Annex G

Swing Bridge

Swing Bridge and Longbird Bridge Replacement

Independent check

2023

Reference Documents

Design Criteria

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 5^{o}C to 34^{o}C.

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.

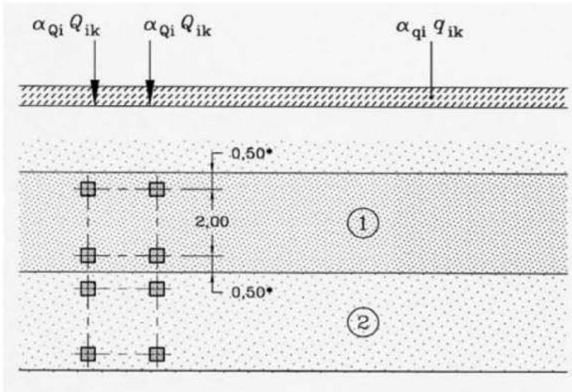
Uniform temperature will be assumed along the entire length of the structure.

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.

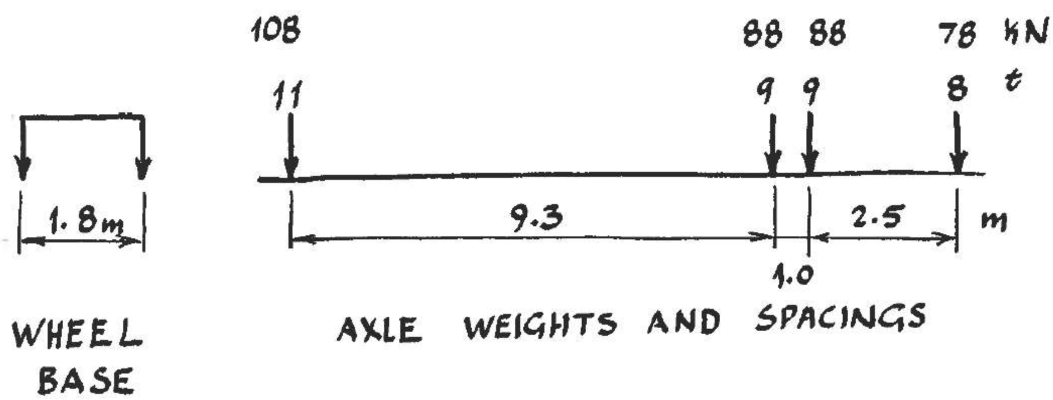


Figure 2 – Proposed Evaluation Truck for Bermuda

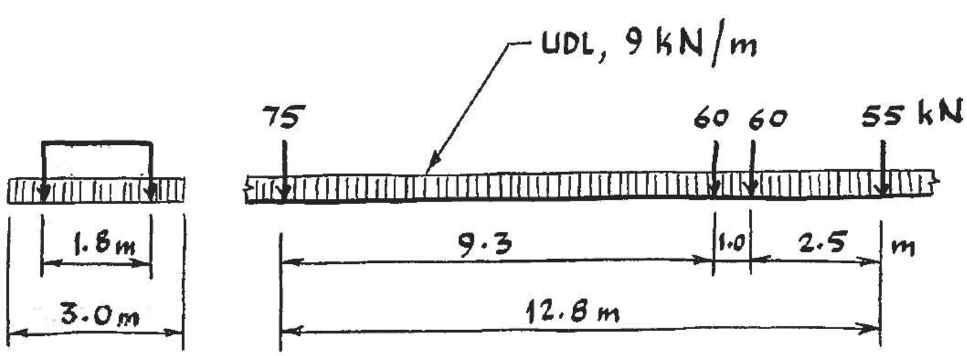


Figure 3 – Proposed Evaluation Lane Load for Bermuda

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

Vessel Impact

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.

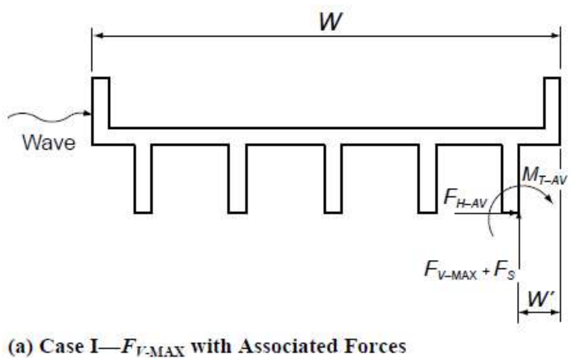


Figure 4 - Diagrams extracted from AASHTO BVCS (2008) illustrating the applied maximum vertical force and associated horizontal force, slamming force, and moment, applied along the length of the span or bridge

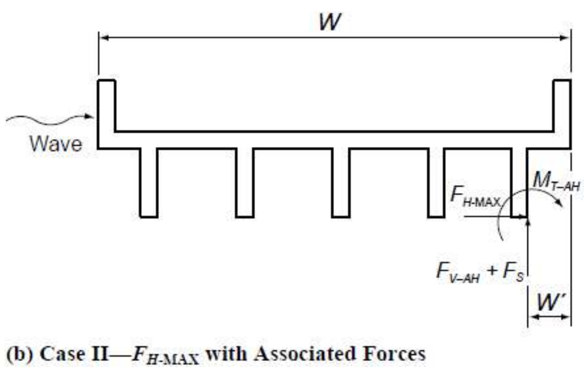


Figure 5 - Diagrams extracted from AASHTO BVCS (2008) illustrating the applied maximum horizontal force and associated vertical force, slamming force, and moment, applied along the length of the span or bridge

Rev P02

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

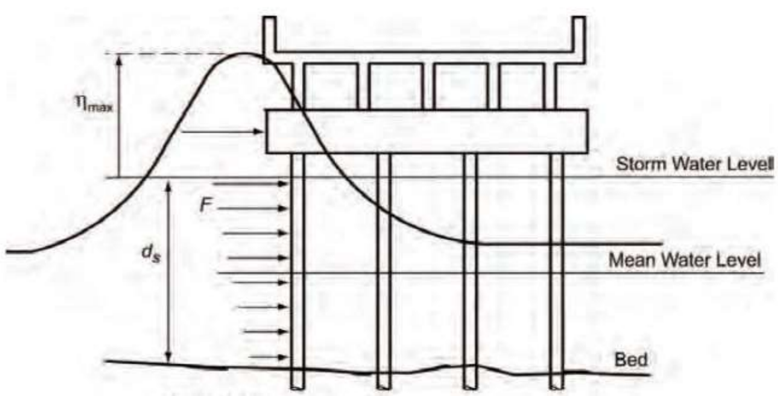


Figure 6 – Extract from AASHTO BVCS (2008) Illustrating the Interaction of Waves with the Bridge Structure

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.

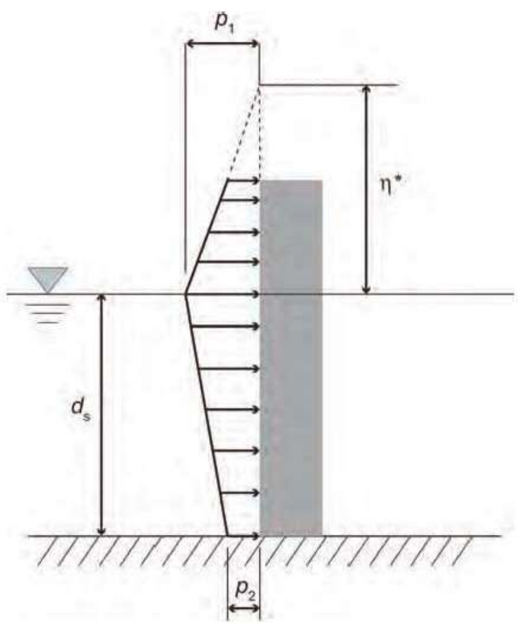


Figure 7 - Extract from AASHTO BVCS (2008) Showing Wave Force Profiles on Large Elements

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 $\gamma I = 1.00$.

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

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.

Rev P02

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.

Wind load shall be considered in the calculation of the fatigue load range during opening cycles of
Rev P03 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 γ_{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 γ_{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.

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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.

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

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

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

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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

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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).

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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.

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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.

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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 γMf = 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.

Loading within Steel work Box of Lift Span for Inspection and Maintenance

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/m² over a total area of 10 m² of any shape, which may be continuous or divided to give the most adverse effect, together with a UDL of 0.75 kN/m² 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.

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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 up of stresses resulting from the two previously mentioned analysis cases. Rev P02

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

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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

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.

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

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

Reference Documents

Design Criteria Longbird Bridge Replacement

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/m3
Reinforced Concrete will have a density of 2500kg/m3
Wet Concrete will have a density of 2600kg/m3

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. Wind loading will be considered using a fundamental design wind speed of 150 mph in accordance with the Bermuda Building Code 2014.

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

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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 5°C to 34°C. 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.

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Differences in the uniform temperature component between different structural elements will be considered in accordance with clause 6.1.6 of BS EN 1991-1-5:2003 along with the UK National Annex. In particular, a 15 degree Celsius differential will be considered between the main structural elements (arch top chord, arch bottom chord/deck and hangers).

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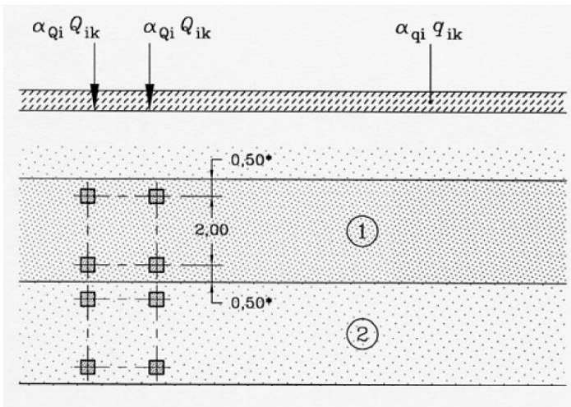
For temperature gradient the superstructure will be considered as Type 2.

No snow loading will be considered.

4.1.3 Actions relating to normal traffic under AW regulations and C&U regulations

The structure has been 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

Rev P02 By way of comparison Figure 2 and Figure 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 Rev P02 Ministry of Public Works. The loading arrangements depicted in Figure 2 and Figure 3 are based upon actual vehicles typical to Bermuda.

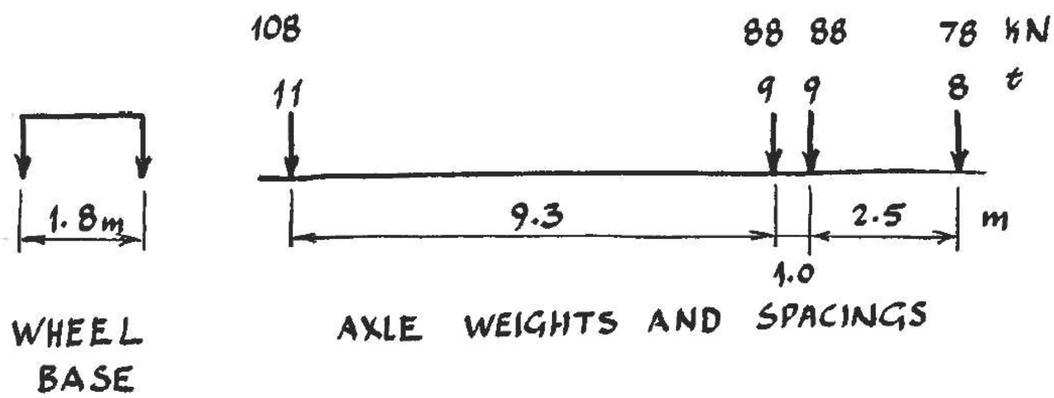


Figure 2 -Proposed Evaluation Truck for Bermuda

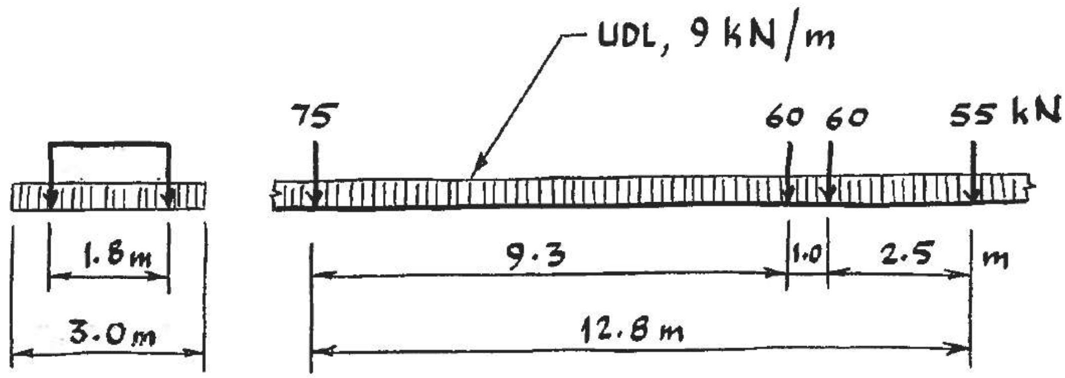


Figure 3 - Proposed Evaluation Lane Load for Bermuda

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 and are appropriate for detailed design in Phase III.

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.
For the footway, where the vehicle access is prevented by the VRS, a point load of 10kN will be considered acting on a 100mm x 100mm in accordance with BS EN 1991-2:2003 Cl 5.3.2.2(1).

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>Vehicle impact</u>

On Longbird bridge the primary structural elements are above the deck and although they are protected by the high containment kerb and VRS there is potential risk of vehicle losing control and striking a structural element. To address such a risk the arch ribs will be designed to sustain an impact force from the vehicle. The structure will also be designed to sustain a sudden loss of a hanger in accordance with clause 2.3.6 of BS EN 1993-1-11.

Vessel Impact

Ship impact into the superstructure and substructure has been considered.

As a fixed bridge span, navigation through Longbird Replacement is only possible for motorboats with low enough air draft to pass under the bridge. If a motorboat loses steerage or the skipper miscalculates, then the boat deckhouse or mast could impact the superstructure.

The design hull substructure impact load from the 50ft motorboat design vessel was calculated to be 840kN in accordance with AASHTO (1991). For further details of the design vessel and derivation of the associated vessel impact loads please refer to section 4.9 of the Phase II Feasibility Report (document number 3502-RAM-XX-XX-RP-CB-20001 rev. 02).

The substructure load will not be applied to the bridge as the substructure is out of the waterway and therefore vessel collision is not an issue. However, as per AASHTO, the vessel deckhouse load is estimated to be 20% of the substructure load, and the vessel mast impact load is estimated to be 10% of the deckhouse load.

Table 6 - Vessel Impact Summary

Impact Case	**Impact load**	**Location**
Head on impact of motor boat hull on bridge substructure	840kN	Not applied.
Glancing impact of motor boat hull on bridge substructure	420kN Applied separately to head on impact case.	Not applied.
Impact of motor boat deckhouses on bridge superstructure	168kN	Action applies from MHW + 1.5m to MHW + 3.5m*, in a direction parallel to the main channel axis.
Impact of motor boat mast on bridge superstructure	17kN	Action applies from MHW + 3.5m to MHW + 4m*, in a direction parallel to the main channel axis.

*These values are estimated based on a 4m air draft

Given the information on water levels and surge levels in relation to the proposed +4.2mOD soffit level, it is predicted that the worst collision case would be a deckhouse collision on the bridge superstructure. The structure shall be designed to be robust enough to withstand this force on the bridge deck.

Vessel Impact Protection

Rubbing strakes of durable timber or plastic will be provided along the sides of the bridge abutments to protect the structural elements from damage by minor glancing impacts.

Accidental vessel impact creates a risk to life, or injury, both to bridge users and to vessel users. During an impact, the vessels bow and/or deckhouse might be crushed, or the mast and rigging may collapse. The design of the bridge will aim to mitigate these risks. For example, the Longbird Bridge Replacement abutments have been profiled in plan to increase the likelihood that vessels will be deflected into the channel rather than suffer head on impact. Aids to navigation will also be provided to further reduce the risk of a collision.

Wind/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. The connections between the substructure and superstructure, will be provided to ensure that the bridge decks remain in place during the hurricane event. The hydrodynamic loading on the pier and abutments has been considered in accordance with section 4.9.12 of the Phase II Feasibility Report as replicated below.

Wave loads on bridge deck

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 geometry of the proposed Longbird Bridge Replacement have been idealised to represent the
Rev P02 AASHTO girder type cross sections to be in-line with the code

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 separately. 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 4 and Figure 5, 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.

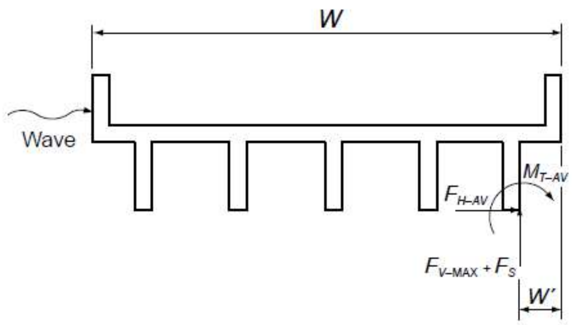


Figure 4 - Diagrams extracted from AASHTO BVCS (2008) illustrating the applied maximum vertical force and associated horizontal force, slamming force, and moment, applied along the length of the span or bridge

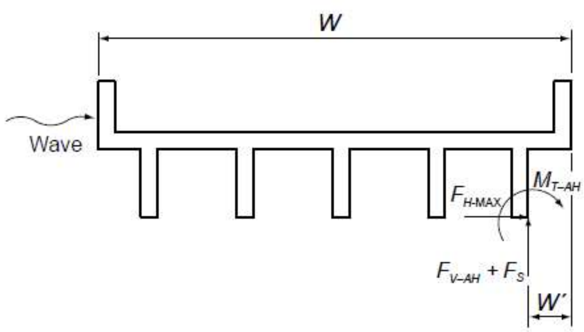


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Figure 5 - Diagrams extracted from AASHTO BVCS (2008) illustrating the applied maximum horizontal force and associated vertical force, slamming force, and moment, applied along the length of the span or bridge

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

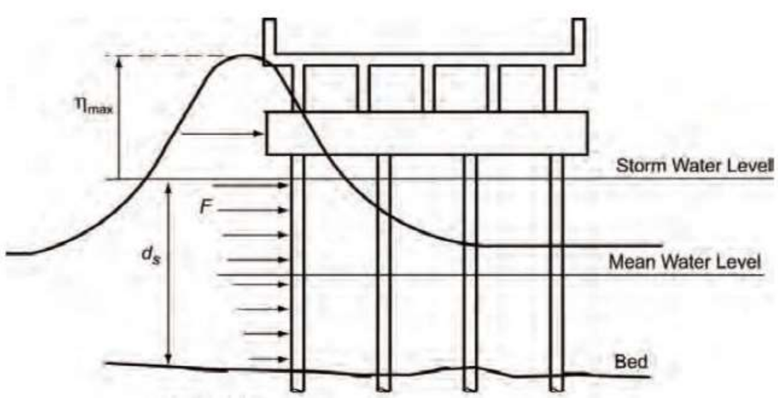

Figure 6 – Extract from AASHTO BVCS (2008) Illustrating the Interaction of Waves with the Bridge Structure

Longbird Bridge Parameters

The following parameters have been used to derive wave forces on Longbird Bridge. The water depth at both bridges has been taken as the deepest based on the review of bathymetric information available.

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S_L	Bridge Soffit Level above OD	4.20	m	13.78	ft
H_{max*}	Max wave height (limited)	**5.58**	m	18.29	ft
H_{max}	Max wave height	5.99		19.65	ft
Tp	Peak wave period	5.00	s	5.00	s
λ or L	Wave length	**39.03**	m	**128.06**	ft
d	Water depth below OD	6.00	m	19.69	ft
SLR	Relative sea level rise above water level by 2100	0.86	m	2.82	ft
$Surge$	1:150yr predicted surge level, mOD	2.20	mOD	7.22	ftOD
d_s	Storm water level (by 2100) above seabed	**9.06**	m	**29.72**	ft
η_{max}	Distance from the storm water level to design water crest	**3.90**	m	**12.81**	ft
	Non-linear wave assymetry factor	0.70			
r	Rail height	0.65	m	2.13	ft
γ_w	unit weight of water taken as 0.064 kip/ft^3			0.06	kip/ft^3
W	Bridge width	11.65	m	38.22	ft
Z_c	Vertical distance from the bottom of the cross section to d_s	**1.14**	m	**3.74**	ft
d_b	Depth of bridge deck	1.55	m	5.09	ft
d/L (present)		**0.15**			
d/L (by 2100)		**0.23**			
0.65 d_s		**5.89**			
λ /7		**5.58**			

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

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

Table 7 - Summary Wave Forces Case I

	Design Case I
	Longbird Bridge Replacement
$F_{V\text{-}MAX}$ (kN/m)	147.5
$F_{H\text{-}AV}$ (kN/m)	74.2
F_S (kN/m)	75.6
$M_{T\text{-}AV}$ (kNm/m)	1828.2

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For the Design of the bridge deck the actions in Table 7 above will be applied to the soffit at W/2 (=5.825m) from the centreline as illustrated in Figure 4.

Table 8 - Summary of Wave Forces Case II

	Design Case II
	Longbird Bridge Replacement
$F_{H\text{-}MAX}$ (kN/m)	92.3
$F_{V\text{-}AH}$ (kN/m)	129.8
F_S (kN/m)	75.6
$M_{T\text{-}AH}$ (kNm/m)	1253.8

Rev P02

For the Design of the bridge deck the actions in Table 8 above will be applied to the soffit at W/2 (=5.825m) from the centreline as illustrated in Figure 5.

Wind load coexisting with Case II wave loading will not be applied to the deck below top of parapet level as this zone is loaded by wave action.

<u>Results of wave forces on bridge decks with SLR taken as 0m</u>

Table 9 - Summary of Wave Forces Case I

	Design Case I
	Longbird Bridge Replacement
$F_{V\text{-}MAX}$ (kN/m)	59.9
$F_{H\text{-}AV}$ (kN/m)	61.4
F_S (kN/m)	47.0
$M_{T\text{-}AV}$ (kNm/m)	912.7

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For the Design of the bridge deck the actions in Table 9 above will be applied to the soffit at W/2 (=5.825m) from the centreline as illustrated in Figure 4.

Table 10 - Summary of Wave Forces Case II

	Design Case II
	Longbird Bridge Replacement
$F_{H\text{-}MAX}$ (kN/m)	87.0
$F_{V\text{-}AH}$ (kN/m)	75.1
F_S (kN/m)	47.0
$M_{T\text{-}AH}$ (kNm/m)	1115.7

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For the Design of the bridge deck the actions in Table 10 above will be applied to the soffit at W/2 (=5.825m) from the centreline as illustrated in Figure 5.

Wind load coexisting with Case II wave loading will not be applied to the deck below top of parapet level as this zone is loaded by wave action.

Reducing the SLR value to 0m, has a significant impact. For Longbird Bridge, it not only reduces the storm water level but also reduces the design wave height as it is limited by the water depth.

Rev P02 Overtopping Case

The design surge wave crest is above the bridge deck level; hence, an overtopping case will be considered. The bridge will be designed for a loading of 70kN/m applied along the length of the deck between inner faces of arch bottom chords. This will represent the loading from the static weight of water accumulated on the deck once the surge wave crest has passed.

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
Rev P02 bridge substructures based on Goda's method.

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

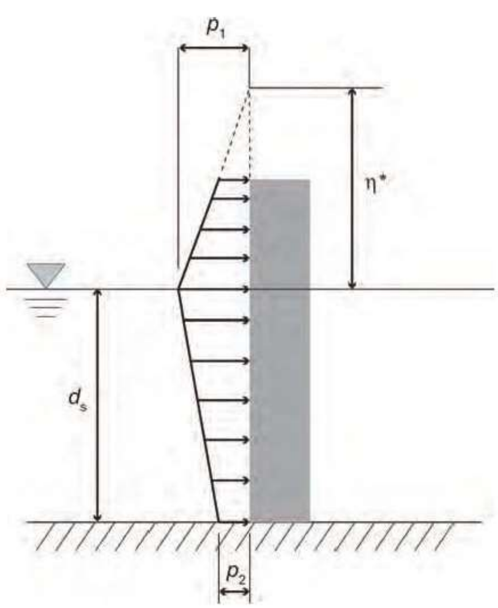

Figure 7 - Extract from AASHTO BVCS (2008) Showing Wave Force Profiles on Large Elements

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

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The results obtained for Longbird Bridge Replacement are presented as follows:

Table 11 - Summary of Wave Loads on Abutments and Walls – SLR=0.86m

	Longbird Bridge Replacement
p1 (kN/m^2)	36.4
P2 (kN/m^2)	16.1
η* (m)	8.4
d_s (m)	9.1

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When considering SLR=0.86m the value of peak pressure p_1, its application level of +3.10m OD and the dimension η* are common for all abutments/walls. Pressure P_2 and the dimension from storm water level to bed level, $d_{s,}$ shown in Table 11 are based upon an assumed bed level of -6.00m OD. For abutments/walls with bed depths other than -6.00m OD, p_2 is to be determined by linear interpolation from the p_1 and p_2 values from Table 11 using the dimension d_s applicable for the bed depth at the location under consideration.

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

The results obtained for Longbird Bridge Replacement are presented as follows:

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Table 12 - Summary of Wave Loads on Abutments and Walls – SLR=0.0m

	Longbird Bridge Replacement
p1 (kN/m^2)	36.0
P2 (kN/m^2)	18.0
η* (m)	8.0
d_s (m)	8.2

Rev P02

When considering SLR=0.0m the value of peak pressure p_1, its application level of +2.20m OD and the dimension η* are common for all abutments/walls. Pressure P_2 and the dimension from storm water level to bed level, $d_{s,}$ shown in Table 12 are based upon an assumed bed level of -6.00mO D. For abutments/walls with bed depths other than -6.00m OD, 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 location under consideration.

Wave Loading Calculation Approach

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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.

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 three separate scenarios will be considered as follows:

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1. Wave loads with SLR considered as 0.86m - Extreme Event Limit State [factor of 1.0] – wave loads on deck and abutments/walls will considered as coincident.
2. Wave loads with SLR considered to be 0m – Strength Limit State [factor of 1.75] – wave loads on deck and abutments/walls will considered as coincident.
3. Overtopping case with SLR considered as 0.86m – Extreme Event Limit State [factor of 1.0] – overtopping loading as noted above to be applied to the bridge deck - no wave loads considered on the deck and abutments/walls.

<u>Seismic loading</u>

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 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:

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

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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 the above case is taken as $\gamma I = 1.00$.

The spectral accelerations of the reference return period and the return periods of the seismic loading are tabulated below.

Table 13 - Spectral accelerations

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

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

<u>Superimposed Dead Load</u>

Load factors for bridge deck surfacing 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 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.

<u>Scour</u>

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.

Rev P04

The flow/tidal velocity appropriate to assess scour and design for mitigation measures is 0.92m/s based on maximum modelled tidal currents from the proposed Longbird Bridge 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)

A vehicle restraint system (VRS) will be installed on Longbird Bridge. It is proposed the VRS will be a tubular CHS positioned with its centroid 600mm above the adjacent carriageway and set back a minimum of 600mm from the traffic face. Additional protection will be provided by high containment kerbs on each side of the carriageway.

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

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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).

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 bridge shall be based on the travelled lane configuration; i.e. 2No. travelled lanes at 3.5m wide and shall comprise 0.5×10^6 ($=N_{obs}$) 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.

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 $\gamma Mf = 1.1$ will be adopted according to clause NA.2.5.3 of NA to BS EN 1993-1-9:2005.

Wind Induced Fatigue in Hangers

Rev P03

The requirement to design for fatigue under various sources of loading, including environmental loads such as wind, is inherent in the Eurocodes BS EN 1993-1-9, associated NA and PD 6695

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outlining the recommendations for the design of structures to this standard. As part of the detailed design process we will calculate the fundamental frequency of the hangers to determine the ratio between the first bending (translational) frequency and first torsional frequency, including a sensitivity check to take account of the likely range of tension within the hanger.

The results of the above will be used to determine the susceptibility of each hanger to divergence or flutter in accordance with clause A.4.2 of PD 6688-1-4:2015. No further specific wind induced fatigue check will be necessary for the hangers demonstrated not to be prone to divergence or flutter.

Loading for Abutment Inspection Galleries and Associated 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/m^2; 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.

Loading within the Deck Steelwork Box for Inspection and Maintenance

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 10m^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. 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.

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 Headroom provided

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The main bridge structure bridge will be designed with a mid-span headroom clearance of 3.67m above highest astronomical tide.

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.

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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 Longbird 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

<u>Superstructure</u>

The superstructure will be analysed as a three-dimensional model using the linear elastic analysis computer program LUSAS. Both thick shell elements and beam elements as appropriate will be assigned to different parts of the structure to form the three dimensional model.

If required, dynamic mode shapes and frequencies will also be determined from a three-dimensional model using LUSAS.

The substructure and pile caps will be analysed using standard elastic methods and hand calculations.

Pile loads will be determined 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

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. The transverse diaphragms will be designed to act compositely with the reinforced concrete deck slab.

The stiffness of the substructure concrete elements and piles 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

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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 Longbird 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

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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 Longbird 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

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

Reference Documents

Geotechnical Information Summary

APPENDIX 5

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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 Settlement (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	Not Applicable					
Maximum Movement (mm)						
Measures to Deal with Movement						

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APPENDIX 5
GEOTECHNICAL REPORT – HIGHWAY STRUCTURE SUMMARY INFORMATION

GEOTECHNICAL REPORT
HIGHWAY STRUCTURE SUMMARY INFORMATION

STRUCTURE NAME Longbird Bridge Replacement	CHAINAGE and OS Grid Reference 554153.13m E 139766.18m N	Comments
STRUCTURE TYPE Fixed Single Span Bridge	AIP Ref No 3502-RAM-LB-XX-RP-CB-30001	
DESIGN LIFE 75 Years		
RELEVANT TRIAL HOLES BH101, BH102 *(Report: Geotechnical Investigation for Two Bridges in Bermuda Islands: Longbird and St. George's Bridge, Final Report, October 2018)*		

Strata Thickness

Borehole		Thickness of Stratum (m)								
		Fill Material	Coralline Deposits	Clayey Silt Design Layer			Coralline Deposits	Silty Clay	Weathered Basalt/Basalt Breccia	Unweathered Basalt/Basalt Breccia
				Sandy Silt	Organics	Silty Clay				
Longbird Bridge	BH101 (North Abutment)	0.7	13.2	-	-	2.1	2.4	5.1	6.5	Extent not proven
	BH102 (South Abutment)	1.5	10.8	5.9	3.3	4.5	2.5	-	5.0	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 levels are assumed to be equivalent to that of sea level.

EARTH PRESSURE VALUE k_0*	Coralline Deposits	Silty Clay/Clayey Silt
	0.48	0.64

SOIL PARAMETERS

Stratum	Bulk Density, γ (kN/m^3)	Strength Parameters: Undrained Shear Strength, c_u (kN/m^2)	Strength Parameters: Effective Shear Strength Parameters Angle of Friction, Φ′ (°): Effective angle of shearing resistance, Φ′ (°)	Strength Parameters: Effective Shear Strength Parameters Angle of Friction, Φ′ (°): Effective cohesion (kN/m^2)	UCS (MPa)	Hoek Brown: mb	Hoek Brown: a	Hoek Brown: s
Fill	18.0	20						
Coralline Deposits	16.0		34	0				
Clayey Silt	19.0	70	21	0				
Silty Clay	19.0	70	21	0				
Weathered Basalt	21.3		36	181	3.2	3.383	0.51	0.002
Basalt	23.0		62	404	30	5.99	0.5	0.01

PILE DESIGN

Structure Element	Founding Stratum	Founding Rock Head Level (mOAD)	Pile Cap Head Level (mAOD)	Pile Toe Level (m AOD)	Pile Length (m)	Pile Diameter (mm)	Ultimate Bearing Resistance (kN)	Pile Tensile Load (kN)	Pile Compressive Load (kN): SLS	Pile Compressive Load (kN): ULS (Set C)	Notes
Northern Abutment (BH101)	Weathered Basalt	-24.6	-1.1	-27.1	26.0	900	6437	-1000	2000	2500	2.5 m rock socket
Southern Abutment (BH102)	Weathered Basalt	-29.6	-1.1	-31.6	30.5	900	6649	-1000	2000	2500	2 m rock socket

Pile type……………………………………………………Steel Tubular - Driven
Criteria for selecting pile toe level……………………. Founding Strata Strength/Stiffness
Allowance for negative skin friction within design…...Potential for settlement of supporting soil due to placement of rock armour and abutment fill to be considered in detailed design

Continued overleaf

SETTLEMENT

Structural Element	Founding Level (m AOD)	Immediate Settlement (mm)	Total Settlement (mm)	Time for 90%	Settlement Remaining at Completion	
Northern Abutment (BH101)	To be completed on receipt of design loads					
Southern Abutment (BH102)	To be completed on receipt of design loads					
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	Not Applicable					
Maximum Movement (mm)						
Measures to Deal with Movement						