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DC STRUCTURES STUDIO

LONGFORD DESIGN STATEMENT

Longford Shared Path (#2019/11) –Design Statement

Prepared for

Gore District Council

Issued date

June 2020

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Document Status

Rev	Description	Author	Date
A	Tender Design	Dan Crocker	Sep 2019
B	Preliminary Design	Dan Crocker	Nov 2019
C	For Peer Review	Dan Crocker	May 2020
D	For Construction	Dan Crocker	June 2020

Review and Approval

Description	Person	Signature	Date
Author	Dan Crocker CPEng		26 June 2020

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(Above image showing an artistic visualization of the proposed bridge)

1. Project Overview

1.1. Introduction

This document summarises the scope, scope exclusions, design codes, geometric constraints, construction methods, programme, design loads, design parameters, material characteristics and the design and planning methodology to be used for the "Longford Shared Path Design Contract (#2019/11)" for Gore District Council (GDC) being led DC Structures Studio Ltd (DC Studio).

1.2. Scope

The overall scope is to come up with cost effective cycle bridge option that will form part of the proposed cycle network. The bridge will cross the Mataura River 500m north of the State Highway 1 (SH1) road bridge.

The connecting cycleway (circa 1km) will run west across the flood plain and onto the existing western flood stop-banks. The cycleway then runs south (underneath SH1 bridge and the rail bridge) until becoming parallel with River Street and terminating at Hyde Street alongside Hyde Park (on the other side of the road).

The purpose of this report is to summarise and discuss the key design criteria.

The intention of the bridge proposed by our team is to provide Gore District Council (GDC) with a cost effective, functional, but low maintenance bridge, with enough visual appeal that it can successfully capture the imagination of the surrounding region and encourage take-up of cycling and walking. We believe this meets the fundamental goals of this project.

1.3. Design Team

Role	Organisation	Key Contact
Bridge Architect & Engineer (PS1)	DC Structures Studio Ltd.	Dan Crocker, CPEng
Geotechnical and Hydrological Engineer (PS1)	Riley Consultants Ltd.	Eli Maynard CPEng
Independent Structural Peer Review (PS2)	Beca Ltd.	Liam Edwards, CPEng
Independent Geotechnical Peer Review (PS2)	Geosolve Ltd.	Colin Mcacdiarmid, CPEng
Independent Hydrological Peer Review	Geosolve Ltd.	Hank Stocker / Colin Macdiarmid CPEng

2. Bridge Design Basis

2.1. Concept

We are proposing to use a 90m span x 3.0m wide cable stay footbridge with a 32m high steel mast. The crisp white circular hollow section (CHS) steel mast will be tilted backwards by 15 degrees to create a striking visual statement. The steel mast is a key structural element but it has also been arranged and styled to form a sculptural landmark in its own right. The sculptural mast has been positioned on the western side so that it can be viewed by the community and prospective tourists using the SH1 road bridge (further down-stream). It is hoped that having this architectural bridge visible to passing traffic will help encourage commuters/tourists to stop (rather than pass through). Whilst people and families are checking out the bridge, it is expected that they will also use local amenities and visit shops.

The superstructure will be suspended from the mast using high strength spiral strand cables connected with elegant proprietary cable forks. The cables will be Galfan™ coated (5% aluminium + 95% zinc) to optimize durability.



Figure 2-2 – Artist's impression of Longford Shared Path Mataura River Bridge

The balustrade will be formed from glulaminated timber posts combined with a stainless-steel handrail and stainless-steel anti-climb infill mesh. This will be combined with timber or recycled plastic decking. This balustrade and decking system was recently used as a cost effective but architectural and low-maintenance solution for the Water of Leith Cyclebridge (Dunedin). See example below.



Figure 2-1 – Example of balustrade system as used on Water of Leith Cyclebridge

By using traditional bridge building materials such as concrete and steel and combining them effectively with the soft modernity and sustainability offered by engineered timber and recycled composite plastic decking, we have tried to create a truly unique bridge which is economical, durable, and architecturally striking.



Figure 2-3 – Artist's impression of Longford Shared Path Mataura River Bridge

2.2. Design Requirements and Standards

To demonstrate compliance with the New Zealand Building Code parts B1 (structure), B2 (durability) and F4 (Falls from height) the bridge has been designed to the New Zealand Transport Agency bridge Manual v3.3 (NZTABM) as modified by project specific clarifications, departures, or augmentations, as listed below:"

Clarifications:

The following key clarifications apply to our approach (parameters not otherwise listed in the design standards):

1) ≤80km/h wind considered for bridge in combination with pedestrians (refer section 5.13).

Departures:

The following design philosophies/parameters are considered departures from the typical NZTABM v3.3 methodology:

1) The 90m long bridge is designed for a 2.6KPa load everywhere or a 5KPa loading over the middle 30m (1/3 of the span). This is consistent with the NZTABM. However, such an extreme 5KPa loading event will not be considered over the end spans (1/3 either side of mid-span). Noting the desire for economic solutions it is not considered appropriate in this rural location given there are no vantage points or activities that would justify this highly localised density of pedestrians at the ends of the bridge (refer section 5.3 for more discussion). (Ref NZTABM Cl 3.4.14 a).

2) Keeper plates will be used to retain bearings in place to the underside of the steelwork. Using keeper plates in lieu of vulcanizing (as requested by NZTABM) has been preferred to minimize the attachment complexity to the underside of the primary beam. This improves the ability to corrosion protect all elements above the bearing to the same standard. Keeper plates are widely used internationally and are deemed appropriate for the scale and economy of footbridges. (ref NZTABM Cl 4.7.2.d).

Augmentation:

The following inputs and standards have been used to augment the documents referenced in the NZTABM v3.3:

- Longford Bridge - Geotechnical and Hydraulic Detailed Design Report Rev 2 (Riley, June 2020);
- Recommendations for Stay-Cable Design Testing and Installation (PTI, 2012);
- Design of Lightweight Footbridges for Human Induced Vibrations (JRC-ECCS, 2009);
- ECCS guide for pedestrian vibration.
- Wind Effects on Cable-Supported Bridges (Xu, 2013);

Key standards as referenced within the NZTABM are listed below:

- NZS1170.0 to .5 Australia and New Zealand Structural Design Actions;

- Austroads Design Guide Part 6A;
- AS 5100.6 Bridge Design, Part 6 – Steel & Composite 2004;
- AS 5100.4 Bridge Design, Part 4 – Bearings and Deck Joints;
- NZS3404:1997 Steel Structures Standard;
- NZS 3101 Design of Concrete Structures 2006;
- AS 1720.1—2010 Timber Structures;
- NZS3603-1993 NZ Timber Structures Standard;
- NZTA Protective Coatings for Steel Bridges 2013 including amendment September 2017;
- BS EN-1991-2:2003 UK National Annex;
- BD49/01 Design Rule for Aerodynamic effects on Bridges (Highways Agency, UK);
- AS 5216-2018 Design of post-installed and cast-in fastening in concrete.

2.3. Functional Requirements

- Minimum bridge width = 3.0m measured at the deck surface;
- Minimum vertical clearance (on bridge) = 2.5m measured at the deck surface;
- Maximum allowable pathway gradient = 7%
- Minimum barrier height = 1.4m;
- Sloped surfaces to have non-slip surfacing.
- Barrier infill to be non-climbable as per NZBC F4;

2.4. Bridge Decking

We are proposing to use proprietary FRP composite decking. The main benefit of using composite decking (compared to concrete or steel decking options) comes from its lightweight properties and proven performance in exposed environments. The composite deck planks will only need limited maintenance before being replaced 50 years into the bridge's design life.

Because the anti-slip treatment is integral with the base material there is no maintenance demand for touching up the surfacing.

Timber options can also be considered as a cost-effective option. The only draw-back of timber decking is that it requires plastic mesh overlay to improve traction and these can look low-quality and/or incur further maintenance issues. Precast concrete decking options may provide a suitable alternative but these would add significant weight and can increase overall price.

2.5. Bearings and Expansion Joints

- Articulation will be provided using elastomeric bearings and pads conforming to AS5100.4;
- The eastern abutment will have elastomeric bearing with ability for large longitudinal displacement to enable cable stay load take-up and thermal movement;
- Anchor bolts and/or shear keys will be used to connect the superstructure to the abutment beams.
- A chequered stainless-steel sliding plate will be used at the eastern sliding end of the superstructure to bridge the potential gap caused by cable stay load take-up and thermal movements.

2.6. Services

2.6.1. Water-main

- During early kick-off discussion it was confirmed that the bridge will need to carry two 350mm diameter watermains. This will require welded attachment details for two twin steel pipe sleeves to accommodate 350DIA water pipes. Sleeves will be attached to the underside of the bridge and will need to pass through the concrete abutment walls.
- Pipe sleeves will have a 1600mm gap at mid-span for the inclusion of a pressure release valve that can be access from above (by removing one side of the deck panel);
- The “utilities” themselves will be designed by Beca ltd. and installed by the utility providers (pipes, trenches, joints, etc.).
- Pipe are considered discontinuous at abutment face and over midspan and at superstructure splice locations. Sleeves are not considered to have any structural influence on the bridge.
- The “utilities” themselves will be designed by Beca ltd. and installed by the utility providers (pipes, trenches, joints, etc.).
- Steel sleeve and waterpipe loadings are discussed in section 5.

2.6.2. Future Services

- It is not economical or practical to accommodate the normal NZTA future service load of 0.25KPa across the full width of the bridge in combination with the 2-350DIA watermains. There is very limited space for any significant future services.
- Future service provision has been discussed with both GDC and Beca Ltd (Watermain designers) and it was agreed that two additional continuous fusion welded 160 OD HDPE ducts will be provided for future services.
- A green duct (future data) and blue duct (future water) will be provided. These (empty) ducts will be pre-installed as part of the permanent works.
- If in future more water (rather than data) is required, it can be run inside the blue duct. However, it is noted that Chorus requires a green duct for data/fibre connections.

2.7. Drainage

- Stormwater run-off will not be collected from bridge decks and will be allowed to pass directly into the waterway. A 2.5% cross fall is provided on the FRP panels to accommodate this.

2.8. Bridge Lighting

- Bridge lighting is not currently part of scope;
- Functional and architectural lighting can be allowed for on request.

2.9. Bridge Inspections & Maintenance

- A detailed bridge inspection shall be performed every 6 years by a competent engineer (CPEng engineer with experience designing bridges);
- Detailed inspection of the underside can be done by small boat at high tide. Clearance is circa 3.5m which is reasonable for detailed inspection;
- The steel mast and mast cable connections can be inspected by Cherry Picker (or similar);

- In the future it is envisaged that drone inspection maybe more common place and an economic choice for Council.

2.10. Waterway Design

2.10.1. Design Basis

- Basis for design to be NZTABM Section 2.3 Waterway Design with reference to AS5100.2-2017;
- Riley have performed hydrological assessment as per the ***Longford Bridge - Geotechnical and Hydraulic Detailed Design Report Rev 2*** (Riley, June 2020). Refer to that report for more information;
- No permanent piers are permitted in the permanent river channel;
- The western embankment required as part of the works (for landing the bridge) is to be constructed over a flood plain (refer to General Arrangement Plan in Appendix A). The influence of this construction within the flood plain has been modelled hydraulically. Protection of the abutments and approach embankments are as contained in the Riley Report;
- Recommended freeboard above SLS2 event is 1200mm as per NZTABM v3.3 cl 2.3.4. This assumes large logging and tree debris is possible upstream;
- Adopted design flood levels:
  - 1/50 year event + climate change = 75.8mRL at East and 75.5m RL at west (linear change);
  - 1/500 year event = 76.7m RL (limited to existing eastern stop-bank level);
- To limit significant increase in height to the eastern stop-bank (circa 76.7m RL) the minimum NZTABM 600mm freeboard has been adopted locally. The consequences of this are that a small 9m and 15m stretch at the eastern region will collect debris during the SLS2 and ULS flooding events respectively. Since the eastern 20m of bridge is over existing footway, removal of this debris is considered simple maintenance and debris loading has been considered in the superstructure design (refer to Section 5);
- It is noted that the “1 in 50 year + climate change” design event is larger than the recent 2020 event (which was the largest on record). The minimum clearance of 600mm shown on the drawings would increase to 1500mm when considered for the 2020 event (and more than 3m and 2m at midspan and west embankment respectively).
- As per AS5100.2 2004 cl 15.5.3 debris forces will only be applied to superstructure where soffit level is less than 1/500 year event (ULS) + 600mm = 77.3m RL.
- Curvature has been added to the superstructure so that the majority of the bridge is above the 77.3m RL debris threshold (so that loads can be more easily transferred to abutments).
- Adopted soffit levels:
  - East = 76.4mRL (600mm freeboard)
  - Mid-span = 77.9m RL (2250mm freeboard)
  - West = 77.04mRL (1540mm freeboard)

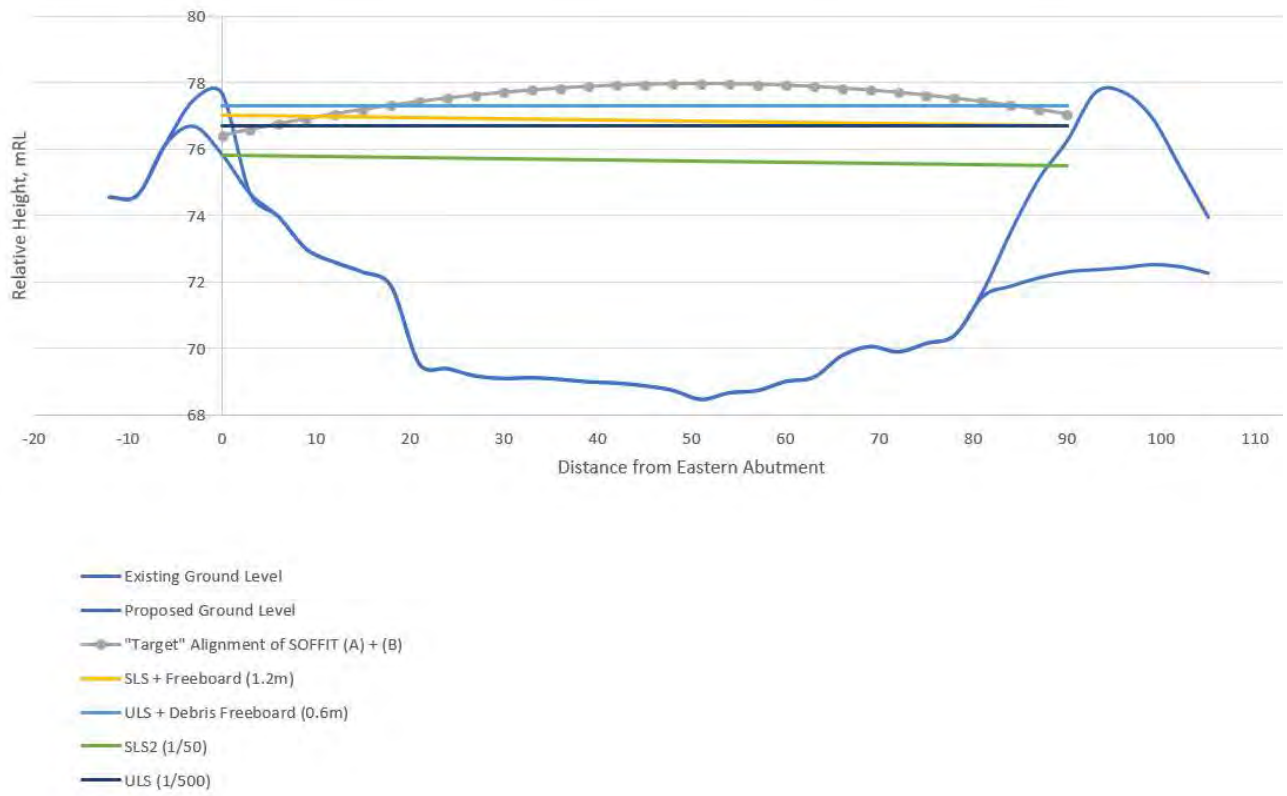


Figure 2-4 – Vertical alignment and flood clearance

2.11. Geotechnical Design

2.11.1. Geotechnical Design

Geotechnical design was performed by Riley Consultants Ltd. Some of the key ground conditions are summarised below. Refer to "***Longford Bridge – Geotechnical and Hydraulic Detailed design Report"*** (Riley, April 2020) for more information.

2.11.2. Geological setting

The regional geological setting of the site comprises an interbedded sequence of quaternary fluvial sediments eroded from the inland schist basement, overlying weakly metamorphosed sandstone (greywacke) and siltstone (argillite) of the North Range Group (Murihiku Terrane).

The GNS Science 1:250,000-scale QMAP and accompanying bulletin for the area (Map 20 "Murihiku Area" 2003) indicates the site is underlain by modern to postglacial floodplain deposits (Q1a) consisting of unconsolidated gravel, sand, silt and clay, and minor peat. These alluvial deposits are underlain by the North Range Group which consists of well-bedded siltstone and fine sandstone. The North Range Group rock unit dips 60° to 70° to the southwest and trends approximately northwest – southeast where it is mapped adjacent to the alluvium near the site. .

2.11.3. Seismicity

For the purposes of structural design, the site is assessed as subsoil category Class B – Rock, in accordance with NZS1170 – Structural Design Actions, based on the shallow depth to rock observed in the boreholes and the fact that the bridge will be supported on a series of bored concrete piles socketed into the rock. For the purposes of liquefaction assessment, we have also considered the site as Class C – Shallow Soil owing to the thickness of embankment fill observed.

No active faults requiring specific consideration for design are mapped within 20km of the site. However, two active faults, the Blue Mountain No. 1 Fault and Spylaw Fault are mapped within 30km of the site, along with several unnamed inactive faults, are mapped by GNS Science.

2.11.4. Stratigraphy

A simplified geological model was developed for the site for the purposes of evaluating foundation conditions and the implications for design. The model comprises embankment/stopbank fill where present, overlying a thin mantle of alluvium overlying basement rock:

Embankment Fill consists of a layer of medium dense, Silty SAND overlying very stiff gravelly SILT. Uncorrected SPT's of between 16 to 22 were recorded in the gravelly SILT. No testing was undertaken on the sand.

Paleosoil or possible buried topsoil was identified below the embankment fill in BH6 only.

Alluvium was encountered below the topsoil on the west bank and below the fill on the eastern embankment. This layer is typically described as soft to firm, low plasticity, sandy SILT and medium dense, poorly graded fine SAND with some Silt, overlying medium dense to dense, sub-rounded, sandy GRAVEL.

North Range Group Siltstone and Sandstone is present across the site below the alluvial sediments on the banks and is present on the riverbed. Sandstone and siltstone layers were recorded as interbedded and typically slightly to moderately weathered, and moderately strong. Rock fragments were recovered as slightly to moderately weathered, strong to very strong, with extremely to closely spaced joints and occasional sheared/shattered zones. A zone of completely weathered to highly weathered rock is indicated below the eastern embankment in BH6. Standard Penetration Tests (SPTs) recorded uncorrected N values of 50+ to bouncing, i.e. medium dense to dense conditions.

2.11.5. Liquefaction Assessment

No liquefaction is predicted for either serviceability limit state (SLS) nor ultimate limit states. No specific consideration for strength loss of the subsoils during seismic shaking is therefore required. Notwithstanding that, some minor co-seismic instability of the riverbank, west embankment and/or stop-bank is possible, with resultant lateral loading applied to the piles. It is considered that such loads can be readily accommodated by the proposed reinforced concrete piles.

2.12. Design Life & Whole of Life Cost

2.12.1. Design Life Basis

Our design is based on the following:

- When assessing loads in accordance with NZ Transport Agency Bridge Manual design life for bridge is 100 years;
- 100 years will be used for concrete elements (when specifying covers);
- 100 years for stay cables;
- ≥ 40 years to first maintenance for all steelwork protective coatings;
- ≥ 50 years will be used for replacement of structural timber components.
- ≥ 50 years will be used for replacement of FRP decking.

2.12.2. Durability and Maintenance Intervention

By only requiring intervention once in the life of the structure, we believe timber and/or man-made plastics/composite will minimise the maintenance activities of GDC. We have used glulaminated timber joists combined with composite plastic decking planks both of which have a 50-year maintenance life. We have used timber posts combined with marine grade stainless steel horizontal rail and flexible mesh wire infill. Timber combined with stainless steel provides a very modern but durable appearance.

We have used steel for the mast and underlying superstructure frame so that a 100-year design life can be achieved for the bridge's primary load path elements. This also provides a permanent platform from which the timber and man-made elements can easily be replaced. We had considered a glulam primary beam as a more cost-effective option (eliminating steelwork altogether) but this would require total replacement of the superstructure after 50 years (albeit re-using the same abutments and anchorage). The steel-mast can be easily accessed and painted from either a cherry picker, man-cage, or scaffold without any risk of contaminants being released into the river. Full re-painting would be required every 40 years with paint touch-ups of the colour system required every 15 to 20 years.

The steelwork skeleton used in the main-span can be overcoated with compatible paint systems e.g. an acrylic elastomeric that would not require blasting over the waterway. Removal of FRP panels could be undertaken for access to steelwork for repainting and/or inspection. A movable gantry/scaffold could also be developed by the maintenance contractor supported between the bottom flanges to recoat the steelwork. Weight of these systems are usually negligible compared to the assumed pedestrian loading with 1.76 factor but pedestrian access to the bridge could be reduced or more likely prevented during these maintenance activities as there are alternative temporary routes (e.g. over the existing SH1 bridge).

Detailed inspection of the underside can be done by boat at high tide. Clearance is circa 3.5m which is considered reasonable for detailed inspection. In the very near future it is envisaged that drone inspection will be a common place and economic choice for Council.

Cable stays are to be marine grade zinc/aluminum coated spiral strand wire rope. Each of the individual spiral wires that make up the strand are to be protected with a minimum 180g/m^2 coating consisting of a 95% zinc + 5% aluminum. This protective system in the exposed C2 corrosion zone will prevent the need for cable replacement during the 100-year design life. A damage or vandalism case is also considered as discussed in the Loadings Section of this report which assumes a modified live load (to account for the lower probability of high pedestrian volumes during vandalism or damage). The bridge design will allow for bridge cable replacement based on assuming the bridge is closed for such maintenance (not safe for pedestrians to use the bridge whilst cables are being hoisted into place).

Abutments and concrete piles will be designed as “maintenance-free”. Reinforced concrete piles, abutments, and the backstay anchor surrounds have been used with concrete additives and cover to provide 100-year life with minimal maintenance.

Elastomeric bearings use on the eastern side are assumed to last circa 50 years (although it is very rare for these to need replacement). In this instance the deck can be nominally jacked by ≤ 15mm to enable the removal and replacement. The jack will need to be placed in front of the existing bearing and positioned directly under the main girders. Elastomeric rubber pads are assumed to have 100 year life (as typically used for NZTA highway bridges constructed from precast concrete hollow-core beams).

2.12.3. Whole of Life Value

The materials and maintenance schedule we are proposing is similar to that adopted for the recently completed Water of Leith Cable-stay Bridge. For that bridge, a Whole of Life Cost Estimate (which considers the maintenance interventions discussed above) was included as part of the design. Because timber does not need intervention until year 50, Whole of Life Costing indicated only a 1% increase in the Capital Cost over the full 100-year life. This assessment was done as per NZTA Bridge Inspection Manual 2001, Section 2 (2.9 - Economic Evaluation) using a 10% discount factor. By only needing a 1% increase in 2017 capital over the 100-year life the design represented great long-term value and performance for Dunedin City Council.

The Water of Leith Bridge was designed for a marine environment and thus cables needed replacement every 30 years. Owing to the much milder atmospheric condition of the inland Gore location, the cables do not require any replacement and thus we anticipate that the Whole of Life Costing for the Longford Mataura River Bridge we will be substantially improved i.e. negligible long-term additional cost (after allowing industry standard discount factors).

2.13. Safety in Design

Safety in Design (SiD) was undertaken to improve safety of the construction, operation, maintenance, and demolition of the structure. This work is documented in the SiD register of Appendix D.

3. Connecting Cycleway Design Basis

3.1. Geometric Design

3.1.1. General approach

The cycleway alignment and geometrics will be designed in accordance with recommendations made in Austroads Design Guide Part 6A.

3.1.2. Pathway widths

Following consultation with Erik Teekman of NZTA (October 2019) it has been agreed that the following approach to pathway widths will be adopted:

- Bridge width to be 3m wide between barriers;
- Path width through flood plain (and any other regions not over stop-banks) to be 3m wide;
- Path over existing flood stop-banks to be 2.2m wide (or as large as possible without the need for stop-bank modification);
- If required stop-banks can be widened in the future as a new project or as part of ongoing stop-bank maintenance;

3.1.3. Pathway radii and grades

- Designed to New Zealand Cycle Trail Design Guide, Feb 2015, 4th Edition (NZCTDG);
- Cycleway to be ≤ Grade 2 cycleway (easy riding);
- Bridge and approach to be ≤ Grade 3 (intermediate riding);
- Maximum allowable pathway gradient =
  1. <5% where practicable (Austroads 6A);
  2. < 8% for no more than 10m length (NZCTDG Grade 3);
- Radius of turns:
  1. 9m centre line radius desirable (Austroads 6A)
  2. 4m minimum with at least 5m desirable to outside of turn (NZCTDG Grade 2);
- Minimum barrier height = 1.4m (where fall from height > 1.0m);
- Sloped surfaces to have non-slip surfacing (NZ Building Code).

3.2. Pavements and surfacing

- Surfacing along stop-banks will be asphalt or exposed aggregate concrete (to be confirmed during tendering);
- Surfacing over the flood plain will have a surface level to match the existing ground level to prevent any detriment to the flood zone. Permeable paving will be preferred to eliminate the risk of hydraulic uplift during a large flood event (an issue with concrete pathways subject to hydraulic actions). This can be discussed further with Environment Southland following award to maximise likelihood of obtaining resource consenting.
- Pavements and surfacing approaching the bridge will be based on recognised NZ council standard details consistent with the adjoining paths that we will nominate and show on our drawings.

3.3. Drainage Design

- The shared footpath/cycle-path will have a minimum 2.5% cross fall and will drain to surrounding permeable landscaping e.g. grass and/or soil.
- No drainage design beyond that described above is proposed.

3.4. Signs and markings

- We are proposing simple painted shared path symbols that will be shown on our drawings. This is typical for New Zealand cycleways.

3.5. Lighting and associated electrical works

- Pathway lighting is not currently requested in the RfP. Where this is requested by council it can be added to our cycleway layout drawings on the basis that it will be consistent with existing light systems (i.e. same light poles) and spacings used elsewhere in Gore.
- If detailed lighting design for the cycleway is required this will need to be discussed as it may require us to engage a lighting professional.

3.6. Utilities Design

- Existing utilities as provided in the tender documents / tender survey will be shown on our drawings.

4. Engagement and Consenting

4.1. Resource Consenting

Land Pro Ltd. will undertake the planning assessment to determine what approvals may be required under the Resource Management Act (RMA) for the new cycleway and bridge.

The following planning requirements are deemed required for obtaining a Resource Consent and shall be allowed for by the contractor in pricing the works:

- The contractor is to prepare and have in place a Construction Flood Management Plan that clearly outlines procedures and processes for removal of all plant, stockpiles, materials, people, temporary works, etc. from the floodplain within 20 hours of a flood warning being raised and/or increased levels being recorded at the “Fairlight” or “Piano Flat” monitoring stations. This Plan is to be submitted to Environment Southland 14 days prior to the start of construction for review and confirmation. The Contractor is responsible for submitting a plan acceptable to Environment Southland. Embankment that has been constructed above a level of 74.1mRL (3.25m above normal flow) may be considered permanent works and not require removal. For embankment to be considered permanent works it must be constructed and protected with rip rap as per drawings and signed off by the Geotechnical/Hydraulic Engineer. More information relating to monitoring station contact details can be found here: https://www.es.govt.nz/repository/libraries/id:26gi9ayo517q9stt81sd/hierarchy/online-services/flood%20warning/documents/flood-warning-factsheets/mataura_flood_warning_web.pdf

- No contaminants, other than sediment released from the bed, are to be discharged to water (from Rule 55A of the Southland Land and Water Plan). Suitable procedures and methods to achieve this shall be identified in the Contractor’s Construction Management Plan. This Plan is to be submitted to Environment Southland 14 days prior to the start of construction for review and confirmation. The Contractor is responsible for submitting a plan acceptable to Environment Southland.

4.2. Building Consent & Producer Statements

It is understood that GDC consider these works exempt from requiring building consent.

Our bridge design, earthworks design, and Geotechnical Interpretive Report will be supplied with a Design Certificate (PS1) signed by Chartered Professional Engineers (CPEng) with competencies in bridge and geotechnical engineering respectively.

Given the nature and complexity of the proposed cable-stay bridge, we recommend the bridge design is also submitted with an Independent Peer Review Checker Certificate (PS2) signed by an independent Chartered Professional Engineer (CPEng) with competency in bridge engineering.

5. Design Loads

5.1. Dead Loads (DL)

Dead loads were derived from unit material weights and structural component dimensions. Weights of materials have been allowed for as follows:

5.1.1. Structural Self weight

- Reinforced / prestressed concrete 25 kN/m^3 (23kN/m^3 if beneficial)
- Structural steel 77 kN/m^3;
- Sawn radiata timber 8 kN/m^3;
- Glulaminated Timber 6 kN/m^3;

5.1.2. Superimposed loading

- An allowance of 0.2 kN/m per side has been assumed for timber post and mesh fill barriers;
- An allowance of 0.15 KPa for FRP decking panels;
- No allowance has been made for any surfacing or future surfacing over decking;
- 0.4 kN/m has been allowed for future services (2-160 HDPE ducts);
- Steel water-pipe sleeves, associated cleats, and pipe spacers are taken as 0.7kN/m per sleeve;
- Full PE water-pipe is taken as 0.94kN/m per pipe;
- Deflection estimates during construction are based on 60%SDL to consider empty pipe sleeves.

5.2. Construction Loads (CN)

- A nominal 4kN point load is assumed;
- This is based on two people at 100kg each + 200kg of equipment for stressing of cables. Point load is applied at cable subject to stressing (considered at each stage of construction);
- Access along steel beam is by means of a guide wire with fall arrest equipment attached to either abutment end. No temporary staging across the steelwork is assumed.

5.3. Cyclist Live Loads (FP)

Live loads have been discussed with Erik Teekman of NZTA (October 2019). It has been agreed that the following represents the “best value” outcome for council, NZTA and ratepayers:

- Bridge width to be 3m wide between barriers;
- Live load is as per Section 3.4.14 of the NZTABM v3.3;
  - Live loads based on NZTABM loading assumes no crowd loading effect. This reflects the non-urban setting (semi-rural) and the bridge not being in close proximity to any stadia/universities/malls etc;
  - Localised members such as decking, glulam joists, steel cross beams, and associated connections, will be designed for 5KPa live loading;

- When considering global members such as the primary steelwork, cables, mast, and foundations, the large 90m span reduces the typically applied 5KPa live load to 2.60KPa;
- The factored ultimate limit state (strength case) will thus be based on a factored design case of (γf=1.76) x (FP=2.60KPa) = 4.58KPa (rather than 8.8KPa for a crowd loaded structure).
- Factored ULS load vs. span are shown in Figure 5-1 below for various scenarios/codes;
- The strength case adopted, equates to (4.58KPa x 90m length x 3m width) / (1.2kN per person) = 1,030 people using the bridge in a single instance;
- We will report/document loading assumptions on the drawings for future use;
- All live load is considered for the full width of the 3.0m usable deck width;
- Three live load cases are considered along the length of the bridge:
    - 2.6KPa Live load everywhere;
    - 5.0KPa Live load to the middle 1/3 of the bridge;
    - 5KPa applied locally over glulam beams and steel cross beams;

The overall bridge has been designed for a 2.6KPa live load. A middle 1/3 of the bridge has been loaded to the full 5KPa loading (this is consistent with adopting increased demands where reduced loaded area is used). We note that 30m of 5KPa loading equates to 64% of the total 2.6KPa x full 90m of bridge. Thus, this scenario assumes that almost 2/3 of all expected users are standing on 1/3 of the bridge (for the ULS event this equates to 660 of 1030 people standing on 1/3 of the bridge). Designing the bridge for circa 1000 users was discussed and agreed with both Council and NZTA during the project kick-off meeting (noting Gore's semi-rural township).

We propose the following approach be adopted for the purposes of creating an economic structure:

1) This loading scenario will only be considered for load combination ULS1A noting that congregation of a huge density of pedestrians on one area of the bridge is considered an extreme live load event. As such this scenario does not need consideration in combination with other extreme loads such as wind, temperature, etc.

2) This scenario will only be considered applied to the middle third. This assumes that an event may occur on the river in the future to justify 2/3 of the total design live load being concentrated over only 1/3 of the bridge length. There are no obvious vantage points for viewing from the bridge that would result in such a highly localised scenario at the ends of the bridges.

We note that all FRP decking, cross beams and glulam beams have been designed for the higher 5KPa load noting that a smaller length of only 6m is more likely to be subjected to crowd loading (compared to a whole 1/3 of the bridge).

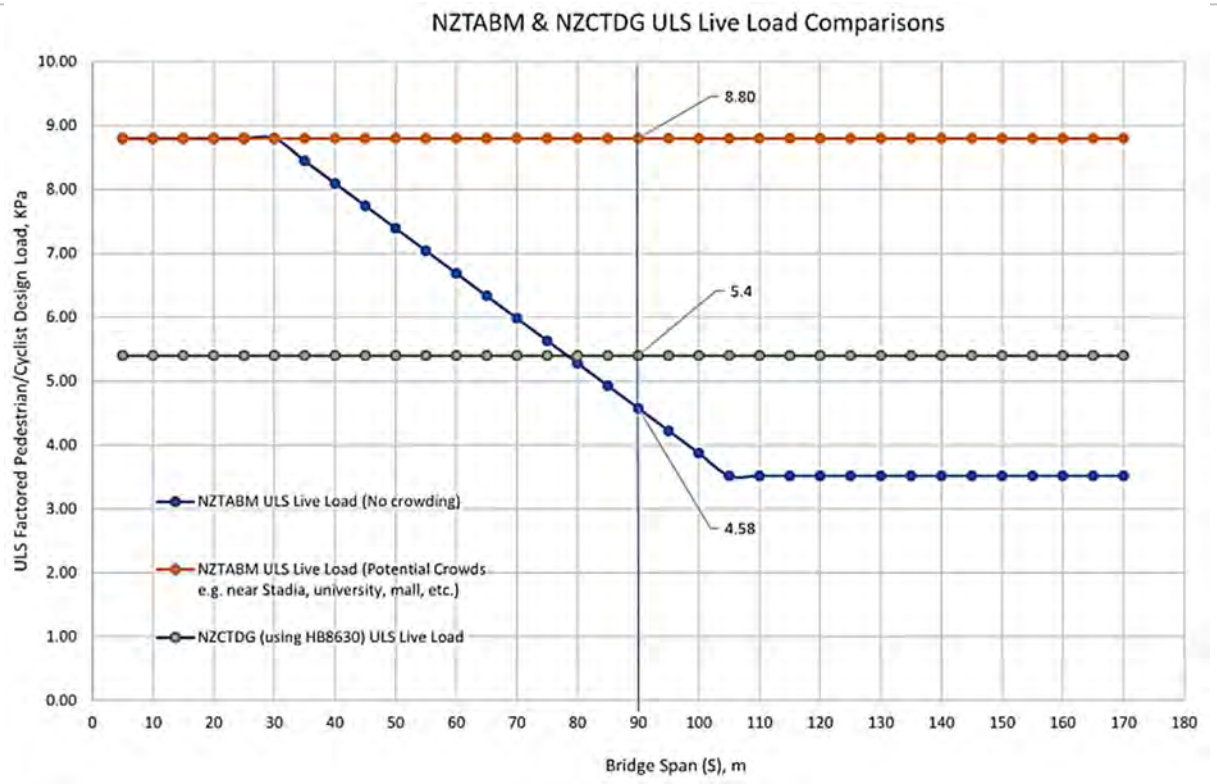


Figure 5-1 – Comparison of ULS factor live loads for a 90m span footbridge

5.4. Seismic Loads (EQ)

- Seismic design is undertaken in accordance with Section 5 of the NZTABM v3.3;
- Seismic loads have been derived in accordance with NZS 1170.5:2004 Structural design actions;
- NZS 1170.5 criteria are as follows:
  - Importance Level = 2 (footbridges);
  - SLS return period: 25 years, Rs = 0.25 (100 years design life);
  - DCLS[1] (ULS) return period: 500 years, Ru = 1.0 (100 years design life);
  - CALS[2] = 1.5*DCLS = (Ru=1.5);
  - Immediate post-earthquake function = none;
  - Zone Factor, Z = 0.18 (Gore);
  - Near fault factor, N(T,D) = 1.0;
  - Soils Category, Class = C (shallow soil);
  - Structural Performance Factor, Sp = 1.0 (no longer applicable to NZTABM v3.3);
  - Maximum allowable ductility (μ) ≤ 3.0 for piles (ductility has not be needed in the design).

5.5. Barrier Loading, Q_b

- Loads on barriers will be assessed in accordance with NZTABM;
- As per NZTABM Section B6.4 the combinations below (each row) are applied as separate load-cases.
- Demands are combined with the 0.2kN/m deadload of the barrier system (x 1.35 at ULS);

Table 7-1 Barrier Loads

Element	Direction	Imposed Load*	ULS Factor (SLS)
Top Edge and Rail	Horizontal	1.75 kN/m	1.7 (1.0)
	Vertical	1.75 kN/m	1.7 (1.0)
Barrier infill	Horizontal	1.5 KPa	1.7 (1.0)
	Point load any direction**	0.5 kN	1.7 (1.0)

* NZ Bridge Loads defined in Section B6.4 of the NZTA Bridge Manual 3rd Edition 2013.
** Infill point loads can be distributed between two infill members spaced ≤ 100mm apart.

5.6. Liquefaction Analysis

- Liquefaction does not occur at either SLS or ULS events. Refer to Section 2.11 above.

[1] DCLS = Damage Control limit State (refer NZTABM v3.3 5.1.1)
[2] CALS = Collapse Avoidance Limit State (refer NZTABM v3.3 5.1.1) commonly referred to as the maximum considered earthquake. Material strength reduction factor at CALS is taken as 1.0 (NZTABM 5.1.3).

5.8. Vehicle Live Loads, LL

- No vehicle loading is allowed for over the bridges.
- Bridges are assumed for the sole purpose of pedestrian and cycle access only;
- Bollards are incoporated to prevent unwanted vehicle access.
- The FRP decking can take a point load of up to 3KN with a 1.5 factor. This would allow for access of a 1.2 tonne 2-axle vehicle. Bridge is to be closed during access by maintenance vehicles and is therefore not considered a governing load-case (compared to the localised 5KPa pedestrian loading case).

5.9. Shrinkage and Creep, SG

- Superstructure and mast are all made of steel which experiences negligible creep and shrinkage change over time. Shrinkage and creep effects are not considered for these elements;
- As per section 3.2.1 of the PTI Stay-cable Recommendations (2012), cable relaxation and/or creep is not expected in low relaxation cables when stressed below 50% UTS for permanent SLS loads;
- The specified cables have a creep of 0.15mm/m (circa 4mm for midspan cables) when held at 45% of minimum breaking load and are supplied pre-stretched;
- Cable creep will be checked for a +40°C cable load corresponding to a 2mm elongation (50% of maximum creep) in combination with DL & SDL. This case will be checked to confirm bridge does not creep towards a sagging condition.
- Under permanent load the overall bridge is in axial compression towards the western abutment. To model this long-term effect on piles the bridge is checked for an Ec/2 permanent load condition for the western piles. The influence of this scenario on the deflected profile of the bridge has been assessed as negligible (within the +/-5KN and 5mm tolerance used for cables and steelwork deflection respectively) and thus can be ignored.

5.10. Pedestrian Dynamics

Although crowd loading is not considered in our strength checks, bridge performance will still consider large volumes of people. This will be achieved by assessing the bridge dynamically and conforming to best-practice design guidance for user comfort (we will use European design codes based on footfall synchronization and associated accelerations etc.).

- Pedestrian dynamics are designed to Design of Lightweight Footbridges for Human Induced Vibrations (JRC-ECCS, 2009);
- This design guide supplements Eurocode 2 UK Annex (BS EN-1991-2:2003 UK National Annex) which is now the standard for the UK and supersedes BD37/01 (as referenced by the current NZTABM).
- JRC-ECCS is considered directly applicable to the design of the Longford Shared Path Cyclebridge;

5.11. Earth Pressures

5.11.1. Abutment and wingwall pressures

- Soil pressures are assumed to be “at rest” for applying static loads to the abutment back wall and wingwall.
- Soil parameters are summarised in table below;
- Soil pressures are only applied to eastern abutment backwall and west wingwalls;
- Earth pressure effects applied to centroid of western abutment beam:
  From backwall and wingwalls:
  1. Active earth pressure force = 9.2kN/m
  2. Active earth pressure moment = 2.9kNm/m
  3. At-rest earth pressure force = 18.4kN/m
  4. At-rest earth pressure moment = 5.8kNm/m
- Soil pressures are not considered for the western abutment / mast foundation. Western abutment effects are in a permanent state of compression 9whjen considered for all load combinations), thus the influence of soil is modelled as a passive spring to account for superstructure movement towards the back fill. Spring is modelled as a bi-linear spring of 2340kN/m per metre length of backwall up to a maximum passive force of 89kN/m x (SRF=0.5) = 45kN/m applied at beam centroid.
- 5KPa Pedestrian live load surcharge effects are considered applied to the eastern wingwalls and backwall (noting they are governed by ULS1A combination).

Table 7-2 Recommended Earth Pressure Coefficients:

	Granular Fill
Bulk Density, γ	20 kN/m3
Friction angle θ’	35°
At rest earth pressure coefficient, Ko	0.4
Active earth pressure coefficient, Ka	0.2
Passive earth pressure coefficient Kp	5.9

5.11.2. Seismic Earth Pressures

- The incremental increase in lateral earth pressures due to seismic loading was applied in accordance with the method recommended RRU Bulletin 84, Wood & Elms (1990) for a stiff wall, as shown in the figure below. The document defines a stiff wall as that which moves outward between 0 to 0.2% of the walls height.
- Under seismic loads, the increment was then added to the active pressure;
- Earth pressure effects applied to centroid of western abutment beam (in combination with active soil effects noted above):
  From backwall and wingwalls:
  1. Seismic earth pressure force = 5.8kN/m
  2. Seismic earth pressure moment = 3.9kNm/m

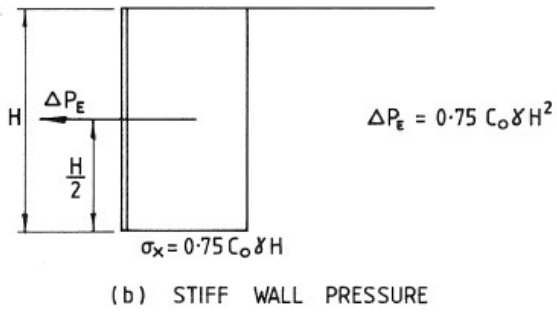


Figure 5-2 Increment in Active Earth Pressure Due to Earthquake

5.12. Temperature Effects

- Temperature effects are applied in accordance with NZTABM Section 3.4.6;
- No temperature effects are assumed in developing construction phase target deflections and forces (assumed bridge is constructed at circa 20°C);
- Steel, cable, and concrete coefficient of thermal expansion is modelled as 12x10-6.

5.12.1. Global Temperature TP-G(-) & TP-G(+)

- A global temperature change of +/- 25°C is applied to the SAP model;
- Applied to all above ground elements;
- Cables are also checked for a +/-10°C degrees;

5.12.2. Differential Temperature – TP-DIF

- NZTABM Figure 3.3 Temperature differential is applied assumed to top and bottom of steel flanges (timber ignored):
  - Top of flange, T = 32°C
  - Soffit = 1.5°C
- Resultant effects from restraint are modelled as an equivalent:
  - Temperature gradient = 22.7°C/m;
  - Average temperature increase = + 10.0°C;
- Unrestrained stresses are manually added to the resultant primary beam design stresses:
  - Unrestrained top flange stress = 20MPa (6% of θfy);
  - Unrestrained bottom flange stress = 12MPa (4% of θfy).

5.13. Wd, Wind Loads and dynamics

- The design gust wind speeds acting on the bridge are derived from section 3.4.5 of the NZTA Bridge Manual, AS / NZS 1170 and BD37;
  - NZS1170.2 SLS (1/25) and ULS (1/500) wind speeds = 35.9 and 43.7m/s respectively;
  - When considering wind + live load effects (and allowing for an additional 1.25m height) the wind speed is limited to 22m/s (80 km/hr) for ULS and SLS events;
- Solid balustrade height taken as 1.4m x 30% opacity = 0.42m;
- Wind applied to superstructure is based on height above ground = 7m and Cd for cyclebridge = 2.0;
  - SLS load applied to leading beam = 2.61kN/m (1.58KPa) without live load;

  - ULS load applied to leading beam = 3.87kN/m (2.34KPa) without live load;
  - SLS & ULS load applied to leading beam = 1.20kN/m (0.59KPa) with live load;
- When combined with flooding debris loads (NZTABM Combination ULS3B) height of superstructure above water is taken as 1.5m (@ midspan). This translates to a 15% reduction in normally applied wind demands. Thus 0.85WD is used in combination with flooding.
- Wind applied to mast is based on height above ground = 27m and Cd of 0.6 (from BD37 Table 9):
  - SLS load applied to mast = 0.53kN/m without live load;
  - ULS load applied to mast = 0.78kN/m without live load;
  - SLS & ULS load applied to mast = 0.16kN/m with live load;
- Wind loads applied to cables are applied using PTI Section 5.1.5 recommended values Cd=1.2, turbulence gust factor (gb)=2.0, and dynamic response factor (gr)=1.1:
  - Assumed SLS wind applied to all cables simultaneously = 0.05kN/m (26mm), 0.07kN/m (35mm);
  - Assumed ULS applied to all cables simultaneously = 0.08kN/m (26mm), 0.11kN/m (35mm);
  - Assumed ULS and SLS load applied to cable with full pedestrian loads = 0.02kN/m (26mm), 0.03kN/m (35mm) (limited to 80km/hr)
- Wind vibration / aerodynamic effects on the bridge are assessed using BD 49/01 “Design Rules for Aerodynamic Effects on Bridges” (refer to Section 9 of this report for detailed discussion about the aerodynamic assessment);
- Vertical vortex loading is considered as a static loading (to model dynamic effects) acting on the superstructure based on BD49/01 Table 1. Since the estimated Kd is 5 mm/s2 vertical vortex demands of circa 4%*(DL+SDL) can be ignored (as suggested by BD49/01 Table 1);
- Assessment of cable vibration due to wind will be made using current best practise. The critical wind speed (“onset” wind speed) for cable galloping and Scruton Number for assessing rain-wind induced vibrations are derived in accordance with the *Recommendations for Stay Cable Design, Testing, and Installation (Post Tensioning Institute, 2012).* The PTI Guide is applied using design parameters suggested in the corresponding commentary sections:
  - PTI Guide Section 5.2.3.2 *Rain-wind induced vibrations*;
  - PTI Guide Section 5.2.3.3 *Cable galloping*;

5.14. Hydrology and Floods

5.14.1. General

- Basis for design to be NZTABM Section 2.3 *Waterway Design with reference to AS5100.2-2017*;
- Riley have performed hydrological assessment as per the ***Longford Bridge - Geotechnical and Hydraulic Detailed Design Report*** (Riley, April 2020);
- As per AS5100.2 2004 cl 15.5.3 debris forces will only be applied to superstructure where soffit level is less than 1/500 year event (ULS) + 600mm = 77.3m RL.
- Some parts of the superstructure are not 600mm above the ULS flooding event. Thus a debris impact load is applied to any stretch of the superstructure that is below the 600mm ULS debris threshold;

5.14.2. Flood Debris Loads

- Superstructure flood demands are governed by the ULS debris demands. These have been determined in accordance with AS5100.2-2017 Section 16.6.4 as summarised in Table 7-3 below;

- As per AS5100.2-2017 all superstructures will be designed for the “upstream flood velocities as measured at the surface”. The 2D hydrological modelling is assumed to output river velocities “averaged over the depth of flow”. It is assumed for design purposes that the “surface velocity” is equal to the average and indicated in the Figure below. This terminology is as per AS5100.2-2017 cl16.2.
- Flood velocities vary across the bridge long-section as indicated in Figure 9 of Riley Report (an extract shown below). These velocities have been used as the upstream velocities for calculating debris on the eastern regions (largely unaffected by bridge). The pre-construction velocities have been provided by Riley to be used for the western region (as these are slightly higher and represent the likely up-stream velocities without the presence of the embankment). These velocities are show in the table below.
- Regardless of the depth of the flood relative to the structural depth, all effects have been conservatively applied directly to steel girder bottom flanges (noting this is the weakest region of the superstructure);
- Ultimate limit state (ULS) flood effects have not been considered in combination with any live loading. At the SLS and ULS event the western approach paths will be significantly inundated and it is assumed that either A) people will naturally avoid using the bridge or B) Council close the bridge during large flood events;
- ULS debris loads have been considered in combination with 85% of the ULS wind loading. The 15% reduction reflects the reduction in height from Z=7.0 to 1.5m above water:
  - ULS3B = 1.1DL + 1.1SDL + FW_{ULS} + 0.85WD_{ULS}

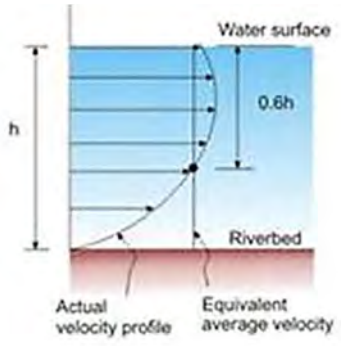


Figure 5-3: Vertical velocity distribution and average velocity for flow in river

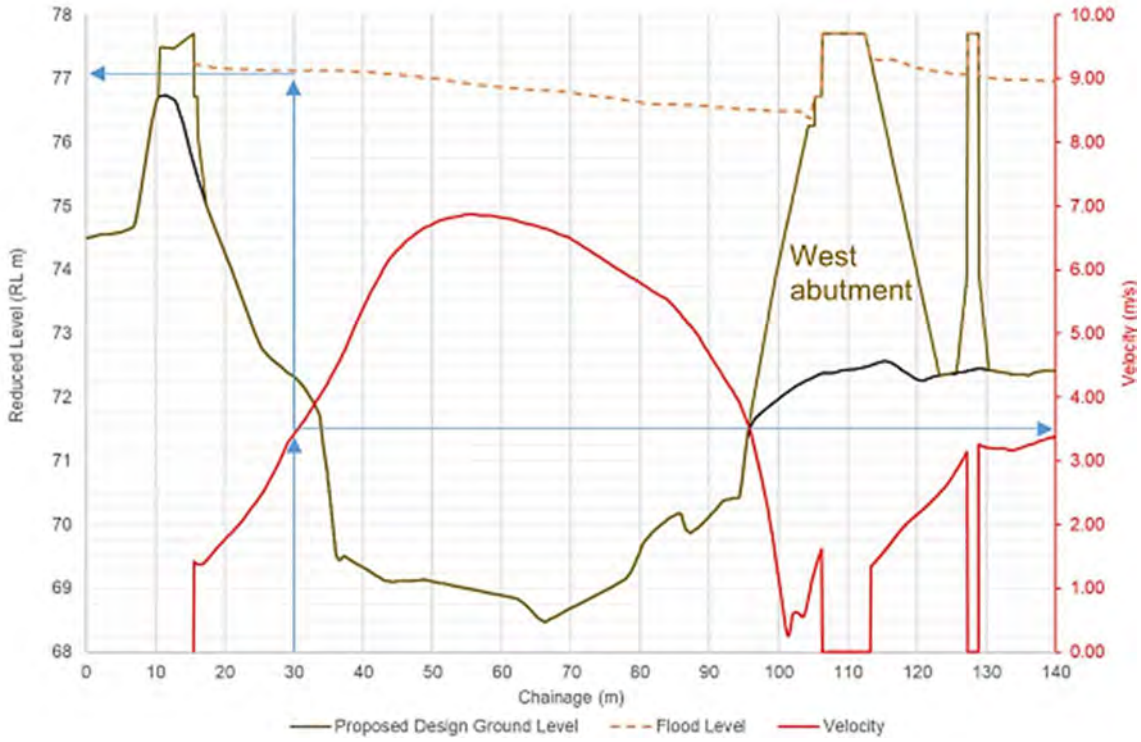


Figure 5-4: Extract from Riley Report – "*Figure 9 Predicted Flood Levels and Velocities for ULS Event*"

Table 5-1: Estimated Flood Debris Forces acting on superstructure

16.6 Forces on Superstructures due to Debris

	E-Abut +3m	E-Abut +9m	E-Abut +15m	W-abutment -6m	W-abutment	
SPAN LENGTH, L =	90	90	90	90	90	m
SPAN WIDTH, B =	4.1	4.1	4.1	4.1	4.1	m
Girder Depth, Dg =	1.235	1.235	1.235	1.235	1.235	m
Slab depth, Ds =	0	0	0	0	0	m
Upstream Solid Barrier Depth, Dbs =	0	0	0	0	0	m
Upstream Open Barrier Depth, Dbo =	0	0	0	0	0	m
TOTAL SUPER DEPTH, D =	1.235	1.235	1.235	1.235	1.235	m
Superstructure design Velocity = 1.4*Upstream average = Vsuper	1.52	2.34	3.45	3.89	1.2	m/s
Upstream ULS Flood Level, FL =	76.7	76.7	76.7	76.7	76.7	mRL
Bridge Soffit level, SL =	76.74	77.04	77.3	77.26	77	mRL
Bridge Barrier Top level, BL =	77.975	78.275	78.535	78.495	78.235	mRL
Bridge Barrier (solid) Top level, BLs =	77.975	78.275	78.535	78.495	78.235	mRL
Assumed Average Bed Level, BedL =	74.662	72.993	72.304	75.119	75.9	mRL
Vertical Flood distance, dwgs = FL-SL =	-0.04	-0.34	-0.6	-0.56	-0.3	m
Total Submerged (wetted) depth = MIN(FL-SL,BL-SL) = dsp	-0.04	-0.34	-0.6	-0.56	-0.3	m
Total SOLID Submerged (wetted) depth = MIN(FL-SL,BLs-SL) = dss	-0.04	-0.34	-0.6	-0.56	-0.3	m
Fig 16.5.2 Submergence =	PARTIALLY	PARTIALLY	PARTIALLY	PARTIALLY	PARTIALLY	"FULLY" submerged or "PARTIALLY" submerged or ">0.6m" clear
Average vertical distance from girder to bed = ygs = SL-BedL =	2.078	4.047	4.996	2.141	1.1	m
net wetted area of bridge span, As = dsp x L =	-3.6	-30.6	-54	-50.4	-27	m2, per span

Fd = 0.5*Cd*V^2*As
units of m^4/s^2
Force based on F = ma = kg.m/S^2
Drag force based on m3 of water weighs 1000kg.

Thus keep V and As in m units (not mm)
1m3water x m/s^2 = 1000kg.m/s^2 = 1000N = 1kN
Thus resultant value is in kN.

16.6.4 Debris Load

Average flow depth, y =FL-BedL =	2.04	3.71	4.40	1.58	0.80	
Debris Depth, Ddeb =	1.2	1.9	2.2	1.2	1.2	m (1/2 water depth, 3m max, 1.2m min)
Area of debris, Adeb =	108.0	166.8	197.8	108.0	108.0	
F = Vsuper/((g*y)^(0.5)) =	0.34	0.39	0.53	0.99	0.43	
Proximity Ratio, Pr = ygs/dss =	-147.50	-17.35	-9.83	-10.54	-19.67	
Proximity Ratio, Pr(debris) = ygs/(dss+0.6) =	3.71	10.00	10.00	10.00	3.67	< use this value if normal proximity value is zero
Debris Coefficient, Cddebris =	1.8	1.4	1.2	0.8	1.6	< From Fig 16.5.2(A)&(B)
Debris Pressure on Super = 0.5*Cd*(Vsuper)^2 =	2.1	3.8	7.1	6.1	1.2	Kpa
Debris Force on Super = 0.5*Cd*(Vsuper)^2*As =	225	639	1,413	654	124	kN (see note above)
Debris Force on Super = 0.5*Cd*(Vsuper)^2*As =	2.5	7.1	15.7	7.3	1.4	kN/m

5.15. Fatigue

5.15.1. Live load fatigue

- Fatigue issues are not expected within the primary steel superstructure and/or the mast due small number of cycles for pedestrian bridges

5.15.2. Wind fatigue

- Fatigue effects are considered in the cable stays, anchors, mast, and superstructure, and are based on stress ranges induced by vertical vortex excitation as defined by section 5 of BD49/01 (refer to Section 7 of this report for fatigue assessment outcomes);
- Fatigue life of a spiral strand cable is discussed in Section 6;

5.16. Loss of cable case

- Allowance for single cable damage case considered based on the load combination recommended by Section 5.5 of the PTI stay-cable design recommendations (2012):
  - Cable Failure Load-case = 1.1DL + 1.35SDL + 0.75FP(@2.6KPa) + 1.1CLDF;
  - CLDF is the cable loss dynamic force = 1.5 x cable tension force prior to failure;
  - CLDF is applied as a vertical and horizontal point load of equivalent force and location as the failed cable (based on load in the failed cable from failure load-case);
  - Strength reduction factor applied to cables in a cable loss case is θ = 0.95 (and is considered as a ULS load-case);
- Direction of dynamic failure force CLDF is taken as:
  - Downward at superstructure cleats (reversal of stay tension load);
  - Toward mast at top (reversal of stay tension load)
  - Upward at backstay assembly (stay tension load);
- No live load assumed during cable replacement. This is a short-term maintenance case and handling large cables is safer without public access (existing alternative routes can be used alongside existing road bridge until replacement is completed).

5.17. Load Combinations

- Load factors and combinations are in accordance with NZTABM Table 3.1 (SLS) and Table 3.2 (ULS).

5.18. Deflection limits

- Bridge Beams are assumed precambered to increase flood clearance;
- Vertical Deflection of Superstructure ≤ Span / 200 for cyclist live load[3];
- Live Load Longitudinal deflection of the Mast ≤ Height / 150.

[3] Deflection ≤ span / 200 for cyclist live load has been taken from NZ Cycle Trail Design Guide v4 which is supported by NZTA for the design of bridges not associated with roads (neither NZTABM or AS5100.2 give guidance on pedestrian deflection). This guide references SNZ HB 8630:2004 “Tracks and Outdoor Visitor Structures” of which section 3.3.2 gives a deflection limit of span/200 (450mm for Longford Bridge).

6. Material Properties

6.1. Concrete

Concrete strengths, exposure classifications and cover requirements are set out below. Covers have been specified in accordance with NZS 3101:2006 Table 3.7, to achieve the 100-year design life. Note, the covers outlined below are the minimum covers required by NZS3101, in order to ensure that these covers are achieved nominal covers inclusive of an additional +5mm will be specified on the drawings.

Table 6.1: Exposure Classification, Concrete Grade and Cover

Location	Exposure Classification	Concrete Grade	Min Required Cover
Reinforced concrete substructure	A2 (inland)	40 MPa	35 mm

The modulus of elasticity, Ec, for the various grades have been evaluated following the provisions of NZS 3101:2006 clause 5.2.3, for normal weight concrete. These are shown in Table 6.2:

Table 6.2: Concrete Modulus

Concrete Strength	Young's Modulus, E_c
30 MPa	25,080 MPa
40 MPa	27,900 MPa

Concrete stiffness' for deep abutment beams will be based on the gross section.

6.2. Reinforcing Steel

All reinforcement will be Grade 500E (fy=500MPa), and will comply with the requirements outlined in AS/NZS 4671:2001.

6.3. Timber

Timber properties and design principles will be based on AS 1720.1—2010 Timber structures Part 1 and Part 2. All timber will be treated to H3.2 unless it is in contact with ground where H4 will be used.

Glulaminated beams:

- Glulam beams grade GL10;

- Density = 600 kg/m^3
- Modulus of elasticity = 10,000 MPa for GL10 from AS/NZS 1328.2 table 1.2
- Characteristic bending = 22 MPa for grade GL10 from AS/NZS 1328.2 table 1.2
- Characteristic shear = 3.7 MPa for grade GL10 from AS/NZS 1328.2 table 1.2
- Creep factor for glulam = 150% from NZS3603 8.7.4
- K1 = 0.8 (medium term loading for dead + live);
- K3, k4, k5 taken as 1.0;
- K6 and K8 vary in accordance with NZS3603 where K6 is based on 1/4D and K8 is based on the 3000mm distance between fastening cleats.

6.4. Structural Steel

- All structural steelwork will comply with AS/NZS 3678 and will be designed in accordance with AS5100.6.
- Structural steelwork grades for the fabricated beam shall be as follows:
  1. Fabricated plate <12mm fy = 360 MPa
  2. Fabricated plate 12mm < t ≤ 20mm fy = 350 MPa
  3. Fabricated plate 20mm < t ≤ 80mm fy = 340 MPa
  4. Rolled UB and UC sections fy = 300 MPa
  5. Circular hollow sections fy = 350 MPa

6.5. Structural steelwork and Cables corrosion protection

A C2 corrosion category has been identified as presented in the SNZ TS 3404:2018 for Gore. This is a corrosion rate of 1.3 to 25 microns of steel per year.

- Time to first maintenance = 40 years;
- Atmospheric Corrosivity Category[4]:
  - C2 where washed by rain
  - C3 where sheltered by other parts of structure.

Proposed systems in accordance with Protective Coatings for Steel Bridges (NZTA, 2017):

- Mast (assume “exposed” = C2)
  - Polyurethane paint system in white with high gloss anti-graffiti top-coat (PUR5);
  - Or other systems (as agreed with the Engineer) to achieve 40 years in C2 in accordance with AS/NZS2312;
- Superstructure (assumed “Sheltered” between primary beam webs):
  - Primary beam and cross beams: 100 micron thermal zinc spray (TSZ100);
  - Visible faces of beams (outside web and flange faces): 100 micron thermal zinc spray + 75 microns of high gloss polyurethane (TSZ100s + 75µPU);
  - Bracing: 600g/m^2 zinc hot dipped galvanizing (HDG600) or TSZ100;
  - Or other systems (as agreed with the Engineer) to achieve 40 years in C3 in accordance with AS/NZS231;
- Exposed Cables:
  - 180g/m^2 of 95% zinc + 5% aluminium coating (AKA galfan^{tm} or bezinel^{tm} systems);
  - This system has a design life of 100 years in a C2 corrosion zone;
  - Cables are considered as an exposed area and washed by rain to reduce salt deposits.

[4] AS/NZS2312.1

- All cable components (e.g. spelter, turnbuckles, etc.) To be hot dipped galvanized to 600g/m2 (or a minimum 80 microns of zinc).

6.6. Cable Stays (and bars)

- Spiral strand cable (1620MPa minimum specified Ultimate Tensile Strength UTS) used for main and back-stays. Spiral strand cables are supplied pre-stretched;
- Williams Form 150KSI bar supplied to ASTM A722 (1035MPa minimum specified Ultimate Tensile Strength UTS) used for anchoring. Anchors are locked-off at factored design load;
- Stay design is as per Post Tensioning Institute (PTI) ***Recommendations for Stay-Cable Design, Installation and Testing***,6th edition 2012 with Addendum #1 Nov 2014 and Figure C3.1 errata Jan 2014;
- Cables are governed by the NZTABM SLS1A and ULS1A cases. These are considered similar to AASHTO Table 3.4.1-1 “Strength I” load-case combinations and as such resistance factors (AKA strength reduction factors) considered in the PTI guide section 5.3.3 are assumed equally applicable to NZTABM and NZ conditions:
  1. θ = 0.65 for ULS1A, ULS1B, ULS2A and ULS2B
  2. θ = 0.55 for SLS cases (based on Yield = 0.85*UTS as per PTI 3.2.1.1 & 3.2.4.1)
  3. θ = 0.95 for ULS3A (seismic)
  4. θ = 0.95 for ULS cable loss case
- As per PTI 5.3.5 the unfactored fatigue demand is factored by 1.4 * 0.75 = 1.05;
- Fatigue testing of cables and bars is based on Table 3.1 and Figure C3.1 of PTI Recommendations (Nov 2014 Addendum #1) for strand assuming fatigue test stressed to 0.55f’s with a quality assurance Δt of 69MPa.:
  - at $1x10^5$ cycles taken as 405 MPa;
  - at $5x10^5$ cycles taken as 264 MPa;
  - at $2x10^6$ cycles taken as 190 MPa;
    at $>2x10^6$ cycles taken as 175 MPa;
- As per PTI 5.3.5 the unfactored fatigue demand is factored by 1.4 * 0.75 = 1.05;
- Fatigue threshold $(\Delta F)_n$ of spiral strand cable and high strength bars is taken at 110MPa as per PTI Table 5.1. Fatigue demands below $G = (\Delta F)_n / 2 = 55$MPa are considered to have an infinite fatigue life as per PTI 5.3.5 and can be ignored.

Table 6.3: Properties of Spiral Cable

	21.9mm 1 x 37	26.4mm 1 x 37	30.9 1 x 61		35.4 1 x 61	
Cable Breaking Load (Ultimate Tensile Strength) (kN)	430	626	859		1129	
End termination	Swaged (0.9MBL)	Swaged (0.9MBL)	Swaged (0.9MBL)	Cast (1.0MBL)	Swaged (0.9MBL)	Cast (1.0MBL)
System Breaking Load (Ultimate Tensile Strength) (kN)	430	626	775	859	1015	1129
Strength assumed at ULS (kN)	250	365	505	560	660	735
Strength assumed at SLS (kN)	215	300	425	470	560	620
Strength at Seismic and Cable Loss ULS (kN)	350	505	695	775	915	1015
Nominal metallic cross section (mm^2)	288	419	575	575	753	753
Mass (kg/m)	2.3	3.3	4.5	4.5	5.9	5.9
Elastic Modulus of spiral cable (MPa)	160,000 (+/-10,000)					

Table 6.4: Properties of Threaded Bar

	26mm	36mm	46mm
System	Williams Form 150 KSI		
Bar Breaking Load (Ultimate Tensile Strength) (kN)	567	1054	1734
Strength assumed at ULS (kN)	370	685	1125
Strength assumed at SLS (kN)	310	580	955
Strength at Seismic and Cable Loss ULS (kN)	540	1000	1645
Nominal metallic cross section (mm^2)	549	1019	1664
Mass (kg/m)	4.6	8.5	13.5
Min hole to pass bar (mm)	32	43	54
Elastic Modulus of spiral cable (MPa)	200,000	200,000	200,000

7. Design Summary

7.1. Modelling Methodology

Structural modelling is on the basis of a 3D analysis model created in SAP2000 v20. Steel and concrete elements have been modelled as “frame” elements and stay-cables have been modelled as pre-tensioned “cable” elements with catenary action. Structure-soil interaction effects on piles are considered by using bi-linear winkler springs. See Figure 7-1 below.

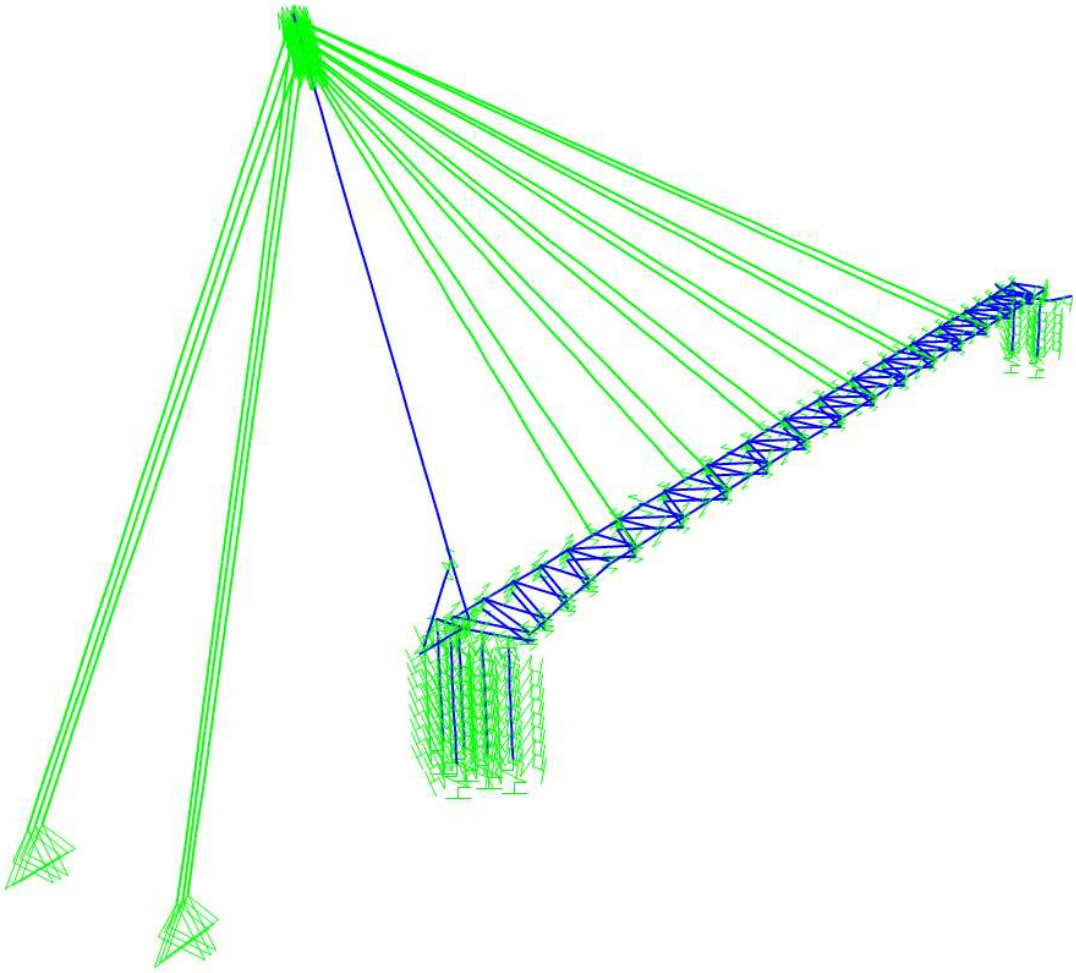


Figure 7-1 3D Analysis Model

Cable stresses have been optimized to achieve a balance between achieving the SLS and ULS cable limits contained in Section 6 when considering all NZTABM load combinations, whilst also targeting a smooth vertical curve in the permanent load case (without live load). All loads are applied to the model as non-linear loads in order to enable the cables forces and displacements to respond to the influence of increasing load.

Three soil models were created using the spring data in the Interpretive Geotechnical Report:

- "Soft Model" = 33% of average spring values;
- "Average model" = 100% of average soil spring values;
- "Hard Model" = 300% of average spring values;

The Hard Model and Soft Model were used to bound the assessment for ultimate limit state structural demands on superstructure steelwork, mast steelwork, piles, abutment beams, and anchorage during static load-cases. The range of variation accounts for the uncertainty in estimating soil behaviour.

The average model was developed for assessing and predicting displacement behaviour including estimates for displacement during the stressing sequence of construction. It is also used for dynamic analysis (wind and pedestrians).

For closely spaced piles, the influence of pile shielding needed to be considered when using the winkler springs. The shielding effect is considered when a trailing pile (considered in the orthogonal direction of the applied force) is too close to a leading pile to develop full resistance. This pile group influence was allowed for by reducing trailing pile springs by a factor of between 25% to 100% for a spacing of 3 x pile width to 8 x pile width respectively. Results were interpolated between these values. Because of wide pile spacings no pile shielding is applied at the East Abutment.

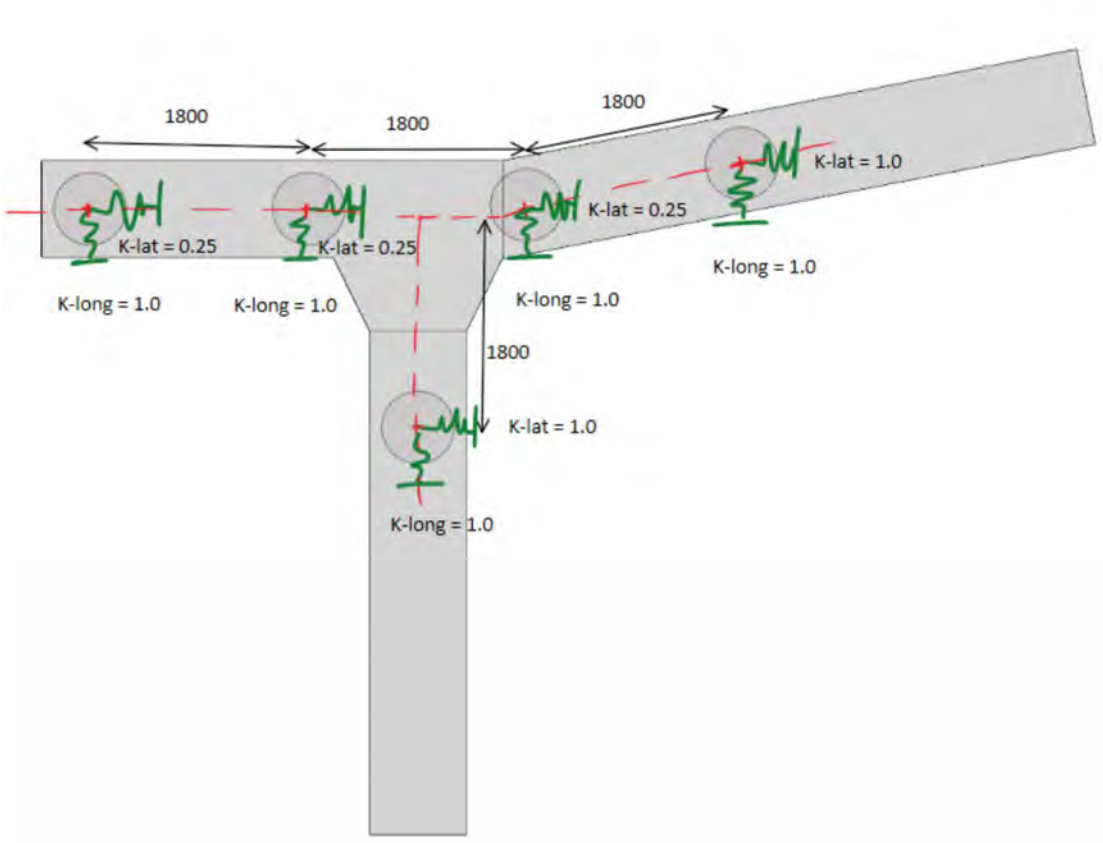


Figure 7-2 West abutment spring reduction factors

Where possible the modelling was run as non-linear load-cases. The only exception to this rule was the “modal” case as used for seismic, wind dynamics and pedestrian dynamics. The “modal” loadcase is run based on the stiffness at the end of the non-linear full permanent load case. This enables the modal case to consider the deflection behaviour and load take-up expected in the structure of the bridge when assessing dynamic performance.

Once the final condition was set, the structure was de-constructed in reverse order so that the staged stressing forces and anticipated deflection for construction could be determined. Since there are no creep/shrinkage related components (mast, super, cables are all steel) this reverse method is assumed adequate for design purposes. Outcomes of the staged construction analysis are contained on drawing S132 and these will be further developed in close coordination with the Constructor and the Constructor’s temporary works designer during the subsequent construction phases. Stressing forces and/or estimated deflections will be recalibrated/updated once initial measurement data becomes available (e.g. after stressing the first cable sets).

Table 7-1 Loadcase Modelling Approach

Load Type	Modelling Approach
Dead Load (DL)	Non-Linear
Super Imposed Dead Load (SDL)	Non-Linear
Wind Loading (WD)	Non-Linear
Live Loading (FP)	Non-Linear
Temperature effects (TP)	Non-Linear
Pedestrian Dynamics	Non-Linear time History
Wind Vortex Excitations	Non-Linear time History
Modal (for seismic, pedestrian and wind dynamics)	Linear Modal Analysis
Seismic Loads (EQ)	Linear Modal Response Spectrum Analysis
Construction Stages (DL, SDL, CN)	Non-Linear

Damping of the structure is modelled as percentage of critical damping. Values used are as per the table below:

Table 7-2 Damping

	Log Decriment δd	Damping Ratio (AKA "% of critical damping" as used in SAP2000 modelling) ξ	Reference
Seismic – vertical bending		2.50%	NZS3404 Table 12.2.9
Seismic – transverse bending		6%	NZS3404 Table 12.2.9
Wind – vertical bending	0.02	(0.3%)	BD49 Cl 3.1.2
Wind - torsion	(0.04)	0.5%	
Pedestrian Dynamics - vertical	(0.03)	0.4%	ECCS Table 4-5
Pedestrian Dynamics - lateral	(0.04)	0.5%	

TABLE 7-2 NOTES:

1. A damping ratio of 0.04 has been taken where lateral and/or torsion are based on bolted steel connections. A lesser amount is taken for vertical effects based on using continuous welded beams with friction bolt splices. This approach is consistent with NZS3404 Table 12.2.9.

2. For assessing wind the vertical damping of cable supported structures have been factored by 0.75 as per BD49/01 cl 3.1.2.

7.2. Stressing and Erection Analysis

Once the final condition was set, the structure's analysis model was de-constructed in reverse order so that the staged stressing forces and anticipated deflection for construction could be determined. Since there are no significant creep/shrinkage related components (mast, super, and cables are all steel) this reverse method is assumed adequate for design purposes.

Outcomes of the staged construction analysis are contained on drawing S132 and these will be further developed in close coordination with the Constructor and the Constructor's temporary works designer during the subsequent construction phases. Stressing forces and/or estimated deflections will be recalibrated/updated once initial measurement data becomes available (e.g. after stressing the first cable sets).

7.3. Seismic Assessment

Transverse seismic forces are assumed transferred by the plan cross bracing and taken at both abutments transferred by the hold down bolts. In the longitudinal direction the east abutment is free to move because of the slotted hold down bolts.

Modal Response Spectrum Analysis Seismic design is run in SAP2000 v22 to NZS1170.5. Structural elements were found suitable for designing to elastic demands for both bending and shear in accordance with NZTABM Section 5. No over-strength or pushover design methods were required.

As expected, the cable stays significantly improve the vertical performance of the structure during a seismic event. The vertical seismic is not a governing case as it is applied without pedestrian live load. On this basis, the linear behaviour of cables assumed in the modal response load-case is not considered a significant issue.

7.4. Pedestrian Vibration Assessment

The Pedestrian dynamics assessment for the bridge in contained in Appendix B. The bridge is considered adequate for both vertical and transverse pedestrian excitation effects.

7.5. Aerodynamic and Wind Performance

Wind effects on the bridge have been considered in accordance with BD49/01 ***Design Rules for Aerodynamic effects on Bridges*** as referenced by the NZTABM v3.

7.5.1. Outcomes of BD49/01 Aerodynamic Assessment

Table 7-3 below shows the key outputs for the various aerodynamics effects when considered against the BD49/01 criteria as supported by the PTI Stay Cable Guide for cable effects.

As can be observed from the summary table:

- Typical “aerodynamic susceptibility” applies to the bridge;
- Torsional vortex excitations do not apply;
- Stay cables and associated turbulence therein are within acceptable limits;
- Galloping effects are not expected to occur;
- Classical flutter will not occur within the ULS return period.

However, the assessment to BD49/01 has determined that further study is required to:

- Confirm loads (including fatigue) and comfort levels on the bridge due to vortex excitations;
- Determine if (non -oscillating) torsional divergence is an issue.

Table 7-3 Aerodynamic Assessment Summary

DB49/01 Section	Description	Governing Equation	Outcome	Assessment outcome
2.1	Aerodynamic Susceptibility	$P_b = \left(\frac{\rho b^2}{m}\right)\left(\frac{16V_r^2}{bLf_B^2}\right)$	Pb = 0.97 Vr = 27m/s	Classified as a “Category B” structure and therefore BD49/01 applies.
2.1.1	Limited Amplitude Response – Vortex Excitation	fB > 5Hz or Bending VcrB = 11.32 fB d4 #	fB = 0.88Hz d4 = 1.235m VcrB = 7.06m/s	Limited criteria not met for simple check. BD49/01 cl 3.1 vortex excitation considered.

		(Torsion VcrT = 11.4 fT d4) # < 1.25 x Vr	(VcrT = 31.0 m/s) 1.25Vr = 34 m/s	⇩
3.1	Limited Amplitude Response – Vortex Excitation	Sensitivity Factor kD = Ymax fB^2 < 30m/s^2 Ymax vertical = (c*b^0.5*d4^2.5*ρ)/(4*m*δ) Ymax torsion ignored for Type 6 structures	Ymax vertical = 60mm fB = 0.88Hz kD = 46mm/s^2	Kd > 30mm/s^2 Comfort levels are potentially an issue. Detailed Wind Analysis required.
2.1.2	Limited Amplitude response - Turbulence	fB > 1Hz & fT > 1Hz or PT < 1 $P_T = \left(\frac{\rho b^2}{m}\right)\left(\frac{V_s}{f_B b}\right)^2 \frac{\sigma_{flm}.b}{\sigma_c}$	fB = 0.88Hz fT = 2.2Hz Cable PT > 1.0 (Cable #4 PT = 6.9)	Vertical bending frequency is less than 1Hz. Cables are potentially susceptible to wind oscillations. Assess to PTI cable guidelines. ⇩
PTI Guidelines Cl 5.2.3.2	Rain-wind induced vibrations	Sc > 10 $Sc = \frac{m\xi}{\rho D^2} \geq 10$	Sc = 20	Cables not subject to rain-wind oscillations.
PTI Guidelines Cl 5.2.3.3	Cable Galloping	V_{ONSET} < VULS $V_{onset} = U_R fD,\ U_R = \frac{8\pi S_c}{-\left(\frac{dC_L}{d\alpha} + C_D\right)}$	VULS = 44m/s V_{ONSET} = 3225m/s (worsecase at Cable #6)	Cables not subject to galloping.
2.1.3 a)	Divergent Amplitude Response – Galloping (AKA 1D-vertical flutter)	Only required for Type 3, 3A, 4 and 4A structures.	Type 6 structure	No vertical galloping issues for a Type 6 Structure.
2.1.3 b)	Divergent Amplitude Response – Stall Flutter (AKA 1D torsional divergence)	Vg = 3.3 fT b ≥ V_{ULS}	Vg = 30m/s V_{ULS} = 44 m/s	BD49/01 checks for torsional divergence have not been satisfied. Detailed Wind Analysis required.

2.1.3.3	Divergent Amplitude Response – Classical Flutter	Vf = Vrf fT b > V_{ULS} #[5] $V_{Rf} = 1.8\left(1\text{-}1.1\left(\frac{f_B}{f_T}\right)^2\right)^{½}\left(\frac{mr}{\rho b^3}\right)^{⅓}$ but not less than 2.5.	Vf = 57 m/s V_{ULS} = 44 m/s	Classical flutter is not an issue.

7.5.2. Aerodynamic CFD Study

To close-out the further work highlighted by the BD49/01 checks, an independent Wind Study was performed. The work was performed by independent aerodynamic specialists ***Ingenium*** (UK). Refer to Appendix C for full report associated with the study.

Key conclusions of the report were as follows:

No aerodynamic instability has been discovered; the results show:

- *Small vortex pairs form on the trailing corners of the bridge, rather than the large alternating pattern of a Von Karman vortex, which both reduces oscillatory loads and total drag force on the bridge.*
- *Bridge 2 [chosen cross section] features gaps between the deck and beams to allow flow between and equalise the pressure. This does reduce the vertical force relative to bridge 1 [cross section without gaps or pipe], with a small improvement in the oscillatory components and bridge stability. It also increases the local flow acceleration around the bridge such that users will experience a 4% higher wind speed.*
- *A sweep of incidence for bridge 2 shows no divergence with torsional deflection, and up to ±2.5deg there is a negative feedback restoring torque. At large incidences the flow system changes as the wake encompasses one of the shedding edges, and while this makes the load oscillations larger the stability margin is still high.*
- *The primary lateral and torsional modes of the bridge are insensitive to the input frequency, however the vertical mode does become less stable close to its natural frequency of 0.88Hz with the worst-case at 9.3m/s causing a limit cycle oscillation of ±60mm. Higher speeds become increasing stable.*

[5] BD37 5.3.2.2.1 is limited to a return period of 120 years in calculating the maximum gust speed. Thus, VWO is needed to factor this to an ultimate limit. Since NZTABM has a V_{ULS} for a 1/500 year return period wind already considered we will adopt this as our credible upper limit.

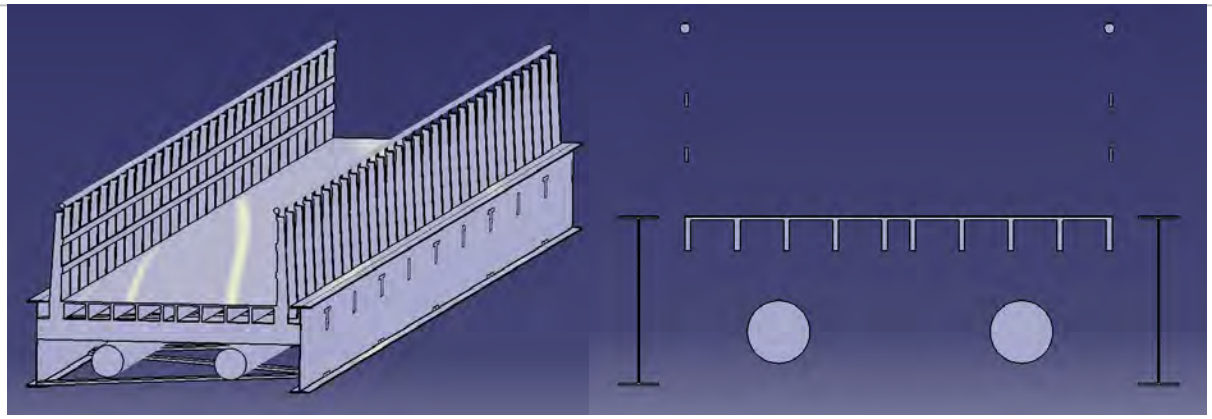


Figure 7-3 CFD Modelling of the Longford Cross Section (in Openfoam software)

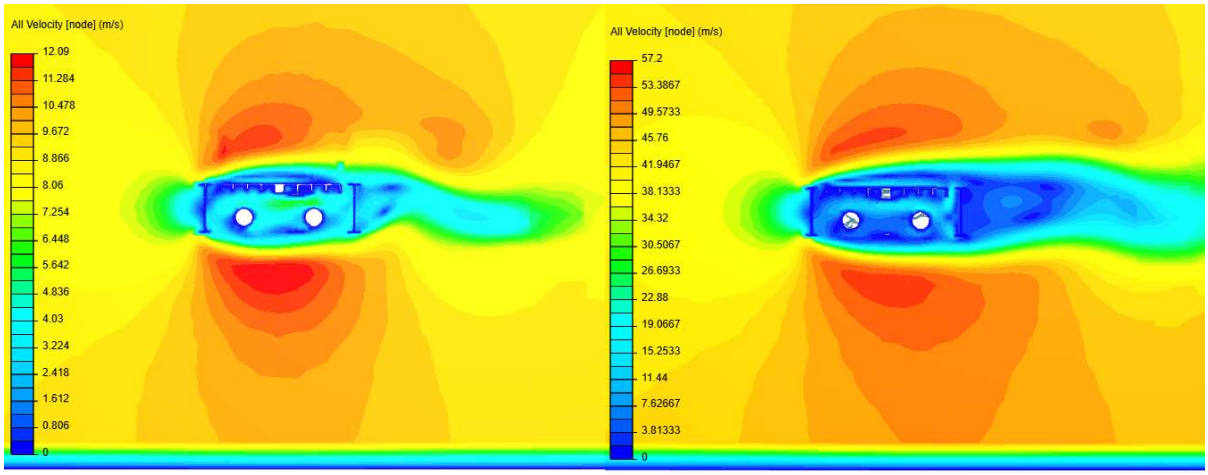


Figure 7-4 CFD Modelling of the Longford Cross Section (left) 9.3m/s onset of vortex excitation (right) 44m/s ULS event

7.5.3. Aerodynamic Vortex Excitation Discussion

Critical wind speed for onset of vertical vortex excitation based on the BD49/01 section 2.1.1 is estimated to occur at circa 12.3m/s. However, the detailed work performed during the CFD modelling confirmed that 9.3m/s is the wind velocity where vortices are created at a frequency matching the natural 0.88Hz vertical bending frequency of the bridge. We have progressed the work based on the 9.3m/s velocity.

Using a simple spring analogy the Wind Study estimated a maximum oscillation amplitude of Ymax = 60mm. On this basis, comfort issues might be experienced by pedestrians (when considered against Table 1 of BD49/01). As per the recommendation of the study, this 60mm was verified by applying the oscillating load direct to the SAP2000 analysis model. The oscillating uplift load estimated by CFD is +/- 12.8N/m (refer Figure below) and this was applied as a co-sine non-linear time-history function. This detailed analysis confirmed a peak deflection of 49mm (refer figure below). If we apply this deflection to the BD49/01 empirical check we generate a Kd = Ymax * fb^2 = 49mm x $(0.88Hz)^2$ = $38mm/s^2$. Using Table 1 of BD49/01 we would have a situation with potential discomfort for users.

However, it is noted that BD49(2001) was developed at a time where non-linear time-history analysis was not readily available. The Kd check for acceleration issues is considered an empirical

approximation. We can use the vortex excitation from the time-history analysis to calculate the accelerations directly. From this analysis it is estimated that vertical accelerations at mid-span are only expected to reach 1.5m/s^2. For the design of the Longford Bridge vertical acceleration have already been checked for comfort by the criteria set in the "***Design of Lightweight Footbridges for Human Induced Vibrations (JRC-ECCS, 2009)***". As discussed in Appendix B, the ECCS 2009 approach permits vertical accelerations (with regard to user comfort) all the way up to 2.5m/s^2. In fact the 1.5m/s^2 is close to what would normally be considered medium comfort (rather than minimum comfort). It should also be noted that this is expected to occur when wind is 9.3m/s which is a 33km/hr wind. In such winds some "discomfort" would be expected when setting out for a walk or traversing a bridge. On the basis of a likely 1.5m/s^2 vertical acceleration during a 33km/hr wind, the comfort of the Longford Pedestrian Bridge when exposed to resonating wind vortex excitation effects has been deemed acceptable.

For determining the effects of fatigue caused by vortex excitation the time-history analysis has been used directly to determine the applicable stress range applied to key components. Refer to Section 7.10 of this report.

In terms of the load applied by vortex excitation we have considered a loading effect as per Table 1 of BD49/01. Although we consider this approach to be conservative, it would estimate an applied load from vortex effects of (0.4*Kd)/100%*DL = (0.4 * 38mm/s^2)/100% * DL = 0.15DL. We note that the loadcase for considering vortex effects is 1.2DL + 1.2SDL + 1.2WD(vrtx) + 1.35FP + PT. Since the most onerous case for vertical demands is ULS1A (which already has 15% more deadload plus significantly more live load) the contribution of the 15% deadload from vortex effects is not considered a governing design case.

Frequency Components		9.3m/s			21m/s			44m/s		
		Fy	Fz	Mx	Fy	Fz	Mx	Fy	Fz	Mx
	Const (N or Nm)	-2575	-500	975	-13025	-450	4925	-58350	-2250	20075
1st	Amp (N or Nm)	130	385	125	25	1475	400	260	5000	1425
	Freq / Hz	0.87	0.87	0.87	2.0	2.0	2.0	3.93	3.93	3.93
	Phase / deg	3	2	-191	-113	32	-86	-78	99	-23
2nd	Amp (N or Nm)	155	375	0	250	400	75	155	1800	475
	Freq / Hz	0.43	0.44	0.43	0.5	1.0	1.0	0.43	0.44	0.58
	Phase / deg	-67	-68	157	131	91	175	-69	168	107
Minimum Stable Damping Ratio		0	0	.0006	0	0	0	0	0	0

Figure 7-5 Extract of loads table from CFD Wind Study for a 30m long section of Longford Bridge (Fz = +/- 385N/30m = +/- 12.8N/m @ 0.87Hz)

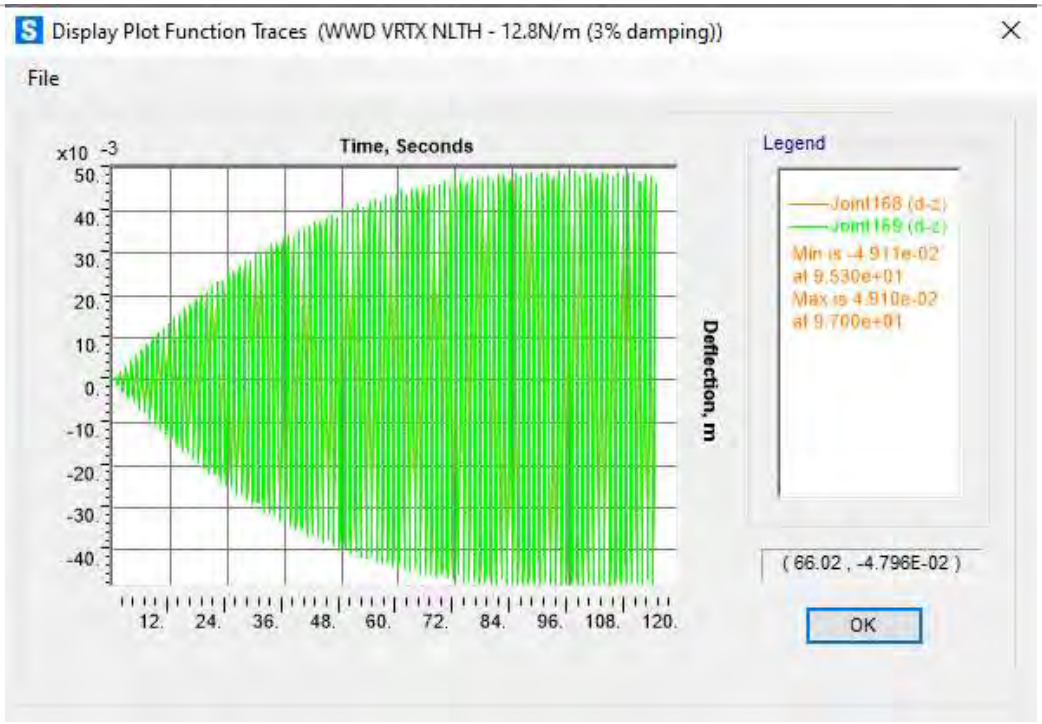


Figure 7-6 Time-history "deflection at midspan" during vortex excitation (Y_{MAX} = 49mm @ 95 seconds)

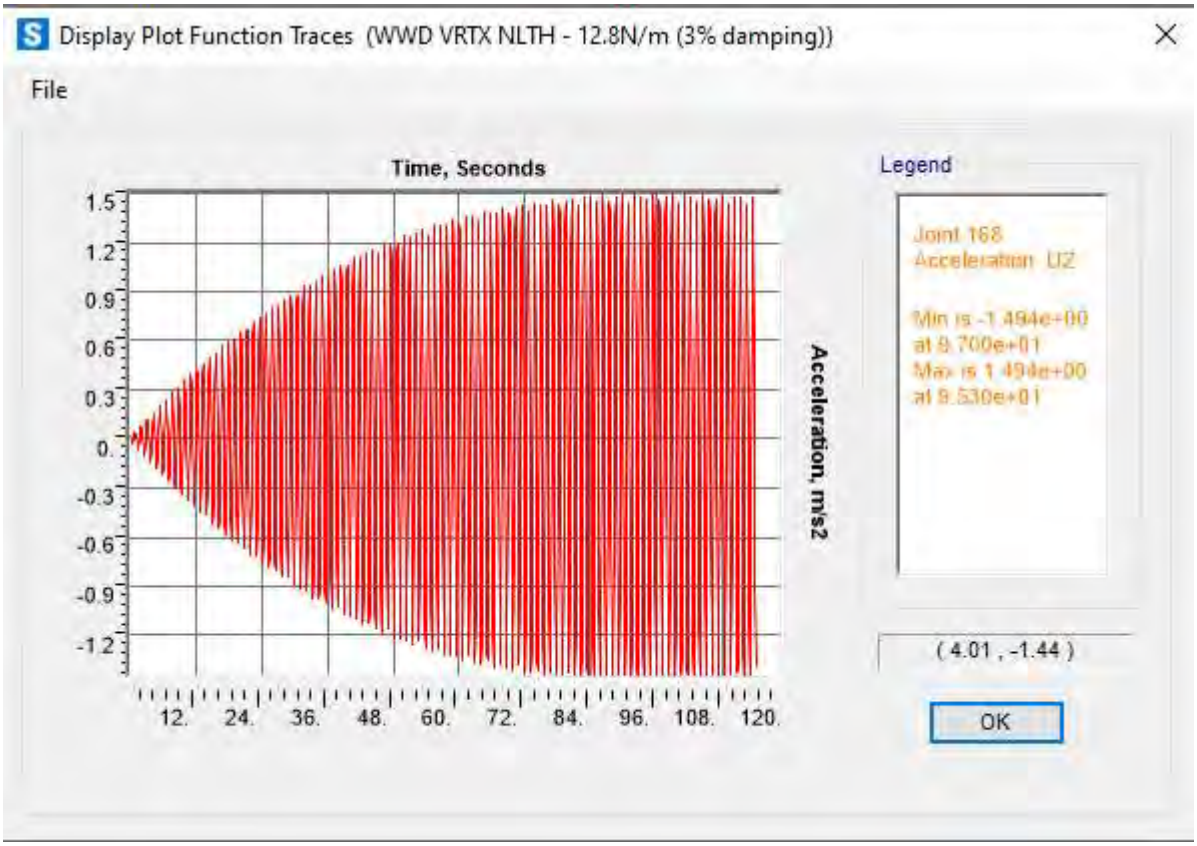


Figure 7-7 Time-history "acceleration at midspan" during vortex excitation (amax = 1.5m/s² @ 97 seconds)

K_D mm/s² (See Note 1)	Vertical load due to vortex excitation expressed as a percentage (α) of the total unfactored design dead plus live load on the bridge		Motion discomfort only for V_{cr} < 20m/s (see Note 2)
	A	B	
	All bridges except those in B	Simply supported highway bridge and all concrete footbridges	All bridges
≥100	α may be greater than 20%: Assess by analysis using derived y_{max}	α may be greater than 25%. Assess by analysis using derived y_{max}	Pedestrian discomfort possible (see Note 2)
50			
30	Assess by analysis using derived y_{max} or for simplicity use upper bound load, $\alpha = 0.4K_D$		Unpleasant
20			
10			
5	α is less than 4% and may be neglected	Assess by analysis using derived y_{max} or for simplicity use upper bound load, $\alpha = 2.5K_D$	Tolerable
3			
2			Acceptable
1		α is less than 5% and may be neglected	Only just perceptible

38mm/s2

Table 1 Assessment of Vortex Excitation Effects

Figure 7-8 Extract from BD/49 for the assessment of Vortex excitation

7.5.4. Aerodynamic Torsional Divergence

The wind study reviewed various angles of attack and concluded that divergence would not occur. Refer Appendix C.

This conclusion is consistent with BD49/01. We note that the BD49/01 equation 2.1.3.2 b) for checking divergence:

$V_g = 3.3\, f_T\, b$ for bridge types 1, 1A, 2, 5 and 6;

is a direct derivation of the equation contained in Eurocode EN 1991-1-4(2010) Annex E.4.3[6]:

$$v_{div} = \left[\frac{2 \cdot k_\Theta}{\rho \cdot d^2 \cdot \frac{dc_M}{d\Theta}}\right]^{\frac{1}{2}} \qquad \text{(E.24)}$$

[6] The derivation steps are explained in ***Wind effects on Cable Supported Bridges***, Xu, 2013 (section 3.3).

The Eurocode approach is a more complex variant of the BD49/01 check and requires that the designer knows the relationship of the "*rate of change of aerodynamic moment coefficient with respect to rotation about the torsional centre. Θ is expressed in radians*". This relationship can only be determined by wind tunnel or CFD testing (and hence why it is not covered in the empirical approach of BD49/01).

However, the CFD study for Longford has determined the relationship and we can apply it directly to the Eurocode equation. A negative gradient represents a "self-correcting" behaviour. However, a positive gradient implies potential divergence (increasing torque as further rotation occurs). On this basis we only need to consider the gradient between 2.5 and 5 (refer to Figure 7-10 below):

$$c_M = \frac{M}{\frac{1}{2} \cdot \rho \cdot v^2 \cdot d^2}$$

Thus,

Cm(2.5°) = (-2300/30m)/[(0.5 * 1.225kg/m^3 * (21m/s)^2 * (4.1m)2] = -0.0169

Cm(5°) = (2400/30m)/[(0.5 * 1.225kg/m^3 * (21m/s)^2 * (4.1m)2] = 0.0176

Thus dCM/dΘ = (0.0176 + 0.0169) / 0.0436 rads = 0.791

K_θ is the torsional stiffness of the bridge which has been derived from the SAP model as 15,873,000Nm/rad, d is the width of the cross section taken as 4.1m, and density of air, ρ is taken as 1.225kg/m^3.

Thus, **Vdiv = 1396m/s >> V_{WO} = V_{ULS} = 44m/s**

Since the critical wind speed is well above the ULS design case of 44m/s it is evident that torsional divergence does not occur. The gradient of change is basically too shallow to create divergence and will always be corrected by the structure's torsional stiffness. This is consistent with the findings of the independent study.

21m/s Bridge 2	Fy	Fz	Mx
5deg LE Up	-15636	-3692	2400
2.5deg LE Up	-17864	-9293	-2301
0deg	-13029	-280.4	4837
2.5deg LE Down	-17968	-423.5	23075
5deg LE Down	-16263	1545	7460

Figure 7-9 Extract of angle of attack variations taken from CFD Wind Study

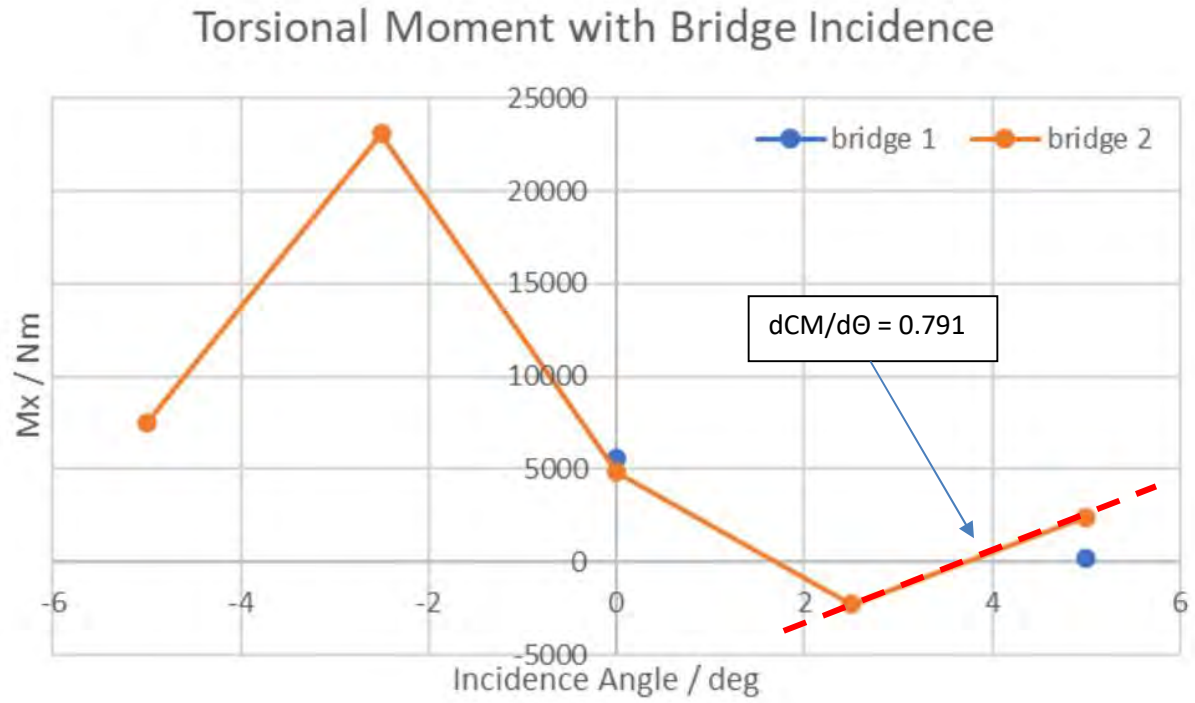


Figure 7-10 Extract of angle of attack variations vs moment graph taken from CFD Wind Study

7.5.5. Summary

BD49/01 checks in combination with an independent CFD wind study have confirmed that no wind issues are expected to arise during normal operation of the bridge.

7.6. Steelwork Design

Steelwork has been sized for all loads and load combinations discussed in Section 5 of this report. The primary beams, cross beams, and the main mast are all governed by the maximum dead + live loadcase ULS1A: 1.35DL + 1.35SDL+ 1.76FP(everywhere) + 1.35EP + PT. The bracing is generally governed by the transverse wind case ULS2B: 1.35DL + , flood, or seismic cases ULS3A: EQ_{LAT} + 30%EQ_{LONG} +1.35DL + 1.35SDL + 1.35EP + PT. Steelwork demands have been enveloped for the two variations of soil stiffness and the "Soft Model" was found to govern. This was also considered for the cable loss case where any one cable can be lost (and replaced by a dynamic cable loss force). Below is a summary of the models assumed and the resultant NZS3404:1997 design demand over capacity ratio (%). All steelwork demands are considered for non-linear P-delta affects where applicable.

Table 7-4 NZS3404 Demand over Capacity ratios for steelwork

Element	Critical location	Governing loadcase	NZS3404:1997 critical demand over capacity ratio
Primary Beam	Girder 1 with 25mm Flange (XB4 to XB5)	ULS1B(+): 1.35DL + 1.35SDL + 1.69TPG(+) + 1.69(TPD) + 1.82EP + Pt	74%*
		ULS1A: 1.35DL + 1.35SDL + 1.76FP + 1.82EP + Pt	55%
	Girder 2 with 25mm flange (XB1 to XB2)	ULS1B(+): 1.35DL + 1.35SDL + 1.69TPG(+) + 1.69(TPD) + 1.82EP + Pt	89%*
	XB4 – XB5	ULS1A: 1.35DL + 1.35SDL + 1.76FP + 1.82EP + Pt	70%
	Girder 1 with 16mm flange	ULS2B(2): 1.35DL + 1.35SDL + WD(28m/s) + 1.35FP + 1.35EP + Pt	86%
	Girder 2 with 16mm flange (Xb25 to XB26)		84%
Cross Beam	360UB-57 XB2	ULS2B(2): 1.35DL + 1.35SDL + WD(28m/s) + 1.35FP + 1.35EP + Pt	88%
	360UB-45 XB5	ULS1A: 1.35DL + 1.35SDL + 1.76FP + 1.82EP + Pt	42%
Bracing	150x150x16 top brace XB29-XB30	ULS3B: 1.1DL + 1.1SDL + FW + 0.85WDu + 1.38EP + Pt	89%
	125x125x12 top brace XB10-XB11		80%
	125x125x8 EA bottom brace XB29-XB30		70%

Mast	Main CHS Above base stiffener	ULS2B(2): 1.35DL + 1.35SDL + WD(28m/s) + 1.35FP + 1.35EP + Pt Construction case: 1.35DL(mast) + WDs	55% (including non-linear analysis of P-delta effects with 40mm mast imperfection in UX and UY direction)
	Mast stiffener	ULS2B(1): 1.35DL + 1.35SDL + WDu + 1.35EP + Pt	21%
RB32 back-span tie	Connected to Backspan Stay #1	ULS1A: 1.35DL + 1.35SDL + 1.76FP + 1.82EP + Pt	75%

*High transverse demand in the Super steel at the support is based on the steelwork being pinned in transverse for temperature effects. In reality the oversized bolt holes will remove these transverse temperature demands. Governing case for 25mm flange primary beam then becomes ULS1A at circa D/C = 55% and 70% for G1 and G2 respectively.

Primary beam stiffeners are welded either side (and in line with) the 360UB cross beam web. This detail has been preferred for eliminating all eccentricities at the connection and permitting economic and simplified detailing. For design purposes when designing to AS5100.6 cl 15.14 stiffeners are assumed full height 100x7 stiffeners (consistent with the cross beam 360UB45 7mm web). We note that the crossbeam flanges (which stiffen the stiffener) are likely to significantly increase the capacity of the stiffeners. However, since the maximum ultimate shear demand of 150kN does not exceed the "unstiffened" shear capacity of the beam (770kN) this fabrication decision is not deemed significant to warrant further study. Drawings show a 10mm plate stiffener either side of the cross beam (7mm is not an available size) but we note that 12mm may be used by constructor (depending upon plate availability).

7.7. Mast Design

7.7.1. Mast Linear Elastic Buckling

Buckling analysis of the steel mast confirms that the mast does not behave like a cantilever when considered for compression buckling. The buckling analysis confirms the first buckling mode of the mast is in the transverse direction when subjected to the influence of deadload and the cable-stay forces. The resultant buckling shape is not the buckling shape of a typical cantilever (Effective length 2H) and is instead consistent with the effective length of a mast pinned at both ends (effective length = H). These findings are consistent with the discussion in the cable stay bridge steel pylon design section and Figure 15 of the ***Bridge Engineering Design Manual*** (ICE, 2008). The stay-cables

which are triangulated on plan (from a central mast connection out towards the edge of the deck) form a system of forces that ensure the stay-cable loads are directed through the footing (Mast weight remains vertical but this is small by comparison). The linear elastic buckling analysis run in SAP2000 v22 confirms the stay-cable arrangement is sufficient to laterally restrain the top for preventing early cantilever buckling. For design purposes the effective length of the mast in compression is taken as the height to the centre of the stay cables. The SAP2000 v22 linear buckling analysis predicts a buckling capacity of 10,200kN in the transverse direction.

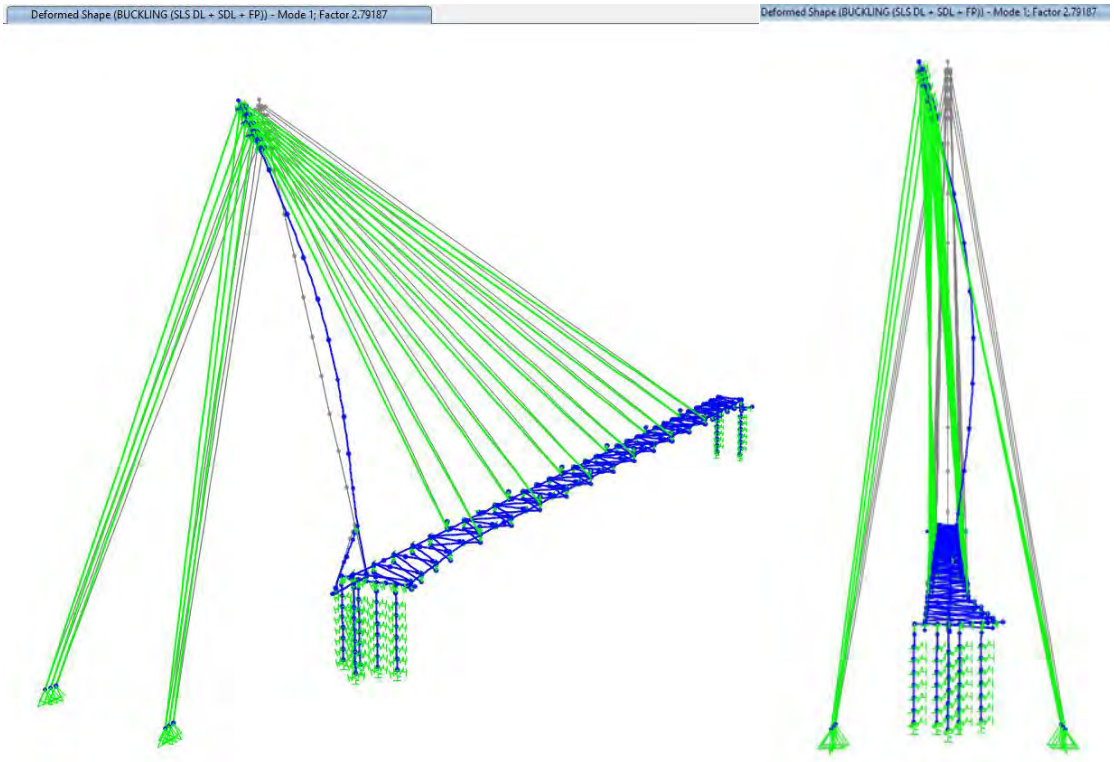


Figure 7-11 – Transverse Linear Elastic Buckling of the Mast (Mode 1 Buckling Factor = 2.8 times SLS1A design combination)

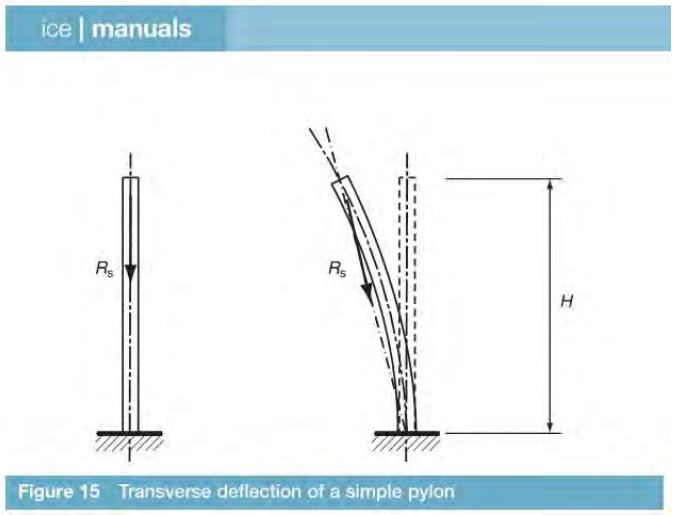


Figure 7-12- Figure 15 of the *Bridge Engineering Design Manual* (ICE, 2008) showing assumed effective length of a cable stay bridge mast in the transverse direction.

When we consider the mast in the longitudinal direction the buckling analysis confirms improved performance compared to the transverse direction. This is because the mast is directly restrained by the deck prop and/or the dead-man anchorage. This system is a more effective restraint than the triangulated plan-like bracing discussed above. The buckling length in the longitudinal direction is taken as 0.7H (not critical) which again is consistent with the ICE Manual of Bridge Engineering recommendation for cable stay steel pylons. The SAP2000 v22 linear buckling analysis predicts a buckling capacity of 11,665kN in the longitudinal direction.

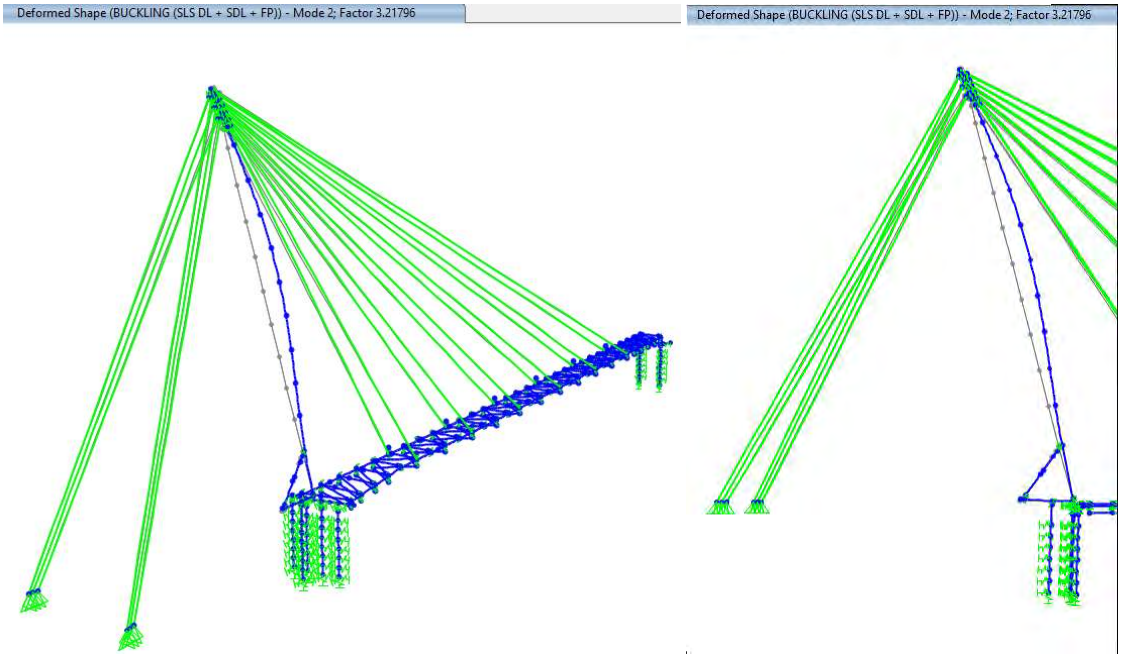


Figure 7-13 – Longitudinal Linear Elastic Buckling of the Mast (Mode 2 Buckling Factor = 3.2 times SLS1A Design Loads)

7.7.2. Mast Non-Linear Buckling Analysis

Noting the inherent issues with linear elastic buckling (that typically overestimate the axial buckling capacity of a member), we have also performed non-linear buckling analysis. To account for the influence of construction tolerances on the compression capacity of the mast an additional 40mm eccentricity has been allowed for. This “construction inaccuracy allowance” is input into the design process directly as a 40mm mid-height curvature in the “straight” mast member. This straightness inaccuracy is consistent with the tolerances permitted by NZS3404 section 3.3.4.1 so is therefore deemed suitable. For non-linear buckling the longitudinal direction was found to govern. Using a non-linear load-case of 800% of live load (starting from deflected shape and stiffness at the permanent unfactored load case) longitudinal displacement at top of mast Vs. applied axial were plotted. Axial buckling capacity was taken as the axial demand prior to “kick-back”. As can be observed in Figure below this was taken as 10,000kN (compared to 11,665kN longitudinal linear elastic buckling). It is noted that both the minimum linear elastic buckling capacities and the non-linear buckling capacity exceed the maximum axial demand of 5130kN (from ULS1A) by a factor of almost 2. Non buckling issues are expected to occur.

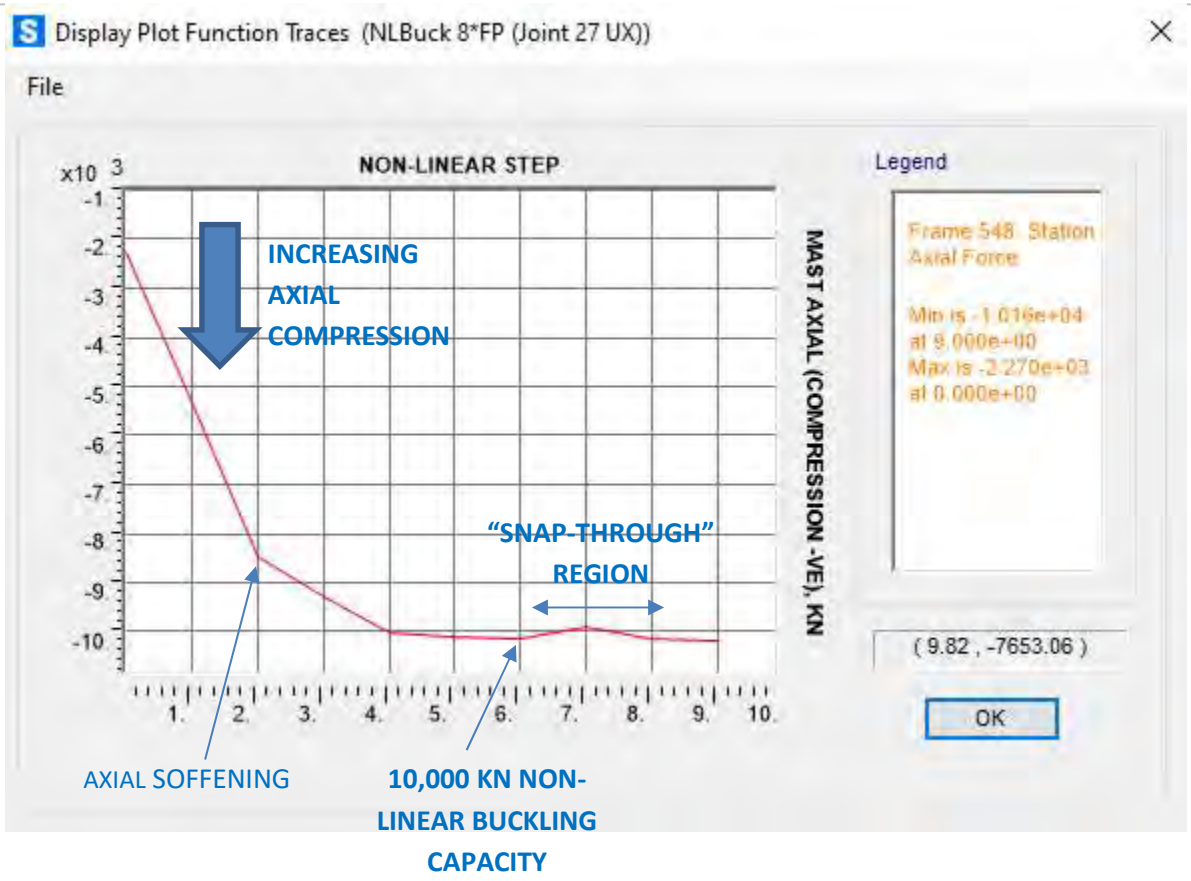


Figure 7-14 – Longitudinal Non-linear Buckling Analysis of the Mast (axial demand measured at top of mast when considered for limiting displacement of mast top in the UX direction)

7.7.3. Mast Combined bending and axial effects

When considering the mast for the effects of combined axial and bending as defined in NZS3404, the stay cables are considered a partial restraint for bending with αm taken as 1.75 (from T 5.6.1) at each critical section down the mast rather than the higher αm value derived from NZS3404 Appendix H for cantilevers unrestrained at the top. Using an effective member length of 39m, the slenderness reduction factor, αs, has been estimated at 1.03 using NZS3404 section 5.6. Since αm x αs is greater than 1, the nominal member bending capacity of the CHS has been taken equal to the section capacity θMb = θMs = 0.9 * 3950 kNm = 3555kNm.

The worse-case mast design demand over capacity ratio of 55% is governed by the combined effects of axial + longitudinal bending + transverse bending as defined by section 8.3.4.1 of NZS3404. This effect is most critical at the base just above the prop.

7.7.4. Mast Deflection

Deflection at the top of the mast is predicted at 40mm and 130mm toward the river under the serviceability "permanent" and "permanent + live" load cases respectively. This is a net effect which considers the 8mm and 11mm movement respectively of the western abutment away from the river due to the propping force in the superstructure reacting to the main-span stay cables. The

permanent deflection of 40mm equates to “mast height / 1000” and is thus deemed negligible (well within the H/200 limit set for design purposes).

7.7.5. Mast connections

Connections for the mast base are based on a finite element model analysis created in the connections software Ideastatica™ V10. The design is governed by the erection case for factored deadload mast in combination with the serviceability wind: 1.35*DL(mast) + WD_{SLS}. It is assumed that the mast is erected in two halves and that the base grout is installed prior to the second mast part being erected. The non-shrink base grout is assumed to transfer shear by friction once it achieves ≥ 30MPa strength. **Figure 7-13 – Finite element model of mast base connection**

7.8. Backstay concrete surround

7.8.1. General approach and loadings

The backstay concrete surround is used to protect the backstay from flood actions, particularly in regard to debris loads, that would otherwise strike the backstays and cause damage.

The concrete surround is shown on drawing S115. Top level has been set to a height of 77.3mRL to achieve a 300mm freeboard above the ULS flood height of 77.0mRL. Design of the concrete surround is governed by stability (overturning and sliding) based on a debris impact force of 175kN and 550kN striking the concrete 1.7m and 2.9m above the top of the footing for SLS and ULS event respectively. Loads have been developed in accordance with AS5100.2-2017 Section 16.6 as summarised in the table below.

Table 7-5 – Debris flood actions on concrete surround

16.6 Forces on Superstructures due to Debris

	ANCHOR ULS	ANCHOR SLS		
DEBRIS WIDTH, W =	15	15	m	sum of adjacent spans/2 or ≤ 15m (NZTABM2.3.5)
Cross Section Flood Design Velocity, V =	3.25	2	m/s	Fd = 0.5*Cd*V^2*As units of m^4/s^2
Upstream ULS Flood Level, FL =	77	75.8	mRL	Force based on F = ma = kg.m/S^2
Bridge Soffit level, SL =			mRL	Drag force based on m3 of water weighs 1000kg.
Bridge Barrier Top level, BL =			mRL	
Bridge Barrier (solid) Top level, BLs =			mRL	Thus keep V and As in m units (not mm)
Assumed Average Bed Level, BedL =	72.4	72.4	mRL	1m3water x m/s^2 = 1000kg.m/s^2 = 1000N = 1kN Thus resultant value is in kN.

16.6.4 Debris Load

Average flow depth, y = FL-BedL =	4.60	3.40	
Debris Depth, Ddeb =	2.3	1.7	m (1/2 water depth, 3m max, 1.2m min)
Area of debris, Adeb =	34.5	25.5	
V^2y	48.6	13.6	
Proximity Ratio, Pr(debris) = ygs/(dss+0.6) =			< use this value if normal proximity value is zero
Debris Coefficient, Cddebris =	3.0	3.4	< From Fig 16.5.2(A)&(B)
Debris Pressure on Super = 0.5*Cd*(Vsuper)^2 =	15.8	6.8	Kpa
Debris Force on Super = 0.5*Cd*(Vsuper)^2*As =	547	173	kN (see note above)

7.8.2. Sliding

Sliding resistance to the ULS debris load (550kN) is provided by two concrete shear keys at either end of the concrete surround. The leading shear key is able to mobilise 1.5m of passive earth pressure (=232kN) and the trailing shear key is able to mobilise 1.9m of passive earth pressure (=372kN)[14]. Total sliding resistance is 605kN.

During the ULS event the shear keys will need to move by circa 1900mm x 2.5% = 5mm in order to mobilize the passive earth pressure. This translation in the backstay anchors is accommodated by double bending of the 46mm Williams Form bar over the 4.7m unbonded length (bending and shear loads in the bar for 5mm deflection are negligible).

During an SLS event the resistance can be provided by direct friction at the underside of the footing (no translation required in the bar).

The downstream anchor is assumed shielded by the upstream anchor noting the anchors are aligned normal to the river flow. A single shear key is still used on the leading edge on the downstream anchor surround as the deep shear key also acts as a means of scour protection to prevent undermining of the foundation during extreme/unforeseen events. Providing this shear key also provides the shielded surround with sliding resistance equivalent to the SLS event.

7.8.3. Overturning

The rock anchors have been assumed to provide resistance when considering resistance to overturning. They are assumed anchored into the concrete surround concrete (above) and the rock anchor embedment (below).

Rock anchor load is assumed equal to the back-stay cable load (ignoring beneficial effects of concrete surround). Maximum backstay demand (ignoring cable failure case) is 560kN at ULS1A. For the flood cases ULS3B backstay demand (prior to including any allowance for overturning resistance) is significantly less at 370kN (noting no live load and a reduced deadload factor of 1.1 during ULS flooding). Capacity of the 150 diameter backstay anchors is estimated at 660kN (based on a 7m bonded length as per the Geotechnical Report provided by Riley). With this in mind it is considered appropriate that that cable and anchor are available to provide 560kN (maximum cable-stay design load) minus 370kN (flood stay load) = 190kN per anchor. This provides a total available resistance of 570kN for three anchors. The anchors have been placed with an average offset of 0.5m from the concrete pad centre-line. Refer to figure below for overturning design basis.

Key design basis:

- Overturning demand (@ULS) = 550kN x (2.9m + 0.4m) = 1815kNm
- Resistance = 570kN * 3.65m = 2080kNm;
- Demand / Capacity = 87%;
- Onsite geotechnical engineer to confirm a 350KPa ultimate geotechnical capacity under footing concrete.

[14] Based on a Kp value of 5.9 as per Table 7-2

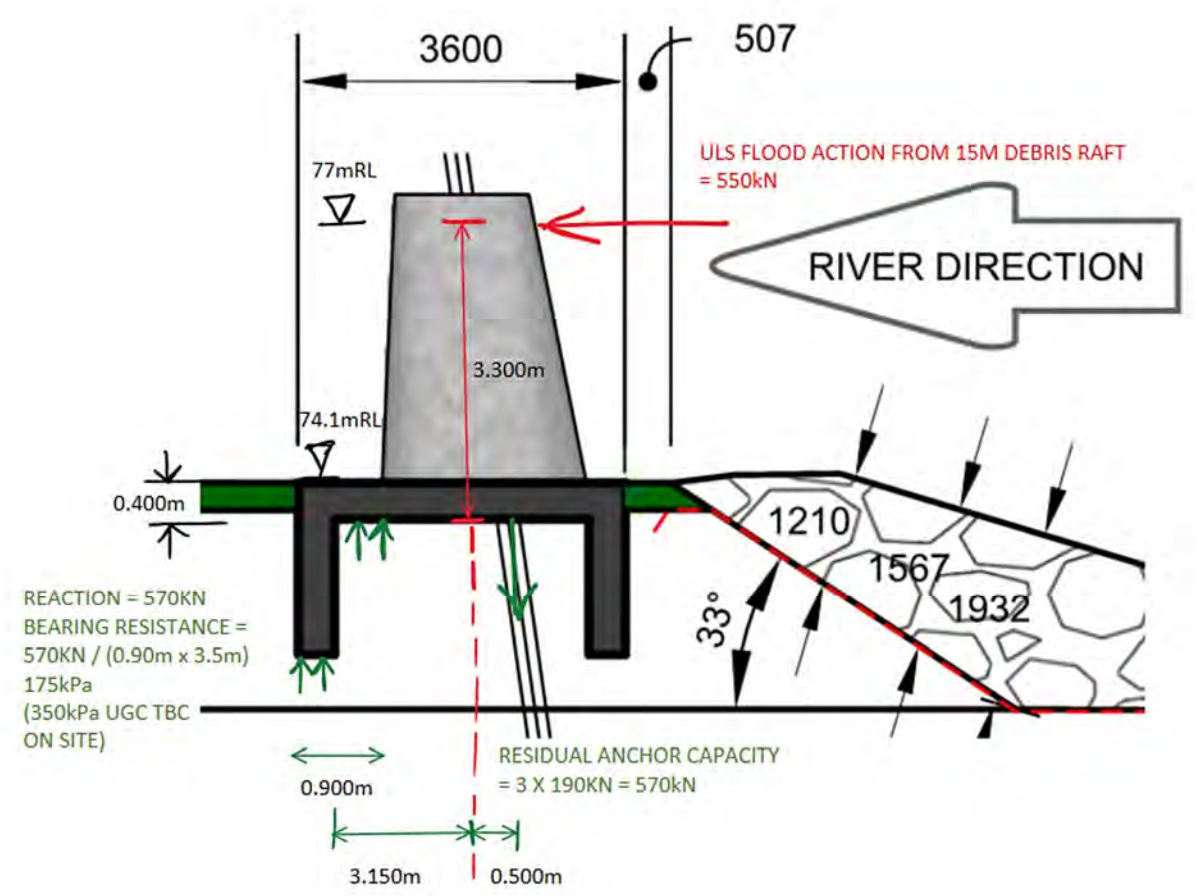


Figure 7-15 – Overturning design of backstay concrete surround.

7.8.4. Minimum reinforcing

The American ACI Guide 224R Part 3.5 "Control of shrinkage Cracking" (as referenced in NZS3101-2006 C2.4.4.8.) indicates that use of rebar to prevent shrinkage cracking is only practical in thin components like walls and slabs and is not used for larger concrete sections "due to the low drying shrinkage of these mass concrete structures". Part 4.7 of this guide "Crack control in deep beams" gives better advise (than NZS3101) on minimum detailing for deep beams. The ACI concrete cracking guide specifically discusses the issue of cracking to the face of deep beams and references the general American concrete code ACI 318. Section 10.6.7 of ACI 318 requires a minimum skin reinforcing based on a recommended spacing "S". To meet this recommendation the longitudinal side bars in the concrete surround have been limited to a maximum vertical spacing of S=150mm.

7.9. Back-stay failure Case

The worse-case backstay is operating at 560kN during the full ULS1A design case. This is confirmed on Drawing S132. The worse-case backstay force (prior to failure) from the cable failure loadcase: 1.1DL + 1.35SDL + 0.75FP(@2.6KPa) is estimated to be 410kN. This occurs in the top back-stay cable which is inclined at 67 degrees to the horizontal. After removing this cable from the modelling we apply a longitudinal force equal to 1.5*410kN*cos(67°) = 320kN to the top of the mast in combination with an upward force of 1.5*410kN*sin(67°) = 565kN. Both loads are applied in the

opposite direction to the original tension force thus representing the dynamic effects caused by the sudden release of cable force. See Figure below.

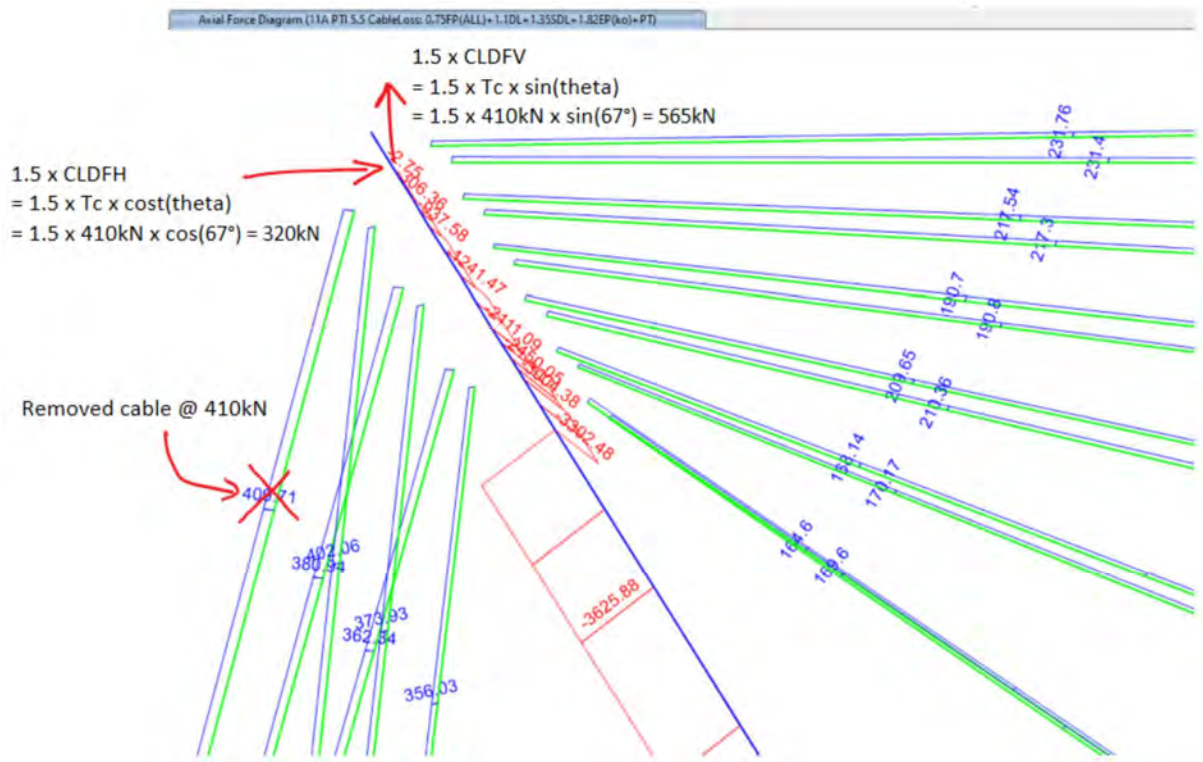


Figure 7-16 – Cable forces prior to cable loss.

For this cable loss scenario the backstays adjacent to the failed cable increase from 380kN and 360kN, to 760kN and 715kN for middle and bottom stays respectively (refer Figure below). We note this is 35% greater than the ULS1A demands. However, it should be noted that the SRF of 0.95 applies (as per PTI C5.5) rather than 0.65 as used for ULS1A (46% more strength) and thus the case does not govern the design of backstays. The same SRF applies to the Williams Form bar which are also designed to PTI standards. The backstay assembly shown on S131 is designed to the 760kN load generated by this case in combination with normal AS5100.6 strength reduction factors for steelwork and welds (this is intended to confirm no localised failure of the stressing assembly).

Backstay anchor have been sized for a 660kN load when using a standard strength reduction factor, θ = 0.5. It is noted that once the dynamic "snap-back" load subsides the maximum tension in the adjacent backstays is 570kN and 545kN for the middle and bottom stay respectively. For this very sudden (and short-duration) dynamic cable loss case a θ = 0.6 has been assumed giving a capacity of 790kN and thus in excess of the 760kN demand. 0.6 is consistent with the range nominated for all design situations in NZBC B1/VM4 Table 1 (where a range of 0.45 to 0.6 is considered acceptable).

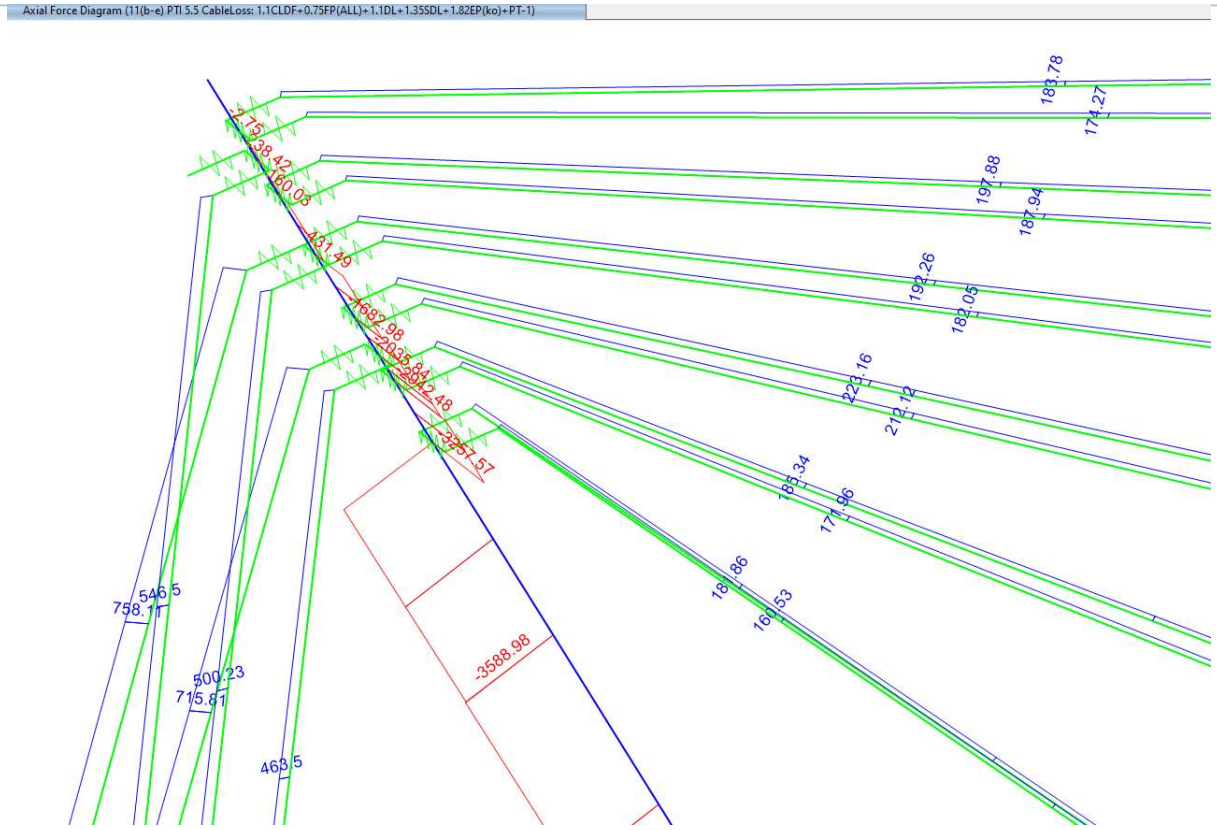


Figure 7-17 – Cable forces after back-stay cable loss.

It is noted that the sudden failure of a backstay cable may also release a dynamic compression into the Williams Form bar (of the failed cable) equal to 410kN x 1.5 = 615kN. During replacement of the failed/vandalised cable it will be necessary to cut away (or straighten) any buckled Williams Form bar and use an additional Williams Form Coupler to add new straight lengths to the assembly. This is assumed part of the replacement methodology.

7.10. Wind Fatigue

7.10.1. Stay cable & bar Fatigue

When the vortex shedding loads that occur at 9.3m/s are applied to the analysis model as a non-linear time-history oscillating loadcase, the maximum axial forces generated in the cables are 15kN and 20kN for mainstay and backstay cables respectively (refer to the figure below). This equates to 36MPa and 27MPa for the mainstay and backstay respectively during the high-cycle maximum amplitude oscillation scenario (as confirmed by the independent wind study). Since these values are below the fatigue threshold $(\Delta F)_n$ divided by 2 = 55MPa, the strands can be considered to have infinite fatigue life as per PTI 5.3.5 and fatigue effects can be ignored (refer table below for more info).

We note that BD49/01 Section 5.2 requires a load factor of 1.2 be applied to vortex excitation stress cycles when assessing demands but that the PTI stay cable design guide (section 5.3.5) permits the

fatigue load to be scaled down by a factor of 0.75. For simplicity with have assessed based on the nominal demand.

Table 7-6 – Fatigue Assessment of stay systems

Element	(A) Vortex Axial Effect, N[15]	(B) Cross Sectional Area, mm^2	Vortex Stress (wind fatigue demand) A/B, MPa	Stress range where fatigue effects can be ignored (PTI 5.3.5), MPa	Further fatigue analysis required?
26mm Spiral Mainstay	15,000	419	36	55	No
35mm Spiral backstay	20,000	753	27	55	No
26mm backstay bar	10,000	531	19	55	No
46mm anchor bar	20,000	1662	12	55	No

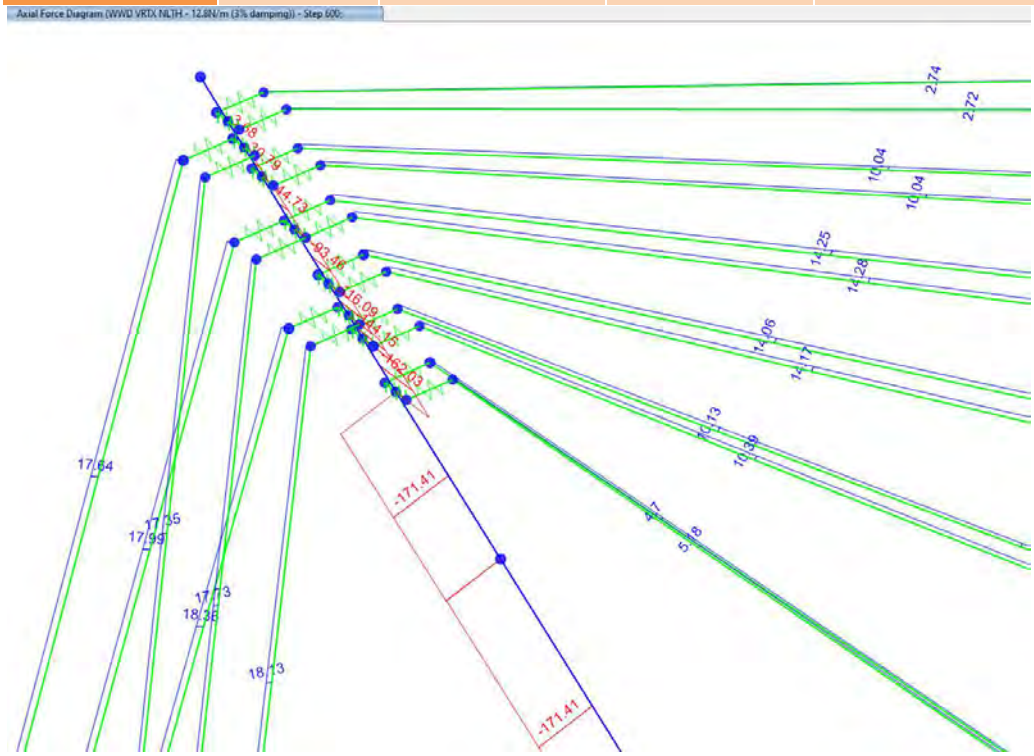


Figure 7-18 Axial effects caused by the vortex excitation non-linear time-history case (shown a T=60 seconds).

[15] Rounded to nearest 5,000N

7.10.2. Mast Fatigue

When the vortex shedding loads that occur at 9.3m/s are applied to the analysis model as a non-linear time-history oscillating loadcase, the stress range in the mast is an axial compression of 170kN (refer to Figure above) in combination with a maximum bending demand of -22kNm at the top and 30kNm at above the mast stiffener. Refer Figure below. This corresponds to a fatigue stress of 4MPa +/- 3MPa. With a maximum fatigue stress of 7MPa (only 2% of the yield strength) the vortex excitations are deemed satisfactory (stress range not considered significant enough to warrant further study).

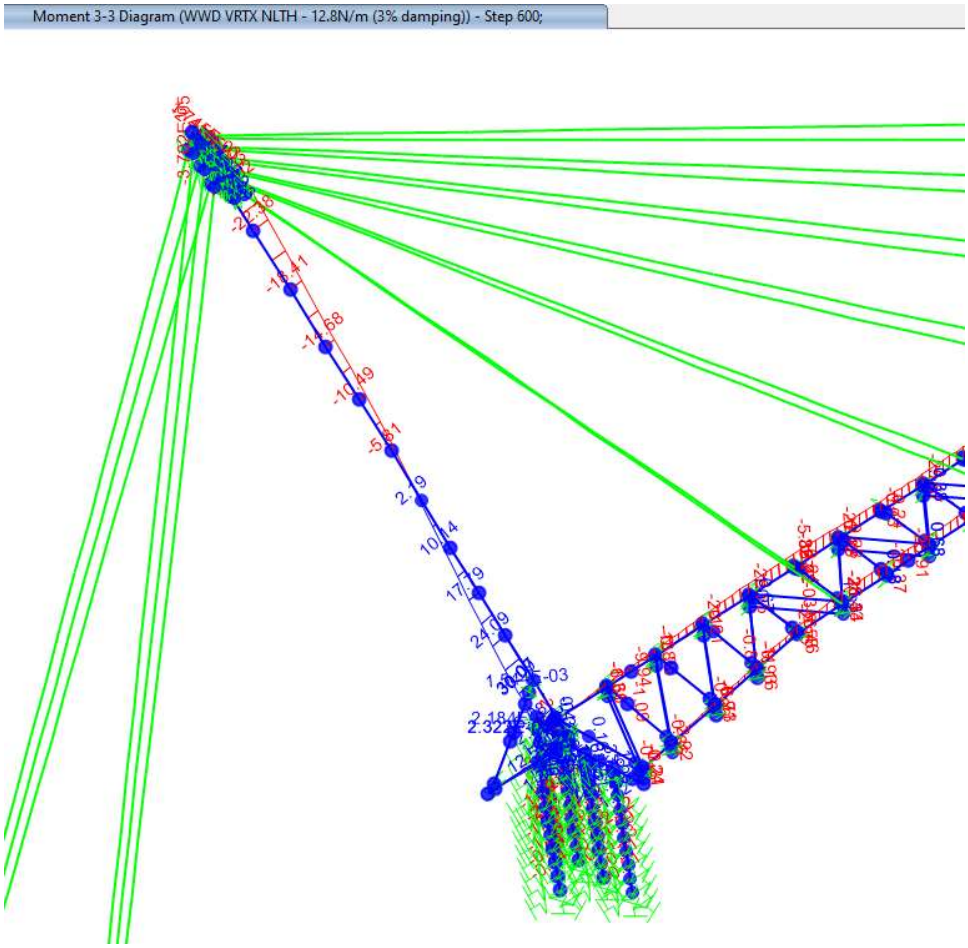


Figure 7-19 Bending demands caused by the vortex excitation non-linear time-history case (shown a T=60 seconds).

7.10.3. Superstructure Fatigue

When the vortex shedding loads that occur at 9.3m/s are applied to the analysis model as a non-linear time-history oscillating loadcase, the stress range in the superstructure is a maximum bending demand of 110kNm in combination with an axial compression of 3kN (between stay #5 and #6). This corresponds to a stress range in the primary beam flanges of 0.1MPa +/- 14MPa With a maximum fatigue stress of 14MPa (only 4% of the yield strength) the vortex excitations are deemed satisfactory (stress range not considered significant enough to warrant further study).

7.11. Shear bolt design basis at Eastern abutment

The shear bolts on the eastern abutment are designed for the loads as indicated on drawing S117. It is noted that the most onerous case is the ULS flood case that applies 140kN to each shear bolt from a single (upstream) direction applied to the bottom flange. The next largest case is the ULS wind case that applies 100kN in either transverse direction and assumed applied to top and bottom flanges.

It is noted that issues can occur if one shear bolt contacts early (possibly as a result of construction tolerances in setting the shear bolt in 2mm or 3mm oversized transverse holes). This can result in one shear bolt taking all load and failing prior to the next bolt activating. To mitigate this issue the design utilizes the flexibility of the U-frame created by the cross beam and primary beam bearing stiffeners. The force required to push the U-frame 3mm (so that both shear bolts can engage) is estimated to be 10kN from SAP2000 modelling of the U-frame.

Three cases have been considered:

- Case #1: Load is applied at deck level and/or cross beam level and either shear bolt engages first,
- Case, #2: Load is applied to the bottom flange and the trailing shear bolt engages first
- Case, #3: Load is applied to the bottom flange and the leading shear bolt engages first

Case #1 and #2 would both successfully engage the U-frame action and enable both shear keys to activate. The first shear bolt to engage would only see 10kN more than the second shear bolt prior to both bolts taking the load evenly. Since this is less than 10% of the critical demand it is considered an advantageous scenario.

In Case #3 there is no ability for the trailing shear bolt to take the load as the U-frame flexure cannot sufficiently engage. Case #3 only applies to a flood condition where debris loads apply direct load to the bottom flange adjacent to the support. All other transverse demands such as wind and seismic have at least 50% of the demand concentrated in the upper flanges and associated bracing. To provide added confidence that the trailing shear key activates first during the ULS flood event the slotted hole on S123 has been set transversely 2mm oversized on the trailing shear bolt and 4mm oversized on the leading shear bolt. This approach aims to prevent Case #3 from occurring and thus the U-frame action will always engaged and activate both shear keys within 10kN.

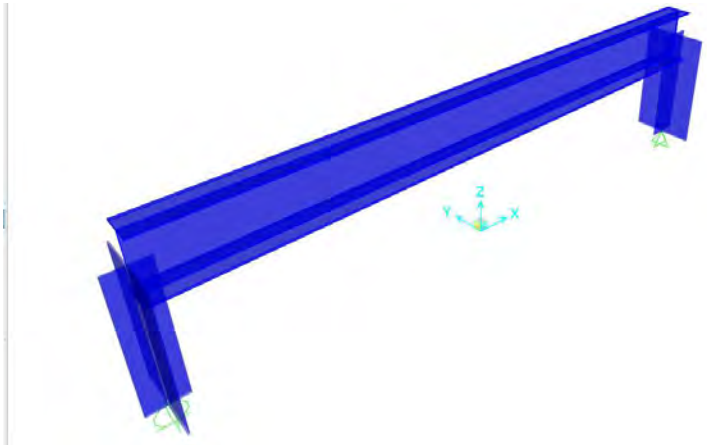


Figure 7-20 Simple SAP2000 model of East Abutment U-frame.

8. Construction Methodology

For the purposes of detailed design the Designer has assumed the following construction sequence[16]:

1. Western embankment is constructed to enable a safe piling platform;
2. Piles and soil anchors are installed;
3. Abutments and associated bearings are installed;
4. Soil anchor concrete surround is cast;
5. Additional temporary piers placed on the edge of the main river channel.
6. 45m western and eastern steelwork sections lifted into place. Constructor's temporary piers include provision for temporary Reidbar stays to support the midspan steelwork (refer to Figure 8-1 and 8-2 below). These temporary stays will be installed and stressed prior to connecting at midspan;
7. ≥ 50% of permanent stay-cables can then be installed and stressed. Cable stressing loads (kN force in cable) and surveyed bridge shape (mm from horizontal at each cross beam) shall be performed and documented by the constructor to be reviewed by the Designer. This shall occur following the stressing of each cable set. A "set" is taken as a pair of cables either side of the bridge width;
8. Temporary piers can be removed at this stage (or remain in place until end of construction if preferred);
9. Timber joists, decking, and balustrade are installed;
10. Remaining cables stressed. Cables loads and surveyed bridge shape shall be performed and documented by the constructor to be reviewed by the Designer. This shall occur following the stressing of each cable set.
11. Western approach embankment, all other connecting paths, and landscaping completed. All outstanding work completed;
12. Final inspections of bridge;
13. Watermain installed from western end through sleeves;
14. Bridge opened to public.

[16] This concept methodology is consistent with the methodology proposed by the construction team during tendering (Concrete Structures Ltd., Dec 2019).

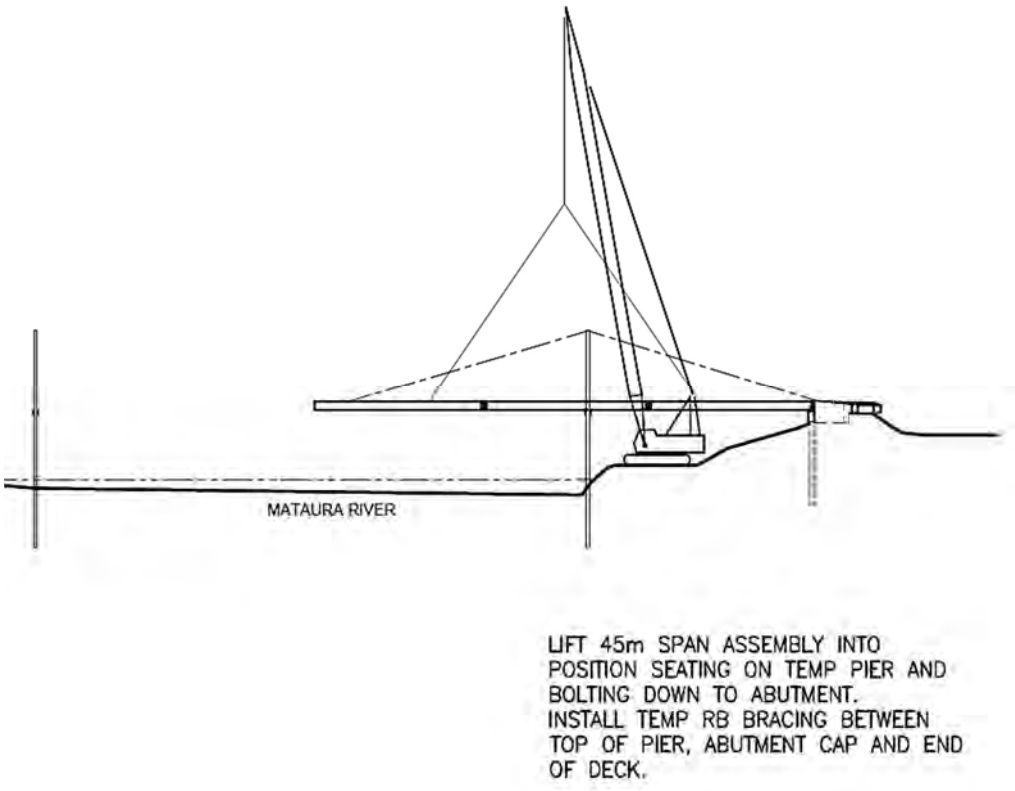


Figure 8-1: Extract from Concrete Structures tender Construction Methodology (Dec 2019)



Figure 8-2: Extract from Concrete Structures tender Construction Methodology (Dec 2019)

9. Construction Support & Site MSQA

During the construction phase our lead bridge designer (Dan Crocker, CPEng structural, CEng Civil) and geotech designer (Eli Maynard, CPEng Geotech) will be visiting site regularly to work alongside the construction team to perform quality testing to construction monitoring level 4 (CM4). This testing is intended to confirm construction meets the design documentation which significantly reduces the risk of poor performance of the council asset.

Dan and Eli are expecting to perform the following QA checks to provide confidence that the bridge and associated works are fit for purpose:

1. General Structural design support during construction phase (responding to any interpretation issues, technical queries, or proposed changes);
2. General Geotechnical design support during construction phase (technical queries including consideration of quarry and alternative fills/rip-rap materials);
3. Collaboration with temporary works designer (providing 3D geometry/modelling, geotech inputs, confirming wind/flood levels, and discussing our design, etc.);
4. Review of temporary works for impacts relating to permanent works (this is not a PS2 for temporary works just a check to make sure there are no negative effects to permanent works when using Constructor's temporary piers and cable stay erection method. We will strive to provide useful recommendations/changes if there is an issue);
5. Design of any temporary lifting lugs and/or review of soft sling arrangements for erection of steelwork (general design support for lifting permanent steelwork);
6. Review and confirmation of cable supply (where alternatives are proposed) and response to technical cable queries;
7. Review and acceptance of cable supply testing certificates (confirmation of weight, tensile capacity, stiffness, coatings, fatigue, etc. corresponding to the design);
8. General coordination with fabrication team (during shop drawing drafting);
9. Detailed steelwork shop drawing review (we will provide redline mark-ups etc.);
10. MSQA of steelwork:
    a. Review/acceptance of nominated weld inspector qualifications;
    b. Review/acceptance of nominated paint inspector qualifications;
    c. Correspondence with Independent Weld Inspector and Independent Paint Inspector to confirm all weld checks and paint systems have passed and any recommended mitigations/remedies have been employed;
    d. Review/acceptance of steel mill testing certificates;
    e. Visit to fabricators for spot checks on detailing and geometry (normally 1 for super and 1 for mast);
    f. Review/acceptance of weld inspection reports;
    g. Review/acceptance of paint inspection reports;
11. Review and agreement of onsite survey pegs as installed by Constructor's registered surveyor (e.g. earthworks, piles, and concrete);
12. Onsite inspection of earthworks (including confirming appropriate batters, compaction, materials, weight, etc.);
13. Onsite inspection of boreholes and technical design support during piling;
14. Review of pile reinforcing cages (checking bar diameters, grade, number, etc.);

15. Abutment pre-pour concrete inspections (cover, formwork, set-out, bar diameters, grade, number, spacing, etc.);
16. Review of concrete supply certificates (confirming applicable 7 day and 28 days strengths, use of applicable additives, etc.);
17. Review of survey following erection of steelwork, mast, abutments, and anchorages prior to start of cable stressing;
18. Review and confirmation of contractor proposed cable stressing methods (we have allowed to provide blank QA check sheets for the stressing operation to help with this);
19. Review of cable stressing records including onsite presence during at least 20% of all cable stressing;
20. Review of the survey following each stage of cable installation;
21. Review of structural timber production certificates (strength and treatment confirmation);
22. Spot checking of timber connections to confirm use of fixings as per drawings;
23. Review of balustrade mesh infill supply certificates (stainless steel grade, wire diameter, spacing, etc.);
24. Spot checking of balustrade cable infill detailing and anti-climb mesh sizes;
25. Liaison with FRP flooring supplier to confirm testing in accordance with drawings;
26. Review of final survey to confirm the bridge is erected and ready for opening;
27. Snag checking prior to Practical Completion;
28. PS4 sign-off for Structural;
29. PS4 sign-off for Geotechnical.

Remedies for non-conformance will vary depending upon the issue identified. We are using experienced staff competent in the design and construction of cyclebridges over water. All rectifications will be based on achieving a standard at least as good as the original design documentation (which will already have been agreed and confirmed by Council prior to construction).

Where non-conformance is identified it will be raised onsite with the Contractor's key personnel immediately so that it can be rectified without delay. We will also notify GDC where necessary.

APPENDIX A
GENERAL ARRANGEMENT DRAWINGS

Longford Shared Path - Project 2019/11

GO DISTRICT COUNCIL RE



D

DRAWING NUMBER		DESCRIPTION	LATEST REV
GENERAL:			
GDC_2019/11_S	001	COVER SHEET	D
GENERAL ARRANGEMENT DRAWINGS:			
GDC_2019/11_S	100	BRIDGE PLAN	D
GDC_2019/11_S	101	BRIDGE ELEVATION	D
GDC_2019/11_S	102	BRIDGE CROSS SECTIONS	D
GDC_2019/11_S	103	BRIDGE ISOMETRIC	D
GDC_2019/11_S	104	WEST BRIDGE APPROACH	D
GDC_2019/11_S	105	EAST BRIDGE APPROACH	D
GDC_2019/11_S	102	WEST EMBANKMENT	D
ABUTMENTS AND ANCHORAGES:			
GDC_2019/11_S	110	PILE SETOUT & SPEC	D
GDC_2019/11_S	111	EAST ABUTMENT	D
GDC_2019/11_S	112	EAST ABUTMENT REBAR	D
GDC_2019/11_S	113	WEST ABUTMENT	D
GDC_2019/11_S	114	WEST ABUTMENT REBAR	D
GDC_2019/11_S	115	BACKSTAY ANCHORAGE	D
GDC_2019/11_S	116	ANCHORAGE DETAILS	D
GDC_2019/11_S	117	BEARINGS	D
STEELWORK DRAWINGS:			
GDC_2019/11_S	120	STEELWORK NOTES & ISOMETRIC	D
GDC_2019/11_S	121	SUPER STEELWORK - SHEET 1	D
GDC_2019/11_S	122	SUPER STEELWORK - SHEET 2	D
GDC_2019/11_S	123	SUPER STEELWORK - SHEET 3	D
GDC_2019/11_S	125	MAST STEELWORK - SHEET 1	D
GDC_2019/11_S	126	MAST STEELWORK - SHEET 2	D
GDC_2019/11_S	127	MAST STEELWORK - SHEET 3	D
CABLES			
GDC_2019/11_S	130	STAY-CABLE DETAILS	D
GDC_2019/11_S	131	BACK STAY-CABLES	D
GDC_2019/11_S	132	CABLE STRESSING SEQUENCE	D
GLULAM TIMBER			
GDC_2019/11_S	140	GLULAM BEAMS	D
BALUSTRADE			
GDC_2019/11_S	150	BRIDGE BALUSTRADE - SHEET 1	D
GDC_2019/11_S	151	BRIDGE BALUSTRADE - SHEET 2	D

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A	PRELIM DESIGN	NOV 19	DC	PS	DC
B	PRELIM DESIGN	NOV 19	DC	PS	DC
C	FOR PEER REVIEW	MAY 20	DC	PS	DC
D	FOR CONSTRUCTION	JUN 20	DC	PS	DC

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PROJECT LONGFORD SHARED PATH
PROJECT NO. 2019/11

LONGFORD SHARED PATH
MATAURA CYCLEBRIDGE
COVER SHEET

DWG: GDC_2019/11_S001

S001
Rev D

PLOT DATE: June 25, 2020



NOTES:

1. ALL DIMENSIONS ARE IN MILLIMETERS UNLESS NOTED OTHERWISE.

BRIDGE PLAN

BP	-	*1:500 @ A3*

REV	DESCRIPTION	DATE	DESIGN	DRAWN	APPROVED
A	PRELIM DESIGN	NOV 19	DC	PS	DC
B	ROAD SAFETY AUDIT	APR 20	DC	PS	DC
C	PEER REVIEW	MAY 20	DC	PS	DC
D	FOR CONSTRUCTION	JUN 20	DC	PS	DC

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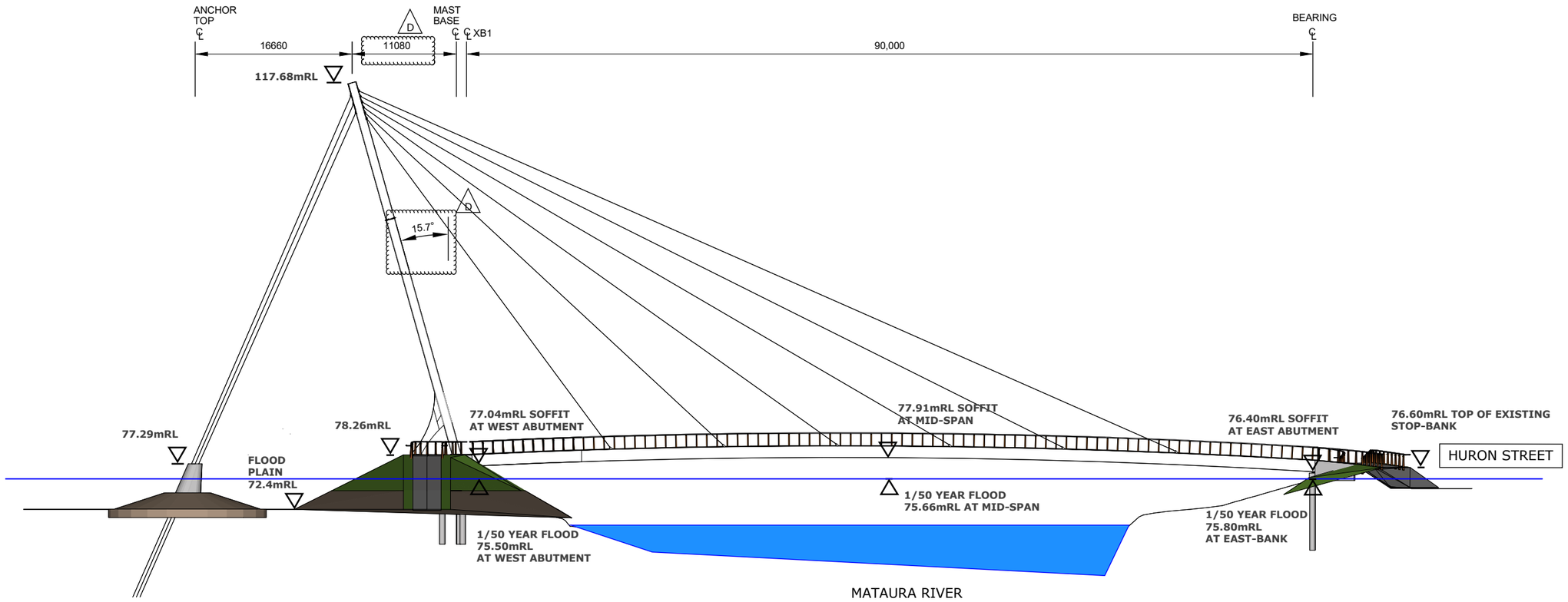
PROJECT
LONGFORD SHARED PATH
PROJECT NO.
2019/11

LONGFORD SHARED PATH
MATAURA CYCLEBRIDGE
BRIDGE PLAN

DWG: GDC_2019/11_S100

S100
Rev D

PLOT DATE: June 25, 2020



BRIDGE ELEVATION (FROM SOUTH LOOKING UPSTREAM)

BE | - | *1:500 @ A3*

INSPECTION/MAINTENANCE ACTIVITY	REPLACEMENT	REFERENCE
RE-APPLY / CLEAN ANTI-GRAFITTI CONCRETE COATINGS	AS REQUIRED	NOTE 6. S111
REPAIR / RE-APPLY STEELWORK ANTI-CORROSION SYSTEM	≤ 40 YEARS	S120
CABLE REPLACEMENT AND RESTRESS #	≤ 100 YEARS	S130, S131, S132
REPLACE GLULAM TIMBER BEAMS	≤ 50 YEARS	S140
REPLACE GLULAM BALUSTRADE POST	≤ 50 YEARS	S150

CABLE REPLACEMENT GUIDANCE IS CONTAINED ON S132 AND THE "LONGFORD CABLE-STAY FOOTBRIDGE DESIGN STATEMENT" REV D

BRIDGE MAINTENANCE SCHEDULE

MS | -

MAINTENANCE AND INSPECTION:

1. BRIDGE INSPECTION AND REPORTING SHALL BE IN ACCORDANCE WITH THE *NZTA BRIDGE INSPECTION POLICY TNZ S6-2000* WHICH REFERENCES THE DEPARTMENT OF TRANSPORT (UK) *BRIDGE INSPECTION GUIDE, (DOT, 1983).*
2. AS PER TNZ S6-2000 A "GENERAL INSPECTION" SHALL BE CARRIED OUT EVERY 2 YEARS AND A "DETAILED INSPECTION" EVERY 6 YEARS.
3. THE DETAILED INSPECTION SHALL BE PERFORMED BY A SUITABLY QUALIFIED BRIDGE INSPECTOR OR BRIDGE ENGINEER (AS PER TNZ S6-2000). IT IS RECOMMENDED THAT THE DETAILED INSPECTION (EVERY 6 YEARS) IS CARRIED OUT BY A BRIDGE ENGINEER WITH FAMILIARITY WORKING WITH CABLE STAY STRUCTURES.
4. THE INSPECTIONS SHALL RECORD ANY REQUIRED MAINTENANCE/REPLACEMENT ACTIVITIES AND THEN BE FORWARDED TO THE MAINTENANCE TEAMS FOR COMPLETION WITHIN A PRE-SET TIMEFRAME.
5. TO IMPROVE AWARENESS OF DESIGN LIVES, SPECIFICATIONS, AND GENERAL DETAILS OF REPLACEABLE ITEMS, PLEASE REFER TO THE MAINTENANCE SCHEDULE OPPOSITE. THE INTENTION OF THIS TABLE IS TO IDENTIFY KEY INSPECTION ITEMS.
6. THE MAINTENANCE TABLE ALSO ENABLES INSPECTION TEAMS TO APPRECIATE "WHEN" THE DESIGNERS ARE ANTICIPATING THAT STEELWORK IS TO BE RECOATED, CABLES ARE TO BE REPLACED, AND TIMBER REPLACED, ETC. THIS SHOULD PREVENT THE INSPECTOR NEEDING TO AIR ON THE SIDE OF CAUTION (WHEN SUCH INFORMATION IS NOT OTHERWISE PRESENTED).
7. IT IS STRONGLY RECOMMENDED THAT THE "LONGFORD CABLE-STAY FOOTBRIDGE DESIGN STATEMENT" (REV D) SHALL BE READ PRIOR TO ANY SIGNIFICANT CHANGES BEING MADE TO THE BRIDGE (E.G. ADDITIONAL SERVICES BEING ADDED, CHANGE OF BALUSTRADES, ETC.).
8. ALLOWABLE MAINTENANCE VEHICLE LOADS ARE DISCUSSED ON DRAWING S100.
9. REPLACEMENT OF CABLES IS DISCUSSED IN MORE DETAIL ON DRAWING S132.

FLOOD MANAGEMENT:

1. THE CONTRACTOR IS TO PREPARE AND HAVE IN PLACE A CONSTRUCTION FLOOD MANAGEMENT PLAN THAT CLEARLY OUTLINES PROCEDURES AND PROCESSES FOR REMOVAL OF ALL PLANT, STOCKPILES, MATERIALS, PEOPLE, TEMPORARY WORKS, ETC. FROM THE FLOODPLAIN WITHIN 20 HOURS OF A FLOOD WARNING BEING RAISED AND/OR INCREASED LEVELS BEING RECORDED AT THE "FAIRLIGHT" OR "PIANO FLAT" MONITORING STATIONS. THIS PLAN IS TO BE SUBMITTED TO ENVIRONMENT SOUTHLAND 14 DAYS PRIOR TO THE START OF CONSTRUCTION FOR REVIEW AND CONFIRMATION.
2. THE CONTRACTOR IS RESPONSIBLE FOR SUBMITTING A PLAN ACCEPTABLE TO ENVIRONMENT SOUTHLAND.
3. EMBANKMENT THAT HAS BEEN CONSTRUCTED ABOVE A LEVEL OF 74.1MRL (3.25M ABOVE NORMAL FLOW) MAY BE CONSIDERED PERMANENT WORKS AND NOT REQUIRE REMOVAL.
4. TO BE CONSIDERED PERMANENT WORKS IT MUST BE CONSTRUCTED AND PROTECTED WITH RIP RAP AS PER DRAWINGS AND SIGNED OFF BY THE GEOTECHNICAL/HYDRAULIC ENGINEER.
5. MORE INFORMATION RELATING TO MONITORING STATION CONTACT DETAILS CAN BE FOUND HERE: HTTPS://WWW.ES.GOVT.NZ/REPOSITORY/LIBRARIES/ID:26GI9AYO517Q9STT81SD/HIERARCHY/ONLINE-SERVICES/FLOOD%20WARNING/DOCUMENTS/FLOOD-WARNING-FACTSHEETS/MATAURA_FLOOD_WARNING_WEB.PDF

REV	DESCRIPTION	DATE	DESIGN	DRAWN	APPROVED
A	PRELIM DESIGN	NOV 19	DC	PS	DC
B	SPECS ADDED	NOV 19	DC	PS	DC
C	FOR PEER REVIEW	MAY 20	DC	PS	DC
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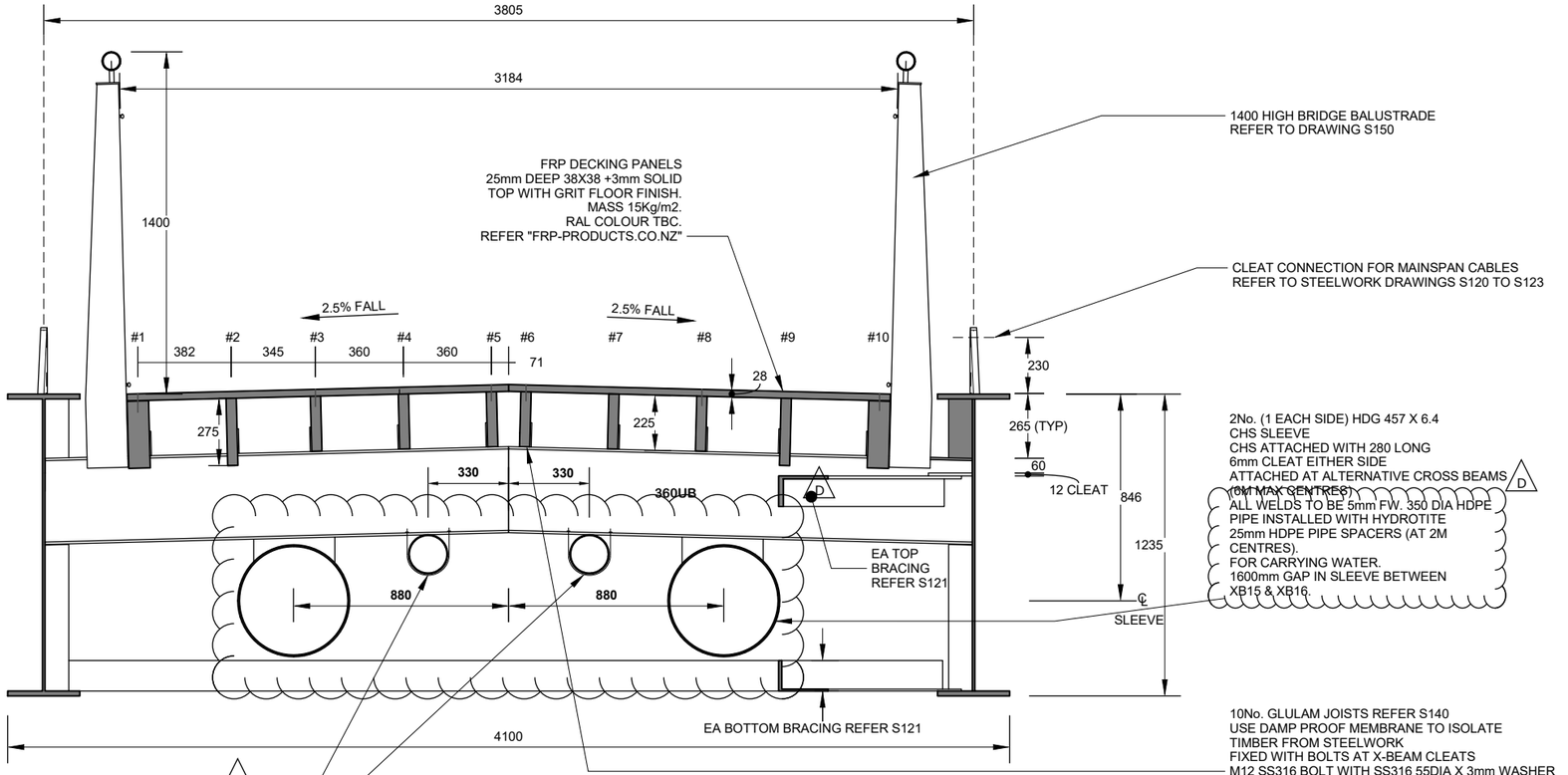
PROJECT
LONGFORD SHARED PATH
PROJECT NO.
2019/11

LONGFORD SHARED PATH
MATAURA CYCLEBRIDGE
BRIDGE ELEVATION

DWG: GDC_2019/11_SK02

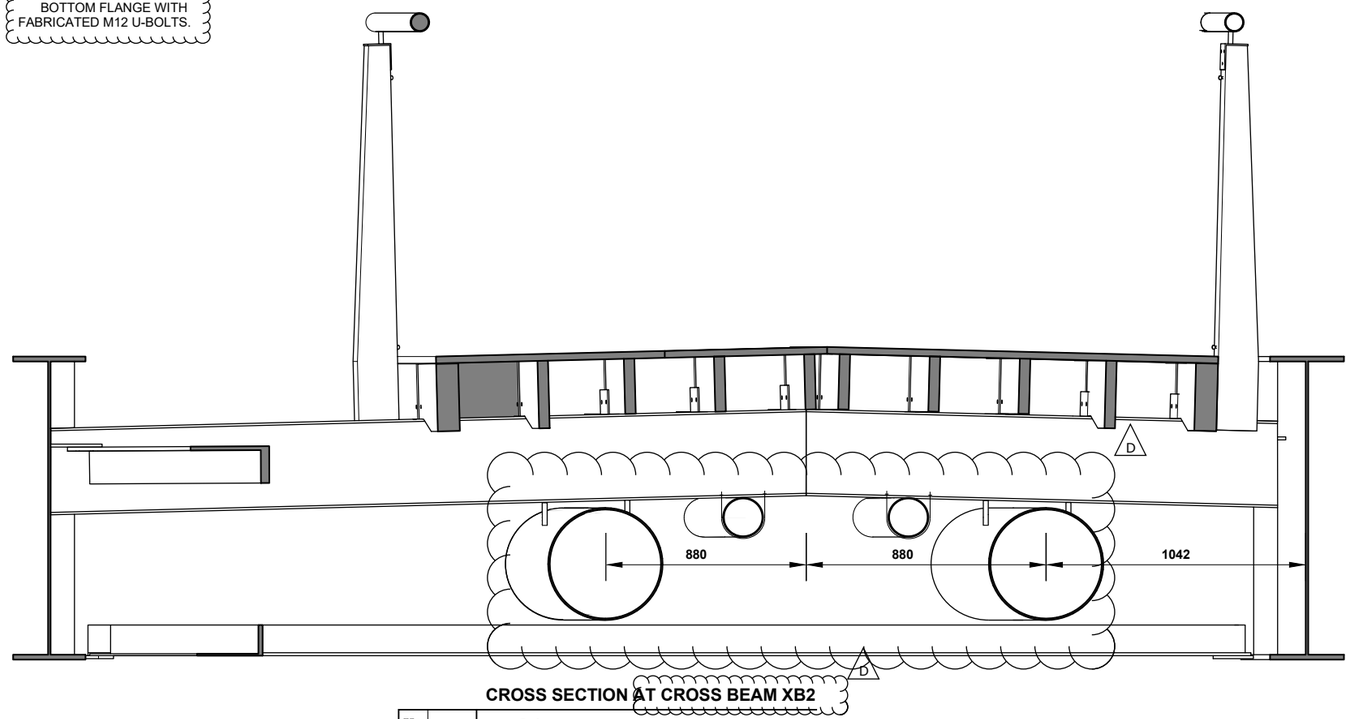
S101
Rev D

PLOT DATE: June 22, 2020



CROSS SECTION AT CROSS BEAM (& CABLE STAY)

X1 | - | *1:25 @ A3*



CROSS SECTION AT CROSS BEAM XB2

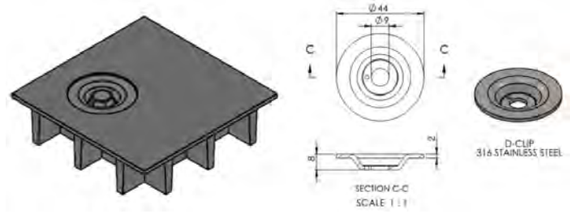
X2 | - | *1:25 @ A3*

NOTES:

1. REFER TO DRAWINGS S120 TO S123 FOR ALL SUPERSTRUCTURE STEELWORK.
2. REFER TO DRAWING S140 FOR ALL TIMBER NOTES.

FRP DECKING AND FIXINGS:

1. FIBRE REINFORCED PLASTIC (FRP) DECKING PANELS ARE TO BE PROVIDED BY "FRP PRODUCTS LTD. (NZ).
2. PANELS ARE TO BE 25mm thick 38 x38 FRP grid with a 3mm solid panel topping (28mm total depth).
3. PANEL COLOUR TO BE "TBC" (RAL XXXX).
4. INDEPENDENT LABORATORY TESTING IS TO BE PERFORMED, AND CERTIFIED BY A CPEng ENGINEER (OR RECOGNISED TESTING LAB), TO CONFIRM:
   - A. ULS STRENGTH USING A LOAD FACTOR OF 1.76 FOR:
     - i. 5KPA LIVE LOAD APPLIED OVER THE WHOLE PANEL;
     - ii. 2.2KN POINT LOAD (600x600);
   - B. 5KPa DEFLECTIONS < SPAN/200 = 3mm;
   - C. WET PENDULUM (SLIP RESISTANCE) TESTING AS/NZS 4586 2004 SRV >39
   - D. ≥ 40 YEARS DURABILITY / UV EXPOSURE
5. FIXINGS FOR ATTACHING THE FRP PANELS AND ALUMINIUM EXPANSION JOINT PLATES TO THE GLULAM JOISTS WILL BE A 14g (M6.3) 125 LONG SS316 SCREW WITH HEX HEAD (FROM ANZOR). 4 FIXINGS ARE REQUIRED ALONG A JOIST FOR ATTACHING A SINGLE PANEL (12 FIXINGS PER PANEL). FIX AT JOISTS: #1, #3, #5, #6, #8, & #10.
6. FIXINGS WILL BE COUNTERSUNK INTO THE FRP DECK PANELS USING THE FRP PRODUCT "D-CUP" WASHER (REFER FIGURE BELOW).



REV	DESCRIPTION	DATE	DESIGN	DRAWN	APPROVED
A	PRELIM DESIGN	NOV 19	DC	PS	DC
B	ROAD SAFETY AUDIT	APR 20	DC	PS	DC
C	FOR PEER REVIEW	MAY 20	DC	PS	DC
D	FOR CONSTRUCTION	JUN 20	DC	PS	DC

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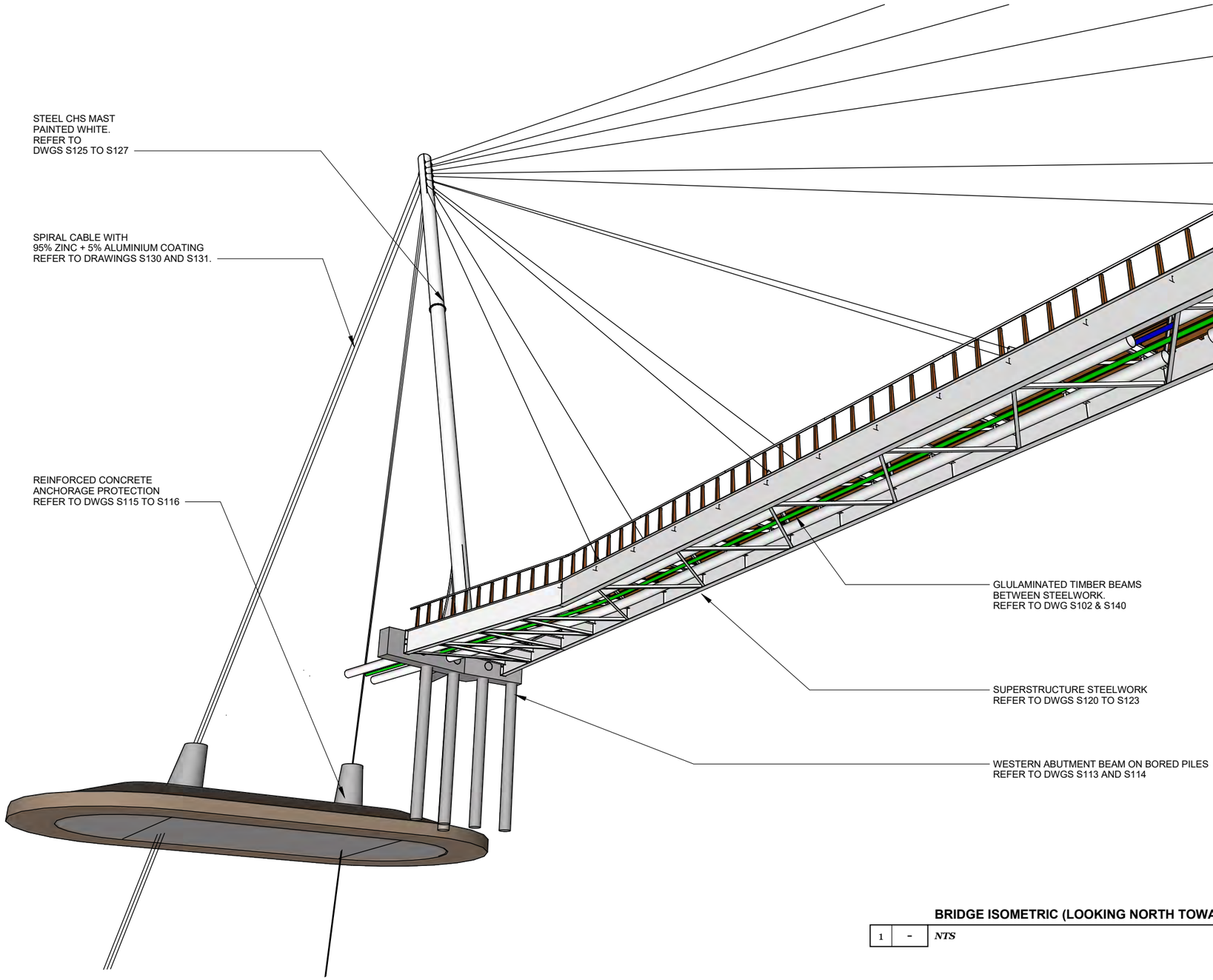
PROJECT
LONGFORD SHARED PATH
PROJECT NO.
2019/11

LONGFORD SHARED PATH
MATAURA CYCLEBRIDGE
BRIDGE CROSS SECTIONS

DWG: GDC_2019/11_S102

S102
Rev D

PLOT DATE: June 22, 2020



BRIDGE ISOMETRIC (LOOKING NORTH TOWARDS WEST ABUTMENT)

1	-	*NTS*

REV	DESCRIPTION	DATE	DESIGN	DRAWN	APPROVED
A	PRELIM DESIGN	NOV 19	DC	PS	DC
B	(NOT USED)				
C	FOR PEER REVIEW	MAY 20	DC	PS	DC
D	FOR CONSTRUCTION	JUN 20	DC	PS	DC

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PROJECT NO.
2019/11

LONGFORD SHARED PATH
MATAURA CYCLEBRIDGE
BRIDGE ISOMETRIC

DWG: GDC_2019/11_S103

S103
Rev D

APPENDIX B
PEDESTRIAN VIBRATION ASSESSMENT

DC STRUCTURES STUDIO

Longford Cable-stay Bridge Pedestrian Dynamics Study

Longford Cable-stay Bridge – Pedestrian Dynamics Study

Prepared for

Gore District Council

Issued date

June 2020

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Document Status

Rev	Description	Author	Date
A	Issued for Peer Review (PS2)	Dan Crocker	May 2020
B	Updated following PS2 comments	Dan Crocker	June 2020

Review and Approval

Description	Person	Signature	Date
Author	Dan Crocker		22 June 2020

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2. Design Basis 5
3. Structural dynamic behaviour 7
4. Assessment 10
   4.1. General 10
   4.3. Vertical and Lateral Vibrations 10
   4.4. Lateral “Lock-in” 12
5. Conclusions 18

1. Pedestrian Vibration Assessment

The Longford Cable-Stay Footbridge is a 90m clear span steel cable-stay footbridge. Because of its large span and relative flexibility it has been deemed important to assess the likely dynamic performance and check that adequate comfort for users can be achieved.

Pedestrian dynamics are considered based on the detailed methods discussed in the ***Design of Lightweight Footbridges for Human Induced Vibrations*** (JRC-ECCS, 2009) referred to herein as the "ECCS Guide". Further inputs are also derived from ***Assessment of vibrational behaviour of footbridges*** (SETRA, 2006) referred to herein as the "SETRA Guide". Reference is also made to the expected user densities as discussed in ***Eurocode 2 Design Loads for Bridges - UK Annex (BS EN 1991-2 NA)*** referred to herein as the "UK Eurocode 2".

2. Design Basis

The following scenarios are design conditions as proposed for the Longford Cable-Stay Footbridge are shown in the table below.

It should be noted that the bridge is a semi-rural bridge built on the outskirts of the Gore town centre. The closest description deemed applicable is UK Eurocode 2 is "Bridge class B: sub-urban route" described as a "location likely to experience slight variation in pedestrian loading intensity on an occasional basis". This user group has an expected design density of 0.4 users per square meter of bridge (total pedestrians using the bridge = 108 people).

In reviewing the likely class for this bridge it was noted that the population of Gore District is approximately 12,400 and that the number living in "Gore Central" (west of bridge) and "East Gore" are 180 and 1443 respectively[1]. As such the total people living and working in the township, and in close proximity to the bridge, is circa 1623. Adopting the semi-urban density would suggest that 108 of these 1623 people are using the bridge at the same time (7% of everyone). This makes the projection of 108 seem rather high. With this in mind, it is likely that user densities lower than a typical Class B bridge would be more applicable. As such we believe a density of 0.2 is likely to be most appropriate for assessing comfort criteria for the bridge. In this instance 54 people are assumed to be using the bridge in a single instance which equates to 3% of all people using the bridge (which still seems a high event). Comfort levels have been assessed on this basis in setting the criteria of Table 2-1.

In determining the acceleration limits, and noting the slender nature of the bridge, it is anticipated that users using the footbridge/cyclebridge will be expecting a greater level of movement compared to lesser span and/or concrete bridges (this effect of user perception for lightweight bridges is discussed in ECCS Section 4.3.2)

[1] From https://www.stats.govt.nz/tools/2018-census-place-summaries/gore-district

Table 2-1 – Assumed Pedestrian Dynamic Serviceability Criteria

Design Situation	Description	Traffic Density, D (persons/m2)	Expected Occurrence	Comfort Class
2	Large event including inauguration of the bridge (108 people)	0.40*	Once every 5 years	CL3 (lowest) – upper region < 2.5m/s^2 vertical < 0.8m/s^2 lateral
3	Higher use (54 people)	0.20	Once every 6 months	CL3 (lowest) Lower region < 1.8m/s^2 vertical < 0.55m/s^2 lateral
4	General use (27 people)	0.10	Once per week	CL2 (medium) < 1m/s^2 vertical < 0.3m/s^2 lateral

* Crowd density of 0.4 people per SQM is equivalent to UK Eurocode 2 "bridge class B: sub-urban route" described as a "location likely to experience slight variation in pedestrian loading intensity on an occasional basis".

Comfort class	Degree of comfort	Vertical a_{limit}	Lateral a_{limit}
CL 1	Maximum	< 0,50 m/s²	< 0,10 m/s²
CL 2	Medium	0,50 – 1,00 m/s²	0,10 – 0,30 m/s²
CL 3	Minimum	1,00 – 2,50 m/s²	0,30 – 0,80 m/s²
CL 4	Unacceptable discomfort	> 2,50 m/s²	> 0,80 m/s²

Note that the given acceleration ranges are just comfort criteria; lock-in criteria for horizontal vibrations are given in section 4.6.

Figure 2-1 – Defined comfort classes with common acceleration ranges (Table 4-4 of ECCS Guide)

3. Structural dynamic behaviour

The structural analysis model is as discussed in the ***Longford Cable-Stay Footbridge– Design Statement (DC Structures Studio, 2020)***. Generally speaking we have developed a 3D finite element model in SAP2000 v22 utilizing steel superstructure elements, steel frame mast elements, and high strength steel catenary stay-cables.

The natural frequencies of response are tabulated below. The modes are checked based on the non-linear stiffness at the end of full deadload and wind tie stressing case.

As per the ECCS Guide Section 4.2 vertical effects are only considered in the natural frequency range of ***1.25Hz ≤ fi ≤ 2.3Hz*** *and* ***2.5Hz ≤ fi ≤ 4.6Hz*** for considering the first harmonic and second harmonic pedestrian loads respectively. Lateral effects need only be considered in the natural frequency range of ***0.5Hz ≤ fi ≤ 1.2Hz***. It is also noted by the ECCS Guide that lateral effects are only considered for the first harmonic mode shapes.

Table 3-1 – Structural dynamic performance

Mode No.	Natural Frequency, Hz	Description of mode shape	Mass participation[2]	No. of half waves (N)	Comments	To be assessed?
1	0.89	Vertical (UZ)	70%	N=1	Outside of critical range.	No
2	1.10	**Lateral (UY)**	79%	N=1	**In the critical range for first lateral harmonic.**	**YES**
3	1.78	**Vertical (UZ)**	12%	N=2	**In the critical range for first vertical harmonic.**	**YES**
4	2.53	Torsion (RX)	67%		Torsion modes not considered.	No
5	2.93	**Vertical (UZ)**	12%	N=3	**In the critical range for second vertical harmonic.**	**YES**
		Longitudinal (UX)	6%		Longitudinal not considered	No.
7	3.81	Longitudinal (UX)	59%		Longitudinal not considered	No.
8	4.12	Longitudinal (UX)	10%		Longitudinal not considered	No.
12	4.90	Longitudinal (UX)	9%		Longitudinal not considered	No.
13	5.58	Lateral (UY)	8%	N=3	Outside of critical range.	No.
		Torsion (RX)	3%		Torsion modes not considered.	No

[2] Only showing scenarios where superstructure mass participation exceeds 4%.

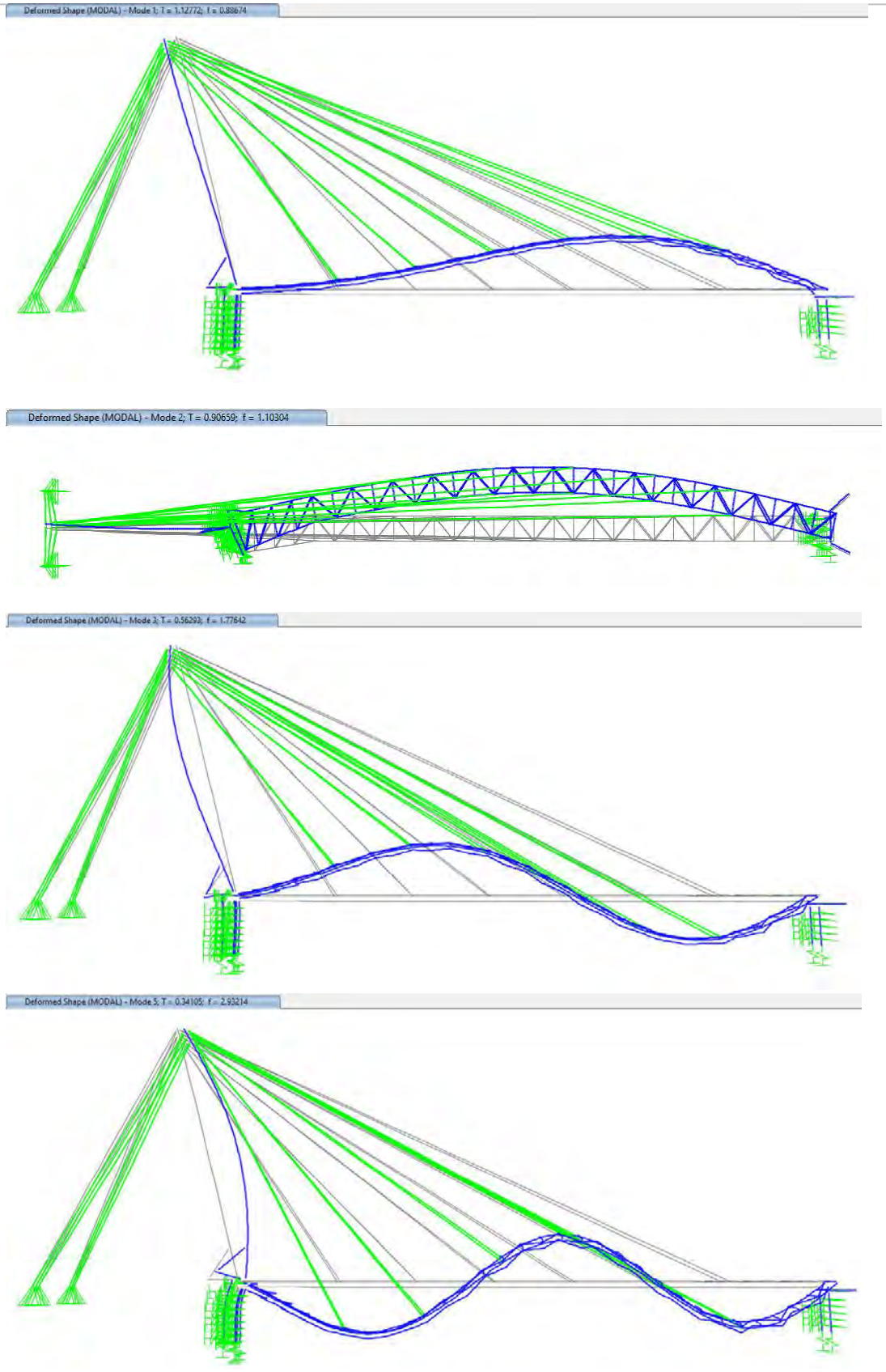


Figure 3-1 extracts from SAP2000 model of key mode shapes (scale = 50).

4. Assessment

4.1. General

We have opted to assess the dynamic performance of the bridge using the ***Harmonic Load Modelling (HLM)*** approach applied to our FE analysis model. This method is described in Section 4.5.1 of ECCS 2009. Step-by-step examples of how this method has been applied is contained in Appendix A to C.

4.2. Vertical Vibrations

From Table 3-1 it can be observed that mode shape #3 (fv=1.78Hz) and Mode shape #5 (fv=2.93Hz) require assessment for vertical pedestrian vibration effects. All other vertical cases are outside of the critical ranges defined by Section 4.2 of the ECCS Guide.

Using the harmonic load approach, we have found that Mode 3 was the most susceptible to pedestrian dynamics and produces significant accelerations. Mode 3 was tested for the first harmonic of pedestrian loading for varying user densities. Inputs/outputs are summarised in Table 4-1 below and full methodology is contained in Appendix A.

The accelerations of Mode 5 when subjected to the second harmonic were found to be minor in comparison and did not govern (only 0.4 users/m^2 density was assessed). Inputs/outputs are summarised in Table 4-1 below and full methodology is contained in Appendix C.

For vertical effects we have adopted the average viscous coefficient of critical damping, $\xi = 0.4\%$ for our steel superstructure (mast, cables, and superstructures are all steel). This value is as recommended in the ECCS Guide Table 4-5.

In summary:

Users / m^2	Total users	Measured Acceleration, m/s^2	Proposed Limit, m/s^2
0.10	27	1.25 (medium)	1.00 (medium)
0.20	54	1.77 (lower minimum)	1.80 (lower minimum)
0.40	108	2.51 (upper minimum)	2.50 (upper minimum)

As can be observed from the summary, all of the crowd densities that were assessed have maximum accelerations corresponding closely to the comfort criteria set in Table 2-1. We note that the 0.1 user density (27 people) doesn't quite achieve the medium comfort level we were targeting but it does still fall in the region of acceptable. Our maximum user density of 0.4 is right on the limit of what would be deemed acceptable.

In recognising that the limits set are about defining the overall user experience it should be noted that the values summarised above represent the absolute worse-case acceleration which is only

applicable to finite points along the bridge. As can be seen in the Figure below, the majority of the bridge easily achieves the intended comfort levels. It is only a very small portion of the bridge that will subject the users to a less than optimal experience. In the pursuit of economic solutions for GDC, and noting all values are within the “acceptable” limits, this is regarded as a good outcome without the need for significantly modifying the bridge response or adding tuned mass dampers.

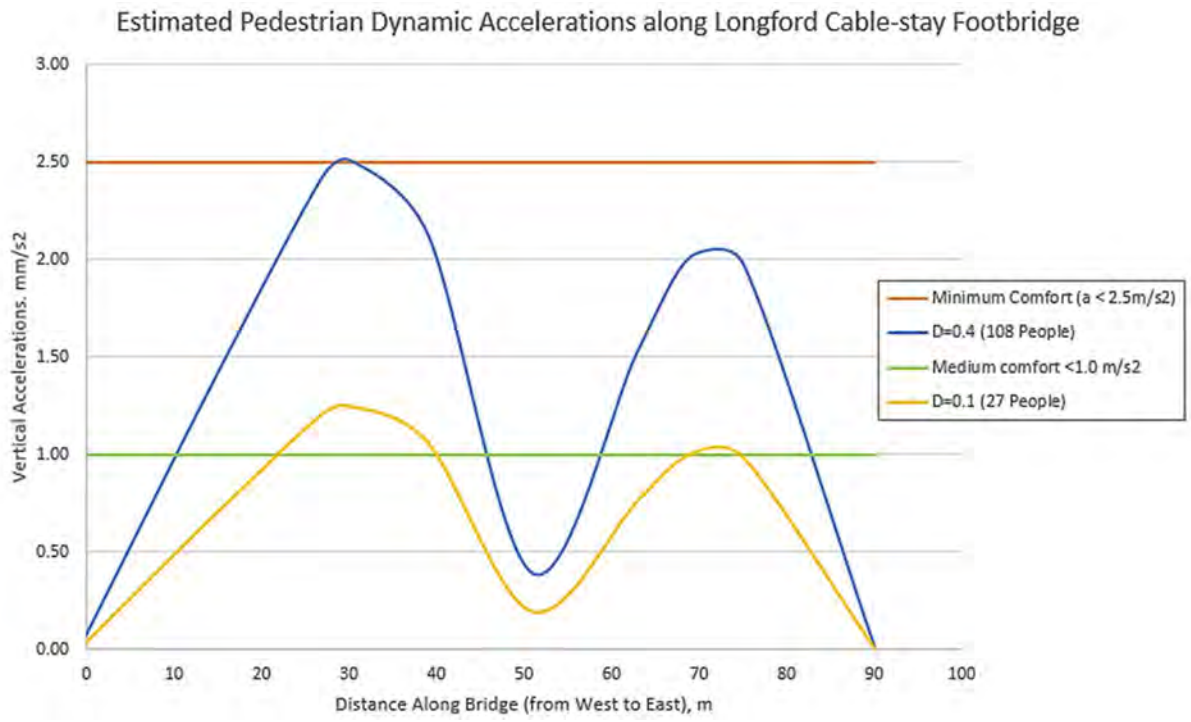
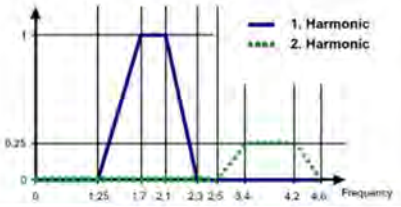


Figure 4-1 Estimated vertical dynamic accelerations as experienced along the bridge

Table 4-1 Summary of the Vertical Harmonic Load Modelling Approach applied to Mode 3 & 5

Direction considered	Vertical (Mode 3)			Vertical (Mode 5)	
Mode Shape	N=2			N=3	
Natural Frequency, Hz	1.78			2.93	Hz
Natural Period of Vibration, T	0.56			0.34	secs
Harmonic Type	FIRST HARMONIC VERTICAL			2nd Harmonic	
Damping Ratio, %	0.40%			0.40%	
User Density, No./m^2	0.1	0.2	0.4	0.4	No./ m2
Applied single user load, N	280				N
Bridge Length, L	90				m
usable Brideg Width, B	3				m
Loaded surface, S	270				m2
Total number of users, n	27	54	108	108	No.
Synchronized Number of users, No/m^2	n′ = 10.8*√(ξ x n) / S				
	0.013	0.019	0.026	0.026	No./ m2
% of users synchronized	13%	9%	7%	7%	
Reduction coefficient, Ψ	1.00			0.12	
Synchronized load applied	280N x n′ x Ψ =				
	3.7	5.2	7.4	0.9	N/m2
Trubutory Width (bridge width), Bt	3				m
Tributory Length (between cross beams), Lt	3				m
Harmonic point load at cross beam	33.1	46.8	66.3	8.0	N
	0.0331	0.0468	0.0663	0.0080	KN
Comfort limit set, amax	1.00	1.80	2.50	2.50	m/s2
Acceleration (From SAP2000 Time-history analysis), a	1.25	1.77	2.51	0.32	m/s2

P [N]		
Vertical	Longitudinal	Lateral
280	140	35
Reduction coefficient ψ		
Vertical and longitudinal		Lateral
		
Equivalent number n^* of pedestrians on the loaded surface S for load model of:		
TC1 to TC3 (density d < 1,0 P/m²): $n^* = \frac{10.8\sqrt{\xi \times n}}{S}$ [m^{-2}]		
TC4 and TC5 (density $d \geq$ 1,0 P/m²): $n^* = \frac{1.85\sqrt{n}}{S}$ [m^{-2}]		

where ξ is the structural damping ratio and,
n is the number of the pedestrians on the loaded surface S ($n = S \times d$).

Figure 4-2 - Extract from ECCS Guide (Table 4-8) Parameters for load model of TC1 to TC5.

4.3. Lateral Vibrations

From Table 3-1 it can be observed that mode shape #2 (fv=1.10Hz) requires assessment for lateral pedestrian vibration effects. All other lateral cases are outside of the critical ranges defined by Section 4.2 of the ECCS Guide.

Mode 2 was tested for the first harmonic of pedestrian loading for varying user densities. Inputs/outputs are summarised in Table 4-2 below and full methodology is contained in Appendix B.

For transverse effects we have adopted the average viscous coefficient of critical damping, $\xi = 0.5\%$ for our steel superstructure (mast, cables, and superstructures are all steel). This value is slightly above the average as recommended in the ECCS Guide Table 4-5 in recognition of the bolted connections of the transverse bracing members.

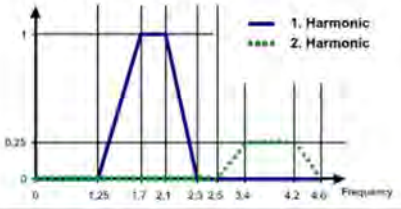
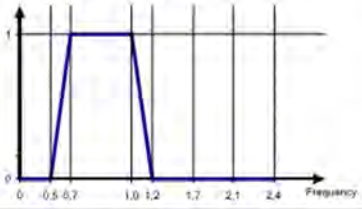
In summary:

Users / m^2	Total users	Measured Acceleration, m/s^2	Proposed Limit, m/s^2
0.10	27	0.078 (maximum)	0.30 (medium)
0.20	54	0.112 (medium)	0.55 (lower minimum)
0.40	108	0.157 (medium)	0.80 (upper minimum)

As can be observed from the summary, all of the crowd densities that were assessed for lateral excitation produce accelerations far smaller than the comfort criteria set in Table 2-1. In general the comfort for day-to-day use can be considered maximized. Even for the extremely rare occasion where the bridge experience 108 users the lateral comfort level only drops to a “medium” user comfort. On this basis the bridge can be considered a high performer when considering user comfort levels during lateral excitations.

Table 4-2 Summary of the Lateral Harmonic Load Modelling Approach applied to Mode 2

Direction considered	Lateral (Mode 2)			
Mode Shape	N=1			
Natural Frequency, Hz	1.10			Hz
Natural Period of Vibration, T	0.91			secs
Harmonic Type	FIRST HARMONIC LATERAL LOADING			
Damping Ratio, %	0.50%			
User Density, No./m^2	0.1	0.2	0.4	No./ m2
Applied single user load, N	35			N
Bridge Length, L	90			m
usable Brideg Width, B	3			m
Loaded surface, S	270			m2
Total number of users, n	27	54	108	No.
Synchronized Number of users, No/m^2	n' = 10.8*√(ξ x n) / S			
	0.01	0.02	0.03	No./ m2
% of users synchronized	15%	10%	7%	
Reduction coefficient, Ψ	0.50			
Synchronized load applied	35N x n' x Ψ =			
	0.26	0.36	0.51	N/m2
Trubutory Width (bridge width), Bt	3			m
Tributory Length (between cross beams), Lt	3			m
Harmonic point load at cross beam	2.315	3.274	4.630	N
	0.0023	0.0033	0.0046	kN
Comfort limit set, amax	1.00	1.80	2.50	m/s2
Transverse lock-in initiation	0.1 to 0.15			m/s2
Acceleration (From SAP2000 Time-history analysis), a	**0.078**	**0.112**	**0.157**	m/s2

P [N]		
Vertical	Longitudinal	Lateral
280	140	35
Reduction coefficient ψ		
Vertical and longitudinal		Lateral
		
Equivalent number n' of pedestrians on the loaded surface S for load model of:		
TC1 to TC3 (density d < 1,0 P/m²): $n' = \frac{10.8\sqrt{\xi \times n}}{S}$ [m^{-2}]		
TC4 and TC5 (density d ≥ 1,0 P/m²): $n' = \frac{1.85\sqrt{n}}{S}$ [m^{-2}]		

where ξ is the structural damping ratio and,
n is the number of the pedestrians on the loaded surface S ($n = S \times d$).

Figure 4-3 - Extract from ECCS Guide (Table 4-8) Parameters for load model of TC1 to TC5.

4.4. Lateral "Lock-in"

> *In recent years some footbridges were excited laterally by dense pedestrian streams in which pedestrian motion and bridge vibration were strongly coupled, in which case a self-excited response of large amplitude may take place and cause discomfort. Footbridges should be designed in such a way that this pedestrian-bridge-interaction phenomenon, also called 'lock-in', does not arise.*
>
> *Design of Lightweight Footbridges for Human Induced Vibrations* (JRC-ECCS, 2009)

Whilst also considering general comfort we must also consider "lateral Lock-in". The phenomena of lateral lock-in is synonymous with the opening of the Millennium Footbridge in London (year 2000) whereby the footfall induced vibration caused by large crowds (on opening day in the case of the Millennium Footbridge) can lead to significant side-to-side sway motion that continues to increase and cause significant discomfort and alarm for users. This situation of structure-pedestrian interaction is caused by users changing their step frequency and width (gait) to counter-act the sway motion and by doing so synchronize with both the bridge motion and one-another. Where the sway is at a particular frequency consistent with the lateral walking frequency (1/2 the vertical step frequency) this can then lead to an increase in movement as the two actions self-excite each other.

Lateral lock-in is expected to initiate somewhere in the range of 0.1m/s^2 and 1.5m/s^2 and over a frequency range of 0.5 to 1.2Hz. As can be observed from Table 4-2 the occurrence of a 0.1m/s2 and 0.15m/s2 is expected to occur in the region of 50 to 100 users respectively.

However, in reviewing the lateral acceleration data (refer Figure 4-4 below) it should be noted that it takes 46 seconds of resonating at the bridge's natural frequency for the 52 people (user density, D=0.2) to generate an acceleration in excess of 0.1m/s^2. Table NA.8 of the UK Eurocode 2 estimates an average walking pace of 1.7m/s. On this basis it would take 53 seconds to walk the entire bridge. To maintain the estimated user density for the duration of the analysis it is assumed that the same number of people must be entering the bridge as leaving the bridge. Thus to maintain "***a continuous stream of people totaling 54 people walking on the bridge in the same instance for 46 seconds we require a continuous flow of 100 people***" derived from (1+46/53) x 54 users = 100 users[3]. It should also be noted that this would be at the onset of "lock-in" at a single point on the bridge. For more wide-spread lock-in to occur (i.e. larger portions of the bridge exceeding 0.1m/s^2) further time, and thus an increase in the total number of users in the steady stream, would be required. Since 108 people has been determined as an upper limit for the credible number of visitors to the bridge, "the first possible onset of lock-in" occurring at 100 people seems reasonable.

If we consider the upper range of 0.15m/s^2 for lock-in activation the required number of visitors required in the continuous stream jumps to (1+50/53) x 108 users = 210 people. This is assuming 108 people initiate 0.15m/s^2 of lateral accelerations at 50 seconds as per Figure 4-6 below.

[3] By way of example the number of users needed to excite the bridge for 53 seconds (time it takes to walk the full length) would be twice the number assumed on the bridge at any one time.

On the basis that lock-in will require a steady stream of 100 to 200 visitors prior to the start of the lock-in phenomena occurring, the bridge is considered suitable.

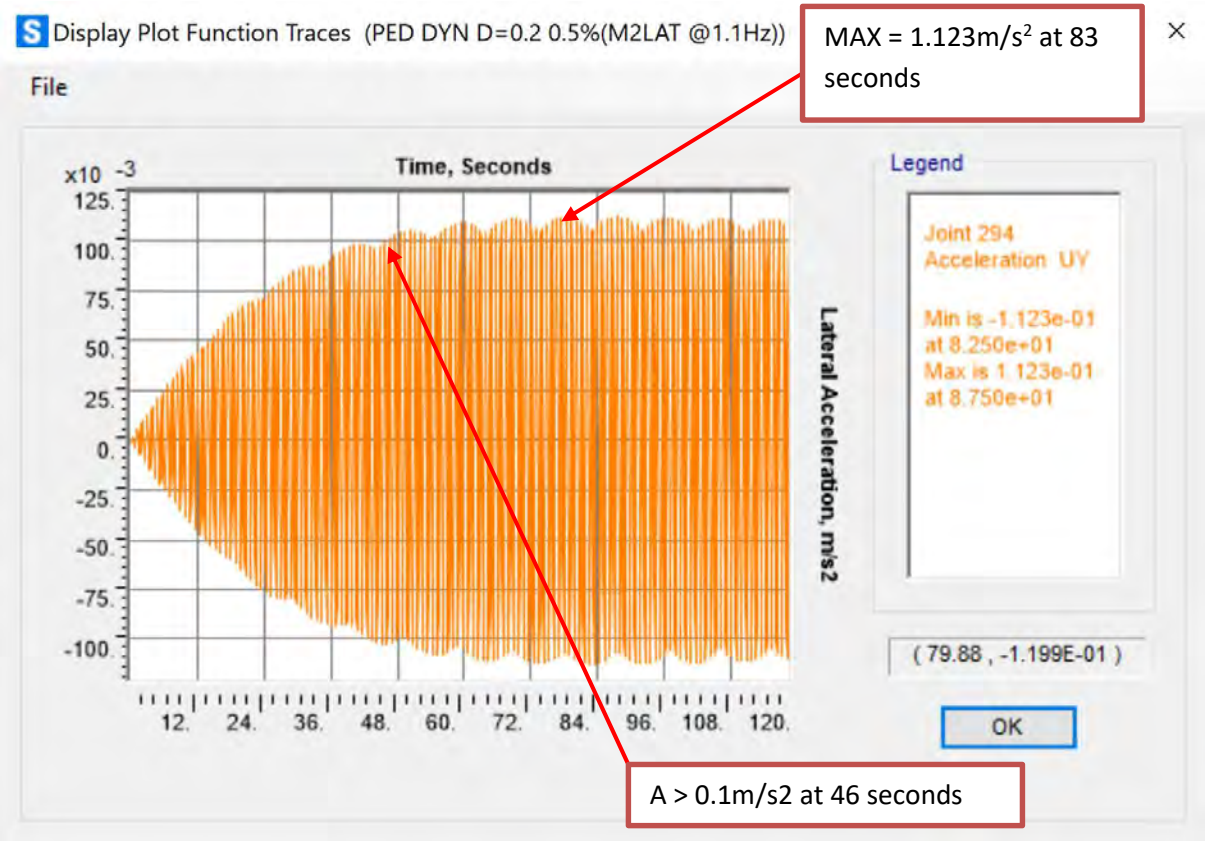


Figure 4-4 – Lateral Accelerations for 54 people at worse-case point on bridge (from SAP2000 Non-linear Time-history Analysis).

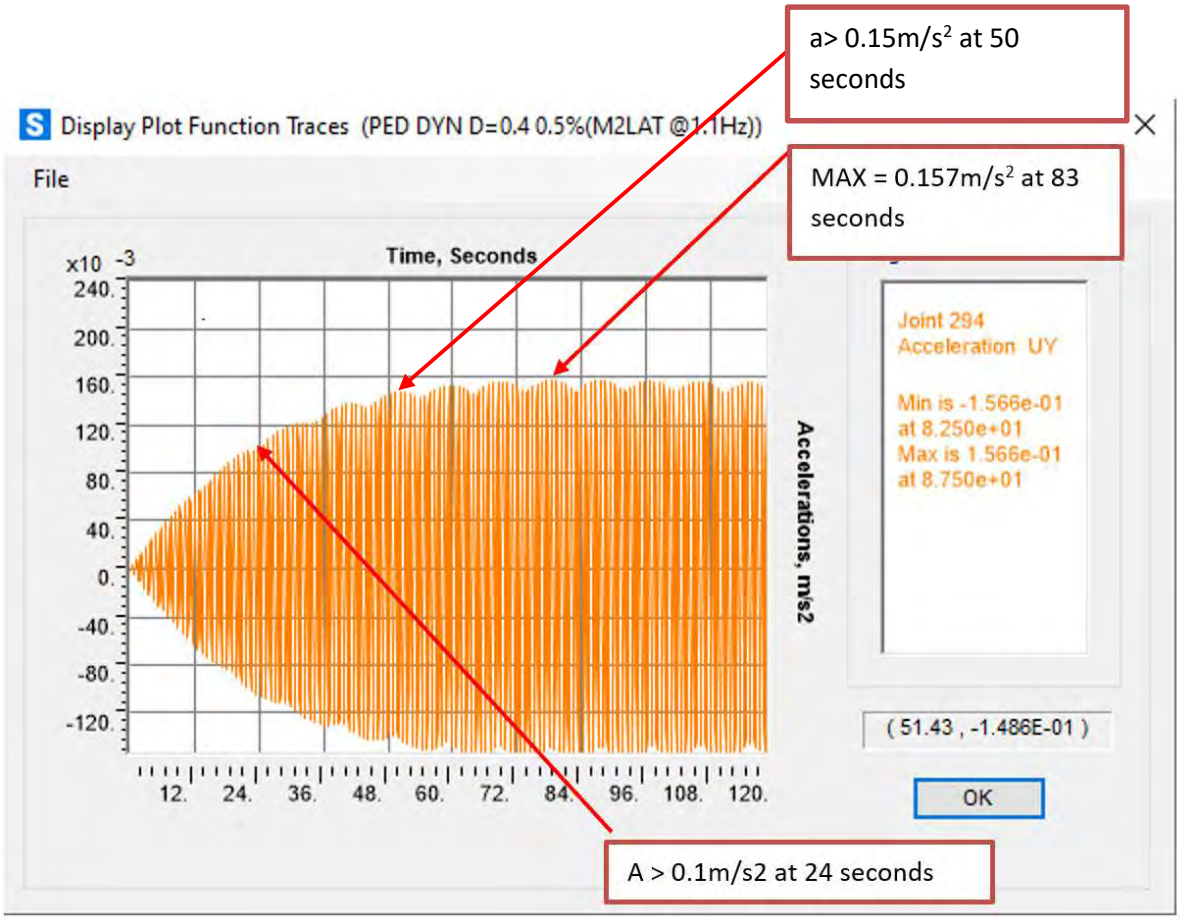


Figure 4-5 – Lateral Accelerations for 108 people at worse-case point on bridge (from SAP2000 Non-linear Time-history Analysis).

6. Summary & Conclusions

Assessment Approach

Pedestrian dynamics are considered based on the detailed methods discussed in the ***Design of Lightweight Footbridges for Human Induced Vibrations*** (JRC-ECCS, 2009) referred to herein as the "ECCS Guide"

For assessment of the Longford Cable-stay Bridge Mode #2 (lateral N=1 mode shape) and Mode #3 (vertical N=2 mode shape) have been identified as the primary modes of concern. All other cases are either outside of the critical frequency ranges defined by Section 4.2 of the ECCS Guide or as per Mode #5 (vertical N=3) generate accelerations well below the set criteria.

The Harmonic Load Modelling (HLM) approach (applied to our FE analysis model) has been used in order to assess accelerations under various different user densities. The method is described in detail in Section 4.5.1.2 of the ECCS Guide. We have adopted comfort criteria as set out by Table 4-4 of the ECC Guide. All crowd densities considered achieve the comfort criteria set:

Vertical Assessment at Mode #3 (N=2 @ 1.78Hz):

Users / m^2	Total users	Measured Acceleration, m/s^2	Proposed Limit, m/s^2
0.10	27	1.25 (medium)	1.00 (medium)
0.20	54	1.77 (lower minimum)	1.80 (lower minimum)
0.40	108	2.51 (upper minimum)	2.50 (upper minimum)

As can be observed from the data, the 0.1 user density (27 people) doesn't quite achieve the "medium" comfort level we were targeting and our maximum user density of 0.4 is right on the limit of what would be deemed acceptable. In recognising that the limits set are about defining the overall user experience it should be noted that the values summarised above represent the absolute worse-case acceleration which is only applicable to finite points along the bridge. The majority of the bridge easily achieves the intended comfort levels (refer Figure 4-1). It is only a very small portion of the bridge that will subject the users to a less than optimal experience. In the pursuit of economic solutions for GDC, and noting all values are within the "acceptable" limits, this is regarded as a good outcome without the need for significantly modifying the bridge response or adding tuned mass dampers.

Lateral Assessment at Mode #2 (N=1 @ 1.10Hz):

Users / m^2	Total users	Measured Acceleration, m/s^2	Proposed Limit, m/s^2
0.10	27	0.078 (maximum)	0.30 (medium)
0.20	54	0.112 (medium)	0.55 (lower minimum)
0.40	108	0.157 (medium)	0.80 (upper minimum)

As can be observed from the summary, all of the crowd densities that were assessed for lateral excitation produce accelerations far smaller than the comfort criteria set in Table 2-1. On this basis the bridge can be considered a high performer when considering user comfort levels during lateral excitations.

Lateral Lock-in Assessment:

Lateral lock-in is expected to initiate somewhere in the range of $0.1m/s^2$ and $1.5m/s^2$ as discussed in Section 4.6 of the ECCS Guide.

In reviewing the lateral acceleration data it was noted that it takes 46 seconds of resonating at the bridge's natural frequency for the 52 people (user density, D=0.2) to generate an acceleration in excess of $0.1m/s^2$. Table NA.8 of the UK Eurocode 2 estimates an average walking pace of 1.7m/s. On this basis it would take 53 seconds to walk the entire bridge. To maintain the estimated user density for the duration of the analysis it is assumed that the same number of people must be entering the bridge as leaving the bridge. Thus, to maintain "***a continuous stream of people totaling 54 people walking on the bridge in the same instance for 46 seconds we require a continuous flow of 100 people***"

If we consider the upper range of $0.15m/s^2$ for lock-in activation the required number of visitors required in the continuous stream jumps to 210 people. This is based on the assessment that 108 people initiate $0.15m/s^2$ of lateral accelerations at 50 seconds.

On the basis that lock-in will require a steady stream of 100 to 200 visitors prior to the start of the lock-in phenomena occurring, the bridge is considered suitable.

As-built testing

It should be noted that dynamic assessment as contained in this report is a good step in preventing unwanted serviceability performance. However, it is noted that since all dynamic assessment is based on estimated material properties (stiffness, damping, etc.) unforeseen behaviour structural and dynamic can still occur. If during operation unwanted movements are experienced, further study on the as-built structure should be undertaken.

APPENDIX A
Vertical Harmonic Load Modelling
1.78Hz & ξ = 0.4%

Vertical Harmonic Load Modelling 1.78Hz & ξ = 0.4%

Applied loading is based on Eq 4-15 of ECCS Guide:

$p(t) = P \times \cos(2\pi f_s t) \times n' \times \Psi$

Since cross beams are spaced at 3m centers we have applied a coincidental cosine harmonic load of 3.0m wide x 3.0m spacing x 7N/m^2 = 66 N at each cross beam centre along the bridge.

$p(t) = 66N \times \cos(2\pi \times 1.7Hz \times t)$

$p(t) = 0.066kN \times \cos(2\pi \times 1.7Hz \times t)$

The first task is to assign a vertical unit load in the direction that corresponds with the mode shape. The vertical mode is an N=2 mode shape, thus unit loads have been applied as shown in the Figure below.

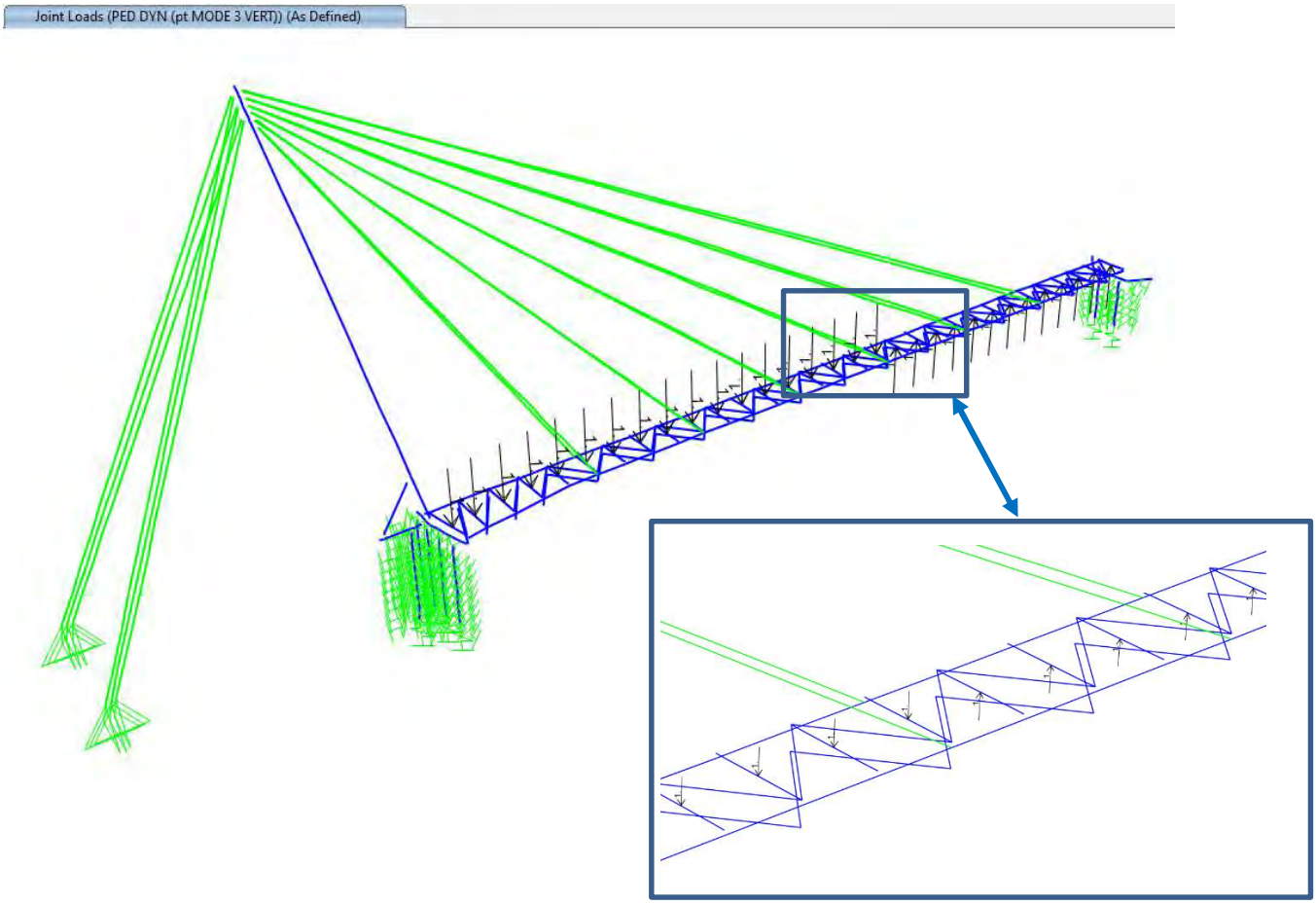


Figure A1 – Unit load. Direction applied to match the N=2 vertical mode shape being considered.

Following application of the unit load, a time-history co-sine function is then setup to match the vertical bending period (1/f = 0.56 seconds) and the amplitude e.g. 0.066kN for this example. See Figure below. The step frequency is assumed equal to the bridge natural frequency = 1.78Hz (e.g. 1.78 steps per second).

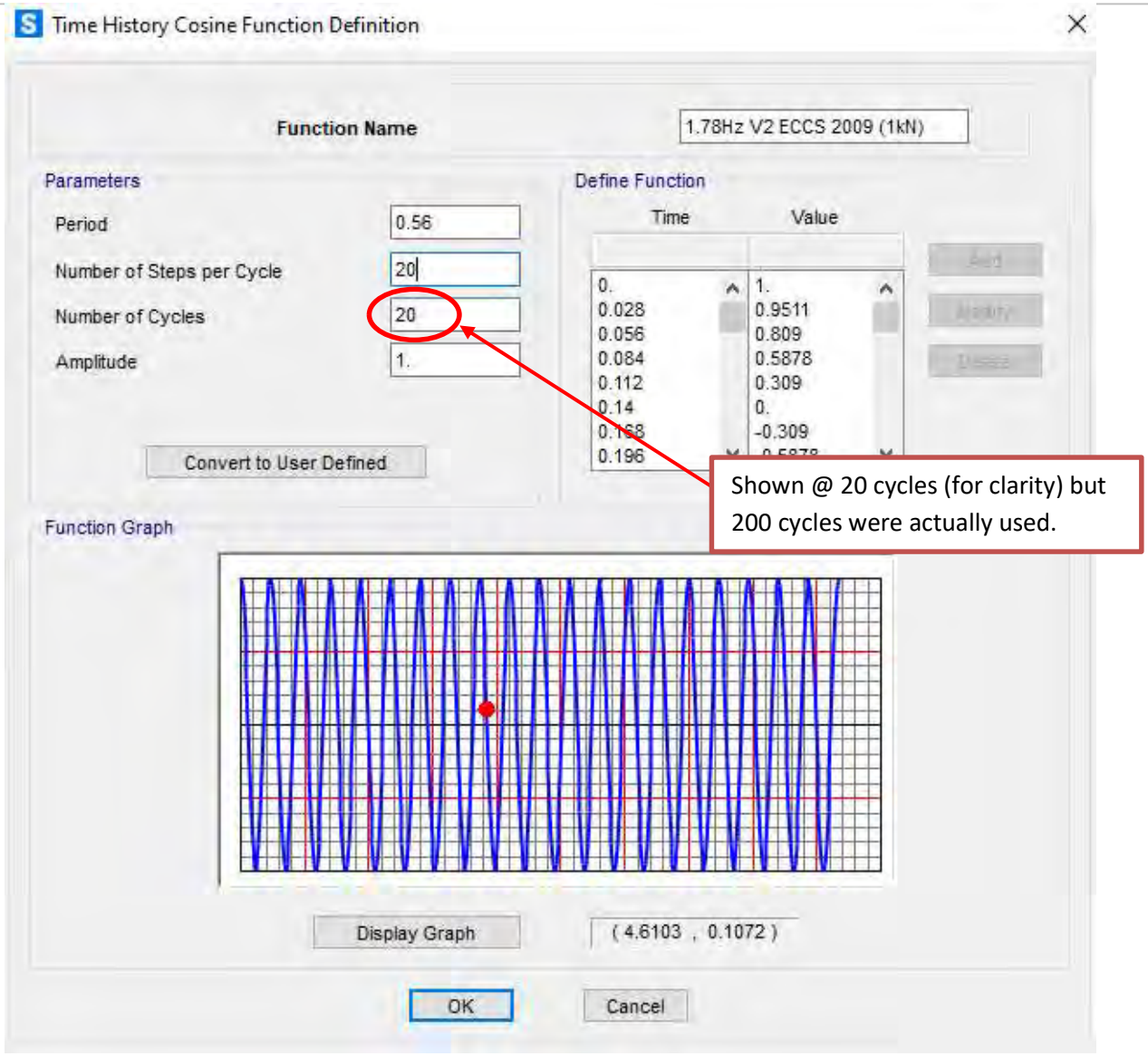


Figure A2 – Applied SAP2000 time-history harmonic load (200 cycles of synchronization ≈ 120 seconds)[4].

Using these time history-inputs we create the non-linear time-history load-case in SAP 2000 v22. In this setup we can set the duration and the damping ratio. SAP2000 is based on a critical damping coefficient input (not logarithmic decrement). See figure below.

[4] Note that "steps" in the SAP2000 interface are "analysis steps" not actual footsteps.

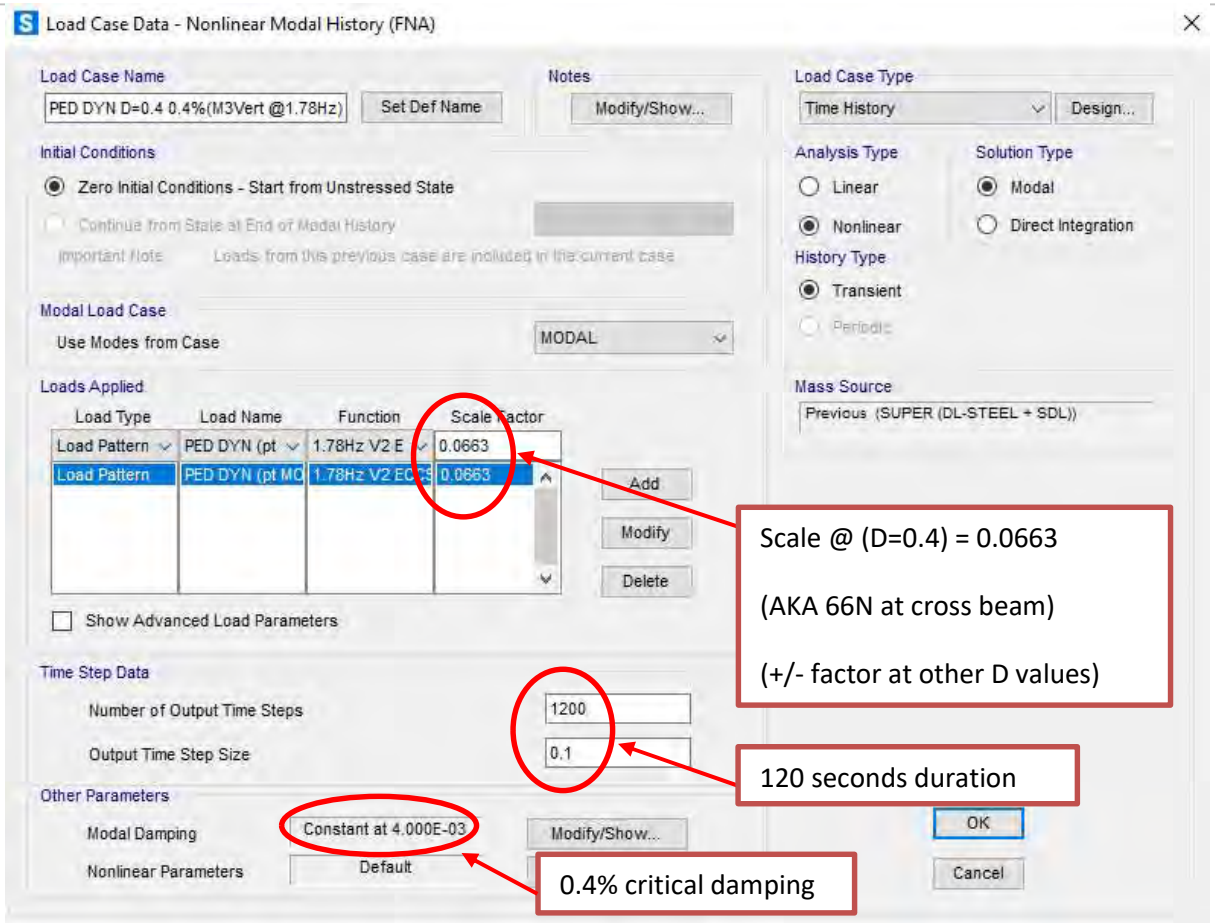


Figure A3 – SAP2000 time-history load-case.

Once this loadcase is established we then analyze the system and check the deflection and accelerations.

The series of images below shows the start of the bridge as it begins to oscillate and increase in deflection.

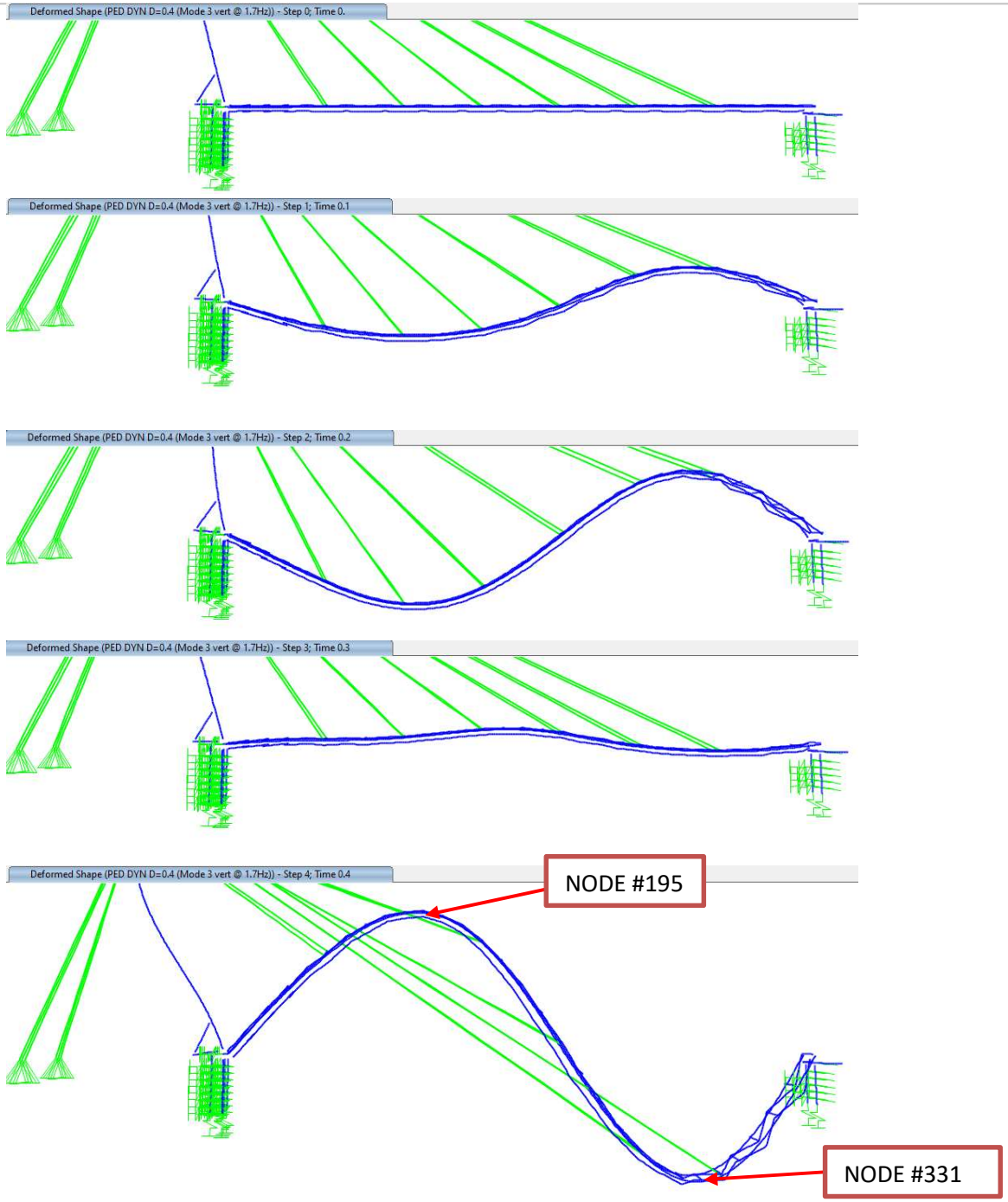


Figure A4 – SAP2000 time-history deflection for t = 0.2, 0.3, 0.4, 0.6 seconds (scale = 50,000)

Using the SAP2000 plot functions we can plot the required nodal deflections and accelerations.

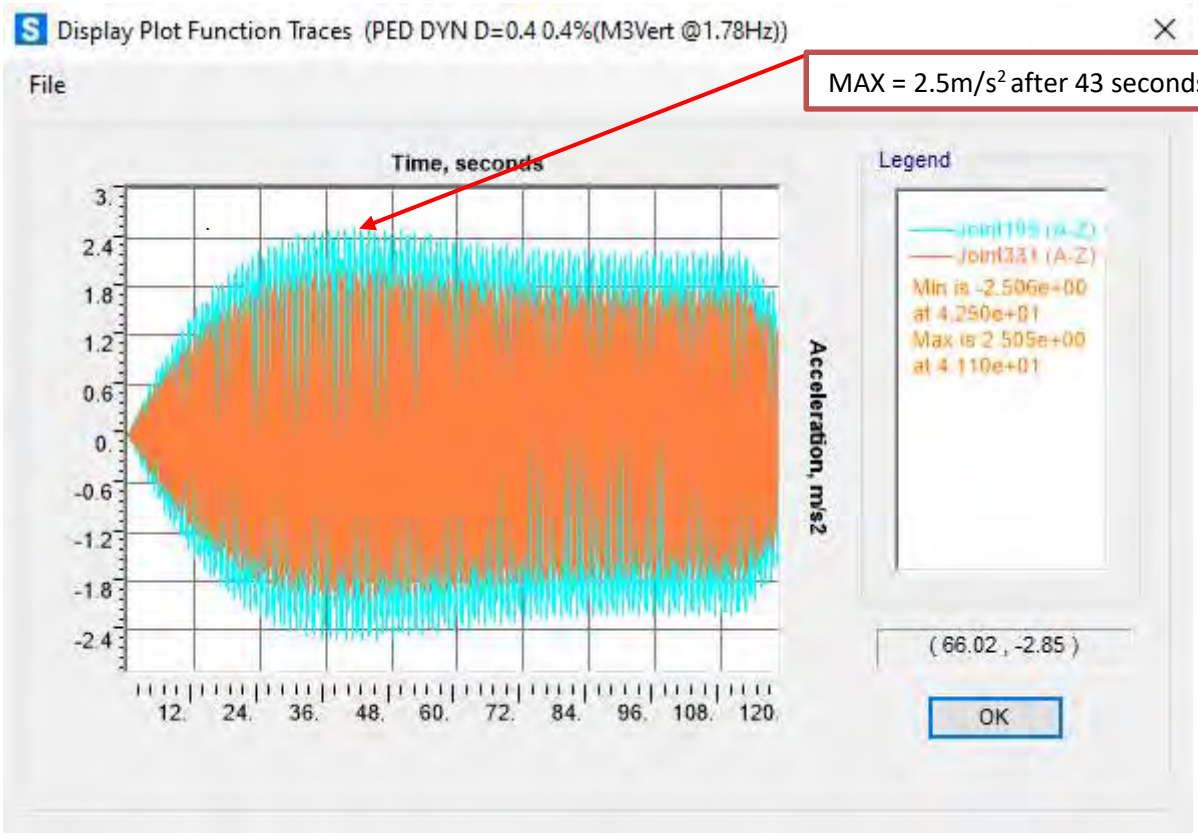


Figure A5 – Accelerations: blue at Node #195, orange at node #331

APPENDIX B
Lateral Harmonic Load Modelling
1.10Hz & ξ = 0.4%

Lateral Harmonic Load Modelling 1.10Hz & ξ = 0.4%

Applied loading is based on Eq 4-15 of ECCS Guide:

$p(t) = P \times \cos(2\pi f_s t) \times n' \times \Psi$

Since cross beams are spaced at 3m centers we have applied a coincidental cosine harmonic load of 3.0m wide x 3.0m spacing x 0.46N/m^2 = 4 N at each cross beam centre along the bridge.

$p(t) = 4.6N \times \cos(2\pi \times 1.1Hz \times t)$

$p(t) = 0.0046kN \times \cos(2\pi \times 1.1Hz \times t)$

The first task is to assign a horizontal unit load in the direction that corresponds with the mode shape. The lateral mode is an N=1 mode shape, thus unit loads have been applied as shown in the Figure below.

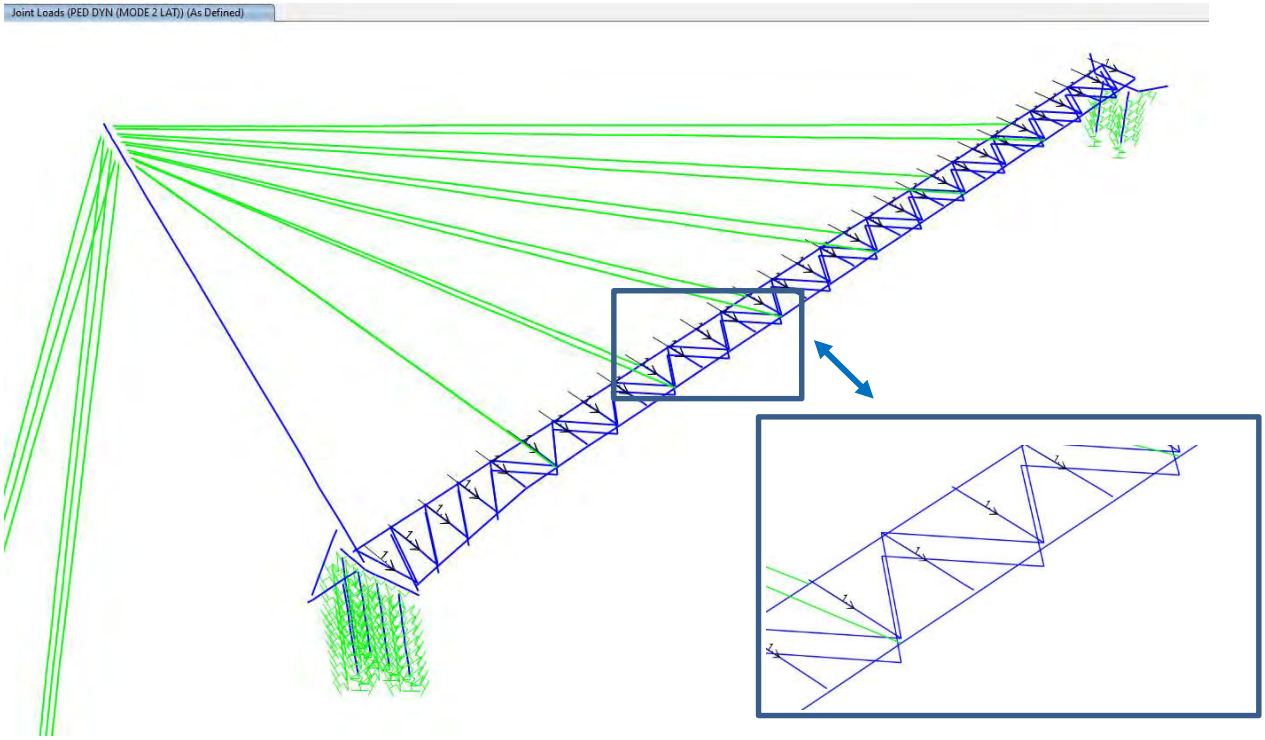


Figure B1 – Unit load. Direction applied to match the N=1 lateral mode shape being considered.

Following application of the unit load, a time-history co-sine function is then setup to match the transverse bending period (1/f = 0.91 seconds) and the amplitude e.g. 0.0046kN for this example. See Figure below. The step frequency is assumed equal to the bridge natural frequency = 1.1Hz (e.g. 1.1 steps per second).

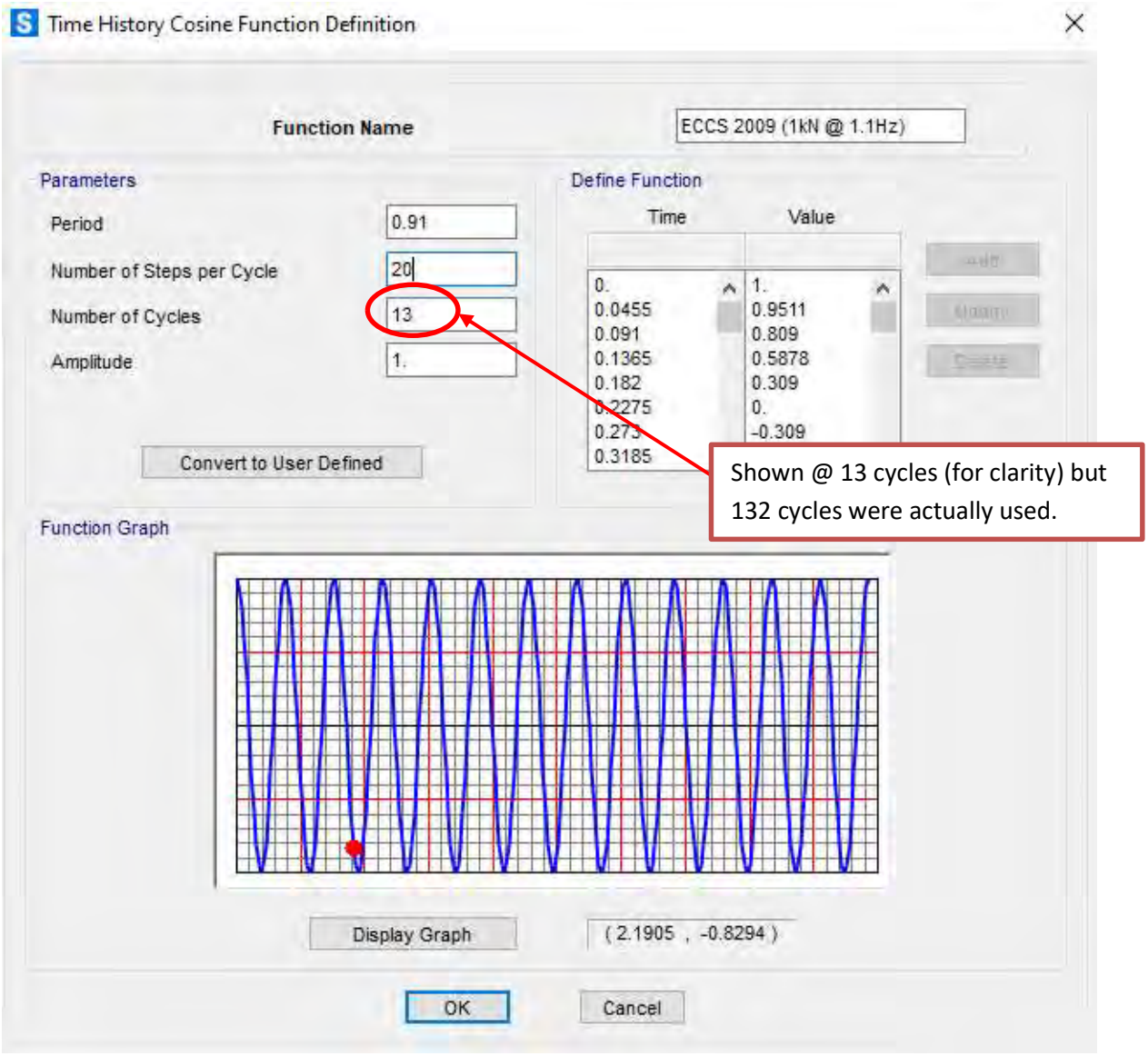


Figure B2 – Applied SAP2000 time-history harmonic load (132 cycles of synchronization ≈ 120 seconds)[5].

Using these time history-inputs we create the time-history load-case in SAP 2000. In this setup we can set the duration and the damping ratio. SAP2000 is based on a critical damping coefficient input (not logarithmic decrement). See figure below.

[5] Note that "steps" in the SAP2000 interface are "analysis steps" not actual footsteps.

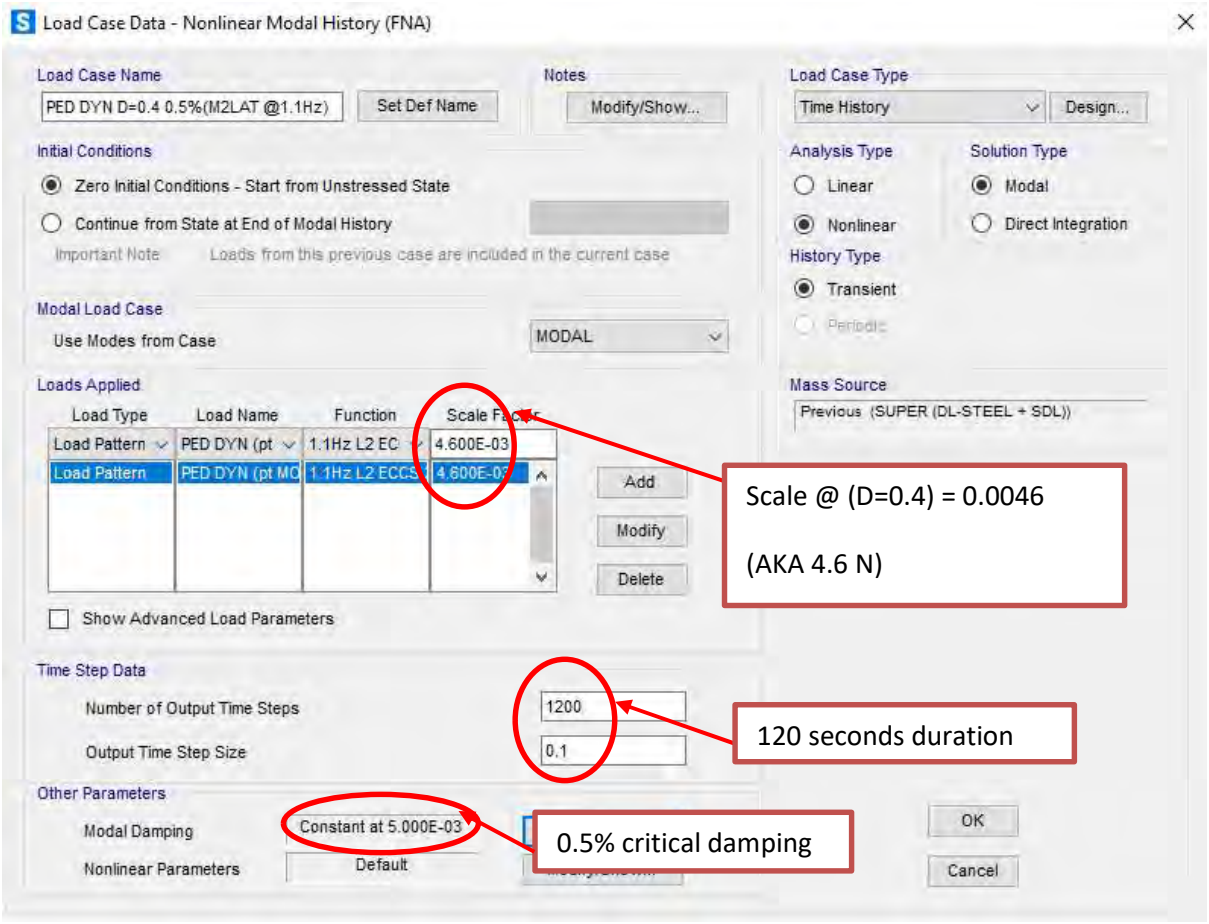


Figure B3 – SAP2000 lateral time-history load-case.

Once this loadcase is established we then analyze the system and check the deflection and accelerations.

The series of images below shows the start of the bridge as it begins to oscillate and increase in deflection.

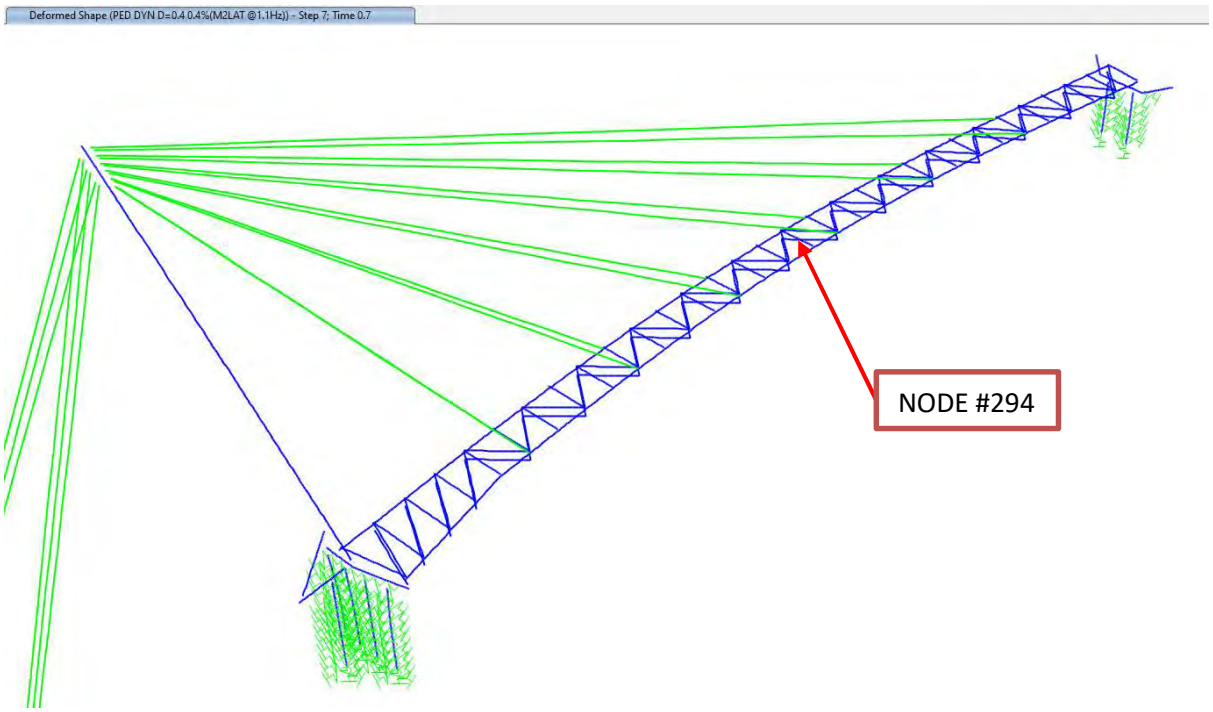


Figure B4 – SAP2000 time-history deflection for t = 0.7 seconds (scale = 50,000)

Using the SAP2000 plot functions we can plot the required nodal deflections and accelerations.

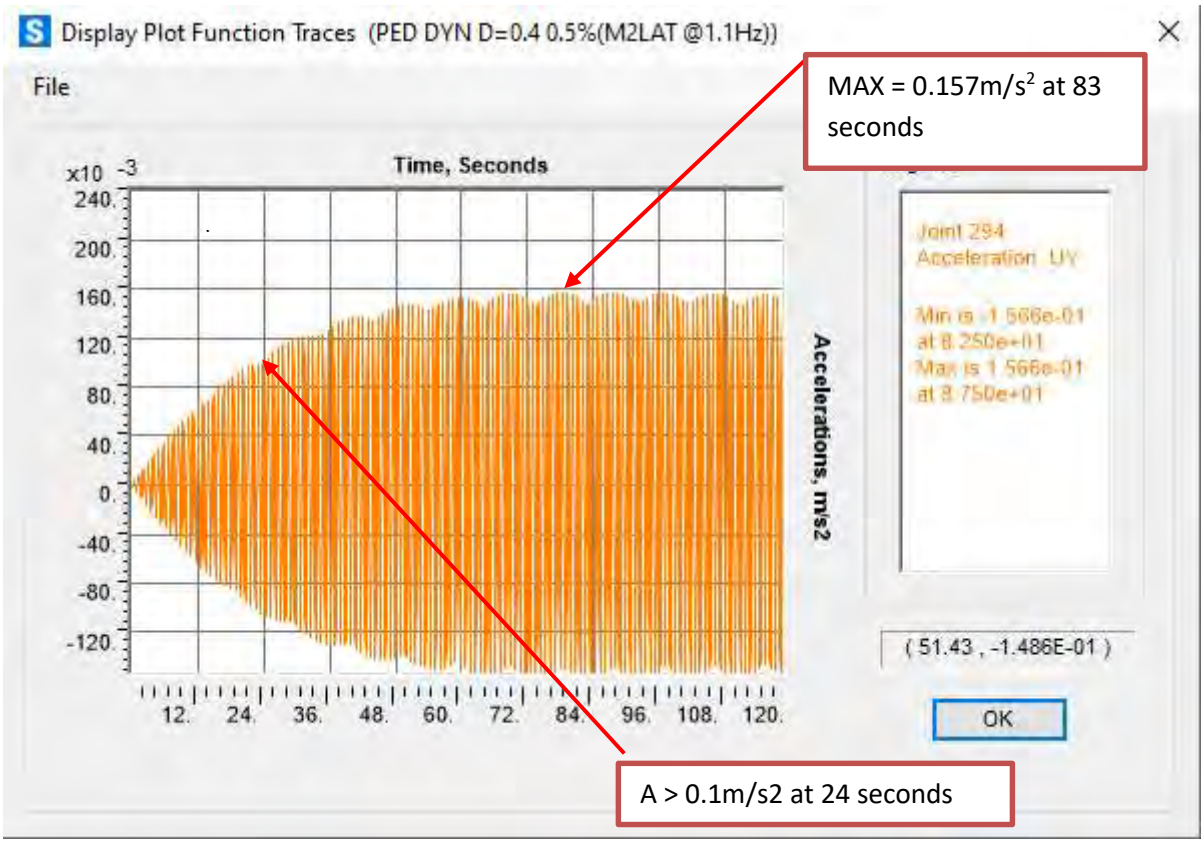


Figure B5 – Accelerations: orange at Node #294

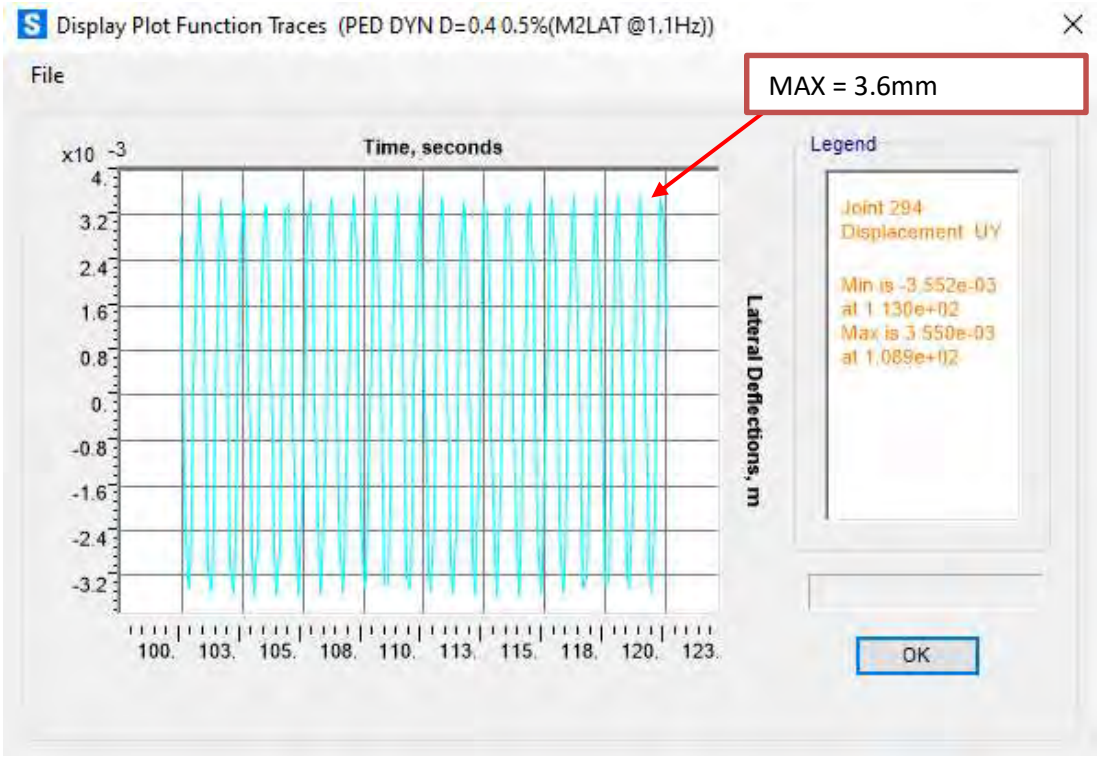


Figure B6 – Deflections: blue at Node #195, red at node #331

APPENDIX C
Vertical Harmonic Load Modelling
2.93Hz & ξ = 0.4%

Vertical Harmonic Load Modelling 2.93Hz & ξ = 0.4%

Applied loading is based on Eq 4-15 of ECCS Guide:

$p(t) = P \times \cos(2\pi f_s t) \times n' \times \Psi$

Since cross beams are spaced at 3m centers we have applied a coincidental cosine harmonic load of 3.0m wide x 3.0m spacing x 0.6N/m^2 = 8 N at each cross beam centre along the bridge.

$p(t) = 8N \times \cos(2\pi \times 1.7Hz \times t)$

$p(t) = 0.008kN \times \cos(2\pi \times 1.7Hz \times t)$

The first task is to assign a vertical unit load in the direction that corresponds with the mode shape. The vertical mode is an N=3 mode shape, thus unit loads have been applied as shown in the Figure below.

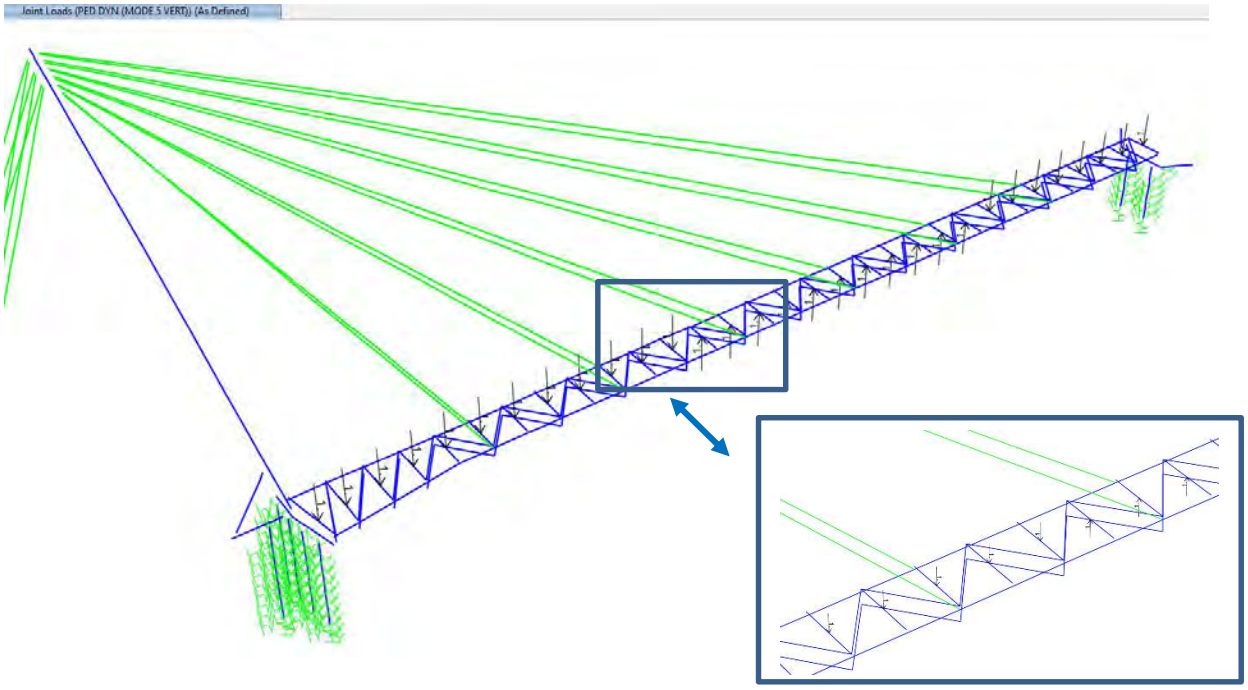


Figure C1 – Unit load. Direction applied to match the N=3 vertical mode shape being considered.

Following application of the unit load, a time-history co-sine function is then setup to match the vertical bending period (1/f = 0.34 seconds) and the amplitude e.g. 0.008kN for this example. See Figure below. The step frequency is assumed equal to the bridge natural frequency = 2.93Hz (e.g. 2.93 steps per second).

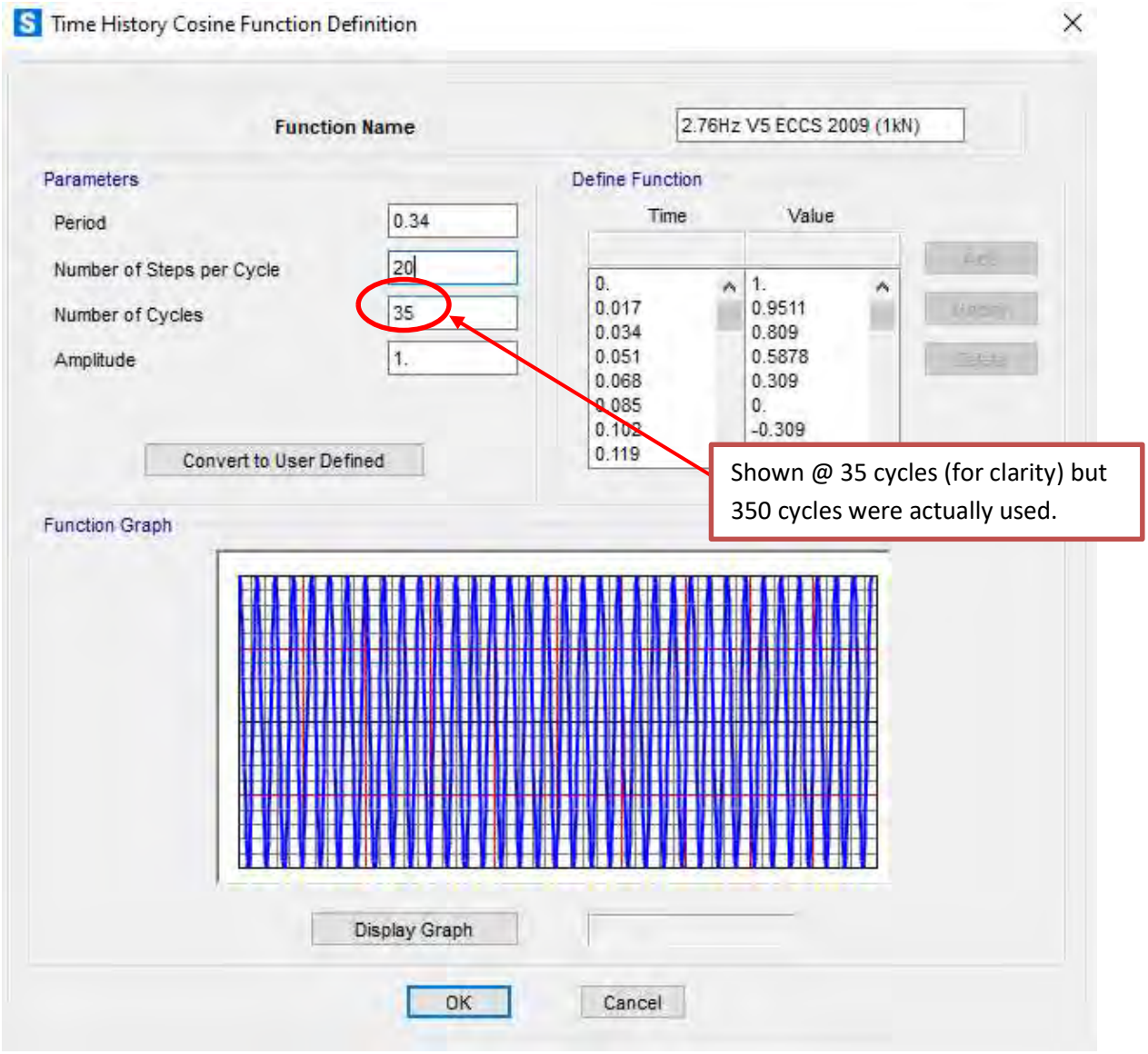


Figure C2 – Applied SAP2000 time-history harmonic load (350 cycles of synchronization ≈ 120 seconds)[6].

Using these time history-inputs we create the time-history load-case in SAP 2000. In this setup we can set the duration and the damping ratio. SAP2000 is based on a critical damping coefficient input (not logarithmic decrement). See figure below.

[6] Note that "steps" in the SAP2000 interface are "analysis steps" not actual footsteps.

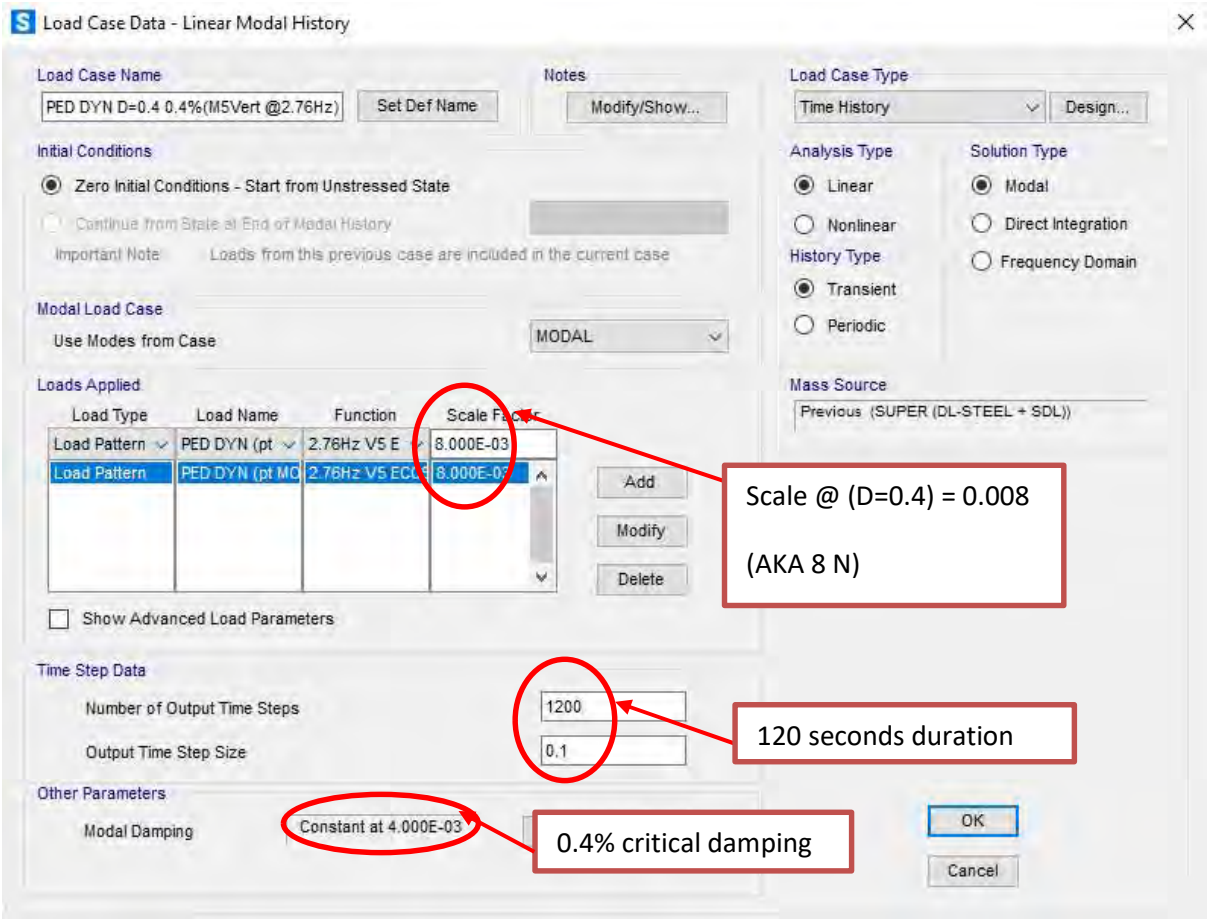


Figure C3 – SAP2000 time-history load-case.

Once this loadcase is established we then analyze the system and check the deflection and accelerations.

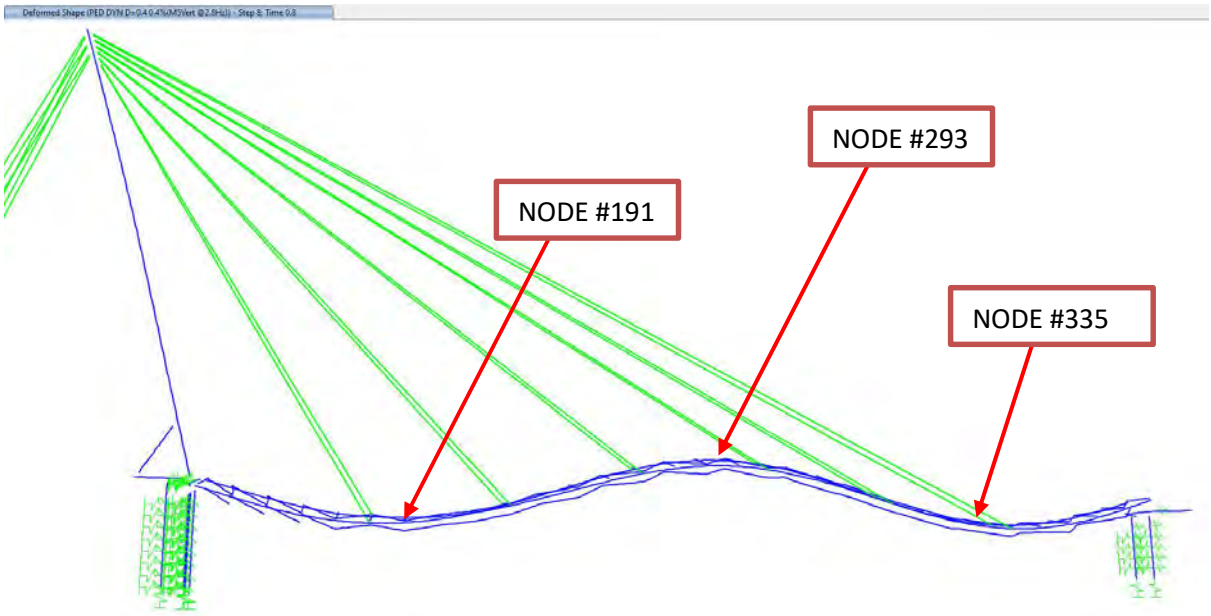


Figure C4 – SAP2000 time-history deflection for t = 0.8 seconds

Using the SAP2000 plot functions we can plot the required nodal deflections and accelerations.

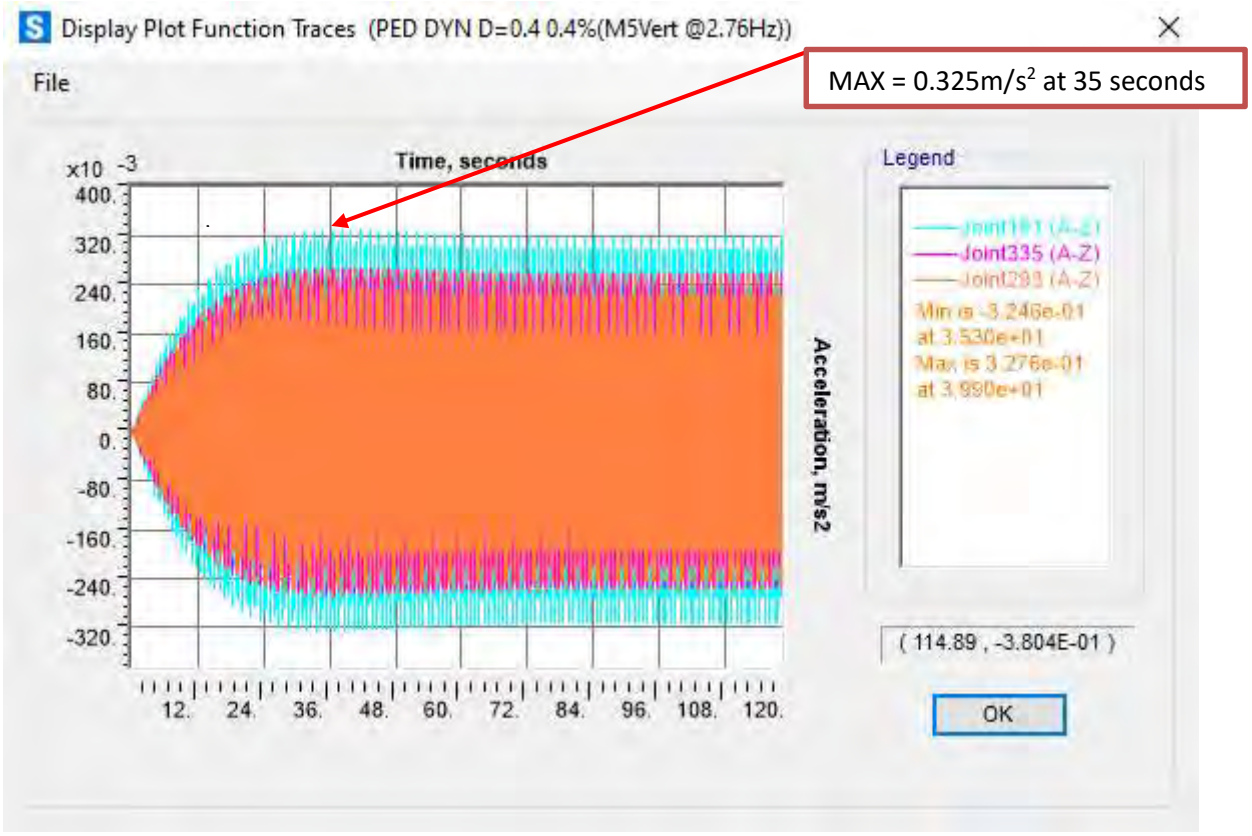


Figure C5 – Accelerations: blue at Node #191, purple at node #335, orange at node #293

APPENDIX C
INDEPENDENT CFD WIND STUDY

DC Structures – Longford Bridge Aerodynamic Analysis

1. Summary

An aerodynamic analysis has been performed on the proposed design of the Longford pedestrian and cycle cable stayed bridge, Gore, New Zealand, based on elevated risk factors identified in the Highways Design Manual.

Transient OpenFOAM CFD has been conducted with optimised settings on two bridge design options to assess the mean aerodynamic loads and extract the frequency components. The CFD suggests that neither bridge exhibits significant Strouhal Instability, with the primary aerodynamic mode shedding a couplet of two small vortices. Playing this data back through a mass-spring-damper model of the bridge calculated from supplied structural sensitivities shows small deflections with a damping stability margin ratio of at least 5.

The key difference between the two bridge designs is that one is porous between the deck and beams, allowing communication between the two to reduce the vertical aerodynamic load. This also slightly reduces the oscillation but does increase the speed of the air over the bridge by 4% which may affect users.

A range of wind speeds from 9.3 to 44m/s have been simulated; the worst case for the bridge is 9.3m/s which is predicted to cause a large amplitude vertical oscillation, but does not exhibit flutter instability in any axis. Suggestions have been made as to how to address the large amplitude oscillation if this will cause user discomfort. At a mid wind speed of 21m/s a range of torsionally deflected bridge profiles have been simulated and show the bridge to be torsionally stable.

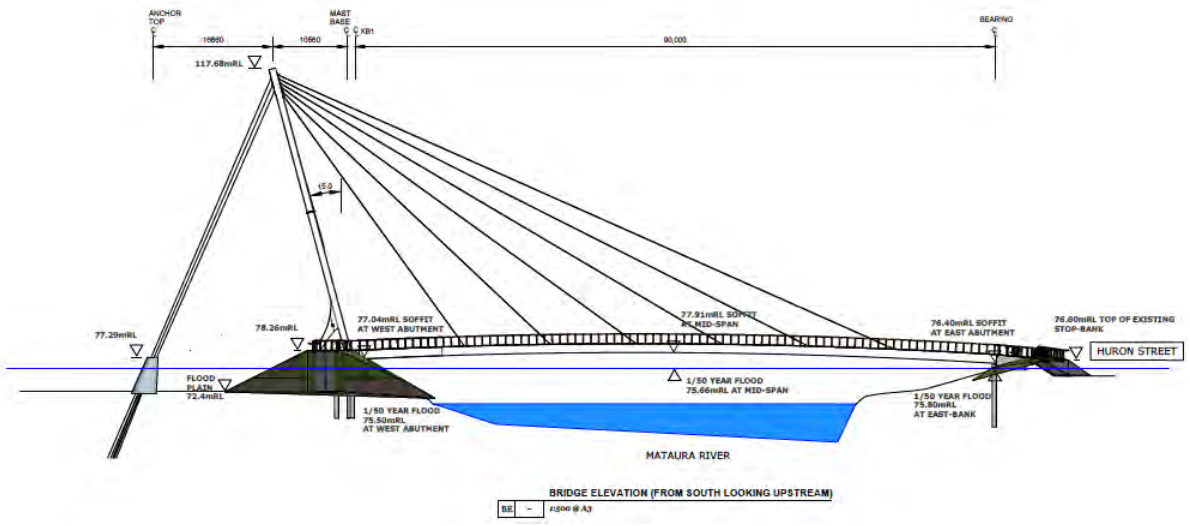
This is only an aerodynamic investigation of the beam-deck structural box, the CFD is not coupled with the structure, and no FEA has been conducted to assess how the energy is transferred between modes using the CFD derived aerodynamic forcing function. This study is comparable to a solid model wind tunnel test, which can identify aerodynamic instability, but further investigation should be performed if there are indications of a coupled aero-structural mode. No analysis has been conducted on the mast or cables, where aerodynamic instability could also occur.

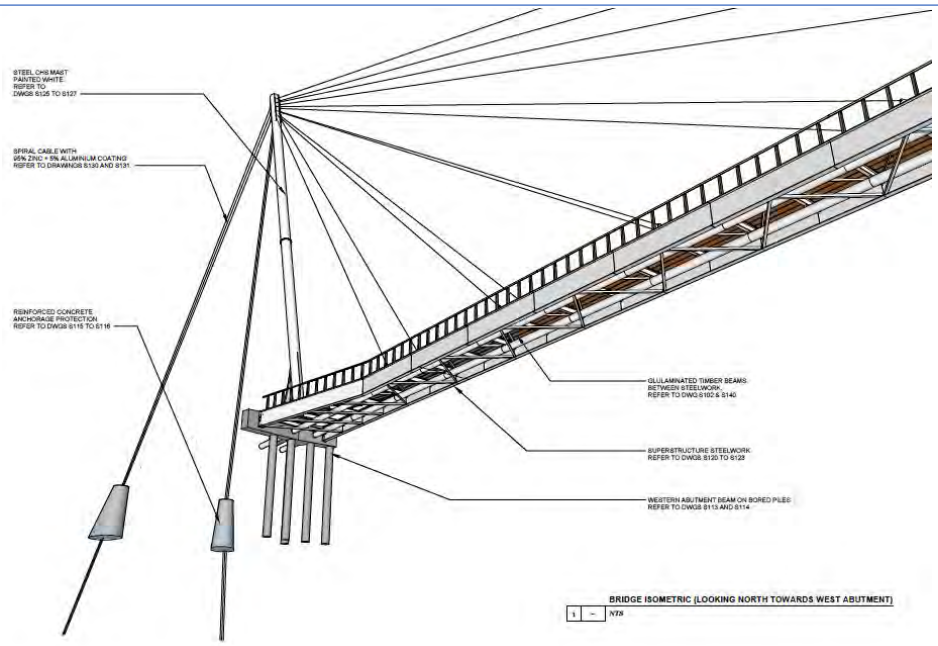
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3. Background

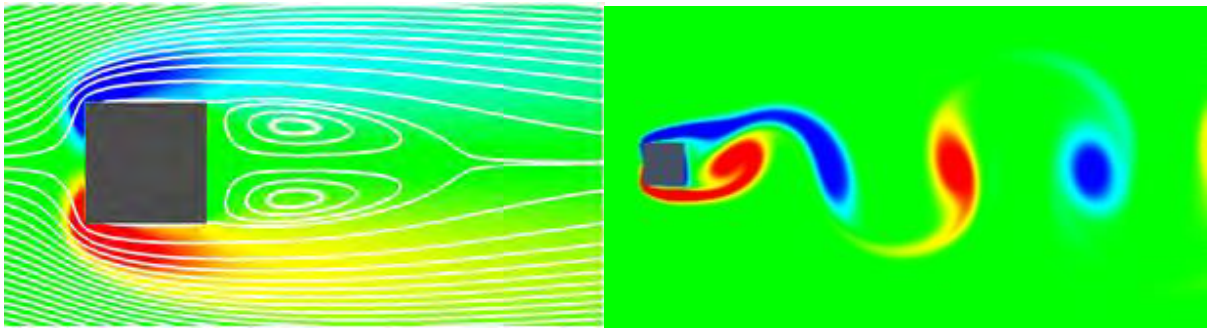
The Longford foot and cycle bridge, over the Mataura River in Gore, New Zealand, is in the process of being designed. According to Highways Design Manual (section DB49) it is at risk of aerodynamic and fluid-structure interaction (flutter) instability.





The aerodynamic risk comes principally from Strouhal instability, with large scale aerodynamic vortices forming an alternating pattern of loading called a Von Karman Vortex street. Typically, only forming at Reynolds Numbers (Re) of above 40, this is expected to correlate to the usual working range of the bridge. This analysis only considers the beam-deck structural box, and not the mast or cables, which can be subject to the same aerodynamic phenomena.

Low Re **High Re**

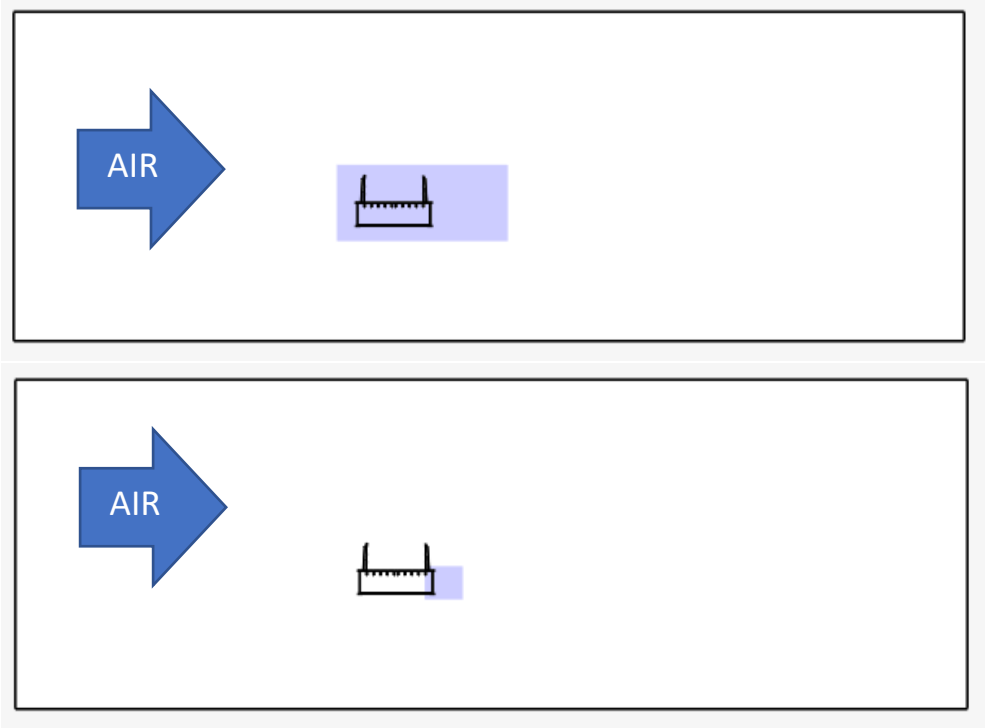


4. Methodology

4.1 Computational Fluid Dynamics

CFD has been conducted using the Simscale platform, starting from the suppliers recommended settings and optimising the model refinement and finite volume mesh to achieve the most detailed and repeatable result. This transient CFD simulation allows for the estimation of unsteady forces and moments on the bridge, but it is not a coupled structural solver so each case holds the bridge surfaces steady and simply shows the time variance of the aerodynamic flow field.

- CAD for each bridge deck has been supplied as a 2D .dxf for extrusion, inspection of the images provided also shows some regular repeating patterns of supports and bracing which may be significant.
- The bridge has been modelled at full scale so that velocities, forces and frequencies do not have to be scaled.
- It is required to simulate a 30m section of an infinite bridge, rather than a particular bridge in-situ, this is modelled with symmetry boundary conditions on the side walls of the fluid volume.
- All forces and moments reported are for the 30m mid section, and so need to be divided by 30 to convert to per metre length values.
- The top and rear wall boundary conditions are total pressure inlet-outlets to allow a natural far field flow to exist without very large volumes and computational overheads.
- The bridge is expected to sit 6m above the usual water surface, this is modelled as a stationary ground plane, so will develop a boundary layer, and with a height to chord ratio of only 1.4, is expected to influence the flow over the bridge.
- The Highways Design Manual assesses a risk of aerodynamic excitation in the range of 8-33m/s, in an area with likely wind speeds of up to 44m/s. Initial simulations have been run at 21m/s as a mid-point and likely upper limit for pedestrian use, to assess the frequencies and magnitudes of the aerodynamic oscillations and identify the conditions of interest. This air speed is applied as a uniform profile on the upwind face of the flow volume and is maintained constant throughout the simulation.
- To ensure the ground plane boundary layer is sufficiently developed, and the full wake of the bridge is explored the fluid volume extends 18m upwind and 28m downwind of the bridge, and 10m above the deck.
- A parametric hex mesh is then generated for each option, with 3 levels of refinement cells added in an inner fluid volume where large gradients are expected (grey rectangle in first image below, where the outer box is the whole fluid volume), and a fourth level of refinement on the rear corner shedding edges (grey square in second image) which are expected to dominate the flow field.



- OpenFOAM PIMPLE transient CFD simulations have been run over at least 2s (1s of fully developed flow), using the k-ω SST turbulence model, and a target timestep of 0.005s.
- Integrated forces and moments have been calculated for the surfaces of the bridge, and converted to coefficients using the true frontal area of the bridge (48.4m^2), the beam height (1.24m) and with moments taken around the structural centre 0.785m below the centre of the deck.
- The flow field is inspected in Paraview to explore the mechanisms exhibited, as well as the risks and limitations of the flow field and modelling techniques.

4.2 Structural Dynamics

A simple mass-spring-damper model of the bridge has been built to examine the effect of the oscillatory aerodynamic loads on the primary vertical, lateral and rotational modes of the bridge. This solves the following independent differential equations over time, where m is mass, c is damping, k is stiffness, different for each axis; and y, z and α are delta deflections in the vertical, lateral and rotational sense. The axis system assumes x is along the bridge, positive y is upwind, positive z is upwards and a positive rotational moment around the structural centre is upwind side upwards.

$$F_{z,aero}(t) = m\frac{d^2z}{dt^2} + c\frac{dz}{dt} + kz$$

$$F_{y,aero}(t) = m\frac{d^2y}{dt^2} + c\frac{dy}{dt} + ky$$

$$M_{x,aero}(t) = I\frac{d^2\alpha}{dt^2} + c\frac{d\alpha}{dt} + k\alpha$$

Including the cable stays, the structural deck and beam box has the following properties and modes, with the relevant mass or inertia for the mode calculated from the natural frequency and stiffness below. As the structural analysis is for the whole 90m bridge, the aerodynamic forces and moments have been scaled by a factor of 3 to match the same length.

Sense	Load	Deflection	Stiffness	Damping Ratio
Vertical	10kN	0.008m	1250kN/m	0.003
Lateral	10KN	0.005m	2000kN/m	0.003
Rotational	10kNm	0.00063rad	15873kNm/rad	0.005

This model gives an indication of the effect of a change in aerodynamic forcing on an individual mode, but it is not the same as a fully coupled fluid-structure flutter analysis. It is not an FEA analysis which can assess the transfer of energy between modes as they are excited, but sufficient aerodynamic data will be generated to allow this to be done if further study is required.

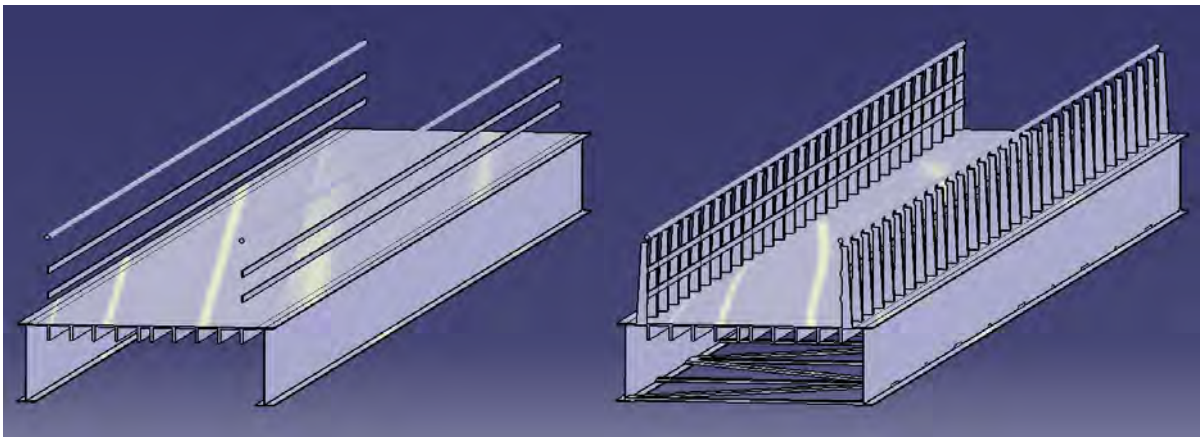
Mode No.	Natural Frequency, Hz	Description of mode shape	Mass participation[1]	No. of half waves (N)
1	0.88	Vertical (UZ)	73%	N=1
2	1.10	**Lateral (UY)**	79%	N=1
3	1.70	Vertical (UZ)	5%	N=2
4	2.20	Torsion (RX)	73%	
5	2.76	Vertical (UZ)	11%	N=3
12	5.23	Lateral (UY)	5%	N=3
		Torsion (RX)	6%	
14	6.44	Lateral (UY)	7%	N=3

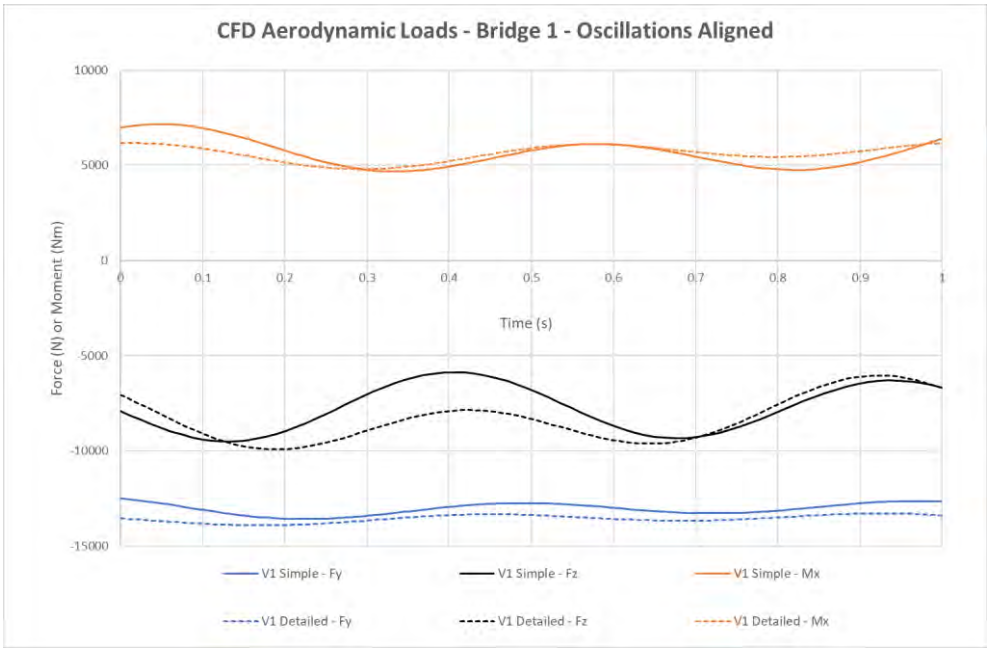
5. Results

5.1 Model Detail

It is standard practice to reduce CFD models to the minimum detail to avoid numerical instability and unnecessary computational overhead. Adding details such as guardrail posts and cross beams may break up the vortex structures so these are key elements to investigate.

Simple Extrusion **Detailed Model**

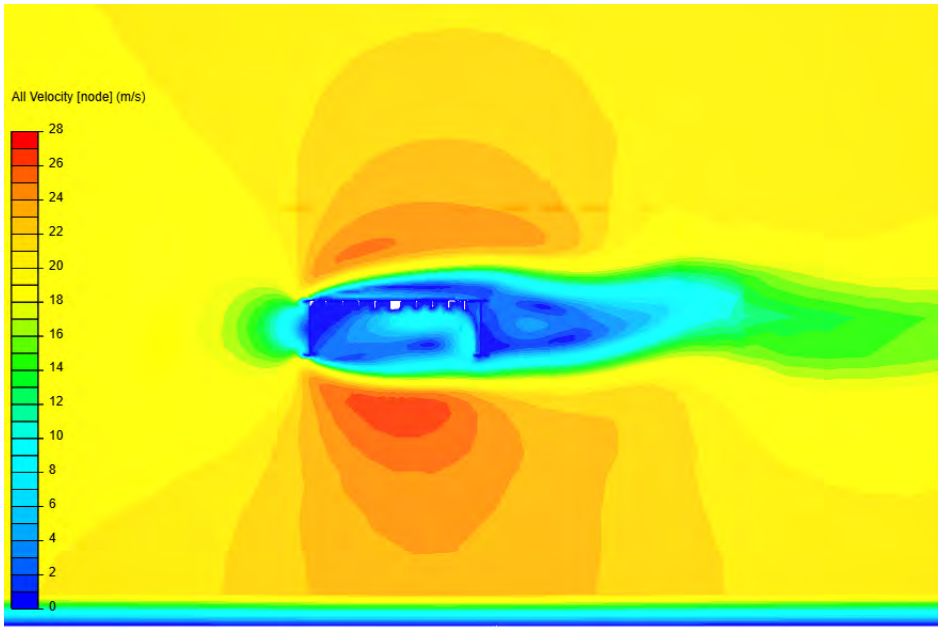




Average aerodynamic loads (last 1s of simulation with fully developed flow) show a slight increase for the detailed model. The frequency analysis shows the constant force is higher (moment similar), perhaps due to the increased surface area, but the amplitude of the oscillations is smaller.

21m/s 0deg	Fy	Fz	Mx	CFy	CFz	CMx
V1 Simple	-13065	-7181	5632	-1.105	-0.607	0.548
V1 Detailed	-13599	-8274	5613	-1.150	-0.700	0.553

		Bridge 1- SImplified			Bridge 1 - Detailed		
Frequency Components		Fy	Fz	Mx	Fy	Fz	Mx
	Const (N or Nm)	-13050	-7725	5675	-13575	-8325	5625
1st	Amp (N or Nm)	475	1650	975	225	1375	500
	Freq / Hz	1.9	1.9	1.9	2.0	2.0	2.0
	Phase / deg	-222	-182	-299	-293	78	-14
2nd	Amp (N or Nm)	250	225	525	125	925	325
	Freq / Hz	1.3	1.1	1.0	0.8	1.0	1.0
	Phase / deg	96	275	64	-158	92	125

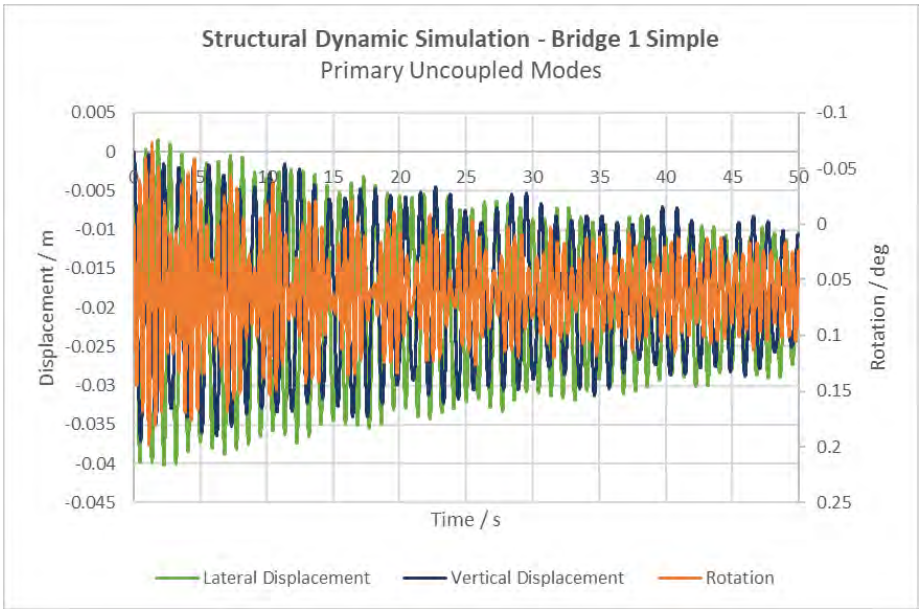


Both models have a strong aerodynamic mode at 1.9 or 2.0Hz, and this is characterised by a couplet of vortices being shed from the top and bottom edges of the rear beam (small blue ovals in the wake above), rather than Strouhal Instability. These vortices have unequal strengths so merge quickly and lead to a reduction in wake cross sectional area and low levels of oscillation. The next strongest oscillation is due to the unequal velocities with the upper flow slowed by the guardrail and the lower flow accelerated by the proximity of the ground plane. These effects also give rise to the net drag, downforce and nose up pitching moment; which although stable for a static structure may lead to positive feedback if the bridge pitches nose up or deflects down towards the ground and the relevant force or moment increases.

Replaying this data through the structural model for the 3 primary and uncoupled modes shows very small displacements and flutter only occurs with small damping ratios. The ratio of this limit damping to expected damping gives a safety margin factor of at least 5 for all 3 modes and both model detail levels. The model detail does make a difference: in vertical and torsional modes it shows smaller oscillations and more stability margin; so, will be carried forwards.

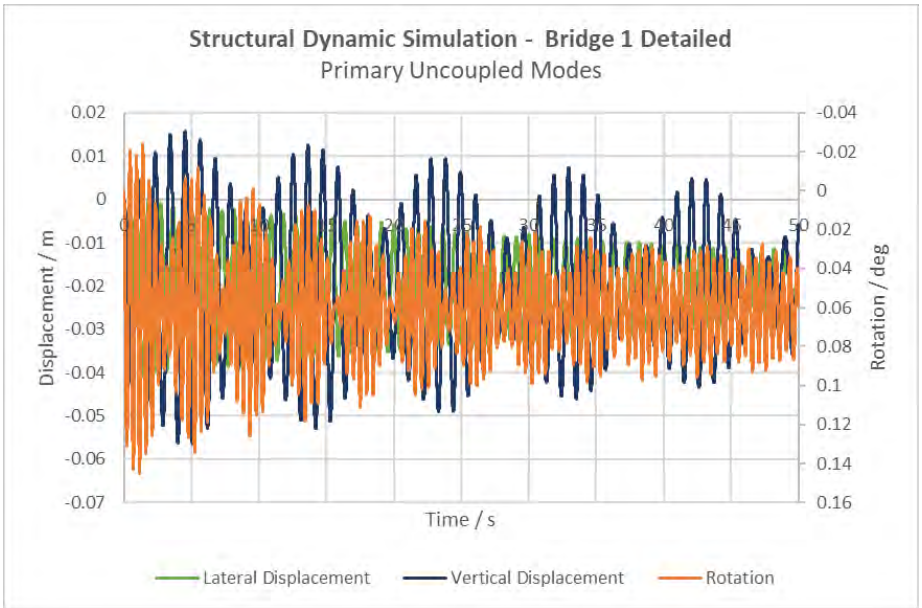
Bridge 1 – Simple - Dynamic Simulation – 21m/s

Primary Mode	Natural freq/Hz	Damping Ratio	Stability Limit Damping Ratio	Safety Factor
Vertical	0.88	0.003	0.0002	14.0
Lateral	1.10	0.003	0.0005	5.0
Rotational	2.20	0.005	0.0008	5.3



Bridge 1 – Detailed - Dynamic Simulation – 21m/s

Primary Mode	Natural freq/Hz	Damping Ratio	Stability Limit Damping Ratio	Safety Factor
Vertical	0.88	0.003	0.0000	Inf
Lateral	1.10	0.003	0.0005	5.0
Rotational	2.20	0.005	0.0006	7.3



5.2 Mesh Sensitivity

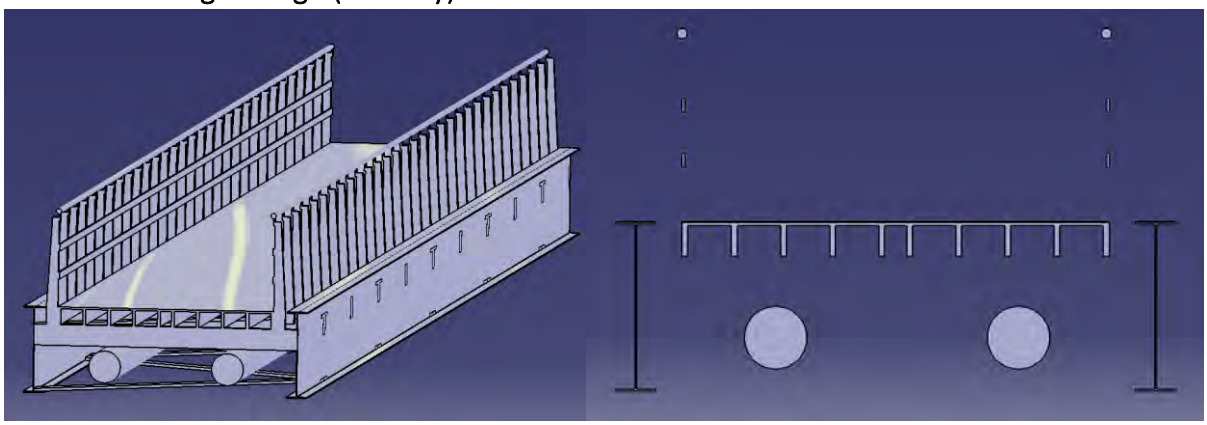
Using the detailed model from above, a mesh sensitivity study was performed to ensure the full detail was being captured, while avoiding numerical instability.

21m/s 0deg	Fy	Fz	Mx	CFy	CFz	CMx
Coarse Mesh (0.8M cells)	-12948	-5091	6793	-1.095	-0.431	0.626
Medium Mesh (1.4M cells)	-13599	-8274	5613	-1.150	-0.700	0.553
Fine Mesh (1.7M cells)	-14610	-9811	5395	-1.236	-0.830	0.551

Frequency Components		Coarse Mesh			Medium Mesh			Fine Mesh		
		Fy	Fz	Mx	Fy	Fz	Mx	Fy	Fz	Mx
	Constant (N or Nm)	-12925	-5675	7075	-13575	-8325	5625	-14400	-9375	5750
1st	Amp (N or Nm)	325	1500	600	225	1375	500	250	1475	425
	Freq / Hz	1.9	1.7	1.9	2.0	2.0	2.0	2.0	2.0	2.0
	Phase / deg	-100	-44	-182	-293	78	-14	-311	64	-49
2nd	Amp (N or Nm)	75	200	150	125	925	325	225	1625	725
	Freq / Hz	1.2	0.4	1.0	0.8	1.0	1.0	0.9	0.9	1.0
	Phase / deg	-236	-53	125	-158	92	125	-160	131	156
Minimum Stable Damping Ratio		.0004	.0007	.0008	.0005	0	.0006	.0005	.0001	.0006

The flow fields and results for the different mesh settings are within transient CFD simulation repeatability, the averages show an increase with cell count but this is due to the phase and number of peaks included in the averaging window. The frequency analysis confirms how similar the results are, with the medium and fine mesh being nearly identical on the primary oscillation. The coarse mesh is very close but produced some warnings for numerical instability, and both the frequency and structural analysis show a slight variance in results relative to the higher cell count options. The medium mesh will be used through this analysis.

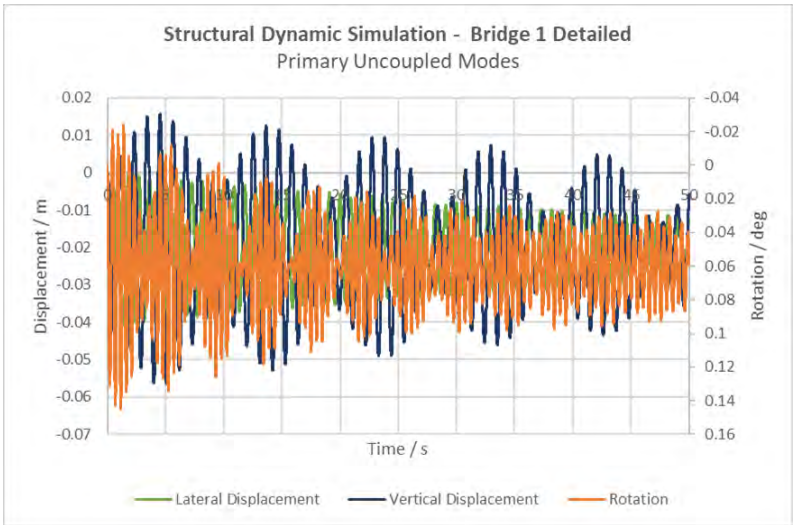
5.3 Bridge Design (Porosity)

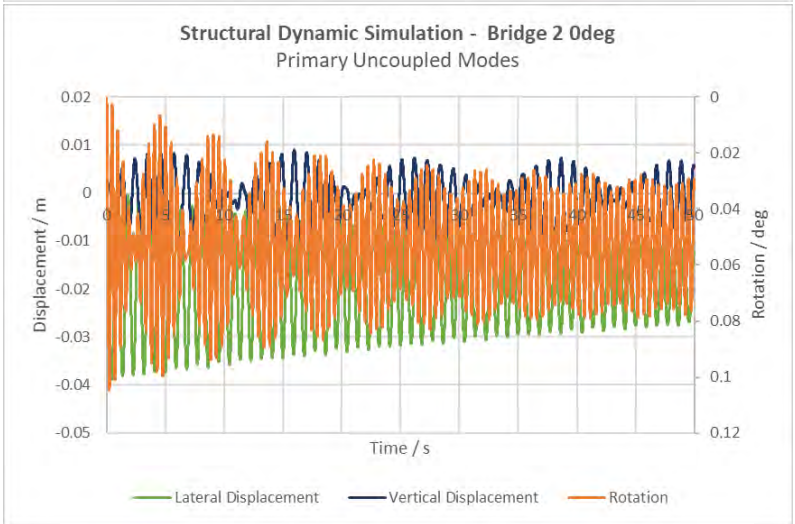


Bridge 2 has the same structural properties as Bridge 1, but features 2 pipes under the deck, and most importantly for the aerodynamics a porous gap between beam and deck on each side.

21m/s 0deg	Fy	Fz	Mx	CFy	CFz	CMx
Bridge 1	-13599	-8274	5613	-1.150	-0.700	0.553
Bridge 2	-13029	-280.4	4837	-1.102	-0.024	0.493

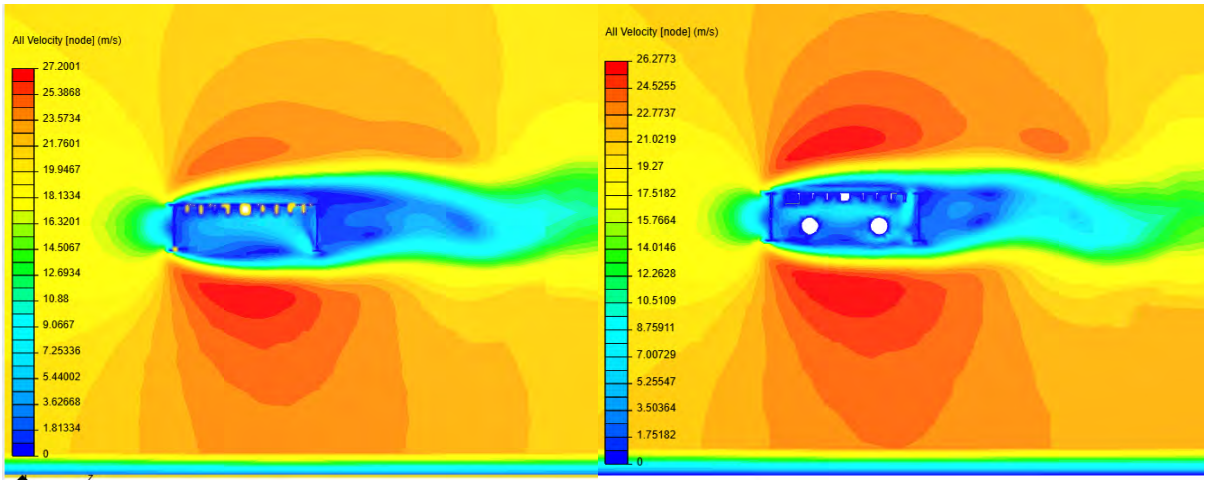
Frequency Components		Bridge 1			Bridge 2		
		Fy	Fz	Mx	Fy	Fz	Mx
	Const (N or Nm)	-13575	-8325	5625	-13025	-450	4925
1st	Amp (N or Nm)	225	1375	500	25	1475	400
	Freq / Hz	2.0	2.0	2.0	2.0	2.0	2.0
	Phase / deg	-293	78	-14	-113	32	-86
2nd	Amp (N or Nm)	125	925	325	250	400	75
	Freq / Hz	0.8	1.0	1.0	0.5	1.0	1.0
	Phase / deg	-158	92	125	131	91	175
Minimum Stable Damping Ratio		.0005	0	.0006	0	0	0





The gaps between the deck and beams have dramatically reduced the Fz, with a small reduction in Fy and Mx. The average and min to max variation in all three loads are lower for Bridge 2, so the deflections are smaller and the stability improved.

Bridge 1 **Bridge 2**

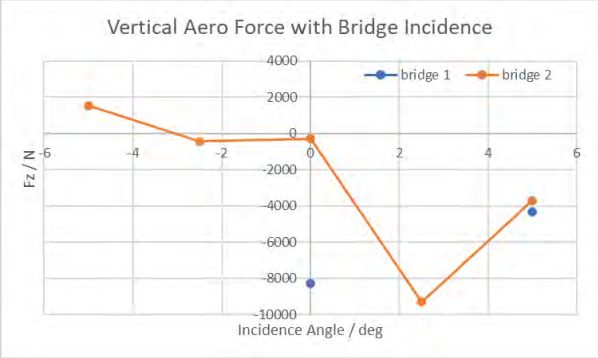


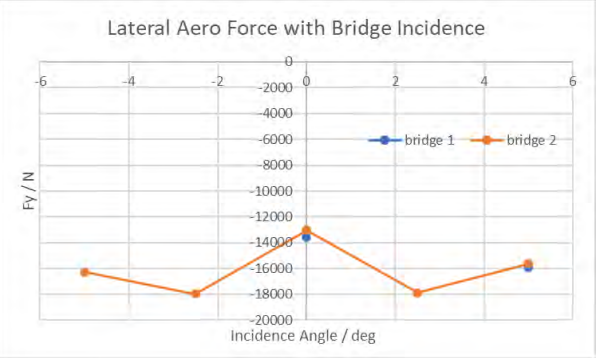
The porosity of the structural box has equalised the flow speeds and pressure above and below the bridge, and results in the reduced vertical force, but the wake size and vortex strengths are similar. The change in wake feeds back to an increased incidence on the upwind shedding edges and greater flow acceleration. This means the wind speed 1-2m above the deck, and experienced by users of the bridge, has increased from 25.8m/s to 26.9m/s (for 21m/s wind speed) and may limit traffic.

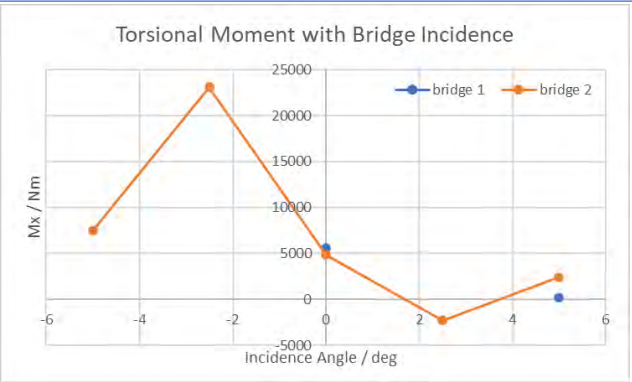
5.4 Bridge Incidence (Torsional Divergence)

As an uncoupled simulation it is essential to check that deflections do not lead to divergence, giving a positive feedback and acting to increase the forces and oscillations in the same sense. A full sweep of incidence has been conducted, with steps every 2.5deg.

21m/s Bridge 2	Fy	Fz	Mx	CFy	CFz	CMx
5deg LE Up	-15636	-3692	2400	-1.323	-0.312	0.358
2.5deg LE Up	-17864	-9293	-2301	-1.511	-0.786	0.064
0deg	-13029	-280.4	4837	-1.102	-0.024	0.493
2.5deg LE Down	-17968	-423.5	23075	-1.520	-0.036	1.803
5deg LE Down	-16263	1545	7460	-1.376	0.131	0.713

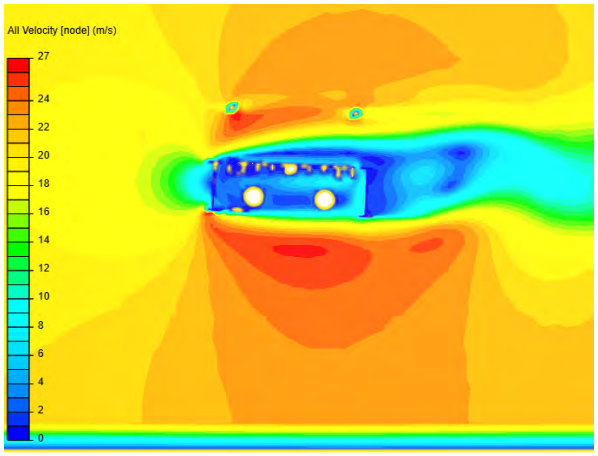




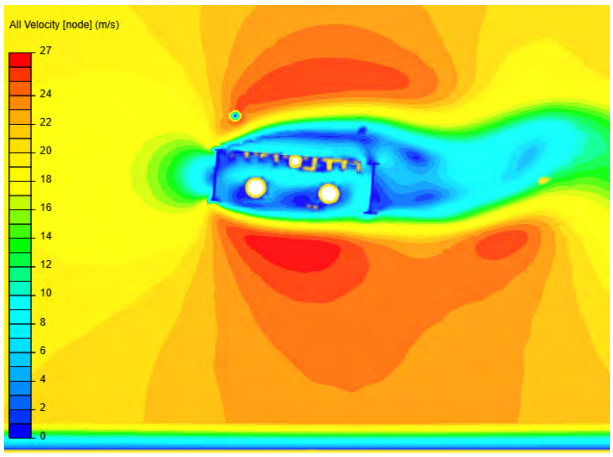


Between -2.5deg and +2.5deg there is a natural restoring moment, and across the whole range the nose up torsional moment tends to reduce as the bridge is deflected nose up, so is stable. However, there is a non-linear response for the different angles, with the ±2.5deg runs not fitting the trend of -5 to 0 to +5deg. 1 extra angle of Bridge 1 was run to confirm, and other than the higher vertical load at 0deg, it is very similar. The flow fields for these cases show significant changes in the vortex shedding: at ±2.5deg both the vortices are still shed from the downwind beam as at 0deg but the upwind beam makes a contribution to the most leeward edge; while at ±5deg the most leeward edge is completely hidden in the wake and the vortex system is formed from the increased incidence upwind beam and opposite downwind beam edges; so the non-linear response is reasonable.

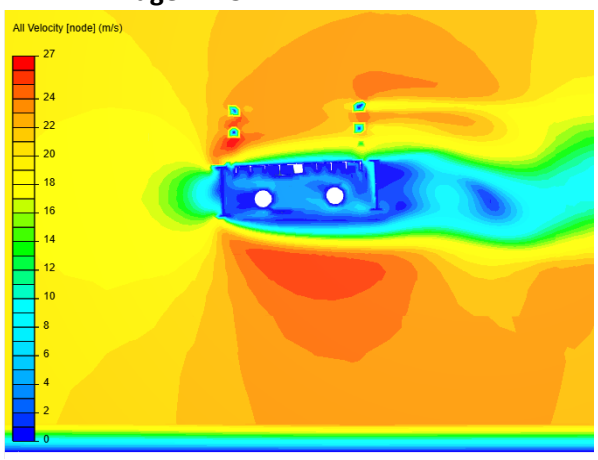
Bridge 2 2.5NU



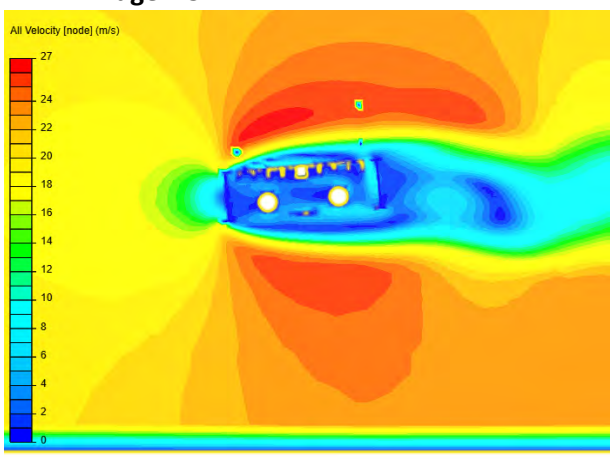
Bridge 2 5NU



Bridge 2 2.5ND



Bridge 2 5ND



Frequency Components		0deg			2.5NU			5.0NU		
		Fy	Fz	Mx	Fy	Fz	Mx	Fy	Fz	Mx
	Constant (N or Nm)	-13025	-450	4925	-17825	-9450	-2175	-15425	-3800	2300
1st	Amp (N or Nm)	25	1475	400	475	1100	500	1025	3250	525
	Freq / Hz	2.0	2.0	2.0	1.9	2.0	1.9	2.0	1.8	2.0
	Phase / deg	-113	32	-86	84	22	238	-28	-29	109
2nd	Amp (N or Nm)	250	400	75	275	300	325	325	1000	125
	Freq / Hz	0.5	1.0	1.0	0.5	1.0	1.0	1.0	1.0	0.7
	Phase / deg	131	91	175	269	237	268	155	-45	53
Minimum Stable Damping Ratio		0	0	0	0	.0003	.0005	.0005	.0003	.0006

Frequency Components		0deg			2.5ND			5.0ND		
		Fy	Fz	Mx	Fy	Fz	Mx	Fy	Fz	Mx
	Constant (N or Nm)	-13025	-450	4925	-17900	375	22600	-16150	1750	6875
1st	Amp (N or Nm)	25	1475	400	600	1800	400	900	2475	625
	Freq / Hz	2.0	2.0	2.0	2.0	2.0	1.9	1.8	1.9	1.9
	Phase / deg	-113	32	-86	-56	-289	298	209	-54	159
2nd	Amp (N or Nm)	250	400	75	325	450	375	150	725	750
	Freq / Hz	0.5	1.0	1.0	0.5	0.7	0.7	1.1	1.0	0.7
	Phase / deg	131	91	175	278	235	43	210	167	43
Minimum Stable Damping Ratio		0	0	0	0	.0004	.0006	0	0	.0001

The magnitude of the aerodynamic load oscillations increases with incidence in both directions, with a slight tendency for the frequency to reduce. While all cases are stable, the margin is slightly reduced for vertical and torsional modes.

5.5 Air Speed Sensitivity

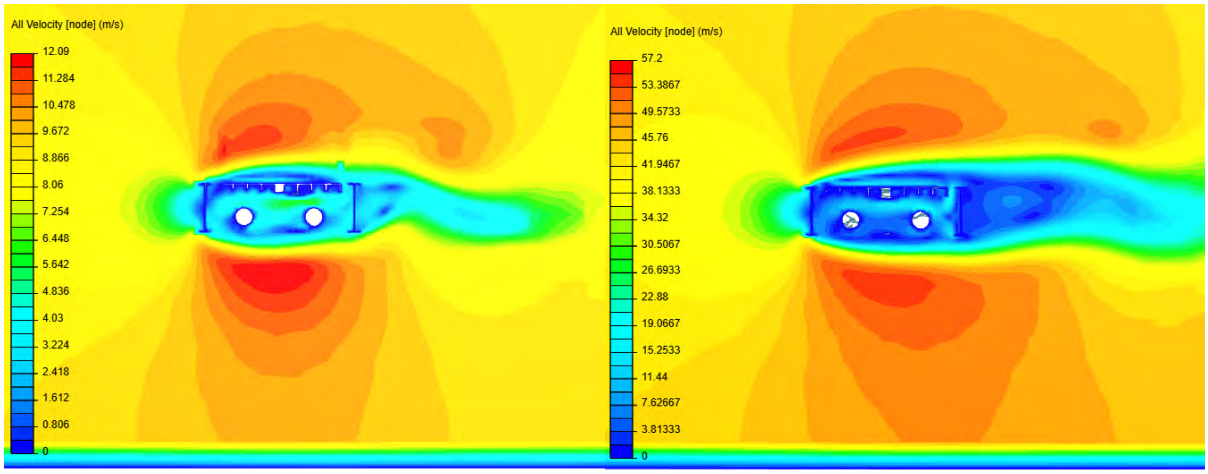
The shedding frequency changes linearly with air speed for the same flow regime (fixed Strouhal Number), while the flow regime can also change with Reynolds Number which is a function of air speed; so the extremes of air speed must be examined to avoid problems.

Exploring the structural model used, using the force and moment amplitudes of Bridge 2 at 0deg, the worst case for dynamic stability is a forcing function at the natural frequency. The rotational and lateral primary modes show a low sensitivity to frequency; but there is a concern over the vertical mode at 0.88Hz, which should occur at 9.3m/s. The maximum expected airspeed for the bridge is 44m/s, so this will also be checked.

Bridge 2 0deg	Fy	Fz	Mx	CFy	CFz	CMx
9.3m/s	-2553	-430.1	963.0	-1.101	-0.185	0.498
21m/s	-13029	-280.4	4837	-1.102	-0.024	0.493
44m/s	-58461	-2300	20514	-1.126	-0.044	0.485

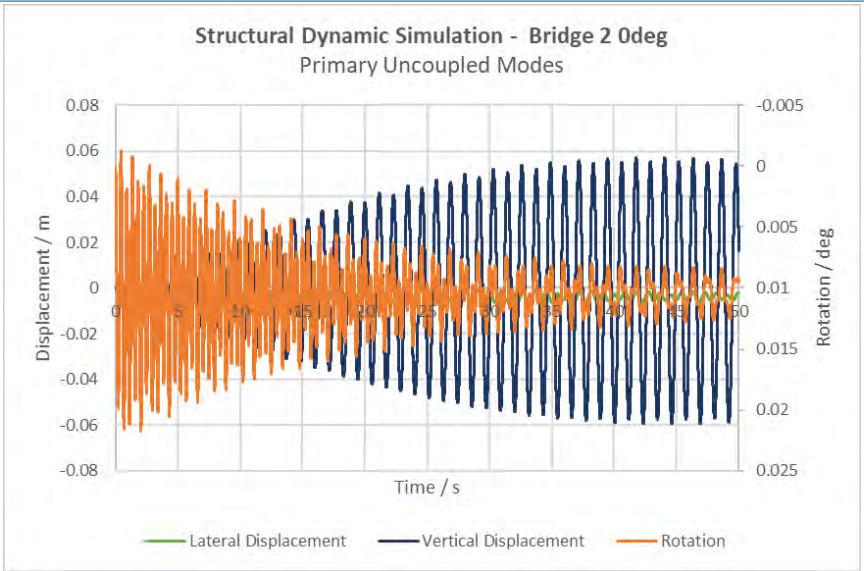
The average aerodynamic load increases with approximately the square of the speed, so in almost all cases the aerodynamic coefficients are unchanged, but the 9.3m/s vertical load is very high. This extra vertical load seems to be due to a disproportionately strong recirculation current under the deck causing a low pressure zone. This is due to the strong acceleration of the flow around the upwind beam shedding edges, worse due to the low Reynolds Number (images below have the speed scale set to be in proportion with the freestream value and the internal recirculation is green and over 50% of 9.3m/s freestream, but only 30% of 44m/s, and with a much bigger and lossier wake). This constant vertical load does not seem to be a problem for deflection or stability.

9.3m/s **44m/s**

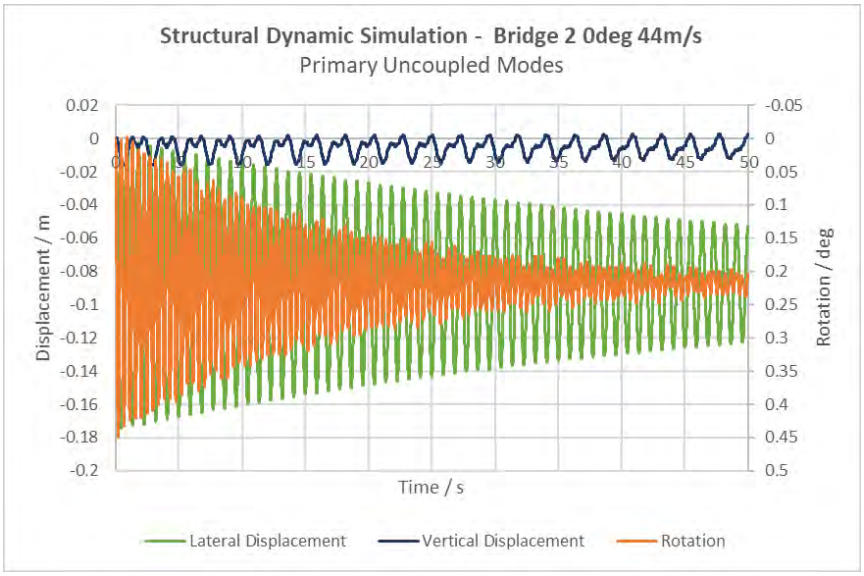


Frequency Components		9.3m/s			21m/s			44m/s		
		Fy	Fz	Mx	Fy	Fz	Mx	Fy	Fz	Mx
	Const (N or Nm)	-2575	-500	975	-13025	-450	4925	-58350	-2250	20075
1st	Amp (N or Nm)	130	385	125	25	1475	400	260	5000	1425
	Freq / Hz	0.87	0.87	0.87	2.0	2.0	2.0	3.93	3.93	3.93
	Phase / deg	3	2	-191	-113	32	-86	-78	99	-23
2nd	Amp (N or Nm)	155	375	0	250	400	75	155	1800	475
	Freq / Hz	0.43	0.44	0.43	0.5	1.0	1.0	0.43	0.44	0.58
	Phase / deg	-67	-68	157	131	91	175	-69	168	107
Minimum Stable Damping Ratio		0	0	.0006	0	0	0	0	0	0

At 9.3m/s the aerodynamic load oscillates at 0.87Hz, close to the primary vertical mode at 0.88Hz. The model predicts a vertical deflection limit cycle oscillation of 120mm peak to peak amplitude, rather than a flutter instability. By scaling the loads and frequency with speed for small speed changes around 9.3m/s and re-running the structural simulation it has been established that this is the worst-case situation. This result has been supported by other studies commissioned by the bridge designer, so while this uncoupled analysis does not investigate how the different modes interact, flutter can be ruled out.



The flow regime remains the same through the speed range, and while the loads and average deflections increase with air speed, the aerodynamic load amplitudes rise proportionately, and the higher frequencies make the bridge less susceptible and more stable. At 44m/s the stable vertical oscillation is reduced to 16mm peak to peak.



6. Conclusions and Recommendations

No aerodynamic instability has been discovered; the results show:

- Small vortex pairs form on the trailing corners of the bridge, rather than the large alternating pattern of a Von Karman vortex, which both reduces oscillatory loads and total drag force on the bridge.
- Bridge 2 features gaps between the deck and beams to allow flow between and equalise the pressure. This does reduce the vertical force relative to bridge 1, with a small improvement in the oscillatory components and bridge stability. It also increases the local flow acceleration around the bridge such that users will experience a 4% higher wind speed.
- A sweep of incidence for bridge 2 shows no divergence with torsional deflection, and up to ±2.5deg there is a negative feedback restoring torque. At large incidences the flow system changes as the wake encompasses one of the shedding edges, and while this makes the load oscillations larger the stability margin is still high.
- The primary lateral and torsional modes of the bridge are insensitive to the input frequency, however the vertical mode does become less stable close to its natural frequency of 0.88Hz with the worst-case at 9.3m/s causing a limit cycle oscillation of ±60mm. Higher speeds become increasing stable.

Aerodynamically, there is no instability or concern for the structure of the bridge. It will not suffer flutter and the torsion mode is stable, although the limit cycle oscillation at around 9.3m/s is large. This is not a fully coupled fluid-structure simulation, but other analyses by the bridge designer support the conclusions of this analysis of transient CFD replayed through a spring-damper model.

It is understood that the vortex excitations estimated from the 0deg / 9.3m/s case will be used by the designer to assess user comfort levels along the bridge. If the estimated value of 60mm amplitude is deemed uncomfortable for users it is recommended that the vertical load is assessed directly in the FE structural analysis model as a time-history oscillation of 0.88Hz to determine if deflections alter compared to the simple spring model used in this study. If oscillations are still considered too large the following measures could be employed to reduce the overall influence of vortex excitation:

- Structurally by adding mass to the middle of the bridge to lower the natural frequency. At 0.84Hz vertical (with an extra 4000kg included in the mode) and 9.0m/s (to match the frequency) the amplitude reduces to ±25mm because the aerodynamic forces are lower (subject to a CFD check at the new speed), and the effective damping higher.
- Dynamically with a mass damper (but it is understood this is not a preferred solution because of the complexity and cost)
- Aerodynamically with less porous railings (as recommended in “Influence of aerodynamic configuration of a streamline box girder on bridge flutter and vortex-induced vibration” Wang et al, Journal of Modern Transportation December 2011), which effectively increases the height of the unit and lowers the shedding frequency such that the worst case occurs with a lower air speed and aerodynamic force so is more stable. This would increase the side load on the bridge.
- Aerodynamically breaking up the vorticity by having alternating sections of high and low porosity railing so that it prevents one driving frequency. This would require additional CFD

modelling to confirm the behaviour and frequencies to be used but essentially it would involve replacing the highly porous wire mesh barrier with (for example) a bolt on laser cut panels. The FE model would be used to determine the portions of barriers to be replaced.

No analysis has been done on the mast or cables of the bridge. It is important to avoid a regular or symmetric profile as this can exhibit Strouhal Instability both amplifying the oscillation of the bridge and adding unexpected fatigue cycles to the components.

APPENDIX D
SAFETY IN DESIGN RISK REGISTER

SAFETY IN DESIGN RISK REGISTER

Project:	**Longford Cable-stay Cyclebridge**	**Revisions:**
Package or Element:	**Cable-stay Bridge**	Rev A - 14 April 2020
Prepared by:	D. Crocker (DC Structures Studio)	
Checked by :	H. Ramezan-zadeh (Gore District Council)	
Approved by:	H. Ramezan-zadeh (Gore District Council)	

Workshop Attendees:

Workshop not possible because of Covid 19. Circulated via email for review by all.

SiD Circulation Register:

Hashem Ramezan-zadeh of Gore District Council (Asset Owner)
Peter Standring of Gore District Council (Asset Owner)
Erik Teekman of NZTA (Funding Agency)
Colin Young of Environment Southland (Regional Council - Flood Control)
Paul Romanes of Concrete Structures Ltd. (Constuctor)
James Kelly of Concrete Structures Ltd. (Constuctor)
Eli Maynard of Riley Consultants (Geotech & Hydrology)
Dave Hughes of Beca Water Team (Water Utilities)
Hamish Weir of Landpro (Planning Team)
Dan Crocker of DC Structures Studio (Bridge Designer)

RISKS ASSOCIATED WITH DESIGN ELEMENTS				Risk Matrix			PROPOSED & APPROVED MITIGATIONMEASURES	Mitigated Risk Matrix						RESIDUAL RISK	
Ref	Hazard (Guideword)	Raised / discussed by	Cause & Outcome	Existing controls, if any	L	C	R	Proposed Control (1 Eliminate 2 Minimise 3 Manage)	L	C	LR	Risk Owner	Design Status / Rev	Risk Owner	Action required
1	**Construction Phase**														
1.01	*Clear Span*	Hashem Ramezan-zadeh of Gore District Council (Asset Owner)	**Hazard:** Working inside the river can be dangerous given river is suseptible to flooding. **Outcome:** Loss of life or damage to other downstream structures (such as SH1 bridge)	Flooding can be monitored and managed.	3	4	H	***Mitigation:*** Reduce ***Measure:*** Construction in river channel has been minimised by nominating a clear 90m span. No permanent piers are to be used and temporary piers are to be outside of main channel.	1	2	L	Constructor	Detailed Design / A		
1.02	*Flood during construction*	Colin Young of Environment Southland (Regional Council)	**Hazard:** Construction phase is expected to be 6 months. If a flood were to occur during this duration materials/plant could be swept downstream damaging assets such as SH1 bridge. **Outcome:** This could lead to significant damage to the existing bridge and potential collapse. Site staff could also be harmed	N/A	4	4	E	***Mitigation:*** Manage ***Measure:*** Contractor has been asked to provide a detailed Flood Management Strategy that includes monitoring of upstream flood data and evacuation of the flood plain within 24hours of a flood warning. Expected to reduce likelihood of surprise and redcution of consequences.	2	3	M	Constructor	Detailed Design / A	Constructor and Environment Southland	Flood Management requirements to form part of Resource Consenting and to be monitored as part of normal Reginal Council processes.
1.03	*Stressing points*	Dan Crocker of DC Structures Studio (Designer)	**Hazard:** Stressing points on mainspan are difficult to access (@ Preliminary Design). **Outcome:** Falls/drowning	Handrail can be used for fall restraint systems	3	4	H	***Mitigation:*** Reduce ***Measure:*** During detailed design cable atachment cleats have been moved closer to walkway and vertically upwards. This is a significant improvement where access to cable cleats for stressing and/or restressing might be required.	1	2	L	Constructor	Detailed Design / A		
1.04	*Installing water pipes over water and at height*	Erik Teekman of NZTA (Funding Agency)	**Hazard:** The bridge is to carry two watermains over the 90m span. Installing these pipes will require difficult work at height over the water.with risk of falling/drowning	Handrail can be used for fall restraint systems	3	4	H	***Mitigation:*** Remove ***Measure:*** Pipes are to be sleeved so that install can be carried out from either end.	1	2	L	Constructor	Detailed Design / A		
1.05	*Movement of large steelwork*	Paul Romanes of Concrete Structures Ltd. (Constuctor)	**Hazard:** Steelwork in excess of 15m in length will make delivery more dofficult which can lead to traffic incidents.	Use "pilots" and travel out of hours.	2	4	M	***Mitigation:*** Minimise ***Measure: By specifying lengths <15 and width <4.4m the steelwork can be transported using normal trucks.***	1	3	L	Constructor	Detailed Design / A		
1.06	*Large stressing forces*	Designer & Constructor	**Hazard:** The cable-stays have a final stress of circa 15 to 20 tonnes. Such large forces require particularly big stressing equipment which can be difficult to handle at height / over water **Outcome:** leading to falls and/or drowning.	Backstays have been designed with direct stressing module. Not possible for main-span stays.	2	4	M	***Mitigation:*** Minimise ***Measure:*** Stressing forces will be reduced to below 10t by stressing prior to applying superimposed dead loads. 10t can be done with normal cable clamps and tirfor assemblies. Will require work to be done with safety line but lighter equipment should be easier (safer) to manouver.	1	3	L	Constructor	Detailed Design / A		

DC STRUCTURES STUDIO

SAFETY IN DESIGN RISK REGISTER

Project:	**Longford Cable-stay Cyclebridge**	**Revisions:**
Package or Element:	**Cable-stay Bridge**	Rev A - 14 April 2020
Prepared by:	D. Crocker (DC Structures Studio)	
Checked by :	H. Ramezan-zadeh (Gore District Council)	
Approved by:	H. Ramezan-zadeh (Gore District Council)	

Workshop Attendees:

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James Kelly of Concrete Structures Ltd. (Constuctor)
Eli Maynard of Riley Consultants (Geotech & Hydrology)
Dave Hughes of Beca Water Team (Water Utilities)
Hamish Weir of Landpro (Planning Team)
Dan Crocker of DC Structures Studio (Bridge Designer)

RISKS ASSOCIATED WITH DESIGN ELEMENTS					Risk Matrix			PROPOSED & APPROVED MITIGATIONMEASURES	Mitigated Risk Matrix						RESIDUAL RISK	
Ref	Hazard (Guideword)	Raised / discussed by	Cause & Outcome	Existing controls, if any	L	C	R	Proposed Control (1 Eliminate 2 Minimise 3 Manage)	L	C	LR	Risk Owner	Design Status	Rev	Risk Owner	Action required
1.07	*Large steelwork weight*	Dan Crocker of DC Structures Studio (Designer)	**Hazard:** Collapse of cranes during lifting operation **Outcome:** Injury to contractors.	Constructor has considered a lifting approach which inludes temporary cable stay and temporary piers.	1	5	M	***Mitigation:*** Minimise ***Measure:*** Steelwork weight has been redcued during detailed design by circa 20%. Smaller loads reduces the consequence and potentially redcues the crane requirements.	1	4	L	Constructor	Detailed Design	A	Constructor	Contractor to provide adequate lifting plans and procedures
1.08	*Position / Location*	Dan Crocker of DC Structures Studio (Designer)	**Hazard:** Working over deep water **Outcome:** Potential Drowning	Maximise erection over land.	3	4	H	***Mitigation:*** Minimise ***Measure:*** Steelwork to be pre-assembled. Only primary beam splices and connecting bracing will be tightened at height over water.	1	4	L	Constructor	Detailed Design	A		
1.09	*Movement Direction*	Dan Crocker of DC Structures Studio (Designer)	**Hazard:** Lifting operations interfering with road traffic **Outcome:** Accident	Mast has been nominated for erection away from existing roadway (on eastern side)	2	4	M	***Mitigation:*** Manage ***Measure:*** Contractor to provide temporary traffic management to suit lifting and erection methdology. Screens to be used to prevent "rubber-knecking". Speed redcutions will be in place to reduce severity of accidents.	1	3	L	Constructor	Detailed Design	A	Constructor	Contractor to provide temporary traffic management to suit lifting and erection methodology
1.10	*Movement Direction*	Dan Crocker of DC Structures Studio (Designer)	**Hazard:** Steelwork structure unstable during construction **Outcome:** Injury to workers.	Designer to nominate construction assembly stresses and deflection on drawings.	2	4	M	***Mitigation:*** Manage (project risk) ***Measure:*** Coordination of temporary works design and permanent works design.	1	4	L	Contractor	Detailed Design	A	Constructor / Designer	Designer to work with Construction team's temporary works designer to consider influence on permanent works during construction.
2 Operation & Maintenance Phase																
2.01	*Maintaining water pipes over water and at height*	Erik Teekman of NZTA (Funding Agency)	**Hazard:** The bridge is to carry two watermains over the 90m span. Maintenance of these pipes will require work at height over the water.	Handrail can be used for fall restraint systems	3	4	H	***Mitigation:*** Remove ***Measure:*** Pipes are to be sleeved so that install can be carried out from either end.	1	2	L	Constructor / Utilities installers	Detailed Design	A		
2.02	*Maintaining pipe performance*	Dave Hughes of Beca Water Team (Water Utilities)	**Hazard:** The camber of the bridge will lead to the sleeved pipes having a high point at midspan. This could lead to performance issues of this critical utility which could subsequently lead to work on the pipes or early replacement of pipe being required. **Outcome:** Injury / Falling / drowning	N/A	4	2	M	***Mitigation:*** Remove ***Measure:*** An air release value will be added at midspan to remove trapped air from the sleeved pipe. An access hatch will be provided.	1	2	L	Asset Owner	Detailed Design	A		
2.03	*Influence on flood plain*	Colin Young of Environment Southland (Regional Council)	**Hazard:** Building a pathway through the flood plain could have significant consequences for flood protection of Gore. **Outcome:** Widespread flooding and loss of life	N/A	3	4	H	***Mitigation:*** Remove ***Measure:*** The approach paths through the western flood plain will be built at grade so no additional bunding is created. Influence of the new embankment has been hydrollogically modelled and outputs agreed/confirmed by Colin Young.	1	4	L	Environment Southland	Detailed Design	A		

DC STRUCTURES STUDIO

SAFETY IN DESIGN RISK REGISTER

Project:	**Longford Cable-stay Cyclebridge**	**Revisions:**
Package or Element:	**Cable-stay Bridge**	Rev A - 14 April 2020
Prepared by:	D. Crocker (DC Structures Studio)	
Checked by :	H. Ramezan-zadeh (Gore District Council)	
Approved by:	H. Ramezan-zadeh (Gore District Council)	

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2.04	*Path Alignments*	Erik Teekman of NZTA (Client)	**Hazard:** Western and Eastern connection to approach pathways (at Concept stage) included tight radii. **Outcome:** Injury to cylists	Balustrades at bend prevent cyclists continuing down the banks.	4	2	M	***Mitigation:*** Minimise ***Measure:*** Radii have been increased and overall bend has been redcued at the western end (by dog legging the end span by 12 degrees).	3	2	L	Asset Owner	Detailed Design	A		
2.05	*Embankment Scour*	Eli Maynard of Riley Consultants (Geotech & Hydrology)	**Hazard:** Toe scour at the base of the embankment can lead to unforseen erosion at the base. **Outcome:** Partial collapse of embankment causing injury to cyclists	Rip rap is used up the side of the embankment	3	4	H	***Mitigation:*** Minimise ***Measure:*** A rip rap apron to the base has been specified so that unforseen erosion of toe is significantly mitigated.	1	4	L	Asset Owner	Detailed Design	A		
2.06	*Steep Side Batters*	Dan Crocker of DC Structures Studio (Designer)	**Hazard:** Banks are to be 2H:1V with no cyclerail. This is to minimise footprint and obstructions in the flood plain. Austroads cycleway design guidance would typically recommend a 3H:1V side batter where no rail is used. **Outcome:** Injury to cyclists	Batters are consistent with rest of flood banks (that will have a cycle trail)	3	3	M	***Mitigation:*** Minimise ***Measure:*** Planting will be used along the shoulder such as carex grasses and native flaxes to reduce cyclists travelling too close to the edge.	2	3	L	Asset Owner	Detailed Design	A		
2.07	*Vandalism and/or Damage to Cables*	Dan Crocker of DC Structures Studio (Designer)	**Hazard:** Damage may occur to cables (e.g. vandalism) during the life of the bridge resulting in bridge collapse **Outcome:** Falls / drowning	Cables have been specified for 100 years and assessed for fatigue (normal wear and tear resulting in failure is unlikely).	1	5	M	***Mitigation:*** Remove ***Measure:*** The bridge has been designed and checked for redundancy where one cable is suddenly omitted (a dynamic impact effect is also considered). This redundancy will also help with the cable replacement in the event of damage.	1	3	L	Asset Owner	Detailed Design	A	Designer / Asset Owner	The cable damage case and replacement requirements will be documented in the Design Report and on construction drawings.
2.08	*Errant Vehicle on bridge*	Dan Crocker of DC Structures Studio (Designer)	**Hazard:** Cars (particularly stolen cars) being driven on wide cyclebridge is a previous issue we have encountered. Cars can cause significant damage to primary members and particulalry to the balustrades. **Outcome:** Bridge users falling/drowning	Smaller community less subject to stolen vehicles (compared to a large city like Auckland).	1	4	L	***Mitigation:*** Remove ***Measure:*** Bollards to be strategically placed to prevent vehicle entrering bridge. Bollards will need to be made clearly visible to cyclists.	1	1	L	Asset Owner	Detailed Design	A	Asset Owner	Client to consider the use of cycle-friendly bollards to prevent unwanted vehicles accessing the bridge.
2.09	*External safety interfaces*	Dan Crocker of DC Structures Studio (Designer)	**Hazard:** Accessing structure for inspections and/or maintenance work above water **Outcome:** Potential falls / drowning	Normal Safety procedures for carrying out bridge inspections	2	4	M	***Mitigation:*** Minimise ***Measure:*** – Top mounted screws will be used so that walkway panels can be easily removed. Panels are lightweight FRP 15kg/m2. Panels can be easily removed to enable safe inspection of cross beams. Fall restraint systems can be attach with soft sling to balustrade rail. Boat access for normal inspections is recommended.	1	4	L	Asset Owner	Detailed Design	A	Designer / Asset Owner	Maintenance notes will be added to drawings. Asset owners maintenance teams to initiate safe systems for inspection.
2.10	*Egress / Access*	Dan Crocker of DC Structures Studio (Designer)	**Hazard:** Members of Public leave cycleway. **Outcome:** Fall from height or fall from bicyle, in particular children	1.4m high balustrades with anti-climb infill is to be used.	1	4	M	None required.	1	4	L	Asset Owner	Detailed Design	A		

DC STRUCTURES STUDIO

SAFETY IN DESIGN RISK REGISTER

Project:	**Longford Cable-stay Cyclebridge**	**Revisions:**
Package or Element:	**Cable-stay Bridge**	Rev A - 14 April 2020
Prepared by:	D. Crocker (DC Structures Studio)	
Checked by :	H. Ramezan-zadeh (Gore District Council)	
Approved by:	H. Ramezan-zadeh (Gore District Council)	

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Hashem Ramezan-zadeh of Gore District Council (Asset Owner)
Peter Standring of Gore District Council (Asset Owner)
Erik Teekman of NZTA (Funding Agency)
Colin Young of Environment Southland (Regional Council - Flood Control)
Paul Romanes of Concrete Structures Ltd. (Constuctor)
James Kelly of Concrete Structures Ltd. (Constuctor)
Eli Maynard of Riley Consultants (Geotech & Hydrology)
Dave Hughes of Beca Water Team (Water Utilities)
Hamish Weir of Landpro (Planning Team)
Dan Crocker of DC Structures Studio (Bridge Designer)

RISKS ASSOCIATED WITH DESIGN ELEMENTS					Risk Matrix			**PROPOSED & APPROVED MITIGATIONMEASURES**	Mitigated Risk Matrix						**RESIDUAL RISK**	
Ref	Hazard (Guideword)	Raised / discussed by	Cause & Outcome	Existing controls, if any	L	C	R	Proposed Control (1 Eliminate 2 Minimise 3 Manage)	L	C	LR	Risk Owner	Design Status	Rev	Risk Owner	Action required
2.11	*Egress / Access*	Dan Crocker of DC Structures Studio (Designer)	**Hazard:** There is a risk that members of the public climb onto the primary beam. **Outcome:** Slips or falls, potentially into the river, resulting in injury or drowning	Balustrade infill has been designed non-climbable as per current Building Code	2	3	M	***Mitigation:*** Minimise ***Measure:*** We have used anti-climb mesh to prevent people (< age of 6) climbing onto top chord. This is as per the NZ Bulding Code F4 recommendations.	1	3	L	Asset Owner	Detailed Design	A	Asset Owner	Risk that people will still get on top of top of the beam and jump. This is unavoidable without large fencing. GDC to confirm acceptance of this risk.
3	**Demolition Phase**															
3.01	*Load / Force / Energy*	Dan Crocker of DC Structures Studio (Designer)	***Hazard:*** Dismantling of bridge ***Outcome:*** Injury to demolition operatives	Bridge has a 100 year life	1	5	M	***Mitigation:*** Manage (Project risk) ***Measure:*** – All the components can be dismantled in smaller components by dismantling members at bolted and pinned connection interfaces.	1	4	L	Contractor	Detailed Design	A	Demolition Contractor	The Demoltiion Contractor will need to provide a safe demolition methodology to prevent uncontrolled collapse of the steelwork.

NOTES: Hazards / risks considered are those that are project / site specific, non-standard / bespoke designs, special processes, high hazard risks (e.g. non 'business as usual' hazards) that have been identified at the time of the review(s). Other risks will continue to appear during the design life of the project and should be assessed and managed by appropriate parties.

RISK MATRIX (R) ASSESSMENT HEIRARCHY:

	LIKELIHOOD (L):	1 - RARE	2 - UNLIKELY	3 - POSSIBLE	4 - LIKELY	5 - ALMOST CERTAIN
CONSEQUENCE (C):	1 - LOW	LOW (1)	LOW (2)	LOW (3)	LOW (4)	MODERATE (5)
	2 - MODERATE	LOW (2)	LOW (4)	MODERATE (6)	MODERATE (8)	HIGH (10)
	3 - SIGNIFICANT	LOW (3)	MODERATE (6)	HIGH (9)	HIGH (12)	EXTREME (15)
	4 - MAJOR	LOW (4)	MODERATE (8)	HIGH (12)	EXTREME (16)	EXTREME (20)
	5 - CRITICAL	MODERATE (5)	HIGH (10)	EXTREME (15)	EXTREME (20)	EXTREME (25)