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Khelekhele Bridge Design

Engineers in Action Suspended Bridge Design

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ABSTRACT

This project will explore the design of a suspended footbridge that will be placed between the communities of Mbelebeleni and Nkiliji in Eswatini, Africa. Spanning 93.5 meters, this bridge will provide a connection between these communities providing access between the sides of the Black Mbuluzi River. Once completed, this project will serve over 3,300 people and 2,600 children by allowing access to essential resources year-round. Furthermore, the suspended walkway bridge will allow access between the communities during the flood season. The design was created partnering with Engineers in Action (EIA), a non-profit organization that aims to build sustainable infrastructure, supplying resources to underserved communities. The University of Southern Indiana (USI) Engineers in Action (EIA) chapter will collaborate with the chapters of Penn State University and Colorado University (CU) Boulder to create a safe and efficient bridge that EIA will construct in summer 2025. The report details the preliminary designs created early in the design process, as well as the finalized design. The project deliverables include the calculation page, checked by the design engineer in charge (DEIC) and the Engineers in Action team, as well as the full plan set. The plan set includes a bridge layout page, and detail pages for the walkway, steel crossbeams, fencing, suspenders, cables, and decking. The full plan set will be referenced during construction of the bridge beginning in May 2025.

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INTRODUCTION

Engineers in Action (EIA) is a non-profit organization that provides resources to underserved communities globally through infrastructure. By combining the minds of many students, engineers, and EIA personnel across the United States, EIA creates multiple bridges that are designed and constructed each year in underserved communities throughout the world. These bridges allow communities access to essential resources, such as healthcare and education during the wet season when rivers inevitably flood. The University of Southern Indiana (USI) EIA chapter collaborated with student chapters from Penn State University and Colorado University Boulder, EIA professional engineers, and project managers throughout the design process to create the Khelekhele Footbridge.

1 BACKGROUND

1.1 ENGINEERS IN ACTION

Founded in 2006, the non-profit organization, Engineers in Action (EIA), seeks to provide transportation access for underserved communities by safe and sustainable infrastructure. Hundreds of students from over forty universities have designed and built over 110 footbridges, serving thousands of people [1]. Students on the travel teams have been immersed in the cultural traditions of the community in which the bridge will be built. Along with the EIA Bridge Program, this organization also provides services that supply sanitary water in places of most need. These services are provided throughout areas of Latin America and South Africa.

To participate in the EIA Bridge Program design, two bridge design courses must be completed. The first course was an introduction to bridge design calculations, as well as the language that is used throughout EIA design procedures. The second course provides a more thorough approach to bridge design and calculations. During these courses, a review call is completed in which the students taking the course present their findings and receive mentorship from a design engineer in charge (DEIC). The courses and review call process ensures the designers for the project are knowledgeable about the progress on the necessary deliverables and expectations [2].

1.2 PROJECT NEED

As previously mentioned, the EIA Bridge Program designs and builds bridges for communities that are unable to access essential resources. During the wet season, the Black Mbuluzi River floods, restricting access between the communities of Mbelebeleni and Nkiliji of Eswatini, Africa. The typical flood duration lasts three days, but the river remains difficult to cross by foot for approximately three weeks. Flooding causes safety risks upon the communities and directly restricts communal access.

The addition of year-round access between the two communities will serve over 4,250 people, 2,600 of which are children. Walking is one of the main means of transportation in this area which can isolate communities when their access is restricted due to flooding or dangerous conditions. Within the last three years, four deaths have occurred; furthermore, ten injuries have been reported that directly relate to dangerous attempts to cross the river. The nearest all-weather river crossing point is eight kilometers from the proposed location. The current crossing consists of large rocks that span the width of the river. As water levels rise during the rainy season, these rocks would not be available; thus, eliminating a path to cross the river. A bridge in the proposed area is crucial to ensure safe walks to schools, medical centers, churches, markets, and livestock trading during dangerous crossing times. After construction is complete, the bridge will greatly increase the health of community members by limiting casualties caused by crossing the dangerous current crossing. It will provide community members with access to vital resources on either side of the Mbuluzi River. The current river crossing is depicted in Figure 1.2.1.

Figure 1.2.1 Current Crossing

1.3 SCOPE OF WORK

The main deliverable of the project is the design for the Khelekhele Footbridge that will be constructed in the summer of 2025 to connect thousands of people during dangerous bridge crossing conditions. The USI chapter of EIA will be collaborating with the student chapters from Penn State and CU Boulder to create and build this design. Through collaboration with partnering universities and EIA engineers, the design will be the combination of a variety of ideas and requirements [3]. The bridge will be designed in accordance with the standards and specifications within “EIA Bridge Program Volume 2: Design” [4].

Prior to beginning construction, design team members must complete two EIA Education courses; these two courses are EIA 201: Suspended Bridge Design and EIA/BP 211: Advanced Suspended Bridge Design. These courses aid university students in understanding the fundamental ideas prior to design of the pedestrian bridge.

A geometric layout of the bridge and a complete geotechnical and structural analysis will be performed alongside the design to ensure safety and stability. These analyses will accompany a complete drawing set as the final deliverable. Considering the availability and accessibility of construction materials is important, as these items will be limited in this area. These considerations and analyses will ensure the needed materials can be accessed, as well as ensuring the safety and structural integrity of each component of the bridge. Resources that were used to complete the design of the project include the use of AutoCAD and Excel, in addition to the use of the specifications and code provided by Engineers in Action [4].

1.4 PROJECT TEAMWORK

The team for the Khelekhele footbridge was composed of EIA staff and students from the following EIA chapters: University of Southern Indiana, Penn State, and Colorado State University Boulder. Each chapter differs in the number of members; however, each chapter contains a project advisor, project manager, construction manager, and design leads.

The design leaders from this project include Abby Guetling (USI), Megan Muensterman (USI), and Jalen Eccles (Penn State). Collaboration occurred frequently between design leads via phone and video calls. Tasks for the design team were distributed evenly amongst the three members. The design team also communicated with project managers, EIA staff, and the DEIC to discuss design sufficiency and preceding steps on a weekly basis. This design coordination ensured the design followed ethical standards regarding safety, as well as helped the team members gain professional insight into the mentorship process.

Oversight of the project is provided by Engineers in Action staff. A brief overview of the roles and responsibilities of the individuals/ groups involved is as follows:

- Bridge Program Coordinator: Rose Schweitzer
  Provides official Notice to Proceed after each review call.

- In-Country Program Manager: Ana Jovanovic
  Provides community information, survey data, and holds bi-weekly meetings with the project manager.

- Design Engineer in Charge (DEIC): Drew Bishop
  Professional engineer who provides design advice and final approval of the drawings and calculations.
- Bridge Corporation
  Group of qualified professionals who volunteer to provide design support and review.
- Team Ambassador: Melanie Cedeno Morales
  One individual per university volunteers to mentor the team throughout the project. The ambassador is typically a university EIA alum.

2 SITE INFORMATION

2.1 SITE LOCATION

The proposed Khelekhele footbridge is located within the Hhohho and Manzini regions of Eswatini, Africa. The structure will connect the residents of the Mbelebeleni, Nsuka, and Nkiliji communities by allowing all-weather access across the Black Mbuluzi River. Additional information regarding the location of the proposed structure is as follows:

- Coordinates: 26° 17’ 47.3” S, 31° 24’ 22.5” E
- Distance to the nearest major city: 40 km
- Distance to closest paved road: 6.2 km

The proposed bridge location is at a bend in the Black Mbuluzi river, with grazing land owned by the community and a privately-owned garden on either side. The current crossing for the river is composed of large rocks placed and are used as steppingstones. It is EIA’s convention to name the sides of a river as follows: while the viewer is looking downstream, the river’s Right-Hand Side (RHS) is on the viewer’s right-hand side and the Left-Hand Side (LHS) is on the viewer’s left-hand side. At the proposed bridge location, the grazing land is on the right-hand side, and the private garden is on the left-hand side. The garden is located away from the proposed crossing so it is anticipated that this garden will not interfere bridge construction or deliveries.

Figure 2.1.1 shows the proposed bridge location along the bend in the Black Mbuluzi river. The river flows west to east, and the left-hand side and right-hand side configurations are made when looking downstream of the river. It is also important to note the access road leading to the bridge location, as people crossing the river from either direction likely use this path. The proposed bridge location remains near the access road, making the bridge convenient for community members on either side of the river.

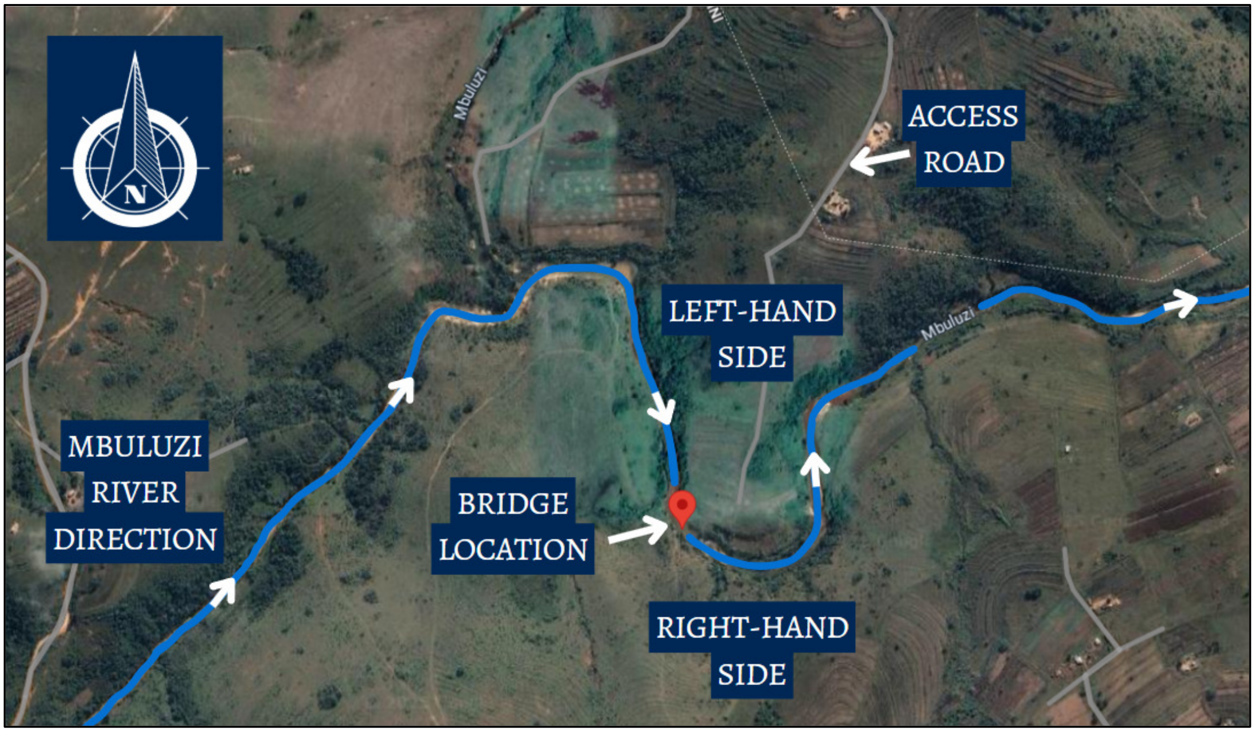


Figure 2.1.1 Proposed Bridge Location

Figure 2.1.2 shows another view of the site location, positioned from the left-hand side looking to the right-hand side. It is important to recognize the elevation difference between the lower left-hand side to the much higher right-hand side. This posed challenges in the design process, explained in greater detail in the preliminary design process.

Figure 2.1.2 Site Overview

2.2 GEOTECHNICAL DATA

The existing soil conditions are classified at four main locations of the proposed bridge, left anchor, left foundation, right anchor, and right foundation. According to the technical assessment provided by EIA, the visual classification from EIA personnel at all four locations was "sandy loam" which can be seen by Figure 2.2.1. The technical assessment provided approximate bearing capacities at each location. Both left-hand side locations yielded a bearing capacity of approximately 67 kPa while both right-hand side locations yielded a bearing capacity of approximately 105 kPa. Since these approximate values were based on rough data, the International Building Code bearing capacity values were utilized within the design checks. When the team arrives on site, more field testing will be performed regarding geotechnical data to ensure the validity of the current design. A layer of crushed stone will be placed beneath the foundation if field results are too low. More information regarding bearing capacity follows in section 4.2.

Figure 2.2.1 Soil Visualization

Due to the unstable nature of sandy soil, a benching technique will be utilized within the excavation process for the abutments. Benching ensures soil stability during excavation by preventing soil sliding and cave-ins. This technique has been previously employed by EIA on projects where granular soils are present. An example of benching during excavation is shown in Figure 2.2.2.

Figure 2.2.2 Example of Benched Excavation

2.3 *HYDROLOGY AND HYDRAULICS*

Data regarding the average rainfall of the larger city of Manzini is shown in Figure 2.3.1 [5]. The heaviest rainfall tends to occur between the months of October and April, peaking near the end of January and the beginning of February. As Eswatini is below the equator, the summer and winter months occur at the opposite times that they typically would in the United States. Please note that flood conditions are not to occur during the planned time of construction which will be from May-July.

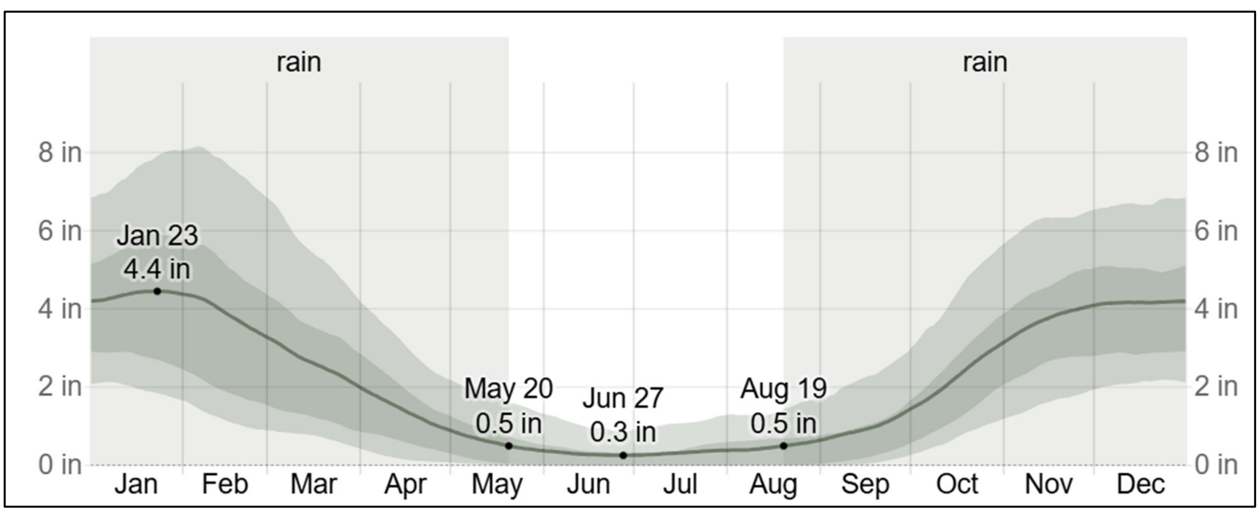


Figure 2.3.1 Average Yearly Rainfall in Manzini, Eswatini

The location of the high-water line (HWL) at the bridge centerline was determined from historical data of one of the area's largest, most recent rainfall events. This event, Cyclone Eloise, devastated many countries due to not only high wind velocities, but the floods that it caused in January 2021. The elevation of the approximate HWL was set at 100 meters. This value serves as a datum for the survey of the river cross section, as was provided to the team as an AutoCAD file by EIA. Please see Figure 2.3.2 for an elevation view of the survey data located at the proposed centerline by EIA. The HWL is located at a depth of 4.57 meters from the bottom of the river channel and has a width of approximately 58 meters.

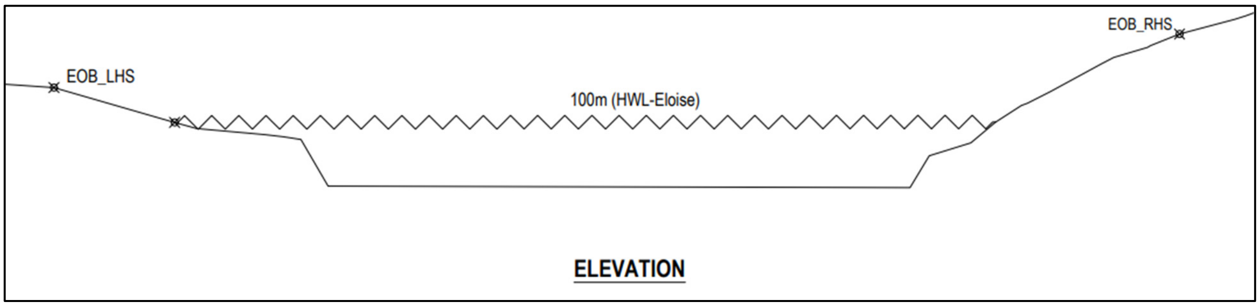


Figure 2.3.2 Survey of Centerline Provided by EIA

A complete hydrologic analysis is not generally required for EIA bridge sites. Although, it is important to consider any potential threats to the bridge caused by hydrologic events. Two of the largest concerns for suspended footbridges are damage due to high water levels and erosion. This site has been classified as a gorge by EIA; therefore, there is a minimum freeboard requirement of 3 meters above the HWL under a dead load sag. This requirement eliminates the concern of the structure being damaged by high water levels. The location of this bridge is on a sharp bend in the river; sharp bends are often associated with erosion/deposition patterns, where the outside of the bend experiences higher stream velocities causing erosion. However, this is not a concern of the Khelekhele bridge site, as information provided by the community has noted that there have been no significant changes to the river's location in recent history.

3 DESIGN PROCESS

3.1 DESIGN CONSIDERATIONS

The primary objective of an engineer should always be the safety and welfare of the public. This not only involves structural integrity and serviceability, but it also involves user safety. The design of a pedestrian bridge must provide safety in multiple aspects such as adequate crossbeams, railing, fencing, ramps, and accommodations for bare feet, as many people in these communities do not have shoes. Other important design considerations discussed in the EIA Bridge Binder Manual include durability, maintainability, economy, and constructability. Aesthetics is also a design consideration, although this is not a prioritized consideration throughout the design process. The local communities have freedom to paint and decorate the bridge as desired. When the local residents are engaged in the construction process, this creates a sense of ownership and pride, increasing the bridge's long-term sustainability and upkeep. Furthermore, the bridge can have a wide range of social impacts by facilitating greater interaction between neighboring community members. Additionally, when university students travel to Eswatini, their involvement in the bridge construction will shape their global perspective, social awareness, and interpersonal skills.

When planning to work in a different area of the world, it is important to consider the community's access to materials. These communities have very limited access to construction equipment; thus, excavation and concrete mixing will be performed using shovels. It is beneficial to create a design that limits the amount of excavation as much as possible. Material acquisition is further discussed in Section 4.1 of this report.

Environmental aspects should also be considered in the design. It is important to not create any environmental hazards within the design or during construction. Environmental hazards will be mitigated by containing concrete washout and providing proper trash disposal. This will minimize hazards and disruptions to nearby habitats.

Ethics and professionalism must be demonstrated throughout the entirety of the project. All details of the design must be considered thoroughly as public safety and welfare are at risk. Regardless of extra effort or cost, the team has, and must continue, to put forth maximum effort in ensuring the safety of this design and construction. Professional conduct is required in all aspects of the project, which includes acting professionally during all review calls, as well as representing the university well.

3.2 GEOMETRIC REQUIREMENTS

For the preliminary design of the Khelekhele suspended footbridge, the procedures and requirements within the EIA Bridge Binder Manual were referenced. EIA has a set of geometric requirements that must be met with all bridge designs. Preliminary designs must only meet these requirements, while final designs must meet more specific requirements. The preliminary design requirements are as follows:

- Maximum span length, L = 120 meters
- Maximum difference in saddle height, ΔH = L/25 meters
- Minimum freeboard = 3 meters
- Minimum abutment setback = 5 meters
- Maximum angle to front of foundation = 35°
- Maximum ground slope behind anchor = 10°
- Maximum ground slope behind anchor to create customized abutments = 20°

The limiting variable for this specific design was the maximum ground slope behind the anchor requirement. As will be further discussed below, the angle determined was between 10° and 20° degrees. The ΔH requirement was also of concern due to the elevation difference between the left and right-hand side edge of bank locations. This was able to be mitigated by a different number of tiers on each abutment.

3.3 PRELIMINARY DESIGN #1

The first preliminary design, shown in Figure 3.3.1, focuses on the right-hand side abutment. Initially, the design team worked to maximize the setback of the abutment from the edge of the bank of the right-hand side of the river. It is important, for erosion control purposes, to ensure the abutment is not placed too close to the edge of the bank of the river, especially during flood season. The preliminary design shows a 7.57-meter setback from the edge of bank. It is also important to note the raised anchor of the design to account for the ground profile slope behind the abutment being greater than ten degrees. This angle is dimensioned at 13.46 degrees, an angle still too great for the use of a standard abutment, as the angle of the ground profile slope behind the abutment needs to be less than ten degrees to use a standard abutment. The design also shows the use of a one-tier abutment to account for the right-hand side of the ground profile being much higher than the left-hand side. To keep the right-hand side elevation minimized, a one-tier abutment was selected.

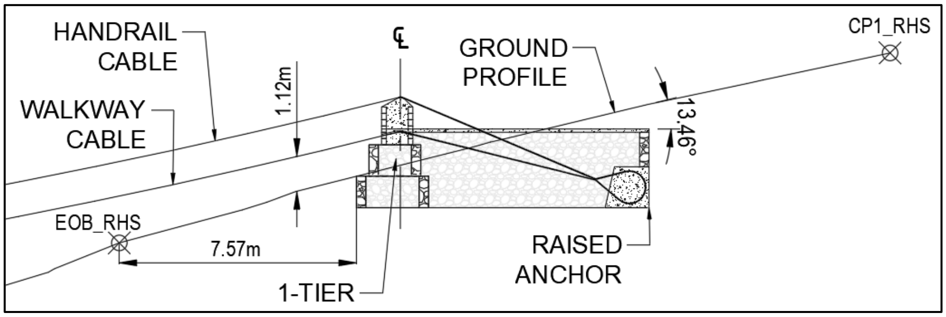


Figure 3.3.1 Preliminary Design #1

3.4 PRELIMINARY DESIGN #2

The second preliminary design, shown in Figure 3.4.1, pictures a two-tier abutment with a raised anchor. This design, again focused on the right-hand side abutment, shortened the setback to 5.86 meters rather than the 7.57 meters from the first preliminary design. The minimum setback, by EIA standards, is five meters from the edge of the bank of the river, so both preliminary designs meet the standards set by the Engineers in Action bridge binder manual. It is also important to note in the second preliminary design the use of a raised anchor to lower the ground profile slope to 9.30 degrees. While the raised anchor mitigated the high ground profile slope behind the abutment, it posed an issue concerning drainage as the walkway was sloped toward the centerline of the abutment. It also posed an issue regarding the safety of people walking down the hill to approach the bridge walkway. This design also required an excess amount of excavation and materials, making it costly and difficult to construct in the field.

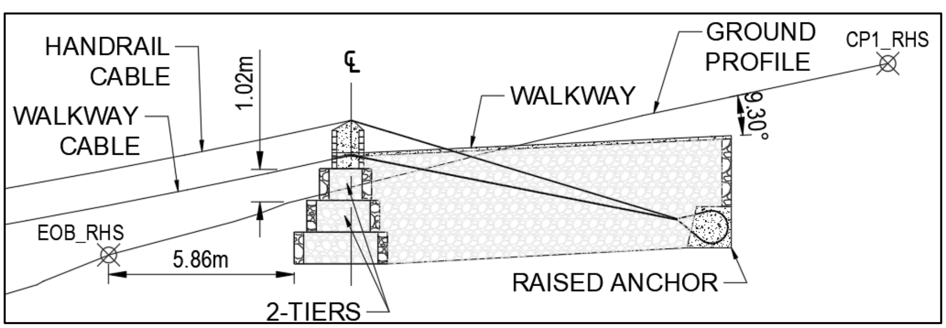


Figure 3.4.1 Preliminary Design #2

3.5 DESIGN CHALLENGES

While exploring the different alternatives throughout the design process, along with the feedback from the design engineer in-charge (DEIC), the team encountered a series of design challenges that were overcome to reach the finalized design. First, as previously mentioned, the left-hand side of the river is at a much higher elevation than the right-hand side. This makes it especially difficult to obtain a design that keeps safety and serviceability as top concerns. It is important for the bridge design to account for the lower side by utilizing a greater number of tiers while keeping the elevation of the abutment on the higher side of the bank as low as possible.

Another key difficulty throughout the design process was the ground profile slope behind the right-hand side abutment being greater than ten-degrees. Since the team noticed this from the beginning of the design process, they were able to accommodate for this angle by creating a non-standard abutment on the right-hand side. The anchor was raised on the right-hand side to limit the ground profile slope behind the back wall of the right-hand side abutment. This promotes the safety of community members as they walk down the hill to approach the bridge approach ramp and walkway.

Lastly, since the site location was proposed along a bend in the river, it was important for the team to research the behavior of the river regarding erosion. Erosion primarily occurs on the outside of river bends where water flows faster and carries more energy. It is important that the bridge design includes a great enough setback to account for the possibility of the river meandering over time. With the use of aerial maps, the team ensured the river had not changed shape much in the past few years to predict its behavior in the coming years after the construction of the bridge. Another issue with the proposed site location was the obstructions, or trees, on the left-hand side of the riverbank. Soil bearing capacity was also an issue due to the high presence of sand at the proposed bridge location. This challenge was previously discussed in section 2.2 regarding geotechnical data.

Despite the listed design challenges, the team was able work with the DEIC to finalize a suspended bridge design to be constructed in the summer of 2025, as teams from the three partnering universities travel to Eswatini to make this design come to life. The finalized design is discussed in detail in the following section.

4 FINALIZED DESIGN

Upon completing the Engineers in Action bridge design courses, mentorship from a DEIC was used throughout the design process to finalize the suspended bridge design. The full design includes a plan set regarding the layout showing the full span of the bridge, the left-hand side and right-hand side abutments, and details regarding the anchor, tower, walkway, crossbeams, fencing, and drainage. Calculations were performed analyzing each of the bridge components, with guidance from the EIA Bridge Binder Manual [4]. The following sections describe the checks performed on the finalized bridge design to ensure the safety and feasibility of the bridge design.

4.1 GEOMETRIC CHECKS

When beginning the design process, the team ensured the design passed the geometric requirements explained in section 3.2. Figure 4.1.1 shows the full bridge span, a view which can also be found in the full plan set in the appendix. The values in the elevation view of this figure were put into the spreadsheet to ensure all geometric requirements were met.

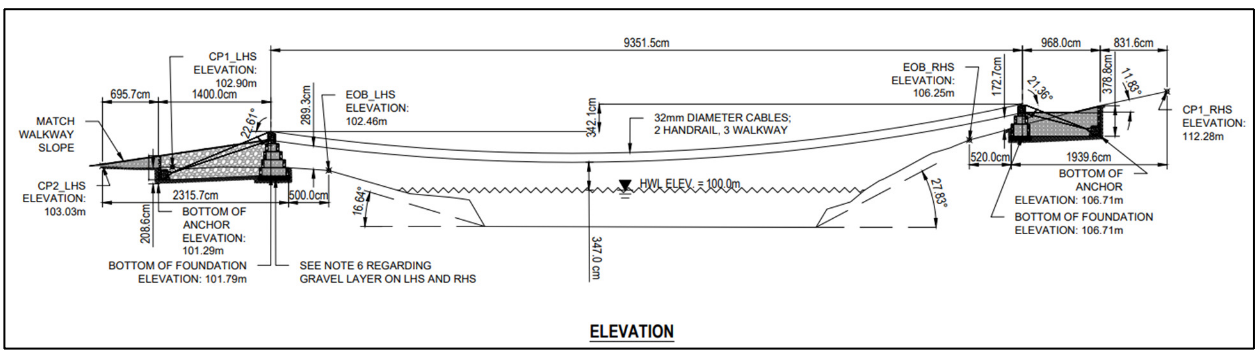


Figure 4.1.1 Bridge Span

Table 4.1.1 shows the geometric requirements and values found throughout the calculation process. All values passed the given requirement, allowing the design to move forward to the next step in the calculation process. This informs the design team the given design is feasible to build; however, other checks must also be met to finalize the design.

Table 4.1.1 Geometric Checks

Parameter	Value	Requirement
Span length, L	93.5 m	< 120 m
Height Differential, ΔH	3.42 m	< 3.74 m
Freeboard	3.47 m	> 3 m
Left Abutment Offset	5.00 m	≥ 5 m
Right Abutment Offset	5.20 m	≥ 5 m
Left Angle to Foundation Front	16.64°	≤ 35°
Right Angle to Foundation Front	27.83°	≤ 35°
Left Angle Behind Anchor	0°	≤ 20°
Right Angle Behind Anchor	11.83°	≤ 20°

4.2 LOAD ASSUMPTIONS

The Engineers in Action bridge binder manual primarily utilizes the Allowable Stress Design (ASD) methodology with safety factors determined from reliability of loading and structural performance [4]. All global checks, including sliding, uplift, overturning, and shear failure, as well as timber material analyses are performed using this methodology. However, when performing calculations regarding reinforced concrete and masonry, the Load and Resistance Factor Design (LRFD) methodology is more common.

In regard to other load assumptions, the load calculations performed below do not account for vehicular loads, as vehicular roadways are not in the vicinity of either side of the proposed bridge location. Furthermore, since the span of the design is less than 120 meters, no wind loads were considered. The EIA bridge binder manual notes that “empirical data has proven that suspended bridges of up to 120-meters in span show no significant dynamics effects due to wind load” [4]. The load values are as follows:

Permanent Load:

Dead Load (DL): 0.861 kN/m

Transient Loads:

Live Load (LL): 4.07 kN/m

Reduced Live Load (RLL): 3.14 kN/m

Primary Load Combination:

Distributed, WC Primary (Dead Load + Reduced Live Load): 4.05 kN/m

Secondary Load Combination:

Distributed, WC Secondary (Dead Load + Live Load): 4.98 kN/m

Point Live Load: 2.22 kN

LRFD Load Combination:

1.4*DL + 1.6*LL: 6.11 kN/m

In addition to the load assumptions listed above, soil bearing capacity was also analyzed. The bearing capacity checks on both abutments did not pass. New soil test data was requested from EIA to ensure the validity of the values provided. After the new values were received, a meeting including Bridge Corporation members and EIA staff was held. This meeting discussed the values provided to the team and the testing methods that were followed to receive these values. The meeting concluded that additional soil tests will be performed once the team arrives on-site this summer, and the design team will assume an allowable bearing pressure of 100 kilopascals. This value was obtained from the International Building Code (IBC) Table 1806.2.

4.3 SUPERSTRUCTURE

The superstructure of the bridge includes cables, suspenders, decking, crossbeams, and fencing. The walkway, crossbeam, and fencing details are all standard for Eswatini and are listed below and further described in each subsection. Each component of the superstructure includes a series of checks performed, as well as figures detailing the geometry of the given component.

W3E Walkway Detail (C5E Steel Crossbeams w/ Nailer and Timber Decking)

C5E Steel Crossbeam Detail [Steel Channel C4x5.4 (SI C100x8)]

F2E Fencing Detail

4.3.1 Cables & Suspenders

The cables of the bridge design are suspended between the towers and anchored on both the left-hand side and right-hand side of the river. Five cables are utilized in the design - three walkway cables and two handrail cables, all with a diameter of 32 millimeters. The only available cable size diameter in Eswatini is 32 millimeters, or 1 ¼ inch, dictating the material choice. In addition to the five main cables, suspenders run vertically and carry the weight of the deck below, upon which the pedestrian traffic crosses. Figure 4.3.1 shows the bridge superstructure, detailing the bridge cables and suspenders.

Figure 4.3.1 Bridge Superstructure

The cable design checks confirm that the height differential ($\Delta H = 3.42$ m) is near but lower than the allowable limit of L/25 (3.74 m). This check is needed because too large of a height difference may cause excessive eccentricity on the abutment tower but also decrease the serviceability by producing steep walkways. Freeboard calculations also meet design standards, with an actual freeboard of 3.47 meters exceeding the 3.00 meters minimum for a gorge. The suspenders need to resist environmental factors and cyclical bending. The standard suspender size of #3 was utilized for this bridge. To prevent the suspenders from breaking under large loads, we met the strength requirements with a factor of safety of 21, well above the required value of 5. The maximum total tension in the cables, 857.39 kN, is effectively distributed over three walkway cables, resulting in a combined factor of safety of 3.52. The cable checks follow in Table 4.3.1; all cable checks pass the minimum requirements.

Table 4.3.1 Cable Design Checks

Cable Design Checks					
Check Height Difference					
Max Height Difference	=	L/25	=	3.74 m	
Height Difference, ΔH	=	3.42 m	≤	3.74 m	OKAY
Check Freeboard					
Distance to the Lowest Point of Cable	=	((4*hsag-ΔH)^2)/(16*hsag)	=	2.72 m	
Required Freeboard	=	3.00 m			
Actual Freeboard	=	3.47 m	≥	3.00 m	OKAY
Suspender Analysis					
Suspender Size			**# 3's**		
Factor of Safety	=	21	>	5	OKAY
Check Cable Load					
Max Cable Capacity	=	604 kN	(For a single cable)		
Max Cable Tension	=	857.39 kN	(Split over 3 walkway cables)		
Factor of Safety (All cables)	=	3.52	>	3	OKAY
Location of Max Cable in Tension	:	Axial Cable Tension, Right Side			

Along with the cable checks, the construction analysis is designed to check if the equipment used for construction is safe for the size of materials that will be used. The cables must be raised above hoisting sag to drop them into place. All checks for the construction analysis pass, most of which are well above the required factor of safety. Table 4.3.2 details the construction analysis checks regarding the winch capacity checks and the erection hook capacity check.

Table 4.3.2 Construction Analysis

Construction Analysis					
Max Total Cable Force	=	42.45 kN			
Winch capacity	=	29.40 kN			
Winch Capacity Check		29.40 kN	>	8.63 kN	OKAY
Tension Capacity of Erection Hook	=	165.9 kN			
Max Single Cable Force	=	8.49 kN			
Factor of Safety	=	13.02			
Erection Hook Capacity Check		19.56	>	3	OKAY

4.3.2 Decking & Crossbeams

The superstructure design checks include the analysis of the timber decking and steel cross beams of the bridge. These calculations are performed under the assumption that crossbeams and decking are simply supported. Steel channels (C100x50) spaced at 1 meter on center are to be used for the bridge. Figure 4.3.2 shows the decking and crossbeams of a similar bridge project through the Engineers in Action bridge program.

Figure 4.3.2 Decking and Crossbeams

Table 4.3.1 depicts the results of the analysis of the steel crossbeams that the timber decking will sit upon. As depicted, these checks meet the requirements, as the capacity is greater than the demand for both the bending moment and the bending stress. Steel crossbeams will be utilized in the Khelekhele bridge; however, the simply support timber decking analysis was also performed.

Table 4.3.3 Steel Crossbeam Analysis

Steel Crossbeam Analysis					
		Capacity		**Demand**	
Bending Moment, Distributed (kN-m)	=	0.9048	>	0.106	OKAY
Bending Stress, Point (kN-m)	=	0.9048	>	0.457	OKAY

Timber decking is standard for the bridges that EIA constructs. As previously mentioned, the results in Table 4.3.2 were calculated from the assumption that the decking is simply supported. The results below prove the timber decking to be sufficient within this design. Although the steel crossbeams will be utilized, the teams still ensured the timber decking checks passed the given requirements.

Table 4.3.4 Timber Decking Analysis

Timber Decking Analysis (Simply Supported)					
		Capacity		**Demand**	
Bending Stress, Distributed (Mpa)	=	4.65	>	1.69	OKAY
Bending Stress, Point (Mpa)	=	9.29	>	9	OKAY
Shear Stress, Distributed (Mpa)	=	1.69	>	0.08	OKAY
Shear Stress, Point (Mpa)	=	3.38	>	0.23	OKAY

4.3.3 Fencing

The fencing component of the bridge superstructure aids in the safety of pedestrians crossing the bridge. F2E is the standard fencing detail for Eswatini [4]. Figure 4.3.3 shows the fencing detail with the necessary notes on how to place and attach the fencing to the post and superstructure of the bridge. Figure 4.3.4 shows a completed bridge project with a view of the fencing running along the bridge superstructure. Mesh fencing is used for its durability and strength, as well as the country’s accessibility to this resource. This fencing type is also built to withstand extreme weather conditions and general wear and tear.

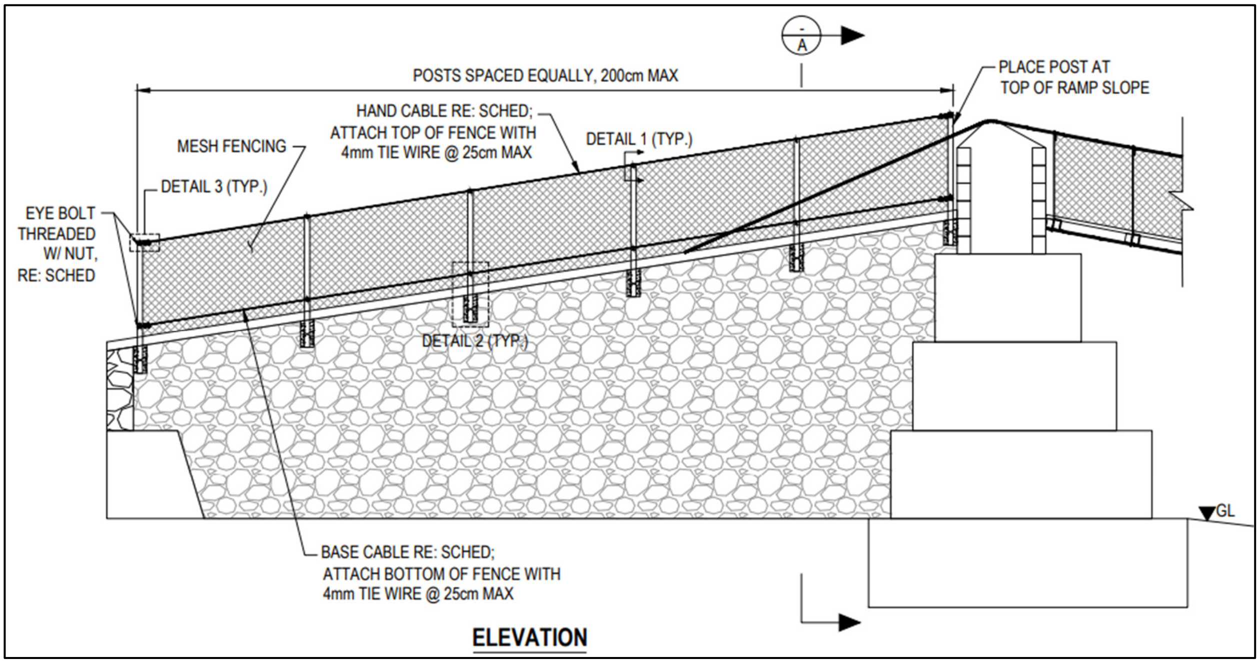


Figure 4.3.3 Fencing Detail

Figure 4.3.4 Mesh Fencing

The fencing also plays an important role in the aesthetics aspect of the bridge. Safety is the most important consideration for design, but aesthetics is also considered. The community has complete creative freedom in decorating the bridge. The fencing is often painted the colors of the community's choosing. In Figure 4.3.4, the fencing was painted blue, red, and yellow.

4.4 ABUTMENTS

The abutments, fundamental components of the bridge design, include the towers, tiers, foundations, anchors, ramp walls, concrete caps, and fill. The left and right-hand side abutments are the substructure components at either end of the bridge supporting its superstructure. The abutments retain the embankment and carry the vertical and horizontal loads from the superstructure to the foundation. The left and right-hand side abutments were both analyzed separately due to the difference in the conditions on either side of the river. Figure 4.4.1 shows the abutment of a previous bridge project, including the ramp, tier, foundation, and anchor. The left-hand side and right-hand side abutments are discussed in more detail in the following sections.

Figure 4.4.1 Bridge Abutment

4.4.1 Left-Hand Side Abutment

For the left-hand side abutment, the team tested many different setback-tier combinations due to the large height difference between the right and left side edge of banks. Due to this extreme height difference, a 3-tier abutment was chosen for the left side. A customized 3G-100A abutment was used for the left-hand side, as this was the optimal option compared to possible alternatives. The standard abutment, 3G-100A, is designed from bridges between 80 to 100 meters in span length. This abutment was customized by raising the anchor upward by 0.5 meters. This was completed in an effort to minimize excavation during construction. Furthermore, the height difference between the left and right side allowed the abutment to be raised as close as possible to the ground profile, also minimizing excavation and accounting for the height difference. Figure 4.5.1 shows the left-hand side abutment design. The callout regarding note 6 refers to the soil being replaced with gravel per the project soil remediation standards. The abutment detail sheets list the gravel layer dimensions.

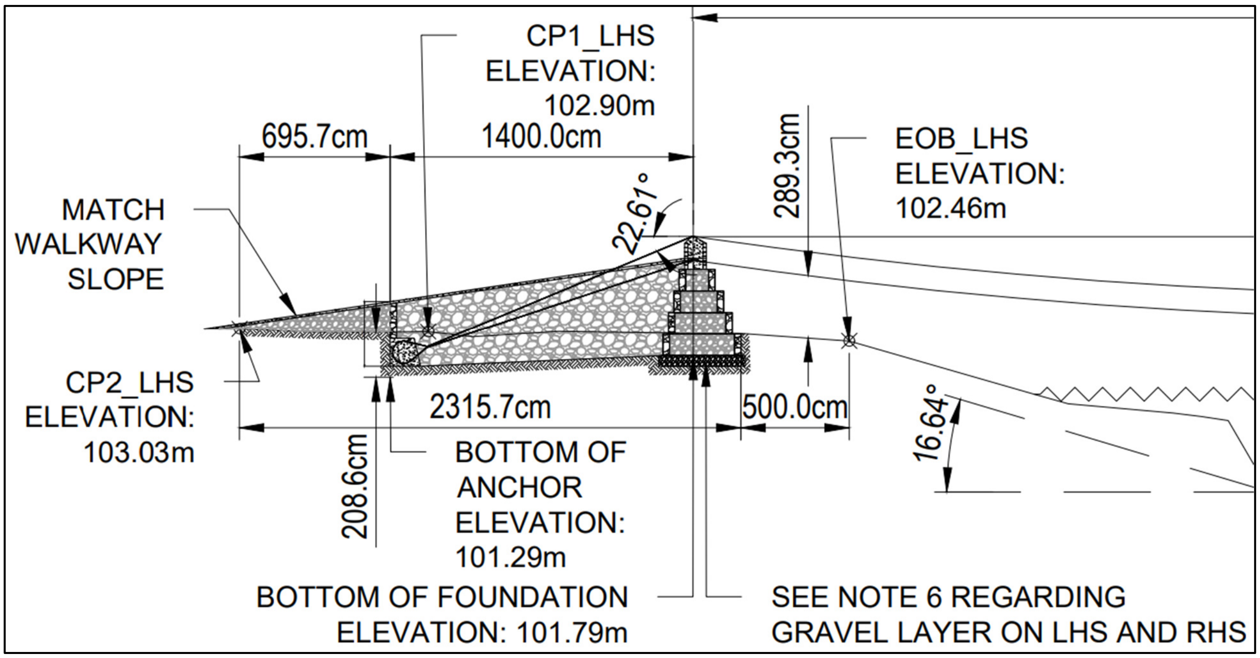


Figure 4.4.2 Left-Hand Side Abutment

The calculations regarding the left-hand side abutment are shown in Table 4.4.1. The overturning toe of the tower and foundation were analyzed based on a factor of safety calculation to ensure the bridge abutment remains stable under the previously mentioned various loading conditions. The left abutment's column eccentricity, tower moment capacity, minimum reinforcement, anchor sliding, and anchor uplift checks all passed the safety requirements. The bearing capacity check did not pass. The bearing capacity check utilized the IBC Table 1806.2. The use of this table's value was previously described in section 4.2 regarding loading assumptions. The bearing capacity issue will be mitigated in the field prior to bridge construction to ensure maximum safety of the design.

Table 4.4.1 Left Abutment Calculations

Left Abutment					
Overturning Toe of Tower Check					
Factor of Safety	=	1.67	≥	1.5	OKAY
Overturning Toe of Foundation Check					
Factor of Safety	=	3.48	≥	1.5	OKAY
Column Eccentricity Check					
Calculated Column Eccentricity	=	0.41 m			
Allowable Column Eccentricity	=	0.45 m			
Eccentricity Check		0.45 m	≥	0.41 m	OKAY
Tower Moment Capacity Check					
Factor of Safety	=	2.02	≥	1.5	OKAY
Required Strength, 1.2M_DL+1.6M_LL	=	33.77 kN-m			
Calculated Design Strength (factored)	=	56.80 kN-m			
Safety Check	=	56.80 kN-m	≥	33.77 kN-m	OKAY
Minimum Reinforcement					
Safety Check		56.80 kN-m	≥	54.75 kN-m	OKAY
Anchor Sliding Check					
Sliding Factor of Safety		2.83	≥	1.50	OKAY
Anchor Uplift Check					
Uplift Factor of Safety		1.53	≥	1.5	OKAY
Bearing Capacity Check					
Bearing Capacity Factor of Safety		0.9	≥	1	NOT OKAY

4.4.2 Right-Hand Side Abutment

As previously mentioned, the extreme height difference between the left and right-hand sides posed potential issues with delta h. This resulted in the use of a 1-tier abutment on the right side. The ground slope profile on the right side is considerably large compared to the left side. The use of a standard abutment posed many issues such as uplift, overturning, sliding, and an excessive amount of excavation. The decision to create a customized abutment was made to mitigate these issues. A 1.5-meter foundation was utilized to improve the sliding factor of safety. A soil block was included in the calculations, not shown in drawings, to aid in minimizing excavation and raising the sliding factor of safety. This decision was made according to the advice from the DEIC.

The anchor of the right abutment was also raised to minimize excavation. The length of the abutment was shortened to help with overturning, and the back of the ramp was angled to be flush with the ground profile to mitigate the failing of the ground slope profile check. These alterations can be seen in Figure 4.4.3. The angled section of the ramp was not included in any calculations (i.e. not used as overburden). This area was not included in the calculations because the community may opt to meet the ramp with the ground in a different way. This was only shown in the drawings to show accessibility onto the walkway. A bill of quantities was created to ensure the team had the necessary materials to begin construction upon arrival; the quantities presented reflect normal masonry construction with concrete cap and placed fill with mortar.

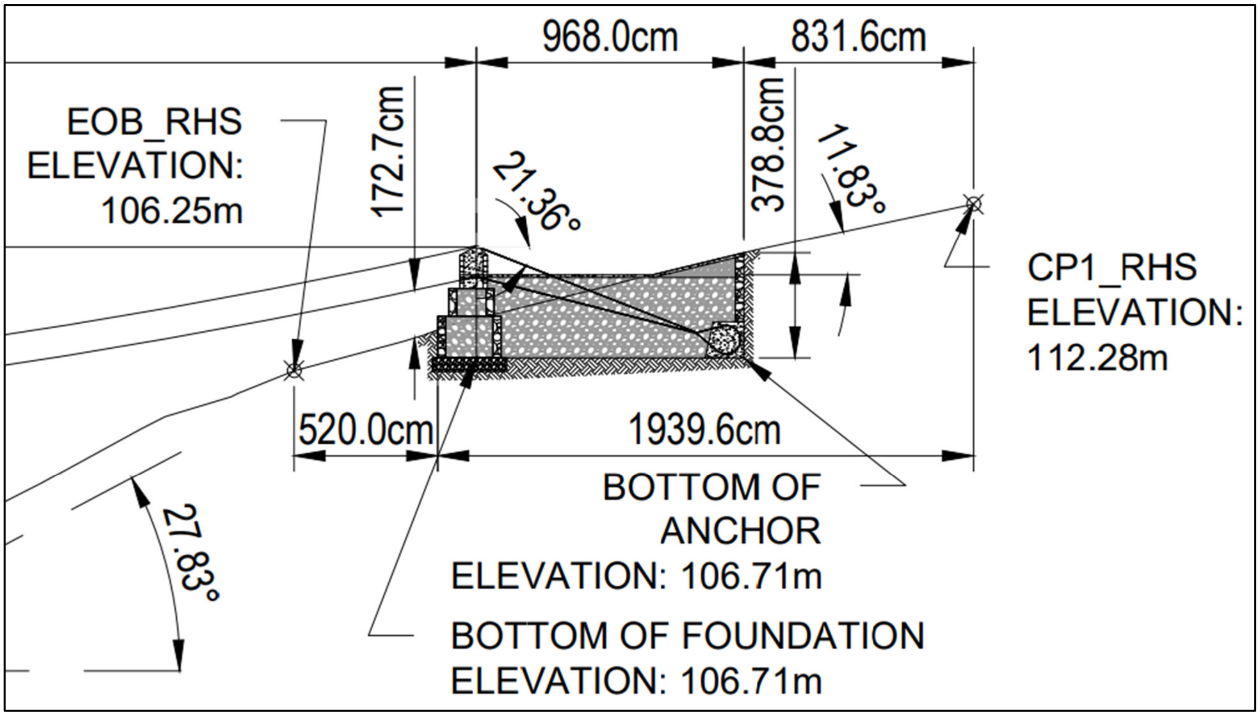


Figure 4.4.3 Right-Hand Side Abutment

Due to the right ground profile being larger than 10 degrees, but less than 20 degrees, a customized abutment was utilized to mitigate this issue. A 1-tier abutment with a 1.5-meter foundation. The anchor of the abutment was raised to minimize excavation, but this made the bottom of the abutment horizontal, which worsened the sliding factor of safety. A 1.5-meter foundation was used to improve the sliding factor of safety. A small portion of the back of the ramp was angled to the same slope as the ground profile to mitigate the ground profile slope angle of 11.83 degrees while also providing a horizontal walkway to the superstructure. The design checks of the right-side abutment are shown in Table 4.4.2.

Table 4.4.2 Right Abutment Calculations

Right Abutment					
Check Overturning Toe of Tower					
Factor of Safety	=	2.04	≥	1.5	OKAY
Check Overturning Toe of Foundation					
Factor of Safety	=	2.38	≥	1.5	OKAY
Column Eccentricity Check					
Calculated Column Eccentricity	=	0.35 m			
Allowable Column Eccentricity	=	0.45 m			
Eccentricity Check		0.45 m	≥	0.35 m	OKAY
Tower Moment Capacity Check					
Factor of Safety	=	2.11	≥	1.5	OKAY
Required Strength, 1.2M_DL+1.6M_LL	=	39.94 kN-m			
Calculated Design Strength (factored)	=	56.80 kN-m			
Safety Check	=	56.80 kN-m	≥	39.94 kN-m	OKAY
Minimum Reinforcement					
Safety Check		56.80 kN-m	≥	53.95 kN-m	OKAY
Anchor Sliding Check					
Sliding Factor of Safety		1.52	≥	1.5	OKAY
Anchor Uplift Check					
Uplift Factor of Safety		1.75	≥	1.5	OKAY
Bearing Capacity Check					
Bearing Capacity Factor of Safety		0.64	≥	1	NOT OKAY

5 ECONOMIC CONSIDERATIONS

5.1 MATERIAL ACQUISITION

Some of the main materials needed for the bridge are available locally, on or near the site. These materials include sand, rocks, and water. These materials will be used in concrete and as the masonry within the bridge. By utilizing materials available locally, the team ensures a cost-effective design. Rocks are located 3 kilometers away from the site and can be carried or hauled to the location in which they are needed. Materials that need to be purchased and brought to site include support cables, timber for framing, safety netting, concrete, gabion stones, gravel and rebar. These materials will be delivered to the site by EIA. Both sides of the river are accessible by four-wheel drive vehicle. The RHS currently is hard to reach, but EIA has confirmed it will be cleared in accordance with proper environmental and construction standards before students arrive on site. Figure 5.1.1 shows a team carrying a cable to a bridge project site; the same process will occur in May 2025 for the Khelekhele bridge project.

Figure 5.1.1 Bringing Cables to Site

6 CONCLUSIONS AND RECOMMENDATIONS

The design team gained valuable experience throughout the design process, as well as accomplishing a list of project outcomes to help people around the world. The Khelekhele bridge design continually changes to ensure the best design proceeds to be built when university teams travel to site May 2025. The following sections explain the current progress of the design, as well as the list of project outcomes resulting from the bridge design.

6.1 CURRENT PROGRESS

Throughout the design process, the team learned about the process of optimizing the bridge design through the use of alternative designs and mentorship from a professional engineer. It is important to continually improve the design based upon comments from the professional engineer with Bridge Corporation, as well as the Engineers in Action team. The review call process gave the design team valuable feedback to improve the design and make it optimal for construction.

The Khelekhele suspended bridge design presented in the second review call was detailed in this report. The design was additionally revised based upon comments received from

Engineers in Action, and the design will continue to be optimized during construction. Especially with the soil bearing issues, the design may need to be revised if the soil bearing does not meet the requirements when the team additionally tests the soil upon arriving on site.

Members of the Engineers in Action student chapters have been preparing to travel by creating excavation drawings, a construction schedule, quality control checks, and a bill of quantities material estimate. Hazard mitigation and travel logistic plans have also been prepared to ensure the safety of everyone involved in the construction. The travel team has also spent time learning about the Eswatini culture to better understand and appreciate what they will be immersed in this summer. The bridge project provided the design team with valuable experience and offered the travel team a meaningful opportunity. Additionally, it is expected to have a broad range of impacts on the Eswatini community, as detailed below.

6.2 Project Outcomes

The construction of the Khelekhele bridge will provide a safe year-round river crossing serving over 4,250 community members living in three Eswatini communities. The design deliverables include a full plan set and a list of calculations, provided in Appendix B. Through the design process, the team continually improved the alternative designs to reach the finalized design, with the mentorship provided by the Bridge Corporation professional engineer, acting as the design engineer in charge of the project. The design team learned valuable aspects of problem solving through the review call process. Additionally, the team was able to mitigate the poor site conditions by implementing design changes to accommodate the specifics of the bridge location. Finally, the team gained insight into the real-world significance of the project, deepening their understanding of how the Khelekhele bridge will affect thousands of lives. The bridge's global impact will not only benefit the Eswatini community but also provide travelers and members of the design team with a once-in-a-lifetime experience.

REFERENCES

[1] "Engineers in Action Bridge Program." *Engineers in Action Bridge Program,* https://www.eiabridges.org/.

[2] "EIA Education." *EIA, https://apps.eiaeducation.org/learner-dashboard/.*

[3] "Eswatini-3_Kelekele (Nsingweni_ Maphalaleni)_CU Boulder, Penn State, USI." *Google Drive*, Google, https://drive.google.com/drive/folders/1H8HukT89byMNIvIx03wIvUkIUiVnBnEQ.

[4] "EIA Bridge Binder, Volume 2 – Design.pdf." *Google Drive*, Google, https://drive.google.com/file/d/13Ay1_dySHcpgYuF9ZtxjIwyA1vTeiHBB/view?usp=drive_link.

[5] "Climate and Average Weather Year Round in Manzini." *Weather Spark,* https://weatherspark.com/y/96804/Average-Weather-in-Manzini-Swaziland-Year-Round.

APPENDIX A: ABET OUTCOMES

ABET Outcome 2, Design Factor Considerations

ABET Outcome 2 states "*An ability to apply engineering design to produce solutions that meet specified needs with consideration of public health safety, and welfare, as well as global, cultural, social, environmental, and economic factors.*"

ABET also requires that design projects reference appropriate professional standards, such as IEEE, ATSM, etc.

Table A.1 indicates the corresponding page number on which each design factor is addressed.

Table A.1: Design Factors Considered

Design Factor	Page Number
Public health, safety, and welfare	3, 10
Global	28
Cultural	2
Social	11
Environmental	11, 18
Economic	27
Ethical & Professional	4, 11, 28
Reference for Standards	3, 17

APPENDIX B: PLAN SET & CALCULATIONS

FULL PLAN SET (PAGES B.1 – B.10)

CALCULATIONS SET (PAGES B.11 – B.32)

KHELEKHELE SUSPENDED BRIDGE

GPS COORDINATES	26°17'47.3"S, 31°24'22.5"E
COUNTRY	ESWATINI
REGION	HHOHHO & MANZINI
INKHUNDLA	MAPHALALENI & MKHIWENI
COMMUNITY	MBELEBELENI, NSUKA, NKILIJI
SPAN	93.5 METERS

GENERAL NOTES:

BRIDGE CONSTRUCTION SHALL BE EXECUTED BY THE MEANS AND METHODS STATED IN THE ENGINEERS IN ACTION, 2022 BRIDGE BINDER VOLUME 2.

CONCRETE:
PORTLAND CEMENT (ASTM C150, TYPE I OR TYPE II) SHALL BE USED. CEMENT MUST BE USED WITHIN 60 DAYS OF PURCHASE.
WATER SHALL BE CLEAN, CLEAR, AND FREE OF HARMFUL MATERIAL.
COARSE AGGREGATE SHALL BE COMPRISED OF GRAVEL (CRUSHED LIMESTONE, GRANITE, OR GRAVEL), NO GREATER THAN 2.5 cm IN DIAMETER. MATERIAL SHALL BE CLEAN AND FREE OF DEBRIS.
FINE AGGREGATE SHALL BE CLEAN, DRY SAND GRADED WITH A 4mm SIEVE, BEFORE MIXING WITH CEMENT.

REINFORCEMENT:
ALL REINFORCEMENT SHALL BE MINIMUM GRADE 280 (GRADE 40) WITH A YIELD STRENGTH OF 275 MPa (40 ksi).
RIBBED STEEL SHALL BE USED FOR ALL REINFORCING BARS INCLUDING SUSPENDERS.
ALL REINFORCEMENT SHALL BE SUPPORTED BY CONCRETE BLOCKS OR STEEL CHAIRS TO AVOID CONTACT WITH GROUND OR FORMS.

MASONRY:
BLOCKS SHALL BE FREE OF CRACKS AND CHIPS. THERE SHALL BE NO DEFORMATIONS. USED BLOCK IS NOT PERMITTED.
MASONRY UNITS SHALL BE WET BEFORE APPLYING MORTAR.
MAINTAIN A CONSISTENT JOINT THICKNESS OF 15mm +/- 5mm. JOINTS BETWEEN BLOCKS SHALL BE COMPLETELY FILLED.
STAGGER BLOCKS IN RUNNING BOND PATTERN.

CABLE:
CABLE IN PERMANENT CONTACT WITH THE GROUND SHALL BE COVERED WITH PLASTIC PIPE AND FILLED WITH GROUT OR COATED WITH TAR.
CLAMPS SHALL BE DROP FORGED AND NOT MALLEABLE.

TIMBER:
TIMBER SHALL BE FREE OF KNOTS, HOLES, AND SPLITS.
WOOD SCREWS AND NAILS SHALL BE GALVANIZED.

DESIGN DATA:

ENGINEERS IN ACTION, 2022 BRIDGE BINDER VOLUME 2.

DESIGN LOADS:

DEAD LOAD	=	1.04	kN/m^2
LIVE LOAD$_{PRIMARY}$	=	3.02	kN/m^2
LIVE LOAD$_{SECONDARY}$	=	4.07	kN/m^2
WIND LOAD	=	0.60	kN/m

MATERIAL PROPERTIES:

CONCRETE	$f'c$	=	10	MPa	(1450	psi)
REINFORCING	F_y	=	275	MPa	(40	ksi)
TIMBER	F_b	=	3.96	MPa	(574	psi)
TIMBER	F_v	=	1.44	MPa	(210	psi)
SOIL*	q_a	=	100	kPa	(15	psi)
FRICTION ANGLE	ϕ	=	30	degrees		
CABLE	P_n	=	883	kN	(198	kips)
CROSSBEAM STEEL	F_y	=	248	MPa	(36	ksi)

*FROM IBC TABLE 1806.2

UNIT WEIGHTS:

STEEL	=	7850 kg/m^3	(490 lb/ft^3)
CONCRETE	=	2400 kg/m^3	(150 lb/ft^3)
TIMBER	=	600 kg/m^3	(38 lb/ft^3)
BROKEN ROCK	=	2000 kg/m^3	(125 lb/ft^3)
MASONRY	=	2100 kg/m^3	(131 lb/ft^3)
SOIL	=	1800 kg/m^3	(112 lb/ft^3)

FACTOR OF SAFETY FOR SLIDING AND UPLIFT = 1.5
FACTOR OF SAFETY FOR CABLE CAPACITY = 3.0

DECK SHALL CLEAR FREEBOARD ENVELOPE WITH A MINIMUM FREEBOARD OF 3.00 METERS.

PROVIDE 7 DROP FORGED CABLE CLAMPS SPACED AT 15 cm O.C.
MAX TORQUE TO 338 ft-lb. PER HANDRAIL CABLE AT EACH ANCHOR.
PROVIDE 7 DROP FORGED CABLE CLAMPS SPACED AT 15 cm O.C.
MAX TORQUE TO 338 ft-lb. FOR THE WALKWAY CABLE AT EACH ANCHOR.

INDEX:

1 TITLE
2 LAYOUT
3 [illegible] LEFT ABUTMENT DETAIL (*NON-STANDARD)
4 [illegible] RIGHT ABUTMENT DETAIL (*NON-STANDARD)
5 A2 ANCHOR DETAIL
6 T1 TOWER DETAIL
7 W3E WALKWAY DETAIL STEEL CROSSBEAM
8 C3E STEEL CROSSBEAM DETAIL
9 F2E FENCING DETAIL
10 D4 DRAINAGE DETAIL

EIA MICROPROJECTS KINGDOM OF ESWATINI Ministry of Economic Planning & Development

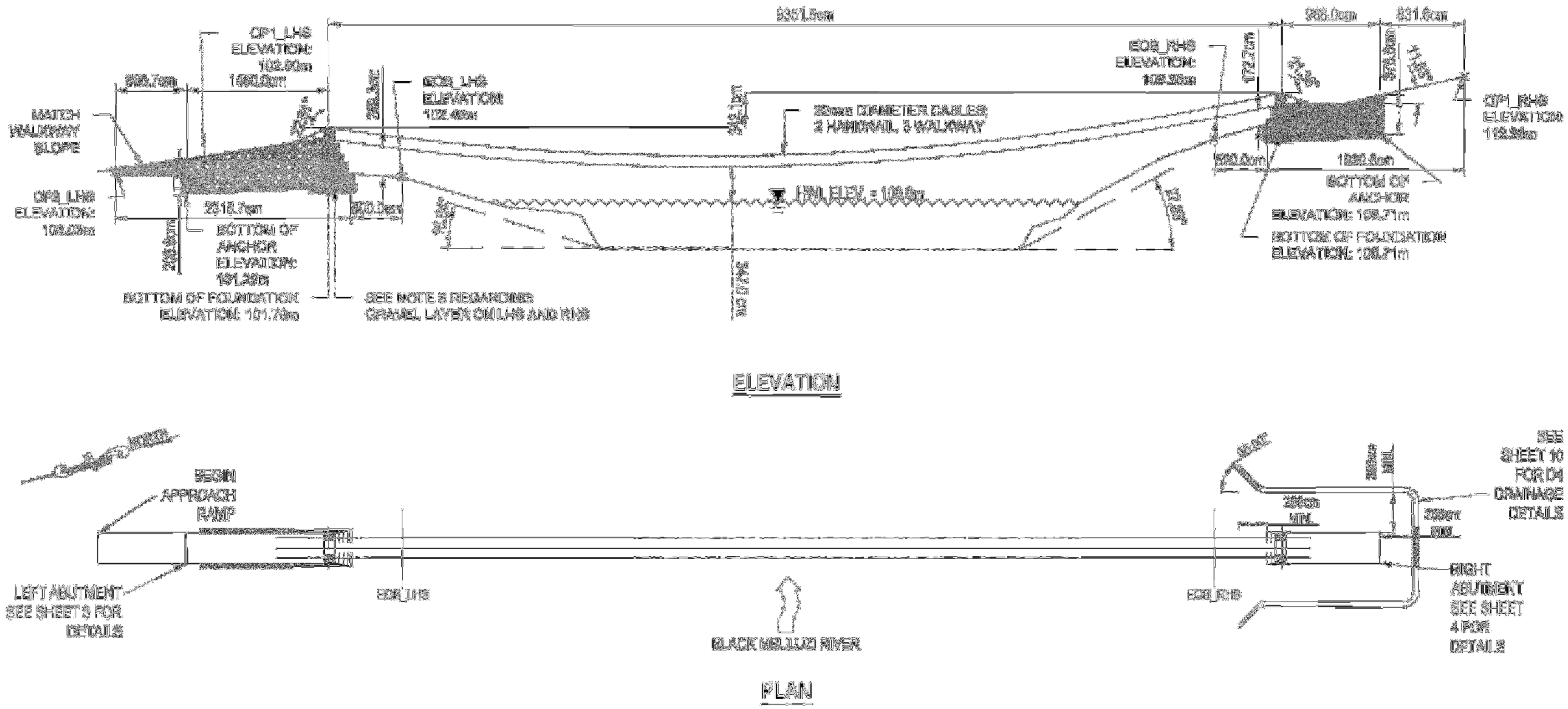
REV.	DESCRIPTION	DATE	ISSUED BY
A	RC1 ISSUED FOR REVIEW	02/26/2025	MMM, AGG
B	RC2 ISSUED FOR REVIEW	03/23/2025	MMM, AGG
C	RC2 REVISED	04/13/2025	MMM, AGG

COUNTRY: ESWATINI
PROJECT: KHELEKHELE BRIDGE
REGION: HHOHHO & MANZINI
PROGRAM MANAGER: ANA JOVANOVIC
INKHUNDLA: MAPHALALENI & MKHIWENI

ENGINEERING RECORD
DATE: 03/23/2025
DRAWN BY: MMM, AGG
CHECKED BY: JAE
APPROVED BY: ARB
REVIEWED BY: IAE

KHELEKHELE BRIDGE SUSPENDED BRIDGE TITLE

PROJECT NUMBER	DRAWING NUMBER
BSZ-BP116	1



NOTES:

1. DESIGN SAG: 4.26m (4.56%)
2. HOISTING SAG: 3.74m (4.00%)
   f_{LEFT} = 2.23m f_{RIGHT} = 6.66m
3. CONSTRUCTION SAG: 2.81m (3.00%)
   f_{LEFT} = 1.38m f_{RIGHT} = 4.79m
4. LIVE LOAD SAG: 4.77m (5.71%)
5. SURVEY INFORMATION PROVIDED BY SIBO, NJABULO & SIMBO ON 31 OCTOBER 2024.
6. SOIL TO BE REPLACED WITH GRAVEL PER THE PROJECT SOIL REMEDIATION STANDARDS. SEE ABUTMENT DETAIL SHEET FOR GRAVEL LAYER DIMENSIONS.

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REV.	DESCRIPTION	DATE	ISSUED BY
A	RC1 ISSUED FOR REVIEW	02/26/2025	MMM, AGG
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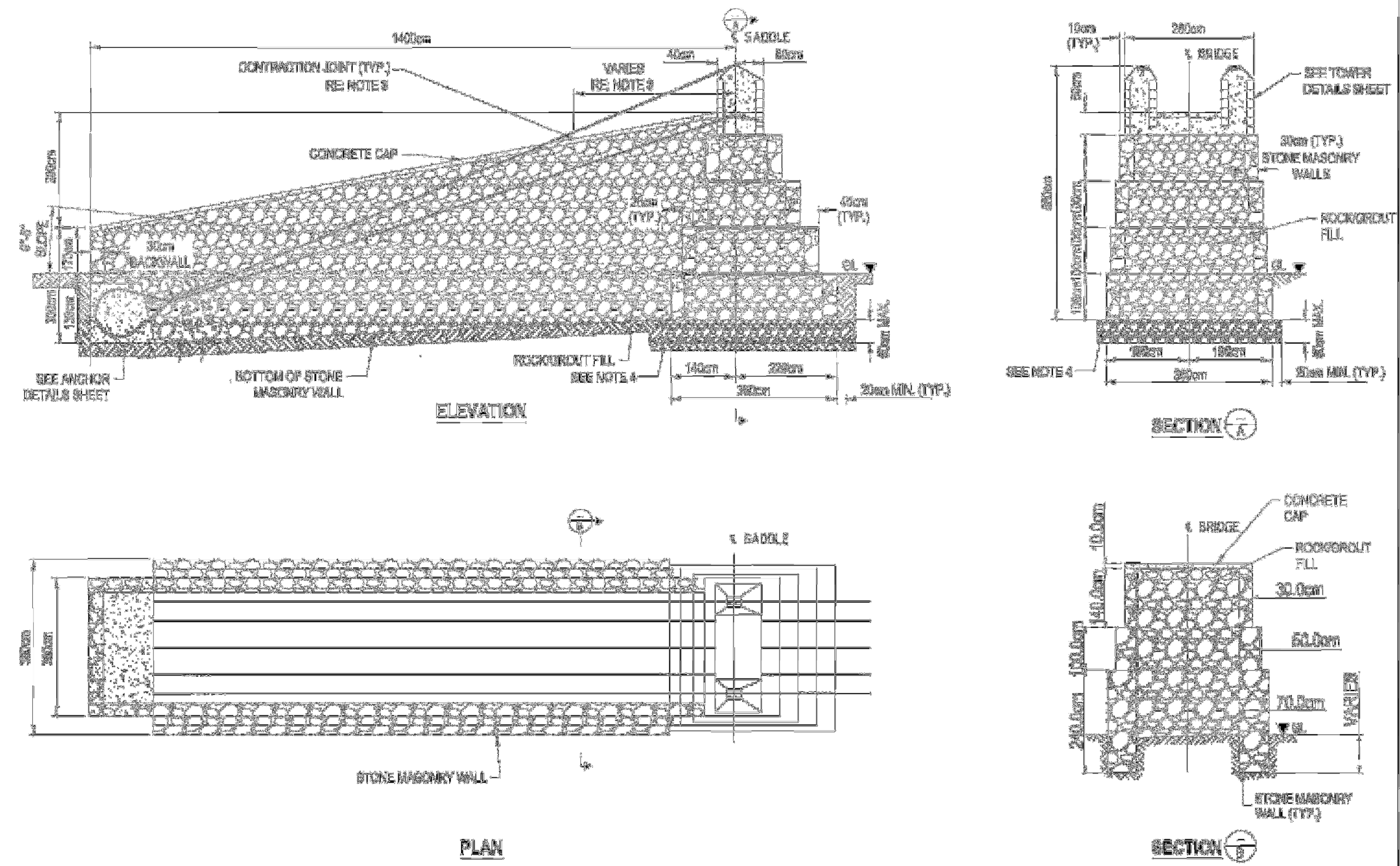
PROJECT: KHELEKHELE BRIDGE
REGION: HHOHHO & MANZINI
PROGRAM MANAGER: ANA JOVANOVIC
INKHUNDLA: MAPHALALENI & MKHIWENI

ENGINEERING RECORD
DATE: 03/23/2025
DRAWN BY: MMM, AGG
CHECKED BY: JAE
APPROVED BY: ARB
REVIEWED BY: IAE

KHELEKHELE SUSPENDED BRIDGE LAYOUT

PROJECT NUMBER	DRAWING NUMBER
BSZ-BP116	2



NOTES:
1. [illegible]
2. [illegible]
3. [illegible] CONTRACTION JOINT OF [illegible] DEPTH [illegible]
JOINTS SHOULD BE SPACED EVENLY EVERY 3-4m ALONG APPROACH RAMP.
4. SOIL TO BE REPLACED WITH GRAVEL PER THE PROJECT SOIL REMEDIATION STANDARDS.

EIA
KINGDOM OF ESWATINI MICROPROJECTS Ministry of Economic Planning & Development

REV.	DESCRIPTION	DATE	ISSUED BY
A	RC1 ISSUED FOR REVIEW	02/26/2025	MMM, AGG
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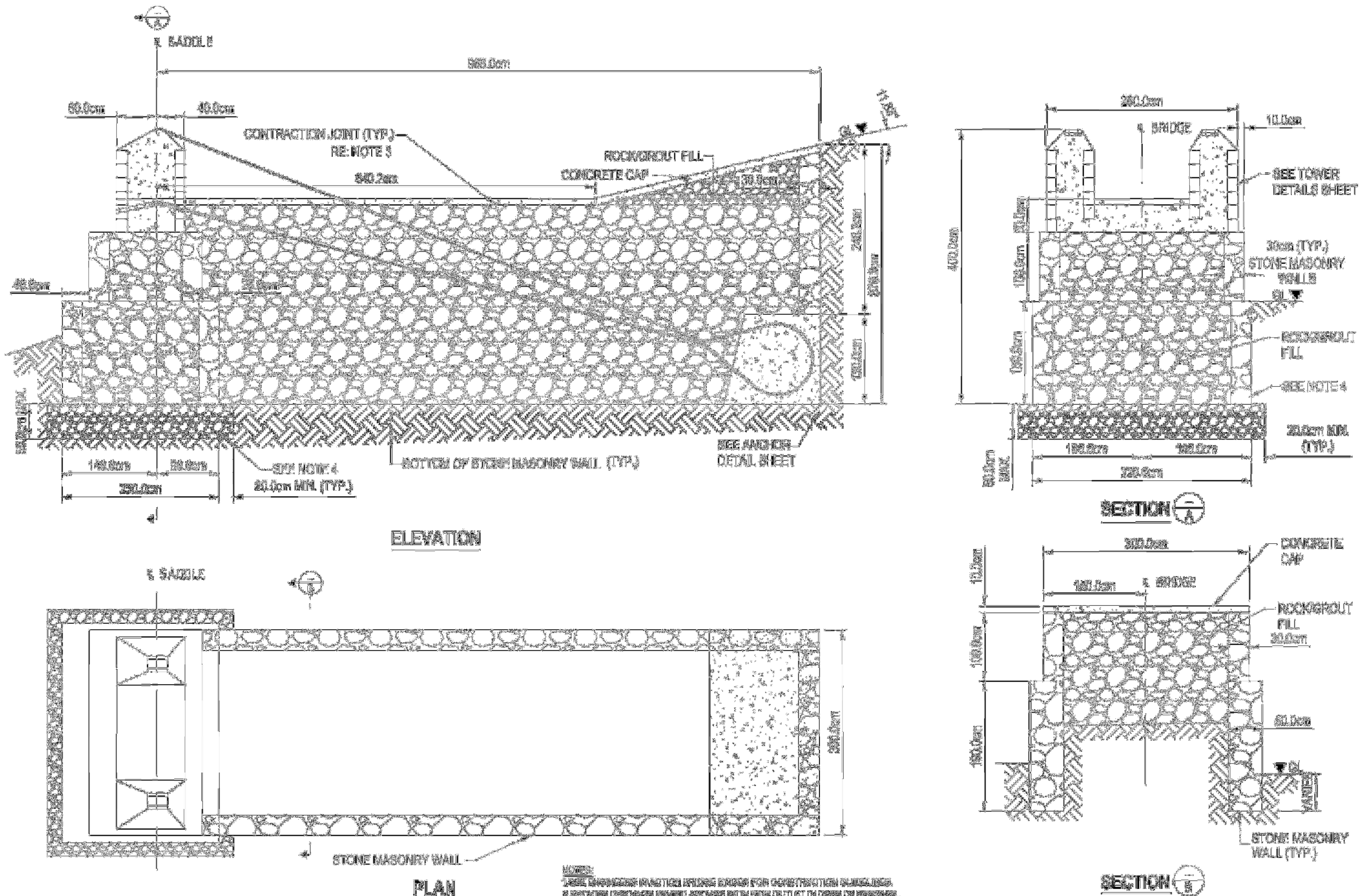
C	RC2 REVISED	04/13/2025	MMM, AGG

COUNTRY: ESWATINI
PROJECT: KHELEKHELE BRIDGE
REGION: HHOHHO & MANZINI
PROGRAM MANAGER: ANA JOVANOVIC
INKHUNDLA: MAPHALALENI & MKHIWENI

ENGINEERING RECORD
DATE: 03/23/2025
DRAWN BY: MMM, AGG
CHECKED BY: JAE
APPROVED BY: ARB
REVIEWED BY: IAE

KHELEKHELE
LEFT ABUTMENT DETAIL
3G-100A

PROJECT NUMBER	DRAWING NUMBER
BSZ-BP116	3



NOTES:
1. SEE ENGINEERS IN ACTION BRIDGE BUILDER FOR CONSTRUCTION GUIDELINES.
2. PROVIDE DRAINAGE BEHIND ANCHORS WITH WIDE OUTLET IN CASE OF SEEPAGE.
3. PROVIDE CONTRACTION JOINT OF 2.5cm DEPTH UNDER MAXIMUM 10cm CONCRETE SLABS. JOINTS SHOULD BE SPACED EVENLY EVERY 3-4m ALONG APPROACH RAMP.
4. SOIL TO BE REPLACED WITH GRAVEL PER THE PROJECT SOIL REMEDIATION STANDARDS.

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C	RC2 REVISED	04/13/2025	MMM, AGG

DISCLAIMER:
THESE DRAWINGS ARE PRODUCED FOR THE STATED BRIDGE PROJECT. ANY CONDITIONS THAT REQUIRE CHANGES FROM THE PLANS MUST BE COMMUNICATED TO THE ENGINEER FOR APPROVAL PRIOR TO CONSTRUCTION. ANYONE USING THIS PLAN SHOULD VERIFY THE CALCULATIONS ACCORDING TO SITE SPECIFIC CONDITIONS AND LOCAL STANDARDS.

USE OF THESE PLANS FOR ANY OTHER PURPOSE SHALL NOT BE ATTRIBUTABLE TO ENGINEERS IN ACTION AND USERS AGREE TO HOLD ENGINEERS IN ACTION HARMLESS TO ANY AND ALL LIABILITY.

COUNTRY: ESWATINI
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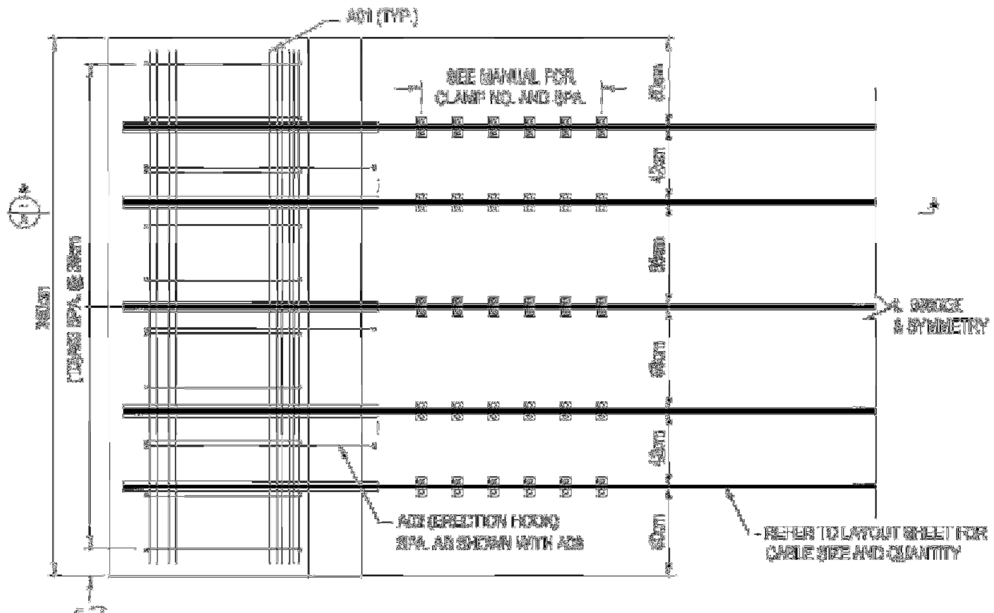
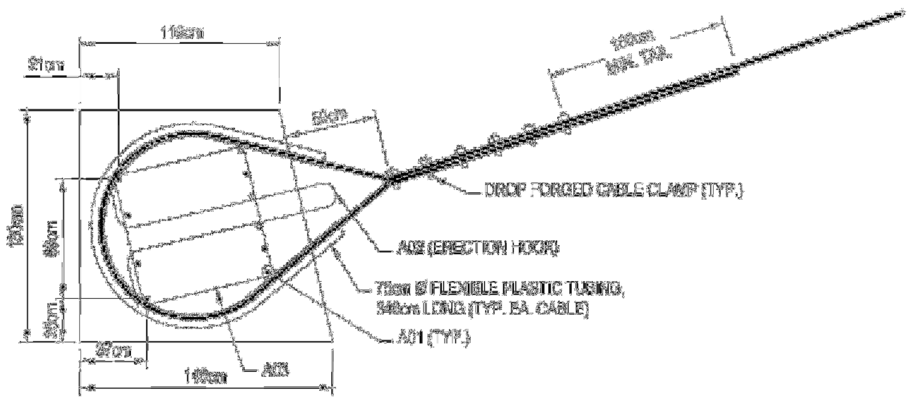
REVIEWED BY: IAE

KHELEKHELE
RIGHT ABUTMENT DETAIL
NON-STANDARD

PROJECT NUMBER	DRAWING NUMBER
BSZ-BP116	4



PLAN



SECTION A

BILL OF MATERIALS

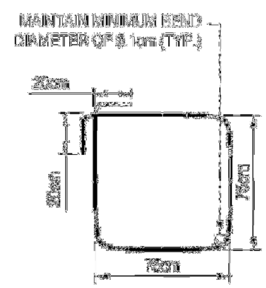
REINFORCING BARS (PER ANCHOR)

NAME	BAR SIZE (mm)	LENGTH (cm)	QUANTITY	LENGTH (m)
A01	16 (#5)	295	10	29.5
A02	16 (#5)	300	3	9.0
A03	13 (#4)	325	10	32.5

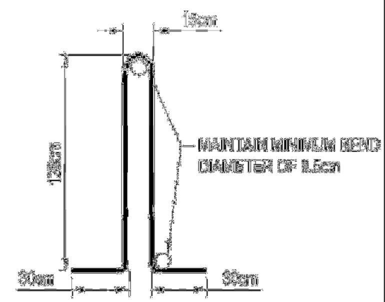
ITEM	QUANTITY
CONCRETE	4.58m^3
FLEXIBLE PLASTIC TUBING x 340cm	VARIES

CLAMP SPACING AND TORQUE REQUIREMENTS

CABLE SIZE		NUMBER OF CLAMPS	SPACING		TORQUE REQUIREMENTS (ft-lb)
(in.)	(mm)		(in.)	(cm)	
1	26	6	6	14	225
1-1/8	29	6	6	16	225
1-1/4	32	7	6	16	360
1-3/8	35	7	6	16	360



A03



A02

NOTES:
1.SEE ENGINEERS IN ACTION BRIDGE BINDER FOR CONSTRUCTION GUIDELINES.
2.7.5cm CLEAR COVER SHALL BE PROVIDED FOR ALL REINFORCING AND PLASTIC TUBING.
3.ERECTION HOOK AND TUBING OPTIONAL FOR ANCHOR DETERMINED TO BE NON-ADJUSTABLE OR FIXED.
4.REINFORCING BAR DIMENSIONS ARE TAKEN TO OUTSIDE OF BAR.
5.IF USING GREATER THAN 3 WALKWAY CABLES, FOLLOW CABLE GEOMETRY ON TOWER DETAILS.

EIA

MICROPROJECTS

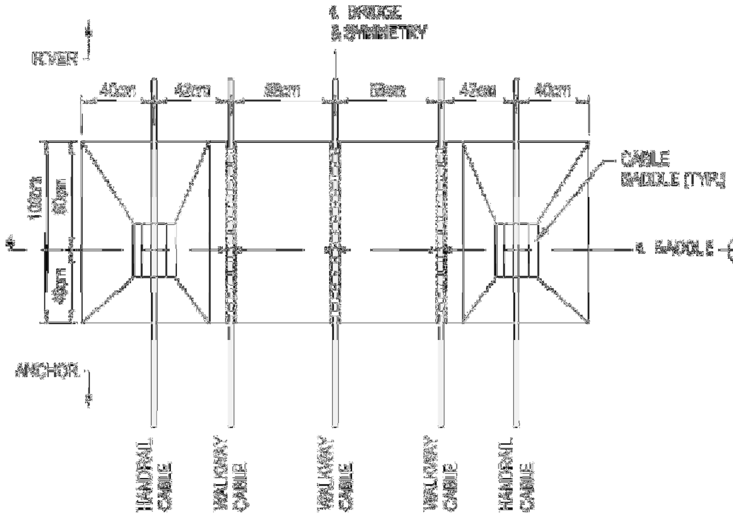
REV.	DESCRIPTION	DATE	ISSUED BY
A	RC1 ISSUED FOR REVIEW	02/26/2025	MMM, AGG
B	RC2 ISSUED FOR REVIEW	03/23/2025	MMM, AGG
C	RC2 REVISED	04/13/2025	MMM, AGG

COUNTRY: ESWATINI
PROJECT: KHELEKHELE BRIDGE
REGION: HHOHHO & MANZINI
PROGRAM MANAGER: ANA JOVANOVIC
INKHUNDLA: MAPHALALENI & MKHIWENI

ENGINEERING RECORD
DATE: 03/23/2025
DRAWN BY: MMM, AGG
CHECKED BY: JAE
APPROVED BY: ARB
REVIEWED BY: IAE

KHELEKHELE A2 ANCHOR DETAIL

PROJECT NUMBER	DRAWING NUMBER
BSZ-BP116	5



PLAN

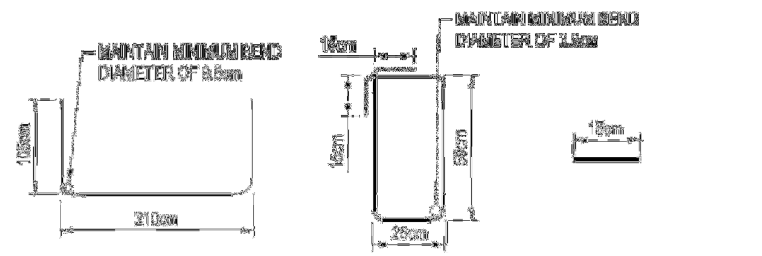
NOTES:
1. SEE ENGINEERS IN ACTION BRIDGE BINDER FOR CONSTRUCTION GUIDELINES.
2. 7.5cm CLEAR COVER SHALL BE PROVIDED FOR ALL REINFORCING AND PLASTIC TUBING.
3. CONSTRUCTION STAGES:
   STAGE 1 - BASE LEVEL MASONRY PERIMETER FILLED WITH CONCRETE.
   STAGE 2 - CONSTRUCT TOWERS IN LIFTS OF 20-40cm.
   STAGE 3 - CAST WALKWAY HUMP AND STAB T03 CABLE GUIDE BARS.
   STAGE 4 - CAST WALKWAY TOPPING SLAB OVER SLEEVED CABLES.

BILL OF MATERIALS

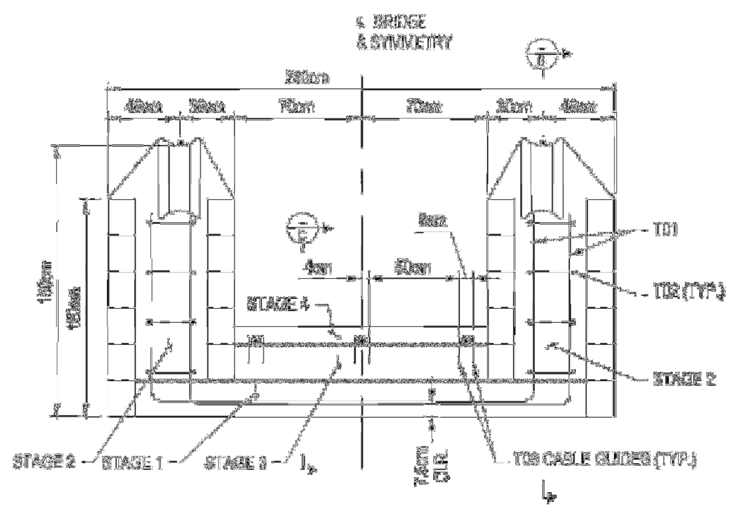
REINFORCING BARS (PER TOWER)

NAME	BAR SIZE (mm)	LENGTH (cm)	QUANTITY	LENGTH (m)
T01	13 (#4)	413	4	16.5
T02	10 (#3)	170	8	13.6
T03	10 (#3)	15	6	0.9

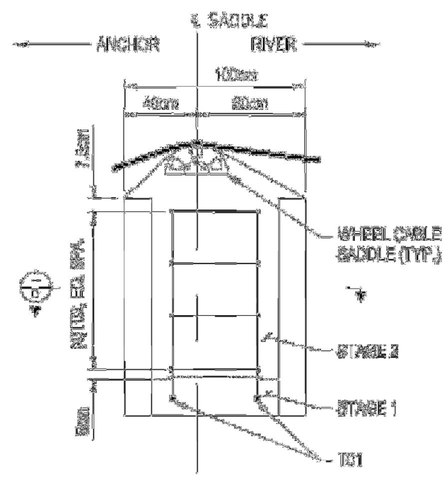
ITEM	QUANTITY
CONCRETE	1.03m³
FLEXIBLE PLASTIC TUBING x 110cm	3



T01 T02 T03



SECTION A



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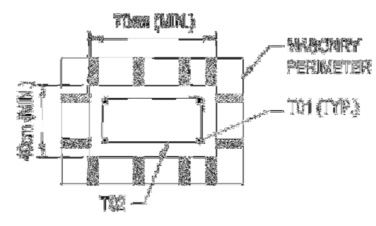
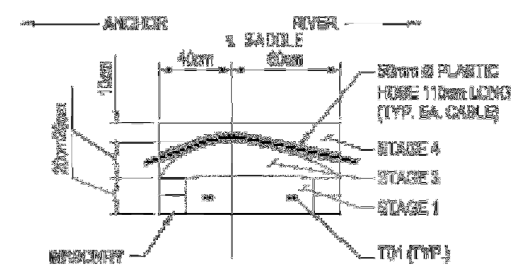


SECTION D



SECTION C

REV.	DESCRIPTION	DATE	ISSUED BY
A	RC1 ISSUED FOR REVIEW	02/26/2025	MMM, AGG
B	RC2 ISSUED FOR REVIEW	03/23/2025	MMM, AGG
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PROJECT: KHELEKHELE BRIDGE
REGION: HHOHHO & MANZINI
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INKHUNDLA: MAPHALALENI & MKHIWENI

ENGINEERING RECORD
DATE: 03/23/2025
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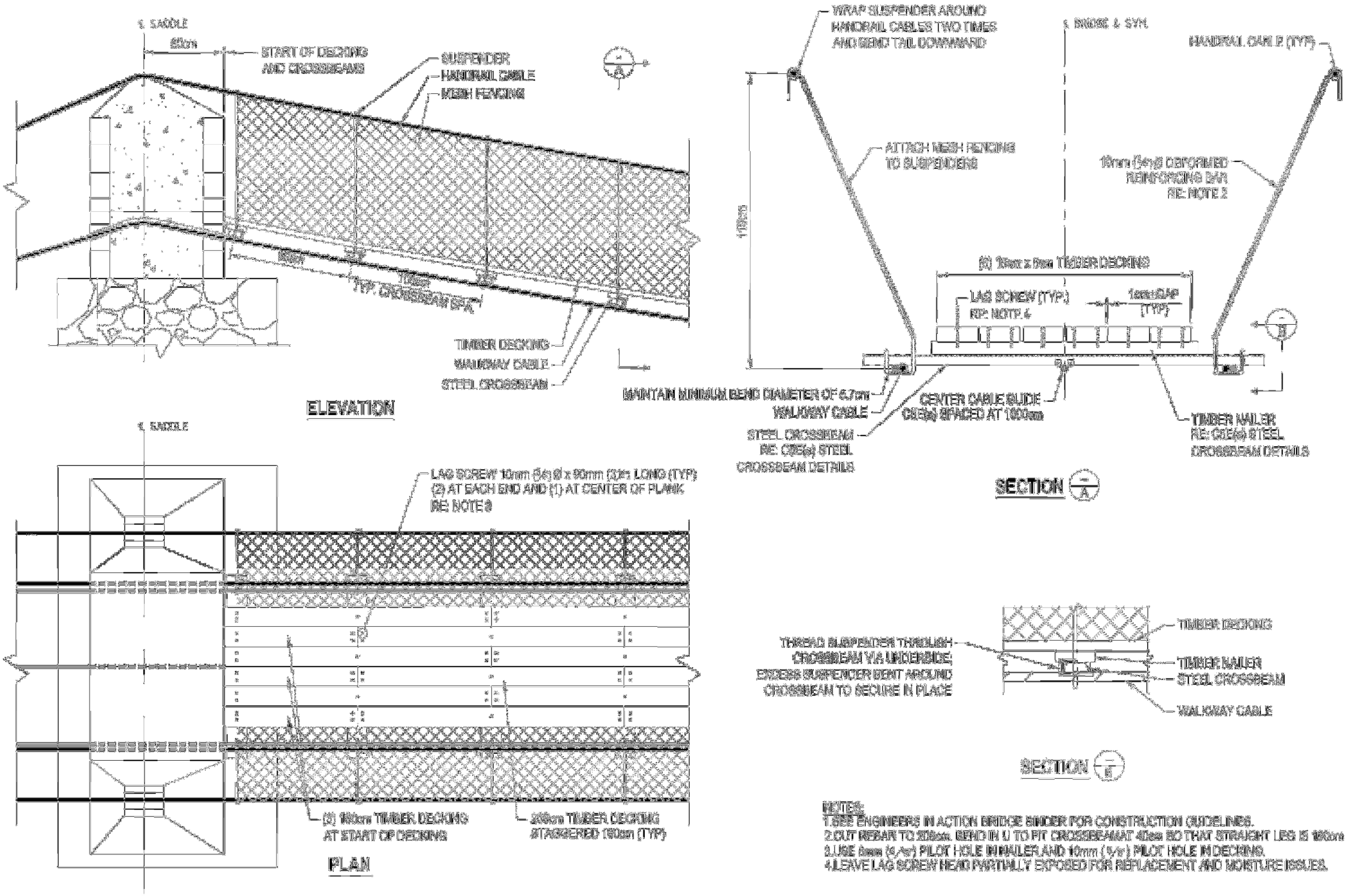
KHELEKHELE T1 TOWER DETAIL

PROJECT NUMBER	DRAWING NUMBER
BSZ-BP116	6



NOTES:
1. SEE ENGINEERS IN ACTION BRIDGE BINDER FOR CONSTRUCTION GUIDELINES.
2. CUT REBAR TO 208cm. BEND IN U TO FIT CROSSBEAM AT 40cm SO THAT STRAIGHT LEG IS 180cm
3. USE 8mm (5/16") PILOT HOLE IN NAILER AND 10mm (3/8") PILOT HOLE IN DECKING.
4. LEAVE LAG SCREW HEAD PARTIALLY EXPOSED FOR REPLACEMENT AND MOISTURE ISSUES.

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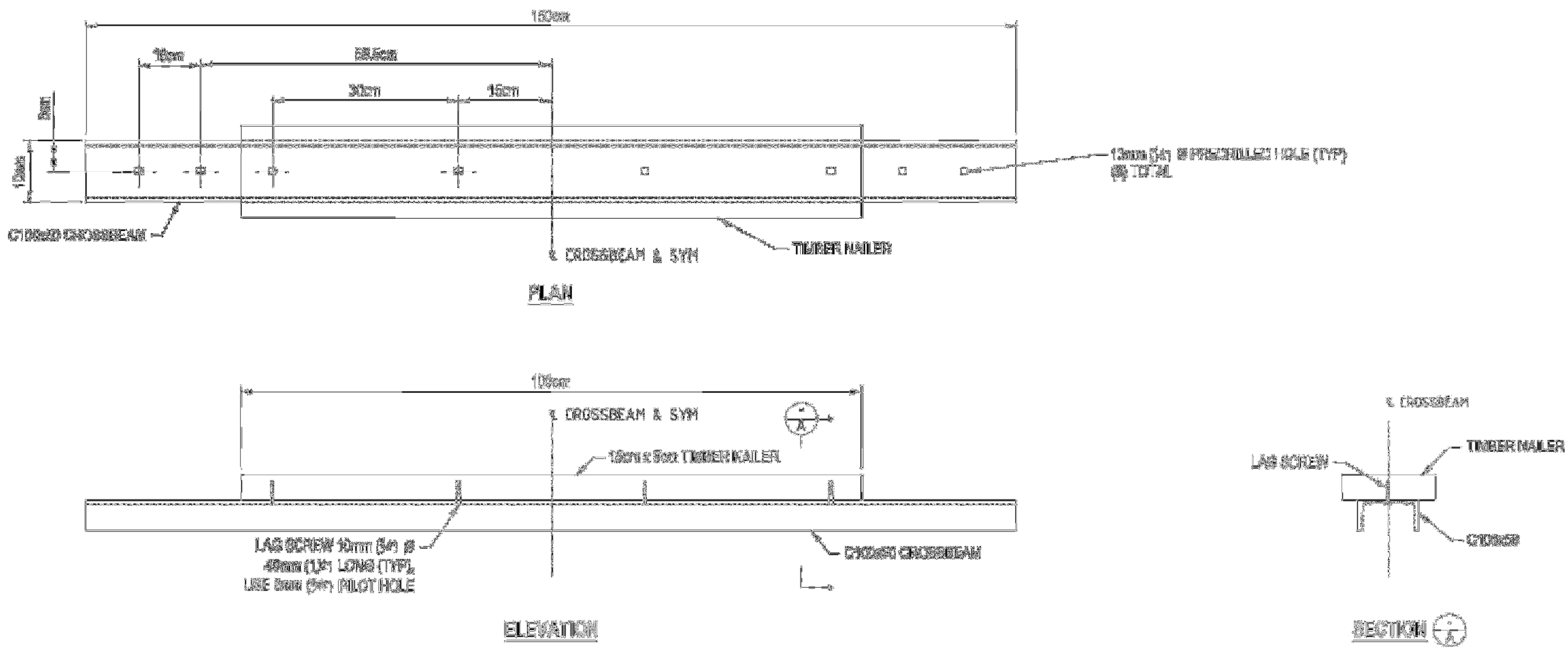
KHELEKHELE
W3E WALKWAY DETAIL
STEEL CROSSBEAM

PROJECT NUMBER	DRAWING NUMBER
BSZ-BP116	7



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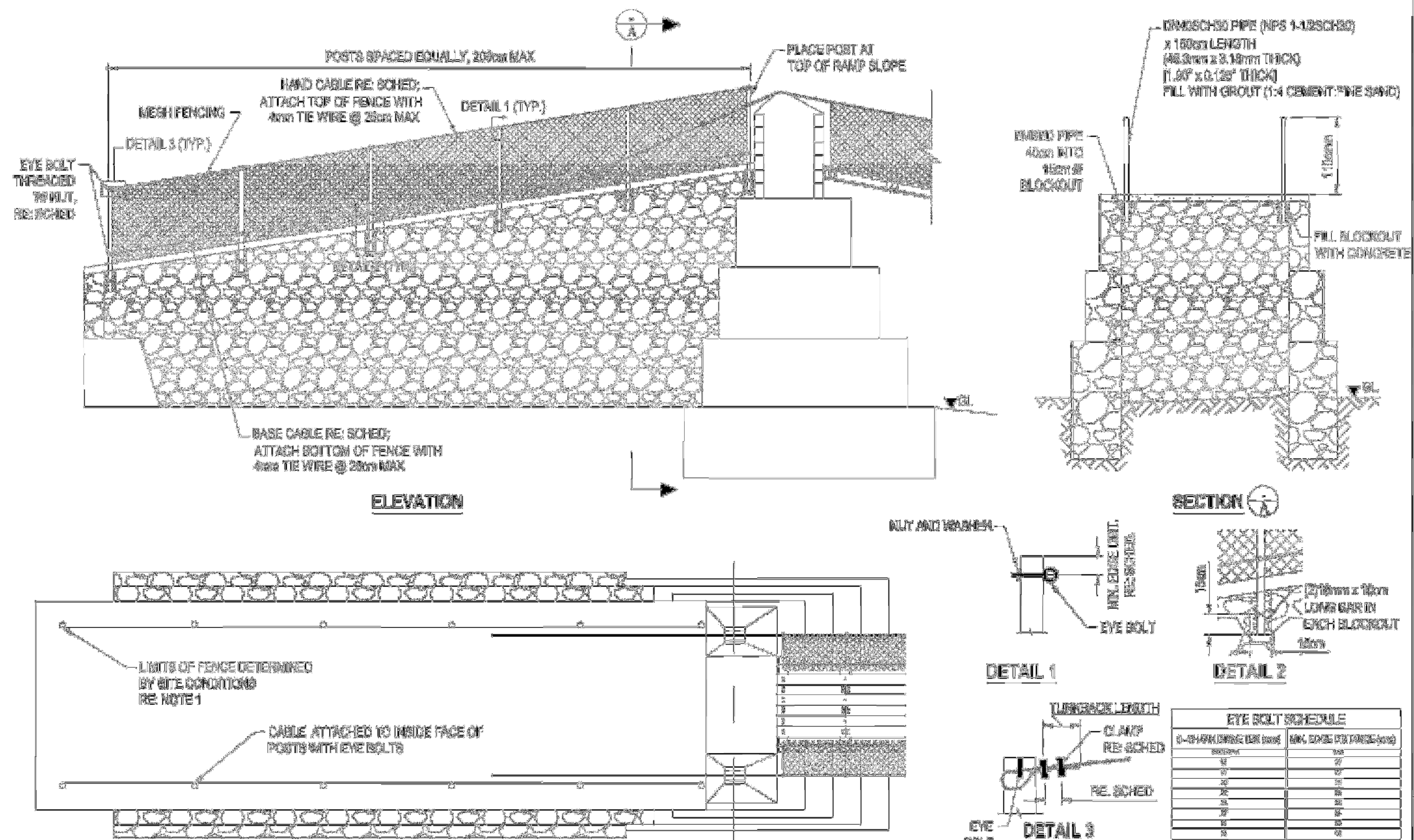
KHELEKHELE
C5E STEEL CROSSBEAM DETAIL

PROJECT NUMBER	DRAWING NUMBER
BSZ-BP116	8



EYE BOLT SCHEDULE	
D-SHANK DIAMETER (mm)	MIN. EDGE DISTANCE (mm)
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]

CABLE & CLAMP SCHEDULE				
CABLE & CLAMP SIZE (mm)	CABLE TURNBACK (mm)	CABLE TENSILE STRENGTH (kN)	CLAMP TORQUE (ft-lbs)	MIN CLAMP SPACING (mm)
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
13	292	101	65	97

NOTES:
1. PROVIDE APPROACH RAMP FENCE WHEN THE DISTANCE FROM GRADE TO THE TOP OF THE RAMP CAP EXCEEDS 1.8m (6ft). FENCE IS NOT REQUIRED WHERE THE VERTICAL DISTANCE IS LESS THAN 1.8m, SUBJECT TO THE PREFERENCE OF COUNTRY MANAGER AND/OR COMMUNITY.
2. PAINT EYE BOLT HARDWARE WITH SIMILAR, ANTICORROSIVE PAINT.
3. SEE EIA BRIDGE BINDER FOR CONSTRUCTION GUIDELINES.

EIA — MICROPROJECTS, KINGDOM OF ESWATINI, Ministry of Economic Planning & Development

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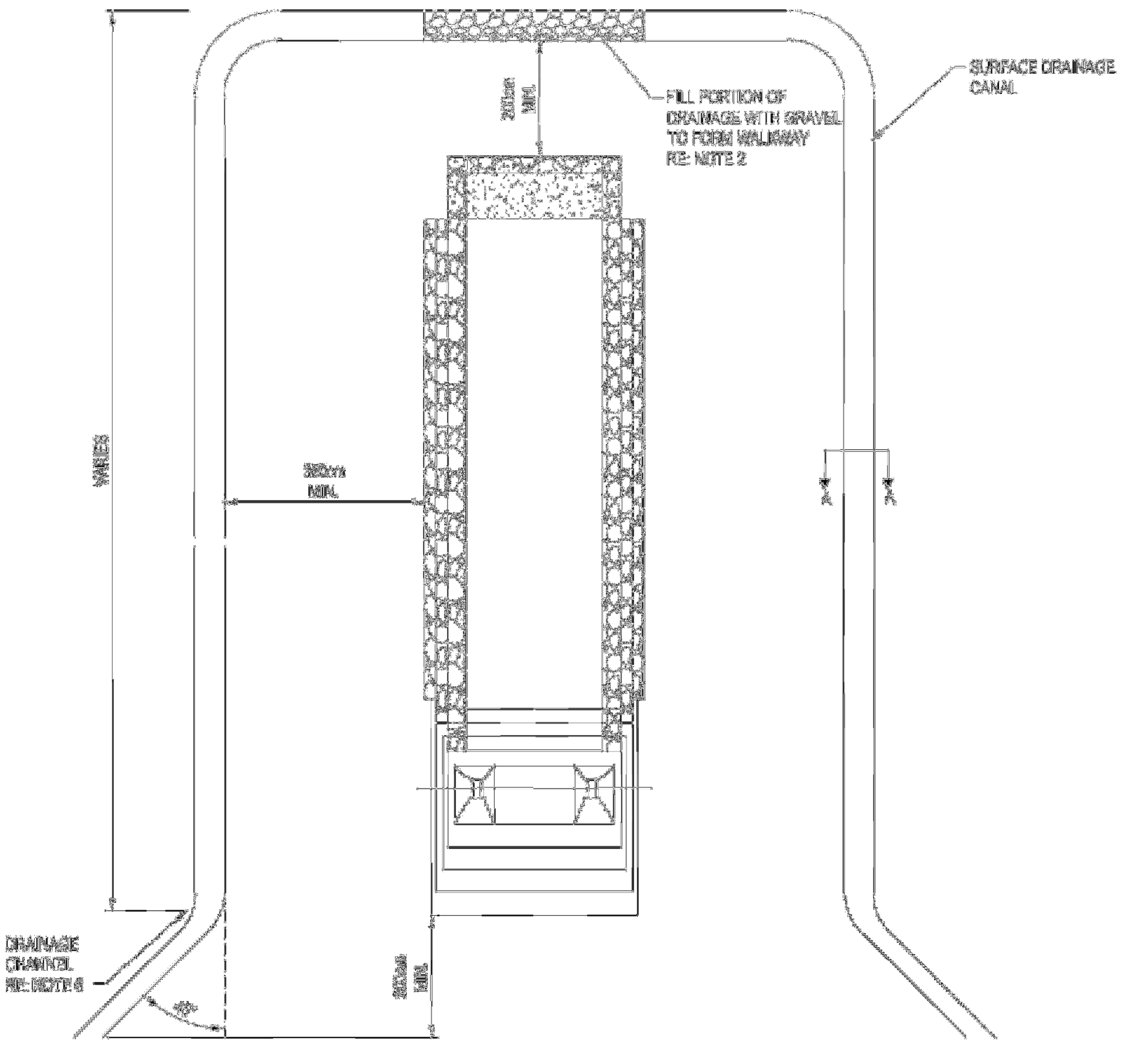
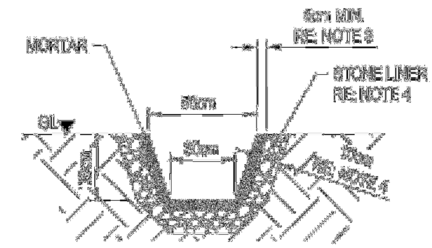
KHELEKHELE F2E FENCING DETAIL

PROJECT NUMBER	DRAWING NUMBER
BSZ-BP116	9



PLAN



SECTION A-A

NOTES:
1. DRAINAGE CONSTRUCTION SHOULD BE COMPLETED WITH LEFTOVER MATERIALS WHEN POSSIBLE.
2. DO NOT FILL DRAINAGE WITH SOIL.
3. DRAINAGE THICKNESS SHOULD BE A MINIMUM OF 5cm.
4. LINE CHANNEL WITH SMALL STONES APPROXIMATELY 10cm THICK.
5. THIS DETAIL IS FOR PLANNING PURPOSES AND USES STANDARD, MINIMUM DIMENSIONS. FIELD CHANGES ARE ACCEPTABLE TO BETTER FIT SITE TOPOGRAPHY.
6. EXTEND DRAINAGE TO A POINT WITH SAME ELEVATION AS FRONT OF FOUNDATION

EIA MICROPROJECTS

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KHELEKHELE D4 DRAINAGE DETAIL

PROJECT NUMBER	DRAWING NUMBER
BSZ-BP116	10

Design Summary

Bridge Layout

Span L	=	93.51	m	306.79	ft	
Deck Width	=	1.04	m	3.41	ft	
Height Difference (ΔH)	=	3.42	m	11.22	ft	
Lower Saddle (type LEFT or RIGHT)		LEFT				
Terrain Type		GORGE				
High Water Elevation, HWL	=	100.00	m	328.08	ft	
Freeboard (F_b)	=	3.47	m	11.38	ft	
		# Cables		Size (mm)		Size (in)
Handrail Cables	=	2.00		32.00		1.26
Walkway Cables	=	3.00		32.00		1.26

Hoisting Geometry

Construction Sag (B_c)	=	3	%		
b_sag (Construction)	=	2.8053	m		
Hoisting Sag (B_h)	=	4	%		
b_sag (Hoist)	=	3.74	m		
Lower tower to low point, f_construction	=	1.36	m	From	RIGHT
Lower tower to low point, f_hoist	=	2.23	m	From	LEFT

Dead Load Geometry

Dead Load Sag (B_d)	=	4.55	%		
h_sag (Dead)	=	4.25	m	13.96	ft
Lower tower to low point, f_design	=	2.72	m	8.91	ft
Left Tower to low point, X_left	=	37.36	m	122.57	ft
Left Tower to low point, Y_left	=	2.72	m	8.91	ft
Right Tower to low point, X_right	=	56.15	m	184.22	ft
Right Tower to low point, Y_right	=	6.14	m	20.13	ft

Left Abutment (Looking Downstream)

Foundation Elevation	=	106.29	m	348.72	ft
Anchor Elevation	=	100.79	m	330.69	ft
No. of Tiers (not include foundation)	=	3.00			
Distance to Back of Anchor	=	14.00	m	45.93	ft
Anchor size	=	60-100	m		
Walkway Cable Angle, α_WalkLeft	=	18.26	°	0.32	rad
Handrail Cable Angle, α_HandLeft	=	22.61	°	0.39	rad
Height of Soil, H_1	=	2.09	m	6.84	ft
Height of Backwall, H_2	=	3.00	m	9.84	ft
Slope of Backfill, β	=	0.00	°		
Ramp Area, A_Ramp_wall	=	45.11	m2	485.56	ft2
Width of Ramp, w	=	3.00	m	9.84	ft
Anchor Height	=	1.30	m	4.27	ft
Anchor Base 1		1.10			
Anchor_ Base 2	=	1.40	m	4.59	ft
Anchor Area	=	1.63	m2	17.49	ft2
Anchor Volume	=	4.88	m3	172.16	ft3

Right Abutment (Looking Downstream)

Foundation Elevation	=	109.72	m	359.97	ft
Anchor Elevation	=	106.71	m	350.11	ft
No. of Tiers (not include foundation)	=	1.00			
Distance to Back of Anchor	=	9.68	m	31.75	ft
Anchor Size	=	60-100	m		
Walkway Cable Angle, α_WalkRight	=	14.39	°	0.25	rad
Handrail Cable Angle, α_HandRight	=	21.36	°	0.37	rad
Height of Soil, H_1	=	3.78	m	12.40	ft
Height of Backwall, H_2	=	3.00	m	9.84	ft
Slope of Backfill, β	=	11.83	°		
Ramp Area, A_Ramp	=	23.80	m2	256.18	ft2
Width of Ramp, w	=	3.00	m	9.84	ft
Anchor Height	=	1.30	m	4.27	ft
Anchor Base 1		1.10			
Anchor_ Base 2	=	1.40	m	4.59	ft
Anchor Area	=	1.63	m2	17.49	ft2
Anchor Volume	=	4.88	m3	172.16	ft3

Strength

Steel, f_y	f_y	=	240	MPa	35.00	ksi
Reinforcing Bar, f_y	f_y	=	275	MPa	40.00	ksi
Concrete (mixed by hand), f'_c	f'_c	=	10	MPa	1450.00	psi
Timber, f_b	f_b	=	3.96	MPa	574.00	psi
f_v	f_v	=	1.44	MPa	209.00	psi
Soil, q_a	q_all	=	143	kPa	2987.00	psf
ϕ		=	30	deg	0.52	rad
Cable, E	E	=	110,000	N/mm2	15949.00	ksi
Breaking strength		=	604	kN		

Densities

Steel, ϒ_st	ϒ_st	=	7850	kg/m3	487	pcf
Concrete, ϒ_c	ϒ_c	=	2400	kg/m3	149	pcf
Timber, ϒ_t	ϒ_t	=	900	kg/m3	56	pcf
Broken Rock, ϒ_r	ϒ_r	=	1900	kg/m3	118	pcf
Masonry, ϒ_m	ϒ_m	=	2100	kg/m3	130	pcf
Soil, ϒ_s	ϒ_s	=	1800	kg/m3	112	pcf
Cable, ϒ_cab	ϒ_cab	=	4.3	kg/m	31	lb/ft
Water, _w	ϒ_w	=	1000	kg/m3	62	pcf

Check Height Difference

				limit	
Max Height Difference	=	L/25	=	3.74	m
Height Difference (ΔH)	=	3.42		3.74	OK

Check Freeboard

				limit	
Distance to Lowest Point of Cable (f)	=	$\frac{(4 * B_d - \Delta H)^2}{16 * B_d}$	=	2.72	
Required freeboard	=	3 m			
Actual Freeboard	=	3.57		3	OK

Check Cable Load

				limit	
Maximum Cable Capacity, Ps	=	3020.00	kN		
Maximum Cable Tension, Pr	=	857.39	kN		
Factor of Safety	=	3.52		3	OK

Check Left Tower Moment Capacity

Nominal Moment Capacity, M_n	=	63.11	m		
Unfactored Moment, M	=	28.14	m		
Factor of Safety	=	2.02	>=	1.5	OK

Checking Left Overturning About Toe of Tower

Overturning Moment	=	-121.71	kN-m		
Restorative Moment	=	203.72	kN-m		
Factor of Safety	=	1.67	>=	1.5	OK

Check Left Overturning about Toe of Foundation

Overturning Moment	=	-661.57	kN-m		
Restorative Moment	=	2302.66	kN-m		
Factor of Safety	=	3.48	>=	1.5	OK

Checking Left Bearing Pressure

q_a	=	111.32	kPa		
Bearing Pressure (kPa)	=	111.32	<=	100	NG

Check Left Anchor Sliding

		Include Side Wall Friction?		YES	NO
		Include Bottom Slope Effects?		YES	NO
Total Horizontal Load, F_b	=	867.17			
Total Frictional Resistance, F_f	=	2453.62			
Factor of Safety	=	2.83	>=	1.5	OK

Check Left Anchor Uplift

		Use additional area of ramp?		YES	NO
Resisting Dead Load Force	=	463.96	kN		
Vertical Uplift Cable Force	=	-264.31	kN		
Factor of Safety	=	1.76	>=	1.5	OK

Check Right Tower Moment Capacity

Nominal Moment Capacity, M_n	=	63.11	m		
Unfactred Moment, M	=	24.96	m		
Factor of Safety	=	2.28	>=	1.5	OK

Checking Right Overturning About Toe of Tower

Overturning Moment	=	-110.05	kN-m		
Restorative Moment	=	224.33	kN-m		
Factor of Safety	=	2.04	>=	1.5	OK

Check Right Overturning about Toe of Foundation

Overturning Moment	=	-400.19	kN-m		
Restorative Moment	=	1049.38	kN-m		
Factor of Safety	=	2.62	>=	1.5	OK

Checking Right Bearing Pressure

q_a	=	155.48	kPa		
Bearing Pressure (kPa)	=	155.48	<=	100	NG

Check Right Anchor Sliding

	Include Side Wall Friction?		YES	NO
	Include Bottom Slope Effects?		YES	NO
Total Horizontal Load, F_b	=	963.60		
Total Frictional Resistance, F_f	=	1147.84		
Factor of Safety	=	1.52 >=	1.5	OK

Check Right Anchor Uplift

	Use additional area of ramp?		YES	NO
Resisting Dead Load Force	=	493.00 kN		
Vertical Uplift Cable Force	=	-230.66 kN		
Factor of Safety	=	2.14 >=	1.5	OK

Suspender Design Check

Suspender Demand	=	1.03 kN		
Suspender Capacity	=	21.60 kN		
Suspender Factor of Safety	=	21.00 >=	5	OK

Check Timber Decking

F'_b (Distributed Load)	=	4.65 Mpa			
f_b (Distributed Load)	=	1.69 Mpa	<=	4.65 Mpa	OK
F'_b (Point Load)	=	9.29 Mpa			
f_b (Point Load)	=	7.31 Mpa	<=	9.29 Mpa	OK
F'_s (Distributed Load)	=	1.69 Mpa			
f_s (Distributed Load)	=	0.08 Mpa	<=	1.69 Mpa	OK
F'_s (Point Load)	=	3.38 Mpa			
f_s (Point Load)	=	0.23 Mpa	<=	3.38 Mpa	OK

Design Loads

Crossbeam Material	=	Steel					
Crossbeam Spacing, s	=	1	m	3.280839895	ft		
Maximum Decking Thickness, t_Deck,max	=	5	cm	2	in		
Deck Width, w_deck, effective	=	0.9	m	35	in		
W_deck	=	$t_{Deck,max} * w_{Deck} * \gamma_t =$		40.5	kgf/m	27.21	lb/ft
Crosbeam Area	=	13.7	cm2	2.12	in2		
Crossbeam Length	=	150	cm	59.06	in		
Nailer Area	=	75	cm2	11.63	in2		
Nailer Length	=	100	cm	39.37	in		
Crossbeam Volume, V_Beam	=	0.002055	m3	0.07	ft3		
Nailer Volume, V_Nailer	=	0.0075	m3	0.26	ft3		
W_Beam	=	$V_{Beam} * \gamma_{s/t} + V_{Nailer} * \gamma_t * \frac{1}{s} =$		22.9	kgf/m	15.38	lb/ft
Suspender Size	=	10	mm	#3			
Suspeder Length	=	2	m	6.6	ft		
Suspender Area	=	78.54	mm2	12.2	in2		
Suspender Volume, V_suspender	=	157.08	cm3	9.6	in3		
W_suspender	=	$\frac{2 * (V_{Suspender} * \gamma_{st})}{s} =$	2.5		kgf/m	1.66	lb/ft
Fencing Height, H_Fence	=	1.2	m	3.94	ft		
Fencing Unit Weight, γ_Fence	=	2.2	kgf/m2	0.44	psf		
W_Fence	=	$2 * (H_{Fence} * \gamma_{Fence}) =$		0.0517968	kN/m	38.203	lb/ft
d_left	=	14.0	m				
d_right	=	9.68	m				
Total length of each cablee, L_cable	=	$L_{cable} = (L + 14 + d_{left} + d_{right})$			=	131.2	m
Cable Weight, W_Cable	=	=		0.21	kN/m	155.56	lb/ft
DL	=	$\sum W_n =$		0.908679701	kN/m	18.978	lb/ft^2
LIVE LOAD REDUCTION? (YES or NO)	:	YES					
Live Load (LL)		=		3.140	kN/m	65.58	lb/ft
Unreduced Live Load		=		4.070	kN/m	85.004	lb/ft
Reduced Live Load	=	$4.07 * \left(0.25 + \frac{4.57}{\sqrt{A}}\right)$		3.140	kN/m	65.58	lb/ft
INCLUDE WIND LOAD (YES or NO)	:	NO					
Wind Load (WL)	=			0.0	kN/m	0	lb/ft
Total Distributed Load, ω_c	=	DL+LL		4.049	kN/m	2986.2	lb/ft
LRFD Load Combination	=	1.2DL+1.6LL		6.114415642	kN/m	4509.8	

Cable Analysis

Cable Size Estimate

		#Cables	Size (in.)	Area (mm2)	Area (in,2)		
Handrail Cable Area, A_Chand	=	2.00	1.26	1608.50	2.49		
Walkway Cable Area, A_Cwalk	=	3.00	1.26	2412.74	3.74		
Total Cable Area, A_Ctotal	=			4021	6.23		

Cable Sag Values

Construction Sag (h_sag)	=	2.81	m	9.20	ft	3	%
Hoisting Sag (h_sag)	=	3.74	m	12.27	ft	4	%
Dead Load Sag (h_sag)	=	4.25	m	13.96	ft	4.55	%
Dead Load + Live Load Sag (h_sag)	=	5.34	m	17.52	ft	5.71	%

Main Span Cable Geometry (Hoisting)

Lower Tower to low point, Y_hoist	=	$\frac{(4 * h_{sag} - \Delta H)^2}{16 * h_{sag}}$	=	2.23	m	7.30	ft

Main Span Cable Geometry (Dead Load)

Lower Saddle?	=	LEFT					
Left Tower to low point, X_left	=	$\frac{L * (4 * h_{sag} \pm \Delta H)^2}{8 * h_{sag}}$	=	37.36	m	122.57	ft
Left Tower to Low point, Y_left	=	$\frac{(4 * h_{sag} \pm \Delta H)^2}{16 * h_{sag}}$	=	2.72	m	8.91	ft
Right Tower to low Point, X_right	=	$\frac{L * (4 * h_{sag} \pm \Delta H)^2}{8 * h_{sag}}$	=	56.15	m	184.22	ft
Right Tower to low Point, Y_right	=	$\frac{(4 * h_{sag} \pm \Delta H)^2}{16 * h_{sag}}$	=	6.14	m	20.13	ft

Main Span Cable Geometry (Fully Loaded)

Left Tower to low point, X_Left	=	$\frac{L * (4 * h_{sag} \pm \Delta H)^2}{8 * h_{sag}}$	=	39.27	m	128.83	ft
Left Tower to Low point, Y_left	=	$\frac{(4 * h_{sag} \pm \Delta H)^2}{16 * h_{sag}}$	=	3.77	m	12.36	ft
Left Tower Cable Angle, θ_Left	=	$tan^{-}\left(\frac{(4 * h_{sag} \pm \Delta H)^2}{\mathrm{L}}\right)$	=	10.86	deg	0.19	rad
Right Tower to low Point, X_right	=	$\frac{L * (4 * h_{sag} \pm \Delta H)^2}{8 * h_{sag}}$	=	54.24	m	177.96	ft
Right Tower to low Point, Y_right	=	$\frac{(4 * h_{sag} \pm \Delta H)^2}{16 * h_{sag}}$	=	7.19	m	23.58	ft
Right Tower to low point, θ_Right	=	$tan^{-}\left(\frac{(4 * h_{sag} \pm \Delta H)^2}{\mathrm{L}}\right)$	=	14.84	deg	0.26	rad

Check Cable Capacity

		#Cables	Size	Capacity	
Handrail	=	2.00	1.26	1208	604
Walkway	=	3.00	1.26	1812	
Total Number of Cables	=	5.00			
Maximum Cable Capacity, Ps (kN)			=	3020	
Factor of Safety	=	3.52	>	3	OK

Cable Force Analysis

Horizontal Tension, Ph_Mtotal	=	$\frac{w_c * L^2}{8 * h_{sag}}$	=	828.79	kN	186312.37	lb
Left Vertical Tension, Pv_Mtotal	=	$Ph_{MTotal} * \tan\theta_{Left}$	=	158.98	kN	35739.63	lb
Left Axial Tension, Pt_Mtotal	=	$\frac{Ph_{MTotal}}{cos\theta_{Left}}$	=	843.90	kN	189709.30	lb
Left Handrail Backstay Tension, Pt_Back,hand	=	$Pt_{MHand} * e^{-\mu(\alpha_{Hand}+\theta_{Left}+0.04)}$	=	292.42	kN	65736.17	lb
Left Walkway Backstay Tension, Pt_Back,walk	=	$Pt_{MWalk} * e^{-\mu(\alpha_{Walk}+\theta_{Left}+0.04)}$	=	446.36	kN	100341.20	lb
Left Total Backstay Tension, Pt_Back	=	$Pt_{BWalk}+Pt_{BHand}$	=	738.78	kN	166077.37	lb
Left Vertical Backstay Tension, Pv,Back	=	$P_T, Back, Hand * \sin(\alpha_{Left,Hand}) + P_T, Back, Walk * \sin(\alpha_{Left,Walk})$	=	252.28	kN	56712.56	lb
Left Horizontal Backstay Tension, Ph,Back	=	$P_T, Back, Hand * \cos(\alpha_{Left,Hand}) + P_T, Back, Walk * \cos(\alpha_{Left,Walk})$	=	693.83	kN	155972.36	lb
Right Vertical Tension, Pv_Mtotal	=	$Ph_{MTotal} * \tan\theta_{Right}$	=	219.61	kN	49367.86	lb
Right Axial Tension, Pt_Mtotal	=	$\frac{Ph_{MTotal}}{cos\theta_{Right}}$	=	857.39	kN	192742.02	lb
Right Handrail Backstay Tension, Pt_Back,hand	=	$Pt_{MHand} * e^{-\mu(\alpha_{Hand}+\theta_{Right}+0.04)}$	=	293.86	kN	66058.62	lb
Right Walkway Backstay Tension, Pt_Back,walk	=	$Pt_{MWalk} * e^{-\mu(\alpha_{Walk}+\theta_{Right}+0.04)}$	=	453.29	kN	101899.49	lb
Right Total Backstay Tension, Pt_Back	=	$Pt_{BWalk}+Pt_{BHand}$	=	747.14	kN	167958.11	lb
Right Vertical Backstay Tension, Pv,Back	=	$P_T, Back, Hand * \sin(\alpha_{Right,Hand}) + P_T, Back, Walk * \sin(\alpha_{Right,Walk})$	=	219.68	kN	49384.46	lb
Right Horizontal Backstay Tension, Ph,Back	=	$P_T, Back, Hand * \cos(\alpha_{Right,Hand}) + P_T, Back, Walk * \cos(\alpha_{Right,Walk})$	=	712.74	kN	160223.63	lb
Maximum Cable tension, Pr			=	**857.39**	**kN**	**192742.0**	**Lb**

Left Tower Analysis

Left Tower Geometry

Tower Height, H_Tower	=	1.50	m	4.92 ft
Tower Width, W_Tower	=	2.80	m	9.19 ft
Tower Depth, D_Tower	=	1.00	m	3.28 ft
Walkway Height, H_Walkway	=	0.50	m	1.64 ft
Walkway Width, W_Walkway	=	1.40	m	4.59 ft
Walkway Depth, D_Walkway	=	1.00	m	3.28 ft
Column Height, H_Column	=	1.20	m	3.94 ft
From the AUTOCAD standard drawings	=	18.26	deg	0.32 rad
From the AUTOCAD standard drawings	=	22.61	deg	0.39 rad
E_SaddleOffset	=	0.10	m	0.33 ft
Reinforcing Bar Size	=	16.00	mm	#5
Area of Bar, Ab	=	201.06	mm2	0.31 in2
Number of Bars	=	4.00		
Reinforcing Bar Cover	=	0.100	m	3.94 in
Masonry Block Width	=	0.30	m	11.81 in
Effective Column Width, W_Ef	=	0.40	m	1.31 ft
Effective Column Depth, D_Ef	=	0.70	m	2.30 ft

Determine Forces Acting on Top of Left Tower Columns

Handrail Axial Tension, Pt_MHand	=	$Pt_{MTotal} * \left(\frac{A_{CHand}}{A_{CTotal}}\right)$	=	337.56 kN	75883.72 lb
Handrail Vertical Tension, Pv_Mhand	=	$Pv_{MTotal} * \left(\frac{A_{CHand}}{A_{CTotal}}\right)$	=	63.59 kN	14295.85 lb
Hadrail Horizontal Tension, Ph_Mhand	=	$Ph_{MTotal} * \left(\frac{A_{CHand}}{A_{CTotal}}\right)$	=	331.52 kN	74524.95 lb
Coefficient of Friction, μ_Saddle,max	=	0.23		Given by Volume 2 of Bridge Binder	
Backstay Axial Tension, Pt_Bhand	=	$Pt_{MHand} * e^{-\mu(\alpha_{Hand}+\theta_{Left}+0.04)}$	=	292.42 kN	65736.17 lb
Backstay Vertical Tension, Pv_Bhand	=	$Pt_{BHand} * sin\alpha_{Hand}$	=	112.42 kN	25272.69 lb
Backstay Horizontal Tenison, Ph_Bhand	=	$Pt_{BHand} * cos\alpha_{Hand}$	=	269.95 kN	60683.89 lb
Vertical Force on Single Column, Pv_Hand	=	$\frac{1}{2}(Pv_{BHand} + Pv_{MHand})$	=	88.01 kN	19784.27 lb
Horizontal Force on Single Column, Ph_Hand	=	$\frac{1}{2}(Ph_{BHand} + Ph_{MHand}))$	=	30.79 kN	6920.53 lb

Determine Column Moment Capacity

Concrete Area, A_c	=	$W_{Eff} * D_{Eff}$	=	0.28 m2	434 in2
Reinforcing Area, A_s	=	No. of Bars$* A_b$	=	402.12 mm2	0.62 in2
Depth to Reinforcing, d	=	$D_{Eff} - Cover$	=	0.59 m	23.1 in
Section Modulus, S	=	$W_{Eff} * \frac{(D_{Eff})^2}{6}$	=	0.03 m3	1993.4 in3
Concrete Modulus of Rupture, f_r	=	$0.52 * \sqrt{f'c}$	=	1.68 Mpa	243.08 psi
Cracking Moment, M_cr	=	$f_r * S$	=	54.75 kN-m	40.38 k-ft
Nominal Moment Capacity, M_n	=	$A_s * f_y * \left(d - \frac{a}{2}\right)$	=	63.11 kN-m	46.55 k-ft
a	=	$\frac{A_s * F_y}{(0.85 * f'c * W_{Eff})}$	=	32.52 mm	1.28 in

Check Tower Moment Capacity

Unfactored Moment, M	=	$Pv_{Hand} * E_{SaddleOffset} + Ph_{Hand} * H_{Column}$ =		28.14 kN-m	20.76 k-ft
Factor of Safety	=	2.02	>	1.5	OK
Load Reduction Factor	=	0.90			
Design Strength, Mn_factored	=	ϕM_n	=	56.80 kN-m	
Required Strength 1.2M_DL+1.6M_LL	=		=	51.67	

Check Minimum Reinforcing

Factored Flexural Resistance, ϕM_n		>		Min(1.33*1.625*M,MCr)	
kN-m*		56.80	>	54.75	OK

Determine Forces Acting on Walkway

Walkway Axial Tension, Pt_Mwalk	=	$Pt_{MTotal} * \left(\frac{A_{CWalk}}{A_{CTotal}}\right)$	=	506.34 kN	113825.58 lb
Walkway Vertical Tension, Pv_Mwalk	=	$Pv_{MTotal} * \left(\frac{A_{CWalk}}{A_{CTotal}}\right)$	=	95.39 kN	21443.78 lb
Walkway Horizontal Tension, Ph_Mwalk	=	$Ph_{MTotal} * \left(\frac{A_{CWalk}}{A_{CTotal}}\right)$	=	497.28 kN	111787.42 lb
Coefficient of Friction, µ_Saddle,max	=	0.23	Given by Volume 2 of Bridge Binder		
Min Backstay Axial Tension, Pt_Bwalk	=	$Pt_{MWalk} * e^{-\mu(\alpha_{Walk}+\theta_{Left}+0.04)}$	=	446.36 kN	100341.20 lb
Min Backstay Vertical Tension, Pv_Bwalk	=	$Pt_{BWalk} * \sin\alpha_{Walk}$	=	139.86 kN	31439.86 lb
Min Backstay Horizontal Tension, Ph_Bwalk	=	$Pt_{BWalk} * \cos\alpha_{Walk}$	=	423.88 kN	95288.47 lb
Vertical Force on Walkway, Pv_Walk	=	$(Pv_{BWalk} + Pv_{MWalk})$	=	235.25 kN	52883.64 lb
Horizontal Force on Walkway, Ph_Walk	=	$(Ph_{BWalk} - Ph_{MWalk})$	=	73.39 kN	16498.95 lb
Angle of Resultant, δ_Walk	=	$\tan^{-}(\frac{Ph_{Walk}}{Pv_{Walk}})$	=	17.33 deg	0.30 rad

Check Overturning about Toe of Tower

Overturning Moment	=	$(2 * Ph_{Hand} * H_{Tower}) + (Ph_{Walk} * (H_{Walk} - .10))$ =	-121.713 kN-m	89.78 k-ft
Restorative Moment	=	$(2 * Pv_{Hand} + Pv_{Walk}) * \left(\frac{D_{Tower}}{2} + E_{SaddleOffset}\right) + \left(P_{Tower} * \frac{D_{Tower}}{2}\right)$ =	203.7238747 kN-m	150.27 k-ft
Factor of Safety	=	1.67 >	1.5	OK

Check Column Eccentricity

Angle of Resultant_ Delta	=	$\tan^{-1}(Ph_{hand}/Pv_{Hand})$	=	19.28 deg	0.34 rad
E_CableMax	=	$\frac{D_{Eff}}{2} + E_{SaddleOffset}$	=	0.45	
E_column	=	$H_{Column} * \tan Delta_{Hand}$	=	0.42	
Check Column Eccentricity		0.42	<	0.45	OK

Left Foundation Analysis

Foundation Geometry

	Depth (r	Width (m)	Height (m)	Tier Offset (m)	Volume (m3)	Weight (kN)	Weight (lb)	Y_Hand (m)	Y_Walk (m)
Tower	1.00	2.80	1.50	0.10	2.59	53.30	11981.84		
Tier1	1.65	3.00	1.00	0.18	4.95	96.93	21790.27		
Tier2	2.30	3.20	1.00	0.25	7.36	142.81	32102.70	3.50	2.40
Tier 3	2.95	3.40	1.00	0.33	10.03	193.52	43503.43	4.50	3.40
Tier4	3.60	3.60	1.00	0.40	12.96	249.08	0.00	5.50	4.40

Cable Forces on Tower

Total Vertical Force on Columns, 2*Pv_hand	=	176.02 kN	39568.54 lb
Total Vertical Force on Walkway, Pv_walk	=	235.25 kN	52883.64 lb
Total Horizontal Force on Columns, 2*Ph_Hand	=	61.57 kN	13841.06 lb
Total Horizontal Force on Walkway, Ph_Walk	=	73.39 kN	16498.95 lb

Check Overturning about Toe of Foundation

Overturning moment = $(2 * Ph_{Hand} * Y_{Hand}) + (Ph_{Walk} * Y_{Walk})$ = -661.57 kN-m -487.97 k-ft

Restorative Moment = $(P_{Saddle} * X_{Saddle}) + \left(\sum P_{TierN} * X_{TierN}\right)$

		Force(P_Saddle)		Arm (X_Saddle)		Moment (kN-m)	Moment (k-ft)
Vertical Reaction	=	411.26 kN	x	2.20 m	=	904.78	667.37
		Force(P_Tier)		Arm (X_Tier)		Moment (kN-m)	Moment (k-ft)
Tower	=	53.30 kN	x	2.10 m	=	111.93	82.56
Tier1	=	96.93 kN	x	2.03 m	=	196.29	144.78
Tier2	=	142.81 kN	x	1.95 m	=	278.47	205.40
Tier3	=	193.52 kN	x	1.88 m	=	362.85	267.64
Tier4	=	249.08 kN	x	1.80 m	=	448.34	330.69
		Totals	=	1146.90 kN	=	2302.66	1698.44

Factor of Safi = 3.48 > 1.5 OK

Check Bearing Pressure

Effective Width of Foundation

Location of Resultant Force, B*/2	=	$\frac{\sum Moments}{\sum Vertical}$	=	1.43 m	4.69 ft	
Check Resultant Located>B/3	=	1.43	>	1.20	OK	
Effective width, B*	=	2.86 m		9.39 ft		
Bearing Pressure	=	$\frac{\sum Vertical}{B^* x\ Width}$	=	111.32 kN/m2		
Check Bearing Pressure	=	111.32 kN/m2	<	100.00 kN/m2		NG
Factor of Safety	=	2.69	>	3		NG

Left Anchor Analysis

Determine Cable Forces on Anchor

Mainspan Cable Forces

Pt_Mtotal	=		843.90 kN	189709.304 lb
Pv_Mtotal	=		158.98 kN	35739.6258 lb
Ph_Mtotal	=		828.79 kN	186312.369 lb
θ_left	=		10.86 deg	0.19 rad

Handrail Cable Forces

Pt_Mhand	=			337.56 kN	75883.7214 lb
Pv_Mhand	=			63.59 kN	14295.8503 lb
Ph_Mhand	=			331.52 kN	74524.9478 lb
α_Hand	=			22.61 deg	0.39 rad
Coefficient of Friction μ_Saddle,max	=	0.15		For Anchor Analysis	
Backstay Axial Tension, Pt_Bhand	=	$Pt_{MHand} * e^{-\mu(\alpha_{Hand}+\theta_{Left}+0.04)}$	=	307.39 kN	69101.80 lb
Backstay Vertical Tension, Pv_BHand	=	$Pt_{BHand} * sin\alpha_{Hand}$	=	118.18 kN	26566.63 lb
Bakstay Horizontal Tension, Ph_Bhand	=	$Pt_{BHand} * cos\alpha_{Hand}$	=	283.77 kN	63790.85 lb

Walkway Cable Forces

Pt_Mwalk	=			506.34 kN	113825.582 lb
Pv_Mwalk	=			95.39 kN	21443.7755 lb
Ph_Mwalk	=			497.28 kN	111787.422 lb
α_Walk	=			18.26 deg	0.32 rad
Bacstay Axial Tension, Pt_Walk	=	$Pt_{MWalk} * e^{-\mu(\alpha_{Walk}+\theta_{Left}+0.04)}$	=	466.37 kN	104839.87 lb
Backstay Vertical Tension, Pv_Walk	=	$Pt_{BWalk} * sin\alpha_{Walk}$	=	146.13 kN	32849.43 lb
Backstay Horizontal Tension, Ph_Walk	=	$Pt_{BWalk} * cos\alpha_{Walk}$	=	442.89 kN	99560.60 lb

Total Cable Forces on Anchor

Vertical Forces, Pv_Anchor	=		-264.31 kN	-59416.06 lb
Horizontal Force, Ph_Anchor	=		726.65 kN	163351.45 lb

Determine Earth Pressure on Anchor

Using Rankine Theory

Internal Angle of Soil Friction, ϕ	=			30.00 deg	0.52 rad
Slope of Backfill, β	=			0.00 deg	0.00 rad
Unit Weight of Soil, Ys	=			1800.00 kg/m3	56.20 lb/cf
Height of Soil H_1	=			2.09 m	6.84 ft
Width, w	=			3.00 m	9.84 ft
Active Earth Pressure Coefficient, K_a	=	$\cos\beta\left(\frac{cos\beta-\sqrt{(\cos^2\beta-\cos^2\phi}}{cos\beta+\sqrt{(\cos^2\beta-\cos^2\phi}}\right)$	=	0.33	
Active Earth Pressure, P_Active	=	$\frac{1}{2}K_a * \gamma_s * H_1^2 * w$	=	38.38 kN	8627.67 lb

Summary of Anchor Forces

Ramp Area 30 cm thick	=	6.40	m2	440.89 ft2	
Ramp Area 50 cm thick	=	11.75	m2	1486.09 ft2	
Ramp Area 70 cm thick	=	26.97	m2	7829.46 ft2	
Stone Masonry Density	=	2100.00	kg/m3		
Backwall Area	=	0.49	m2	2.55 ft2	
Backwall Width	=	3.00	m	9.84 ft	
Concrete cap area	=	1.37	m2	20.20 ft2	
Concrete cap width	=	3.00	m	ft	
Concrete Density	=	2400.00	kg/m2		
Backwall Thickness	=	0.30	m	0.98 ft	
Fill Area 1	=	45.11	m2	21903.61 ft2	
Fill Thickness 1	=	2.40	m	7.87 ft	
Fill Density (using Broken Rock density)	=	1900.00	kg/m3		
Undisturbed Soil Mass	=	$A_{Soil} * (Width_{ramp} - Width_{wall}) * \gamma$	=	550.51 kg	
Weight of Ramp, P_Ramp	=	$\sum A_{Ramp} * width * \gamma$	=	3240.51 kN	728467.12 lb
Bottom Slope Effect Percentage to include	=	50	%		
Bottom Slope Angle, θ	=	2.56	deg	0.04 rad	
Bottom Slope Effect Force, Pslope	=	$P_{Ramp} * sin\theta * percentage$	=	72.37 kN	

Horizontal Forces

P_Active	=	38.38 kN	8627.67 lb	
Ph_Anchor	=	726.65 kN	163351.45 lb	
Ph_Tower	=	102.14 kN	22960.92 lb	

Vertical Forces

Pv_Anchor	=	-264.31 kN	-59416.06 lb
Pv_Tower	=	423.29 kN	95155.69 lb
P_Abut	=	735.63 kN	165370.70 lb
P_Ramp	=	3240.51 kN	728467.12 lb
P_Anchor	=	114.66 kN	25775.57 lb

Check Anchor Sliding

Coefficient of Friction μ_Sliding	=	$\tan(\phi)$	=	0.58		
Total Horizontal Load, F_h	=	$P_{Active} + Ph_{Anchor} + Ph_{Tower}$	=	867.17 kN	194940.04 lb	
Total Vertical Load, N	=	$Pv_{Anchor} + Pv_{Tower} + P_{Abut} + P_{Ramp} + P_{Anchor}$	=	4249.79 kN	955353.02 lb	
Total Frictional Resistance, F_f	=	μN	=	2453.62 kN	551573.32 lb	
Factor of Safety	=	$\frac{F_f}{F_h}$				
Sliding Factor of Safety	=	2.83	>	1.5		OK

Check Anchor Uplift

H_2	=	3.00 m		9.84 ft		
w	=	3.00 m		9.84 ft		
b	=	1.40 m		4.59 ft		
B_1	=	$b + H_2 * \tan(30°)$	=	3.13 m	10.28 ft	
Weight of Anchor Beam, P_AnchorBeam	=	114.78 kN		25801.87 lb		
Weight of Overburden, P_Overburden	=	288.97 kN		64960.01 lb		
Resisting Dead Load Force	=	$P_{AnchorBeam} + P_{Overburden}$	=	403.75 kN	90761.88 lb	
Vertical Uplift Cable Force	=	Pv_{Anchor}	=	-264.31 kN	-59416.06 lb	
Uplift Factor of Safety	=	1.528	>	1.50		OK

Summary of Additional Anchor Forces (Used in LHS)

Check Anchor Uplift

H_2	=	3.00 m		9.84 ft		
w	=	3.00 m		9.84 ft		
b	=	1.40 m		4.59 ft		
B_1	=	$b + H_2 * \tan(30°)$	=	3.13 m	10.28 ft	
α	=	8.37 deg		0.15 rad		
Additional Area	=	1.08 m2		11.63 ft2	0.45	
Weight of Anchor Beam, P_AnchorBeam	=	114.66 kN		25775.57 lb		
Weight of Overburden, P_Overburden	=	349.30 kN		78521.93 lb		
Resisting Dead Load Force	=	$P_{AnchorBeam} + P_{Overburden}$	=	463.96 kN	104297.50 lb	
Vertical Uplift Cable Force	=	Pv_{Anchor}	=	-264.31 kN	-59416.06 lb	
Uplift Factor of Safety	=	1.76	>	1.5		OK

Right Tower Analysis

Right Tower Geometry

Tower Height, H_Tower	=	1.50	m	4.92	ft
Tower Width, W_Tower	=	2.80	m	9.19	ft
Tower Depth, D_Tower	=	1.00	m	3.28	ft
Walkway Height, H_Walkway	=	0.50	m	1.64	ft
Walkway Width, W_Walkway	=	1.40	m	4.59	ft
Walkway Depth, D_Walkway	=	1.00	m	3.28	ft
Column Height, H_Column	=	1.20	m	3.94	ft
From the AUTOCAD standard drawings	=	14.39	deg	0.25	rad
From the AUTOCAD standard drawings	=	21.36	deg	0.37	rad
E_SaddleOffset	=	0.10	m	0.33	ft
Reinforcing Bar Size	=	16.00	mm	#5	
Area of Bar, Ab	=	201.06	mm2	0.31	in2
Number of Bars	=	4.00			
Reinforcing Bar Cover	=	0.100	m	3.94	in
Masonry Block Width	=	0.30	m	11.81	in
Effective Column Width, W_Ef	=	0.40	m	1.31	ft
Effective Column Depth, D_Ef	=	0.70	m	2.30	ft

Determine Forces Acting on Top of Right Tower Columns

Handrail Axial Tension, Pt_MHand	=	$Pt_{MTotal} \cdot \left(\frac{A_{CHand}}{A_{CTotal}}\right)$	=	342.96 kN	77096.81 lb
Handrail Vertical Tension, Pv_Mhand	=	$Pv_{MTotal} \cdot \left(\frac{A_{CHand}}{A_{CTotal}}\right)$	=	87.84 kN	19747.15 lb
Handrail Horizontal Tension, Ph_Mhand	=	$Ph_{MTotal} \cdot \left(\frac{A_{CHand}}{A_{CTotal}}\right)$	=	331.52 kN	74524.95 lb
Coefficient of Friction, µ_Saddle,max	=	0.23		Given by Volume 2 of Bridge Binder	
Backstay Axial Tension, Pt_Bhand	=	$Pt_{MHand} \cdot e^{-\mu(\alpha_{Hand}+\theta_{Right}+0.04)}$	=	293.86 kN	66058.62 lb
Backstay Vertical Tension, Pv_Bhand	=	$Ph_{BHand} \cdot sin\alpha_{Hand}$	=	107.03 kN	24060.31 lb
Backstay Horizontal Tenison, Ph_Bhand	=	$Ph_{BHand} \cdot cos\alpha_{Hand}$	=	273.67 kN	61521.07 lb
Vertical Force on Single Column, Pv_Hand	=	$\frac{1}{2}(Pv_{BHand} + Pv_{MHand})$	=	97.44 kN	21903.73 lb
Horizontal Force on Single Column, Ph_Hand	=	$\frac{1}{2}(Ph_{BHand} + Ph_{MHand})$	=	28.92 kN	6501.94 lb

Determine Column Moment Capacity

Concrete Area, A_c	=	$W_{Eff} \cdot D_{Eff}$	=	0.28 m2	434.00 in2
Reinforcing Area, A_	=	No. of Bars $\cdot A_b$	=	402.12 mm2	0.62 in2
Depth to Reinforcing, d	=	$D_{Eff} - Cover$	=	0.59 m	23.11 in
Section Modulus, S	=	$W_{Eff} \cdot \frac{(D_{Eff})^2}{6}$	=	0.03 m3	1993.45 in3
Concrete Modulus of Rupture, f_r	=	$0.52 \cdot \sqrt{f'c}$	=	1.68 Mpa	243.08 psi
Cracking Moment, M_cr	=	$f_r \cdot S$	=	54.75 kN-m	40.38 k-ft
Nominal Moment Capacity, M_n	=	$A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right)$	=	63.11 kN-m	46.55 k-ft
a	=	$\frac{A_s \cdot F_y}{(0.85 \cdot f'c \cdot W_{Eff})}$	=	32.52 mm	1.28 in

Check Tower Moment Capacity

Unfactored Moment, M, M_DL	=	$Pv_{Hand} * E_{Saddleoffset} + Ph_{Hand} * H_{Column}$	=	24.96 kN-m	18.41 k-ft
Factor of Safety	=	2.28	>	1.5	OK
Reduction Factor	=	0.90			
Design Strength, Mn_factored	=	ϕM_n	=	56.80 kN-m	

Check Minimum Reinforcing

Factored Flexural Resistance, ϕM_n	>	Min(1.33*1.625*M,MCr)		
kN-m*		56.80 >	53.95	OK

Determine Forces Acting on Walkway

Walkway Axial Tension, Pt_Mwalk	=	$Pt_{MTotal} * \left(\frac{A_{CWalk}}{A_{CTotal}}\right)$	=	514.44 kN	115645.21 lb
Walkway Vertical Tension, Pv_Mwalk	=	$Pv_{MTotal} * \left(\frac{A_{CWalk}}{A_{CTotal}}\right)$	=	131.76 kN	29620.72 lb
Walkway Horizontal Tension, Ph_Mwalk	=	$Ph_{MTotal} * \left(\frac{A_{CWalk}}{A_{CTotal}}\right)$	=	497.28 kN	111787.42 lb
Coefficient of Friction, μ_Saddle,max	=	0.23		Given by Volume 2 of Bridge Binder	
Min Backstay Axial Tension, Pt_Bwalk	=	$Ph_{MWalk} * e^{-\mu(\alpha_{Walk}+\theta_{Right}+0.04)}$	=	453.29 kN	101899.49 lb
Min Bakcstay Vertical Tension, Pv_Bwalk	=	$Ph_{BWalk} * sin\alpha_{Walk}$	=	112.65 kN	25324.15 lb
Min Backstay Horizontal Tension, Ph_Bwalk	=	$Ph_{BWalk} * cos\alpha_{Walk}$	=	439.07 kN	98702.55 lb
Vertical Force on Walkway, Pv_Walk	=	$(Pv_{BWalk} + Pv_{MWalk})$	=	244.42 kN	54944.87 lb
Horizontal Force on Walkway, Ph_Walk	=	$(Ph_{BWalk} - Ph_{MWalk})$	=	58.21 kN	13084.87 lb
Angle of Resultant, δ_Walk	=	$tan^{-}(\frac{Ph_{Walk}}{Pv_{Walk}})$	=	13.40 deg	0.23 rad

Check Overturning about Toe of Tower

Overturning Moment	=	$(2 * Ph_{Hand} * H_{Tower}) + (Ph_{Walk} * H_{Walk} - .10)$	=	-110.05 kN-m	81.17 k-ft
Restorative Moment	=	$(2 * Pv_{Hand} + Pv_{Walk}) * \left(\frac{D_{Tower}}{2} + E_{saddleoffset}\right) + \left(P_{Tower} * \frac{D_{Tower}}{2}\right)$	=	224.33 kN-m	165.47 k-ft
Factor of Safety	=	2.04	>	1.5	OK

Check Column Eccentricity

Angle of Resultant	=	$\tan^{-1}(Ph_{hand}/Pv_{Hand})$	=	16.53 deg	0.29 rad
E_CableMax	=	$\frac{D_{Eff}}{2} + E_{Saddleoffset}$	=	0.45	
E_column	=	$H_{Column} * tanD_{Hand}$	=	0.36	
Check Column Eccentricity		0.36	<	0.45	OK

Right Foundation Analysis

Foundation Geometry

	Depth (m)	Width (m)	Height (m)	Tier Offset (m)	Volume (m3)	Weight (kN)	Weight (lb)	Y_Hand (m)	Y_Walk (m)
Tower	1.00	2.80	1.50	0.10	2.59	53.30	11981.84		
Tier1	1.65	3.00	1.00	0.18	4.95	96.93	21790.27		
Tier2	2.30	3.20	1.50	0.25	11.04	214.21	48154.05	4.00	2.90
Tier 3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Tier4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Cable Forces on Tower

Total Vertical Force on Columns, 2*Pv_hand	=	194.87 kN	43807.46 lb
Total Vertical Force on Walkway, Pv_walk	=	244.42 kN	54944.87 lb
Total Horizontal Force on Columns, 2*Ph_Hand	=	57.85 kN	13003.88 lb
Total Horizontal Force on Walkway, Ph_Walk	=	58.21 kN	13084.87 lb

Check Overturning about Toe of Foundation

Overturning moment = $(2 * Ph_{Hand} * Y_{Hand}) + (Ph_{Walk} * Y_{Walk})$ = -400.19 kN-m -295.18 k-ft

Restorative Moment = $(P_{Saddle} * X_{Saddle}) + \left(\sum P_{TierN} * X_{TierN}\right)$

		Force(P_Saddle)		Arm (X_Saddle)		Moment (kN-m)	Moment (k-ft)
Vertical Reaction	=	439.29 kN	x	1.40 m		615.01	453.63
		Force(P_Tier)		Arm (X_Tier)		Moment (kN-m)	Moment (k-ft)
Tower	=	53.30 kN	x	1.30 m	=	69.29	51.11
Tier1	=	96.93 kN	x	1.23 m	=	118.74	87.58
Tier2	=	214.21 kN	x	1.15 m	=	246.34	181.70
Tier3	=	0.00 kN	x	0.00 m	=	0.00	0.00
Tier4	=	0.00 kN	x	0.00 m	=	0.00	0.00
		Totals = 803.73		kN		1049.38	774.02

Factor of Safety = 2.62 > 1.5 OK

Check Bearing Pressure

Determine Effective Width of Foundation

Location of Resultant Force, B*/2	=	$\frac{\sum Moments}{\sum Vertical}$	=	0.81 m	2.65 ft	
Check Resultant Located>B/3	=	0.81	>	0.77	OK	
Effective width, B*	=	1.62 m		5.30 ft		
Bearing Pressure	=	$\frac{\sum Vertical}{B^* \text{x Width}}$	=	155.48 kN/m2		
Check Bearing Pressure	=	155.48 kN/m2	<	108 kN/m2		NG
Factor of Safety	=	0.69	>	1	NG	

Right Anchor Analysis

Determine Cable Forces on Anchor

Mainspan Cable Forces					
Pt_Mtotal	=			857.39 kN	192742.017 lb
Pv_Mtotal	=			219.61 kN	49367.8645 lb
Ph_Mtotal	=			828.79 kN	186312.369 lb
θ_right	=			14.84 deg	0.26 rad
Handrail Cable Forces					
Pt_Mhand	=			342.96 kN	77096.8067 lb
Pv_Mhand	=			87.84 kN	19747.1458 lb
Ph_Mhand	=			331.52 kN	74524.9478 lb
α_Hand	=			21.36 deg	0.37 rad
Coefficient of Friction μ_Saddle,max	=	0.15		For Anchor Analysis	
Backstay Axial Tension, Pt_Bhand	=	$Pt_{MHand} * e^{-\mu(\alpha_{Hand}+\theta_{Right}+0.04)}$	=	310.08 kN	69706.14 lb
Backstay Vertical Tension, Pv_BHand	=	$Pt_{BHand} * \sin\alpha_{Hand}$	=	112.94 kN	25388.84 lb
Bakstay Horizontal Tension, Ph_Bhand	=	$Pt_{BHand} * \cos\alpha_{Hand}$	=	288.78 kN	64918.04 lb
Walkway Cable Forces					
Pt_Mwalk	=			514.44 kN	115645.21 lb
Pv_Mwalk	=			131.76 kN	29620.7187 lb
Ph_Mwalk	=			497.28 kN	111787.422 lb
α_Walk	=			14.39 deg	0.25 rad
Backstay Axial Tension, Pt_Walk	=	$Pt_{MWalk} * e^{-\mu(\alpha_{Walk}+\theta_{Right}+0.04)}$	=	473.69 kN	106484.65 lb
Backstay Vertical Tension, Pv_Walk	=	$Pt_{BWalk} * \sin\alpha_{Walk}$	=	117.72 kN	26463.65 lb
Backstay Horizontal Tension, Ph_Walk	=	$Pt_{BWalk} * \cos\alpha_{Walk}$	=	458.83 kN	103143.86 lb
Total Cable Forces on Anchor					
Vertical Forces, Pv_Anchor	=			-230.66 kN	-51852.49 lb
Horizontal Force, Ph_Anchor	=			747.61 kN	168061.91 lb

Determine Earth Pressure on Anchor

Using Rankine Theory					
Internal Angle of Soil Friction, ϕ	=			30.00 deg	0.52 rad
Slope of Backfill, β	=			11.83 deg	0.21 rad
Unit Weight of Soil, Ɣs	=			1800.00 kg/m3	56.20 lb/cf
Height of Soil H_1	=			3.78 m	12.40 ft
Width, w	=			3.00 m	9.84 ft
Active Earth Pressure Coefficient, K_a	=	$\cos\beta\left(\frac{\cos\beta - \sqrt{(\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{(\cos^2\beta - \cos^2\phi}}\right)$	=	0.36	
Active Earth Pressure, P_Active	=	$\frac{1}{2}K_a * \gamma_s * H_1^2 * w$	=	134.81 kN	30305.76 lb
Summary of Anchor Forces					
Ramp Area 30 cm thick	=			8.83 m2	839.25 ft2
Ramp Area 50 cm thick	=			14.97 m2	2412.20 ft2
Ramp Area 70 cm thick	=			0.00 m2	0.00 ft2
Masonry Density	=			2100.00 kg/m3	
Backwall Area	=			0.48 m2	2.48 ft2
Backwall Width	=			3.00 m	9.84 ft
Concrete area	=			1.03 m2	11.42 ft2
Concrete width	=			3.00 m	ft
Concrete Density	=			2400.00 kg/m3	
Backwall Thickness	=			0.30 m	0.98 ft
Fill Area 1	=			17.23 m2	3195.51 ft2
Fill Thickness 1	=			2.40 m	7.87 ft
Fill Density (using Broken Rock density)	=			1900.00 kg/m3	
Undisturbed Soil Weight	=	$\gamma_s * A_{Soil\ Block} * W_{Soil\ Block}$	=	351.75 kN	79072.81 lb
Weight of Ramp, P_Ramp	=	$A_{Ramp} * w * \gamma_r$	=	1289.40 kN	289857.50 lb

Horizontal Forces

P_Active	=	134.81 kN	30305.76 lb
Ph_Anchor	=	747.61 kN	168061.91 lb
Ph_Tower	=	81.19 kN	18250.46 lb

Vertical Forces

Pv_Anchor	=	-230.66 kN	-51852.49 lb
Pv_Tower	=	450.27 kN	101220.36 lb
P_Abut	=	364.44 kN	81926.16 lb
P_Ramp	=	1289.40 kN	289857.50 lb
P_Anchor	=	114.66 kN	25775.57 lb

Check Anchor Sliding

Coefficient of Friction µ_Sliding	=	$\tan(\phi)$	=	0.58	
Total Horizontal Load, F_h Rs	=	$P_{Active} + Ph_{Anchor} + Ph_{Tower}$	=	963.60 kN	216618.12 lb
Total Vertical Load, N	=	$Pv_{Anchor} + Pv_{Tower} + P_{Abut} + P_{Ramp} + P_{Anchor}$	=	1988.11 kN	446927.09 lb
Total Frictional Resistance, F_f Rn	=	μN	=	1147.84 kN	258033.47 lb
R_Passive	=	$\frac{1}{2} * K_p * g * \gamma_s * (H_2^2 - H_1^2)$	=	321.06 kN	72174.29 lb
Factor of Safety	=	$\frac{F_f}{F_h}$			
Sliding Factor of Safety	=	1.52	>	1.5	OK

Check Anchor Uplift

H_2	=	3.00 m		9.84 ft	
w	=	3.00 m		9.84 ft	
b	=	1.4 m		4.59 ft	
B_1	=	$b + H_2 * \tan(30°)$	=	3.13 m	10.28 ft
Weight of Anchor Beam, P_AnchorBeam	=	114.78 kN		25801.87 lb	
Weight of Overburden, P_Overburden	=	288.97 kN		64960.01 lb	
Resisting Dead Load Force	=	$P_{AnchorBeam} + P_{Overburden}$	=	403.75 kN	90761.88 lb
Vertical Uplift Cable Force	=	Pv_{Anchor}	=	-230.66 kN	-51852.49 lb
Uplift Factor of Safety	=	1.75	>	1.5	OK

Summary of Additional Anchor Forces (Used in RHS)

Check Anchor Uplift

H_2	=	3.00 m		9.84 ft	
w	=	3.00 m		9.84 ft	
b	=	1.40 m		4.59 ft	
B_1	=	$b + H_2 * \tan(30°)$	=	3.13 m	10.28 ft
α (using autoCAD)	=	13.43 deg		0.23 rad	
Additional Area (using autoCAD)	=	1.60 m2		17.22 ft2	
Weight of Anchor Beam, P_AnchorBeam	=	114.66 kN		25775.57 lb	
Weight of Overburden, P_Overburden	=	378.34 kN		85051.74 lb	
Resisting Dead Load Force	=	$P_{AnchorBeam} + P_{Overburden}$	=	493.00 kN	110827.31 lb
Vertical Uplift Cable Force	=	Pv_{Anchor}	=	-230.66 kN	85544.74 lb
Uplift Factor of Safety	=	2.14	>	1.5	OK

Superstructure Analysis

Suspender Design Check

Live Load	=	4.23 kN/m		
Dead Load	=	0.91 kN/m		
Crosbeam Spacing, s	=	1.00 m	3.28 ft	
Gross Load per Crossbeam, P_beam	=	5.14 kN	1155.80 lb	
Net Load per Suspender	=	$0.5 * P_{Beam} * \frac{A_{CHand}}{A_{CTotal}} =$	1.03 kN	231.16 lb
Suspender Size	=	10.00 mm	#3	
Suspender Capacity	=	21.60 kN	4855.33 lb	
Suspender Factor of Safety	=	21.00 >	5	OKAY

Timber Decking Loading and Section Properties

Distributed load, W_LL	=	4.23 kN/m	65.58 psf	
Equestrian Load, P_LL	=	2.25 kN	505.80 lb	
Crossbeam Spacing, s	=	1.00 m	3.28 ft	
Width, b_deck	=	15.00 cm	5.91 in	
Thickness, t_deck	=	5.00 cm	1.97 in	
Moment of Inertia, I_deck	=	$b_{deck} * \frac{t_{deck}^3}{12} =$	156.25 cm4	3.75 in4
Section Modulus, S_deck	=	$\frac{I_{deck}}{t_{Deck}/2} =$	62.50 cm3	3.81 in3

Timber Decking Demand Stressed

M_Dist	=	$\frac{W_{LL} * s^2}{8} =$	0.11 kN-m	78.05 lb-ft
M_point	=	$\frac{P_{LL} * s}{4} =$	0.46 kN-m	337.09 lb-ft
V_dist	=	$\frac{W_{LL} * s}{2} =$	0.42 kN	95.15 lb
V_point	=	$\frac{P_{LL}}{2} =$	1.13 kN	252.90 lb
fb_dist	=	$\frac{M_{Dist}}{S_{deck}} =$	1.69 Mpa	245.57 psi
fb_point	=	$\frac{M_{point}}{S_{deck}} =$	7.31 Mpa	1060.59 psi
fv_dist	=	$\frac{3}{2} * \frac{V_{Dist}}{A_{Deck}} =$	0.08 Mpa	12.28 psi
fv_point	=	$\frac{3}{2} * \frac{V_{point}}{A_{Deck}} =$	0.23 Mpa	32.63 psi

Timber Decking Allowable Stresses

Adjusted Bending stress, F'_b	=	$F_b C_D C_M C_t C_L C_F C_{fu} C_i C_r C_c$		
Adjusted Shear Stress, F'_b	=	$F_v C_D C_M C_t C_i C_H$		
Allowable Bending Stress, F_b	=	3.96 Mpa		
Allowable Shear Stress, F_v	=	1.44 Mpa		
Load Duration Factor, C_D	=	1	(Distributed)	
	=	2	(Point/Impact)	
Bending Wet Factor, C_M	=	0.85	(Bending)	
	=	0.97	(Shear)	
C_t	=	1		
Stability Factor, C_L	=	1		
Size Factor, C_F	=	1.2		
Flat Use Factor, C_fu	=	1.15		
C_i	=	1		
Redundancy Factor, C_r	=	1		
Curvature Factor, C_t	=	1		
Shear Stress Factor, C_h	=	1		
F'_b	=	4.65 Mpa		702 psi
	=	9.29 Mpa		1123 psi
F'_v	=	1.69 Mpa		253 psi
	=	3.38 Mpa		405 psi

Check Timber Decking

Check F'_b >= f_b (Distributed Load)	4.65 MPa	>	1.69 MPa	OKAY
Check F'_b >= f_b (Point Load)	9.29 MPa	>	7.31 MPa	OKAY
Check F'_v >= f_v (Distributed Load)	1.69 MPa	>	0.08 MPa	OKAY
Check F'_v >= f_v (Point Load)	3.38 MPa	>	0.23 MPa	OKAY

Steel Crossbeam Allowable Stresses

Nominal Bending Moment, M_n	=	$F_y * Z$	<=	$1.6 * F_y * S_y$
Allowable Bending Moment, Mr	=	$\frac{M_n}{\Omega}$	=	$\frac{M_n}{1.6}$
Fy	=	240000 kPa		34809043.39 psi
Sy	=	3.77 cm3		0.23 in3
Z	=	8.14 cm3		0.50 in3
Mn	=	1.44768 kN-m		1068 lb-ft
Mr	=	0.9048 kN-m		667 lb-ft

Check Mr >= MDist (Distributed Load)	0.9048 kN-m	>	0.106 kN-m	OKAY
Check Mr >= Mdist (Point Load)	0.9048 kN-m	>	0.457 kN-m	OKAY

Timber Crossbeam Loading and Section Properties

Design for full live load, W_LL	=	4.07 kN/m2	85.0 psf		
Distributed load, W_LL_Beam	=	$\frac{5}{4} * DeckWidth * W_{LL}$=	4.57875 kN/m	313.7	lb/ft
Equestrian Load, P_LL	=	2.25 kN			
Cable Spacing, s_cable	=	1 m	3.28 ft		
Deck Width	=	0.90 m	2.95 ft		
Width, b_deck	=	15 cm	5.91 in		
Thickness, t_beam	=	10 cm	3.94 in		
Moment of Inertia, I_deck	=	$b_{deck} * \frac{t_{beam}^3}{12}$=	1250 cm4	30.03 in4	
Section Modulus, S_deck	=	$\frac{I_{deck}}{t_{Deck}/2}$=	250 cm3	15.26 in3	

Timber Crossbeam Demand Stressed

M_Dist	=	$\frac{W_{LLBeam}*s^2}{8}$=	0.57 kN-m	422.14 lb-ft
M_point	=	$\frac{P_{LL}*s}{4}$=	0.5625 kN-m	414.88 lb-ft
V_dist	=	$\frac{W_{LLBeam}*s}{2}$=	2.29 kN	514.7 lb
V_point	=	$\frac{P_{LL}}{2}$=	1.125 kN	252.9 lb
fb_dist	=	$\frac{M_{Dist}}{S_{deck}}$=	2.289 Mpa	332.05 psi
fb_point	=	$\frac{M_{point}}{S_{deck}}$=	2.25 Mpa	326.33 psi
fv_dist	=	$\frac{3}{2} * \frac{V_{Dist}}{A_{Deck}}$=	0.229 Mpa	33.20 psi
fv_point	=	$\frac{3}{2} * \frac{V_{point}}{A_{Deck}}$=	0.113 Mpa	16.32 psi

Timber Crossbeam Allowable Stresses

Adjusted Bending stress, F'_b	=	$F_b C_D C_M C_t C_L C_F C_{fu} C_i C_r C_c$		
Adjusted Shear Stress, F'_b	=	$F_v C_D C_M C_t C_i C_H$		
Allowable Bending Stress, F_b	=	3.96 Mpa		
Allowable Shear Stress, F_v	=	1.44 Mpa		
Load Duration Factor, C_D	=	1	(Distributed)	
	=	2	(Point/Impact)	
Bending Wet Factor, C_M	=	0.85	(Bending)	
	=	0.97	(Shear)	
C_t	=	1		
Stability Factor, C_L	=	1		
Size Factor, C_F	=	1		
Flat Use Factor, C_fu	=	1		
C_i	=	1		
Redundancy Factor, C_r	=	1		
Curvature Factor, C_t	=	1		
Shear Stress Factor, C_h	=	1		
F'_b	=	3.37 Mpa		702 psi
	=	6.73 Mpa		1123 psi
F'_v	=	1.22 Mpa		253 psi
	=	2.45 Mpa		405 psi

Check Timber Crossbeams

Check F'_b >= f_b (Distributed Load)	3.37 MPa	>	2.29 MPa	OKAY
Check F'_b >= f_b (Point Load)	6.73 MPa	>	2.25 MPa	OKAY
Check F'_v >= f_v (Distributed Load)	1.22 MPa	>	0.229 MPa	OKAY
Check F'_v >= f_v (Point Load)	2.45 MPa	>	0.113 MPa	OKAY

Construction Analysis

Hoisting Cable Force Analysis

Hoisting Sag (h_sag)	=	3.74 m	12.27 ft		
Additional rise while hoisting	=	0.10 m	0.33 ft		
h_SagAdjusted	=	$h_{Sag} - .5$	=	3.24 m	10.63 ft
b_hoistAdjusted	=	$\frac{h_{SagAdjusted}}{L}$	=	3.47 %	
Left Tower Cable Angle, θ_Left	=	$\tan^{-1}\left(\frac{4 * h_{Sag} \pm \Delta H}{L}\right)$	=	10.86 deg	0.19 rad
Right Tower Cable Angle, θ_Right	=	$\tan^{-1}\left(\frac{4 * h_{Sag} \pm \Delta H}{L}\right)$	=	14.84 deg	0.26 rad
Horizontal Tension, Ph_MTotal	=	$\frac{W_c * L^2}{8 * h_{Sag}}$	=	41.00 kN	9216.34 lb
Left Vertical Tension, Pv_MTotal	=	$Ph_{MTotal} * \tan(\theta_{Left})$	=	7.86 kN	1767.94 lb
Left Axial Tension, Pt_Mtotal	=	$\frac{Ph_{MTotal}}{\cos(\theta_{Left})}$	=	41.75 kN	9384.37 lb
Right Vertical Tension, Pv_MTotal	=	$Ph_{MTotal} * \tan(\theta_{Right})$	=	10.86 kN	2442.09 lb
Right Axial Tension, Pt_MTotal	=	$\frac{Ph_{MTotal}}{\cos(\theta_{Right})}$	=	42.41 kN	9534.39 lb
		Maximum Total Cable tension	=	42.41 kN	9534.39 lb
		Maximum Single Cable Tension	=	8.48 kN	1906.88 lb

Erection Hook Size	=	201 mm^2	#5	
Erection Hook Capacity	=	110.58 kN		
Erection Factor of Safety	=	13.04 >	3	OKAY