C0347 Cox's Walk Footbridge Assessment Report

Southwark HAPS

London Borough of Southwark

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Executive Summary

Cox's Walk Footbridge is located in Sydenham Hill Wood in Dulwich about 1km west of Forest Hill train station in the London Borough of Southwark. The structure is a 28m long, 3 span footbridge. Each span consists of a concrete deck supported by 3 longitudinal steel beams. The eastern and western abutments are constructed of brickwork. The two piers are of masonry construction. A triangular timber truss supports the parapets on the edges of each span and are connected by a timber cross beam above the midpoint of each span.

CONWAYAECOM (CA) was commissioned to carry out load assessment on all main structural elements including: the main deck; masonry piers; masonry abutments, and; timber truss elements, in order to determine the strength capacity and utilisation of the bridge elements and recommend feasible repairs options if necessary. The assessment has been carried out in accordance with the Approval in Principle (AiP), which was accepted by LBS on 9th March 2018 (refer document: 60493385-C0347-AIP-0001).

Assessment Results

Steel Deck Girders
The steel beams passed the assessment in accordance with BD 21/01 for 5kN/m^2 pedestrian live loading.

Precast Concrete Deck Panels
The reinforced concrete panels passed the assessment in accordance with BD 21/01 for 5kN/m^2 pedestrian live loading.

Masonry Abutments
The front and side abutment walls both fail the assessment in accordance with BD 21/01 due to lateral earth pressure load effects.

Timber Truss
All elements of the timber truss passed the assessment in accordance with BD 21/01 for 1.4kN/m lateral horizontal loadings from the bridge parapets.

Masonry Piers
The masonry piers passed the assessment in accordance with BD 21/01.

Slope Stability (At abutments and at bridge approaches)
The slope stability passed the assessment in accordance with BD 21/01.

The assessment methodology and results for each component can be found in Sections 2 and 3 of this report.

Repair Options
The following repairs options have been developed based on the results of the assessment and condition of the footbridge. An extensive feasibility study is not recommended for this structure as the proposed repair options are relatively minor providing the best value for money.

Steel Deck Girders
Repairs for the corroded ends of the steel beams at the abutments are shown in Appendix A. The full length of the steel beams should be treated and repainted.

Precast Concrete Deck Panels
The concrete deck panels at the two ends of the bridge should be modified to introduce reinforced concrete end screen walls at the abutments as shown in Appendix A. Minor spalling to the deck soffit around the steel hangers should be patch repaired such as polystyrene blocks or lightweight aggregate fill.

Because of the poor condition of the abutments, if the above recommended work is deferred then temporary propping of the abutments will be required.

Masonry Abutments
The wall sections should be reconstructed as shown in Appendix A. It is proposed that the wall sections are reconstructed with higher strength masonry and mortar. Tree roots need to be removed from behind the abutment walls. If required for an economic design, soil pressures on the abutment wall can be reduced by replacing the backfill with lighter weight material.

Masonry Piers
Minor cracks and spalling of masonry should be patch repaired.

Timber Truss
The timber truss is in an adequate condition and does not require any immediate attention but it is recommended that consideration be given to preservative treatment of the timber trusses and that inspections are carried as part of a standard inspection and maintenance programme. Noting the durability issues with timber, it is expected that the trusses will need to be replaced periodically.

Slope Stability (At abutments and at bridge approaches)
It is recommended that the slopes are inspected as part of the standard inspection and maintenance programme for the structure to report on any signs of distress or sliding of the slopes.

It is recommended that the proposed repair options be implemented if the footbridge is to remain in service.

1. Introduction

1.1 Description of the structure

Cox's Walk Footbridge is located in Sydenham Hill Wood in Dulwich crossing over what used to be the Crystal Palace and South London Junction Railway between Lordship Lane and Crystal Palace. The structure was built in 1865 but the timber deck was replaced sometime after the railway closed in 1956. The current structure is a 26m long, 3 span footbridge with each span about 8.6m long.

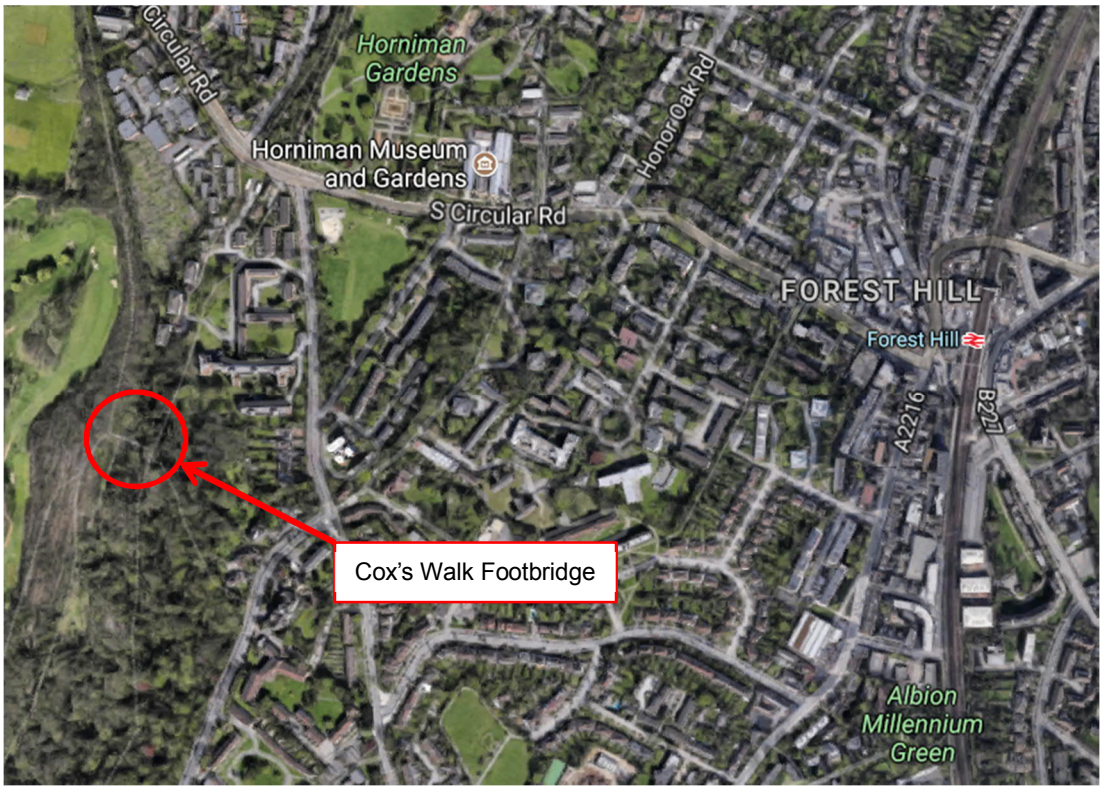


Figure 1: Location of Cox's Walk Footbridge from Google Maps

The deck is reinforced precast concrete panels supported by 3 rolled steel longitudinal beams. The concrete panels are approximately 2000mm longitudinally and 2950mm transversely across the width of the bridge. The centres of the steel beams are spaced 1.185m apart and are double braced in each span by transverse steel beams bolted to the main beams webs. At each end of each span the beams sit on stone bearing shelves which are supported on either brick piers or brick abutments. A timber truss frame on each side supports the bridge parapets. The timber truss extends from 1m below the deck, at each pier/abutment, up to 2.6m above the deck at the mid-span where a timber cross beam connects the trusses. The parapet consists of a welded and bolted steel lattice. The timbers trusses were replaced about 10 years ago.

The substructures consist of masonry wall or brick cladding abutments and two intermediate masonry pier supports. At the surface of the deck, at the two ends of the bridge, there are metal plates across the width of the bridge deck over the gap between the deck end and footbridge approach.

The soil below the eastern and western span slopes down at an angle of about 17˚ towards the ground below the central span. The front wall of the abutments bear onto concrete foundations below the underside of the steel beams.

1.2 Background

The inspection works undertaken by CA in February 2017 indicated cracks in the bridge abutments. Movement of bricks was also identified in the abutment walls. Hammer tapping of the bridge beams also identified a 5mm section loss to the bottom flange on the northern beam of the eastern span, close to the bearings

There are no record details on the construction, design elements, the materials used for the original bridge or the replacement of the deck.

A principal inspection was carried out in 2009 by Waterman Transport and Development which identified cracks in the abutments. A principal inspection was carried out in 2015 by CONWAYAECOM. (Cox's Walk Footbridge PI 2015) on BridgeStation which identified the same problems.

A Special Inspection (SI) (document: 60493385-C0347-REP-0002) was carried out in June 2017 in order to obtain information about construction form and foundations of the abutments as well as mortar tests to determine the mortar class, geotechnical testing to obtain relevant soil parameters, visual inspections and hammer tapping to determine the condition of the steel beams. A cover meter survey was also completed in April 2018 to determine the reinforcement arrangement in the concrete deck.

1.3 Purpose of document

CONWAYAECOM was commissioned to carry out a load assessment to all main structural elements including the main deck, the steel beams, the timber trusses, the masonry piers, the masonry abutments and slope stability in order to inform the recommended repairs and strengthening work. The load assessment has been carried out in accordance with the Approval in Principle (60493385-C0347-AIP-0001).

This document summarises the methodology for the structural analysis of the longitudinal steel beams, reinforced concrete deck, masonry abutments, masonry piers, timber trusses and slope stability, provides results obtained from these analysis and gives the conclusions and recommendations for each of the elements involved in the calculations.

2. Assessment Methodology

The structural assessment was made in accordance with the Approval in Principle, 60493385-C0347-AIP-0001.

2.1 Steel Beams

Longitudinal steel beams are simply supported in each span, and therefore the bending moments and shear forces are derived from simple static analysis. The central steel beam of the longest effective span has been analysed because it carries the largest load. Due to the torsional flexibility of the steel beams and compared to the rigidity of the concrete deck in transverse bending, torsion in the beams will be negligible and was not, therefore, included in the assessment.

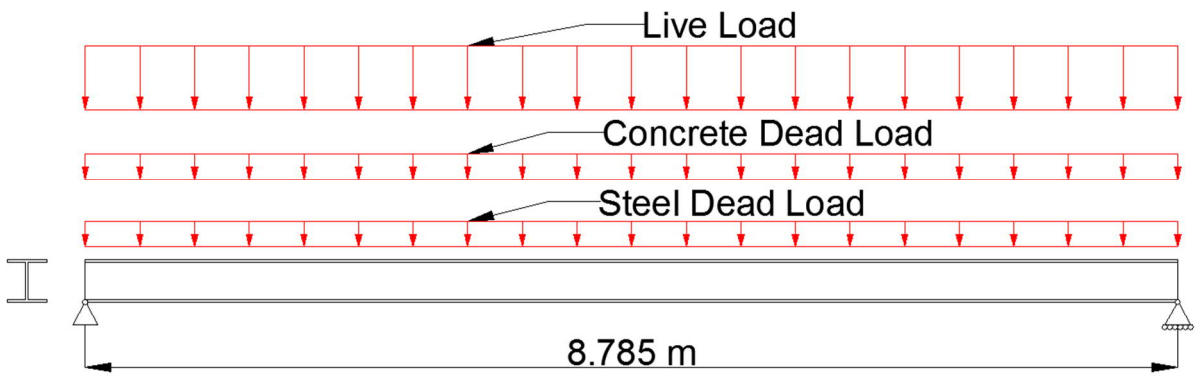


Figure 2: Loading to Centre Span Steel Beam

The steel beam was analysed with the 5mm section loss to in the inside of each flange observed in the Special Inspection Report (document: 60493385-C0374-REP-0002).

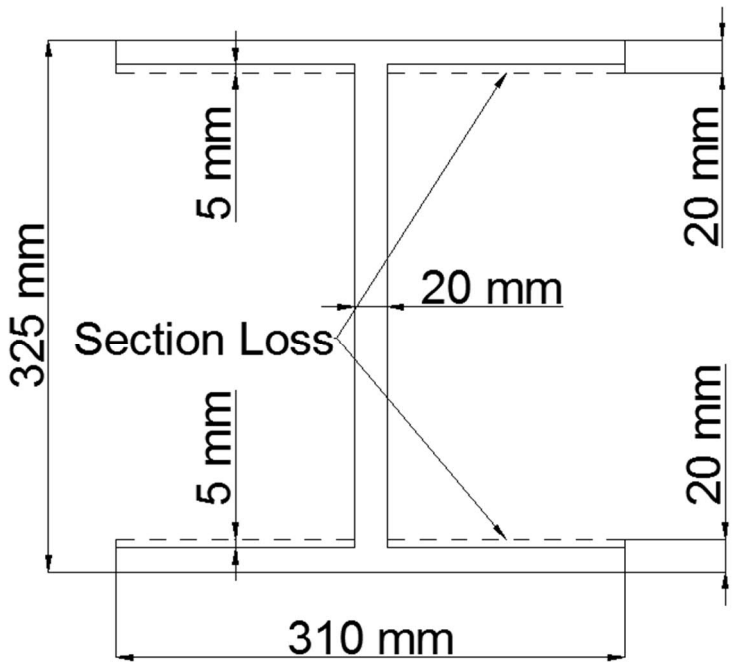


Figure 3: Cross section of steel beam with section loss

The loads applied on the steel beams are the sum of the following:

- Pedestrian live load. A pedestrian live load of 5 kN/m^2, according to BD 37/01, is applied to the surface of the bridge. The internal beam carries more loads than each of the external ones since they support a

greater width of the deck. The parapet load is not supported by the edge beams but by the timber truss. Therefore, the internal beam is more critical than the external ones;

- Concrete deck self-weight. The density of the concrete deck is assumed as 2400 kg/m^3 according to BD 21/01 Section 4.1 and the dimensions are taken from the Inspection for Assessment (IfA)(document: 60493385-C0347-REP-0001); and
- Steel beams self-weight. The density of the steel beams is assumed as 7850 kg/m^3 according to BD 21/01 Section 4.1. The dimensions of the beams are taken from the IfA.

Loads are applied with the partial load factors for Ultimate Limit State for Combination 1 according to BD 37/01. Partial Load factors for the loads considered for the steel beams assessment are given in Table 1.

Load	γ_{fL} for combination 1 ULS
Footbridge live load	1.50
Dead: concrete	1.15
Dead: steel	1.05

Table 1: Partial Load factors for Steel Beam assessment

Source: BD 37/01 Section 4.

Since CA considers the deck of the footbridge was probably constructed after 1956, a worst case scenario of steel material properties before 1955 from BD21/01 clause 4.3 has been used. Material strengths have been applied with partial factors for material strength (γ_m) according to BD21/01 Table 3.2.

Material strength	
Minimum yield stress	230 N/mm^2
γ_m	1.30

Table 2: Steel material properties

Source: BD21/01.

2.2 Concrete Deck

The concrete deck consists of precast concrete panels which are transversely continuous over the longitudinal steel beams. Each concrete panel is 2m long in the longitudinal direction and 2.95m across the width of the bridge. A transverse spanning strip of concrete deck 1m in the longitudinal direction was used for this assessment. The bending moments and shear forces are derived from hand calculations.

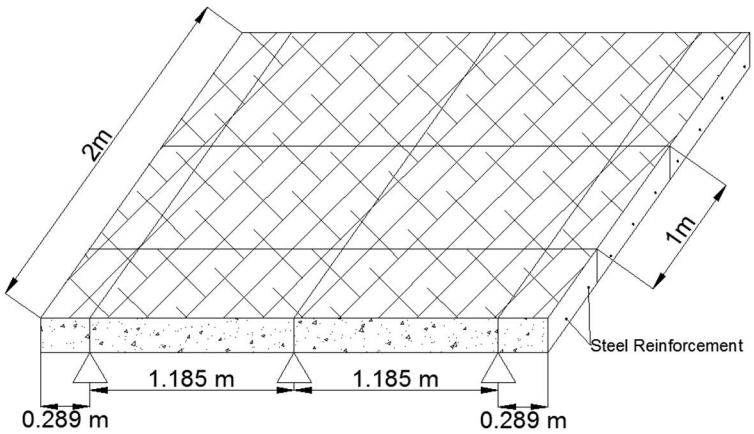


Figure 4: Concrete Panel

2.2.1 Loading Arrangements

The live load has been assessed in 4 different loading arrangements to determine the critical loading arrangement. The concrete dead load is applied in all cases.

The loads applied on the concrete panels are the sum of the following:

- Pedestrian live load. A pedestrian live load of 5 kN/m^2, according to BD 37/01, is applied on the effective area of each loading arrangement on the concrete panel; and
- Concrete deck self-weight. The density of the concrete deck is assumed as 2400 kg/m^3 according to BD 21/01 Section 4.1. This is applied to the full width of the concrete panel for all loading arrangements shown in Figure 5.

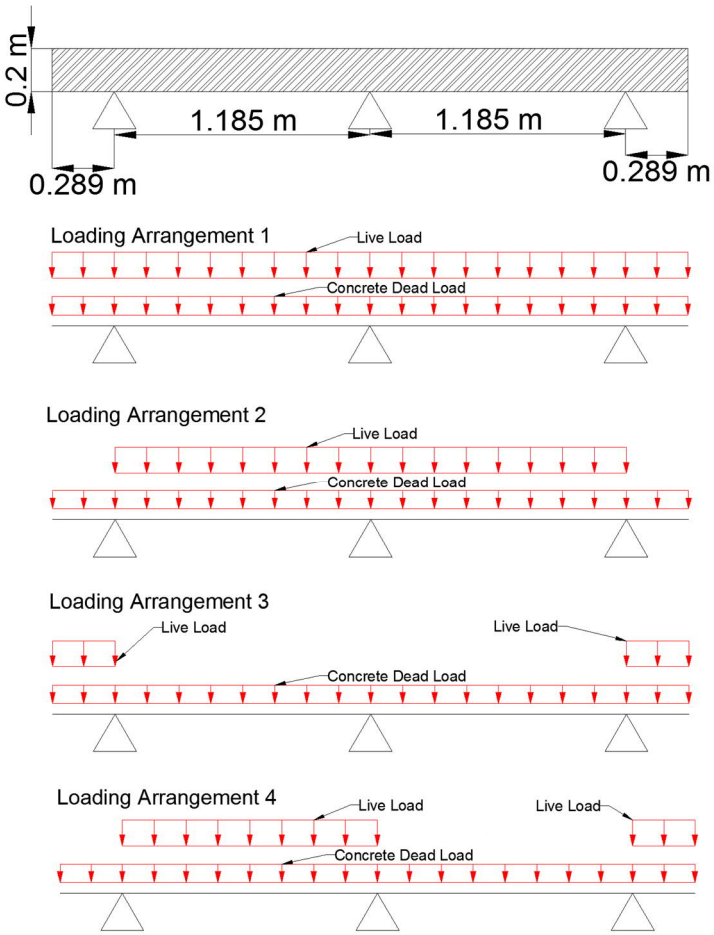


Figure 5: Loading Arrangements

The bending moment and shear stress of the critical loading arrangement has been compared with the calculated bending moment and shear force capacities of the RC panel.

Loads are applied with the partial load factors for Ultimate Limit State for Combination 1 according to BD 37/01. Partial Load factors for the live load and concrete dead load are given in Table 1.

2.2.2 Reinforced Concrete Material Properties

The size and distribution of reinforcing bars in the deck panels were measured by Ferroscan cover meter survey. The steel reinforcement bars were measured to have diameters between 6mm and 12mm, with centres

approximately 155mm apart and cover between 40mm and 70mm from the underside of the panels. Using standard imperial units and conservative estimates, the dimensions used in this assessment are: bar diameters ¼ inch (6.35mm), at 6 inch (152.4mm) centres with cover of 2 inches (50.8mm). The reinforcement arrangements are the same both longitudinally and transversely. A cross section of the concrete panel with steel reinforcement is shown in Figure 6.

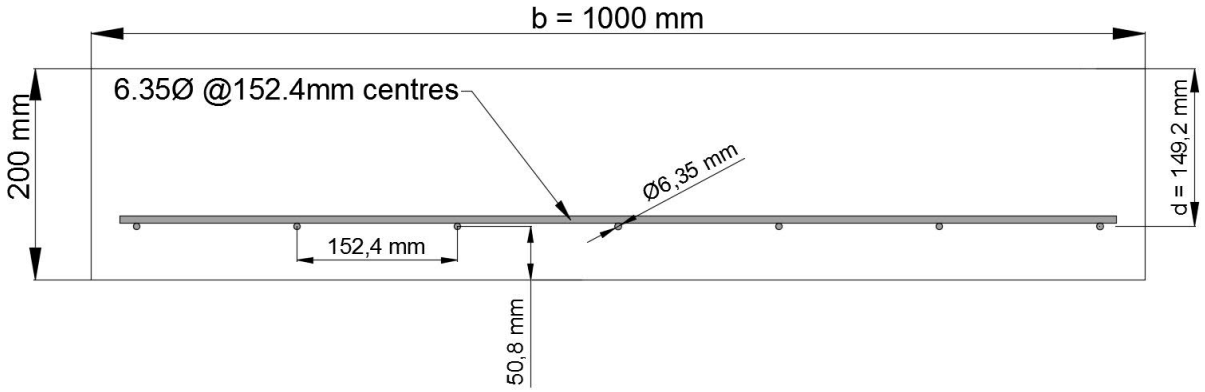


Figure 6: Concrete panel with reinforcement (a 1m length along the bridge)

Concrete cube strength has been taken from BSI CP 114 from 1969 which is now withdrawn but can be used as a reference for concrete that forms the deck of Cox's Walk Footbridge because the deck was constructed in the late 1960s. From this standard, the most conservative concrete was considered; this is, a concrete with a mix proportions of 1 part concrete to 2 parts fine aggregate to 4 parts course aggregate (1:2:4). Yield strength used for steel reinforcement is according to BD 21/01 for steel reinforcement pre-1961. Material strengths have been applied with partial factors for material strength (γ_m) according to BD21/01 Table 3.2.

Material strength

Concrete Cube Strength	21 N/mm^2
Steel reinforcement yield strength	230 N/mm^2
$\gamma_{m\ steel\ reinforcement}$	1.30
$\gamma_{m\ concrete}$	1.5

Table 3: Reinforced Concrete material properties

Source: BD21/01and BSI CP 114 .

2.3 Brickwork Abutments

To evaluate the effectiveness of repairing the abutments, they are assessed assuming that the current defects have been repaired.

The assessment was carried out considering the front abutment wall as cantilevering vertically from the base slab with horizontal "propping" support from the deck. Although it has not been established that there is a formal rigid connection between the steel beams and the abutment there is also no evidence of relative movement between these elements suggesting that some propping of the abutments by the beams is occurring. Noting, below, that as the abutment walls fail assessment even assuming propping they will need to be strengthened. (The utilisation of unpropped abutments would have been much greater, i.e. unpropped abutments would have had a greater overstress). The strengthening design should consider either propped or unpropped abutments consistent with the strengthening scheme adopted.

The side abutment walls were considered to cantilever vertically from the base slab with no horizontal "propping" supports. The dimensions have been obtained in the Special Inspection.

The loads considered for the analysis are shown in Figure 7, these are:

- Pressures from retained fill and pedestrian surcharge in accordance with BS 5628-1. The partial load factors are given in Table 4;
- Vertical forces acting on the stone bearing shelf as a result of the loaded transmitted by the steel beams to the abutments;
- Horizontal Water Pressure; CPT analysis found the water table to be at a depth of 2.5m below the surface of the footpath, and
- Surcharge of the surfacing on top of the front wall will also be taken into account when assessing the front wall.

Load	γ_{fL} for combination 1 ULS
Footbridge live load	1.50
Horizontal Earth pressure: retained fill	1.50
Deck Surfacing	1.75
Dead load (favourable)	1.0

Table 4: Partial Load factors for Abutment assessment from BD 37/01

Source: BD 37/01 Section 4.

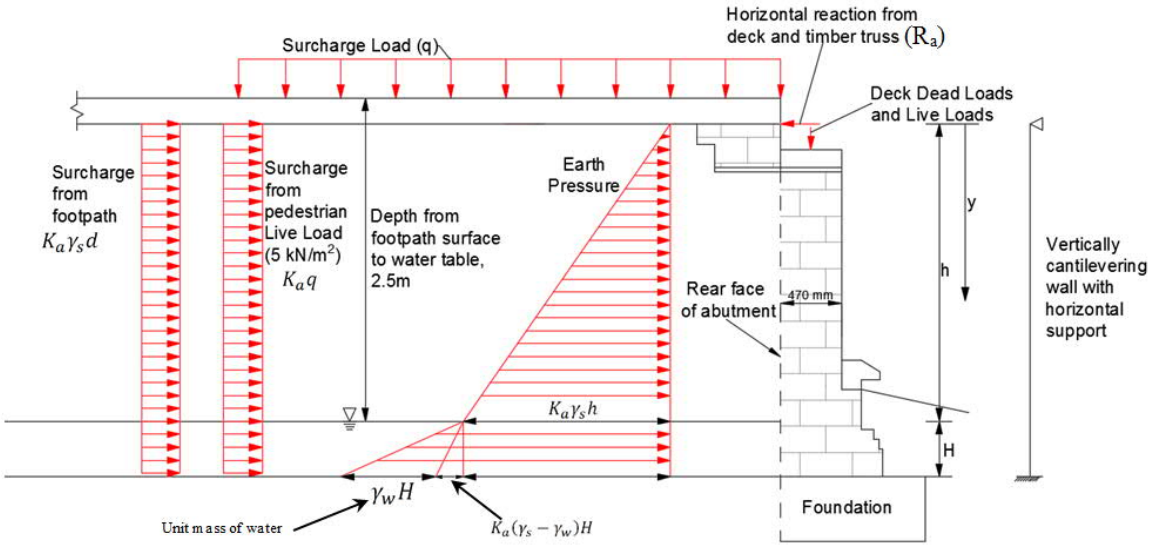


Figure 7: Front Abutment Wall Loads Sketch

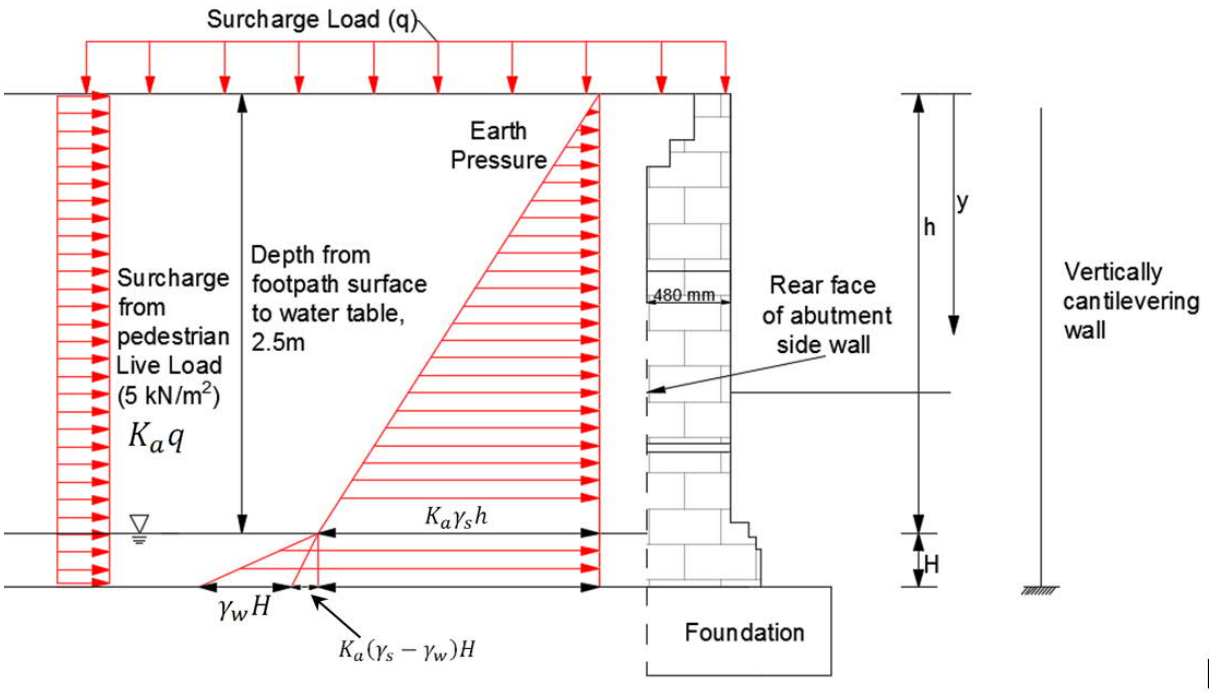


Figure 8: Side Abutment Wall Loads Sketch

Conservative values for the properties of the mortar strength class have been assumed. The characteristic compressive strength, f_k, and flexural strength, f_{kx}, of the masonry in accordance with Tables 2 & 3 of BS 5268-1 has been assumed. Material strengths have been applied with partial factors for material strength (γ_m) according to BD21/01 Table 3.2

Properties	Value
Mortar strength class	M6/(ii)
Characteristic compressive strength, f_k	2.5 N/mm^2
Flexural strength parallel to the bed joints, f_{kx}	0.3 N/mm^2
Flexural strength perpendicular to the bed joints, f_{kx}	0.9 N/mm^2
Characteristic initial shear strength of masonry, f_{vko}	0.15 N/mm^2
Partial factor for masonry material strength, γ_m	1.5

Table 5: Masonry material properties

The soil parameters used in this assessment are all from the Special Inspection Report:

- Friction Angle, $\phi' = 30^o$
- $K_a = 1 - Sin(\phi') / (1 + Sin(\phi'))$
- Unit mass of retained soil, $\gamma_s = 18$ kN/m^3

2.4 Timber Truss

A 3D finite element model of the timber truss has been analysed using Lusas Bridge software. The timber trusses are of a triangular shape connecting from the abutments and piers to a timber cross beam 2.5m above the centre of each span. The trusses are bolted to the outer steel beams to give lateral support. None of the deck dead

loads or live loads are transferred to the timber truss apart from lateral wind and pedestrian live loading from the steel mesh parapets.

Figure 9: Central span of the footbridge

For this analysis, the section of the timber truss considered for the load assessment is the central section because it has the longest effective span as mentioned in the AIP (document: 60493385-C0347-AIP-0001).

2.4.1 Loads and supports

For the Loads, Combination 1 from BD 37/01 has been incorporated to assess parapet loading and wind loading on the timber truss. Dead load of the timber truss, steel handrail and parapet mesh are also included in the assessment according to BD37/01.

Supports:

The bases of the timber frame are connected to the abutments and piers via pinned supports. The main horizontal timber beams are fixed in the transverse direction by bolts to the outer steel beams. The ends of the parapets are fixed transversely by triangular timber frames.

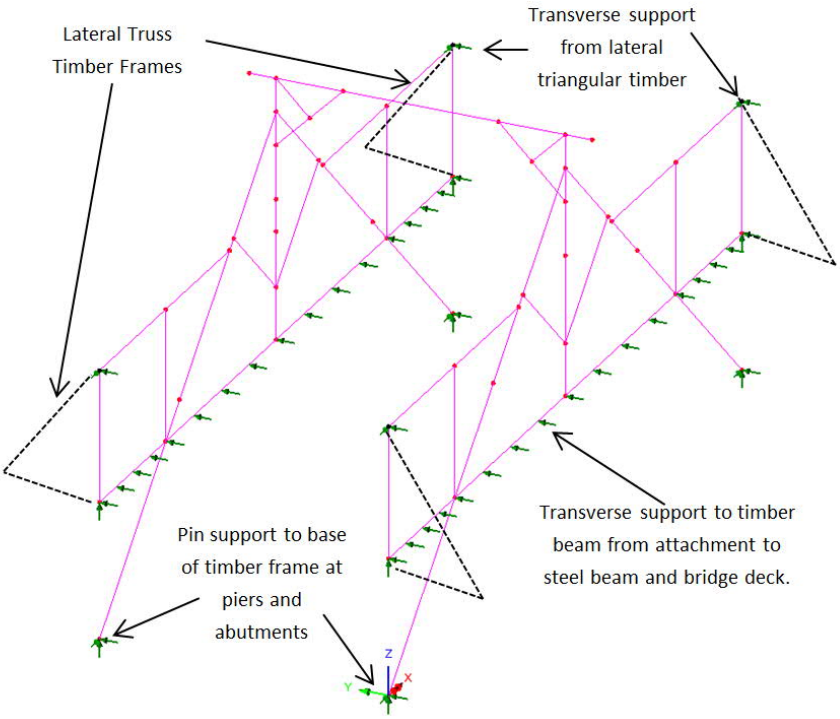


Figure 10: Lusas model of Timber Truss with supports

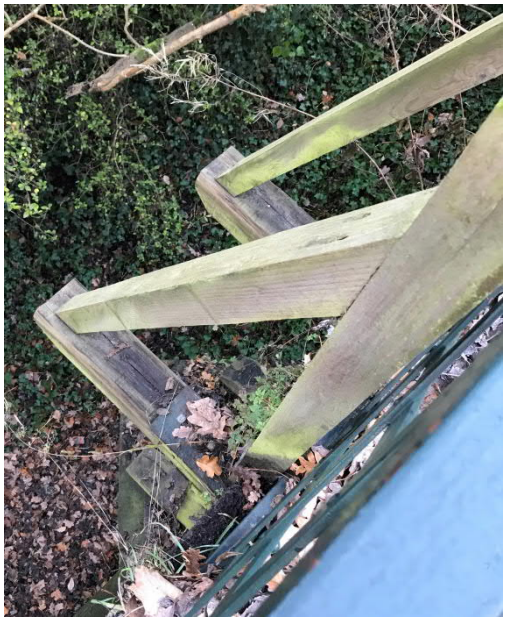


Figure 11: Photo of lateral triangular timber frames

Parapet Loads:

Vertical parapet loading of 1.4 kN/m has been applied to the steel handrails pushing outwards from the deck according to BD37/01 clause 7.1.2.

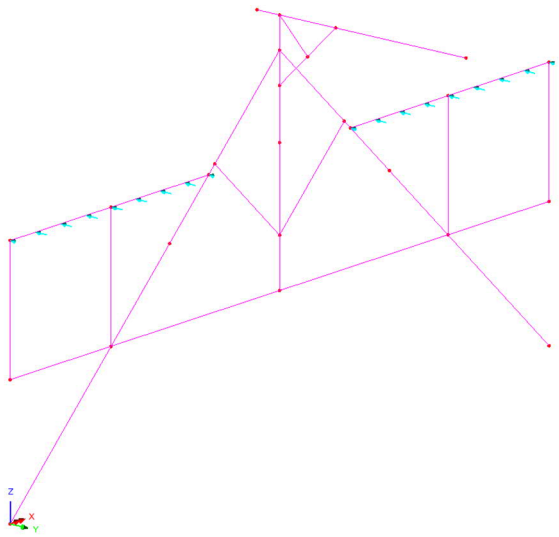


Figure 12: Wire model with parapet loading highlighted

Wind Loads:

Wind loading has been calculated according to BD37/01 clause 5.3. These loads are transmitted to the model of the timber truss as a combination of concentrated loads.

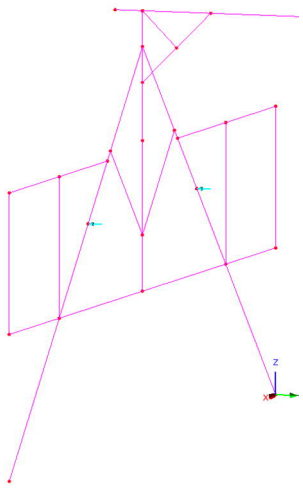


Figure 13: Wire model with wind Loading highlighted

Self-Weight:

Self-weight of the timber truss, steel handrail and mesh parapets has also been applied.

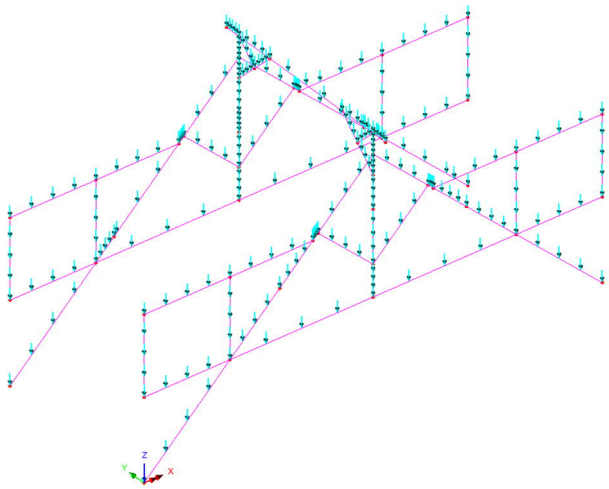


Figure 14: Self-weight of timber truss

The material properties of the timber are unknown so all will be assumed to be a low strength class of C16 according to BS EN 14081-1:2005. This is considered to be a conservative assumption. The resistance of timber is outside the scope of BD 21/01 and is therefore assessed to BS 5268-2. This is a permissible stress code and therefore no partial load factors or material factors are required.

Material properties	Timber stresses
Bending parallel to grain	5.3 N/mm^2
Compression parallel to grain	6.8 N/mm^2
Tension parallel to grain	3.2 N/mm^2
Shear parallel to grain	0.67 N/mm^2

Table 6: C16 Timber material properties

2.5 Masonry Piers

A qualitative assessment of the piers has been undertaken. This assessment involved Compression and bending due to eccentricity.

The Loads applied on each pier for the compression assessment are the following:

- The pedestrian live load and concrete dead load for an area of the deck covering approximately 8.8m longitudinally and 2.95m transversely;
- The steel beam dead load for 3 steel beams approximately 8.8m long with no section loss; and
- The timber truss dead load applied to the stone shelf either side of the masonry wall.

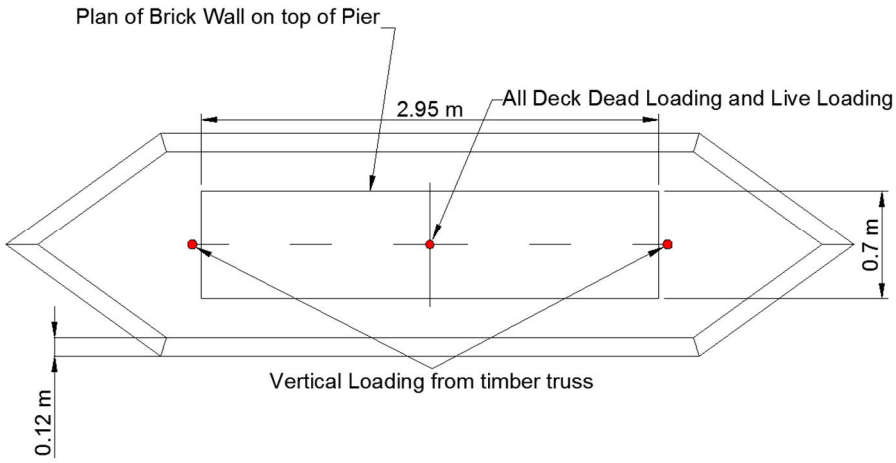


Figure 15: Pier Plan View with compression loads

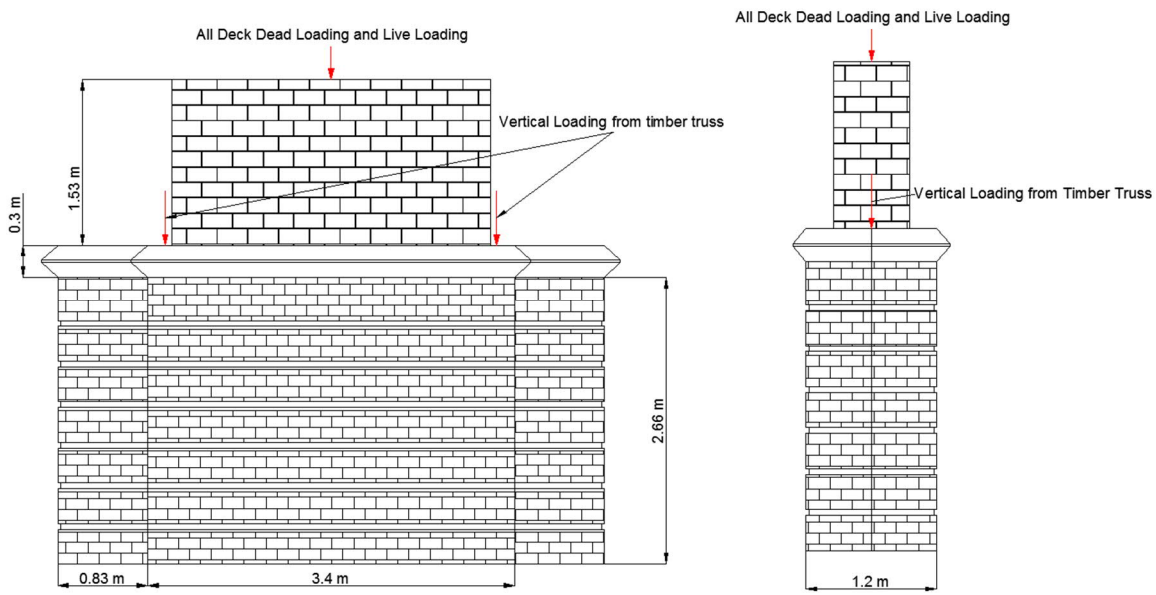


Figure 16: Pier Elevation and End Elevation Views with compression loads

An eccentricity of 10% of the plan dimensions of the top of the pier will be considered. The full compressive load will be applied to the top of the pier 70mm from the pier centre parallel to the direction of the deck and 300mm perpendicular to the deck.

The Loads applied on each pier for the bending assessment due to eccentricity are the following:

- The pedestrian live load, concrete dead load and steel beams dead load will be the same as above but applied 70 mm longitudinally and 300 mm transversely from the centre of the pier as shown in Figure 17; and
- A lateral shear force transferred from the timber truss has also been applied perpendicular to the deck due to pedestrian live load and wind effects.

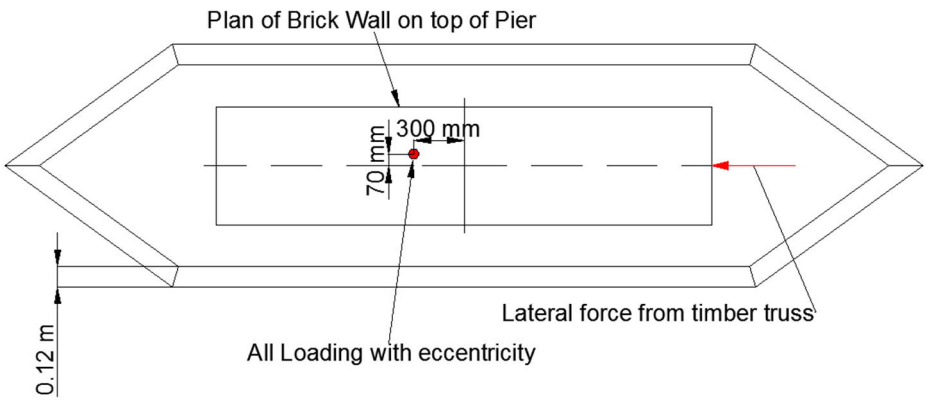


Figure 17: Pier Plan with bending due to Eccentricity

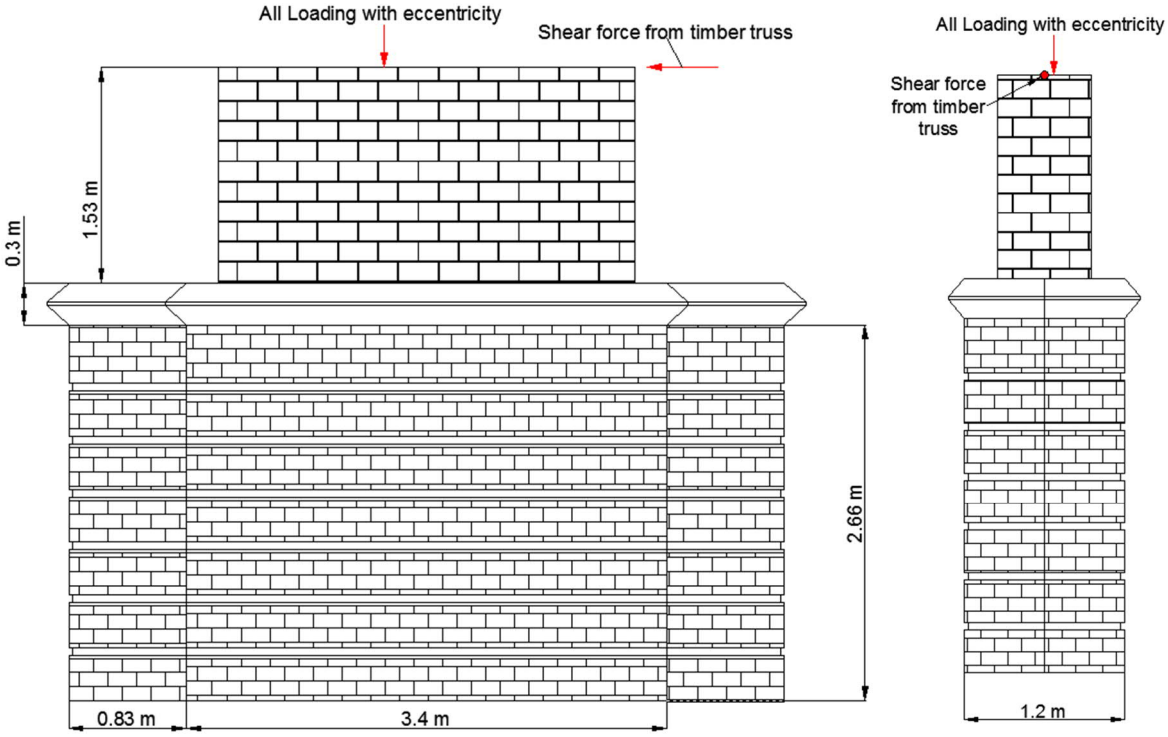


Figure 18: Pier Elevation and End Elevation with bending due to Eccentricity

Partial load factors Table 1. Material load factors and strengths in Table 5.

Loads are applied with the partial load factors for Ultimate Limit State for Combination 1 according to BD 37/01. Partial Load factors for the loads considered for the masonry pier assessment are given in Table 1. Material strengths and material partial factors are shown in Section 2.3 Table 5.

2.6 Slope Stability

Stability of the soil sloping from the western abutment has been assessed using Bishops Method with GeoStudio software SLOPE/W. Analysis will provide with the Factor of Safety (FoS) for any deep-seated slope failure. Global stability refers to a failure mechanism not involving structural capacity, but exclusively ground resistance. This is typically the case when a slip surface is formed in a volume of soil incorporating, but not intersecting, an existing structure. The limit equilibrium Software GeoStudio SLOPE/W will be used to calculate the factor of safety corresponding to the ultimate limit state.

3. Assessment Results

3.1 Steel Beams

3.1.1 Bending

Table 7 shows the bending moments and utilisation ratios for the critical steel beam with and without section loss. Design bending moment refers to the moment applied by the loading and resistance bending moment refers to the bending resistance of the steel section.

Steel Beam	Full Steel Section	With 5mm Section Loss
Combined Load	16.77 kN/m	16.77 kN/m
Maximum Design Bending Moment	161.8 kNm	161.8 kNm
Bending Moment Resistance	406.4 kNm	332.0 kNm
Bending Utilisation	0.40	0.49

Table 7: Bending Utilisation of steel beams

Analyses of the critical steel beam with and without section loss have bending utilisation ratios under 1.00 and thus, comply with the required strength for an Ultimate Limit State analysis.

3.1.2 Shear

Below are the shear utilisation ratios for the critical steel beam.

Steel Beam	Steel Section
Maximum Design Shear Force	73.66 kN
Shear Force Resistance	655.33 kN
Shear Utilisation	0.11

Table 8: Stress Utilisation of steel beams

Analyses of the critical steel beam have utilisation ratio under 1.00 and thus, complies with the required strength for an Ultimate Limit State analysis.

3.2 Concrete Deck

3.2.1 Loading Arrangements

Table 9 shows the maximum bending moment for each of the concrete loading arrangements.

Loading Arrangement	Maximum Bending Moment
1	2.00 kNm
2	2.15 kNm
3	0.68 kNm
4	1.48 kNm

Table 9: Concrete Loading Arrangement Results

Loading arrangement 2 is the critical case and has been used to calculate the utilisation ratios in the concrete reinforcement section.

3.2.2 Strength

The minimum reinforcement dimensions from section 2.2.2 gives the resistance bending moment shown in Table 10. The design bending moment is the maximum bending moment of critical loading arrangement 2.

Reinforced Concrete Deck	Bending Moment
Bending Moment Resistance	4.86 kNm
Maximum Design Bending Moment	2.15 kNm
Bending Utilisation	0.44

Table 10: Maximum bending moments of the reinforced concrete

Analyses of the concrete panels have a bending utilisation ratio under 1.00 and thus, they comply with the required strength for an Ultimate Limit State analysis.

Analyses of the concrete panels have a stress utilisation ratio under 1.00 and thus, they comply with the required strength for an Ultimate Limit State analysis.

3.3 Brickwork Abutments (assuming current defects repaired)

The front wall and wing walls of the brickwork abutment has been analysed for compressive, shear and bending stresses in accordance with combination 1 partial load factors from BD 37/01, using the applied loads stated in Section 2.3. Analysis has been done assuming a repaired face of the abutment front wall. Results for the front wall are shown in Table 11. Results for the side walls are shown in Table 12.

Stress Type	Combination	Assessment Loading	Design Resistance	Utilisation	Pass/Fail
Compressive	Combination 1 ULS	0.12 N/mm^2	1.7 N/mm^2	0.07	Pass
Shear	Combination 1 ULS	0.019 N/mm^2	0.17 N/mm^2	0.11	Pass
Bending	Combination 1 ULS	11.1 kNm	7.4 kNm	**1.51**	**Fail**

Table 11: Abutment assessment results for front wall cantilevering from base slab with horizontal support from the deck

Stress Type	Combination	Assessment Loading	Design Resistance	Utilisation	Pass/Fail
Compressive	Combination 1 ULS	0.39 N/mm^2	1.7 N/mm^2	0.23	Pass
Shear	Combination 1 ULS	0.063 N/mm^2	0.17 N/mm^2	0.37	Pass
Bending	Combination 1 ULS	26.6 kNm	7.7 kNm	**3.46**	**Fail**

Table 12: Abutment assessment results for side walls cantilevering from base slab with no horizontal support

The front and side abutment walls fail in bending.

3.4 Timber Truss

3.4.1 Finite Element Analysis

Table 13 shows the bending utilisation, load bending moments and resistance bending moments for all elements of the timber truss. Figure 19 shows what parts of the truss each element name refers to.

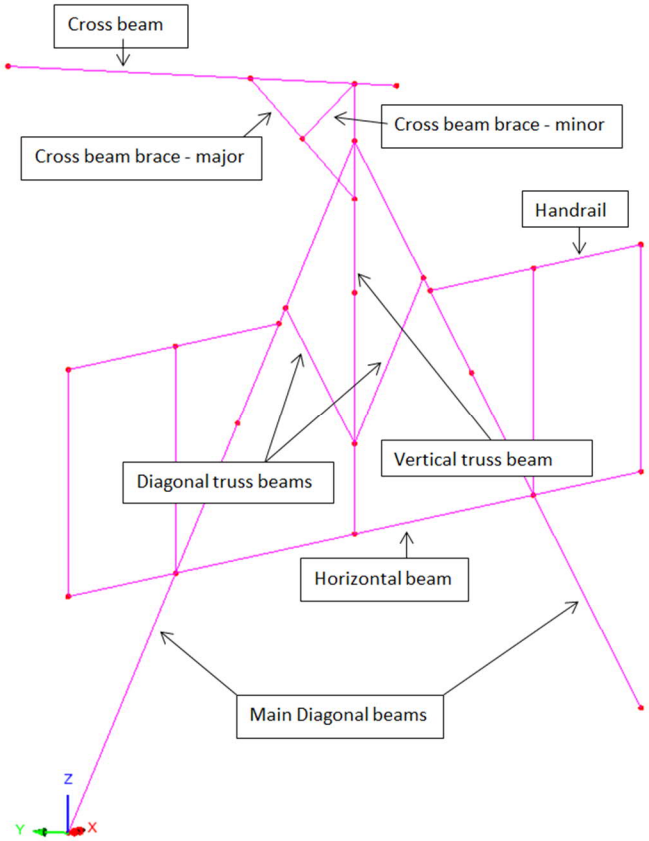


Figure 19: Labelled diagram of the timber truss elements

Beam elements	$M_{pl,Rd}$ (kNm)	My	Bending Utilisation
Horizontal beam	16.7	0.97	0.06
Main Diagonal Beams	7.3	3.00	0.41
Diagonal truss beams	4.2	1.53	0.36
Vertical truss beam	4.2	1.53	0.36
Cross Beam	6.5	1.76	0.27
Cross beam brace – major	1.3	0.29	0.22
Cross beam brace – minor	0.57	0.02	0.04

Table 13: Bending Utilisation of Timber Truss Elements

Analyses of the timber truss show all elements have a bending utilisation ratio under 1.00 and thus, they comply with the required strength for an Ultimate Limit State analysis.

Table 14 shows the bending utilisation, load bending moments and resistance bending moments for all elements of the timber truss.

Beam elements	$N_{pl,Rd}$ (kN)	Fx	Stress Utilisation
Horizontal beam	29.1	2.90	0.10
Main Diagonal Beams	20.5	7.11	0.35
Diagonal truss beams	12.5	6.53	0.52
Vertical truss beam	12.5	6.53	0.52
Cross Beam	19.4	5.29	0.27
Cross beam brace – major	6.7	5.74	0.86
Cross beam brace – minor	3.8	0.87	0.23

Table 14: Shear Stress Utilisation of Timber Elements

Analyses of the timber truss show all elements have a stress utilisation ratio under 1.00 and thus, they comply with the required strength for an Ultimate Limit State analysis.

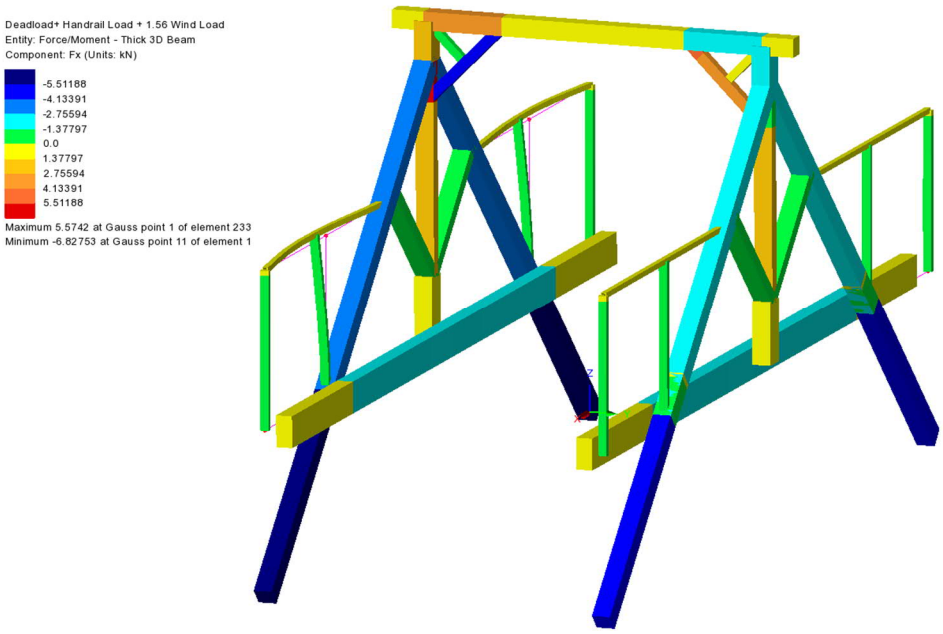


Figure 20: Lusas model of timber truss

3.5 Masonry Piers

The masonry piers have been analysed for compressive and bending in accordance with combination 1 partial factors from BD 37/01, using the applied loads stated in Section 2.5. Results for the piers in compression are shown in Table 15 and results for the piers in bending due to eccentricity are in Table 16.

Masonry Pier	Stress Utilisation
Total Compressive Load	397 kN
Design Resistance load	3442 kN
Utilisation	0.12

Table 15: Utilisation of Pier in Compressive analysis

Bending due to eccentricity	Eccentricity parallel to deck	Eccentricity perpendicular to deck
Eccentricity	70 mm	300 mm
Total Bending moment	26.1 kNm	118.7 kNm
Design Resistance Moment	48 kNm	204 kNm
Utilisation	0.54	0.58

Table 16: Utilisation of Pier in bending analysis

Analyses of the masonry pier have compressive and bending utilisation ratios under 1.00 and thus, comply with the required strength for an Ultimate Limit State analysis.

3.6 Slope Stability

Global stability of the slope has been analysed using Bishops Method in SLOPEW for slope stability. Four different models were created, one through each of the abutments and one adjacent to each of the abutments. The slip surfaces that intersected elements of the footbridge (i.e. abutment walls or abutment or pier foundations) were discarded. The result for the Factor of Safety for the most critical slip surfaces can be found in Table 17.

Table 17: Results for Global Stability Analysis

Section	FoS	Figure Reference
East Abutment	1.31	Figure 21
East slope	1.06	Figure 22
West Abutment	1.21	Figure 23
West Slope	1.12	Figure 24

Table 18: Results for Global Stability Analysis

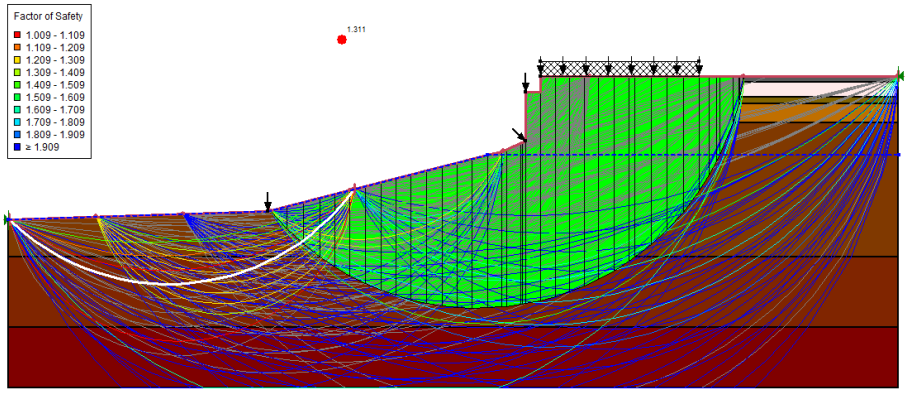


Figure 21: Critical Slip Failure Surface for East Abutment

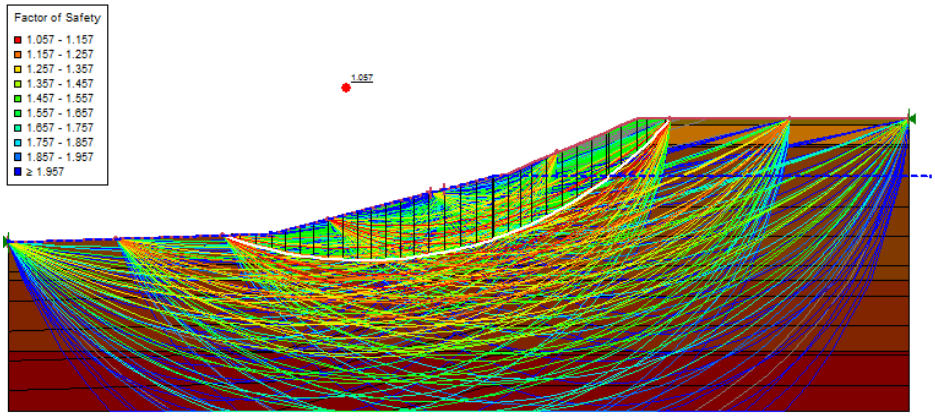


Figure 22: Critical Slip Failure Surface for East slope

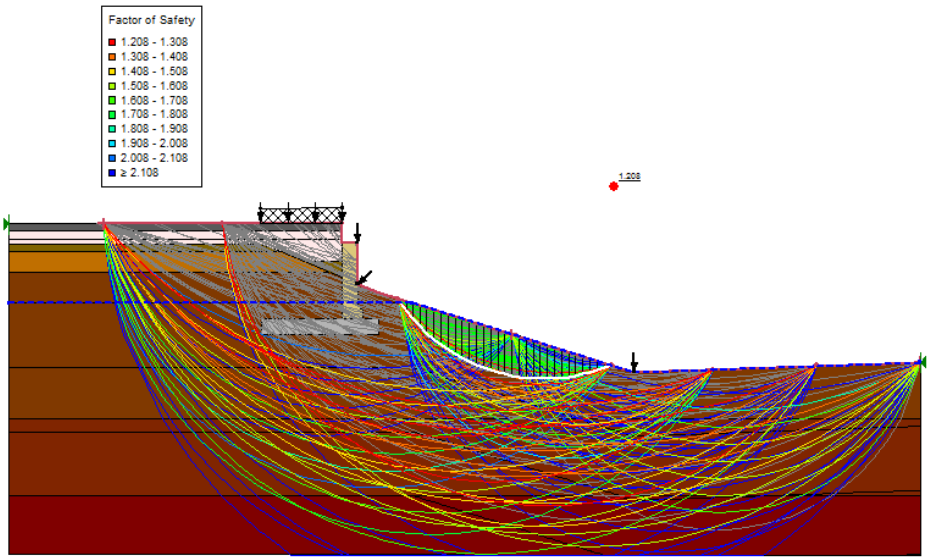


Figure 23: Critical Slip Failure Surface for West Abutment

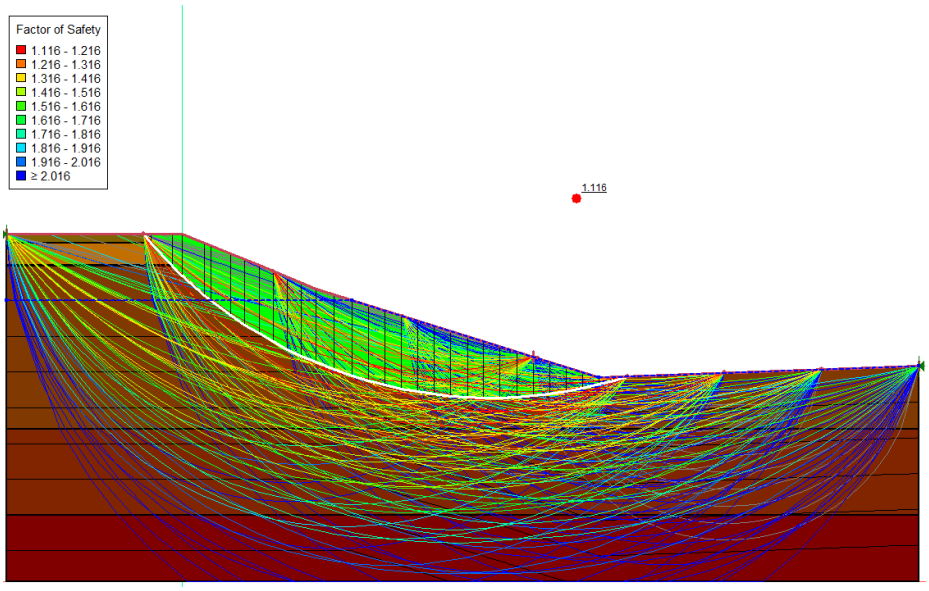


Figure 24: Critical Slip Failure Surface for West slope

All factors of Safety (FoS) are above 1.0 for the parameters considered; and thus, there is no risk associated with global stability failure at both slopes of the footbridge.

4. Summary

4.1 Steel Beams

The steel beams pass the assessment. Treating and repainting is required to resist further corrosion as well as the introduction of waterproofing/drainage system to prevent moisture build up on steel girders at supports.

4.2 Concrete Deck

The concrete deck passes the assessment.

Minor spalling to the deck soffit around steel hangers in the deck should be repaired. Extending the concrete deck slab to form an end screen beyond the ends of the steel beams at the abutments would improve the durability of the structure.

4.3 Brickwork Abutments

As shown in Table 11 and Table 12, the assessment concludes that the front and side abutment walls cannot withstand the earth pressure and surcharge loadings even if they were repaired. The utilisation ratio shows that the bending stresses from lateral pressures are approximately 1.5 times the brickwork capacity for the front abutment wall and approximately 3.5 times the brickwork capacity for the side abutment walls.

Figure 25: Cracking in western abutment south wall

In the SI Report (document: 60493385-C0347-REP-0002) a high presence of roots was found in the backfill of the abutments (Figure 26). It is likely a combination of rooting in the backfill and over utilisation of the abutment walls has resulted in the defects observed during the 2015 Principal Inspection and Inspection for Assessment Report.

Figure 26: Rooting in the backfill of abutment observed during SI Report

4.4 Timber Truss

All elements of the timber truss pass the assessment.

It is recommended that inspections are carried as part of a standard inspection and maintenance programme and that consideration is given to giving the timber preservation treatment.

4.5 Masonry Piers

The piers pass the assessment.

Minor cracks with spalling masonry probably due to freeze thaw was observed in the 2015 Principal Inspection and Inspection for Assessment but this is not significant enough to have any effect on the structural integrity. This should be repaired.

4.6 Slope Stability

The results from the global stability calculations resulted in factors of Safety above 1.0, meeting the loading requirements. Although the obtained values are close to failing according to the calculations, slopes have had the same conditions for the last 150 years, and are currently more vegetated that in the past, so improved equilibrium conditions prevail. Given that the repair works are not going to affect the foundations or the live loads applied, there is no need of strengthening the slopes adjacent to the structure.

5. Recommendations

The general, Cox's Walk footbridge is in fair condition. Based on the condition of the footbridge a feasibility study is not recommended. It is recommended that the following repairs be designed and built. The proposed repairs provide the best value for money ensuring the footbridge can remain in service for the duration of its intended design life. Sketches for each repair option can be found in Appendix A.

5.1 Steel Beams and Concrete Deck

The proposed deck end screens in Appendix A would reduce further corrosion and water damage to the ends of the steel beams. The steel beams should be treated and repainted.

The minor spalling to the concrete deck soffit around the steel hangers is not significant enough to have any effect to the footbridge stability but should be patch repaired to prevent reinforcement corrosion.

5.2 Brickwork Abutment

The front and side abutment walls failed the assessment and require strengthening.

Proposed abutment wall sections of both abutments to be reconstructed are shown in Appendix A. It is proposed that the wall sections are reconstructed with higher strength masonry and mortar to increase their bending resistance. Soil pressure from behind the abutment wall can be reduced by replacing the backfill with lighter weight material. This would reduce the bending moments applied to the walls.

Reconstruction of the abutment walls would require temporary propping of deck beams.

Damage to the abutment walls is also caused by roots observed in the Special Inspection Report (604393385-C0347-REP-0002). Tree roots in the backfill need to be removed from behind the abutment walls to prevent further deterioration. This would provide an opportunity to replace the backfill with a lightweight material if required.

A drainage channel should be installed in the front abutments wall during reconstruction to prevent deterioration to the rear of the abutment walls.

In the short term, in advance of abutment wall reconstruction, it is recommended that temporary horizontal propping is installed to prevent further cracking and possible displacement of the abutments.

5.3 Timber Truss

The timber truss is in an adequate condition and does not require any immediate attention but it is recommended that inspections are carried as part of a standard inspection and maintenance programme. Preservative treatment of the timber should be considered.

5.4 Masonry Piers

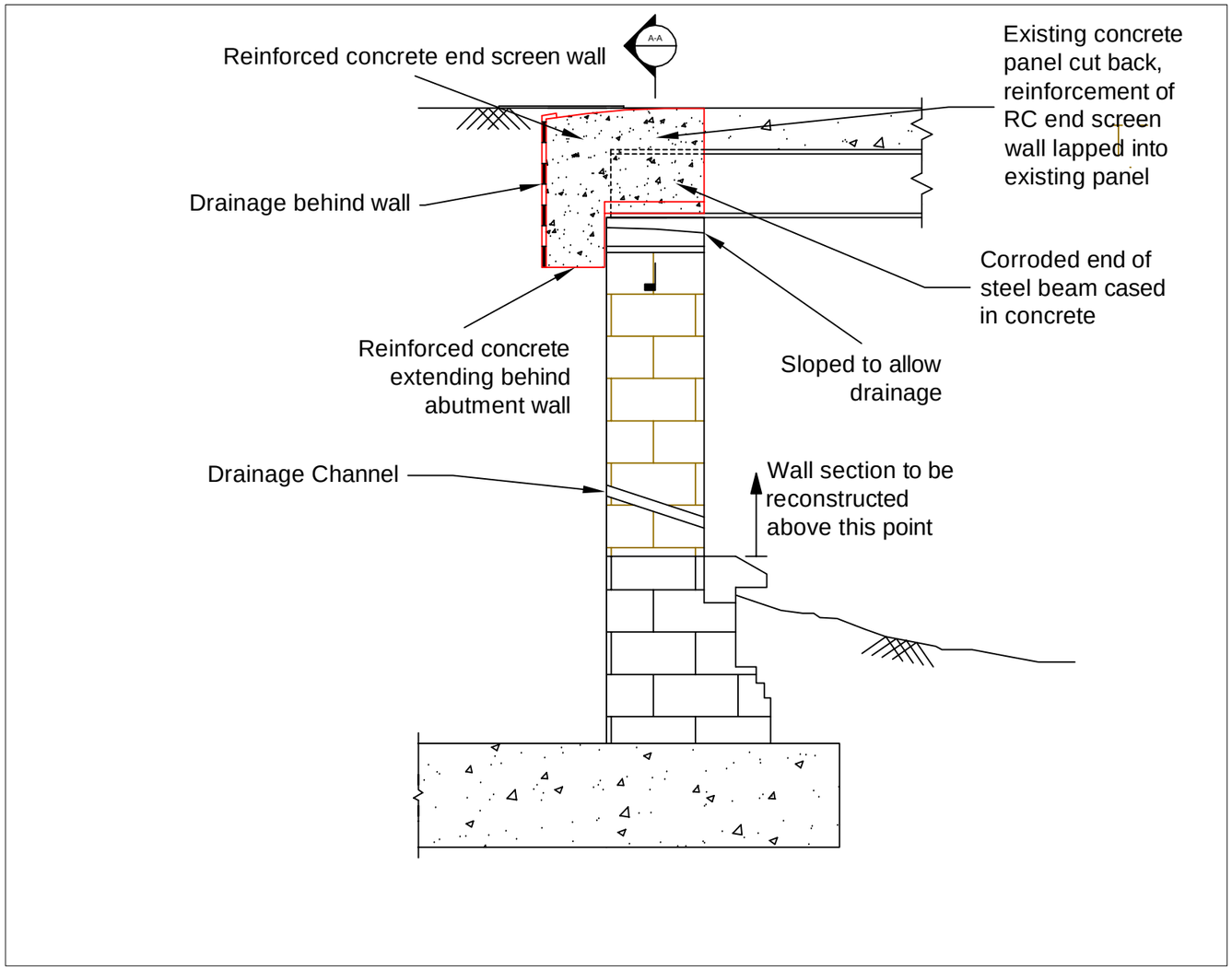
Minor cracks and spalling of masonry should be patch repaired.

5.5 Slope Stability

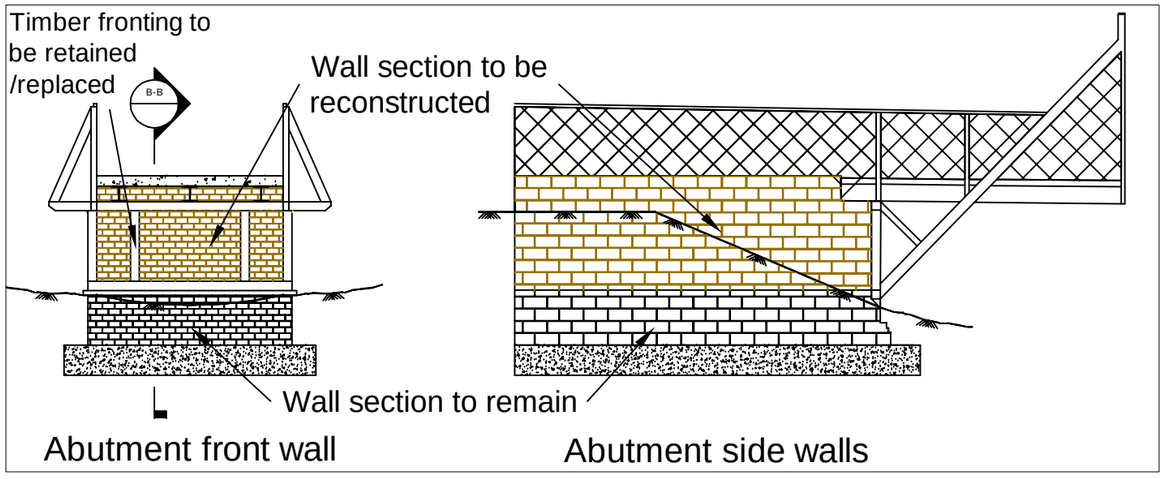
It is recommended that the slopes are inspected as part of the standard inspection and maintenance programme for the structure to report on any signs of distress or sliding of the slopes.

Appendix A Repair Option

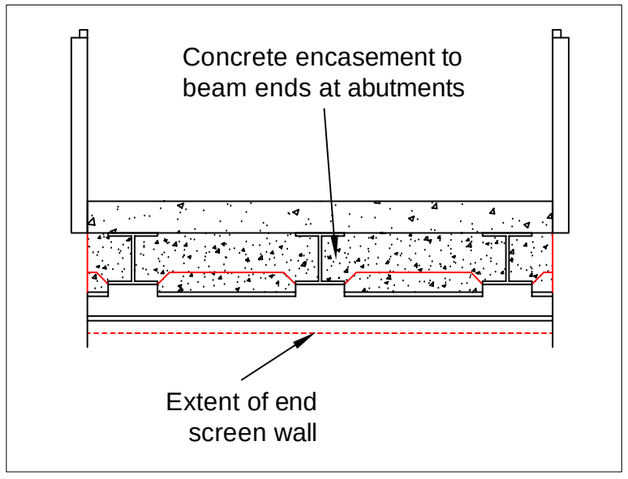
A.1 60493385-C0347-SKE-0001-A



Section B-B through front abutment wall



Sections of abutment walls to be reconstructed



Section A-A: Cross section of deck

LEGEND/NOTES

1. The proposed repairs are for both abutments

Revision	Date	Amendment	Drawn	Design	Checked	Approved

Shaping Southwark's Highways

Project
C0347 Cox's Walk Footbridge

Title
Appendix A - Repair Option Sketches

Contract No. 60493385-C0347		Drawn	SG
		Designed	IG
Scale N/A		Checked	SM
		Approved	DP
Drawing No. 60493385-C0347-SKE-0001		Rev.	A
File Name			
Date Drawn MAY 2018	Date Issued MAY 2018		

Last saved by: STEPHEN.GLENFIELD(2018-05-23)

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