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SWING BRIDGE REPLACEMENT, BERMUDA
APPROVAL IN PRINCIPLE - STRUCTURAL

RAMBOLL

Knight Architects

EADON CONSULTING

SWING BRIDGE REPLACEMENT, BERMUDA
APPROVAL IN PRINCIPLE - STRUCTURAL

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Prepared by	**R Graham-Ward**
Checked by	**N Karamichalis**
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Ramboll
Carlton House
Ringwood Road
Woodlands
Southampton
SO40 7HT
United Kingdom

T +44 (0) 238 081 7500
https://uk.ramboll.com

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Ramboll UK Limited
Registered in England & Wales
Company No: 03659970
Registered office:
240 Blackfriars Road
London
SE1 8NW

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1. HIGHWAY DETAILS

1.1. Type of highway

Rev 02 The Swing Bridge Replacement is a seven-span viaduct that links St. David's with St. George's and comprises of a single two-lane carriage way. The carriageway will have a total width of 7.00m, two traffic lanes with a width of 3.50 m each, and two footways with a width of 1.20 m each on both the East and West side of the bridge. The total width of the carriageway and footways shall be 9.40 m.

1.2. Permitted traffic speed

Rev P02 Road over: 50 kph.

1.3. Existing restrictions

Not applicable.

2. SITE DETAILS

2.1 Obstacle crossed

The viaduct crosses the channel between Ferry Reach and Stocks Harbour.

The Swing Bridge Replacement provides a clear width of 22m and when the bridge is open it will provide an infinite headroom.

3. PROPOSED STRUCTURE

3.1 Description of structure and design working life

The proposed new structure spans between Ferry Reach and Stocks Harbour and provides motorist and pedestrian connectivity between St. David's Island and St. George's. The structure will consist of 6 Piers and 7 spans with a main navigation channel width of 22m and bridge soffit elevation of +4.9m OD along the width of the navigable channel. A clear height channel will be provided when the bridge is in the open to navigation of marine vessels position (closed to motorist and pedestrian traffic).

The design life for this structure will be 75 years, except the elements listed below:

Rev 02

- Bearings — 50 years design working life
- Parapets — 75 years design working life
- Waterproofing system — 25 years design working life
- Surfacing/Expansion joints — 25 years design working life

3.2 Structural type

Lift Span

The lift span comprises a main steel box girder with a curved soffit that has a deep main spine, which tapers toward the edges of the box, resulting in slender wings that project beyond the pedestrian footpath into a crescent opening. The main box beam incorporates a grillage of plated transverse steel diaphragms, longitudinal webs and an external steel plate shell. The upper face of this box supports the running surface of the carriageway and comprises an orthotropic steel deck with trough stiffeners spanning between transverse diaphragms.

Approach Spans

The approach spans comprise a main steel box girder with a composite concrete deck supporting the carriageway. The box girder has a curved soffit, which tapers toward its outer edges and incorporates a grillage of plated transverse steel diaphragms, longitudinal webs and longitudinal plate stiffeners.

There is a pedestrian footpath on either side of the carriageway with a VRS (Vehicle Restraint System) and parapet that protect the edges of the bridge. The footpath is generally supported by the tapered edges of the box girder.

Piers and Abutments

The piers and abutments will be cast in-situ reinforced concrete and connected to the driven steel tubular piles with pile caps. The main pier forms the pivot point for the bridge and will be designed as a hollow reinforced concrete water retaining structure with an access chamber to house the hydraulic cylinders and pivot. The chamber pit will allow the pivot and hydraulic cylinders to be inspected at the base. The bridge terminates at the abutments located on the north and south side of the bridge. The south abutment will be a hollow reinforced concrete structure that will house the mechanical and electrical plant and equipment for the main span lifting mechanism.

3.3 Foundation type

Rev P02

The piers and abutments are formed on reinforced concrete pile caps that are seated on steel tubular piles rock socketed into weathered basalt/unweathered basalt dependent upon location. For further details refer the Geotechnical Report – Highway Structure Summary Information in Appendix 5.

3.4 Span arrangements

Rev P02

The overall length of the Swing Bridge Replacement between abutment bearings is 152.8m. The bridge comprises 7No. spans.

Rev P02

The south approach spans comprise 3No. spans of 18.5m, 22m, and 19.7m from abutment to the lift span.

Rev P02

The lift span (main span) has a length of 30.1m and provides a clear navigation channel for marine vessels of 22m between the faces of the main piers.

Rev P02 The north approach spans comprise 3No. spans of 22m, 22m and 18.5m from lift span to north abutment.

3.5 Articulation arrangements

Lift Span

The lift span box girder is connected with two radial spherical bearings to the main pier and is supported on two elastomeric bearings on the nose pier. To permit passage of marine vessels through the main navigation channel, the lift span pivots through 80 degrees about the main pier. The lifting mechanism is formed from a pair of hydraulic cylinders attached to the deck soffit and anchored within an access chamber within the body of the main pier.

Approach Spans

The approach spans will be fully integral with the intermediate piers and articulated with bearings at the north and south abutments. Directly above the intermediate piers, the box girders will be filled with concrete to form the integral connection to the pier.

3.6 Classes and levels

Rev P02 The superstructure and the substructures including piled foundations are Consultant designed elements. The whole structure category as defined in BD2 is Category 3.
Classes are based on the assumed consequences of failure and the exposure of the construction works to hazard.

3.6.1 Consequence class

The whole structure has been classed as CC2; shown in Table 1 below.

Table 1. Consequence class Table B1 from BS EN 1990:2002.

Consequences Class	Description	Examples of buildings and civil engineering works
CC3	**High** consequence for loss of human life, *or* economic, social or environmental consequences **very great**	Grandstands, public buildings where consequences of failure are high (e.g. a concert hall)
CC2	**Medium** consequence for loss of human life, economic, social or environmental consequences **considerable**	Residential and office buildings, public buildings where consequences of failure are medium (e.g. an office building)
CC1	**Low** consequence for loss of human life, *and* economic, social or environmental consequences **small or negligible**	Agricultural buildings where people do not normally enter (e.g. storage buildings), greenhouses

3.6.2 Reliability class

Reliability classes are represented by beta indexes as per Table B2 in Annex B of BS EN 1990. The indexes allow for moderate differentiation in the partial factors for actions and resistances. They correspond with the Consequence Classes as per Clause B3.2(2) of BS EN 1990.

The whole structure has been classed as a minimum as RC2 with an associated multiplication factor for actions K_{FI}=1.0.

3.6.3 Inspection level

Inspection levels define the inspection characteristics and requirements as shown in Table 2 below.

The inspection level for the whole structure is IL2.

Table 2. Inspection level Table B5 from BS EN 1990:2002

Inspection Levels	Characteristics	Requirements
IL3 Relating to RC3	Extended inspection	Third party inspection
IL2 Relating to RC2	Normal inspection	Inspection in accordance with the procedures of the organisation
IL1 Relating to RC1	Normal inspection	Self inspection

<u>Design supervision level</u>

The design supervision level for the whole structure is considered as DSL3 as shown in Table 3 below.

Table 3. Design supervision levels Table B4 from BS EN 1990:2002

Design Supervision Levels	Characteristics	Minimum recommended requirements for checking of calculations, drawings and specifications
DSL3 relating to RC3	Extended supervision	Third party checking : Checking performed by an organisation different from that which has prepared the design
DSL2 relating to RC2	Normal supervision	Checking by different persons than those originally responsible and in accordance with the procedure of the organisation.
DSL1 Relating to RC1	Normal supervision	Self-checking: Checking performed by the person who has prepared the design

<u>Execution Class</u>

EXC3 for Superstructure steelwork in accordance with EN 1090-2 Table B.3.

Execution Class 3 for Substructure and Superstructure concrete works in accordance with Highways England Specification Appendix 1701.

3.7 Road restraint systems requirements

VRS/parapets on the bridge superstructure will be a bespoke lattice grille structure. The vertical orientation of the bars will prevent them from being climbed.

Rev P02

The parapets will be finished by a longitudinal handrail made in stainless steel. Provision will be made for optional linear LED lighting.

The proposed load criteria and impact force on the VRS, and the design guidance used to ascertain the average impact force applied to the VRS are listed below:

- TD 19/06 – Requirements for Road Restraint Systems.
- TD 27/05 – Road Geometry Links – Cross-sections and Headrooms
- BS EN 1991 and National Annex
- BS EN 1317 – 2:2010 Road Restraint Systems. Performance Classes, Impact Test Acceptance Criteria and Test Methods for Safety Barriers Including Vehicle Parapets.
- BS 7818:1995 – Specification for Pedestrian Restraint Systems in Metal
- Manual of Contract Documents for Highway Works Specification for Highway Works Volume 1 – Series 400 – Road Restraint Systems

3.8 Proposed arrangements for future maintenance and inspection

3.8.1 Traffic Management

To ensure the Swing Bridge Replacement operates safely, the public must be kept away from all hazards. Swing barriers, sounders and signals will be provided on the static approaches at both ends of the lift span. The location of barrier mechanisms have been selected such that no excessive trap points are created on the walkways. Due to the anticipated low volume of pedestrian traffic each barrier will close off a pedestrian walkway and one side of road traffic (i.e. a total of four barriers will be provided, two at each end). Each barrier will be individually controllable to allow the barriers to be opened and closed at will before bridge movements begin.

A warning klaxon and traffic signals (but not pedestrian warning lights) will be provided to warn the public that the barriers and bridge are about to move. No navigation signals for marine traffic are proposed.

The traffic control barriers will be of the swinging type and actuated using a small hydraulic cylinder beneath a hatch in the walkway. This will enable access for maintenance and inspection without needing to interrupt or disrupt road traffic. Each barrier will be powered by a small hydraulic power pack located next to the actuating cylinder, also beneath the hatch. An underwater link will provide power and control for the equipment on the opposite side to the main
Rev P02 plant room (in the south abutment).

A facility will be provided to enable the barrier to be manually rotated in the event of a problem with either the cylinder or its associated hydraulic power unit.

The actuating force achievable by the mechanism will be sufficient to overcome the anticipated maximum operating wind but will be limited to reduce the risk of injury or crushing in the event that the barrier meets resistance. In the fully open and fully closed barrier positions the cylinders will be locked off and hence will be able to resist higher (hurricane) wind loads.

3.8.2 Arrangements for future maintenance and inspection of structure.

Access arrangements to structure.

Inspection and maintenance of external surfaces of bridge decks structural steelwork will be possible by means of temporary barges and access scaffolding. Internal access man-ways generally will be provided through the approach viaduct box girders and lift span box beams to allow access for inspection and maintenance of interior surfaces. This will require confined space access procedures. Access within the boxes will be provided wherever possible; portholes or smaller holes for endoscope will be provided for critical areas where man access is impracticable to permit visual inspection only. Inaccessible internal surfaces of structural steelwork, such as wings outstands will be effectively sealed by welding.

The access to the lift span box will be provided from the pivot pier and from the nose pier. In both locations the access will be through the sealed covered plate in the box girder soffit. There will be single access to each of the approach viaducts, from either the north or south abutment access galleries, through sealed cover plates in the approach span box girders. It is proposed to seal the cover plates at all locations to limit the exposure to air, moisture and contaminants.

Ready access will be provided at the main pier from the bridge footway to facilitate inspection of the pier and maintenance of the pivot bearings and other M&E plant and equipment located in the pier. Additional ladder access will be provided for the access by boat.

A gated access to a recess for bearing, shock absorber and deck inspection, complete with ladder access to the deck box and bearing shelf, will be provided in the nose pier.

Access galleries will be provided at the bridge abutments for inspection and maintenance of the bridge bearings.

It is proposed to provide integral connection between the intermediate piers and the bridge deck and no specific provision will be made for access at these locations.
Major M&E plant will be located in the plant room in the south abutment. In general, it is the design intention that equipment will be maintained in-situ. Ready access will be provided to all M&E plant and equipment with facility for lifting during installation and replacement.

Hydraulic cylinders will be designed to be maintainable in-situ and the bridge shall be operational whilst one cylinder is out of service for maintenance. A maintenance access hatch shall be provided in the top of the main pier to facilitate the removal of the smaller items of mechanical equipment should that be necessary. Removal of items from the main pier will be piece small through the vertical access shaft provided to all levels in the pier. Removable floor panels at each floor level will enable lift in/out of plant and equipment using mobile crane with the bridge in the open position. All M&E equipment will be readily maintainable and allowance will be made within the design for temporary jacking/support points to enable replacement of all bearings.

Rev P02

The lift span bearings at nose pier will be replaceable with the bridge in closed position and traffic restrictions. The lift span spherical pivot bearings will be replaceable with the lift span down and with the bridge closed to both road and marine traffic. The approach span bearings at abutments will be replaceable with the bridge fully operational for marine traffic, but with restrictions for vehicle loading, pedestrians and cyclists during bearing replacement.

Rev P02

In the main pier, the provision of a sump pump will be included to remove any water that may enter the cylinder pit within the pier. The water will be pumped towards the top of the pier, through the deck, to an interceptor beyond the south abutment, from where it will be discharged into the channel.

3.9 Environment and sustainability

Unsustainable and high embodied energy materials have been avoided where possible. The designed solution will be developed to minimise material usage in the superstructure and foundations. The steel for the superstructure can be readily recycled at the end of its working life and is likely to already contain a recycled component. The bridge superstructure will be fabricated off site in the controlled environment to minimise impact on the environment.

3.10 Durability. Materials and finishes

Table 4. Material Specification

Structural Steelwork – superstructure	Steel Grade S355 to BS EN 10025-2:2004 will be used for the bridge superstructures. As an alternative, ASTM A 709/A 709M grade 50 steel, or other equivalent, may be used. Where necessary steel plate shall comply with the requirements for improved deformation properties perpendicular to the surface of the product in accordance with BS EN 10164. In case of ASTM specified materials the yield and tensile strength shall be equal or greater than Steel Grade S355 to BS EN 10025-2. Inside the central cell of the lift span and approach spans, it is proposed to paint all internal steel surfaces. These surfaces will be readily accessed for maintenance painting in the future. There will inevitably be some areas of the lift span steelwork that will become inaccessible as fabrication advances which will result in irreparable damage to internal paintwork during the welding process which cannot then be repaired as part of the fabrication nor repainted as part of the maintenance regime. Where this occurs, sacrificial steel will be provided. Rev P02 The thickness of the sacrificial steel is dependent on the corrosivity category. Clause NA.2.14 a) of UK National Annex to BS EN 1993-2 provides guidance on sacrificial thickness allowances for the various atmospheric corrosion classes. An appropriate allowance for a humid subtropical climate will be considered. The internal boxes will generally be sealed with only occasional opening and venting for inspection and maintenance access through sealed access cover plates. This will significantly limit the exposure to air, moisture and contaminants. In accordance with Annex C of BS EN ISO 9223 Table C.1, corrosivity category C3 is proposed for the internal

	environment of the box elements of Swing Bridge Replacement. The additional sacrificial thickness of steel elements will be considered in accordance with Clause NA.2.14 a) and Table NA.1 of UK National Annex to BS EN 1993-2. It is proposed that steel with sacrificial steel allowance of 4.0mm appropriate to corrosivity category C3 is adopted for all internal unpainted and inaccessible surfaces. The internal voids of lift span trapezoidal trough stiffeners will be effectively sealed by welding, verified by pressure testing. Hence no sacrificial thickness of steel will be applied to the internal surfaces enclosed by these stiffeners.
Steel driven piles	Steel grade S355 to BS EN 10025-2:2004 will be used for the steel tubular piles. As an alternative ASTM A252 Grade 3 (Mod) may be used. The piles will have a design life of 75 years. The interior face and exterior face of the piles will include a sacrificial steel thickness of 2.6mm, which is the estimated loss of section thickness (mm) over a 75 year design life when fully submerged in sea water as specified in Table NA.1 of NA to BS EN 1993-5. In case of ASTM specified materials the yield and tensile strength shall be equal or greater than Steel Grade S355 to BS EN 10025-2.
Parapets/Bearing and tie down pins	Stainless steel grade 1.4404 or 1.4462 to BS EN 10088-2. As an alternative stainless steel grade S31603 or S32205 to ASTM A959 may be used. In case of ASTM specified materials the yield and tensile strength shall be equal or greater than the steel materials specified in BS EN 10088-2.
Access Grating	Stainless steel grade 1.4404 to BS EN 10088-2. As an alternative stainless steel grade S31603 to ASTM A959 may be used. In case of ASTM specified materials the yield and tensile strength shall be equal or greater than the steel materials specified in BS EN 10088-2.
Concrete	Grade C40/50 to BS 8500 (20mm max aggregate size). As an alternative Class P with compressive cylinder strength f'c = 5.8 ksi (=40 MPa) to AASHTO LRFD Bridge Design Specifications (20mm max aggregate size) may be used.

Rev P02

Rev P02

	In case of concrete material specified to AASHTO the nominal concrete cover shall be specified to BS 8500.
Reinforcement	Steel reinforcing bars shall be grade 500B or 500C ribbed bars to BS 4449:2009, to be produced by the hot rolled method. Preferred bar diameters to comply with BS 4449:2009. Alternatively, the reinforcement should conform to ASTM A615 Grade 60 reinforcement with a minimum yield strength of 60,000 pounds per square inch (psi) which is equal to 414 MPa. Reinforcement to be hot dip galvanised to BS EN ISO 1461 or Class 2 standard as per ASTM standard A 767; or zinc metal sprayed to BS EN ISO 17834. The yield strength of reinforcement to be used in the design calculation is to be 414 MPa in order to suit the selection of reinforcement specified to either BS or ASTM.
Surface Treatment – Waterproofing	The top surface of the concrete deck slab on the approach spans, the top surface of the steel deck of the lift span, will be protected by a proprietary waterproofing system to 2000 Series of SHW (Specification of Highways Works). All buried concrete surfaces and the rear face of the abutments down to the level of the deck soffit shall receive two coats of bituminous paint in accordance with Clause 2030 of SHW.
Paint Protective Coatings	All exterior steelwork surfaces and internal surfaces of box sections accessible for maintenance will be painted with a Type II approved paint system complying with Series 1900 of the Specification for Highways Works appropriate to a Marine Environment with difficult access with 20 years to major maintenance. Specialist advice will be sought from the paint system manufacturers at the detail design stage and the further consideration will be given to use of combined metal spray system (zinc or aluminium). The paint system that is likely to be considered will comprise the typical Type II paint system as shown below: - 1st Coat: Zinc Phosphate Epoxy (two-pack) - Min dry film thickness 25 μm - 2nd Coat: High Build Glass Flake Epoxy (two-pack) - Min dry film thickness 400 μm

	- 3rd Coat: choice of Epoxy Acrylic Finish (two-pack), Polyurethane (two-pack), Organic Modified Polysiloxane (two-pack) - Min dry film thickness 50 or 100 μm - 4th Coat: N/A Paint colour to be agreed before construction.

Cover to reinforcement and concrete class shall be as required by BS 8500 and Series 1700 of the Specification for Highway Works for the exposure conditions and buried concrete classification appropriate to the site and specific elements of the structure. The exposure criteria of the concrete for design purposes will be as follows:

Table 5. Concrete exposure criteria

Element	Exposure Class	Cover		
		Minimum cover (mm)	Fixing tolerance (mm)	Nominal cover (mm)
Rev P02 Pile caps	XS2	70	15	85
Abutments/Piers	XS3	70	10	80
Bridge deck top surface, soffit and pier diaphragms	XS1	40	10	50
Retaining wall and their footings	XS3	70	15	85
Rev P02 Parapet edge detail, top, sides and soffit	XS3	70	10	80

3.11 Risks and hazards considered for design, execution, maintenance and demolition. Consultation with and/or agreement from CDM co-ordinator

A Designer's Risk Assessment will be maintained for the duration of the design process with the aim of mitigating construction, operation, maintenance and demolition hazards. Where it is not reasonably practicable to design out hazards, and these are outside the reasonable knowledge of a competent contractor, then these will be communicated to the contractor by means of residual risk register on the construction drawings.

For a detailed breakdown of the risks and hazards considered, refer to the Designer's Risk Assessment for the Swing Bridge Replacement in Appendix 3.

3.12 Estimated cost of proposed structure together with other structural forms considered (including where appropriate proprietary manufactured structure), and the reasons for their rejection (including comparative whole life costs with dates of estimates)

A detailed feasibility study to review other structural forms and full cost estimate of the proposed structure has been prepared as part of Phase 2. For further details refer to Doc Ref. 3502-RAM-XX-XX-RP-CB-20001 – Rev 2 (Replacement of Swing Bridge and Longbird Bridge, Bermuda, Phase II Feasibility Report).

3.13 Proposed arrangements for construction

3.13.1 Construction of structure

Rev P02

Field surveys are to be conducted to establish the on-site locations of the navigation channel and piers/abutments. At the foundation locations, preliminary excavations are to take place to ensure level bed profile for subsequent pile caps. Steel tubular piles are to be driven from a piling rig located either on a barge or temporary piling platform to the required load bearing soil stratum as identified on the drawings

Once the piles are driven to the required depth, sheet pile cofferdams surrounding the driven piles are to be installed, braced as appropriate and secured to the sea bed. Prior to pouring the plug, the sea bed within the cofferdam may need to be further excavated to form a level surface where necessary. The concrete plug or tremie seal should then be poured and the cofferdams dewatered. Excess pile lengths are to be trimmed to the appropriate level in preparation for the pile cap construction.

Steel reinforcement cages for the pile caps can either be fabricated on site and lifted into place within the pile cap formwork or fixed in place. Concrete can then be transported to the site, poured, compacted and cured. The concrete can be pumped from the abutment locations or from the existing Swing Bridge, however this would require traffic management.
The pier steel reinforcement cages can either be fabricated on site and lifted into place or fixed in place. The shuttering for the piers can then be installed and the concrete cast up to c.300mm below the deck soffit. Concrete for the pier construction can then be transported to site, poured, compacted and cured. Once the desired concrete strength is achieved, shuttering can be stripped, the cofferdams removed and the sea bed reinstated.

The abutments can then be constructed, with formwork erected for the abutment walls and the steel reinforcement cage fixed in place. Concrete will then be poured and compacted. The concrete should then be cured and the formwork removed. A waterproof membrane shall be applied to the wall faces retaining the soil fill and a drainage layer installed behind the abutment surrounded by free draining granular material. The abutments and wing walls should then be backfilled to road formation level. Bearings should then be placed at the abutments in preparation for the seating of the bridge deck.

The bridge superstructure will likely consist of full span fabricated deck elements that could be positioned with self-propelled modular transportation (SPMT) units, or temporary jacking stools, from the barge or craned in. Approach spans are to be erected onto permanent bearings at abutments and temporary steel stools on intermediate piers in locations where there are integral diaphragm connections. The bridge deck approach spans shall be erected sequentially from the abutments. As the water depth adjacent to the abutments is too shallow for barge access, the first approach spans are likely to be lifted into place with a suitable crane, but as an alternative could be rolled into place from landside SPMTs on a short temporary causeway access. The remaining approach span steelwork deck sections can then be erected from SPMT units, or temporary jacking stools, from a barge or craned in.

Once the steelwork deck segments are in place, the segments shall be made continuous with full strength butt welds. The approach spans in-situ concrete deck can then be poured to form the composite deck along with the integral diaphragm connections and top of pier to make the fully monolithic connection between deck and intermediate piers.

On completion of the construction of the south abutment, main pier and nose pier, the M&E equipment for lifting the main span can be installed in preparation for installation of the lift span steelwork. This includes the nose bearings, pivot bearings and lifting cylinders. The lifting span can then be erected using the same installation procedure outlined for the approach spans.

Finally, the bridge deck shall be waterproofed and the finishes installed and the M&E equipment for lifting the main span fully commissioned. Only after the Swing Bridge Replacement is fully commissioned can the demolition of the existing Swing Bridge commence.

3.13.2 Traffic management

It is recommended that the construction of the new roundabout (with 4 arms) be undertaken in phases to allow for the all roads to remain operational during the construction of the new bridge and its associated carriageway.

All traffic movements into and out of the existing roundabout and its access points are not to be constrained by the proposed new roundabout. Therefore, the proposed roundabout will be designed to be appropriately located to minimise disruption to the existing road network and to minimise additional offsite construction works.
Due to the requirement of maintaining access and traffic movement on all roads, there will be potential to deliver traffic management or to utilise temporary traffic signals during the construction phase.

All existing splitter and kerb central islands will be removed initially as part of the phasing plan to allow for adequate access to be maintained to all roads whilst construction is occurring. Temporary road markings, and signs will be required.

Rev P02

It is anticipated that the new roundabout proposals are for a normal roundabout in accordance to TD 16/07 with splitter and kerb central island and flared entries and exits to allow two vehicles to enter or exit the roundabout at a specified arm at the same time. The roundabout is to be designed by the Government's, Ministry of Public Works, Highways department.

The circulatory carriageway will need to be maintained as the existing conditions and wide enough to allow for two vehicles to travel alongside each other around the roundabout.

3.13.3 Service diversions

Rev P02

The topographic surveys provided by the Client appear to indicate that, other than the power and control cables for the opening mechanisms, no services cross the existing Swing Bridge at present.

However, there are overhead power lines, traffic signals, drainage gullies and possibly an internet/cable line, which all run along the north-east side of Mullet Bay Road adjacent to the retaining wall to the north-east of the existing Swing Bridge.

There is evidence from the existing BELCO utility drawings that there are underground telephone cables that run under the main road along with a subsea cable that runs from the bridge keeper's house to the existing Swing Bridge pintle pier.

It is unlikely there will be water supply and sewerage pipes feeding into the private residence and bridge operators welfare facility on the North-West of the existing swing bridge as these are likely to be self-contained systems, however, this needs to be confirmed.

On the south side of the existing Swing Bridge there are existing lighting columns at the Kindley Field roundabout. It is anticipated that relocation of these power/data/telephone lines and drainage gullies will be necessary to aid construction of the realigned approach for the existing Swing Bridge.

It is essential for the existing Swing Bridge that a full existing services site survey is performed by the Client and summarised in a combined services drawing to verify the location of each of the services and confirm which are live and which are redundant in order to inform a strategy for diversion and protection of services prior to construction and demolition works.

Service ducts for present or future use within the Swing Bridge Replacement deck will not be provided.

3.13.4 Interface with existing structures

The proposed replacement structure will be constructed parallel to, and offline from, the existing Swing Bridge. The interface with existing structures refers to the demolition of the existing Swing Bridge which is planned to start after Swing Bridge Replacement is fully commissioned.

Rev P02

The existing superstructure for both the swing and approach spans consist of steel main girders, steel cross girders, a steel orthotropic deck, surfacing, concrete verges and pedestrian parapets. The cross girders are bolted to the top flange of the main girders and to the underside of the orthotropic deck. The swing spans are a double cantilever pivoting about the pintle bearing and consist of a pivot span and a counterbalancing back span, whilst the approach spans are simply supported on the bridge piers.

4. DESIGN CRITERIA

4.1 Actions

4.1.1 Permanent actions

Self-weight of the superstructure; Permanent actions shall be in accordance with the relevant parts of BS EN 1991 and the UK National Annex.

Steel will have a density of 7850kg/m^3.

Reinforced Concrete will have a density of 2500kg/m^3.

Wet Concrete will have a density of 2600kg/m^3.

4.1.2 Snow, Wind and Thermal actions

Wind loads will be calculated in accordance with BS EN 1991-1-4:2005 and the UK National Annex. In the closed condition, wind loading will be considered using a fundamental design wind speed of 150 mph in accordance with the Bermuda Code 2014.

In the bridge open conditions, wind velocities are described in detail in Section 4.1.9.

Assessment on the aerodynamic stability of the structure will be performed in accordance with BS EN 1991-1-4 as supplemented by PB 6688-1-4.

Thermal loads will be calculated in accordance with BS EN 1991-1-5:2003 along with the UK National Annex and will be based on the shade air temperature range of 5ºC to 34ºC.

Rev P02

In line with the provisions of NA.2.21 of NA to BS EN 1991-1-5 and taking into account the ambient temperature range of Bermuda, the construction temperature T_0 will be taken as 15 degrees Celsius for expansion and 25 degrees Celsius for contraction.

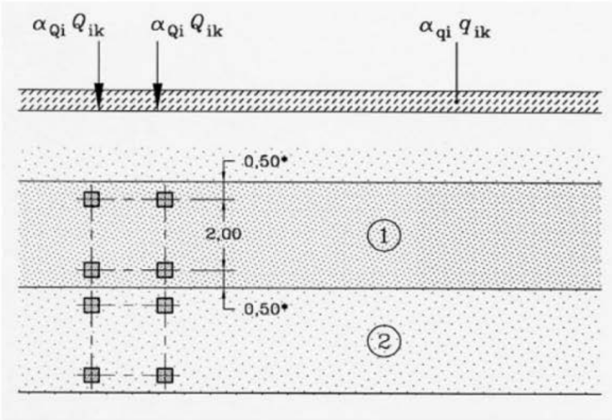
Uniform temperature will be assumed along the entire length of the structure.

For temperature gradient the lift span superstructure (steel orthotopic deck) will be considered as Type 1 whilst the approach span (steel concrete composite deck) will be considered as Type 2.

No snow loading will be considered.

4.1.3 Actions relating to normal traffic under Authorised Weight (AW) regulations and Construction and Use (C&U) regulations

The structure will be designed to the BS EN 1991-2 as modified by UK National Annex for highways traffic ‘Load Model 1’, which includes a Uniformly Distributed Load of 5.5 kN/m2 along with double-axle concentrated loads (tandem systems) per notional lane acting on the most unfavourable part of the influence surface, as indicated in Figure 1 below.



Load Model 1 based on BS EN 1991-2

Key:

Carriageway - Lane 1: Q_{1k} = 300 kN q_{1k} = 5.5 kN/m^2

Carriageway - Lane 2: Q_{2k} = 200 kN q_{2k} = 5.5 kN/m^2

Remaining area of carriageway: q_k = 5.5 kN/m^2

Tandem axle spacing = 1.2 m
Lane width = 3.0 m

Figure 1 – Representation of Load Model 1

By way of comparison Figures 2 and 3 below indicate the assessment live loading for the assessment (or evaluation) of existing bridge structures in Bermuda derived by the Delcan Corporation in their report ‘Evaluation Criteria for Highway Bridges in Bermuda’ produced for the Ministry of Public Works. The loading arrangements depicted in Figure 2 and Figure 3 are based upon actual vehicles typical to Bermuda.

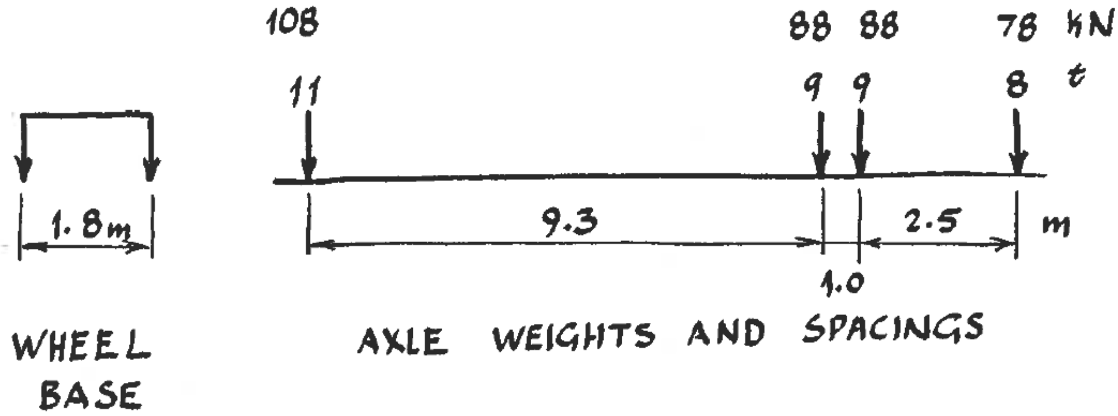


Figure 2 – Proposed Evaluation Truck for Bermuda

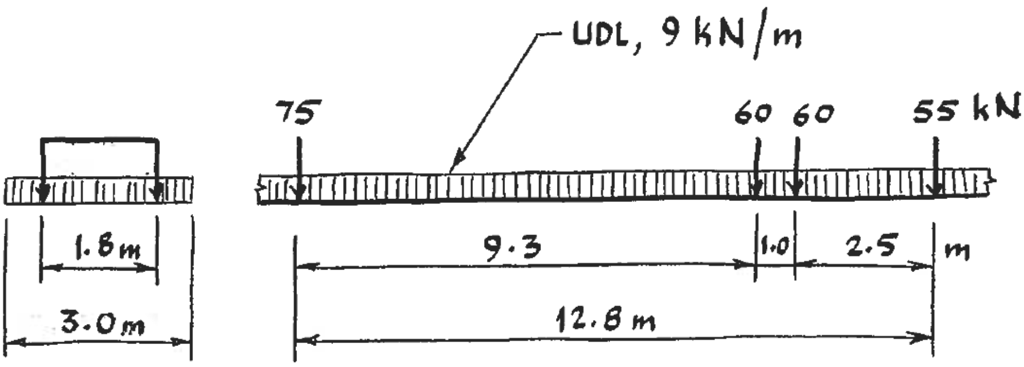


Figure 3 – Proposed Evaluation Lane Load for Bermuda

Whilst the Load Model 1 and the Evaluation loading are not quite the same in that they are not both patterns of design live load, it can be seen by inspection that the Load Model 1 case is more onerous.

It should be noted that in the Delcan report the partial factor for live loads is proposed as 1.6 at ULS. Whereas in BS EN the equivalent load factor is 1.35. However even after taking this difference into consideration it can be seen by inspection that it remains that the BS EN Load Model 1 loads are more onerous. Therefore, BS EN Load Model 1 will be considered for design.

4.1.4 Actions relating to General Order traffic under STGO regulations

N/A.

4.1.5 Footway or footbridge variable actions

The structure will be designed for a vertical uniformly distributed live load of 5kN/m2.

Rev P02

The VRS does not protect footway from vehicles and therefore two separate cases of a single wheel and single axle (pair of wheels orientated parallel with the carriageway) will be considered to be present on the footway in accordance with clause 4.7.3 of BS EN 1991-2 as modified by UK National Annex.

4.1.6 Actions relating to Special Order traffic, provision for exceptional abnormal indivisible loads including location of vehicle track on deck cross-section

N/A.

4.1.7 Accidental actions

<u>Vessel Impact</u>

Using the AASHTO impact formulas for barges, the ferry design impact load on the bridge substructure is calculated to be 2100kN (about 214 tons force).

It is noted that the calculation includes a reduction factor based on the ratio of the widths of the ferry hulls (2nr x 9.7 foot) to the width of the AASHTO barge (35 foot). This reduction factor was included in the AASHTO (1991) edition but has been discontinued in the current AASHTO LRFD (2014) edition. However, it is considered it is reasonable to retain the factor, since the impact force is largely due to crushing and buckling of the hull and deck plates, so that narrower hulls would exert proportionately less force. If the reduction factor were not used, then the impact load would be over 5000kN, which we consider to be unduly onerous for the relatively lightweight aluminium structure of the ferry.

For comparison, use of the impact formulas in AASHTO for ocean going ships, but with a displacement of only 128 tons, gives an impact load of 3500kN. However, this has also been discounted as being unduly onerous.

For further comparison, EuroCode-1 Part 1-7 Accidental Actions on Structures, Table C.3, gives indicative values of the forces due to ship impact on inland waterways. The smallest vessels listed are 200-400 ton mass, for which the indicative dynamic force is stated to be 2000kN.

The indicative force is multiplied by the following factors:

Table 6. Factors for vessel impact

Factor for high consequence of failure	1.3
Dynamic Amplification Factor	1.3 (frontal)
Factor for harbour areas	0.5

The resulting factored force is 1690kN (172 tons force).

It is therefore considered that the proposed design impact force of 2100kN for the ferry is conservative, considering that it is lighter and made of aluminium not steel.

The impact load for the ferry deckhouse is estimated to be 20% of the hull impact load, and the impact load for the mast is estimated to be 10% of the deckhouse load, as per the AASHTO method for ships.

Table 7. Vessel impact loads

Impact Case	Impact load	Location
Head on impact of ferry hull on bridge substructure	2100kN	2.3m above mean high water (MHW) level, in a direction parallel to the main channel axis.
Glancing impact of ferry hull on bridge substructure	1050kN Applied separately to head on impact case.	2.3m above MHW level, in a direction transverse to the main channel axis.
Impact of ferry deckhouses on bridge superstructure	420kN	Action applies from MHW + 2.3m to MHW + 8.4m, in a direction parallel to the main channel axis.
Impact of ferry mast on bridge superstructure	42kN	Action applies from MHW + 8.4m to MHW + 10.1m, in a direction parallel to the main channel axis.

In principle, these loads could be reviewed by detailed assessment of the capacity of the connections of the mast to the deckhouse and the deckhouse to the hull, as well as the hull plates and scantlings, if information were available.

Given the information on water levels and surge levels in the area in relation to the proposed +4.9mOD soffit level, it is predicted that the worst superstructure collision case would be with a deckhouse collision. This assumes that the highest observed water levels would not coincide with the highest predicted storm surge level due to the unlikeliness of this event. The potential of a bow collision with the bridge deck was also considered, however, as the calculation is based on percentages of the pier force collision, >20% of the bow would have to make contact with the deck for it to be the governing condition. In order for this to happen the sea level would have to reach >+2.02mOD, which would only be possible with HAT tides with 1 in 100 year surge, a very improbable occurrence. The improbability of the vessel impacting in conjunction with HAT and 100yr surge means this scenario can be neglected.

The forward end of the ferry's deckhouse is sloping, so the possibility of the ferry wedging itself under a bridge deck should also be considered. This could possibly give rise to an uplift force on the bridge deck which should be assessed later in the design process.

The size and displacements of the sailboats and motorboats are much less than the ferry, so the loading would also be less critical then the ferry for moving bridge options.

Wave loading

The wave loading on the superstructure has been considered at the feasibility stage in accordance with section 4.9.11 of the Phase II Feasibility Report (Doc Ref. 3502-RAM-XX-XX-RP-CB-20001 – Rev 2). The integral connections between the substructure and superstructure (Swing Bridge Replacement approach spans) will be provided to ensure that the bridge decks remain in place during the hurricane event. The radial spherical bearings and the nose locking system with pin will ensure the same for the lift span. The hydrodynamic loading on the pier and abutments will be considered in accordance with section 4.9.12 of the Phase II Feasibility Report.

Wave loads on bridge deck

Rev P02

Guidance from AASHTO BVCS (Bridges Vulnerable to Coastal Storms 2008) is based on bridge geometries of the girder type shown below in Figure 4 and Figure 5. The curved shell type geometries of the proposed Swing Bridge Replacement were idealised to represent the AASHTO girder type cross sections to be in-line with the code. Appropriate assumptions for the application of the wave load and the implementation of the AASHTO BVCS (2008) will be made during the calculation of the loading input.

According to AASHTO BVCS (2008), two different design cases must be analysed to evaluate the forces applied on the bridge deck by the waves. The forces on the piers, abutments, and other retaining walls are addressed. The design cases for wave action on the bridge deck are:

- Design Case I: Maximum quasi-static vertical force and associated horizontal force, moment, and vertical slamming forces
- Design Case II: Maximum horizontal wave force and associated quasi-static vertical force, moment and vertical slamming force

According to AASHTO BVCS (2008), the wave force equations were developed around the trailing edge of the girders as shown in Figure 6, and calculations of force effects on the structure shall start with the forces assumed to be applied at the trailing edge. The forces shall be applied to the full length of one span of the structure at the same time. Although the slamming force is instantaneous, to design against bridge uplift the maximum quasi-static vertical force and the slamming force must be combined.

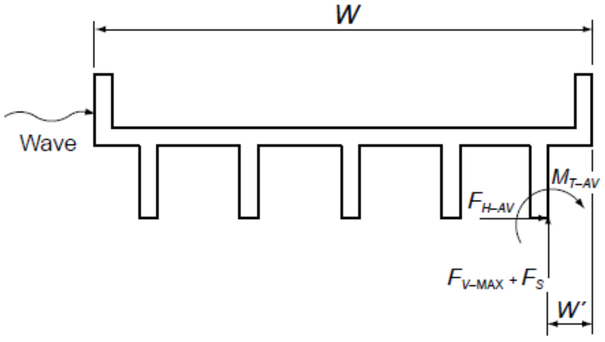


Figure 4 - Diagrams extracted from AASHTO BVCS (2008) illustrating the applied maximum vertical force and associated horizontal force, slamming force, and moment, applied along the length of the span or bridge

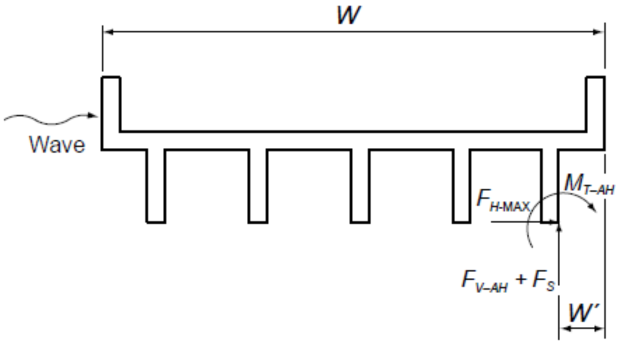


Figure 5 - Diagrams extracted from AASHTO BVCS (2008) illustrating the applied maximum horizontal force and associated vertical force, slamming force, and moment, applied along the length of the span or bridge

Rev P02

Figure 6 illustrates in sketch form the interaction of the wave with a typical bridge structure.

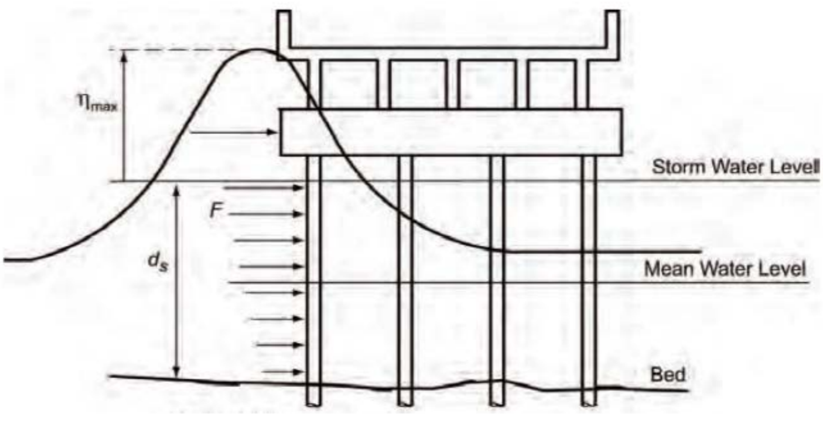


Figure 6 – Extract from AASHTO BVCS (2008) Illustrating the Interaction of Waves with the Bridge Structure

Wave Parameters

The following parameters have been used to derive wave forces on the Swing Bridge Replacement lift span:

Table 8. Wave parameters for Swing Bridge Replacement lift span

S_L	Bridge Soffit Level above OD	4.90	m	16.08	ft
H_{max}	Max wave height	3.01		9.9	ft
H_{max*}	Max wave height (limited)	3.01	m	9.9	ft
Tp	Peak wave period	5.00	s	5.0	s
λ or L	Wave length	39.03	m	128.1	ft
d	Water depth below OD	4.00	m	13.1	ft
SLR	Relative sea level rise above water level by 2100	0.86	m	2.8	ft
Surge	WSP 2016 1:150yr predicted surge level, mOD	2.00	mOD	6.6	ftOD
d_s	Storm water level (by 2100) above seabed	6.86	m	22.5	ft
η_{max}	Distance from the storm water level to design water crest	2.10	m	6.9	ft
	Non-linear wave assymetry factor	0.70			
r	Rail height	1.75	m	5.7	ft
γ_w	unit weight of water taken as 0.064 kip/ft^3			0.064	kip/ft^3
W	Bridge width	11.60	m	38.1	ft
Z_c	Vertical distance from the bottom of the cross section to d_s	2.04	m	6.7	ft
d_b	Depth of bridge deck	1.65	m	5.4	ft
d/L (present)		0.10			
d/L (by 2100)		0.18			
0.65 d_s		4.46			
λ /7		5.58			

The following parameters have been used to derive wave forces on the Swing Bridge Replacement approach spans:

Table 9. Wave parameters for Swing Bridge Replacement approach spans

S_L	Bridge Soffit Level above OD	4.60	m	15.09	ft
H_{max}	Max wave height	3.01		9.9	ft
H_{max*}	Max wave height (limited)	3.01	m	9.9	ft
Tp	Peak wave period	5.00	s	5.0	s
λ or L	Wave length	39.03	m	128.1	ft
d	Water depth below OD	4.00	m	13.1	ft
SLR	Relative sea level rise above water level by 2100	0.86	m	2.8	ft
Surge	WSP 2016 1:150yr predicted surge level, mOD	2.00	mOD	6.6	ftOD
d_s	Storm water level (by 2100) above seabed	6.86	m	22.5	ft
η_{max}	Distance from the storm water level to design water crest	2.10	m	6.9	ft
	Non-linear wave assymetry factor	0.70			
r	Rail height	1.35	m	4.4	ft
γ_w	unit weight of water taken as 0.064 kip/ft^3			0.064	kip/ft^3
W	Bridge width	10.35	m	34.0	ft
Z_c	Vertical distance from the bottom of the cross section to d_s	1.74	m	5.7	ft
d_b	Depth of bridge deck	1.65	m	5.4	ft
d/L (present)		0.10			
d/L (by 2100)		0.18			
0.65 d_s		4.46			
λ /7		5.58			

Results of wave forces on bridge decks with SLR (relative Sea Level Rise) taken as 0.86m

The wave forces on the bridge decks are presented as follows:

Table 10. Summary Wave Forces Case I

	Design Case I	
	Swing Bridge Replacement Lift Span	Swing Bridge Replacement Approach Spans
$F_{V\text{-}MAX}$ (kN/m)	0.8	4.1
$F_{H\text{-}AV}$ (kN/m)	0.0	0.2
F_S (kN/m)	4.8	8.6
$M_{T\text{-}AV}$ (kNm/m)	32.4	85.9

Rev P02

For the Design of the bridge decks the actions in Table 10 above will be applied to the soffit at W/2 (=5.175m) from the centreline as illustrated in Figure 4.

Table 11. Summary of Wave Forces Case II

	Design Case II	
	Swing Bridge Replacement Lift Span	Swing Bridge Replacement Approach Spans
$F_{V\text{-}MAX}$ (kN/m)	0.0	0.9
$F_{H\text{-}AV}$ (kN/m)	0.0	0.0
F_S (kN/m)	4.8	8.6
$M_{T\text{-}AH}$ (kNm/m)	50.8	85.0

Rev P02

For the Design of the bridge decks the actions in Table 11 above will be applied to the soffit at W/2 (=5.175m) from the centreline as illustrated in Figure 5.

Rev P02

Reducing the SLR value to 0m, has a significant impact. The reduction in SLR results in the design wave crest level being lower than the lift span soffit level by at least 0.49m.

Minimum Loads

Rev P02

Clause C5.3 of AASHTO BVCS (2008) states *that 'when the calculated uplift force on a bridge based on the nominal values of surge and wave height approaches zero due to the passing wave below the structure'* the connection between each slab sub-unit and the substructure shall be designed to resist a minimum factored uplift force (unreduced by the slab dead load reaction) of F_a = 8 kips/ft of width at each end (~117 KN/m). This is to account for the residual risk stemming from the random distribution of surge and wave heights.

The requirements of clause C5.3 apply to the lift span nose bolts only. The nose bolts shall be designed to accommodate a minimum force of 117kN/m x width at ULS.

Hydrodynamic loads on wide piers, and walls

Waves encountering vertical, wide structures will behave differently as the full depth of the wave will hit the structure, and the water will be projected upwards above wave crest level. Clause 6.1.3 of AASHTO BVCS (2008) provides guidance on the calculation of hydrodynamic loads on bridge substructures based on Goda's method. The applicability of this method to the bridge piers will be investigated during detailed design so that the most appropriate method for the geometry of the structure is selected.

Figure 7 summarises the wave pressure profile to be applied using the Goda method on such piers and walls.

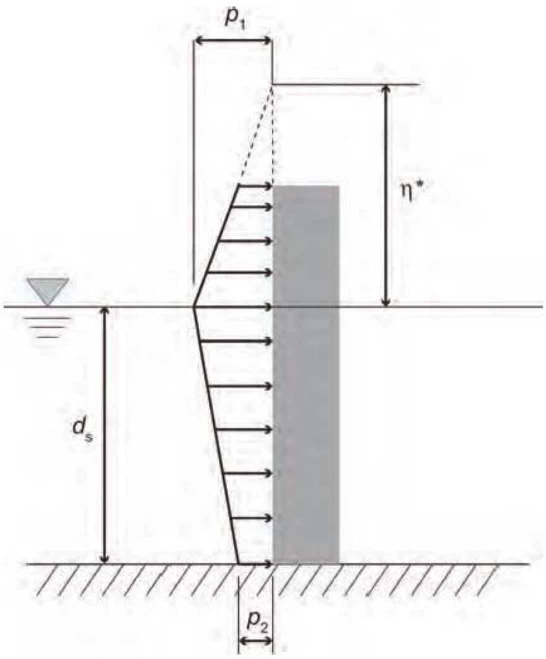


Figure 7 - Extract from AASHTO BVCS (2008) Showing Wave Force Profiles on Large Elements

Results of Wave Forces on Substructure with SLR taken as 0.86m

Rev P02 The results obtained for Swing Bridge Replacement are presented as follows:

Table 12. Summary of Wave Loads on Piers – SLR=0.86m

Rev P02

	Swing Bridge Replacement
p1 (kN/m^2)	21.8
P2 (kN/m^2)	13.0 *(applicable to nose pier only)*
η* (m)	4.5
d_s (m)	6.9 *(applicable to nose pier only)*

Rev P02 When considering SLR=0.86m the value of peak pressure p_1, its application level of +2.90m OD and the dimension η* are common for all piers. Pressure P_2 and the dimension from storm water level to bed level, d_s, shown in Table 12 are applicable to the nose pier only. For all other piers p_2 is to be determined by linear interpolation from the p_1 and p_2 values from Table 12 using the dimension d_s applicable for the bed depth at the pier under consideration.

Results of Wave Forces on Substructure with SLR taken as 0m

The results obtained for both bridges are presented as follows:

Table 13. Summary of Wave Loads on Piers – SLR=0.00m

	Swing Bridge Replacement
p1 (kN/m^2)	23.1
P2 (kN/m^2)	15.3 *(applicable to nose pier only)*
η* (m)	4.5
d_s (m)	6.0 *(applicable to nose pier only)*

Rev P02

When considering SLR=0.00m the value of peak pressure p_1, its application level of +2.00m OD and the dimension η* are common for all piers. Pressure P_2 and the dimension from storm water level to bed level, d_s, shown in Table 13 are applicable to the nose pier only. For all other piers p_2 is to be determined by linear interpolation from the p_1 and p_2 values from Table 13 using the dimension d_s applicable for the bed depth at the pier under consideration.

Rev P02

Wave Loading Calculation Approach

According to AASHTO BVCS (2008) bridges classed as critical/essential should be designed at the strength limit state to achieve a state of "service immediate". Bridges considered secondary to rescue and recovery may be designed at the extreme event limit state. Under the strength limit state, a load factor of 1.75 is applied to the wave loads whereas the load factor is unity for the extreme limit state. These load factors are based on the design event being a 1 in 100yr event whereas the analysis carried out herein has been based on a 1 in 150yr event as agreed with the Client and therefore the load factors can be considered conservative for such an event.

Rev P02

The combined total SLR of 0.86m (0.76m for sea level rise and 0.1m for land subsidence) in conjunction with the 1 in 150yr hurricane event provides a conservative worst-case scenario. Including this scenario under the strength limit state with the associated factor of 1.75 was considered an overly conservative approach, therefore a method has been adopted whereby two separate scenarios will be considered as follows:

1. Loads with SLR considered as 0.86m - Extreme Event Limit State [factor of 1.0] – wave loads on deck and piers will considered as coincident.
2. Loads with SLR considered to be 0m – Strength Limit State [factor of 1.75]. Wave loads will be considered on deck end spans only. Wave loads on deck end spans will be considered with wave loads on all piers.

Seismic loading

Bermuda is known to be situated in an area that is seismically active. The Bermuda Building Code 2014 cl. 1610.1 states that "Consideration of earthquake loads should be taken into account especially when designing multi storey, non-symmetrical eccentrically loaded structures or those containing sensitive equipment.

As part of the Feasibility Study for the crossing of Castle Harbour and Grotto Bay, Halcrow undertook a specialist seismic hazard study to confirm the seismic loading appropriate for Bermuda (refer to report 'Government of Bermuda, MW&E&H, New Crossing, Waters of Castle Harbour / Grotto Bay, Bermuda – Seismic Hazard Study, April 2010).

Site specific uniform hazard spectra for the horizontal component of the ground motion are proposed in this report for return periods of 500 years, 1000 years and 2500 years and for rock site conditions.

The 500-year return period uniform hazard spectrum for rock site conditions will be used as a reference for design, implementing the seismic design provisions of BS EN 1998-1, BS EN 1998-2 and BS EN 1998-5 as appropriate. This return period is approximately equal with the recommended value of the reference return period of Eurocode being 475 years. This return period corresponds to seismic loading with probability of exceedance of 10% in 50 years.

To achieve a level of seismic loading with the same level of probability of exceedance for the 75 years design life of the bridge in the closed condition (open to vehicle traffic) reference is made to Annex A of BS EN 1998-2.

The return period of the seismic loading which corresponds to p=10% in t_L = 75 years (design life of bridge) is given by equation A.1 of Annex A of BS EN 1998-2 as below:

$T_R = 1/(1-(1-p)^{1/tL}) = 1/(1-(1-0.1)^{1/75}) = 712$ years

An acceptable estimation for the spectral acceleration ratio that corresponds to the return period T_R in relation to the reference period T_{NCR} is given by equation A.3 of Annex A of BS EN 1998-2 as below:

$a(T_R) / a(T_{NCR}) = (T_R / T_{NCR})^k = (712/500)^{0.35} = 1.132$

The return period for the estimation of seismic loading for the open bridge condition (closed to vehicle traffic) is calculated as below:

Rev P02

- As an average throughout the year lift span open eight (8) times on a 24 hour cycle for a period of 10mins each time
- This corresponds to 5.55% of a 24 hour period
- Design life of bridge in open condition is therefore 5.55% of 75 years or 4.2 years

Rev P02

Similar to the previous calculation, the return period of the seismic loading which corresponds to p=10% in t_L = 4.2 years (time period of open bridge throughout its design life) is given by equation A.1 of Annex A of BS EN 1998-2 as below:

Rev P02

$T_R = 1/(1-(1-p)^{1/tL}) = 1/(1-(1-0.1)^{1/4.2}) = 41$ years

An acceptable estimation for the spectral acceleration ratio that corresponds to the return period TR in relation to the reference period TNCR is given by equation A.3 of Annex A of BS EN 1998-2 as below:

Rev P02

$a(T_R) / a(T_{NCR}) = (T_R / T_{NCR})^k = (41/500)^{0.35} = 0.417$

The bridge is considered to be of importance class II in accordance with Clause 2.1 (4)P of BS EN 1998-2 therefore the importance factor for both the above cases is taken as γI = 1.00.

The spectral accelerations of the reference return period and the return periods of the seismic loading that correspond to the bridge in closed condition and the bridge in open condition are tabulated below.

Table 14. Spectral accelerations

Rev P02

Rock Soil Conditions	Reference return period T_{NCR} = 500 years	Bridge Closed Return period for 10% probability of exceedance in 75 years T_R = 712 years	Bridge Open (any inclination) Return period for 10% probability of exceedance in 4.2 years T_R = 41 years
Period (sec)	Reference Spectral Acceleration * g (m/sec^2)	Design Spectral Acceleration * g (m/sec^2)	Design Spectral Acceleration * g (m/sec^2)
0 (PGA)	0.06	0.06*1.132=0.068	0.06*0.417=0.025
0.1	0.10	0.10*1.132=0.113	0.10*0.417=0.042
0.2	0.08	0.08*1.132=0.091	0.08*0.417=0.033
0.4	0.06	0.06*1.132=0.068	0.06*0.417=0.025
1.0	0.02	0.02*1.132=0.023	0.02*0.417=0.008
2.0	0.01	0.01*1.132=0.011	0.01*0.417=0.004

The soil amplification factors from Table 3.3 of BS EN 1998-1 will be used for design depending on the founding ground type.

4.1.8 Action during construction

Actions during execution has been considered in accordance with BS EN 1991-1-6:2005. The structure will be designed taking due consideration of the different support conditions during transportation and erection.

4.1.9 Any special action not covered above

Dead load dynamic load allowance

Structural parts, in which the force effect varies with the movement of the span, or in parts which move or support moving parts, shall be designed for an additional load taken as 20 percent of the dead load to allow for dynamic load allowance or vibratory effect. The 20 percent increase shall be the dead load dynamic load allowance. This 20 percent increase shall not be considered in the fatigue loading range.

Dead load dominant load combination

Rev P02

An additional load combination is required to address the lift span opening case with only dead loads present. This load combination shall use $\gamma_{fL\ ULS}$ = 1.80 (i.e. gamma ULS self weight = 1.50 * dynamic amplification factor of 1.20) on structure dead loads. This load factor includes the dynamic amplification allowance of 20 percent.

Superimposed Dead Load

Load factors for bridge deck surfacing over the approach spans shall be $\gamma_{fL\ SLS}$ = 1.00*1.55=1.55 and be $\gamma_{fL\ ULS}$ = 1.20*1.55 = 1.86 (Table NA.A2.4(B) of UK NA to BS EN 1990 and Table NA.1 of UK NA to BS EN 1991-1-1). This allows for the potential increase in self weight of surfacing over the fixed span portion of the bridge caused by maintenance operations by the Government of Bermuda resulting in the increased thickness of total surfacing material e.g. from overlay/surfacing dressing.

Load factors for bridge deck surfacing over the lift span of the bridge shall be $\gamma_{fL\ SLS}$ = 1.00 and $\gamma_{fL\ ULS}$ = 1.20 (Table NA.A2.4(B) of UK NA to BS EN 1990 and Table NA.1 of UK NA to BS EN 1991-1-1). As the weight of the lift span portion of the deck is critical, the Bermuda Government will be responsible for ensuring that the nominal superimposed dead loading from the surfacing materials will not be exceeded during the life of the structure, hence the reduced load factors are appropriate. This will be achieved by ensuring that during maintenance operations no additional surfacing thickness or material density increase will be permitted i.e. overlays or additional surface dressing will not be permissible; surfacing materials will need to be removed and replaced with the same thickness and material densities when the surfacing has reached the end of its serviceable life.

For the dead load dominant load combination noted above the partial load factor for surfacing
Rev P02 shall be taken as $\gamma_{fL\ ULS}$ = 1.80.

Wind Loading During Operation/Opening of the Bridge

During the normal operation/opening of the bridge, the maximum wind gust speed shall be 15.64m/s (35mph). The design for the operation/opening of the bridge will be carried out to a higher wind speed of 20.11m/s (45mph) so that there is a safety margin between the design and operation wind speeds, considering full operation of both cylinders.

To optimise the sizing of the operating equipment, a reduced limit for permissible wind speed shall be enforced if one lift cylinder is out of order. The design wind speed in that case will be 8.05m/s (18 mph).

The maximum wind gust speed for the case when the lift span is closed shall be 67.06m/s (150mph) including hurricane consideration as per Client's brief. During extreme winds (greater than 35mph) the lift span deck will be parked and locked.

An additional extreme wind load condition shall be considered in the design of structural components at ULS. This shall be the design wind load appropriate for operation/opening of the bridge (wind speed of 24.60m/s, or 55mph). This case is intended to allow for the extreme event where the bridge has been opened, or partly opened, under the normal twin cylinder conditions but becomes inoperable due to some fault condition, and then wind gust speeds increase above the normal operating limit. As an extreme event the ULS load factor (γfl) shall be taken as 1.0. This condition is not applied to single cylinder operations.

Rev P03 Wind load shall be considered in the calculation of the fatigue load range during opening cycles of the bridge as defined in the 'Fatigue Loading' section below. See M&E AIP for wind load allowances used in the fatigue loading of M&E components.

Emergency Stop Loading on Lift Span Opening

The inertia loading caused by the application of the emergency stop braking system shall be considered in the design as the application of an angular acceleration value on the mass resulting from dead load. Load factors will be those appropriate to the mass. This shall be included as a separate load case at ULS only during lifting operations at dead load dominant load combination and at combination with leading action being the wind load. The period over which the emergency stop occurs is likely to be in the region 2 to 3 seconds.

Loads imposed by hydraulic cylinder operation.

The cylinders are disengaged and will not provide support to the lift span when the bridge is closed.

The hydraulic cylinders are arranged to be approximately equivalent to a single point of support to provide a statically determinate three point support arrangement for the lift span during lifting. During bridge normal operation lifting it shall be assumed that both cylinders are equally loaded. During single cylinder lifting account shall be taken of the eccentricity of the cylinder connection relative to the centreline of the pair of cylinders.

Loading due to faults in cylinder operation shall be based on the failure scenarios in the Approval in Principle (M&E Installations). Cylinder fault loading shall be calculated based on the pressure release settings but shall not be more than the cylinder load required to lift the span dead load with the addition of dead load dynamic load allowance of 20% and shall be treated as a dead load with a $\gamma fl=1.2$ at ULS. This cylinder fault load shall be considered at ULS in combination with dead load and superimposed dead loading (SDL) only with $\gamma fl = 1.2$ applied to coexistent dead and SDL also. Dead load dynamic load allowance shall not be applied to the dead & SDL loads that coexist with the cylinder load since the deck will not be moving at any significant speed. Cylinder fault loads require additional resistance to lifting being present and these mechanisms shall be included in the application of this load. Such mechanisms may include restraint at bearing seats but not failure to withdraw nose locking pins. Cylinder fault loading is not a serviceability or fatigue condition and shall be considered for structure strength design (ULS) only.

Rev P02

Main Pier Hydrostatic/Hydrodynamic Loading and Flooding

Hydrostatic/Hydrodynamic loading will be considered on its own without vessel collision.

The soffit level of the lift span is raised in the design in order to raise the top surface of the main pier and pivots above level +4.0m OD. This has been necessary to mitigate potential for surge inundation of the cylinder chamber in the event of the 1 in 150 year hurricane surge (excluding SLR).

For purposes of design two cases (upper and lower bound) will be examined for the hydrostatic/hydrodynamic pressures applied onto the main pier chamber.

- Water pressure from 1 in 150 year hurricane surge (excluding SLR) +4.0m OD outside the chamber, with chamber being empty.
- Water pressure from the LAT -0.543m OD level outside the chamber, with the chamber being filled with water.

Rev P02

Buoyancy effects shall also be considered for the upper bound case of hurricane surge (excluding SLR).

The main pier chamber reinforced concrete elements will be designed in accordance with BS EN 1992-3 for liquid retaining structure requirements. The main pier chamber shall be Tightness Class 1 where any leakage is to be limited into small amounts, and some surface staining or damp patches are considered acceptable. To achieve this level of tightness the reinforced concrete elements shall be designed for the crack width values w_{k1} depending on hydrostatic pressure depth over the element thickness ratio h_D/h as below:

- For $h_D/h \leq 5 \rightarrow w_{k1} = 0.2mm$
- For $h_D/h \geq 35 \rightarrow w_{k1} = 0.05mm$

For intermediate values of h_D/h, linear interpolation between 0.2mm and 0.05mm will be used. Limitation of the crack widths to these values is considered to result in the effective sealing of the cracks within a relatively short time.

Scour

Rev P04

Scour and hydraulic actions on the bridge piers and abutments shall be considered via an assessment of scour risk for the proposed bridge foundations using the HEC-18 method. Appropriate scour mitigation measures will be designed as appropriate and if required.

The flow/tidal velocity appropriate to assess scour and design for mitigation measures is 0.3m/s based on maximum measured currents from Ferry Reach taken from Waters of Castle Harbour and Grotto Bay, Halcrow, 2010. Scour from wave action will be considered. Scour from Vessels travelling at 5 knots has been ruled out due to water depth at LAT.

Vehicle Restraint System (VRS) and Parapet

Rev P02

VRS/parapets on the bridge superstructure will be a bespoke lattice grille structure, comprising hollow section posts and hollow section upper and lower rails with vertical infill. The vertical orientation of the infill bars will prevent them from being climbed.

A risk assessment for the Road Restraint System requirement will be prepared and this will confirm the VRS design approach.

Assuming the VRS comply with BS EN 1317-2; with the performance class B (normal containment rigid parapet connections between restraint and kerb or bridge; as per Table 4.9(n), BS EN 1991-2:2003 - Section 4.7.3.3) to determine the equivalent average impact force assuming normal containment level N1 (appropriate for low speed permanent installations) and with 0.1m deflection, the average force is 200kN. This is based on 80kph collision at 20° (Tables 1&2, BS EN 1317-2:2010).

Rev P02

To determine an equivalent load for the situation of a road with the design speed of 50kph, the average force is multiplied by $50^2/80^2$ (i.e. the ratio of the velocities squared as the calculated force is proportional to velocity squared) which gives an equivalent force of 78.1kN.

Fatigue Loading

In accordance with Table NA.4 of UK NA to BS EN 1991-2 the fatigue loading for the approach spans and lift span shall be based on the travelled lane configuration; i.e. 2No. travelled lanes at 3.5m wide and shall comprise 0.5 x 10^6 (=N_{obs}) heavy goods vehicles per slow lane per year as for an all-purpose single carriageway.

The number N_{obs} represents heavy vehicles (maximum gross vehicle weight more than 100 kN), observed or estimated, per year and per slow lane (i.e. a traffic lane used predominantly by lorries).

Fatigue Load Model 3 (single vehicle model) in accordance with Clause 4.6.4 of BS EN 1991-2 will be used for the fatigue assessment from the traffic loads. This vehicle comprises 4 No. axles of 120 KN each resulting to a total vehicle load of 480KN.

Rev P02 In addition, fatigue shall be checked for the lifting span based on the number of times the bridge is to be opened in its 75 year design life; the number of lifting cycles is 2920 per annum, or 219000 for the design life of the structure (assuming an annual average of 8 No. operations from 10am to 4pm every day).

The total stress range used for this fatigue check shall be that arising from the operation of the span from fully closed to the fully opened position, and return, including the effect of the passage of the last fatigue vehicle before opening and the first fatigue vehicle after closing. Fatigue Load Model 3 will be used for this fatigue check.

Rev P03 Dead load dynamic load allowance shall not be included in the calculation of the fatigue stress range. However, 50% of the longitudinal operating wind load shall be considered in combination with dead loads to calculate the range of loads applied to the structural members during a lifting cycle.

For the assessment of the fatigue loading on M&E components 50% of the longitudinal operating wind load shall be considered in combination with dead loads to calculate the range of loads during a lifting cycle.

Steel elements will be assessed for safe life using the detail categories from Tables 8.1 to 8.10 of BS EN 1993-1-9:2005 and the fatigue loading described above. A value of γ_{Mf} = 1.1 will be adopted according to clause NA.2.5.3 of NA to BS EN 1993-1-9:2005.

Loading for Plant Room and Accessways

Access-ways to and within the plant rooms shall be designed for the imposed loading requirements BS EN ISO 14122-1:2016 'Safety of machinery – permanent means of access to machinery. Choice of fixed means and general requirements of access' appropriate for General Duty access. (UDL 5.0 kN/m2; Concentrated Load 1.0kN) γ_{fL} = 1.0 shall be used at the serviceability limit state (SLS) and γ_{fL} = 1.5 at the ultimate limit state (ULS) for all load combinations.

Plant Rooms shall be designed for the imposed loading requirements of BS EN 1991-1-1 'General actions- Densities, self-weight, imposed loads for buildings'. Loading to comply as a minimum with the appropriate loading provisions for buildings in Category E1. Imposed loads on floors due to storage are selected as UDL 10.0 kN/m^2 and Concentrated Load 7.0 kN. Load factor γ_{fL} = 1.0 shall be used at the serviceability limit state (SLS) and γ_{fL} = 1.5 at the ultimate limit state (ULS) for all load combinations.

<u>Loading within Steel work Box of Lift Span for Inspection and Maintenance</u>

Deck soffit plates shall be designed to accommodate live loading within the box structures for inspection and maintenance access. The live loading shall comprise a UDL of 1.5 kN/m^2 over a total area of 10 m^2 of any shape, which may be continuous or divided to give the most adverse effect, together with a UDL of 0.75 kN/m^2 elsewhere. γfL= 1.0 shall be used at the serviceability limit state (SLS) and γfL= 1.5 at the ultimate limit state (ULS) for all load combinations.

4.2 Heavy or high load route requirements and arrangements being made to preserve the route, including any provision for future heavier loads or future widening

Not applicable.

4.3 Minimum headroom provided

In the bridge open (road closed) position, unrestricted clearance will be provided over the 22m wide navigation channel.

In the bridge closed (road open) position, a mid-span clearance will be as below:

- From LAT (-0.543m) to +4.90m OD → Headroom = 5.443 m
- From HAT (+0.890m) to +4.90m OD → Headroom = 4.010 m

4.4 Authorities consulted and any special conditions required

Consultations with Statutory Undertakers are underway.

A full existing services site survey is to be performed by the Client and summarised in a combined services drawing to verify the location of each of the services and confirm which are live and which are redundant in order to inform a strategy for diversion and protection of services prior to construction and demolition works.

4.5 Standards and documents listed in the Technical Approval Schedule

See Appendix 1.

Rev P02

In addition, reinforcement to control early thermal cracking of reinforced elements will be designed in accordance with the requirements of CIRIA document, C766 – *Control of cracking caused by restrained deformation in concrete*. This document supersedes the previous CIRIA document C 660 relating to this subject. CIRIA C 660 is referred to in the Published Documents (PDs) to BS EN 1992-2 (PD 6687-2 cl. 8.2.3) and BS EN 1992-1-1 (PD 6687-1 cl. 2.21.3) and counts in Eurocode terminology as “NCCI” (Non Contradictory Complimentary Information). It is considered that CIRIA C 766 is a direct update of NCCI and therefore should be used immediately for new projects, and on this basis it is proposed for Swing Bridge Replacement.

4.6 Proposed Departures relating to departures from standards given in 4.5

None

4.7 Proposed Departures relating to methods for dealing with aspects not covered by standards in 4.5

None

5. STRUCTURAL ANALYSIS

5.1 Methods of analysis proposed for superstructure, substructure and foundations

Superstructure

The lift span and approach spans will both be analysed as a three-dimensional model (Model A) using the linear elastic analysis computer program. Both thick shell elements and beam elements as appropriate will be assigned to different parts of the structure to form the three-dimensional model.

Rev P02 The lift span will be modelled with shell elements using Sofistik and both bridge closed (open to traffic) and critical bridge opening conditions will be considered. The influence of the approach spans in this model will be considered as single beam elements and a construction stage analysis will be performed to review concrete pouring sequence and long term creep effects.

Dynamic mode shapes and frequencies will also be determined from the three dimensional model.

The composite deck of the approach spans will be modelled as a grillage model (Model B) using the linear elastic analysis computer program Lusas. The wet concrete case with the steelwork only sections will be considered together with the composite section assignment for the permanent in-service case where live loading is applicable. The composite section will be checked from the build
Rev P02 up of stresses resulting from the two previously mentioned analysis cases.

The substructure and pile caps will be analysed using standard elastic methods and/or hand calculations using force effects from the three dimensional model (Model A) as appropriate.

Pile loads will be determined from the three-dimensional model for the design of the piles. Pile loads may also be reviewed using the method of A.J.Francis ref ASCE Journal "Analysis of pile groups with flexural resistance" and expanded by Sawko in a paper in the Structural Engineer "A simplified approach to the analysis of piling systems.

5.2 Description and diagram of idealised structure to be used for analysis

The lift span will be idealised with shell elements using finite element analysis. The approach spans will be idealised with beam elements forming a grillage using finite element analysis.

See Appendix 2.

5.3 Assumptions intended for calculation of structural element stiffness

The stiffness of the steel elements will be based on the gross section properties and steel elastic moduli E=210GPa. For steel concrete composite sections cracked concrete section properties will be assumed over the supports as per the provisions of BS EN 1994-2.

The stiffness of the substructure concrete element will be based on elastic uncracked section properties.

5.4 Proposed range of soil parameters to be used in the design of earth retaining elements

The earth retaining elements identified are the abutments and the retaining walls.

The design of earth retaining elements will be in accordance with PD 6694-1:2011. The backfill material will be assumed as a free draining granular material with properties and grading conforming to Classes 6N or 6P, specified, installed and compacted in accordance with the Highway's Agency's Manual of Contract Documents for Highway Works (MCHW).

The surcharge loading behind the walls will be in accordance with Clause 7.6 of PD 6694-1:2011 for loading from normal traffic.

6. GEOTECHNICAL CONDITIONS

6.1 Acceptance of recommendations of the Geotechnical Design Report to be used in the design and reasons for any proposed changes

Rev P02 The Ground Investigation Report (GIR report no. 3502-RAM-XX-XX-RP-CE-30001) is now complete. Geotechnical parameters for use in the design of Swing Bridge Replacement are provided in the Geotechnical Report – Highway Structure Summary Information 'Form C' in Appendix 5.

6.2 Summary of design for highway structure in the Geotechnical Design Report

The Ground Investigation Report (GIR report no. 3502-RAM-XX-XX-RP-CE-30001) is now
Rev P02 complete. Geotechnical parameters for use in the design of Swing Bridge Replacement are provided in the Geotechnical Report – Highway Structure Summary Information 'Form C' in Appendix 5.

6.3 Differential settlement to be allowed for in the design of the structure

Rev P02 Differential settlement to be allowed in the design of the structure will be 10mm.

6.4 If the Geotechnical Report is not yet available, state when the results are expected and list the sources of information used to justify the preliminary choice of foundations.

Rev P02 The Ground Investigation Report (GIR report no. 3502-RAM-XX-XX-RP-CE-30001) is now complete. Geotechnical parameters for use in the design of Swing Bridge Replacement are provided in the Geotechnical Report – Highway Structure Summary Information 'Form C' in Appendix 5.

7. CHECK

7.1 Proposed Category and Design Supervision Level

Category 3, DSL3.

7.2 If Category 3, name of proposed Independent Checker

Government of Bermuda to appoint Category 3 checker.

7.3 Erection proposals or temporary works for which Types S and P Proposals will be required, listing structural parts of the permanent structure affected with reasons

Not applicable.

8. DRAWINGS AND DOCUMENTS

8.1 List of drawings (including numbers) and documents accompanying the submission

3502-RAM-SB-XX-DR-CB-30001 Rev. P01 - SWING BRIDGE REPLACEMENT, APPROVAL IN PRINCIPLE, GENERAL ARRANGEMENT, SHEET 1 OF 5

Rev P02 3502-RAM-SB-XX-DR-CB-30002 Rev. P02 - SWING BRIDGE REPLACEMENT, APPROVAL IN PRINCIPLE, GENERAL ARRANGEMENT, SHEET 2 OF 5

Rev P02 3502-RAM-SB-XX-DR-CB-30003 Rev. P02 - SWING BRIDGE REPLACEMENT, APPROVAL IN PRINCIPLE, GENERAL ARRANGEMENT, SHEET 3 OF 5

3502-RAM-SB-XX-DR-CB-30004 Rev. P01 - SWING BRIDGE REPLACEMENT, APPROVAL IN PRINCIPLE, GENERAL ARRANGEMENT, SHEET 4 OF 5

Rev P02 3502-RAM-SB-XX-DR-CB-30005 Rev. P02 - SWING BRIDGE REPLACEMENT, APPROVAL IN PRINCIPLE, GENERAL ARRANGEMENT, SHEET 5 OF 5

Rev P02 3502-RAM-SB-XX-DR-CB-30011 Rev. P02 - SWING BRIDGE REPLACEMENT, APPROVAL IN PRINCIPLE, CONSTRUCTION SEQUENCE.

Drawings included in Appendix 4.

9. THE ABOVE IS SUBMITTED FOR ACCEPTANCE

Signed	[signature]
Name	Eur Ing Steve Thompson
Engineering Qualifications	BEng CEng FICE
Position Held	Director
Name of Organisation	Ramboll
Date	16-05-2019

10. THE ABOVE IS REJECTED/AGREED SUBJECT TO THE AMENDMENTS AND CONDITIONS SHOWN BELOW:

Signed	[signature]
Name	Attila Fustos
Position Held	Principal Eng
Engineering Qualifications	MICE, CEng.
TAA	Bermuda Government
Date	27, May 2019

APPENDIX 1

TECHNICAL APPROVAL SCHEDULE (TAS)

Technical Approval Schedule (TAS)

Schedule of Documents Relating to Design of Highway Bridges and Structures
Documents relevant to this project are indicated by a tick.

Eurocodes

All national annexes will be used with the list of documents below. Documents relevant to this project are indicated by a tick

Used	Eurocode Part	Title	Publication Date	UK National Annex Publication Date
Eurocode 0 Basis of Structural Design				
✓	BS EN 1990	Eurocode 0: Basis of structural design	Jul-02	Dec-04
Eurocode 1 Actions on Structures				
✓	BS EN 1991-1-1	Actions on structures. General actions. Densities, self- weight, imposed loads for buildings	Jul-02	Dec-05
	BS EN 1991-1-3	Actions on structures. General actions. Snow loads	Jul-03	Dec-05
✓	BS EN 1991-1-4	Actions on structures. General actions. Wind actions	Apr-05	Sep-08
✓	BS EN 1991-1-5	Actions on structures. General actions. Thermal actions	Mar-04	Apr-07
✓	BS EN 1991-1-6	Actions on structures. General actions. Actions during execution	Dec-05	May-08
✓	BS EN 1991-1-7	Actions on structures. General actions. Accidental actions	Sep-06	Dec 08
✓	BS EN 1991-2	Actions on structures. Traffic loads on bridges	Oct-03	May-08
Eurocode 2 Design of Concrete Structures				
✓	BS EN 1992-1-1	Design of concrete structures – Part 1-1: General rules and rules for buildings	Dec-04	Dec-05
✓	BS EN 1992-2	Design of concrete structures – Part 2: Concrete bridges – Design and detailing rules.	Dec-05	Dec-07
✓	BS EN 1992-3	Design of concrete structures – Part 3: Liquid retaining and containment structures	Jul-06	Oct-07
Eurocode 3 Design of Steel Structures				
✓	BS EN 1993-1-1	Design of steel structures – Part 1-1: General rules and rules for buildings	May-05	Dec-08

Used	Eurocode Part	Title	Publication Date	UK National Annex Publication Date
	BS EN 1993-1-3	Design of steel structures – Part 1-3: General – General rules – Supplementary rules for cold-formed members and sheeting.	Nov-06	Mar-09
✓	BS EN 1993-1-4	Design of steel structures – Part 1-4: General rules – Supplementary rules for stainless steel.	Nov-06	Mar-09
✓	BS EN 1993-1-5	Design of steel structures – Part 1-5: Plated structural elements	Nov-06	May-08
✓	BS EN 1993-1-6	Design of steel structures – Part 1-6: General – Strength and stability of shell structures	May-07	Mar-09
✓	BS EN 1993-1-7	Design of steel structures – Part 1-7: General – Plated structures subject to out of plane loading.	Jul-07	Mar-09
✓	BS EN 1993-1-8	Design of steel structures – Part 1-8: General – Design of joints	May-05	Dec-08
✓	BS EN 1993-1-9	Design of steel structures – Part 1-9: Fatigue	May-05	May-08
✓	BS EN 1993-1-10	Design of steel structures – Part 1-10: Material toughness and through thickness properties.	May-05	Dec-08
	BS EN 1993-1-11	Design of steel structures – Part 1-11: Design of structures with tension components	Nov-06	Dec-08
	BS EN 1993-1-12	Design of steel structures – Part 1-12: Additional rules for the extension of EN 1993 up to steel grades S 700.	May-07	May-08
✓	BS EN 1993-2	Design of steel structures – Part 2 Steel Bridges	Nov-06	May-08
✓	BS EN 1993-5	Design of steel structures – Part 5 Piling	Apr-07	Mar-09
Eurocode 4 Design of Composite Steel and Concrete Structures				
✓	BS EN 1994-1-1	Design of composite steel and concrete structures – Part 1-1: General rules and rules for buildings	Feb-05	Aug-08
✓	BS EN 1994-2	Design of composite steel and concrete structures – Part 2 General rules and rules for bridges.	Dec-05	Dec-07

Used	Eurocode Part	Title	Publication Date	UK National Annex Publication Date
Eurocode 5 Design of Timber Structures				
	BS EN 1995-1-1	Design of timber structures – Part 1-1: General – Common rules and rules for buildings	Dec-04	Oct-06
	BS EN 1995-1-2	Design of timber structures – Part 1-2: General – Structural fire design	Dec-04	Oct-06
	BS EN 1995-2	Design of timber structures – Part 2 Bridges	Dec-04	Oct-06
Eurocode 6 Design of Masonry Structures				
	BS EN 1996-1-1	Design of masonry structures – Part 1-1: General rules for reinforced and unreinforced masonry structures.	Dec-05	May-07
	BS EN 1996-1-2	Design of masonry structures – Part 1-2: General – Structural fire design	Jun-05	May-07
	BS EN 1996-2	Design of masonry structures – Part 2 Design considerations, selection of materials and execution of masonry.	Feb-06	May-07
	BS EN 1996-3	Design of masonry structures – Part 3: Simplified calculation methods for unreinforced masonry structures	Feb-06	May-07
Eurocode 7 Geotechnical design				
✓	BS EN 1997-1	Geotechnical design – Part 1 General rules	Dec-04	Nov-07
✓	BS EN 1997-2	Geotechnical design – Part 2 Ground investigation and testing	Apr-07	Mar 09
Eurocode 8 Design Of Structures For Earthquake Resistance				
✓	BS EN 1998-1	Design of structures for earthquake resistance – Part 1 General rules, seismic actions and rules for buildings	Apr-05	Aug-08
✓	BS EN 1998-2	Design of structures for earthquake resistance – Part 2 Bridges	Dec-05	Feb-09
✓	BS EN 1998-5	Design of structures for earthquake resistance – Part 5 Foundations, retaining structures and geotechnical aspects	Apr-05	Aug-08
Eurocode 9 Design Of Aluminium Structures				
	BS EN 1999-1-1	Design of aluminium structures – Part 1-1 General structural rules	Aug-07	Dec-08
	BS EN 1999-1-2	Design of aluminium structures – Part 1-2: General – Structural fire design	Apr-07	Mar-09

Used	Eurocode Part	Title	Publication Date	UK National Annex Publication Date
	BS EN 1999-1-3	Design of aluminium structures – Part 1-3 Structures susceptible to fatigue.	Aug-07	Dec-08
	BS EN 1999-1-4	Design of aluminium structures – Part 1-4 Cold formed structural sheeting	Apr-07	Mar-09
	BS EN 1999-1-5	Design of aluminium structures – Part 1-5: Supplementary rules for shell structures	Apr-07	Mar-09

BSI Published Documents

Used	Document Reference	Title
BSI Published Documents		
✓	PD 6688-1-1	Background paper to the UK National Annex to BS EN 1991-1-1
✓	PD 6688-1-4	Background paper to the UK National Annex to BS EN 1991-1-4
✓	PD 6688-1-7	Recommendations for the design of structures to BS EN 1991-1-7
✓	PD 6688-2	Recommendations for the design of structures to BS EN 1991-2
✓	PD 6687-1	Background paper to the UK National Annex to BS EN 1992-1 and BS EN 1992-3
✓	PD 6687-2	Recommendations for the design of structures to BS EN 1992-2
✓	PD 6694-1	Recommendations for the design of structures subject to traffic loading to BS EN 1997-1:2004
✓	PD 6695-1-9	Recommendations for the design of structures to BS EN 1993-1-9
✓	PD 6695-1-10	Recommendations for the design of structures to BS EN 1993-1-10
✓	PD 6695-2	Recommendations for the design of bridges to BS EN 1993
✓	PD 6696-2	Background paper to BS EN 1994-2 and the UK National Annex to BS EN 1994-2
✓	PD 6698	Recommendations for the design of structures for earthquake resistance to BS EN 1998
✓	PD 6703	Structural bearings – Guidance on the use of structural bearings
✓	PD 6705-2	Recommendations on the execution of steel bridges to BS EN 1090-2

Execution Standards

Used	Document Reference	Title	Date
✓	BS EN 1090-1 +A1:2011	Execution of steel structures and aluminium structures. Requirements for conformity assessment of structural components	2009
✓	BS EN 1090-2 +A1:2011	Execution of steel structures and aluminium structures. Technical requirements for steel structures	2008
✓	BS EN 1090-3:2008	Execution of steel structures and aluminium structures. Technical requirements for aluminium structures	2008
✓	BS EN 13670:2009	Execution of concrete structures	2009
✓	BS EN 1536:2000	Execution of special Geotechnical Work – Bored Piles	2000

Product Standards referenced in British Standards or Eurocodes

Used	Document Ref	Title	Date
✓	BS EN 206-1	Concrete. Specification, performance, production and conformity	2000
✓	BS EN 1317-1-2010	Road Restraints Systems – Part 1, Terminology and general criteria for test methods	2010
✓	BS EN 1317-2-2010	Road Restraints Systems – Part 2, Performance classes, impact test acceptance criteria and test methods for safety barriers	2010
✓	BS EN 1317-3-2010	Road Restraints Systems – Part 3, Performance classes, impact test acceptance criteria and test methods for crash cushions	2010
✓	DD ENV 1317-4-2002	Road Restraints Systems – Part 4, Performance classes, impact test acceptance criteria and test methods for terminals and transitions of safety barriers	2002
✓	BS EN 1337	Structural Bearings, Parts 1 - 11.	Various
✓	BS EN 10025	Hot rolled products of structural steels, Pt 1 to 6	2004
✓	BS EN 10080	Steel for the reinforcement of concrete. Weldable reinforcing steel	2005
✓	BS EN 10210	Hot finished structural hollow sections of non-alloy and fine grain steels, Parts 1 and 2	2006
	BS EN 15050	Precast concrete products - Bridge elements	2007
	BS EN 14844 (+A2: 2011)	Box culverts	2006
	BS EN 15258	Retaining wall elements	2008

	BS EN 12843	Masts and poles	2004
	BS EN 12794	Foundation piles	2005

British Standards

✓	BS 4449:2005 +A2:2009	Steel for the reinforcement of concrete: Weldable reinforcing steel – Bar, coil and decoiled product – Specification	2009
	BS 5896; 1980 (inc Amdt No.1)	Specification for high tensile steel wire and strand for the prestressing of concrete	1980
✓	BS 8002; 1994	Earth retaining structures	1994
✓	BS 8004; 1986	Foundations	1986
	BS 8006; 1995	Strengthened/reinforced soils and other fills	1995
✓	BS 8500:	Concrete - Complementary British Standard to BS EN 206-1:	
✓	Part 1;	Method of specifying and guidance for the specifier	2016
✓	Part 2;	Specification for constituent materials and concrete	2016
✓	BS 8666:2005	Scheduling, dimensioning, bending and cutting of steel reinforcement for concrete- Specification	2005
✓	BS 7818:1995	Specification for pedestrian restraint systems in metal	1995
	BS EN 13369 (+A1: 2006)	Common rules for precast concrete products	2004

Miscellaneous Standards

✓		International Building Code	2012
✓		Bermuda Building Code	2014
✓		Bermuda Residential Building Code	2014
✓		AASHTO LFRD Bridge Design Specifications 7th Edition	2014
✓		AASHTO BVCS Bridges Vulnerable to Coastal Storms	2008
Rev P02 ✓	CIRIA C 766	Control of cracking caused by restrained deformation in concrete	2018

The Manual of Contract Documents for Highway Works (MCDHW)

Volume 1: Specification for Highway Works, Amendment Feb 2016	✓
Volume 2: Notes for Guidance on the Specification for Highway Works, Amendment Feb 2016	✓
Volume 3: Highway Construction Details, Amendment Nov 2008	✓

The Design Manual for Roads and Bridges (DMRB)

Bridges and Structures, Advice Notes (BA Series)				
BA 9/81	The Use of BS 5400: Part 10: 1980. Code of Practice for Fatigue Amendment No. 1	Dec 1981 Nov 1983	1.3	
BA 16/97	The Assessment of Highway Bridges and Structure Amendment No. 1 Amendment No. 2	May 1997 Nov 1997 Nov 2001	3.4.4	
BA 19/85	The Use of BS 5400: Part 3: 1982	Jan 1985	1.3	
BA 26/94	Expansion Joints for Use in Highway Bridge Decks	Nov 1994	2.3.7	✓
BA 28/92	Evaluation of Maintenance Costs in Comparing Alternative Designs for Highway Structures	Aug 1992	1.2.2	
BA 35/90	Inspection and Repair of Concrete Highway Structures	Jun 1990	3.3	
BA 36/90	The Use of Permanent Formwork	Feb 1991	2.3	✓
BA 37/92	Priority Ranking of Existing Parapets	Oct 1992	2.3.2	
BA 38/93	Assessment of the Fatigue Life of Corroded or Damaged Reinforcing Bars	Oct 1900	3.4.5	
BA 39/93	Assessment of Reinforced Concrete Half-joints	Apr 1993	3.4.6	
BA 40/93	Tack Welding of Reinforcing Bars	Apr 1993	1.3.4	
BA 41/98	The Design and Appearance of Bridges	Feb 1998	1.3.11	
BA 42/96	The Design of Integral Bridges [Incorporating Amendment No.1 dated May 2003]	Nov 1996	1.3.12	
BA 47/99	Waterproofing and Surfacing Concrete Bridge Decks	Aug 1999	2.3.5	✓
BA 51/95	The Assessment of Concrete Structures Affected by Steel Corrosion	Feb 1995	3.4.13	
BA 52/94	The Assessment of Concrete Highway Structures Affected by Alkali Silica Reaction	Nov 1994	3.4.10	
BA 53/94	Bracing Systems and the Use of U-Frames in Steel Highway Bridges	Dec 1994	1.3.13	
BA 54/94	Load Testing for Bridge Assessment	Apr 1994	3.4.8	
BA 55/06	The Assessment of Bridge Substructures and Foundations, Retaining Walls and Buried Structures	May 2006	3.4.9	
BA 57/01	Design for Durability	Aug 2001	1.3.8	✓
BA 58/94	Design of Bridges and Concrete Structures with External Unbonded Prestressing	Nov 1994	1.3.10	
BA 59/94	Design of Highway Bridges for Hydraulic Action	May 1994	1.3.6	✓
BA 67/96	Enclosure of Bridges	Aug 1996	2.2.8	
BA 72/03	Maintenance of Road Tunnels	May 2003	3.2.3	
BA 82/00	Formation of Continuity Joints in Bridge Decks	Nov 2000	2.3.7	
BA 83/02	Cathodic Protection for Use in Reinforced Concrete Highway Structures	Feb 2002	3.3.3	
BA 85/04	Coatings for Concrete Highway Structures & Ancillary Structures	May 2004	2.4.3	
BA 86/06	Advice Notes on the Non-Destructive Testing of Highway Structures	Aug 2006	3.1.7	
BA 87/04	Management of Corrugated Steel Buried Structures Correction No.2	Aug 2004 Nov 2009	3.3.4	

BA 88/04	Management of Buried Concrete Box Structures	Aug 2004	3.3.5	
BA 92/07	The Use of Recycled Aggregates in Structural Concrete	May 2007	2.3.9	✓
BA 93/09	Structural Assessment of Bridges with Deck Hinges	Feb 2009	3.1.5	
Bridges and Structures, Standards (BD Series)				
BD 2/12	Technical Approval of Highway Structures	Aug 2005	1.1.1	✓
BD 7/01	Weathering Steel for Highway Structures	Nov 2001	2.3.8	
BD 9/81	Implementation of BS 5400: Part 10: 1980. Code of Practice for Fatigue	Dec 1981	1.3	
BD 10/97	Design of Highway Structures in Areas of Mining Subsidence	May 1997	1.3.14	
BD 12/01	Design of Corrugated Steel Buried Structures with Spans Greater than 0.9 Metres and up to 8.0 Metres	Nov 2001	2.2.6	
BD 13/06	Design of Steel Bridges. Use of BS 5400-3: 2000	May 2006	1.3.14	
BD 15/92	General Principles for the Design and Construction of Bridges. Use of BS 5400: Part 1: 1988	Dec 1992	1.3.2	
BD 16/82	Design of Composite Bridges. Use of BS 5400:Part 5:1979 Amendment No.1	Nov 1982 Dec 1987	1.3	
BD 20/92	Bridge Bearings. Use of BD 5400: Part 9: 1983	Oct 1992	2.3.1	
BD 21/01	The Assessment of Highway Bridges and Structures	May 2001	3.4.3	
BD 27/86	Materials for the Repair of Concrete Highway Structures	Nov 1986	3.3	
BD 29/04	Design Criteria for Footbridges	Aug 2004	2.2.8	
BD 30/87	Backfilled Retaining Walls and Bridge Abutments	Aug 1987	2.1	✓
BD 31/01	The Design of Buried Concrete Box and Portal Frame Structures	Nov 2001	2.2.12	
BD 33/94	Expansion Joints for Use in Highway Bridge Decks	Nov 1994	2.3.6	✓
BD 35/14	Quality Assurance Scheme for Paints and Similar Protective Coatings	May 2006	2.4.1	✓
BD 36/92	Evaluation of Maintenance Costs in Comparing Alternative Designs for Highway Structures (See Appendix B)	Aug 1992	1.2.1	
BD 37/01	Loads for Highway Bridges	Aug 2001	1.3.14	
BD 43/03	The Impregnation of Reinforced and Prestressed Concrete Highway Structures using Hydrophobic Pore-Lining Impregnants	Feb 2003	2.4.2	
BD 44/15	The Assessment of Concrete Highway Bridges and Structures	Jan 1995	3.4.14	
BD 45/93	Identification Marking of Highway Structures	Aug 1993	3.1.1	
BD 47/99	Waterproofing and Surfacing of Concrete Bridge Decks	Aug 1999	2.3.4	
BD 48/93	The Assessment and Strengthening of Highway Bridge Supports	Jun 1993	3.4.7	
BD 49/01	Design Rules for Aerodynamic Effects on Bridges	May 2001	1.3.3	
BD 51/14	Portal and Cantilever Signs/Signal Gantries	May 1998	2.2.4	
BD 53/95	Inspection and Records for Road Tunnels	Jul 1995	3.1.6	

BD 54/15	Post-tensioned Concrete Bridges Prioritisation of Special Inspections	Apr 1993	3.1.2	
BD 56/10	The Assessment of Steel Highway Bridges and Structures	Nov 1996	3.4.11	
BD 57/01	Design for Durability	Aug 2001	1.3.7	✓
BD 58/94	The Design of Concrete Highway Bridges and Structures with External and Unbonded Prestressing	Nov 1994	1.3.9	
BD 60/04	Design of Highway Bridges for Vehicle Collision Loads	May 2004	1.3.5	
BD 61/10	The Assessment of Composite Highway Bridges and Structures	Nov 1996	3.4.16	
BD 62/07	As Built, Operational and Maintenance Records for Highway Structures	Feb 2007	3.2.1	
BD 63/07	Inspection of Highway Structures	Feb 2007	3.1.4	
BD 65/14	Design Criteria for Collision Protector Beams	Feb 1997	2.2.5	
BD 67/96	Enclosure of Bridges	Aug 1996	2.2.7	
BD 70/03	Strengthened/Reinforced Soils and Other Fills for Retaining Walls and Bridge Abutments. Use of BS8006: 1995, incorporating Amendment No.1 (Issue 2 March 1999)	May 2003	2.1.5	
BD 78/99	Design of Road Tunnels	Aug 1999	2.2.9	
BD 79/13	The Management of Sub standard Highway Structures	Aug 2006	3.4.18	
BD 81/02	Use of Compressive Membrane Action in Bridge Decks	May 2002	3.4.20	
BD 82/00	Design of Buried Rigid Pipes	Aug 2000	2.2.10	
BD 84/02	Strengthening of Concrete Bridge Supports Vehicle Impact Using Fibre Reinforced Polymers	Aug 2002	1.3.16	
BD 85/08	Strengthening Highway Structures Using Externally Bonded Fibre Reinforced Polymer	Nov 2008	1.3.18	
BD 86/11	The Assessment of Highway Bridges and Structures For The Effects of Special Types General Order (STGO) and Special Order (SO) Vehicles	Nov 2007	3.4.19	
BD 87/05	Maintenance Painting of Steelwork	May 2005	3.2.2	
BD 89/03	The Conservation of Highway Structures	Nov 2003	3.2.4	
BD 90/05	Design of FRP Bridges and Highway Structures	May 2005	1.3.17	
BD 91/04	Unreinforced Masonry Arch Bridges	Nov 2004	2.2.14	
BD 94/07	Design of Minor Structures	Feb 2007	2.2.1	
BD 95/07	Treatment of Existing Structures on Highway Widening Schemes	Aug 2007	1.2.3	
BD 97/12	The Assessment of Scour and Other Hydraulic Actions at highway Structures	May 2012	3.4.21	
BD 100/16	The Use of Eurocodes for the Design of Highway Structures	Nov 2016	1.3.19	✓
BD 101/11	Structural Review and Assessment of Highway Structures	Nov 2100	3.4.22	
Bridges and Structures, Technical Memoranda (BE Series)				
BE 13	Fatigue Risk in Bailey Bridges	Apr 1968	3.4	
BE 23	Shear Key Decks	Nov 1970	1.3	

	Amendment No. 1 to Annex	June 1971		
BE 5/75	Rules for the Design and Use of Freyssinet Concrete Hinges in Highway Structures	Mar 1975	1.3	
BE 7/04	Departmental Standard (Interim) Motorway Sign/Signal Gantries	Aug 2004	2.2	
Traffic Engineering and Control, Standards (TA and TD Series)				
TD 9/93	Highway Link Design Amendment No. 1	Jun 1993 Feb 2002	6.1.1	✓
TD 19/06	Requirement for Road Restraint Systems	Aug 2006	2.2.8	✓
TD 27/05	Cross Sections and headroom	Feb 2005	6.1.2	
TD 36/93	Subways for pedestrians and cyclists, layout and dimensions	Jul 1993	6.3.1	
Highways, Advice Notes (HA Series)				
HA 66/95	Environmental Barriers - Technical Requirements	Sep 1995	10.5.2	
Highways, Standards (HD Series)				
HD 22/08	Managing Geotechnical Risks	Aug 2008	4.1	✓

Interim Advice Notes

124/11 (Jul 11)	Use of Eurocodes for the design of highway structures	✓
122/09 (Jun 09)	Rapid Condition Assessment of Hard Shoulder Pavements. Interim guide to data and maintenance advice	
121/09 (Jun 09)	Advice regarding implementation of Integrated Traffic Management	
117/08 Rev 1 (Jun 09)	Certification of combined kerb and drainage products	
116/08 (Oct 08)	Nature Conservation in Relation to bats	
115/08 (Nov 08)	Hard shoulder working	
114/08 (Sep 08)	Highways agency carbon calculation and reporting requirements	
113/08 (Jul 08)	Temporary Automatic Speed Camera System for the Enforcement of Mandatory Speed Limits at Roadworks (TASCAR)	
112/08 (Jun 08)	Managed Motorway Implementation Guidance – Through Junction Hard Shoulder Running [PR 100/08]	
111/08 (Jun 08)	Managed Motorway Implementation Guidance – Dynamic Use of Hard Shoulder [PR 99/08]	
110/08 (Apr 08)	Assessment of Implications (Of Highways Plans and Projects) On European Sites (Including Appropriate Assessment)	
109/08 (Apr 08)	Advice Regarding the Motorway Signal Mark 4 (MS4)	
107/08 (Feb 08)	Variable Demand Modelling As Part Of A Transport Assessment For The Highways Agency	
106/08 (Jan 08)	Guidance Note for Traffic Consultants Employed on Highways Agency Schemes	
105/08 (Jan 08)	Implementation of Construction (Design and Management) 2007 and the withdrawal of SD 10 and SD 11	
104/07 (Dec 07)	The Anchorage of Reinforcement & Fixings in Hardened Concrete	✓
103/08 (Mar 08)	Ramp metering	
100/07 (Oct 07)	Cultural Heritage Asset Management Plans	
99/07 (Nov 07)	Implementation of Local Grid Referencing System for England	

98/07 (Sep 07)	HD 28 Guidance for HA Service Providers on Implementing the Skid Resistance Policy	
97/07 (Aug 07)	Assessment and Upgrading of Existing Parapets	
96/07r1 (Aug 07)	Guidance On Implementing Results Of Research On Bridge Deck Waterproofing	✓
95/07 (May 07)	Revised guidance regarding the use of BS8500(2006) for the design and construction of structures using concrete.	✓
93/07 (Apr 07)	Driver location signs – Interim Performance Specification	
91/07 (Mar 07)	Interim Advice on the identification of 'Particularly at Risk' Supports	
90/07 Rev 01 (Apr 07)	Guidance For The Use Of Rapid Setting Emergency Repair Materials	
87/07 (Mar 07)	The Provision Of Signal Gantries For Motorways With Four Or More Running Lanes	
86/07 (Jun 07)	Amendments to design requirements for Portal and cantilever Sign/Signal Gantries.	
85/07 (Jun 07)	Design of Passively Safe Portal Signal Gantries	
84/07 (Jul 07)	Environmental Information System (EnvIS)	
83/06 (Jun 06)	Principal and General Inspection of Sign/Signal Gantries, and Gantries with low handrails or open mesh flooring.	
75/06 (May 06)	Code of Practice for Emergency Access to and Egress from the Trunk Road Network in England	
73/06 Rev 1 (Feb 09)	Design of Pavement Foundations	
71/06 (Feb 06)	Marker Posts On Lay-By Segregation Islands	
70/06 (Jan 06)	Implementation of New Reinforcement Standards (BS 4449:2005, BS 4482:2005, BS 4483:2005 and BS 8666:2005)	✓
69/05 (Dec 05)	Designing for Maintenance	✓
68/05 (May 06)	Infrastructure changes to improve emergency access to and egress from the trunk road network in England	
64/05 (Apr 05)	Driver Information At Road Works	
63/05r1 (Feb 07)	Asbestos Management Applicable to the Strategic Road Network	
56/04 (Aug 04)	Maintenance Of Traffic Signs With Dew Resistant Coatings	
53/04 (Feb 04)	Concrete Half-Joint Deck Structures	
51/03 (Jul 03)	Hinge Deck Structures	
49/03 (Mar 03)	Use of Warning Signs For New Asphalt Road Surfaces	
48/03 (Jan 03)	Measures to Minimise the Risk of Sulphate Attack (Including Thaumasite) – New Construction and Structures Under Construction	✓
47/02 (Dec 02)	Post Tensioned Grouted Duct Concrete Bridges	
41/02 (Jan 02)	European Cement Standards	✓
39/01 (Jun 01)	Post Opening Project Appraisal (POPE)	
36/01 (Jun 01)	The Use and Application of Micro-Simulation and Traffic Models	
05/96 (July 96)	BD 24/92 The Design of Concrete Highway Bridges and Structures. Use of BS 5400: Part 4: 1990	
04/96 (July 96)	BD 44/95 The Assessment of Concrete Highway Bridges and Structures	
03/96 (June 96)	BA 50/93 Post Tensioned Concrete Bridges	
01/95 (Oct 95)	TD 37/93 Scheme Assessment Reporting	

APPENDIX 2

DIAGRAM OF IDEALISED STRUCTURE TO BE USED FOR ANALYSIS

Rev P02

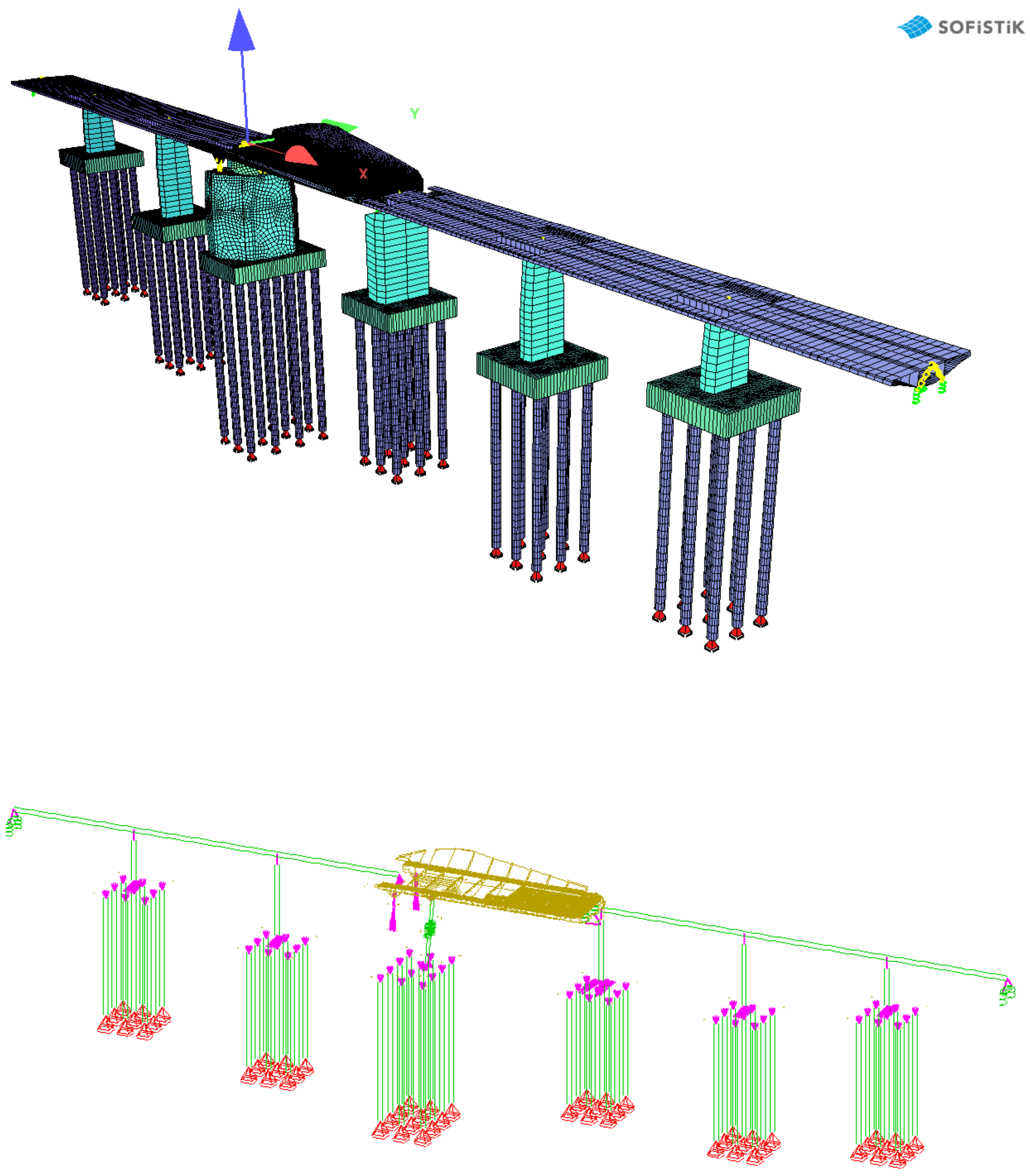


Figure 8 – Swing Bridge Replacement (SBR) Global Model – 3D Isometric view with Lift Span deck in closed position (open to vehicle traffic)

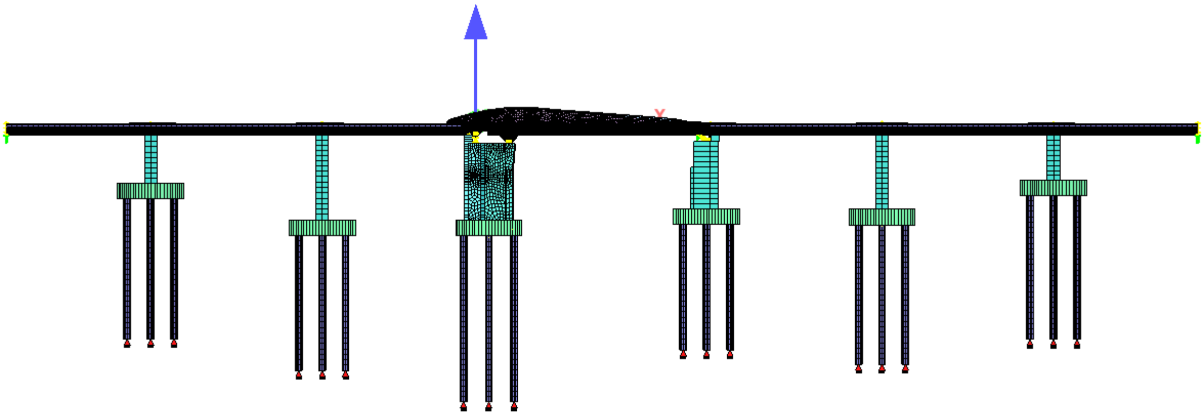


Figure 9 - SBR Global Model – Elevation view with Lift Span deck in closed position (open to vehicle traffic)

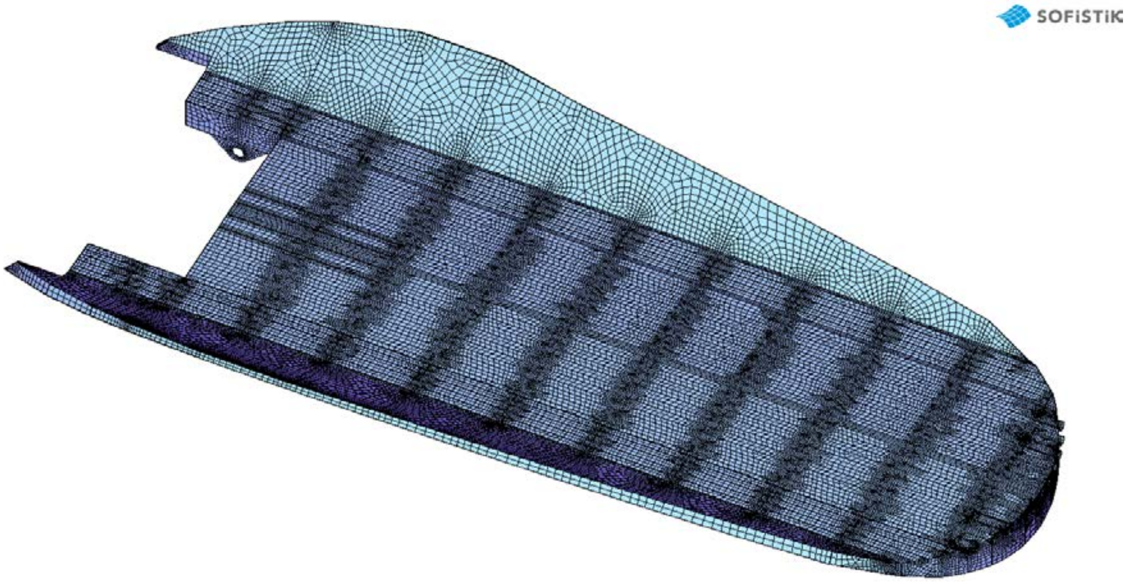


Figure 10 - SBR Global Model – 3D view of Lift Span deck in isolation

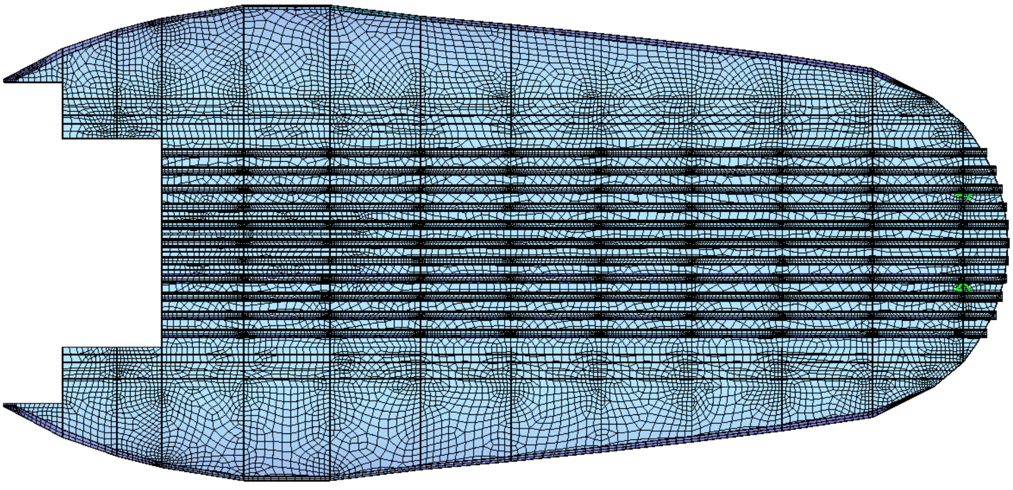


Figure 11 - SBR Global Model – Plan view of Lift Span deck in isolation excluding top flange plate

Rev P02

Rev P02

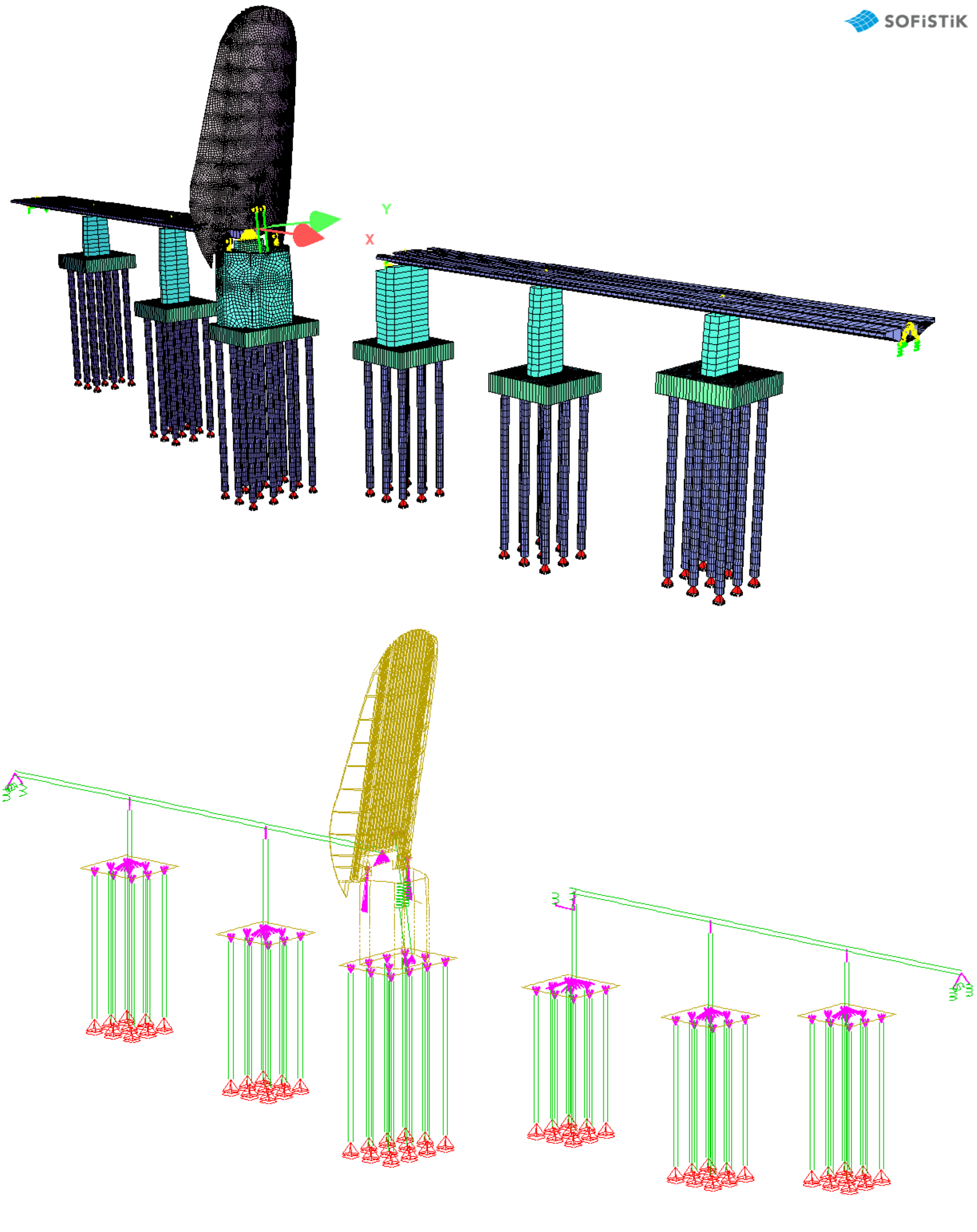


Figure 12 - SBR Global Model – 3D Isometric view with Lift Span deck in open position (closed for vehicle traffic)

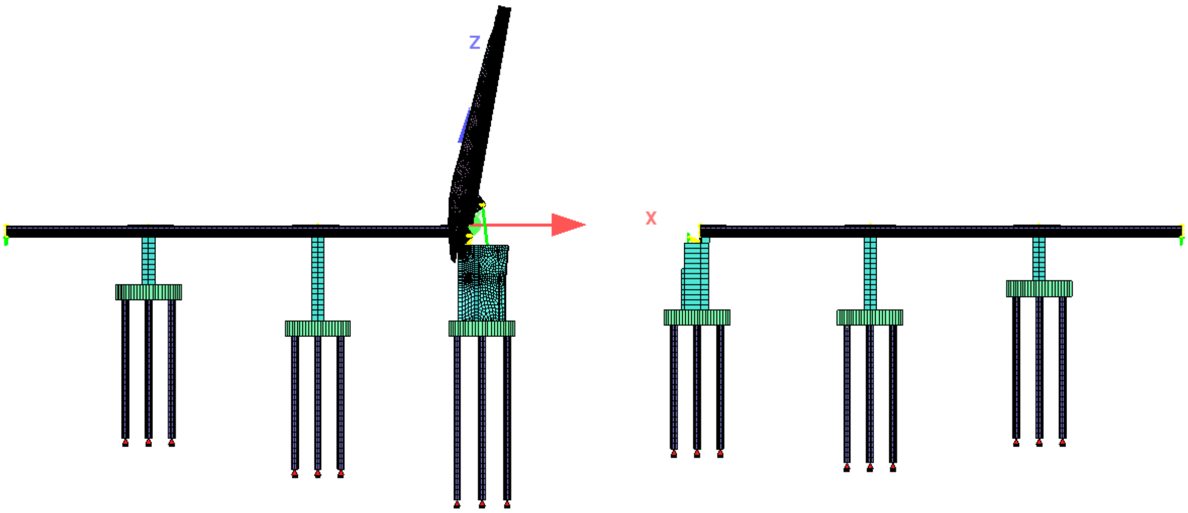


Figure 13 - SBR Global Model – Elevation view with Lift Span deck in open position (closed for vehicle traffic)

Rev P02

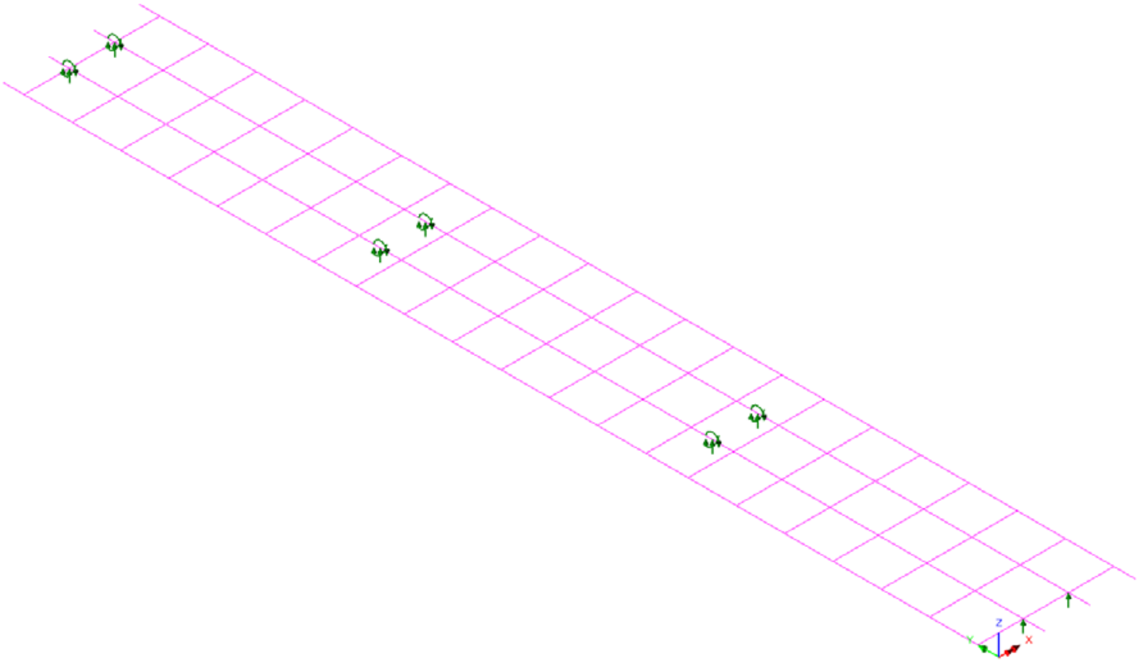
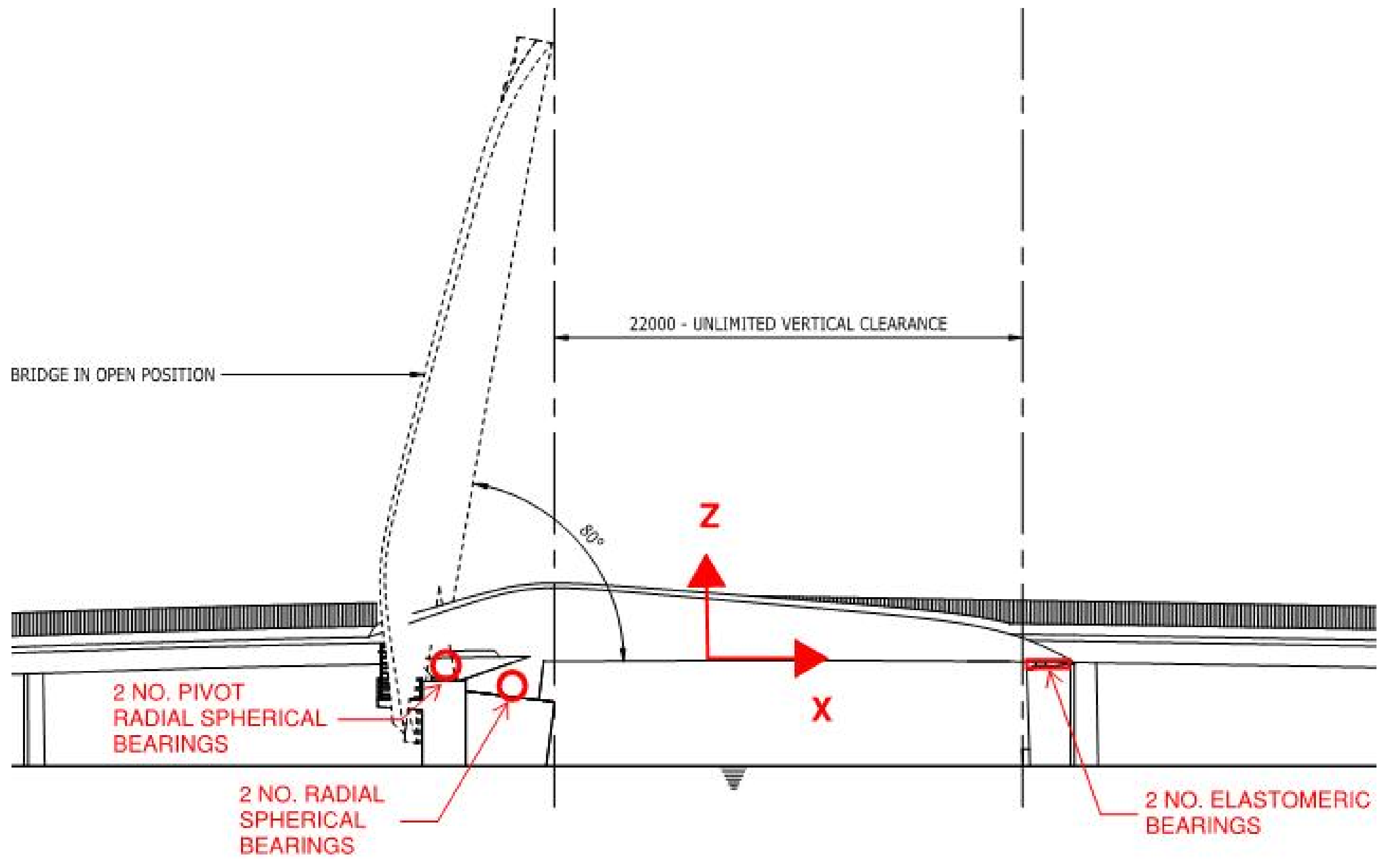


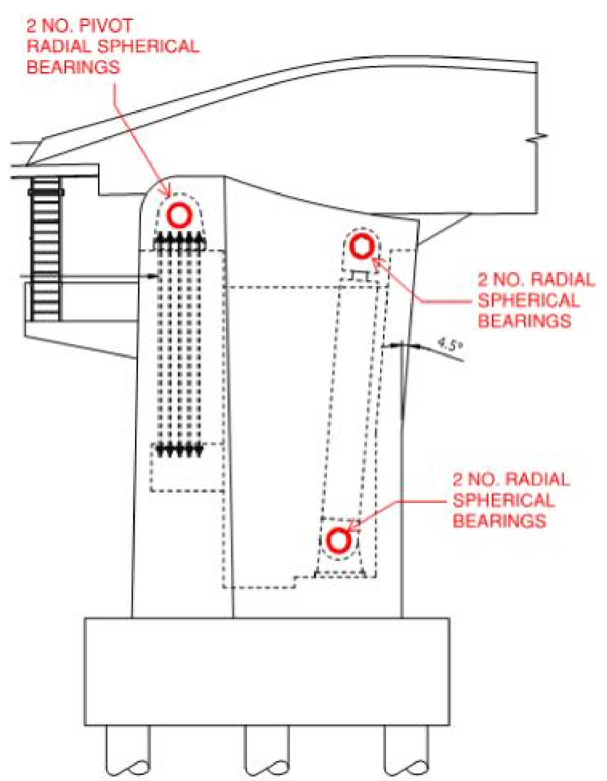
Figure 14 - SBR Grillage Model for the approach spans – 3D Isometric view

Figure 15 - SBR Grillage Model for the approach spans – Plan view

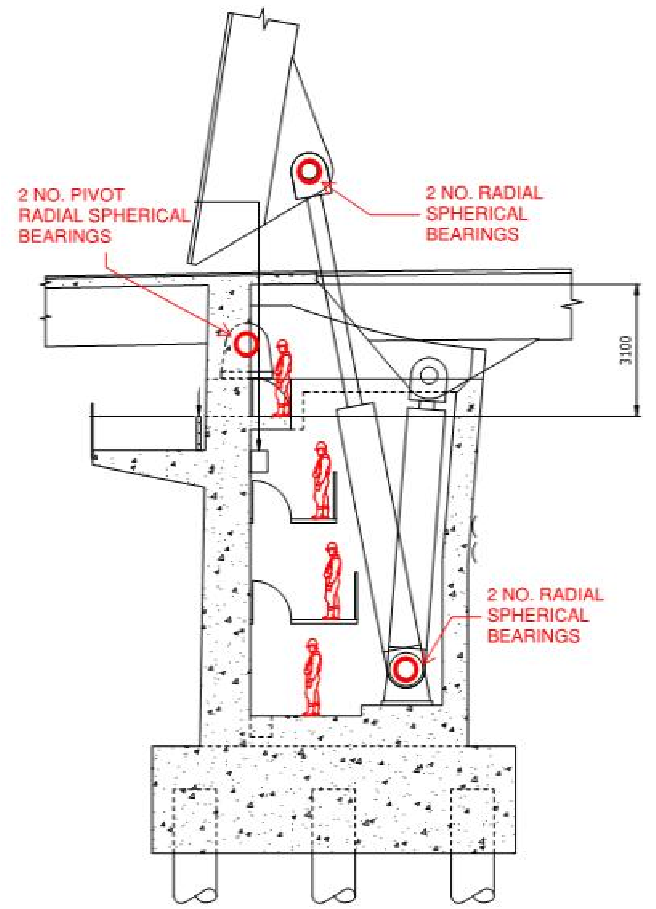
Idealised Structure Diagram



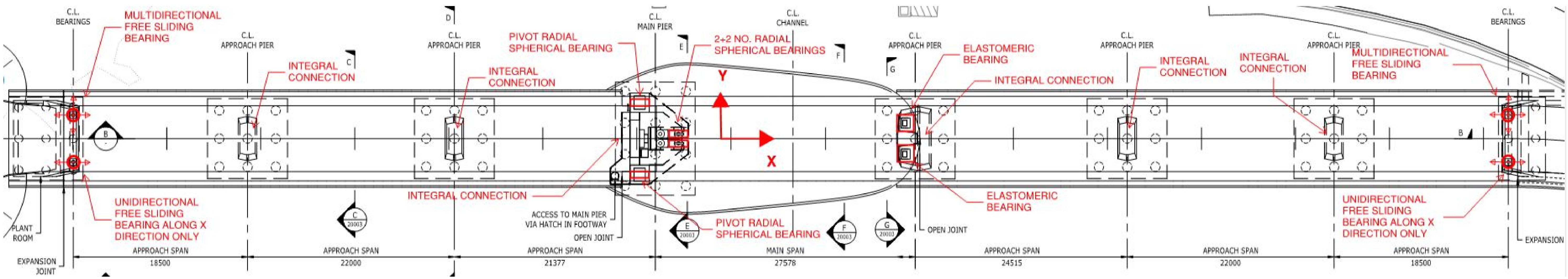
Articulation of Lift Span in Elevation



Articulation through Main Pier (Bridge Closed)



Articulation through Main Pier (Bridge Open)



Articulation of Swing Bridge Replacement in Plan

APPENDIX 3

DESIGNER’S RISK ASSESSMENT

CDM DESIGNER'S RISK ASSESSMENT

Project Name: Swing Bridge Replacement				**Job No: 1620003502**			**Document Ref:**
Stage / Section of works:				**Issue date/rev: 30/11/2018**			**Project Director Approval:**
Item No.	Activity or Element Reference	Design item giving rise to hazard What decisions are being made or what is being designed which is creating a hazard	Consequence of item giving rise to risk What are the probable consequences of this design item	Persons at Risk Site	Persons at Risk Others	Design action to eliminate risk or reduce risk The measures to be taken such as alternatives to be designed, information to be transferred onto drawings or in other documentation	Residual Risks Information about the risks that cannot be designed out and require controls to be developed and implemented by others
A	**General**						
A01	Construction	Construction of carriageway/footway/retaining structures/parapets/fencing/curb	Site workers falling from height. Materials & equipment falling from height and injuring persons below.	**X**		Minimise size/weight of elements with efficient design as far as possible. Show elevation on construction drawings to indicate the height from the water level to top of deck surfacing. Permanent formwork will be used for the bridge deck slab forming a safe working platform for the deck reinforcement fixing and concrete deck pour.	Contractor to implement suitable staff training and certification for working at heights along with ensuring that proper PPE is worn and that safety precautions to the OSHA/NEBOSH regulations are implemented Contractor to ensure qualified persons operate cranes or other lifting equipment. Contractor to ensure that the site is adequately supervised to minimise the risk to the workers. Automated mechanical methodology to be employed where practical to avoid working at height by personnel. Residual work at height/manual handling and ergonomics to be considered at detailed design. Consider temporary safety fencing and vehicle blocks to prevent construction workers and vehicles falling down.

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Item No.	Activity or Element Reference	Design item giving rise to hazard What decisions are being made or what is being designed which is creating a hazard	Consequence of item giving rise to risk What are the probable consequences of this design item	Persons at Risk Site	 Others	Design action to eliminate risk or reduce risk The measures to be taken such as alternatives to be designed, information to be transferred onto drawings or in other documentation	Residual Risks Information about the risks that cannot be designed out and require controls to be developed and implemented by others
A02	Construction	Construction of carriageway/footway/retaining structures/parapets/fencing/curb/piers/foundations	Drowning Plant slippage	**X**		Where practical, cofferdams were proposed to create a safe working area.	Ensure safe site area with adequate life saving equipment and provision of safety boat during works on or near water edge. Consider proximity to water in design and provide suitable welfare facilities and adequate PPE on site.
A03	Construction	Unknown Services, Utilities and Obstruction including Existing underground/overhead servicies	Damage to services (e.g. HV cables) Risk to injury/death by electrocution/explosion	**X**	**X**	Services enquiries will be undertaken and all known service locations will be shown on the drawings. The design will avoid disruption (e.g. clashes and diversion works) to services as much practicable as possible. Drawings will clearly highlight where services may be affected by works.	Services to be traced prior to dig and thus identified on drawings supplied. Contractor to arrange diversions as required. Works to be completed by appropiate personnel. Radio and electrodetection of servicies should be undertaken on site everytime an excavation is to take place
A04	Construction	Working alongside and adjacent to a live carriageway	Risk of injury or death from road users colliding with site workers and machinery. Potential for collision/conflict with local traffic due to road closure or traffic diversions.	**X**	**X**	N/A	Contractor to develop and adopt methodology for safe working on/near live carriageways.

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Stage / Section of works:				**Issue date/rev: 30/11/2018**		**Project Director Approval:**	
Item No.	Activity or Element Reference	Design item giving rise to hazard What decisions are being made or what is being designed which is creating a hazard	Consequence of item giving rise to risk What are the probable consequences of this design item	Persons at Risk Site	Persons at Risk Others	Design action to eliminate risk or reduce risk The measures to be taken such as alternatives to be designed, information to be transferred onto drawings or in other documentation	Residual Risks Information about the risks that cannot be designed out and require controls to be developed and implemented by others
A05	Construction	Hazardous materials/substance (e.g. asbestos and paint vapours)	Release of fibers into air, paint solvent vapours and general exposure causing prolonged injury to contractors workforce and general public.	**X**	**X**	Painting the inside of the box girder was designed out by specifying weathering steel with a sacrificial steel thickness allowance.	Contractor to conduct Hazardous Materials Survey and Asbestos Survey for the site area by certfied inspectors. Contractor to ensure that there is appropriate ventilation and netting prior to painting.
A06	Construction	Contaminated ground	Exposure of hazardous materials/soils to workforce and general public. Release of contaminants/spread of pollution; removal and disposal if contaminated material/soil	**X**	**X**	All known contaminated land areas will be shown on the drawings.	Principal Designer/Contractor to ensure that they are satisfied with the accuracy of the Survey/Reports prior to commencement of any site works/activities. Ensure suitable surveys/desktop study is carried out where necessary to locate areas of contamination.

CDM DESIGNER'S RISK ASSESSMENT

Project Name: Swing Bridge Replacement **Job No: 1620003502** **Document Ref:**

Stage / Section of works: **Issue date/rev: 30/11/2018** **Project Director Approval:**

Item No.	Activity or Element Reference	Design item giving rise to hazard What decisions are being made or what is being designed which is creating a hazard	Consequence of item giving rise to risk What are the probable consequences of this design item	Persons at Risk: Site	Persons at Risk: Others	Design action to eliminate risk or reduce risk The measures to be taken such as alternatives to be designed, information to be transferred onto drawings or in other documentation	Residual Risks Information about the risks that cannot be designed out and require controls to be developed and implemented by others
A07	Construction	Deep excavations and groundworks	Collapse of excavation/trench resulting in injury or death. Flooding of excavations - leading to drowning and loss of stability.	X		Minimise depth of excavation where possible using piles and pilecaps. Where practical, cofferdams were proposed to create a safe working area.	Contractor to implement suitable measures to ensure that the required excavation slope for stability is achieved. Contractor to ensure that excavation works do not undermine nearby structures. Contractor to ensure that they have the required geotechnical information prior to excavation works. Contractor to protect the edges of the excavation pit with substantial barriers to prevent falling injuries.
A08	Construction	Excessive noise	Possible hearing impairments to workers from the pile driving noise.	X	X	N/A	Contractor to implement suitable measures and to drive piles during non-peak hours and to ensure that all workers involved with the pile driving wear the required PPE for noise protection. Contractor to control/limit the amount of workers in the construction zone.

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Stage / Section of works:				**Issue date/rev: 30/11/2018**		**Project Director Approval:**	
		Design item giving rise to hazard	Consequence of item giving rise to risk	Persons at Risk		Design action to eliminate risk or reduce risk	Residual Risks
Item No.	Activity or Element Reference	What decisions are being made or what is being designed which is creating a hazard	What are the probable consequences of this design item	Site	Others	The measures to be taken such as alternatives to be designed, information to be transferred onto drawings or in other documentation	Information about the risks that cannot be designed out and require controls to be developed and implemented by others
A09	Construction	Drainage - Pollution spillage along proposed bridge and carriageway during construction works.	Spillages will cause pollution of waterbody within the area	**X**	**X**	N/A	Suitable maintenance regime should be in place to ensure blockages and spillages are cleared. Suitable drainage will be required to be provided throughout the site, containment will be suggested during the detail design following a pollution prevention risk assessment recommendations
A10	Demolition	Demoltion of existing Bridge deck, foundations and associated highway elements/structural items	Risk of collapse causing damage, injury or death from falling objects. Risk of asbestos	**X**	**X**	N/A	Ensure that demolition is carried out by competant demolition team. Principal Designer/Contractor to ensure Asbestos Survey is carried out prior to commencement of any site works/activities.
B	**Bridges**						
B01	Construction	Construction sequence	Changes to the defined construction sequence resulting in a possible structural collapse	**X**		Provide construction sequence with the construction drawings	The contractor is to agree any proposed changes to the construction sequence with the designer prior to undertaking the works.

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Item No.	Activity or Element Reference	Design item giving rise to hazard What decisions are being made or what is being designed which is creating a hazard	Consequence of item giving rise to risk What are the probable consequences of this design item	Persons at Risk Site	 Others	Design action to eliminate risk or reduce risk The measures to be taken such as alternatives to be designed, information to be transferred onto drawings or in other documentation	Residual Risks Information about the risks that cannot be designed out and require controls to be developed and implemented by others
B02	Construction	Lifting operations	Crane instability due to excessive section loads. Muscular skeletal injuries arising due to manual handling of heavy components. Failure of structural element during lift. Crashing into existing bridge. Crushing of body parts/equipment between the bridge soffit and bearings.	X	X	The steel box girders can be lifted as single span units. The lifting points will be identified in the detailed design stage. New Swing Bridge Replacement offset 5m from the existing Swing Bridge.	Contractor is to ensure the stability and integrity of the steel box girders at all times. Temporary restraint to beam ends might be required to ensure their stability until the integral connection with the piers is cast and has reached the required design strength. Contractor to ensure lifting equipment with a suitable load capacity is used and to ensure that workers wear the required PPE and are trained in lifting/maneuvering heavy objects.
B03	Maintenance	Pivot bearing/pin inspections inside bascule pier chamber	Inspector drowning due to water inundation	X		Solid portion of cylinder shroud raised to +4.0m, to mitigate the risk of inundation fo the chamber during storm surge excluding sea level rise.	Inspector to assess the external water levels on the inspection day, along with the water levels within the chamber prior to conducting the inspection.

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Stage / Section of works:	**Issue date/rev: 30/11/2018**	**Project Director Approval:**

Item No.	Activity or Element Reference	Design item giving rise to hazard What decisions are being made or what is being designed which is creating a hazard	Consequence of item giving rise to risk What are the probable consequences of this design item	Persons at Risk Site	Persons at Risk Others	Design action to eliminate risk or reduce risk The measures to be taken such as alternatives to be designed, information to be transferred onto drawings or in other documentation	Residual Risks Information about the risks that cannot be designed out and require controls to be developed and implemented by others
B04	Maintenance	Pivot bearing/pin inspections in bascule pier chamber	Inspector getting injured at the bottom of the chamber pit and unable to get out on their own	X		Access platform designed to be large enough for emergency personnel and equipment to maneuver. Vertical access stairs into the chamber allow a winch to be attached to the injured person for the medical professional to pull up. Ladder access up to the lift pier platform by boat was included as an alternative option. All health and safety spatial/access requirements have been incorporated into the design.	Inspector to inspect chamber with a team of certified inspectors.
B05	Maintenance	Pivot bearing/pin inspections in bascule pier chamber	Lack of oxygen/mould	X		The cylinder shroud above +4.0m is designed to be mesh/grille to allow natural ventilation of the cylinder pit.	Inspectors to check internal air quality prior to conducting inspection.
B06	Maintenance	Access onto the bascule pier	Inspectors getting hit by moving traffic as they attempt to gain access down onto the bascule pier	X	X	Access down onto the bascule pier was designed to be from the footway to mitigate the risk of the inspector coming into contact with moving traffic. An alternative access route onto the bascule pier from boat was also included.	Appropriate maintenance methodology to be developed by the highways authority, with suitable traffic management if required and appropriately trained operatives. Potential requirement for life saving equipment and provision of safety boat during works on or near parapets and water edge.

CDM DESIGNER'S RISK ASSESSMENT

Project Name: Swing Bridge Replacement | **Job No: 1620003502** | **Document Ref:**

Stage / Section of works: | **Issue date/rev: 30/11/2018** | **Project Director Approval:**

Item No.	Activity or Element Reference	Design item giving rise to hazard What decisions are being made or what is being designed which is creating a hazard	Consequence of item giving rise to risk What are the probable consequences of this design item	Persons at Risk Site	Persons at Risk Others	Design action to eliminate risk or reduce risk The measures to be taken such as alternatives to be designed, information to be transferred onto drawings or in other documentation	Residual Risks Information about the risks that cannot be designed out and require controls to be developed and implemented by others
B07	Maintenance	Inspection of the inside of the steel box girder.	Confined space leading to excessive heat, oxygen deficiency and lack of mobility.	X		Two access points on the lift span were included - one from the main pier and one from the nose pier to allow access and egress, as well as ventilation. The depth of the deck was increased to reduce issues with confinement and mobility.	Inspectors to be trained in the inspection of confined spaces and to wear the appropriate PPE.
C	**Geotechnical**						
C01	Construction	Unexploded Ordnance UXO	Explosion, contamination	X	X		Contractor to use the mitigation measures developed in conjunction with UXO consultant
D	**M&E**						
D01	Operation	Hydraulic leaks - environmental risks	Destroy the local fauna and flora		x	Ensure plant room is bunded to contain any spills/leaks. Pipe runs to in trenches. Cylinder pit to be bunded. Majority of oil to be stored over land to ease access for removal and replacement.	Risk of leakage remains due to pipe run from plant room to pier. Check for leakage needs to be added to visual checks carried out during periodic inspections
D02	Operation	Hydraulic leaks in the plant room - slip and trip	Injury/death risk for the operator/maintainer	x		Consider adding mesh floor in plant room	Operator/maintainer needs to be aware of potential for oil leakage
D03	Operation	Risk to public during operation - public fall into water or are trapped by machinery	Injury/death risk for the public		x	Install barriers, warning lights and sounders	Operator needs to check bridge is clear of public before operation. Needs to be able to see all hazards/ trap points during operation. Will require support staff if CCTV not provided

CDM DESIGNER'S RISK ASSESSMENT

Project Name: Swing Bridge Replacement | **Job No: 1620003502** | **Document Ref:**

Stage / Section of works: | **Issue date/rev: 30/11/2018** | **Project Director Approval:**

Item No.	Activity or Element Reference	Design item giving rise to hazard What decisions are being made or what is being designed which is creating a hazard	Consequence of item giving rise to risk What are the probable consequences of this design item	Persons at Risk Site	Others	Design action to eliminate risk or reduce risk The measures to be taken such as alternatives to be designed, information to be transferred onto drawings or in other documentation	Residual Risks Information about the risks that cannot be designed out and require controls to be developed and implemented by others
D04	Operation	Risk to traffic during operation - vehicle drives into water	Injury/death risk for the public		x	Install barriers, wig wags and sounders	Operator needs to check bridge is clear of public before operation. Needs to be able to see all hazards/ trap points during operation. Will require support staff if CCTV not provided
D05	Operation	Risk to marine traffic - ship impacts against bridge whilst in closed position	Injury/death risk for the public		x	Navigation obstruction lights. Warning lights	Install obstruction marks on side of bridge. Guidance to be given to boat operators to be aware of bridge. Suggest implementing speed limit in vicinity of bridge
D06	Operation	Risk to marine traffic - ships travelling in different directions collide with each other. Or ship doesn't wait until bridge is open to navigate past	Injury/death risk for the public		x	Navigation obstruction lights. Warning lights	Consider further installation of navigation lights (red/green) either side of bridge. Guidance to be given to boat operators to be aware of bridge. Suggest implementing speed limit in vicinity of bridge
D07	Operation	Cylinder failure	Catastrophic failure to deck if lose support	x	x	Bridge supported by two cylinders. Load holding valves mounted directly to cylinder body. Minimise use of hydraulic hose.	Use of bridge under single cylinder operation should be minimised due to lack of redundancy.
D08	Operation / Maintenance	Operator harmed by the mechanism under the bridge	Injury/death risk for the operator	x		Access walkway needs lockable gate.	Operator required training to highlight all risks and hazards

CDM DESIGNER'S RISK ASSESSMENT

Project Name: Swing Bridge Replacement	**Job No: 1620003502**	**Document Ref:**
Stage / Section of works:	**Issue date/rev: 30/11/2018**	**Project Director Approval:**

Item No.	Activity or Element Reference	Design item giving rise to hazard What decisions are being made or what is being designed which is creating a hazard	Consequence of item giving rise to risk What are the probable consequences of this design item	Persons at Risk: Site	Persons at Risk: Others	Design action to eliminate risk or reduce risk The measures to be taken such as alternatives to be designed, information to be transferred onto drawings or in other documentation	Residual Risks Information about the risks that cannot be designed out and require controls to be developed and implemented by others
D09	Operation / Maintenance	Maintainer harmed by the mechanism under the bridge	Injury/death risk for the maintainer	x		Emergency stop connected to the walkway locking system	Operator required training to highlight all risks and hazards. Safe method of work to be implemented when maintenance staff are in cylinder pit.
D10	Operation	Operator becomes distracted during bridge operations	Injury/death risk	x	x	Push and hold button required to operate the bridge. In this way the operator will not be allowed to go away from the control panel and it will be there in case an emergency stop is required. SEE FURTHER DISCUSSION OF THIS IN STAND ALONE DOCUMENT	Operator required training to highlight all risks and hazards
D11	Operation / Maintenance	Heat build-up from drives/ pumps could result in fire or damage equipment	Injury/death risk	x	x	Ensure drives are sized correctly. Oil temperature to be monitored and interlocked with bridge operation.	Check to be carried out on heat generated during commissioning
D12	Operation / Maintenance	Fire in plant room	Injury/death risk	x	x	Fire detection and alarm to be fitted and wired into main fire panel of the building that houses the electrical cabinets	Operator required training to highlight all risks and hazards
D13	Operation / Maintenance	Fire in control room	Injury/death risk	x	x	Automatic unlock of all the doors in case of fire	Operator required training to highlight all risks and hazards
D14	Operation / Maintenance	Risk of water into the pier	Damaging lifting equipment			Pier walls raised to reduce the risk	Pier not water tight, hence sump pump system required to drain small amount of water anticipated.
D15	Operation	Barrier crushing a person	Injury/death risk	x	x	Pushing mechanism just strong enough to resist normal bridge operation wind speed limit.	Operator needs to check bridge is clear of public before operation

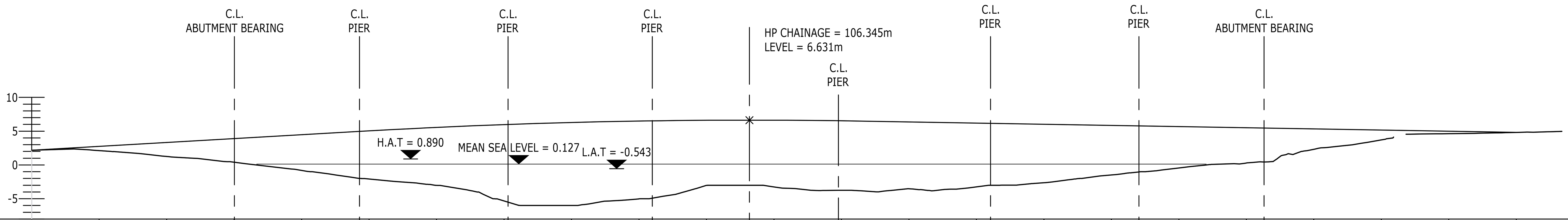
CDM DESIGNER'S RISK ASSESSMENT

Project Name: Swing Bridge Replacement	**Job No: 1620003502**	**Document Ref:**
Stage / Section of works:	**Issue date/rev: 30/11/2018**	**Project Director Approval:**

Item No.	Activity or Element Reference	Design item giving rise to hazard	Consequence of item giving rise to risk	Persons at Risk		Design action to eliminate risk or reduce risk	Residual Risks
		What decisions are being made or what is being designed which is creating a hazard	What are the probable consequences of this design item	Site	Others	The measures to be taken such as alternatives to be designed, information to be transferred onto drawings or in other documentation	Information about the risks that cannot be designed out and require controls to be developed and implemented by others
D16	Operation / Maintenance	Bridge will potentially be operated from the control panel as well as a pendant	Failure of bridge mechanism or injury/ death	**x**		A key will be required to active the control panel and the pendant	Operator required training to highlight all risks and hazards

APPENDIX 4

DRAWINGS



PROPOSED LEVELS TOP OF SURFACING: 2.169, 3.896, 4.962, 5.725, 5.990, 6.631, 6.589, 6.162, 5.781, 5.460

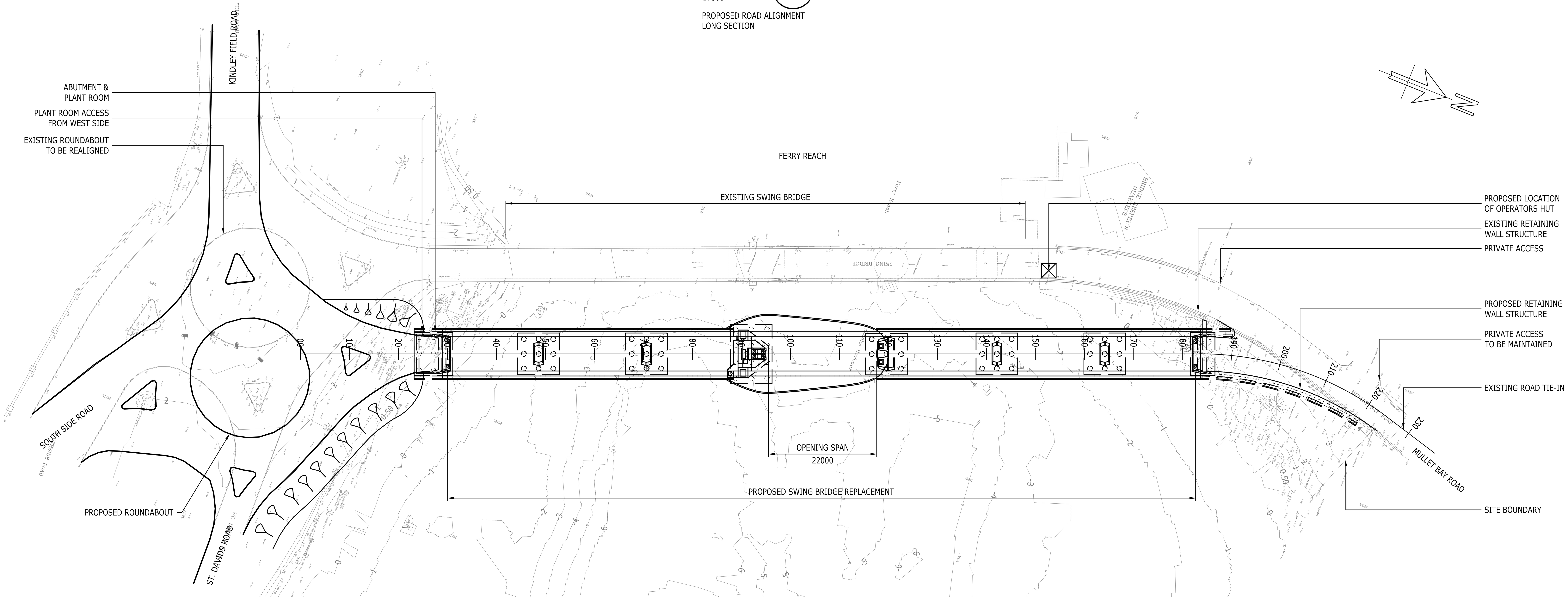
CHAINAGE	EXISTING GROUND & BED LEVELS
00.000	2.169
10.000	2.125
20.000	1.254
30.000	0.410
40.000	-0.808
50.000	-2.079
60.000	-3.012
70.000	-5.326
80.000	-6.001
90.000	-5.022
100.000	-3.021
110.000	-3.249
120.000	-3.758
130.000	-3.528
140.000	-3.247
150.000	-2.628
160.000	-1.487
170.000	-0.439
180.000	0.282
190.000	2.320
200.000	3.707
210.000	
220.000	
230.000	
236.752	

PROPOSED HORIZONTAL GEOMETRY: L = 183.345; R = 63.500, L = 41.222; L = 12.185

PROPOSED VERTICAL GEOMETRY: G = 5.747%, L = 48.969; R = 1000, L = 74.668; G = 1.730%, L = 96.363

SECTION
1:500

PROPOSED ROAD ALIGNMENT LONG SECTION



PLAN
1:500

Notes

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Rev	Description	Date	By / Chk	App
P01	PRELIMINARY	30/11 2018	CAB / JFRW	SPT

PRELIMINARY

REPLACEMENT OF SWING BRIDGE & LONGBIRD BRIDGE, BERMUDA

RAMBOLL
Knight Architects
EADON CONSULTING

SWING BRIDGE REPLACEMENT
APPROVAL IN PRINCIPLE
KEY PLAN
SHEET 1 OF 5

Project No:	Scale (@A1):	Drawn:	Date:
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Drawing No: 3502-RAM-SB-XX-DR-CB-30001 | Rev: P01

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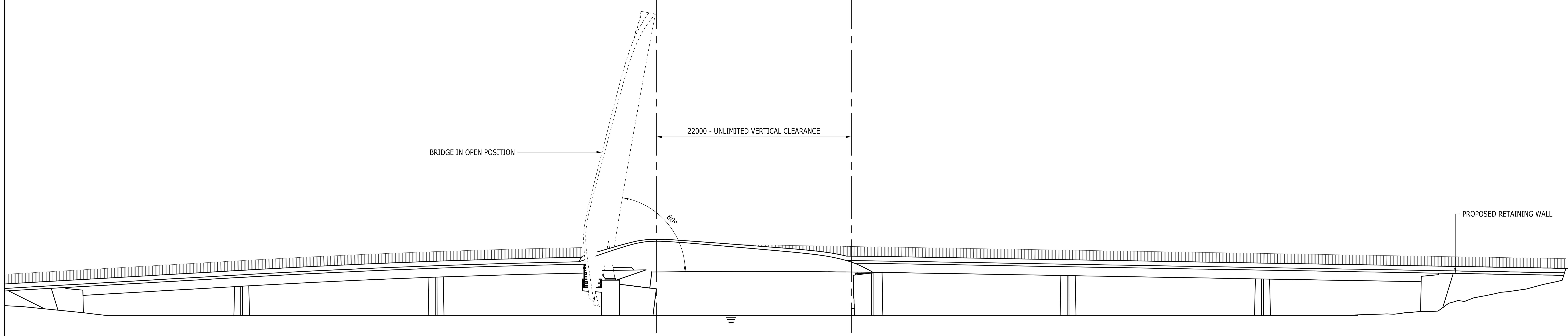

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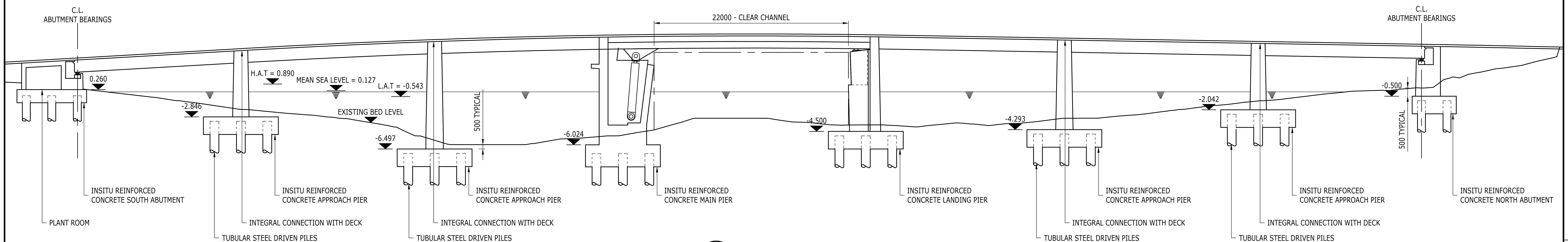
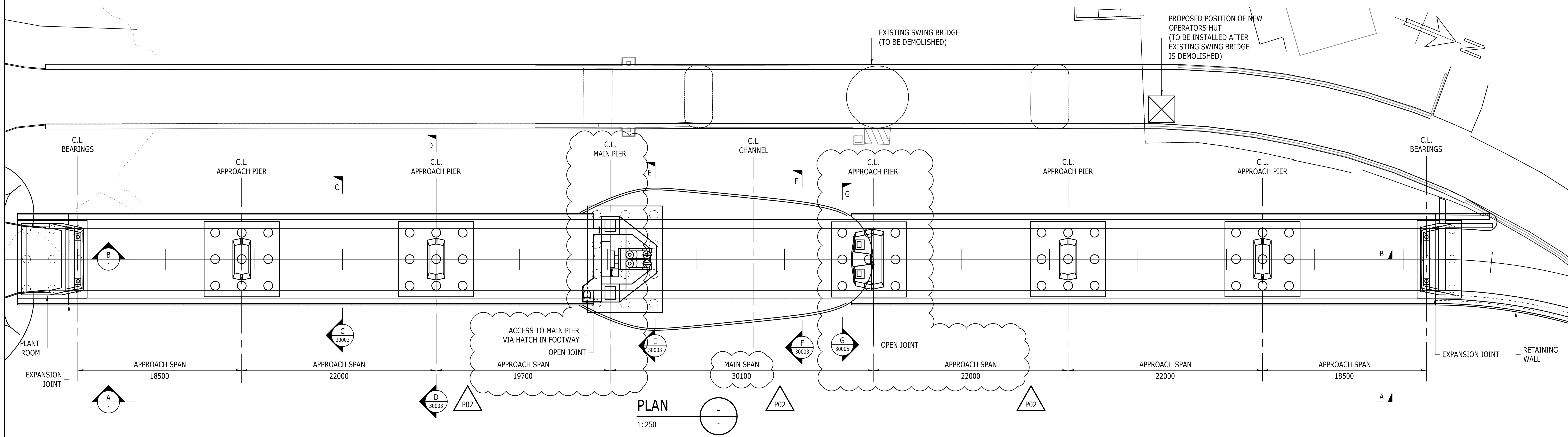
1: 250



SECTION B-B
1: 250



PLAN
1: 250

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P02	REVISIONS CLOUDED	14/02 2019	RZC JFRW	SPT
P01	PRELIMINARY	30/11 2018	CAB JFRW	SPT

PRELIMINARY

REPLACEMENT OF SWING BRIDGE & LONGBIRD BRIDGE, BERMUDA

RAMBOLL

Knight Architects

EADON CONSULTING

SWING BRIDGE REPLACEMENT
APPROVAL IN PRINCIPLE
GENERAL ARRANGEMENT
SHEET 2 OF 5

Project No:	Scale (@A1):	Drawn:	Date:
1620003502	AS SHOWN	CAB	NOV.2018

Drawing No:	Rev:
3502-RAM-SB-XX-DR-CB-30002	P02

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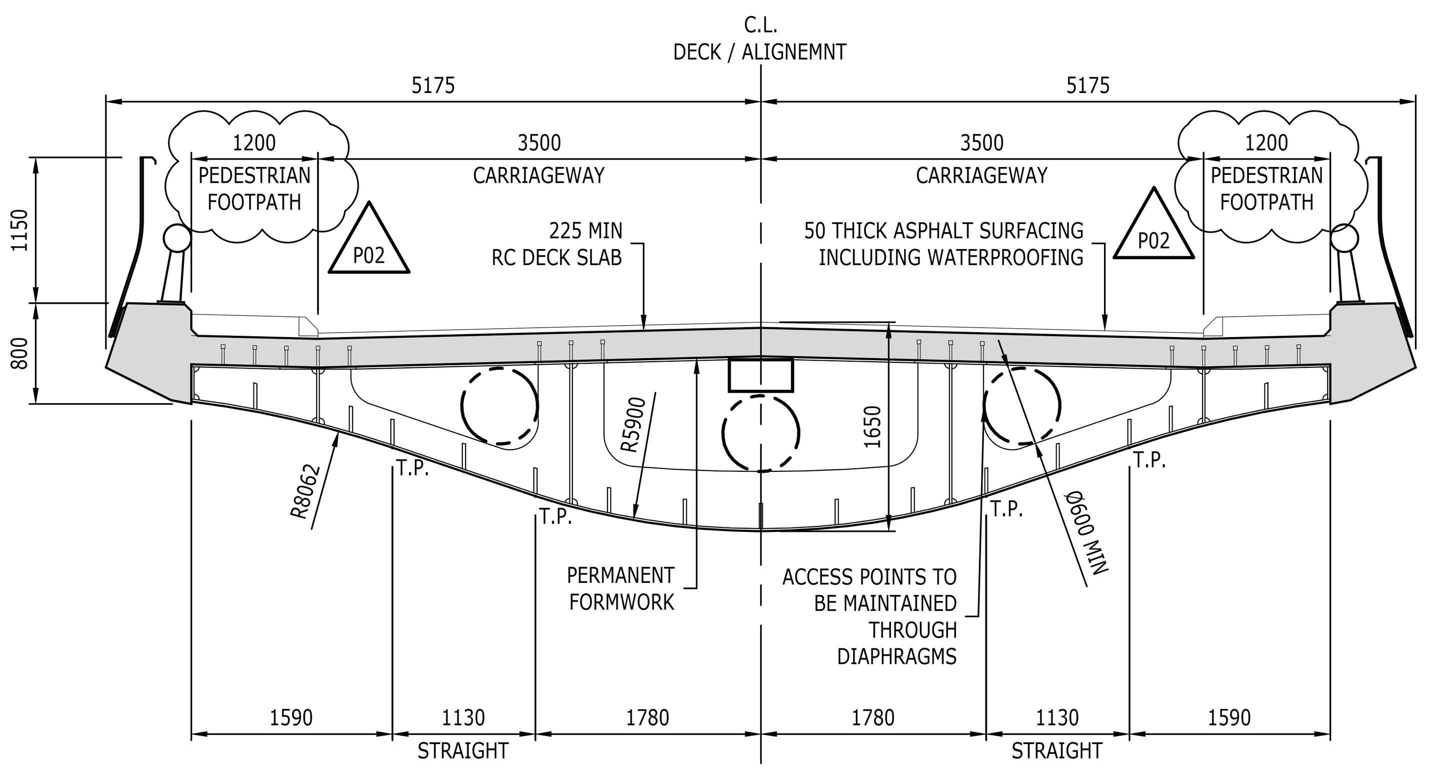

SECTION C-C 30002
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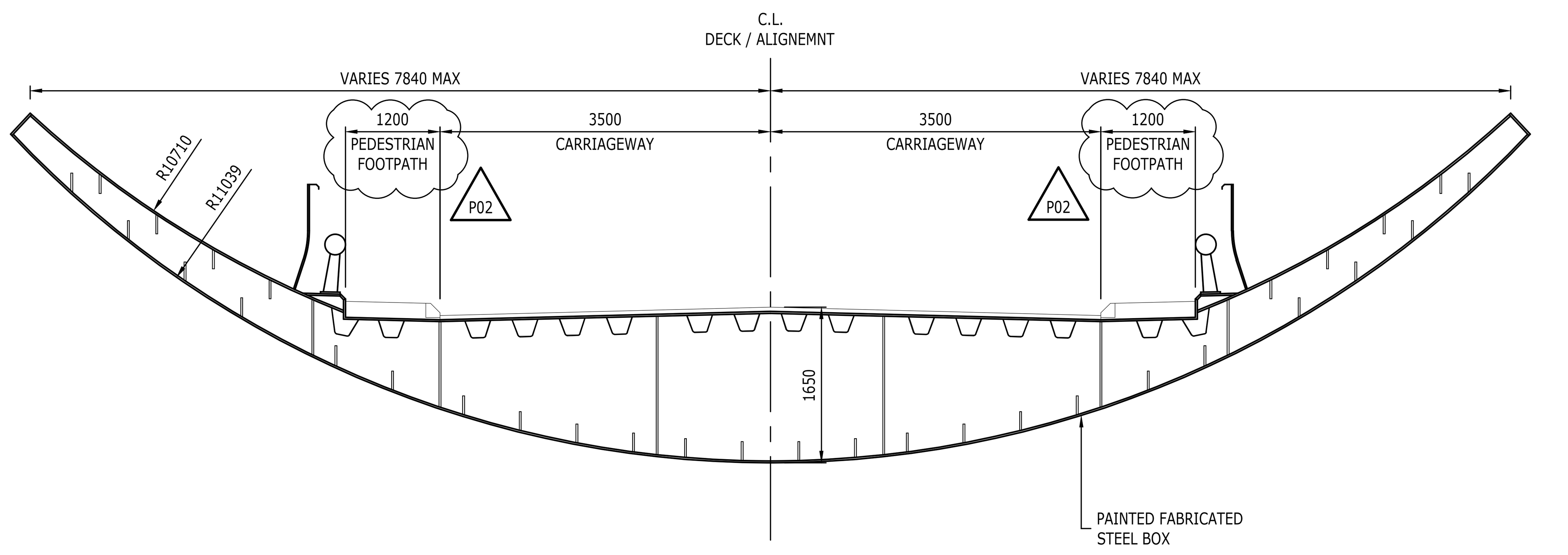

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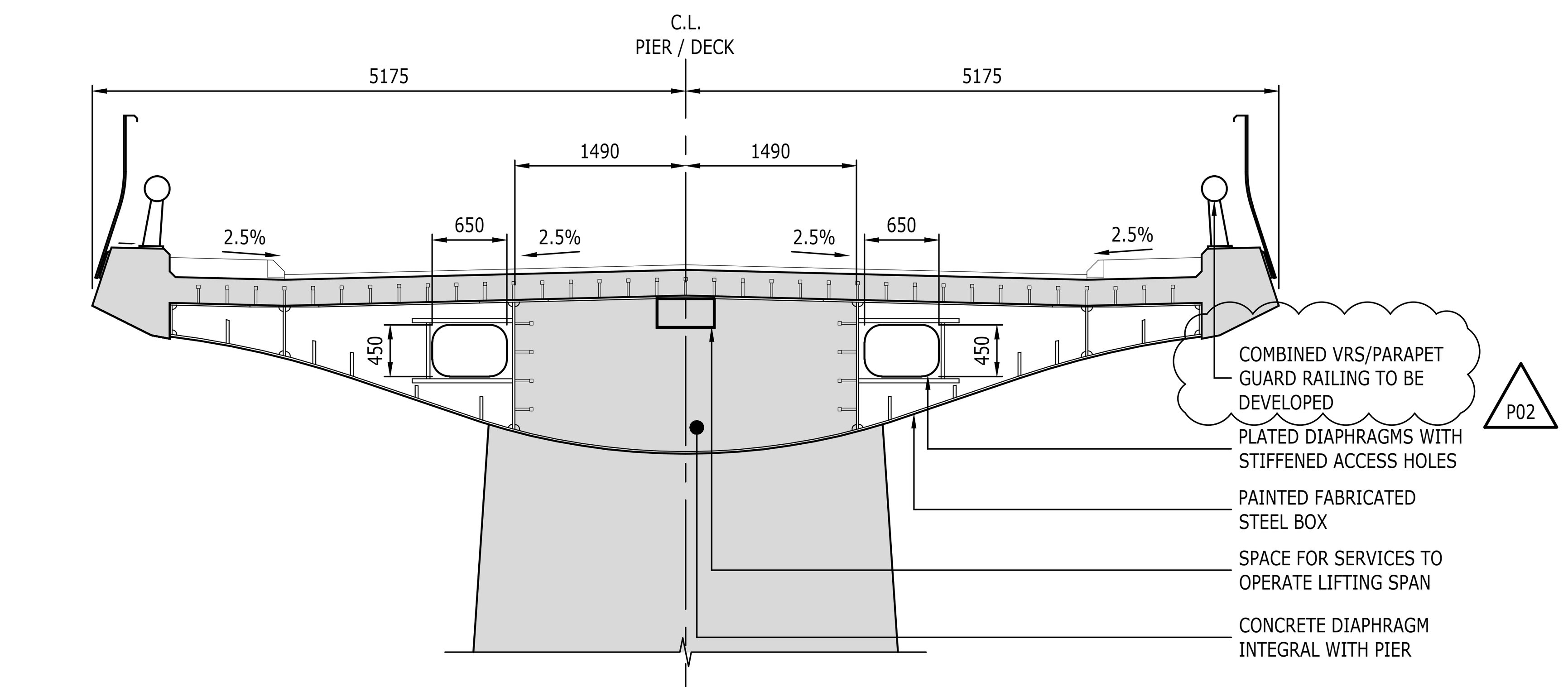

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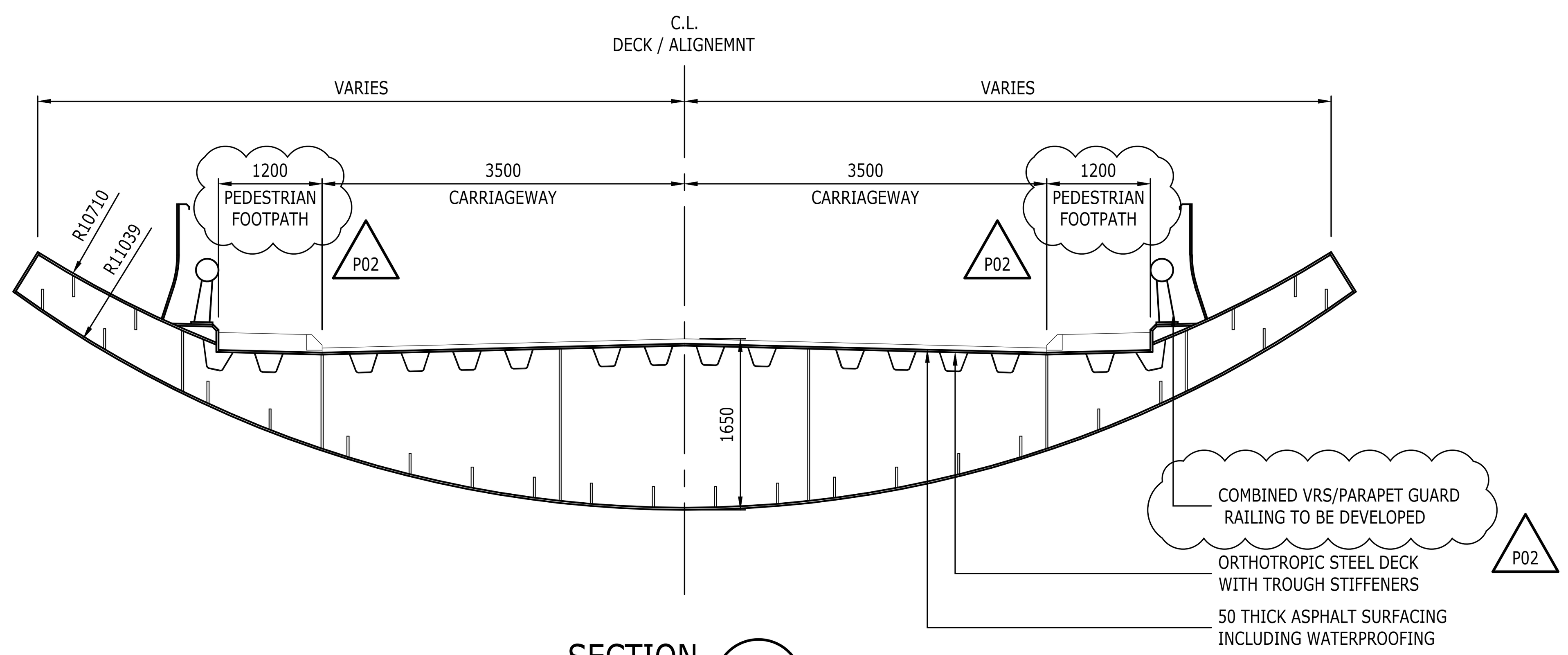

SECTION F-F 30002
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P01	PRELIMINARY	30/11 2018	CAB JFRW	SPT

PRELIMINARY

REPLACEMENT OF SWING BRIDGE & LONGBIRD BRIDGE, BERMUDA

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SWING BRIDGE REPLACEMENT
APPROVAL IN PRINCIPLE
GENERAL ARRANGEMENT
SHEET 3 OF 5

Project No:	Scale (@A1):	Drawn:	Date:
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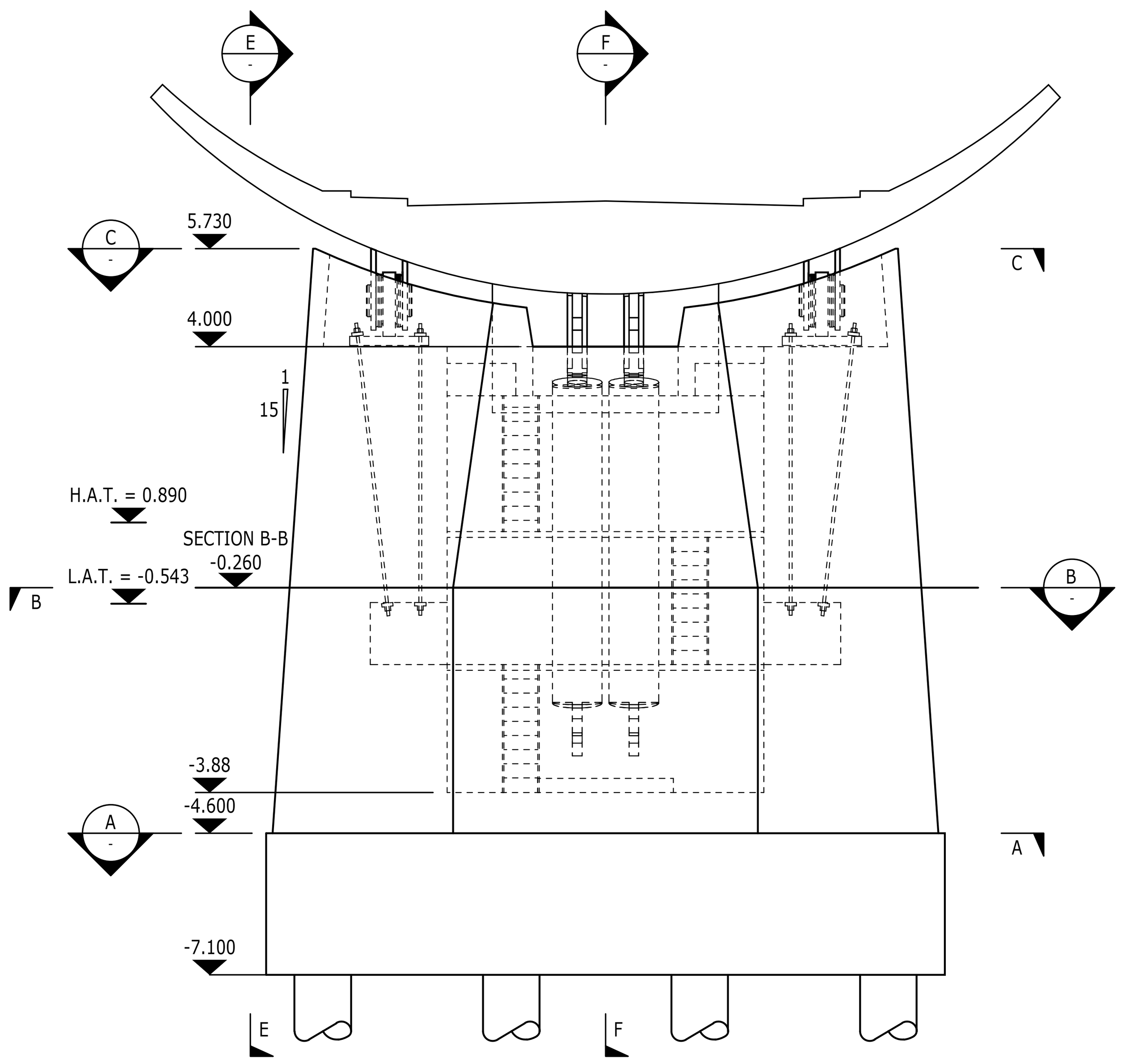

ELEVATION D-D
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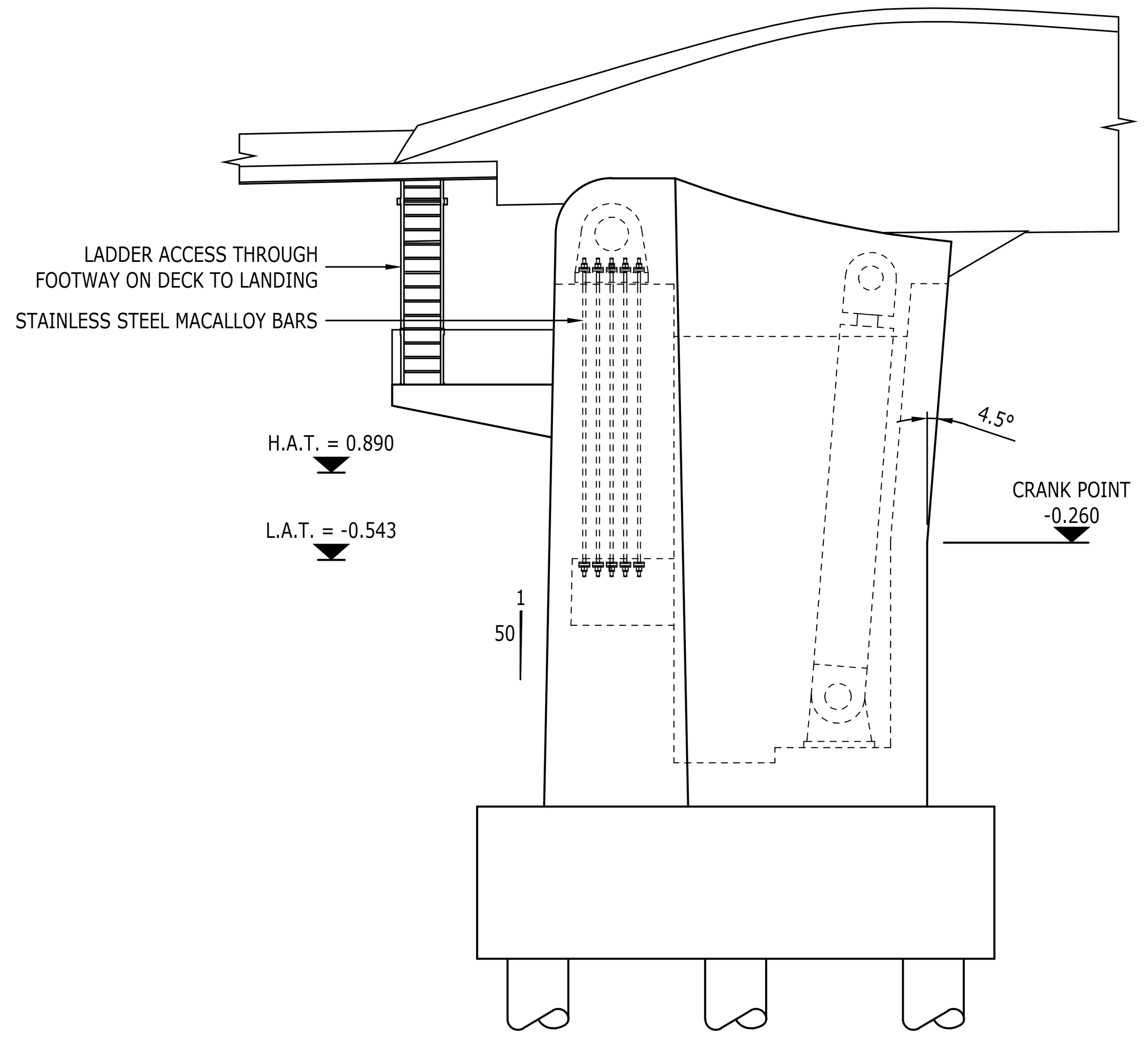

ELEVATION E-E
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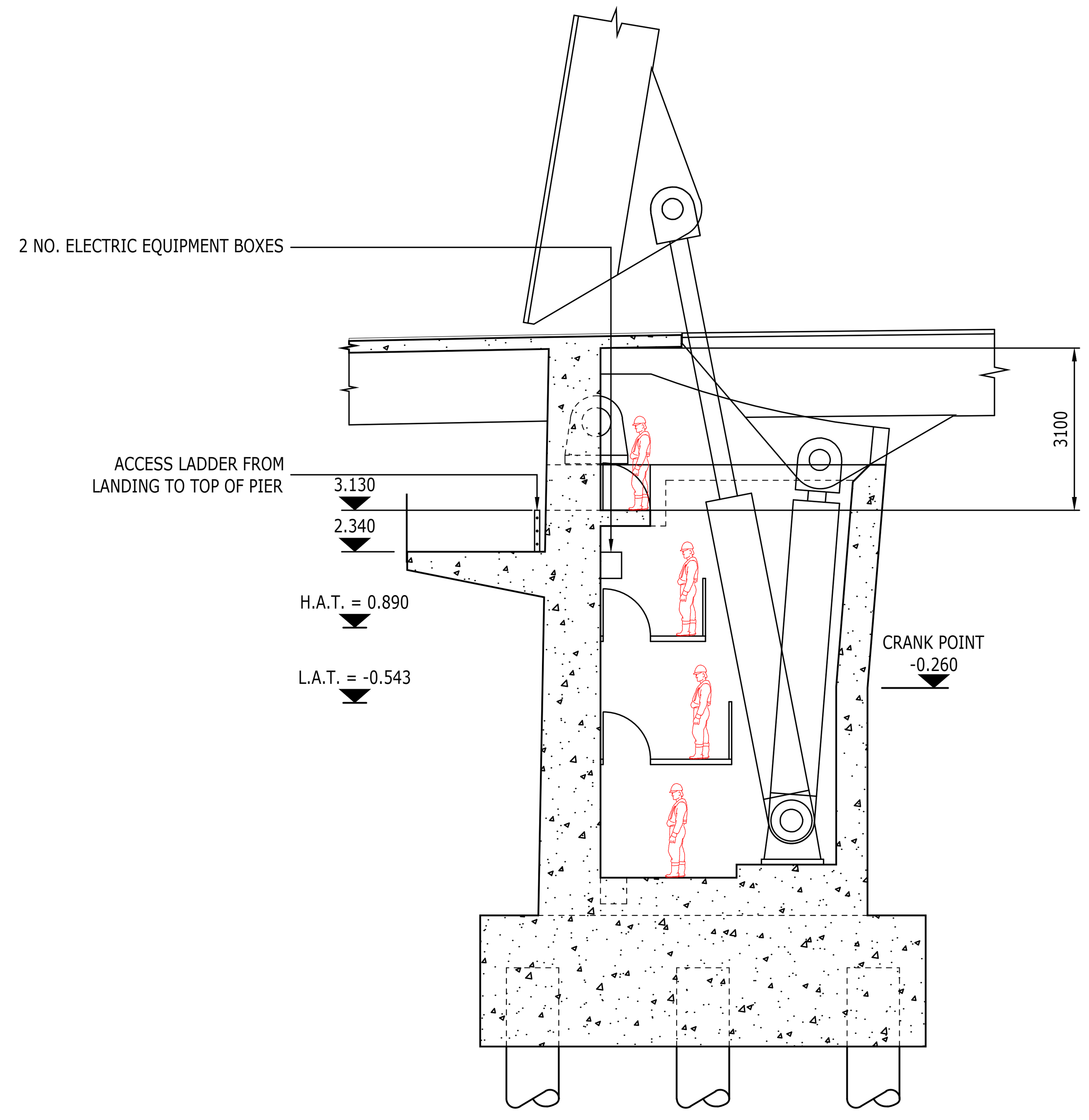

SECTION F-F
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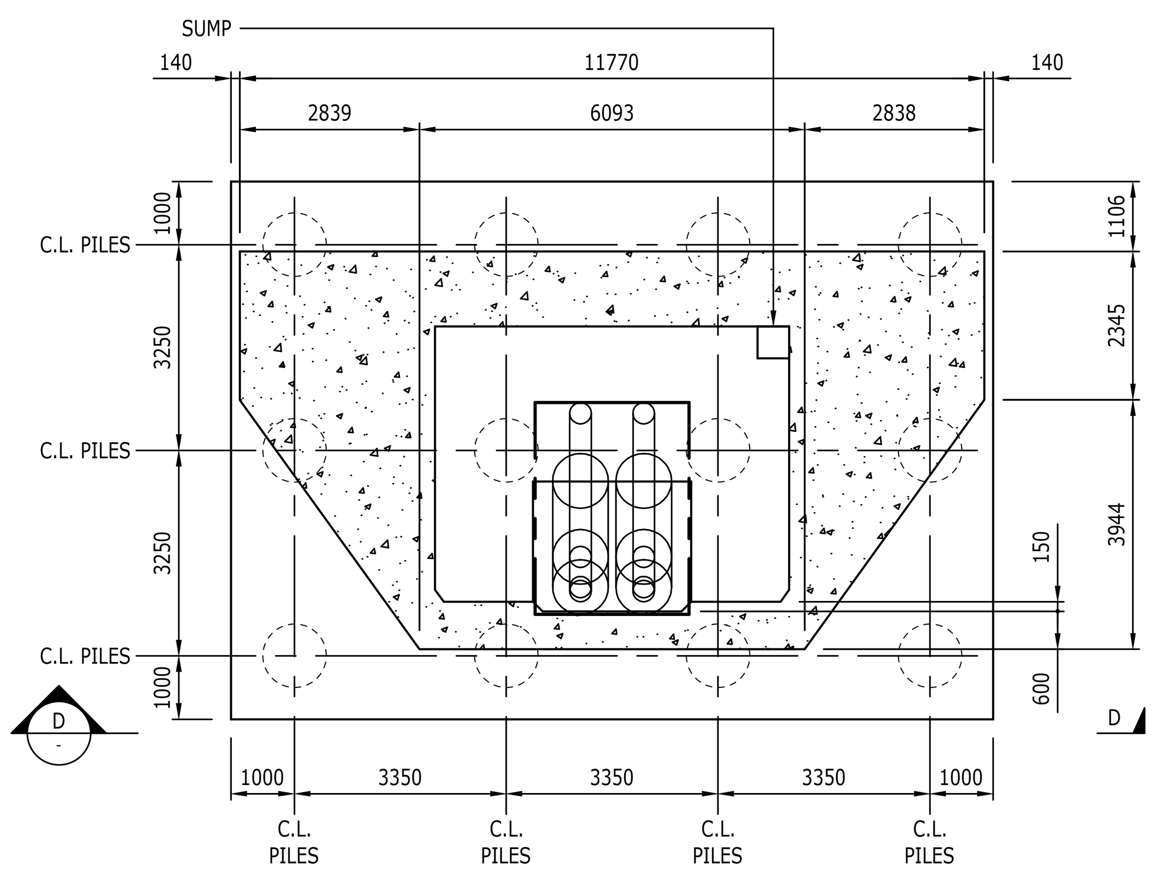

PLAN A-A
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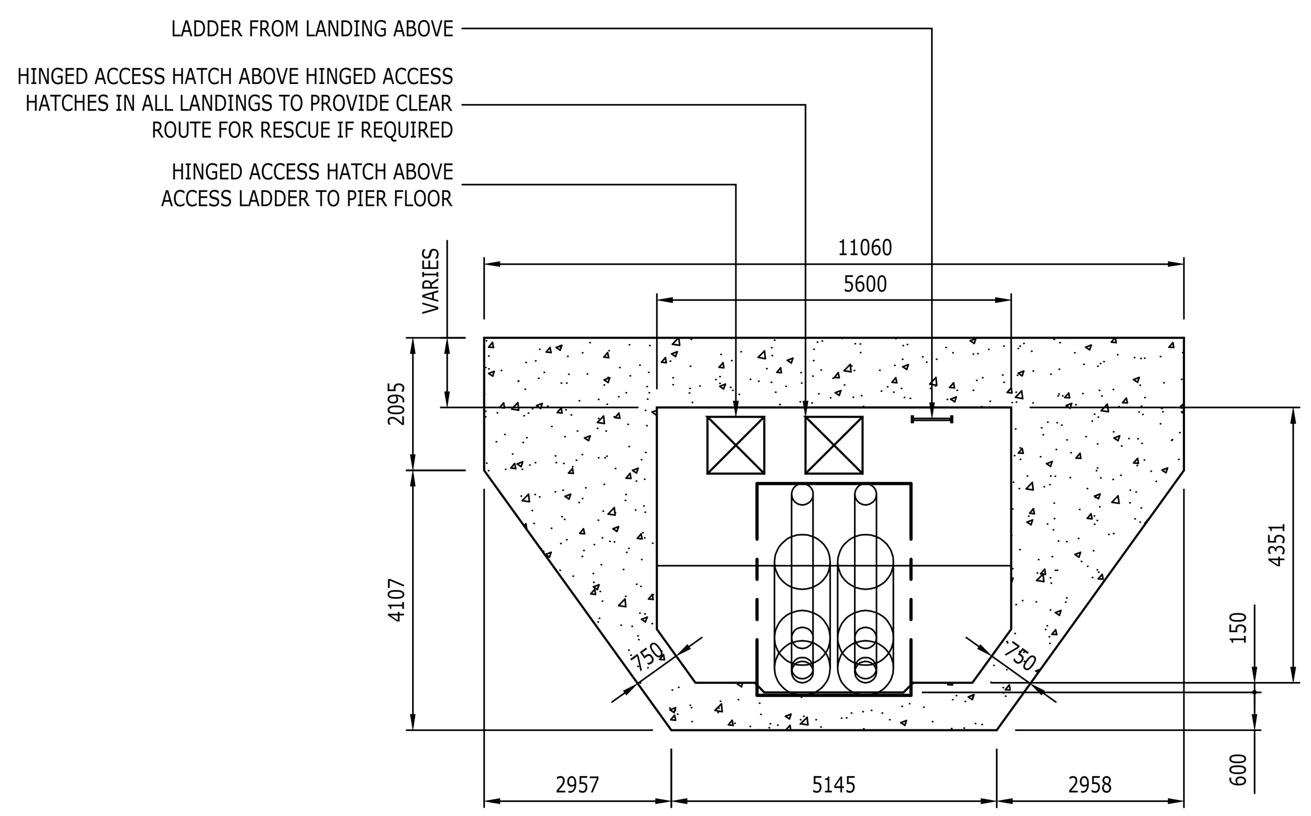

PLAN B-B
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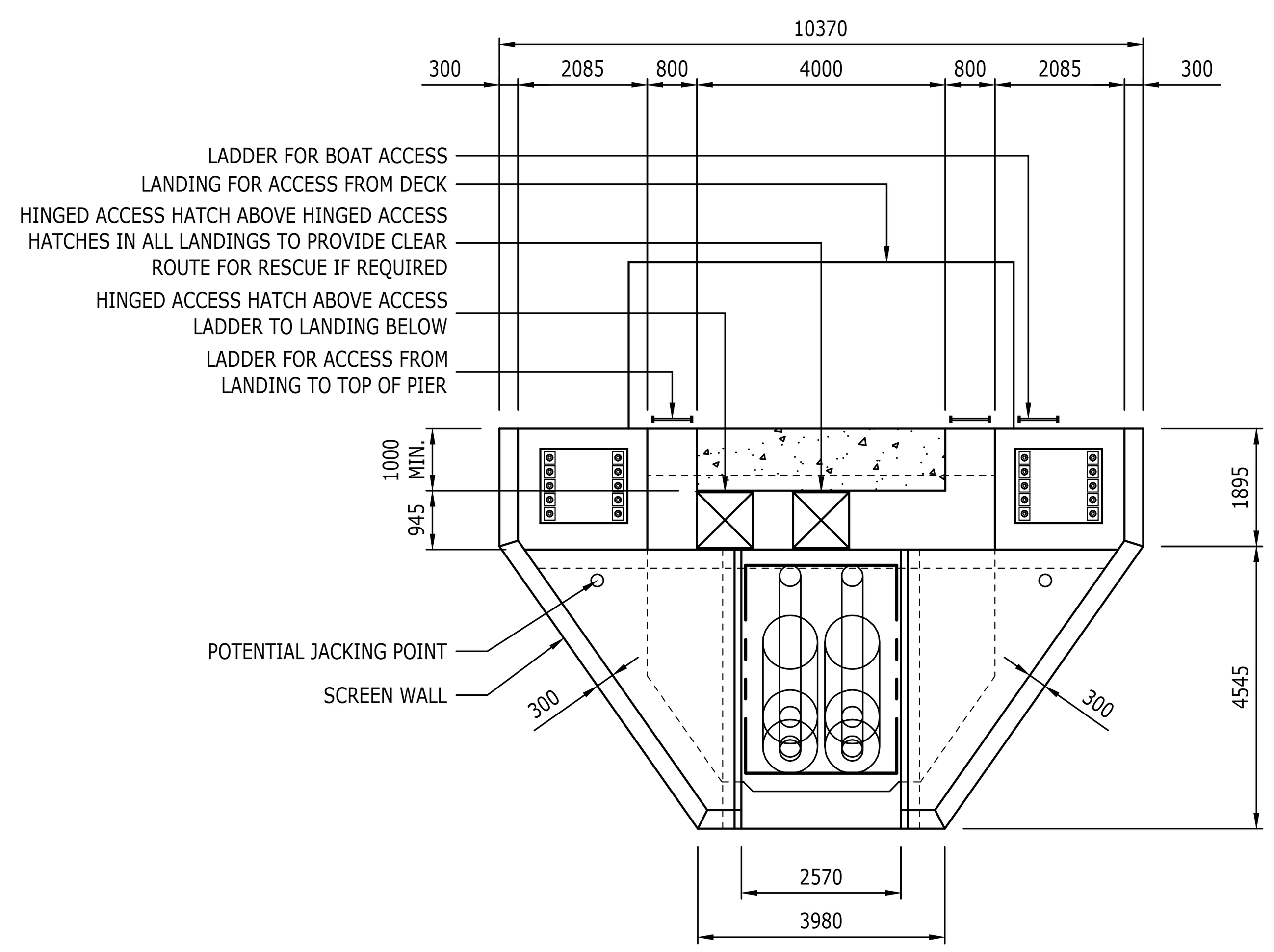

PLAN C-C
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Notes

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PRELIMINARY

REPLACEMENT OF SWING BRIDGE & LONGBIRD BRIDGE, BERMUDA

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SWING BRIDGE REPLACEMENT
APPROVAL IN PRINCIPLE
GENERAL ARRANGEMENT
SHEET 4 OF 5

Project No:	Scale (@A1):	Drawn:	Date:
1620003502	AS SHOWN	CAB	NOV.2018

Drawing No: 3502-RAM-SB-XX-DR-CB-30004
Rev: P01

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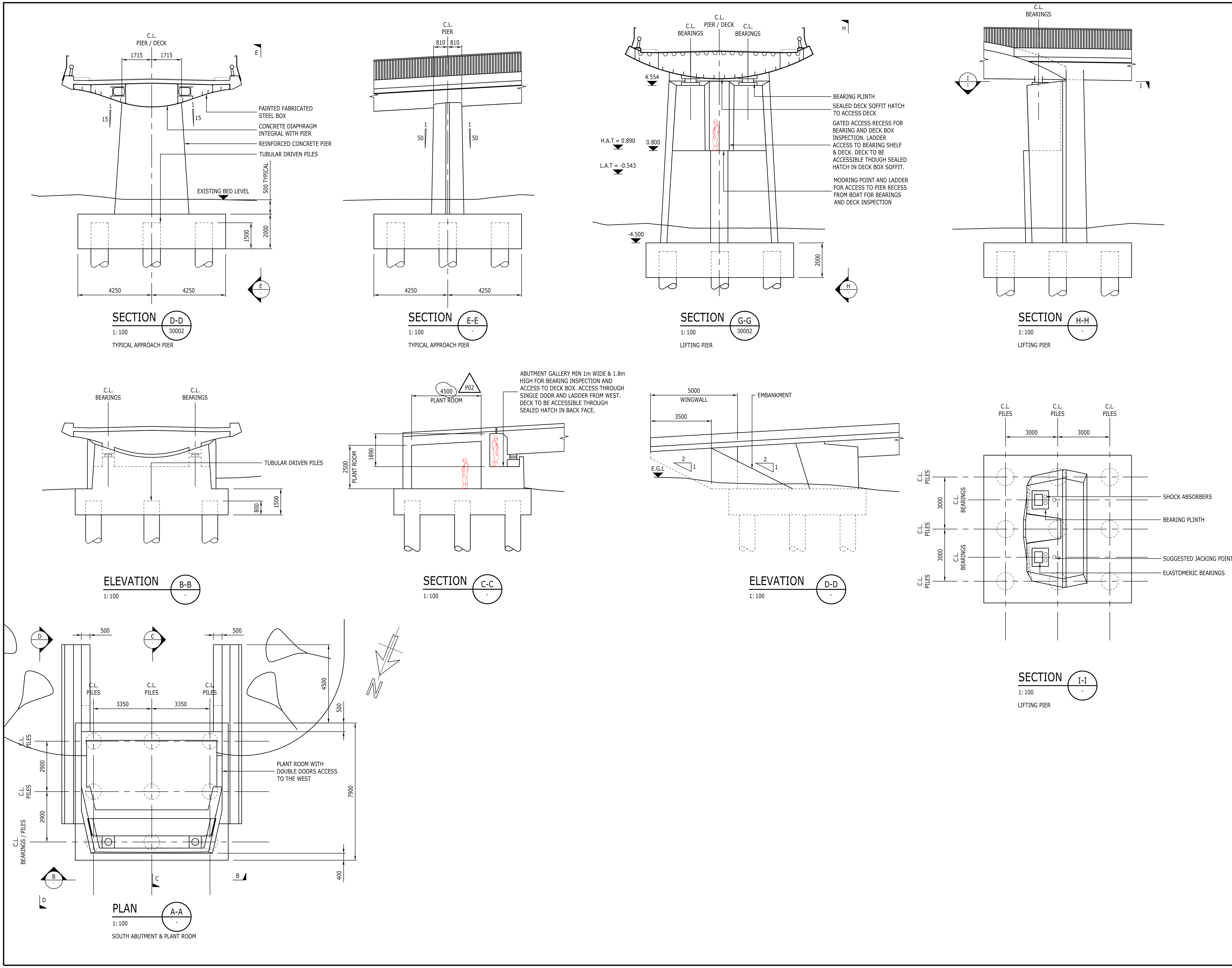

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Rev	Description	Date	By / Chk	App
P02	REVISIONS CLOUDED	14/02 2019	RZC / JFRW	SPT
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PRELIMINARY

REPLACEMENT OF SWING BRIDGE & LONGBIRD BRIDGE, BERMUDA

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SWING BRIDGE REPLACEMENT
APPROVAL IN PRINCIPLE
GENERAL ARRANGEMENT
SHEET 5 OF 5

Project No:	Scale (@A1):	Drawn:	Date:
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Drawing No:	Rev:
3502-RAM-SB-XX-DR-CB-30005	P02

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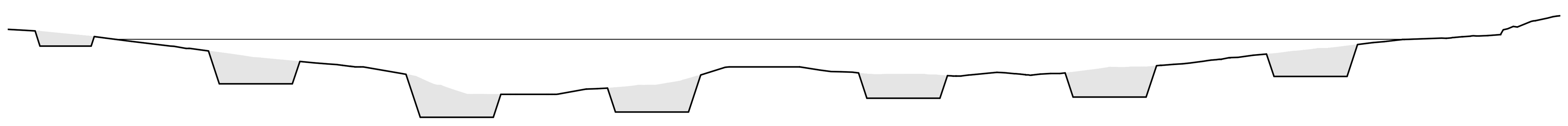
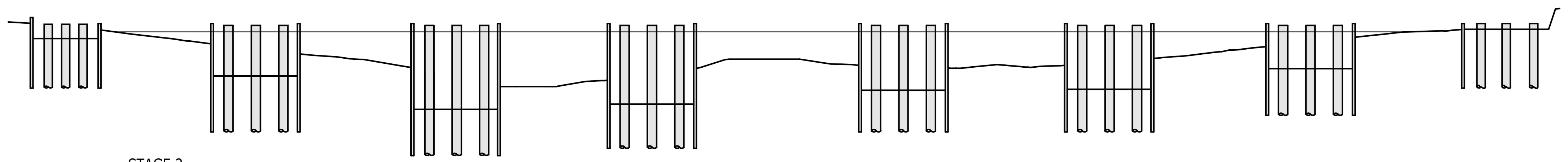
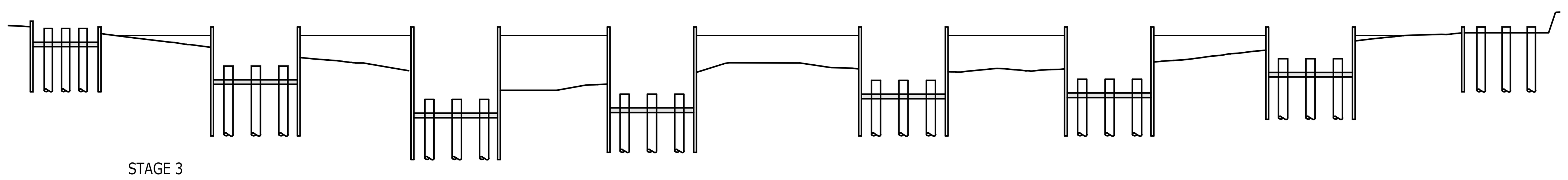


STAGE 1
- SET UP SITE COMPOUND
- ESTABLISH NEW NAVIGATION CHANNEL
- PRE-EXCAVATION FOR PIERS
- EXCAVATE TRENCH TO WEST OF EXISTING SWING BRIDGE AND INSTALL POWER & CONTROL CABLES

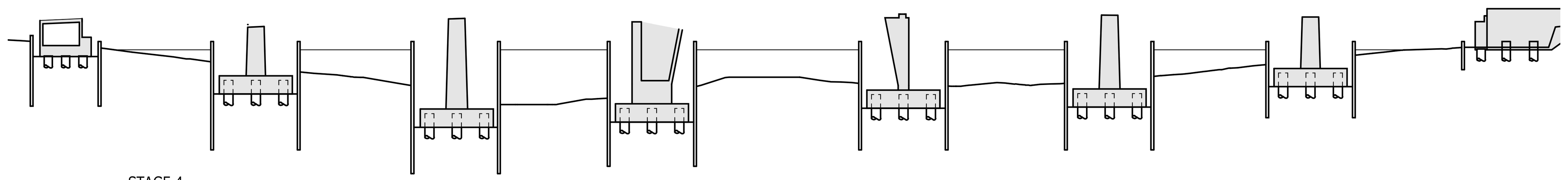
P02



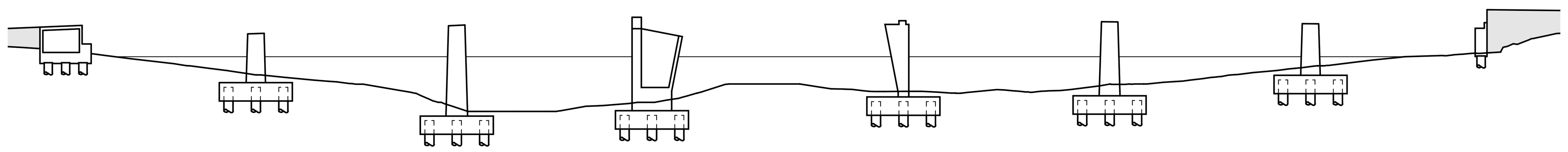
STAGE 2
- DRIVE PERMANENT WORKS PILES TO REQUIRED DEPTH
- CONSTRUCT COFFERDAMS



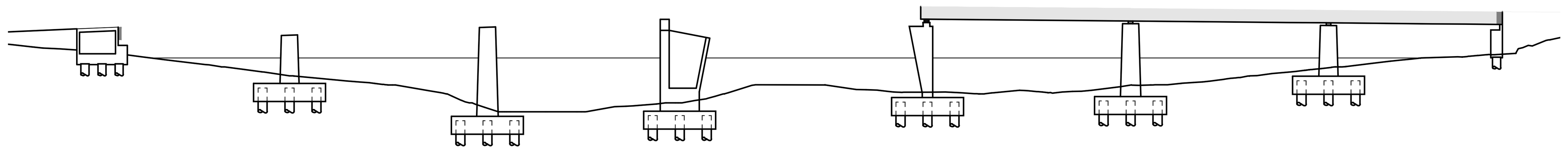
STAGE 3
- EXCAVATE COFFERDAMS
- PLUG COFFERDAM BASES
- DE-WATER COFFERDAM
- CUT PILES TO REQUIRED LEVEL



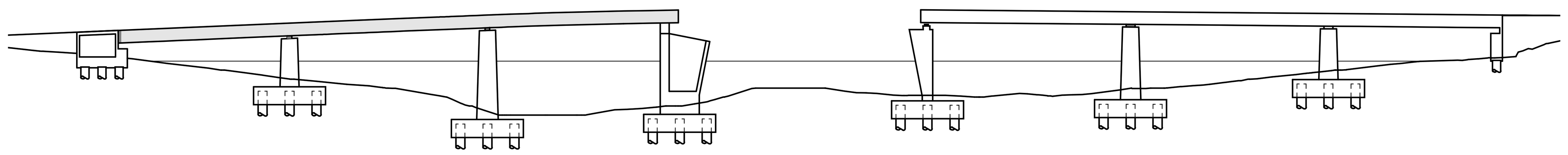
STAGE 4
- CONSTRUCT PILE CAPS
- CONSTRUCT PIERS, ABUTMENTS & RETAINING WALLS



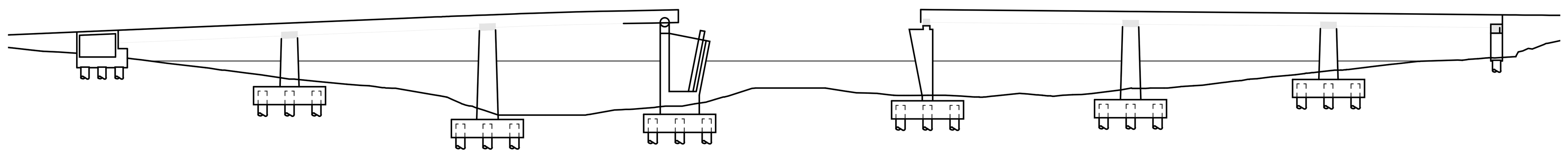
STAGE 5
- REMOVE COFFERDAMS
- REINSTATE SEA BED ABOVE PILECAPS
- INSTALL SCOUR PROTECTION (IF REQUIRED)
- INSTALL M&E EQUIPMENT IN SOUTH ABUTMENT
- BACKFILL ABUTMENTS & NORTH RETAINING WALL



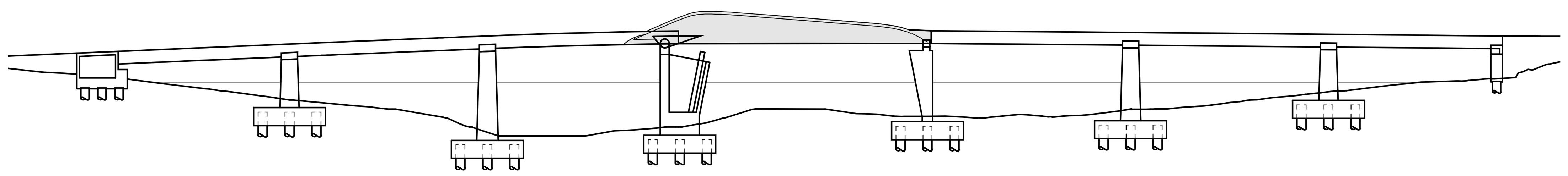
STAGE 6
- INSTALL BEARINGS AT NORTH ABUTMENT
- ERECT NORTH APPROACH STEELWORK SPANS & CONNECT THEM TOGETHER



STAGE 7
- INSTALL BEARINGS AT SOUTH ABUTMENT
- ERECT SOUTH APPROACH STEELWORK SPANS & CONNECT THEM TOGETHER
- INSTALL M&E SERVICES IN SOUTH APPROACH VIADUCT



STAGE 8
- CAST DECK & DIAPHRAGMS AT ALL PIERS
- INSTALL BEARINGS FOR LIFT SPAN AT NOSE PIER
- INSTALL CYLINDERS AT PIVOT PIER



STAGE 9
- ERECT LIFT SPAN AT PIVOT PIER
- CONNECT SUPERSTRUCTURE TO SUBSTRUCTURE AT PIVOT PIER
- COMPLETE INSTALLATION OF M&E EQUIPMENT

STAGE 10
- CAST DECK STRINGCOURSES IN APPROACH SPANS
- INSTALL KERBS, PARAPETS, SURFACING & ALL OTHER FINISHES.
- COMMISSIONING OF LIFT SPAN
- OPEN BRIDGE TO TRAFFIC

STAGE 11
- DEMOLISH EXISTING
- MAKE GOOD EMBANKMENTS
- SCOUR PROTECTION

Notes

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Rev	Description	Date	By / Chk	App
P02	REVISIONS CLOUDED	14/02 2019	RZC JFRW	SPT
P01	PRELIMINARY	30/11 2018	CAB JFRW	SPT

PRELIMINARY

REPLACEMENT OF SWING BRIDGE & LONGBIRD BRIDGE, BERMUDA

RAMBOLL

Knight Architects

EADON CONSULTING

SWING BRIDGE REPLACEMENT
APPROVAL IN PRINCIPLE
CONSTRUCTION SEQUENCE

Project No:	Scale (@A1):	Drawn:	Date:
1620003502	AS SHOWN	CAB	NOV.2018

Drawing No:	Rev:
3502-RAM-SB-XX-DR-CB-30011	P02

15/02/2019 10:48:57

APPENDIX 5

Rev P02

GEOTECHNICAL REPORT - HIGHWAY STRUCTURE SUMMARY INFORMATION

GEOTECHNICAL REPORT
HIGHWAY STRUCTURE SUMMARY INFORMATION

STRUCTURE NAME Swing Bridge Replacement	OS Grid Reference 555412.16m E 141133.99m N	Reference/ comments
STRUCTURE TYPE 7-span bridge with main navigation channel	AIP Ref No 3502-RAM-SB-XX-RP-CB-30001	
DESIGN LIFE 75 Years		
RELEVANT TRIAL HOLES BH201, BH202, BH203, BH204, BH205, BH206, BH207 *(Report: Geotechnical Investigation for Two Bridges in Bermuda Islands: Longbird and St. George's Bridge, Final Report, October 2018)*		
Strata	Typical Thicknesses	

Unit Thickness (m)		Sand and Gravel	Coralline Deposits	Karst Limestone	Clayey Silt Design Layer		Karst Limestone	Silty Clay	Weathered Basalt/Basalt Breccia	Unweathered Basalt/Basalt Breccia
					Clayey Silt	*Sandy Silt*				
Swing Bridge	BH201 (Northern Abutment)	-	2.1	4.7	3.8	-	3.0	11.6	3.1	Extent not proven
	BH202 (Piers 3+4)	1.3	1.3	13.4	-	-	-	4.0	6.5	Extent not proven
	BH203 (Nose Pier)	1.7	-	9.8	-	-	-	5.6	3.6	Extent not proven
	BH204 (Lift Pier)	1.4	-	-	2.3	5.8	-	9.3	3.0	Extent not proven
	BH205 (Pier 2)	1.2	-	-	3.3	-	-	9.5	3.8	Extent not proven
	BH206 (Pier 1)	5.8	-	-	2.7	-	6.0	4.5	1.8	Extent not proven
	BH207 (Southern Abutment)	-	6.6	-	14.7	-	-	-	1.5	Extent not proven

PREVIOUS GROUND HISTORY

The historic ground use adjacent to the site is that of an airport development found on reclaimed land but more locally, an existing bridge structure.

Previous ground investigations have been undertaken on and around the site:

- Geotechnical Investigation for proposed new apron and widening of existing taxiway LF Wade International Airport (2016)
- Preliminary Geotechnical Assessment New Grotto Bay/Castle Harbour Crossing Bermuda (2007)
- St George's Town Cut Project, Geotechnical Data Report (2015)

GROUNDWATER

Groundwater was encountered on the southern (airport side) abutment location in TP203 and TP205 (On shore Test Pits dug to 3.5m)

EARTH PRESSURE VALUE k_0*	Coralline Deposits	Sand and Gravel	Silty Clays	Clayey Silts
	0.48	0.44	0.64	0.64

SOIL PARAMETERS

Stratum	**Bulk Density, γ (kN/m³)**	**Undrained Shear Strength Parameters**		**Drained Shear Strength Parameters**		**UCS (MPa)**	**Hoek Brown**		
		Undrained Shear Strength, c_u (kN/m²)	*Change with depth, z*	**Effective Angle of Shearing Resistance Φ' (°)**	**Drained cohesion (kN/m²)**		**mb**	**a**	**s**
Sand and Gravel	17.0			31	0				
Coralline Deposits	16.0			34	0				
Clayey Silt	19.0	50	0	21	0				
Silty Clay	19.0	50	21.4 kN/m²/m depth	21	0				
Karst Limestone	24.8			48	247	16	1.6	0.50	0.01
Weathered Basalt	21.3			36	181	3.2	3.383	0.51	0.002
Unweathered Basalt	23.0			62	404	30	5.99	0.50	0.01

PILE DESIGN									
Structure Element	Founding Stratum	Founding Rock Head Level (mOAD)	Pile Cap Head Level (mAOD)	Pile Length (m)	Pile Toe Level (m AOD)	Pile Diameter (mm)	Ultimate Bearing Capacity (kN)	Pile Compressive Load (Tensile Load) (kN)	
Northern Abutment	Weathered Basalt	-25.7	-0.50	25.7	-26.2	900	22135	1100 (600)	0.5m rock socket
Pier 4	Weathered Basalt	-22.0	-2.04	20.5	-22.5	900	35102	1900 (500)	0.5m rock socket
Pier 3	Weathered Basalt	-24.3	-4.29	20.5	-24.8	900	35102	2400 (900)	0.5m rock socket
Nose Pier	Weathered Basalt	-21.6	-4.50	18.1	-22.6	900	27327	2400 (900)	1m rock socket
Lift Pier	Basalt	-27.8	-6.02	23.3	-29.3	900	33889	2900 (2200)	1.5m into Basalt
Pier 2	Basalt	-24.3	-6.50	19.3	-25.8	900	35582	2400 (900)	1.5m into Basalt
Pier 1	Weathered Basalt	-21.8	-2.85	20	-22.8	900	18620	1900 (500)	1m rock socket
Southern Abutment	Basalt	-22.5	0.26	23.3	-23.0	900	30135	1100 (600)	0.5m into Basalt
Pile type……………………………………………………………………Steel Tube (Driven) Criteria for selecting pile toe level……………………………………Strength/Stiffness of founding stratum Allowance for negative skin friction within design…………………..None									

SETTLEMENT						
Structural Element	Founding Level (m AOD)	Immediate Settlement (mm)	Total Settle ment (mm)	Time for 90%	Settlement Remaining at Completion	
Not Applicable						
GROUND MOVEMENTS						

Associated Earthworks	Settlement due to Embankment loading	Heave due to Cutting Excavation	Subsidence Due to Mineral Extraction	Flowing Water	Other	
Cause of Movement						
Maximum Movement (mm)	Not Applicable					
Measures to Deal with Movement						

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