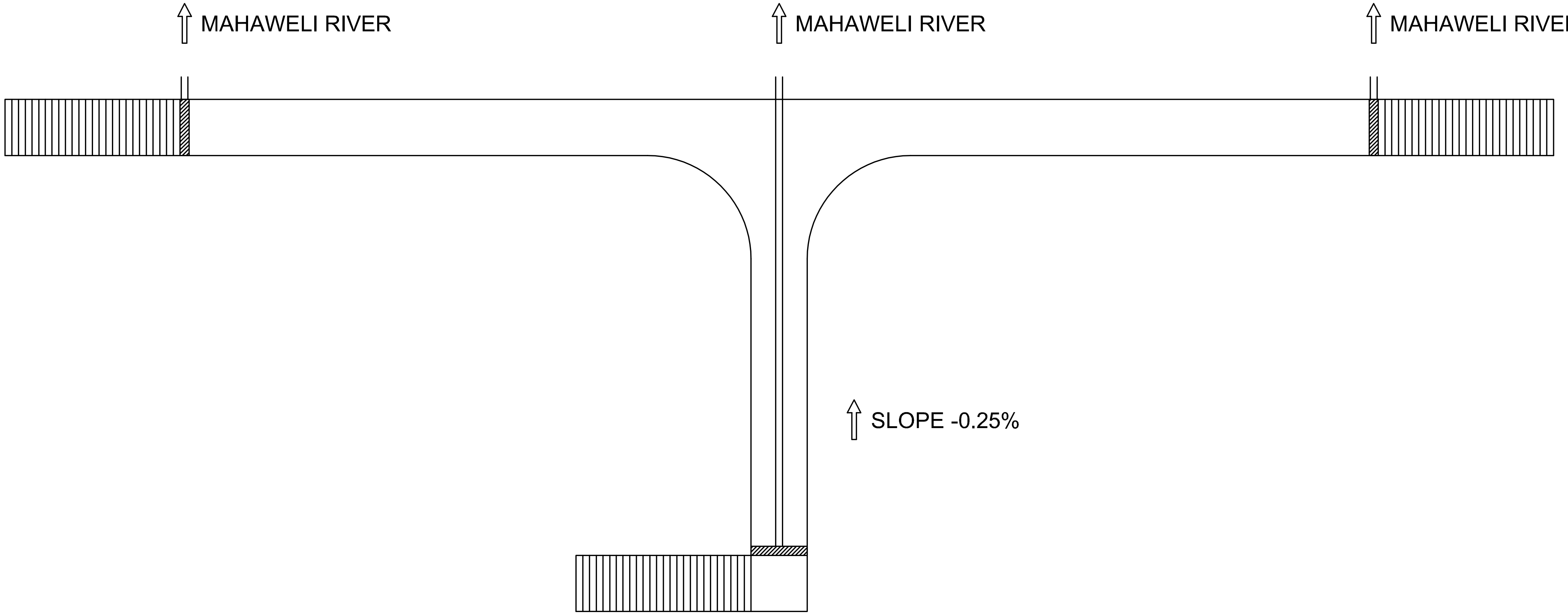
VIEW AA

CLIENT : PANEL D CE 402	NEW BRIDGE FOR PERADENIYA	CONSULTANTS AND ARCHITECT : GROUP - D1	TITLE : CROSS SECTIONAL VIEWS		
			DRAWN BY : PATHIRANA A.P.U.M.	DESIGN BY : KALABAN P.	DATE : 05/06/2020
					SCALE : 1:25
			ALL DIMENSIONS ARE IN 'mm'	SIGNATURE :	DRAWING NO. :



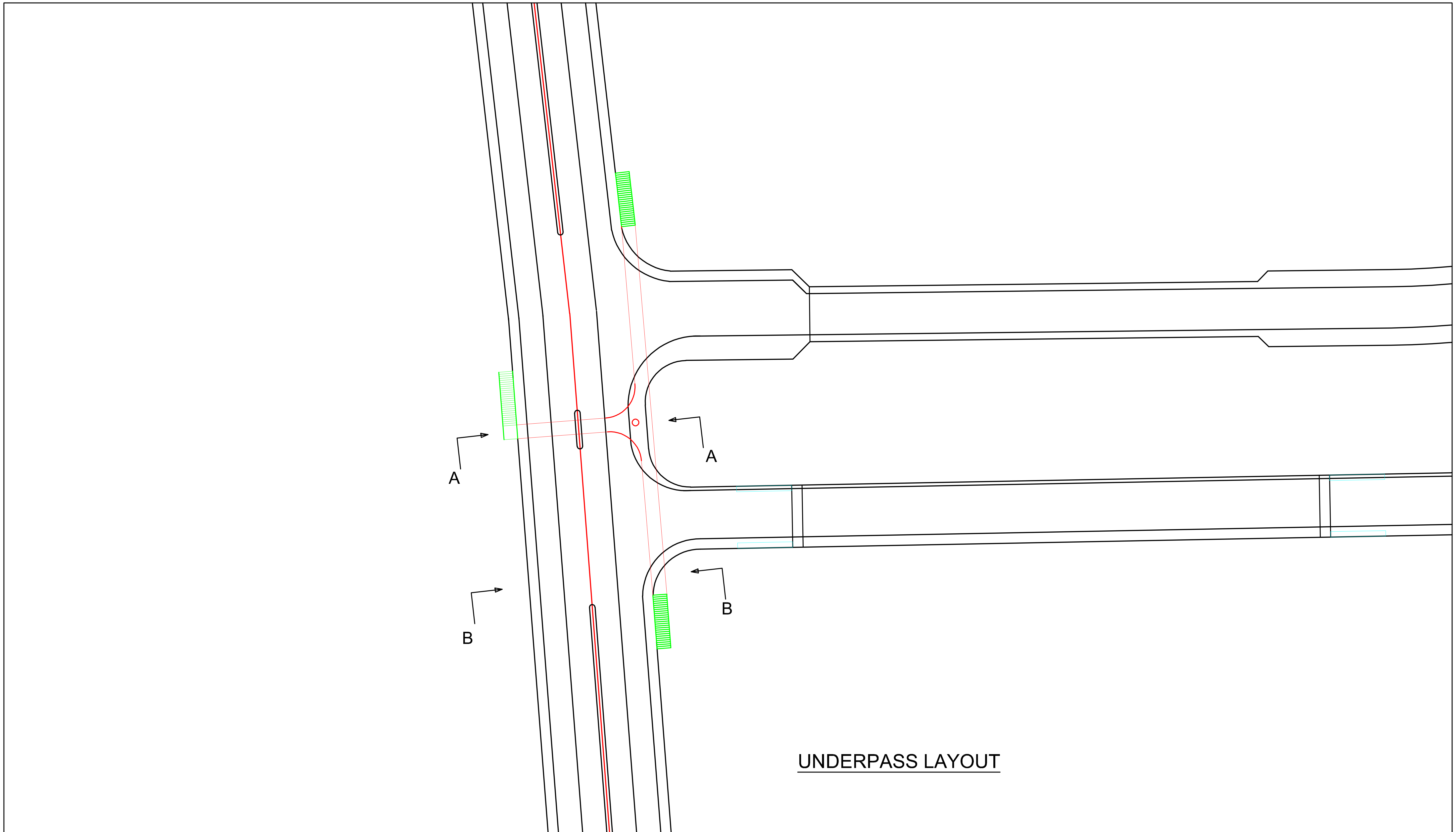
VIEW BB

CLIENT : PANEL D CE 402	NEW BRIDGE FOR PERADENIYA	CONSULTANTS AND ARCHITECT : GROUP - D1	TITLE : CROSS SECTIONAL VIEWS		
			DRAWN BY : PATHIRANA A.P.U.M.	DESIGN BY : KALABAN P.	DATE : 05/06/2020
					SCALE : 1:25
			ALL DIMENSIONS ARE IN 'mm'	SIGNATURE :	DRAWING NO. :



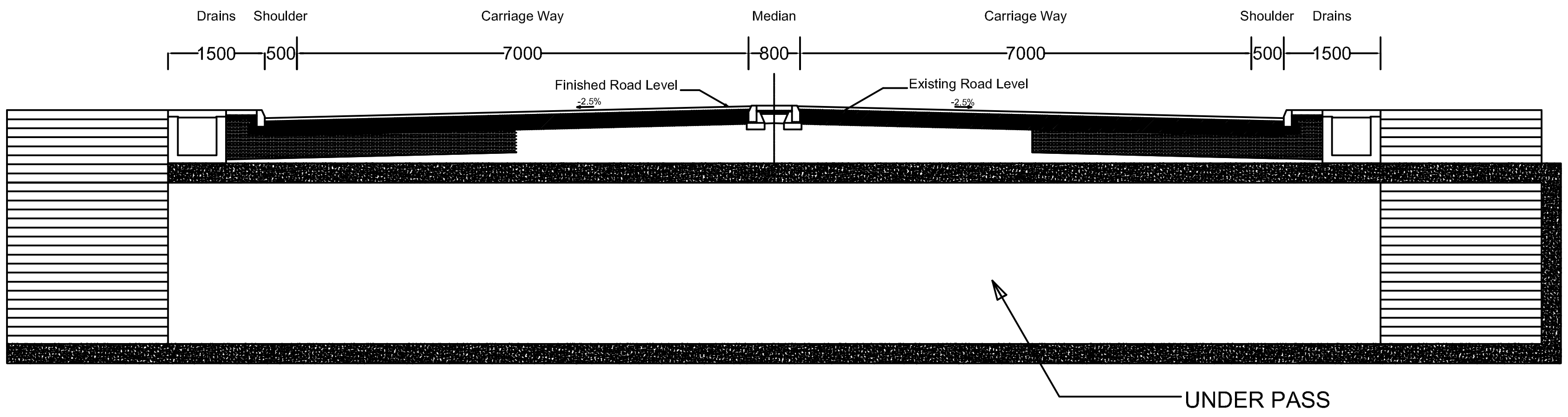
UNDERPASS DRAINAGE

CLIENT : PANEL D CE 402	NEW BRIDGE FOR PERADENIYA	CONSULTANTS AND ARCHITECT : GROUP - D1	TITLE : UNDERPASS DRAINAGE		
			DRAWN BY : PATHIRANA A.P.U.M.	DESIGN BY : KALABAN P.	DATE : 05/06/2020 SCALE : 1:200
			ALL DIMENSIONS ARE IN 'mm'	SIGNATURE :	DRAWING NO. :

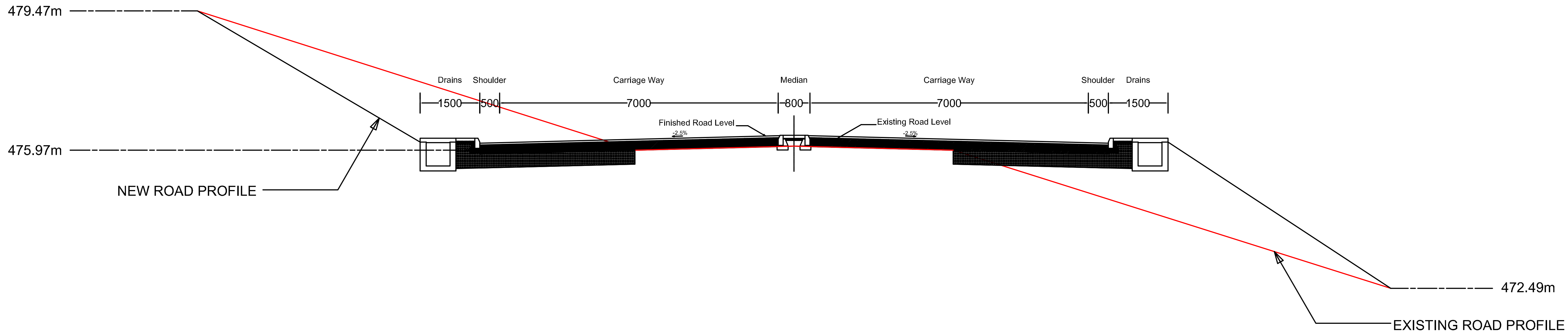


UNDERPASS LAYOUT

CLIENT : PANEL D CE 402	NEW BRIDGE FOR PERADENIYA	CONSULTANTS AND ARCHITECT : GROUP - D1	TITLE : UNDERPASS LAYOUT		
			DRAWN BY : PATHIRANA A.P.U.M.	DESIGN BY :	DATE : 05/06/2020
					SCALE : 1:500
			ALL DIMENSIONS ARE IN 'mm'	SIGNATURE :	DRAWING NO. :

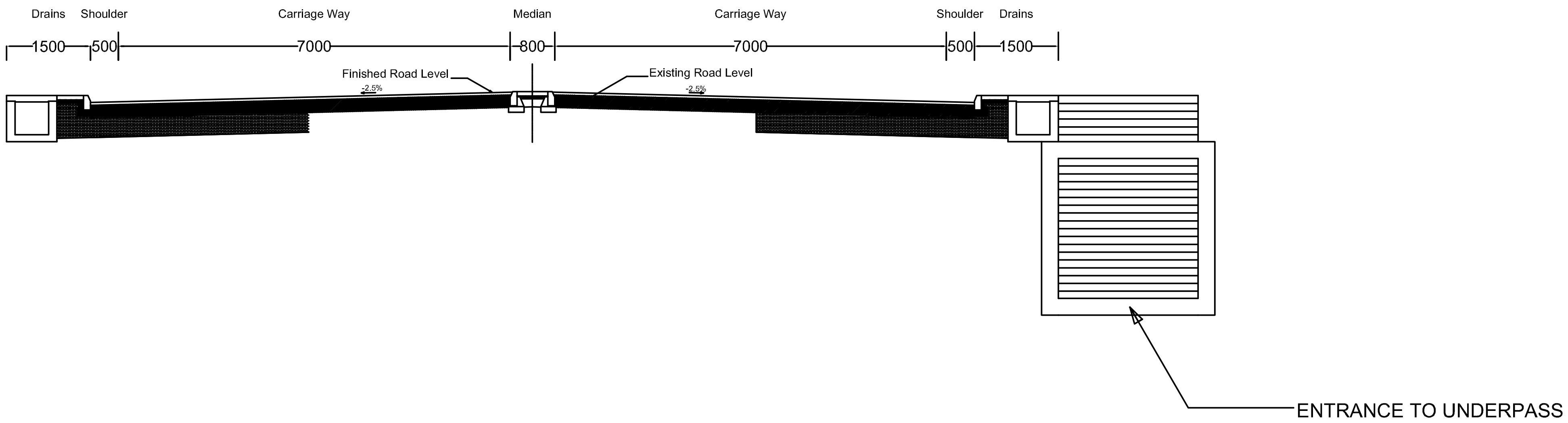


TYPICAL CROSS SECTION OF THE ROAD (VIEW AA)

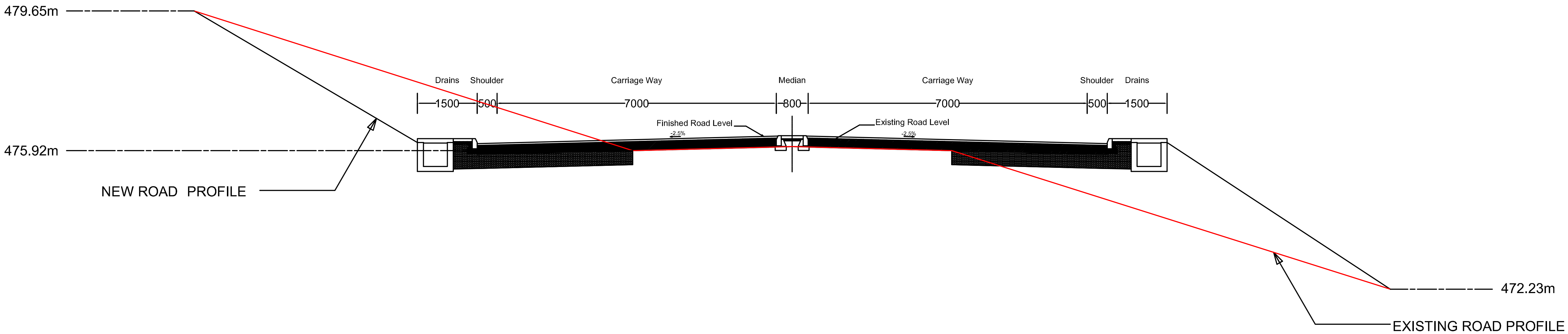


PROFILE OF THE ROAD

CLIENT : PANEL D CE 402	NEW BRIDGE FOR PERADENIYA	CONSULTANTS AND ARCHITECT : GROUP - D1	TITLE : CROSS SECTION OF THE ROAD(VIEW AA)		
			DRAWN BY : PATHIRANA A.P.U.M.	DESIGN BY :	DATE : 05/06/2020
					SCALE : 1:100
			ALL DIMENSIONS ARE IN 'mm'	SIGNATURE :	DRAWING NO. :



TYPICAL CROSS SECTION OF THE ROAD (VIEW BB)



PROFILE OF THE ROAD

CLIENT :

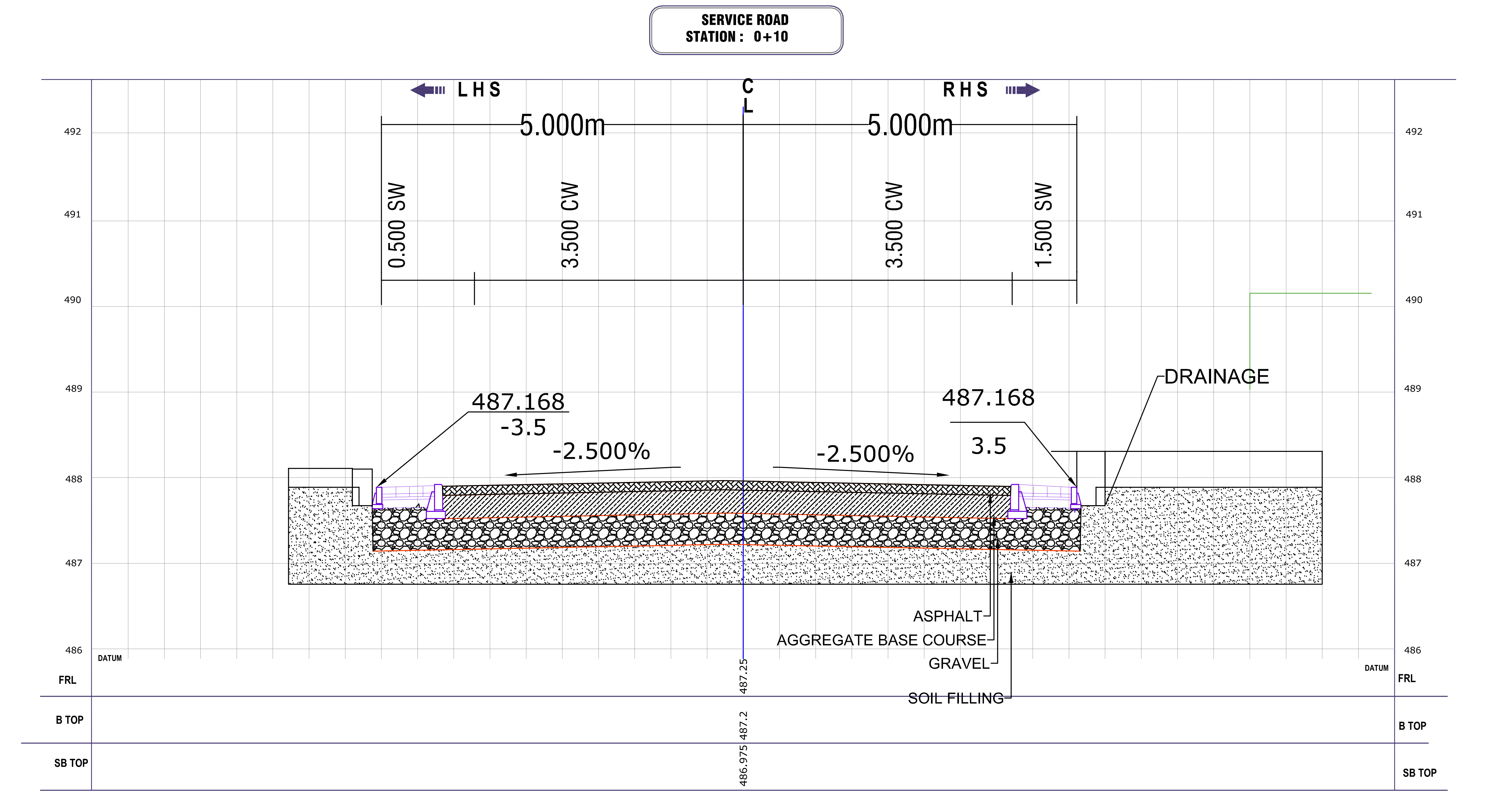
PANEL D
CE 402

NEW BRIDGE
FOR
PERADENIYA

CONSULTANTS AND ARCHITECT :
GROUP - D1

TITLE : CROSS SECTION OF THE ROAD(VIEW BB)

DRAWN BY : PATHIRANA A.P.U.M.	DESIGN BY :	DATE : 05/06/2020
		SCALE : 1:100
ALL DIMENSIONS ARE IN 'mm'	SIGNATURE :	DRAWING NO. :



CLIENT :

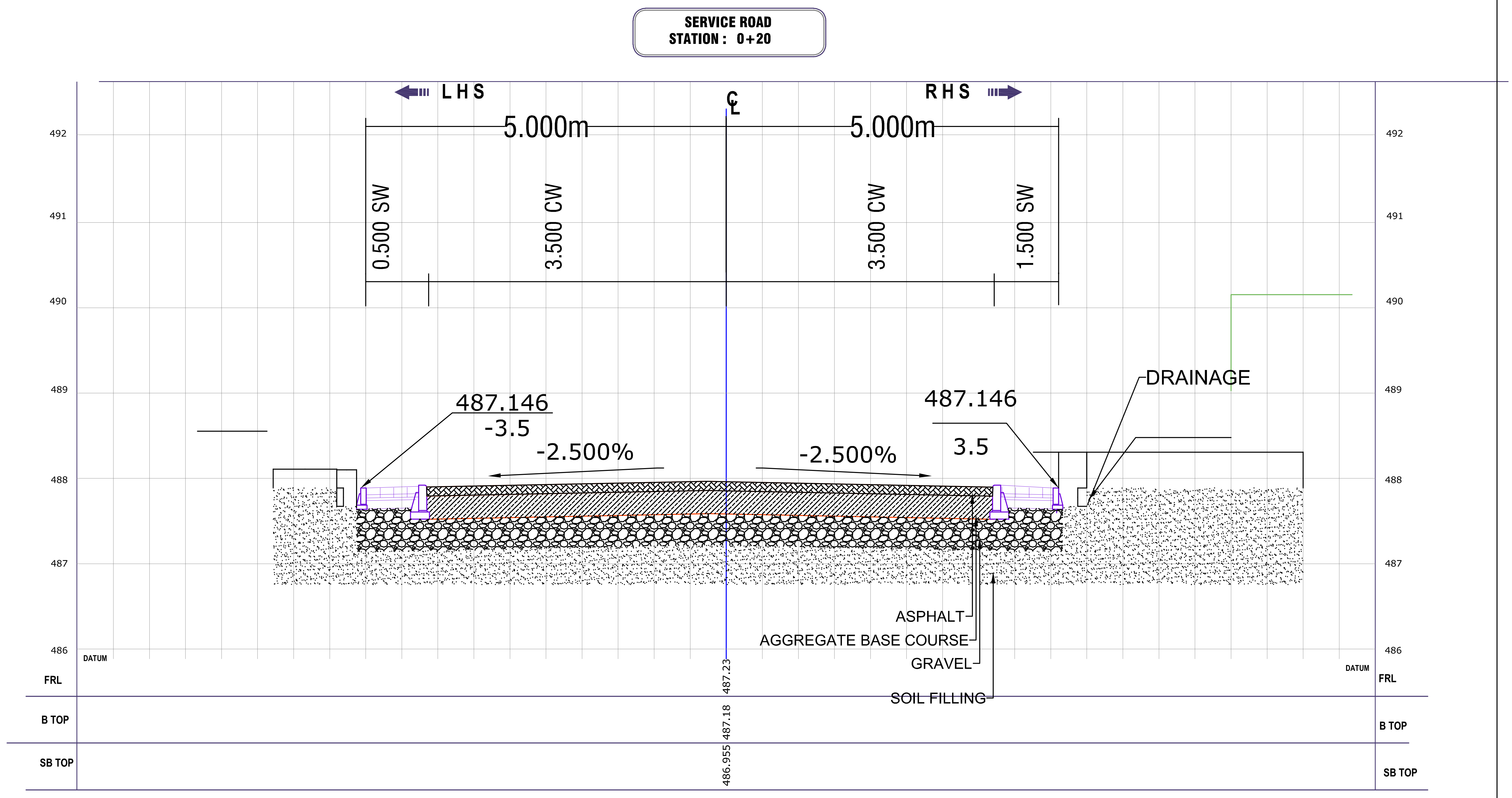
PANEL D
CE 402

NEW BRIDGE FOR PERADENIYA

CONSULTANTS AND ARCHITECT :
GROUP - D1

TITLE : ROAD CROSS SECTION 0+10

DRAWN BY : BRANAWAN K.	DESIGN BY : THANIKARUBAN T.	DATE : 05/06/2020
		SCALE : 1:50
ALL DIMENSIONS ARE IN 'mm'	SIGNATURE :	DRAWING NO. :



CLIENT :

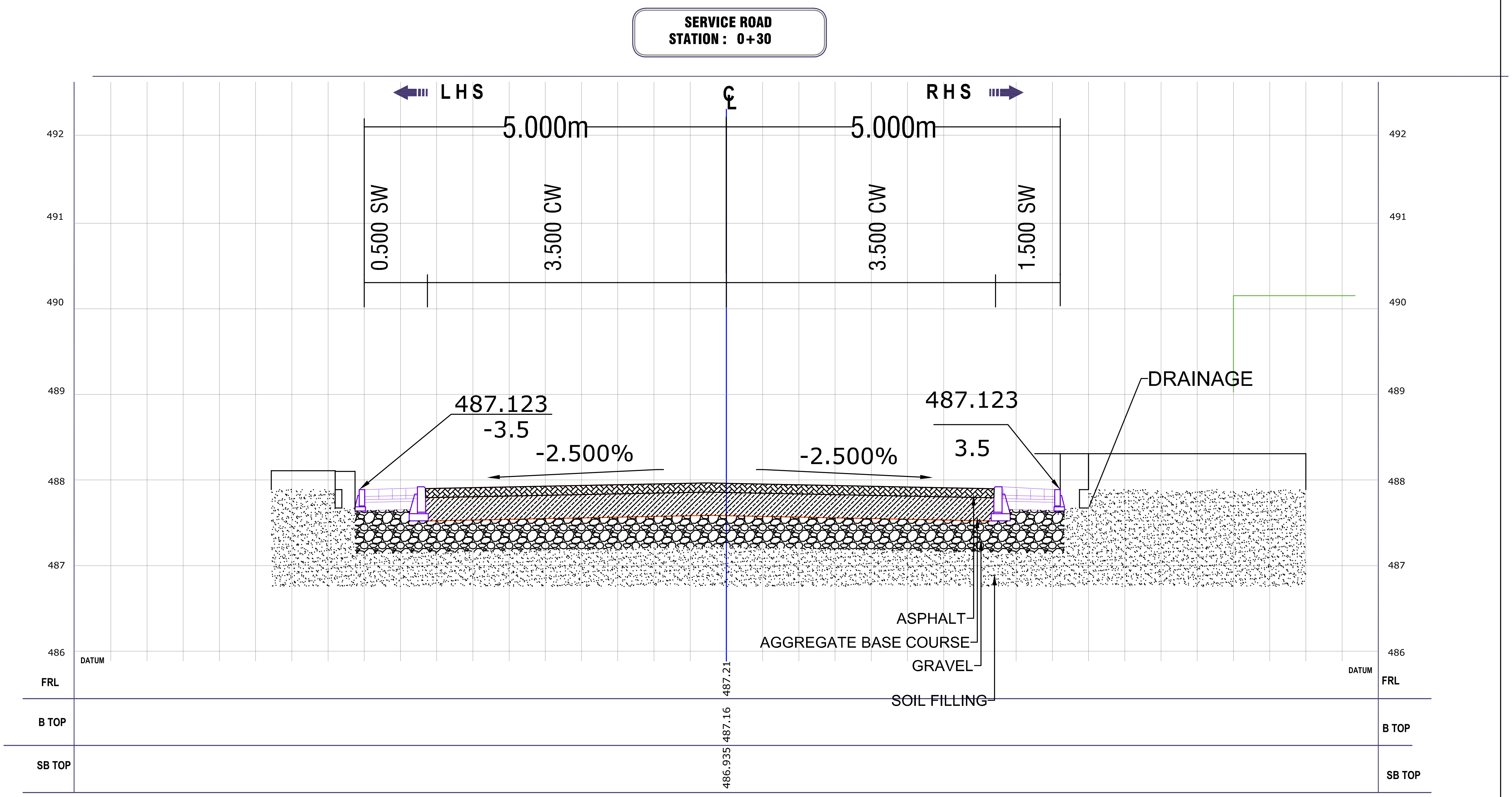
PANEL D
CE 402

NEW BRIDGE
FOR
PERADENIYA

CONSULTANTS AND ARCHITECT :
GROUP - D1

TITLE : ROAD CROSS SECTION 0+20

DRAWN BY : BRANAWAN K.	DESIGN BY : THANIKARUBAN T.	DATE : 05/06/2020 SCALE : 1:50
ALL DIMENSIONS ARE IN 'mm'	SIGNATURE :	DRAWING NO. :



CLIENT :

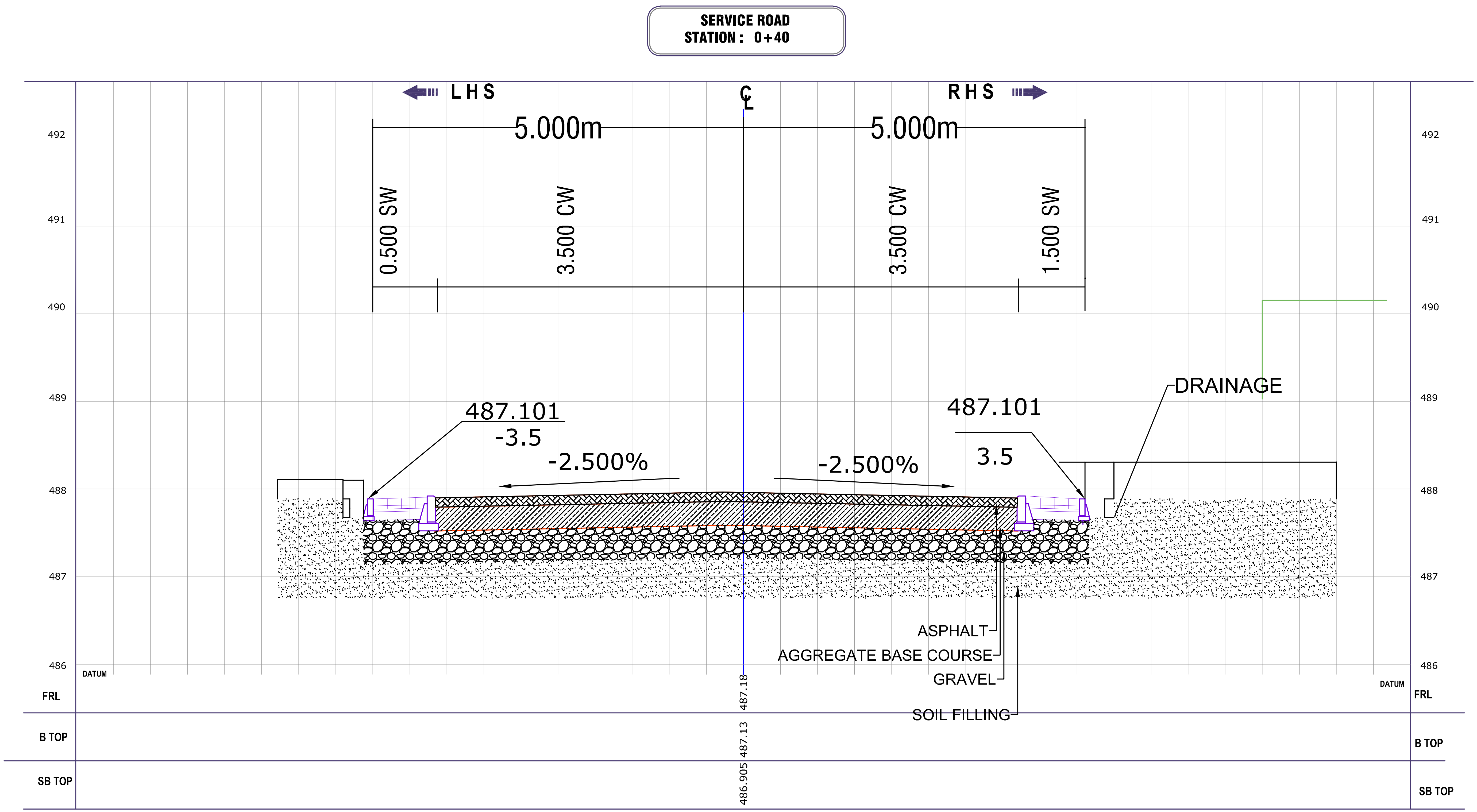
PANEL D
CE 402

NEW BRIDGE FOR PERADENIYA

CONSULTANTS AND ARCHITECT :
GROUP - D1

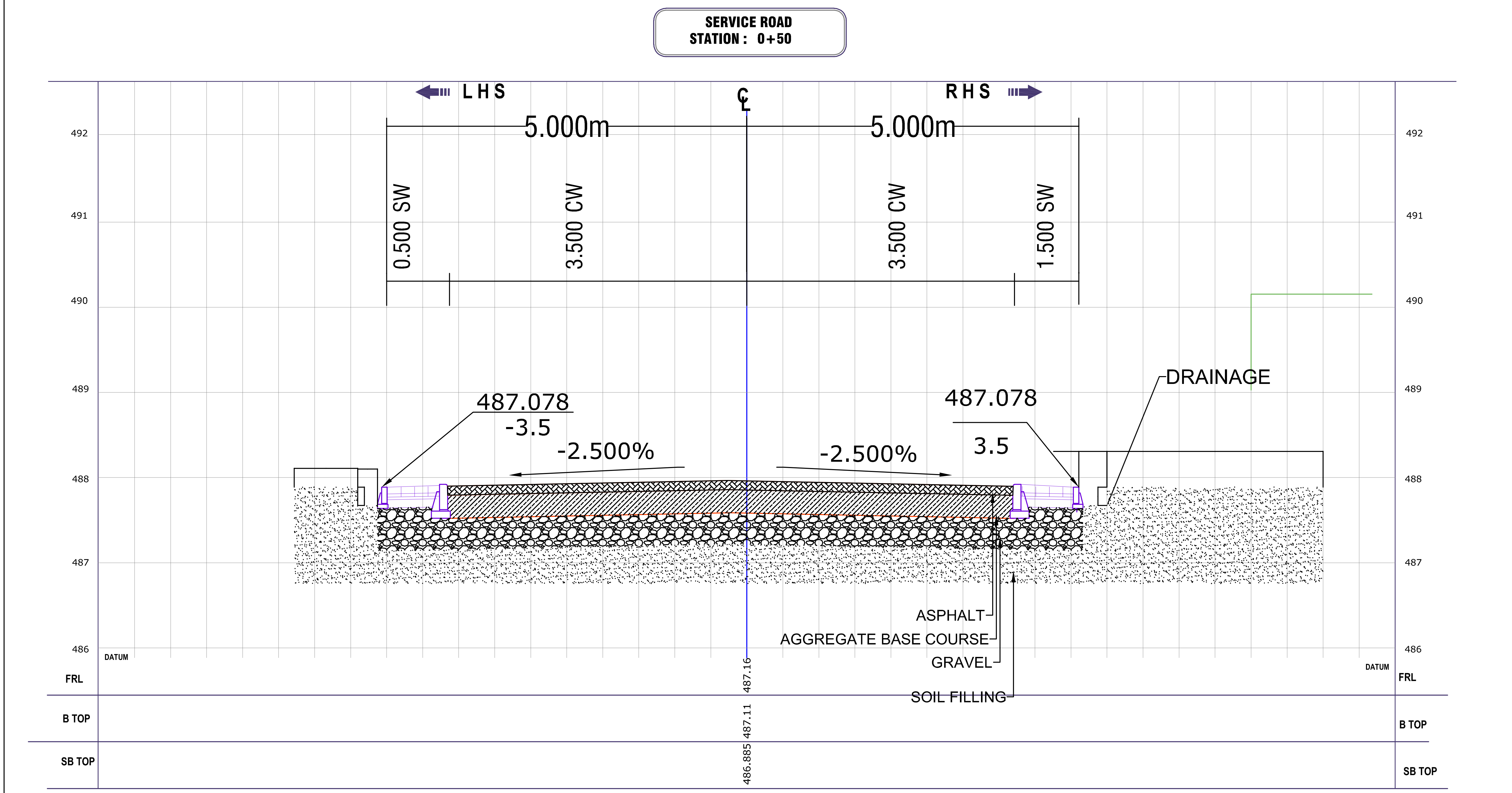
TITLE : ROAD CROSS SECTION 0+30

DRAWN BY : BRANAWAN K.	DESIGN BY : THANIKARUBAN T.	DATE : 05/06/2020
		SCALE : 1:50
ALL DIMENSIONS ARE IN 'mm'	SIGNATURE :	DRAWING NO. :



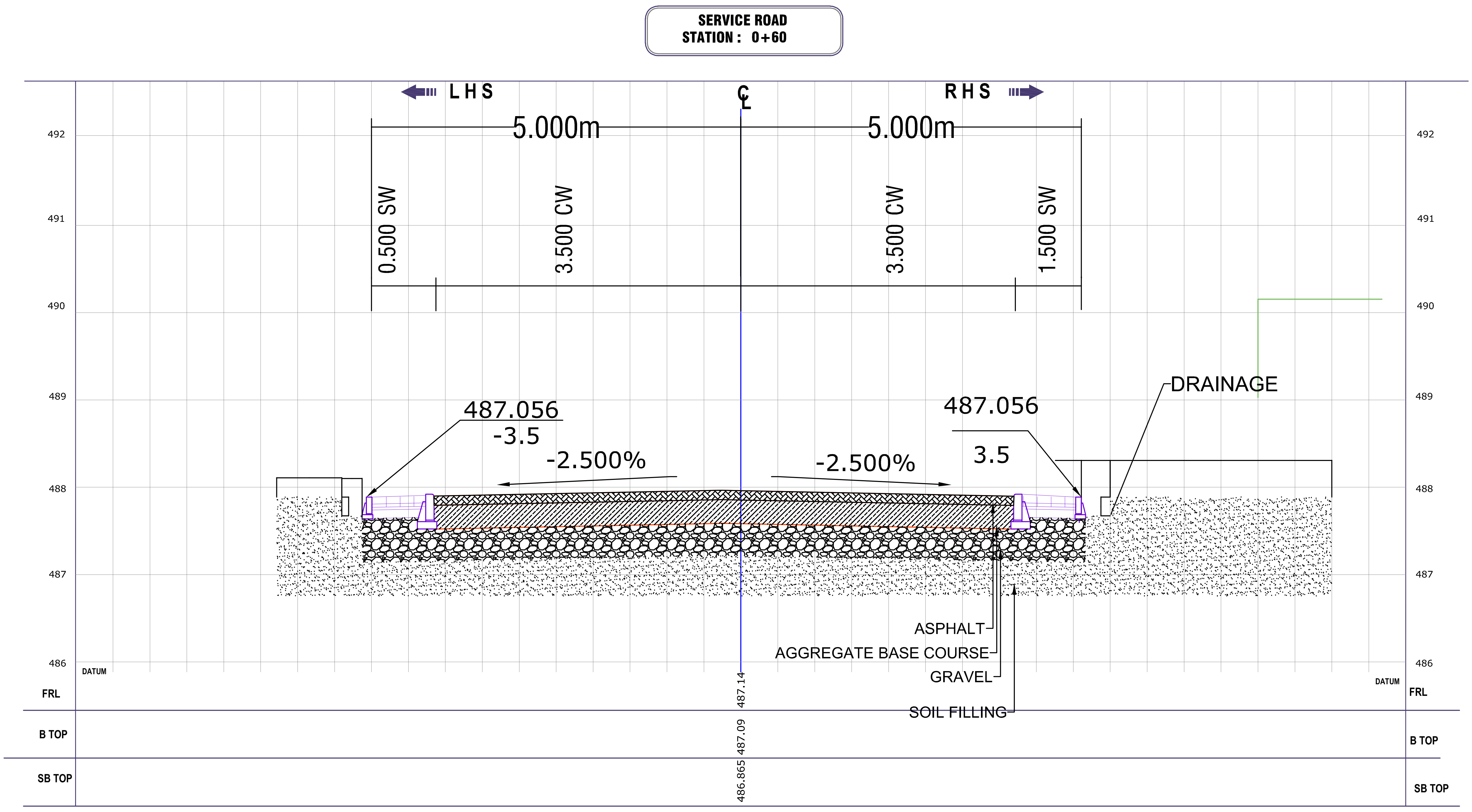
CLIENT : PANEL D CE 402	NEW BRIDGE FOR PERADENIYA	CONSULTANTS AND ARCHITECT : GROUP - D1	TITLE : ROAD CROSS SECTION 0+40

DRAWN BY : BRANAWAN K.	DESIGN BY : THANIKARUBAN T.	DATE : 05/06/2020 SCALE : 1:50
ALL DIMENSIONS ARE IN 'mm'	SIGNATURE :	DRAWING NO. :



CLIENT : PANEL D CE 402	NEW BRIDGE FOR PERADENIYA	CONSULTANTS AND ARCHITECT : GROUP - D1	TITLE : ROAD CROSS SECTION 0+50

DRAWN BY : BRANAWAN K.	DESIGN BY : THANIKARUBAN T.	DATE : 05/06/2020
		SCALE : 1:50
ALL DIMENSIONS ARE IN 'mm'	SIGNATURE :	DRAWING NO. :



CLIENT :

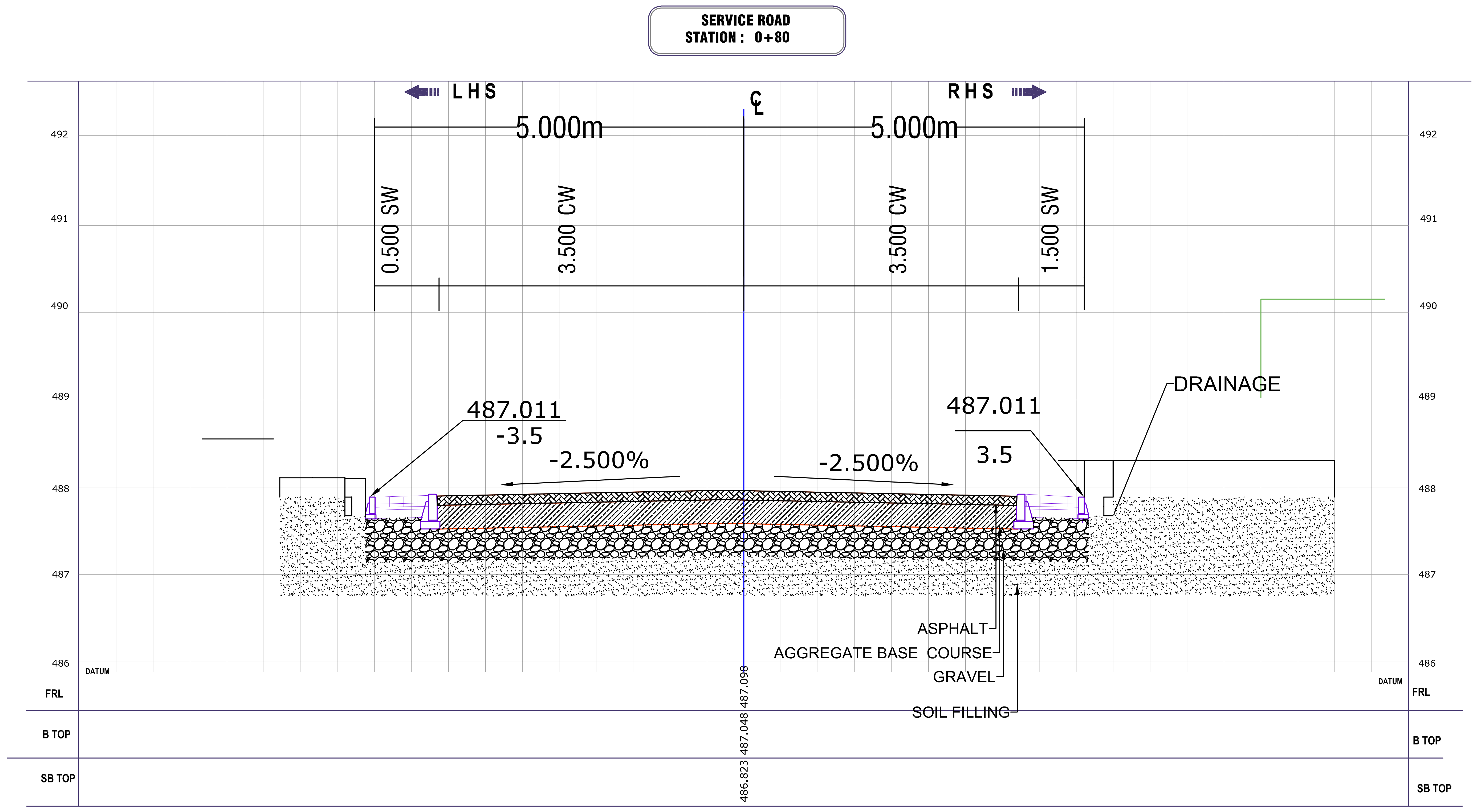
PANEL D
CE 402

NEW BRIDGE
FOR
PERADENIYA

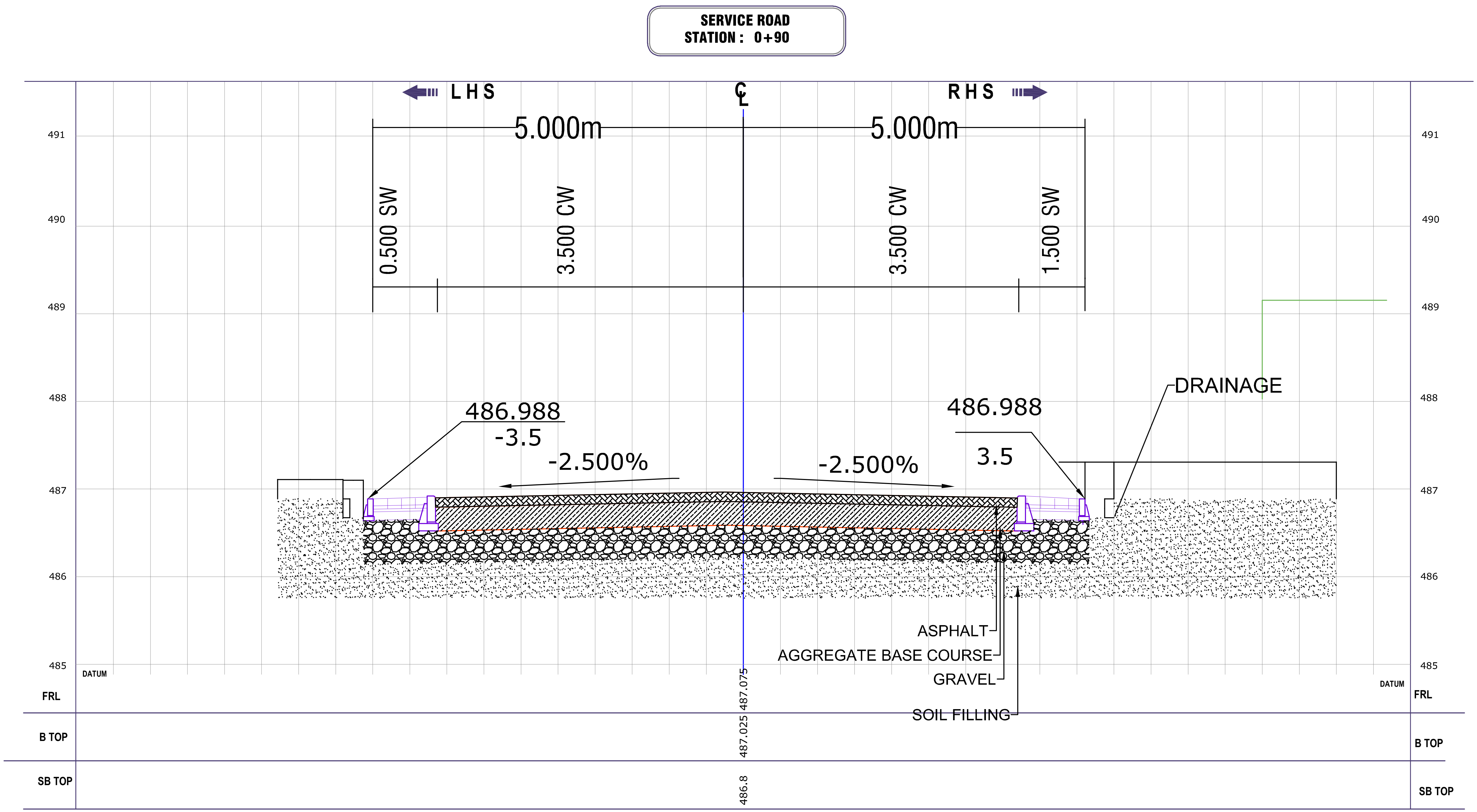
CONSULTANTS AND ARCHITECT :
GROUP - D1

TITLE : ROAD CROSS SECTION 0+60

DRAWN BY : BRANAWAN K.	DESIGN BY : THANIKARUBAN T.	DATE : 05/06/2020
		SCALE : 1:50
ALL DIMENSIONS ARE IN 'mm'	SIGNATURE :	DRAWING NO. :



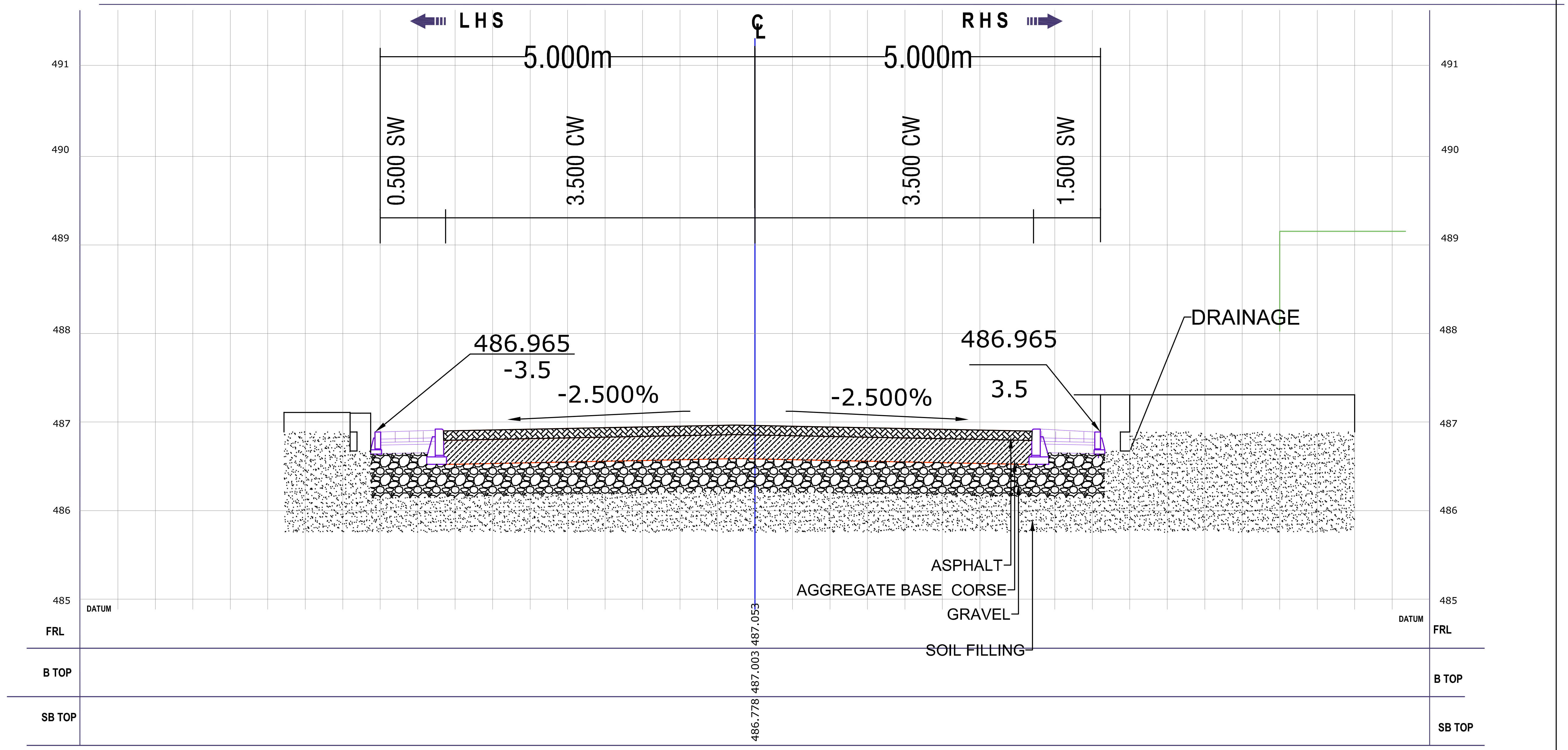
CLIENT : PANEL D CE 402	NEW BRIDGE FOR PERADENIYA	CONSULTANTS AND ARCHITECT : GROUP - D1	TITLE : ROAD CROSS SECTION 0+80 DRAWN BY : BRANAWAN K. DESIGN BY : THANIKARUBAN T. DATE : 05/06/2020 SCALE : 1:50 ALL DIMENSIONS ARE IN 'mm' SIGNATURE : DRAWING NO. :



CLIENT : PANEL D CE 402	NEW BRIDGE FOR PERADENIYA	CONSULTANTS AND ARCHITECT : GROUP - D1	TITLE : ROAD CROSS SECTION 0+90

DRAWN BY : BRANAWAN K.	DESIGN BY : THANIKARUBAN T.	DATE : 05/06/2020
ALL DIMENSIONS ARE IN 'mm'	SIGNATURE :	SCALE : 1:50
		DRAWING NO. :

SERVICE ROAD
STATION : 0+100



CLIENT :

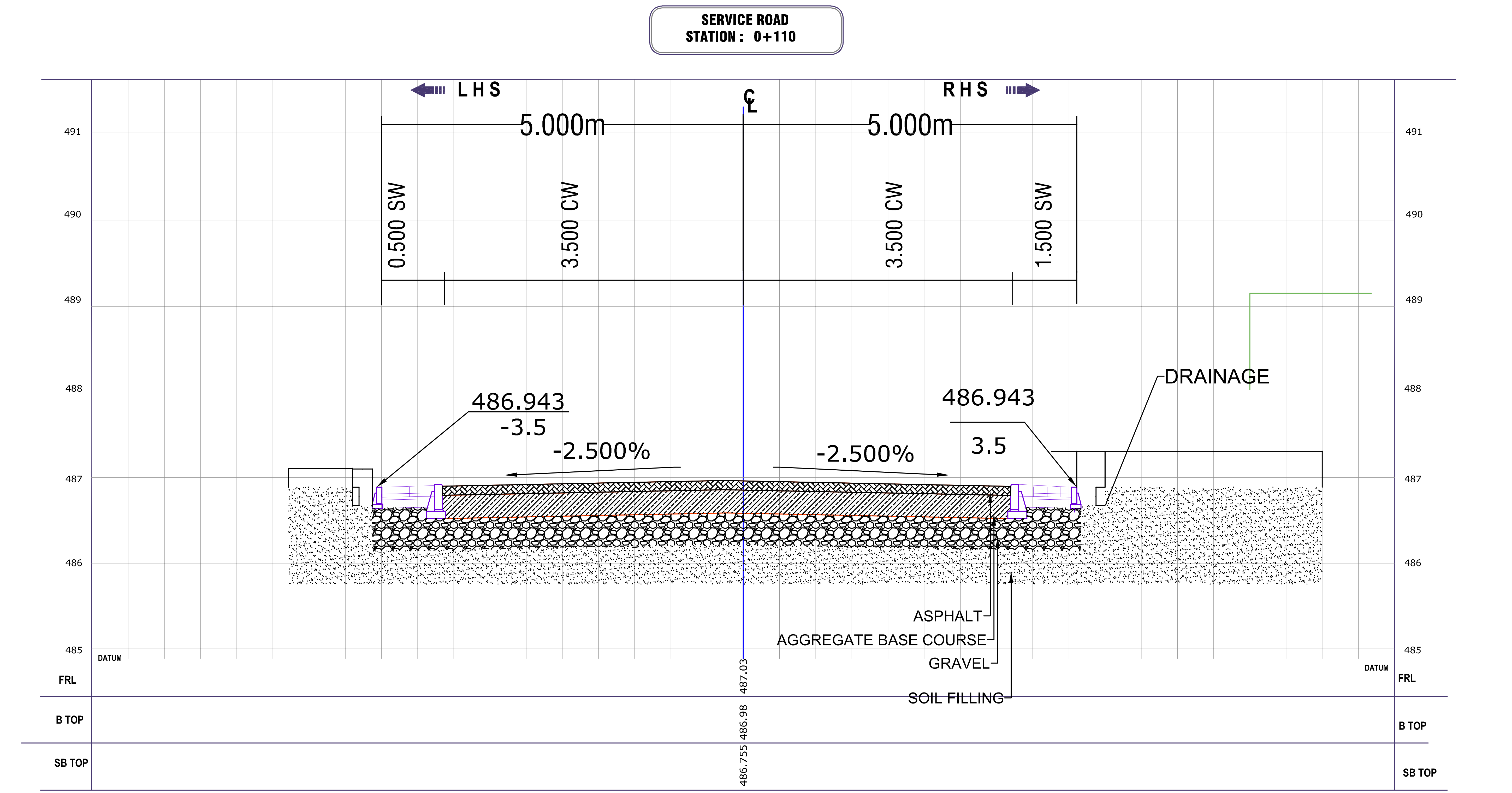
PANEL D
CE 402

NEW BRIDGE FOR PERADENIYA

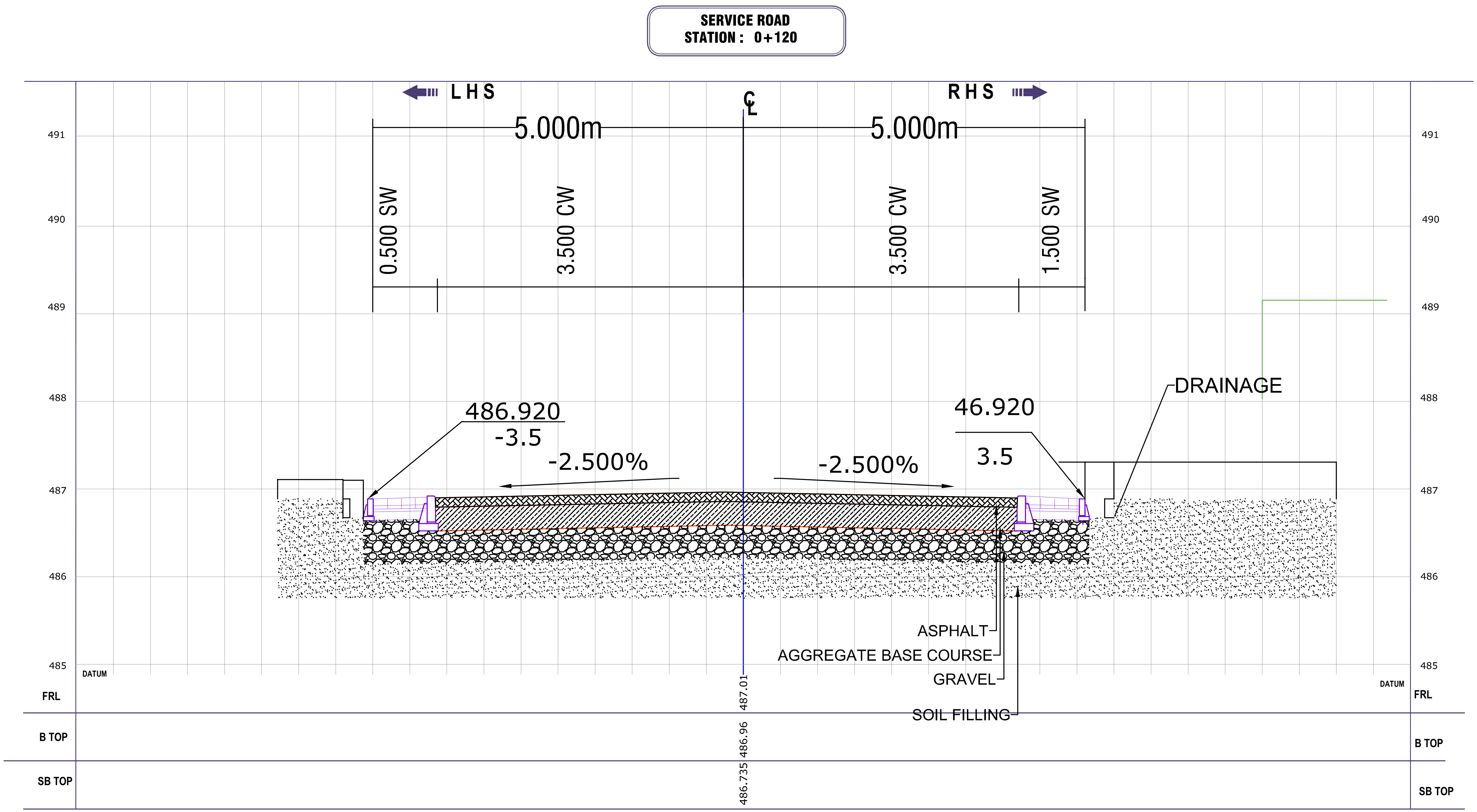
CONSULTANTS AND ARCHITECT :
GROUP - D1

TITLE : ROAD CROSS SECTION 0+100

DRAWN BY : BRANAWAN K.	DESIGN BY : THANIKARUBAN T.	DATE : 05/06/2020
		SCALE : 1:50
ALL DIMENSIONS ARE IN 'mm'	SIGNATURE :	DRAWING NO. :



CLIENT : PANEL D CE 402	NEW BRIDGE FOR PERADENIYA	CONSULTANTS AND ARCHITECT : GROUP - D1	TITLE : ROAD CROSS SECTION 0+110		
			DRAWN BY : BRANAWAN K.	DESIGN BY : THANIKARUBAN T.	DATE : 05/06/2020 SCALE : 1:50
			ALL DIMENSIONS ARE IN 'mm'	SIGNATURE :	DRAWING NO. :



CLIENT :

PANEL D
CE 402

NEW BRIDGE FOR PERADENIYA

CONSULTANTS AND ARCHITECT :
GROUP - D1

TITLE : ROAD CROSS SECTION 0+120

DRAWN BY :
BRANAWAN K.

DESIGN BY :
THANIKARUBAN T.

DATE :
05/06/2020

SCALE :
1:50

ALL DIMENSIONS ARE IN 'mm'

SIGNATURE :

DRAWING NO. :