OCTOBER 2022
UK HIGHWAYS A55 LTD.

MENAI BRIDGE - HANGER LOSS ASSESSMENT

FINAL REPORT

COMMERCIAL – IN CONFIDENCE

COWI

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CONTENTS

1 Introduction

1.1 Introduction

The Grade 1 Listed Menai Suspension Bridge was designed by Thomas Telford and opened in 1826. Today it continues to connect mainland Wales to the Island of Anglesey. The bridge is managed by UK Highways A55 Ltd. and has seen several major maintenance interventions during its lifetime, most notably the following:

Year(s)	Works package
1938-41	Replacement of the superstructure (chains, saddles, hangers, deck)
1988-1993	Replacement of 40No. of the 1938-41 hangers
2000	Deck replacement

In 2022 UK Highways A55 Ltd. appointed COWI UK Ltd. to prepare a specification for repainting the approach span hangers. Whilst reviewing historic reports to inform the specification COWI became aware that significant concerns regarding the ductility and thus capacity of the hangers were identified during structural investigations in the late 1980's and early 1990's. These structural investigations led to the replacement of 40 hangers and a recommendation to replace the remaining hangers on a rolling basis thereafter.

COWI undertook a qualitative review (COWI Report Ref A238719-RP01-v2.0) of documentation arising from these structural investigations and confirmed that there is an ongoing concern of brittle fracture of the sockets of the remaining 1938-41 hangers. On this basis the structure has been classified as a substandard structure in accordance with DMRB CS450 and interim management measures have been implemented (see COWI Report A238719-RP03-v1.0). Most significantly the bridge is now subject to a 7.5T weight restriction.

UK Highways also instructed COWI to prepare a specification for the design, testing, manufacture and installation of replacement hangers with sockets made from modern ductile steel. This work is underway, however there is a significant lead time for new hangers. Spencer Group Ltd. have been appointed to deliver the works and it is expected that work on site will commence in spring 2023.

As previously noted, COWI's document review was qualitative in nature and did not include any quantitative analysis. Therefore, UK Highways asked COWI to prepare a model of the bridge and investigate the consequence of a hanger failing in service. This report presents the findings of that quantitative review.

1.2 Analysis

1.2.1 Description of Analysis

The purpose of the assessment work covered by this report is to explore and conclude on the following questions:

- What influence would the sudden brittle failure of one hanger have on adjacent hangers?
  - Will load redistribution (and associated short-term dynamic effects) cause these to become overloaded, leading to a progressive failure?
- What level of traffic loading is the bridge then able to withstand, in the aftermath following an event where one hanger has failed?

It should be noted that the assessment work covered by this report does **not** constitute a full assessment of the bridge, rather it is a limited investigation concerned with exploring the consequences of hanger failure.

1.2.2 Basis of Analysis

This study is defined in TAF document *A238719-TN09-v3*.0 (see Appendix A), this provides some further details that have not been restated in this report.

Generally, the analysis has been undertaken in accordance with:

- DMRB CS 454 (Rev 01) Assessment of highway bridges and structures

However, this document does not define an approach for assessing the dynamic failure of a hanger in service and thus the approach given in BS EN 1993-1-11 is adopted for this item.

1.2.3 Analysis Approach

Any analysis of this type if multi-faceted and incorporates the following aspects, each of which are considered in turn in the following sections of the report.

- Analysis Methodology
- Finite Element Model
- Loads
- Capacities
- Condition of the Structure
- Engineering Judgement (to interpret the results)

2 Structure Description

2.1 General Description

The bridge consists of a pair of 52.7m high masonry towers which support two paired sets of steel link chains across the 176.75m main span. The approaches are formed of masonry arch viaducts, four spans on the Anglesey side and three spans on the Bangor Mainland side.

The chains support spiral strand hangers which are connected to a truss in the main span and are anchored to rods embedded in the masonry viaducts on the approaches.

The superstructure comprises a lightweight reinforced concrete deck composite with a 9.5mm thick deck plate and rolled steel cross girders at 0.6m (2' 0") centres. These are supported from the lower flanges of the two 2.6m (8' 6") deep longitudinal steel stiffening through trusses. The deck is 7m wide with a two-lane carriageway with a 1.5m wide footway cantilever out on each side.

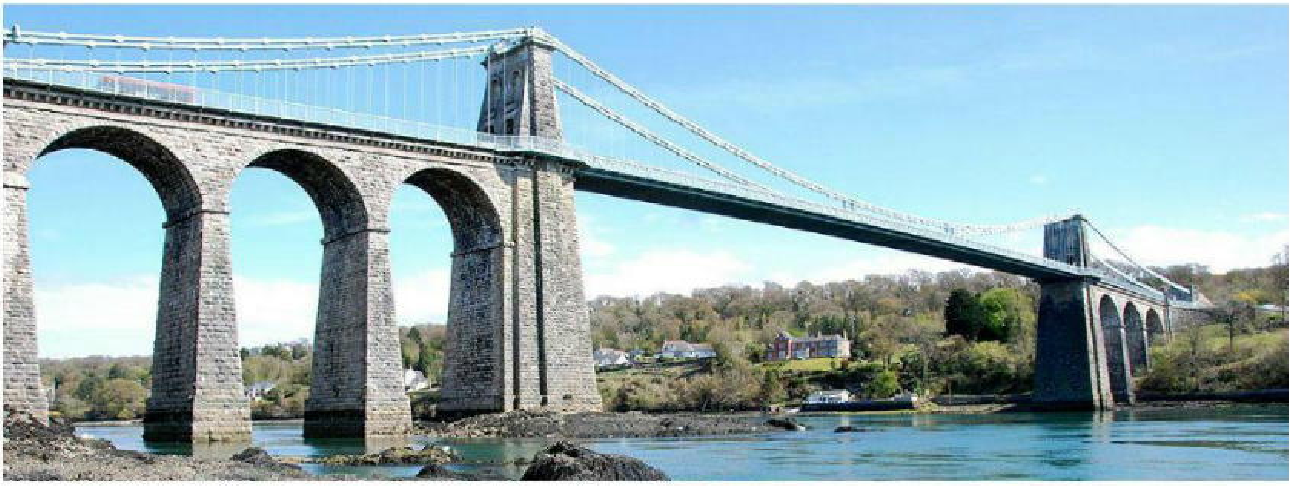


Figure 1: General view of Menai Suspension Bridge

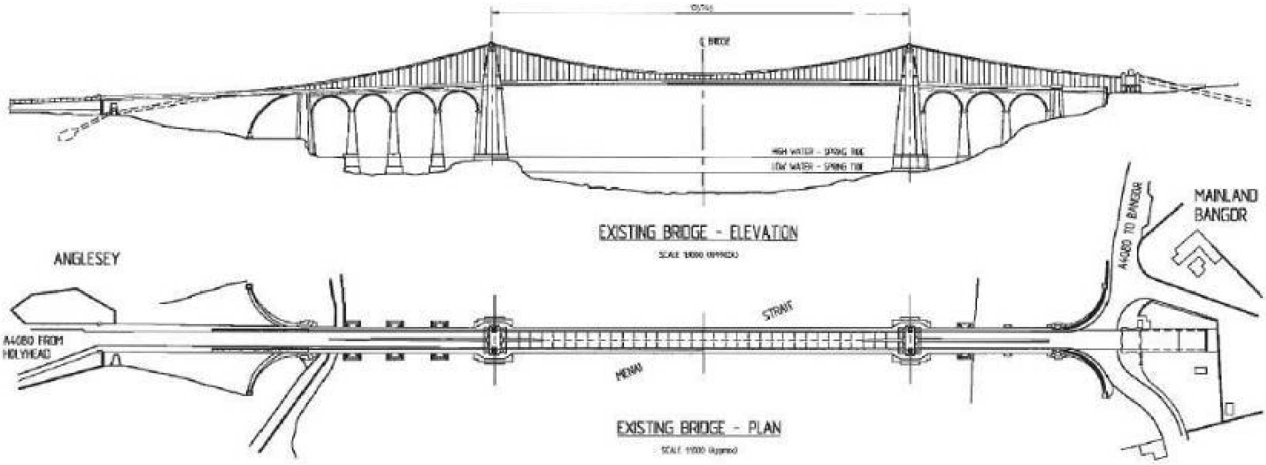


Figure 2: General Arrangement Drawing

2.2 Description of Hangers

The hangers on the structure fall into three groups:

1 Square section steel rods used in the central part of the suspended span (between sway braces)
2 Original (1938-41) spiral strand hangers used on the suspended span and approach spans
3 Replacement (1988-91) spiral strand hangers used on the suspended span and approach spans

The spiral strand hanger assemblies consist of a 1 3/8" (34mm) diameter spiral strand embedded into a cone shaped steel casting with two eyelets through which a pin is passed to form a connection with the main chains. The hangers are alternately pinned to the upper and lower chain levels via $^1/_2$"-1"(12.7-25.4mm) thick suspender plates or 1 1/8"-2" (28.6-50.8mm) suspension links also referred to as "drop links".

On each side of the bridge there are 22No. hanger assembles on the Bangor Mainland (BA) Approach, 69 in the main span (MS) and 30 in the Anglesey (AG) Approach. All hangers are spaced at c.8' (2.4m) intervals.

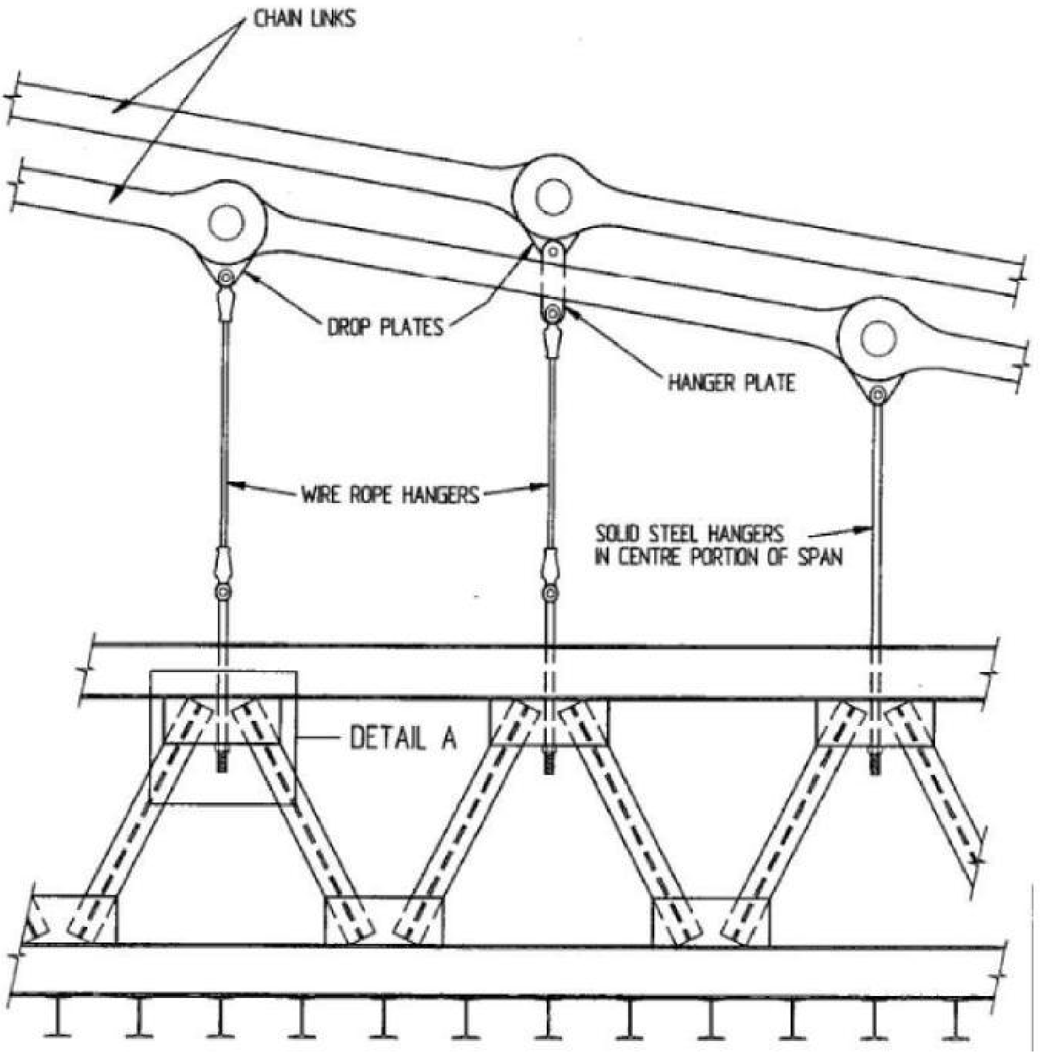


Figure 3: Hanger arrangement on suspended span (from 1994 Maintenance Manual Ref ERB894

Hanger tensions were measured in 1988 and the results are presented in Appendix A of the Gibb & Partners Load Assessment Report (NMWTRA Ref – ER208B).

On the approach spans the hangers serve to tie down the chains, thereby ensuring the chain catenary profile matches the original chain profile. Therefore, these hangers attract dead and live load, but the hanger tensions are generally less than on the suspended spans.

Figure 4: Anglesey Approach Span

Hanger tensions can be adjusted through the turnbuckle arrangement on the approach spans and by turning the seating nut on the main span hangers.

Between 1989 and 1991 extensive investigation of the structural capacity of the hangers was undertaken. These investigations led to the replacement of 40 hangers as outlined below.

Table 1: Hanger replacements to date

Works Phase	Anglesey Approach	Main Span	Bangor Approach
Phase 1 – 4No. replaced at random for testing purposes (1989)		M45E, M45W, M10E, M09W	
Phase 2 – 25No. replaced as deemed most critical due to corrosion of top sockets (1990)	A04E, A12E, A16E, A19E, A20E, A03W, A04W, A14W, A16W, A19W	M12E, M22E, M46E, M48E, M58E, M61E, M64E, M68E, M52W	C06E, C07E, C12E, C18E, C10W, C11W
Phase 3 – 11No. replaced for assessment reasons (1991)		M01E, M01W, M02E, M02W, M68W, M69E, M69W	C21E, C21W, C22E, C22W

2.3 Technical Background

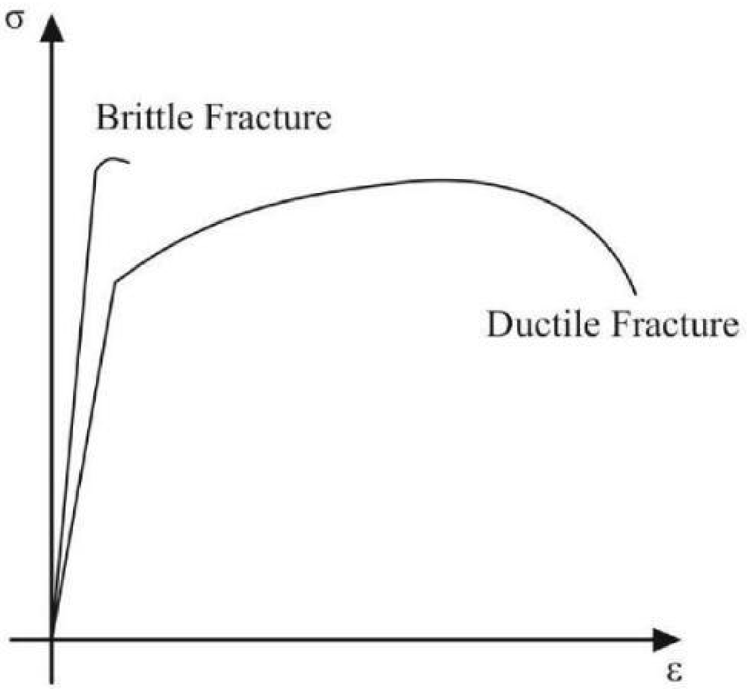
The issue with the original hangers, as identified by Gibb & Partners in the later 1980's, centres on the brittle behaviour of the hanger sockets. This is most unusual as steel structures are designed to behave in a ductile manner.

This section provides a short technical briefing on ductile and brittle behaviour to aid the reader's understanding of later sections of this report and the issues discussed therein.

2.3.1 Ductile v. Brittle Behaviour

The strength and toughness of steel is determined by its chemical composition, metallurgical structure and the manufacturing process e.g. rate of cooling, rolling and subsequent heat treatments.

Modern structural steels are always designed to behave in a ductile manner i.e. to exhibit significant plastic (unrecoverable) deformation before eventual failure. This results in a 'cup and cone' style failure of the steel specimens when submitted to tensile forces (see Figure 5). By contrast brittle materials are not able to plastically deform to the same extent and thus failure occurs suddenly, often with little or no prior warning.



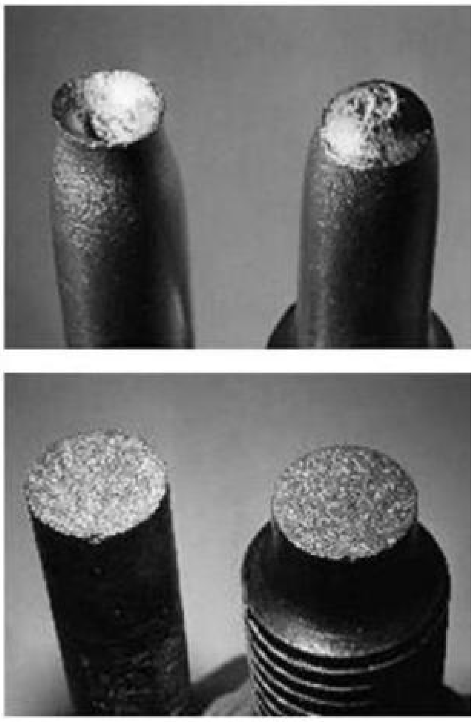


Figure 5 - Comparison of ductile and brittle failure modes

2.3.2 Toughness

In material science, toughness is defined as the ability of a material to absorb energy and plastically deform without fracturing. It is an important parameter for ensuring the material is able to sustain impact loads and/or resist crack initiation and propagation that may lead to the eventual failure of the member.

This parameter is related to the area under the stress-strain curve, see Figure 6. Thus, in order to be a tough material, a material must be both strong and ductile - due to their lack of ductility brittle materials are not tough.

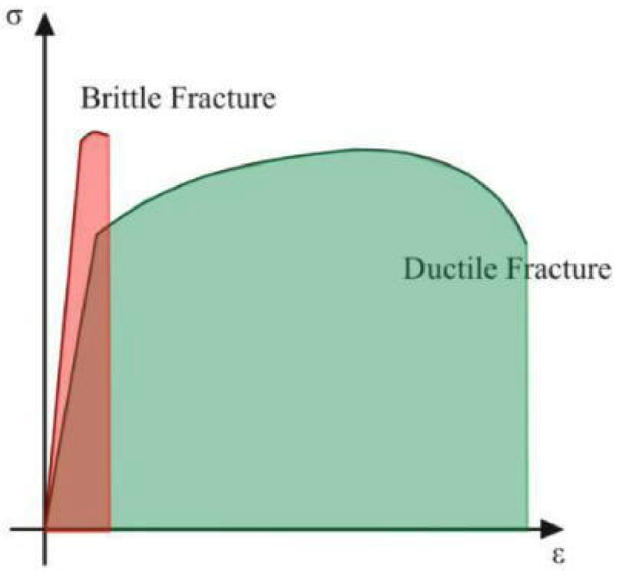


Figure 6 - Toughness of brittle v ductile material

2.3.3 Fracture of Steel

The following factors exacerbate the tendency of steel to fracture:

> Underlying brittle material – the lack of toughness reduces the material's ability to resist fracture

> Low temperatures – these reduce the ductility and thus toughness of an otherwise ductile steel, see Figure 7. The point at which the material moves from ductile to brittle behaviour is known as the 'transition temperature' and structures should be designed to ensure that material transition temperature is below the in-service minimum design temperatures to avoid the onset of brittle behaviour during service.

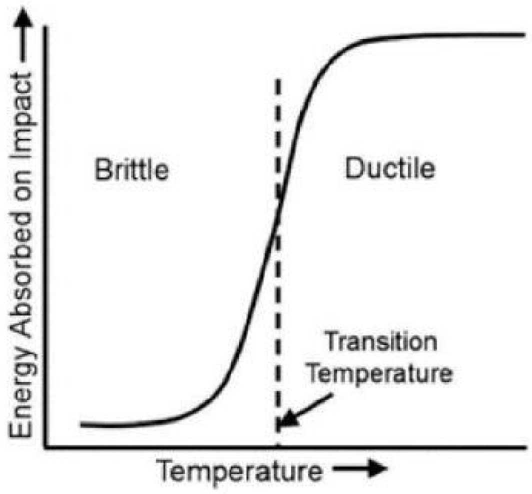


Figure 7 - Transition temperature of steel

- Tensile stress
- High loading rates (e.g. vehicle braking)
- Grain Structure – the size of grains formed in the steel during the original casting process will affect the steel's ability to resist fracture
- Crack initiating defects – a notch or stress concentrating defect that may cause a crack to initiate e.g. an original casting defect, mechanical damage or corrosion.

Brittle fracture of structural steelwork in service must be avoided at all costs, thus, modern design codes require the designer to:

- use steels with suitable ductility at the in-service minimum design temperature (achieved through the selection of a suitable steel sub-grade).
- detail the steelwork to avoid introducing crack initiating defects.
- adopt fabrication methods that do not alter the properties of the steel adversely.

3 Finite Element Model

To undertake the analysis outlined in Section 1.2 COWI has created a Finite Element (FE) model of the bridge.

A FE model contains the following information for each element:

> Geometric properties

> Material properties

> Boundary conditions e.g. supports and rotational freedoms

Once the model is assembled, loading information is added and the forces acting on, and within, each member can be extracted and compared to their calculated capacities.

COWI have created this model of the bridge using LUSAS, a proprietary and widely used FE modelling package.

The following report sub-sections describe the main features of the model and screenshots from the model showing key aspects discussed are provided below.

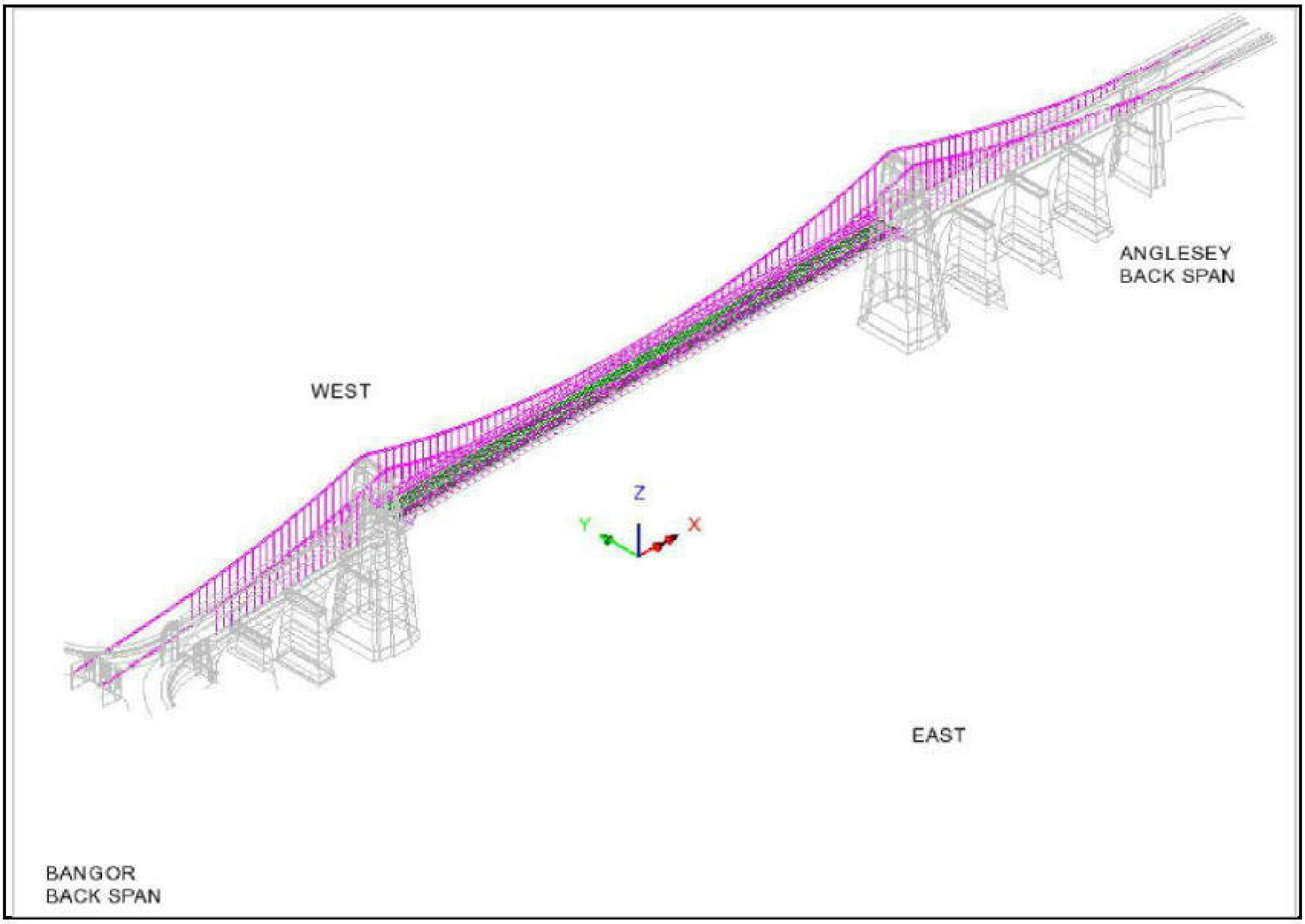


Figure 8: General arrangement of COWI's LUSAS finite element model. Only the coloured lines and surfaces have been meshed; other lines give context

3.1 Geometry

The geometry of the LUSAS model, in particular the main chain and deck profiles, has been imported from the federated model drawings prepared by Mott MacDonald. Lines in the LUSAS model were imported from .DXF file format, via Rhino and .IGES intermediaries, without further manipulation. Hence whilst generally similar, each main chain profile (upper/lower, East/West) is distinct. Accordingly, the structural model is not precisely symmetric in an East/West or North/South sense, due to small differences in surveyed positions of connection nodes, e.g., main chain pin levels.

The imported geometry has been rationalised, split-up and reconnected to establish points and proper line connectivity. Geometric data not provided in the .DXF file have been obtained from reference drawings as listed in the TAF. Refer Figure 8 for the geometry of the completed model.

3.2 Materials

The values in Table 2 have been assigned to relevant lines and surfaces in the model; refer Figure 9.

Table 2: Elastic material properties as used in COWI's LUSAS model

Material	Elastic modulus [kN/m^2]	Poisson ratio [-]	Mass density [t/m^3]	Coefficient of thermal expansion [/°C]
Concrete (short-term [(1)])	30.8×10^6	0.2	2.548	12.0E-6
Steel	205×10^6	-	7.850	12.0E-6
Spiral strand	100×10^6 (2)	-	-	12.0E-6

(1) Regarding use of a short-term elastic modulus value for C40 concrete (calculated in accordance with C455 Cl. 3.5), this is considered most relevant to this study, which is assessing the effects of sudden loss of any one hanger. However, the concrete deck stiffness is not considered especially important; responses are not sensitive to different (lower) stiffness values

(2) This *E* value has been used in conjunction with appropriate cross-sectional area *A* to give the correct extension stiffness of the spiral strand, *EA*.

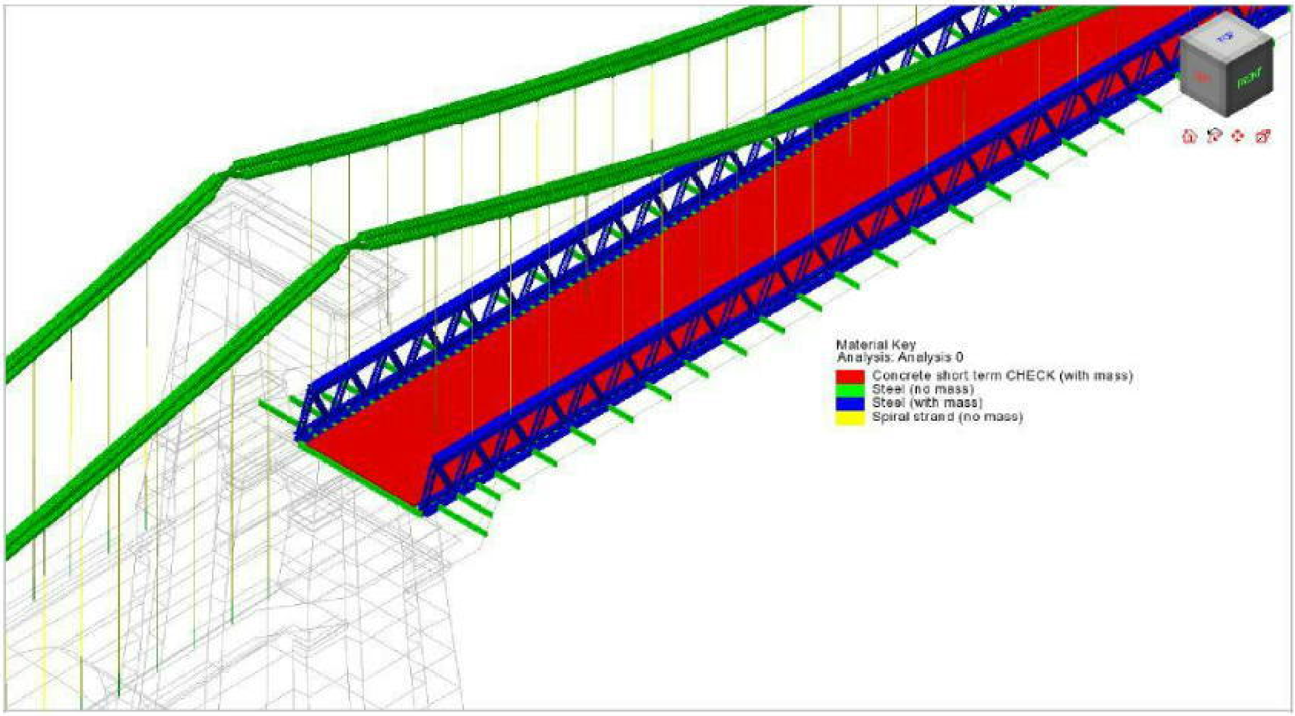


Figure 9: Fleshed mesh geometry, coloured by material assignment

3.3 Mesh

3.3.1 General

The lines representing the main structural components (main chains, hangers, stiffening truss, deck crossbeams) have been meshed via linear thick beam elements. These have section properties assigned to them according to their cross-section geometries, as obtained from reference drawings. Refer Figure 10 and Figure 11 that visualise these assignments.

It should be noted that only the superstructure components, comprising main chains, hangers, stiffening truss and deck, have been modelled; other lines / curves in the model are included for context only. The masonry towers and viaducts are assumed to behave as comparably stiff supports; refer Section 3.3.4 for discussion of model boundary conditions.

The deck concrete has been represented via thick shell elements. These are assumed to act compositely with the deck cross beams (via shear connection) but do not connect directly to the stiffening truss bottom boom lines; refer Figure 12. The tapering thickness of the deck due to cross-fall has been accounted for via linear thickness variation.

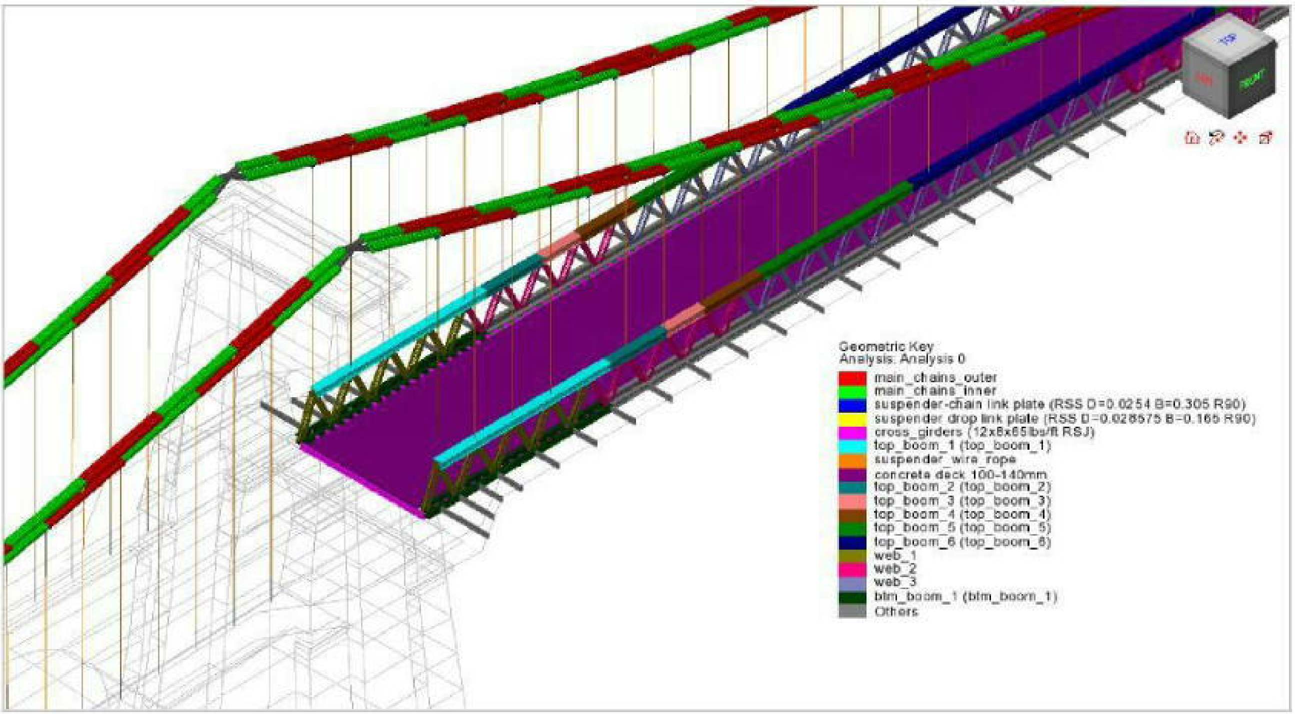


Figure 10: Fleshed mesh geometry, coloured by geometric assignment

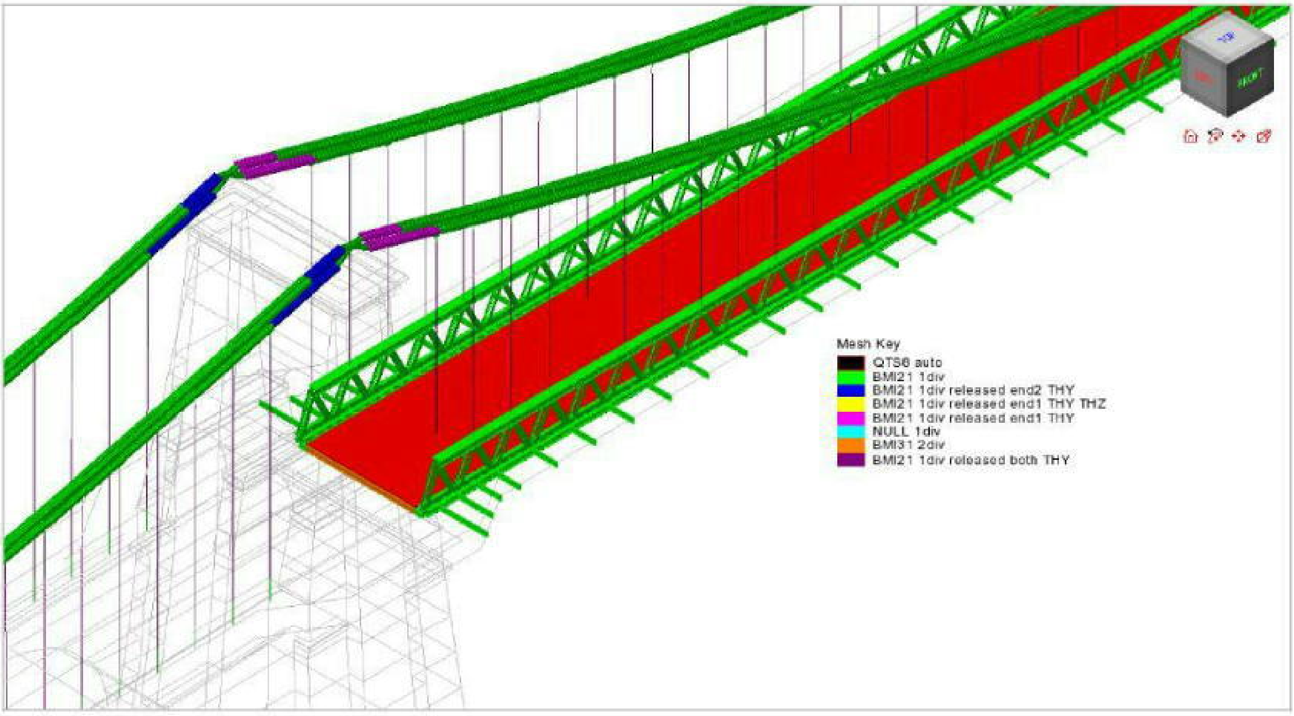


Figure 11: Fleshed mesh geometry, coloured by mesh assignment

3.3.2 Modelling of main chain links

Main chain links are represented by beam elements with no rotational releases defined at pins connecting successive sets of links (e.g., across hanger connections). The only exception to this is at the main chain saddles, where THY rotation releases are provided.

This modelling is informed by our experience from Clifton Suspension Bridge, another historic suspension bridge that has similar main chain detailing.

3.3.3 Modelling of hangers

When pins at provided at the top and bottom sockets of wire rope hangers, these are represented by rotational THY releases about the transverse Y direction, such that no moment is transferred via the pins. It is recognised that this could be a simplification due to friction in pins, but this is a conservative representation that potentially neglects stiffness and eliminates participation of the hangers in potential alternative load paths.

For solid bar hangers near midspan, which have pin details at their upper connections only, similar THY releases are provided at these upper connection nodes only.

All hangers have relevant linkage members included, to represent the link plates and other connection rods. Bending stiffnesses have been included for all hanger types, but spiral strand hangers only bend out-of-plane (e.g., in response to sway due to horizontal wind loading) due to their end releases.

3.3.4 Boundary conditions

External supports are provided to relevant points in the model as tabulated below; refer Figure 12 and Figure 13 for spatial locations of these supports.

Table 3: Boundary Conditions

Location	Restrained nodal freedoms (global axes)					
	DX	DY	DZ	THX	THY	THZ
Main chain anchorages	x	x	x	x		x
Back span hangers; lower connections to viaducts	x	x	x	x	x	x
Tower top main chain saddles		x	x	x	x	x
Stiffening truss end bearings		x	x			

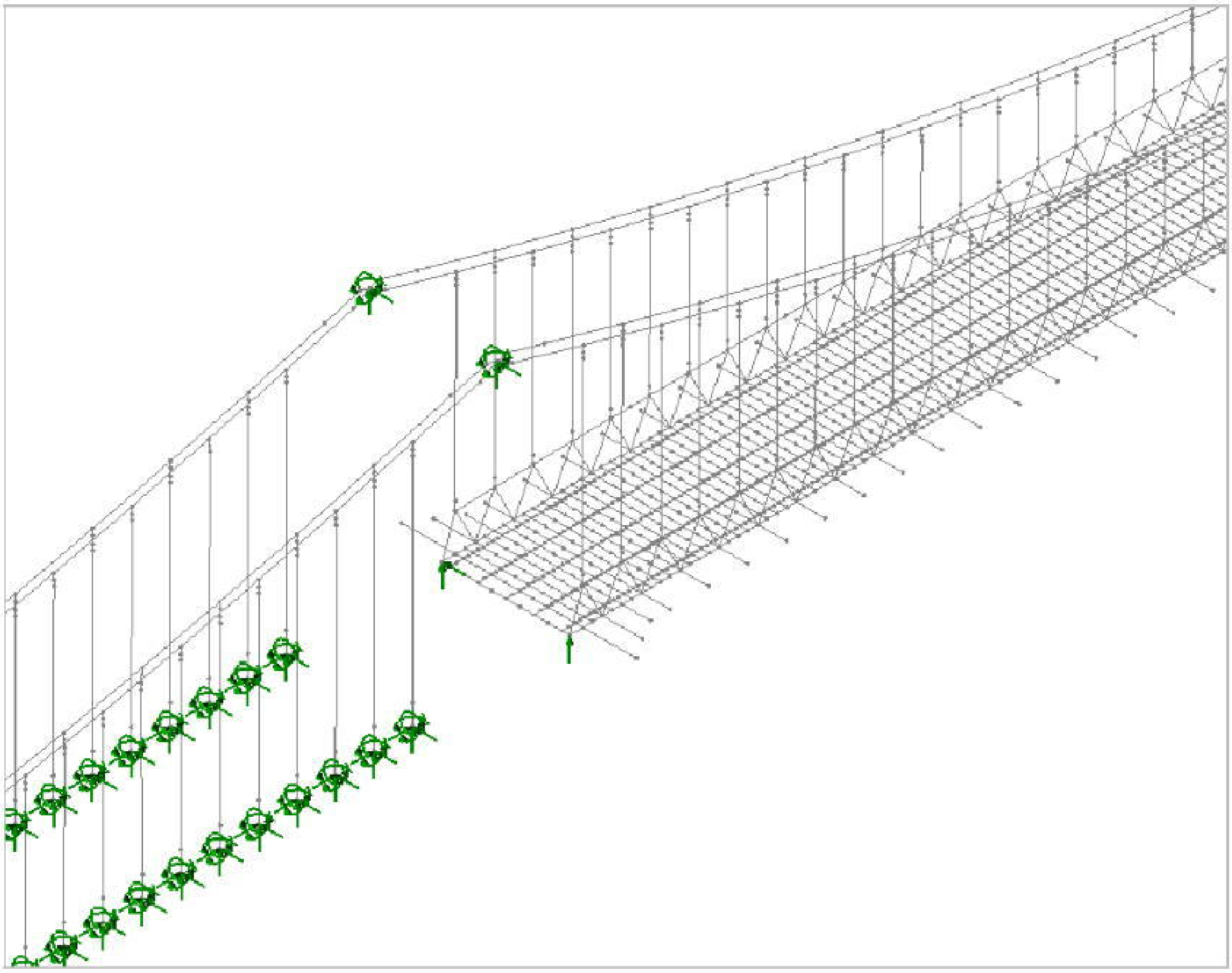


Figure 12: Mesh in the vicinity of the South tower, with supports visualised. End releases, e.g., at the top and bottom of hangers and main chains at the tower saddles, have been omitted for clarity

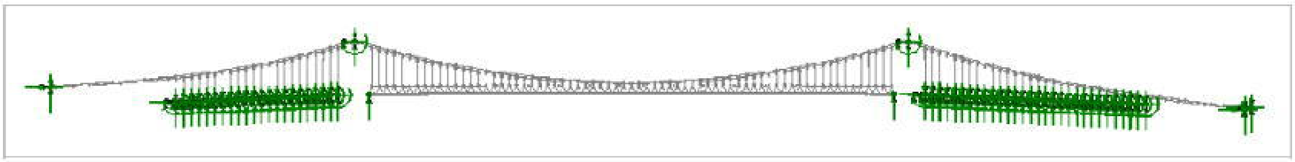


Figure 13: External boundary conditions. Note the back span viaducts and main towers are not modelled; these are represented by fixed translational and rotational supports

4 Loading

4.1 Introduction

4.1.1 Limit States

Modern structural design and assessment codes check a structure for various 'limit states', as follows:

- Ultimate Limit State (ULS) – this considers the strength and stability of a structure under worst case design loads e.g. ensuring the bridge won't collapse.

- Serviceability Limit State (SLS) – this considers the useability of a structure and comfort of users under typical in-service loads e.g. ensuring deflections are not excessive.

- Accidental Limit State (ALS) – this considers the effect of exceptional circumstances e.g. fire, explosion, impact or the consequences of localised failure on the wider structure.

A structure must satisfy all relevant limit states to be considered acceptable. Acceptability is defined as the capacity of all members and connections being equal to or greater than the applied loads for a given service state.

The limit states considered in this assessment are ULS and ALS as outlined in Sections 7 and 8 of this report.

4.1.2 Loads

The loads acting on a structure are split into two categories:

- Permanent or Dead Load e.g. the weight of the structure

- Variable or Live Load e.g. traffic, wind, thermal effects

The loads considered in this assessment are outlined in Sections 4.2 and 4.3 of this report.

4.1.3 Partial Factors

Partial factors are applied to the loads to factor-up the load effects to account for the uncertainty in the loads. Since partial factors serve to make the nominal loads more extreme, it follows that larger partial factors are applied when assessing the Ultimate Limit State than at other limit states

The partial factors used in this assessment are outlined in the relevant sections of the report.

4.2 Permanent Load

4.2.1 Global Assessment of Loads

Reviewing the historical reference drawings for the bridge and using the material densities defined in Table 2, the nominal weights of all superstructure components has been estimated; refer Table 4.

Note: the weights of the masonry towers and back-span viaducts have not been evaluated, since these are not relevant to the current study or modelling.

Partial factors as defined in Table A.1 of CS 454 (refer Table 5) have been applied to the different weight components to obtain the ULS-factored main span and full superstructure weights for the different limit states; refer Table 6.

Table 4: Nominal permanent load breakdown by component and span

Component	Main span (kN)	Back spans (kN)	TOTAL (kN)
Main chains	4272	4965	9237
Hangers	201	163	364
Stiffening truss	3677	0	3677
Deck	7201	0	7201
Surfacing	1200	0	1200
Walkways	867	0	867
Parapet	340	0	340
Non-structural	270	0	270
TOTAL	**18,028**	**5,128**	**23,156**

Table 5: Partial factors for dead load, from CS454

Action	ULS partial factor	SLS & ALS partial factor
Steel dead load	1.05	1.00
Concrete dead load	1.15	1.00
Surfacing dead load	1.75	1.20
Other superimposed dead loads	1.20	1.00

Table 6: Permanent load totals for different limit states

Limit state	Main span totals (kN)	Full superstructure totals (kN)
Nominal	18,028	23,156
SLS (also ALS)	18,268 (=Nominal + 1%)	23,396
ULS	20,530 (=Nominal + 14%)	25,914

4.2.2 Estimation of permanent hanger forces

Since the main chain profiles have been established from survey data, the hanger forces are estimated via the same method as was applied in COWI's previous assessment of the approach span hangers (COWI Ref *A238719-TN02-v1.0)*. This is simply a geometric analysis of the 4No. main chains, considering the static equilibrium at each of the hanger-chain connection nodes. This comprises the following main steps:

1 Given the total suspended permanent load for a given limit state, by assuming equal load share between the main chains and between the 4No. corners of the main span the total vertical load carried by each set of main chains is determined.

2 Given the surveyed inclination of the main span chains where they connect to the towers is similar (15.7°, all chains), the horizontal component of tension being carried in each set of chains is back calculated using the tangent of this angle and the common vertical tension component carried by chains (from step 1).

3 Since the hangers are all very close to vertical, as established by the survey data, the horizontal tension carried by all links in each chain must be uniform within the main span, being equal to the value at the tower saddles.

4 Since the self-weight of the main chain links and upper connection nodes has been estimated from reference drawings, and the change in inclination across each hanger is known from the survey data, the tension force being carried by a given hanger is estimated from consideration of vertical equilibrium.

It is recognised that the calculated hanger permanent loads could differ from reality due to one or more of the following:

> Errors in the calculation of suspended dead load; if the main span dead loads were 10% larger than estimated, estimated hanger forces would also be 10% greater (aside from any weight differences attributed to the chains / upper hanger connections)

> Errors in the angular deviation of the main chains across a given hanger, e.g., due to the finite precision of the survey upon which the main chains profile is based

Regarding handling of the 14% additional permanent load as implied by ULS partial factors (refer Table 6), the above approach has been applied using the factored weights as input. Hence the ULS permanent loads in the hangers are distinct from SLS (being approx. 14% larger). ULS permanent loads have been used for ULS assessment not considering hanger loss. ALS permanent loads, which corresponds to SLS permanent loads due to similarity of partial factors, have been used in the hanger loss analysis.

ULS permanent loads in individual main span hangers determined in this way are visualised via Figure 15. Similar results for the Bangor and Anglesey back-span hangers are given in Figure 16 and Figure 14 respectively. Note that the nomenclature for hangers adheres to the convention established via the DBFO reference drawings. SLS/ALS hanger loads are approximately 13% less than these values, due to the ratio of total suspended dead load; refer Table 6.

It can be seen from these Figures that there is significant variability in permanent hanger forces as estimated by this approach, i.e., some hangers carry higher or lower than average tensions. Statistics describing the variability, for the different hanger groups, are given in Table 7.

Table 7: Statistics to describe ULS hanger tension variations

Hanger group	Mean ULS hanger force [kN]	Standard deviation [kN]	Coefficient of variation
Main span	116	20	17%
Bangor back span	119	22	19%
Anglesey back span	78	20	26%

Examining the spatial variation in permanent hanger loads within the main span, where a given hanger has above-average load, the adjacent hangers *connecting to the same main chain*, i.e., alternate hangers, typically have below-average load. This points to the physical cause of the uneven tensions; since the structure is highly statically indeterminate, owing to the very deep stiffening truss, it is possible to over-tension or under-tension a given hanger significantly by making relatively small length adjustments.

It should be noted that COWI is not aware of the effective bridge temperatures when the Mott MacDonald survey was carried, from which hanger forces have been back-calculated. If temperatures were significantly different from the ambient yearly average value (say 15°C) this could further reduce the precision by which the permanent hanger loads can be estimated.

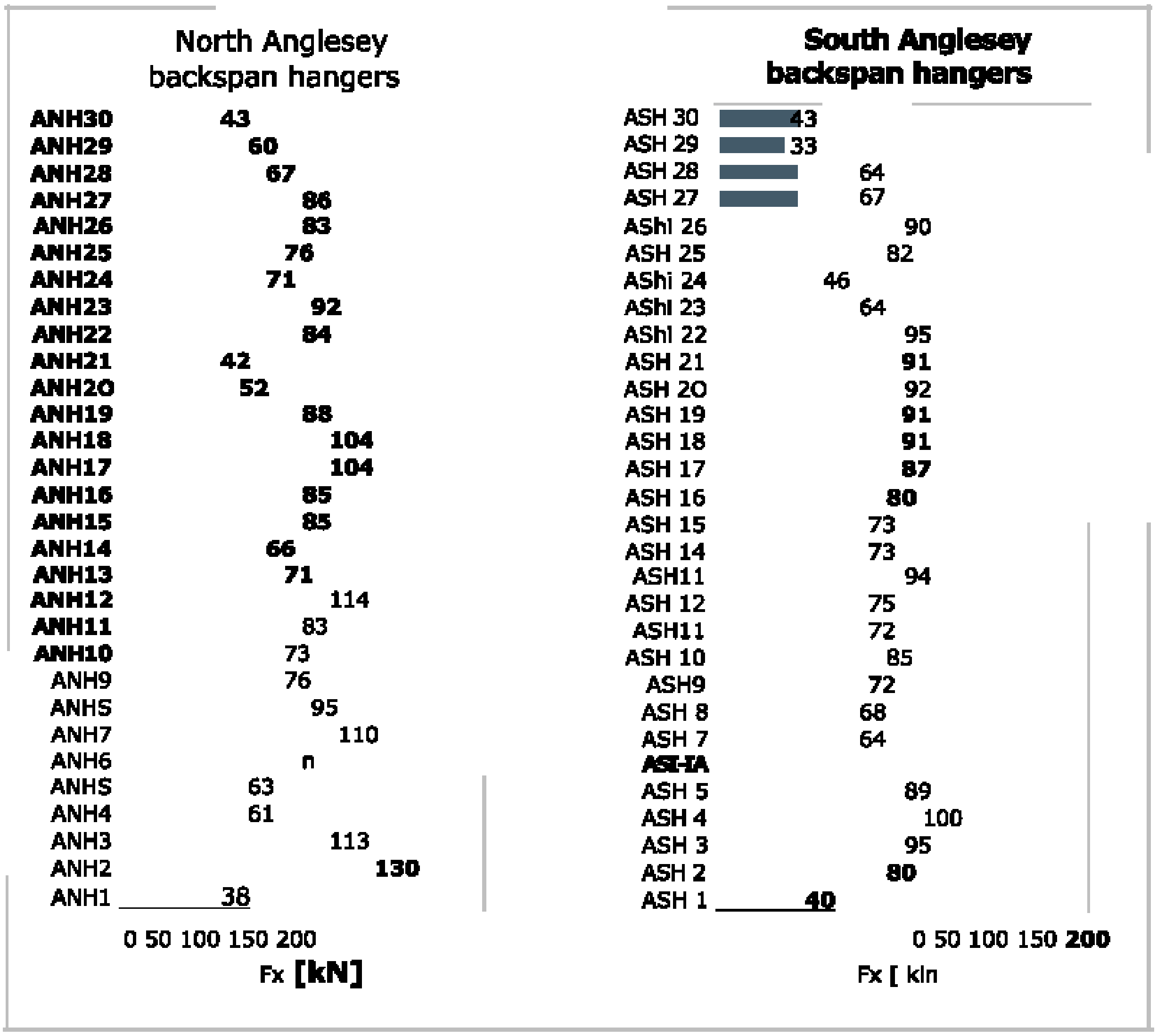


Figure 14: ULS permanent hanger loads: Anglesey back span

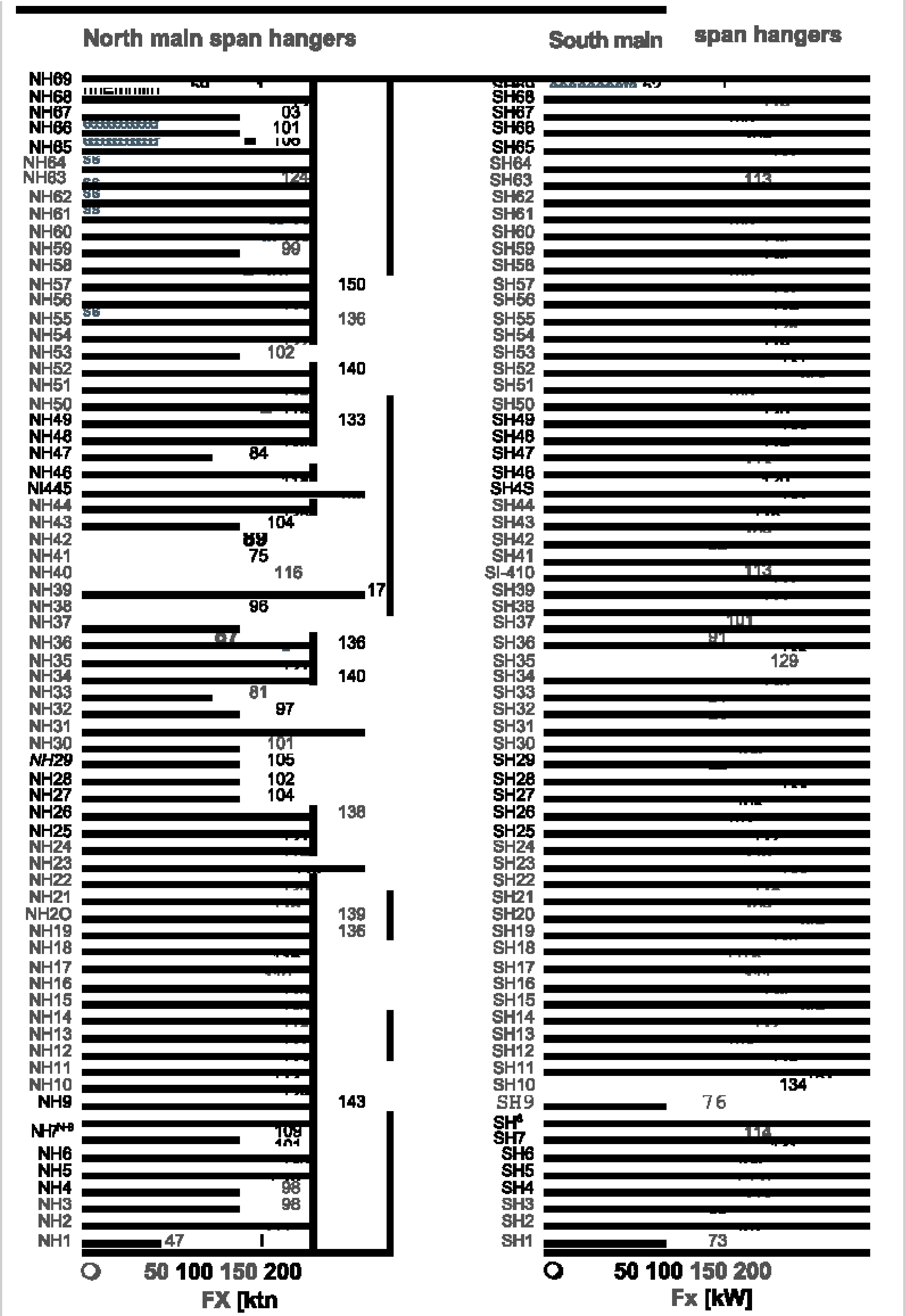


Figure 16: ULS permanent hanger loads: main span

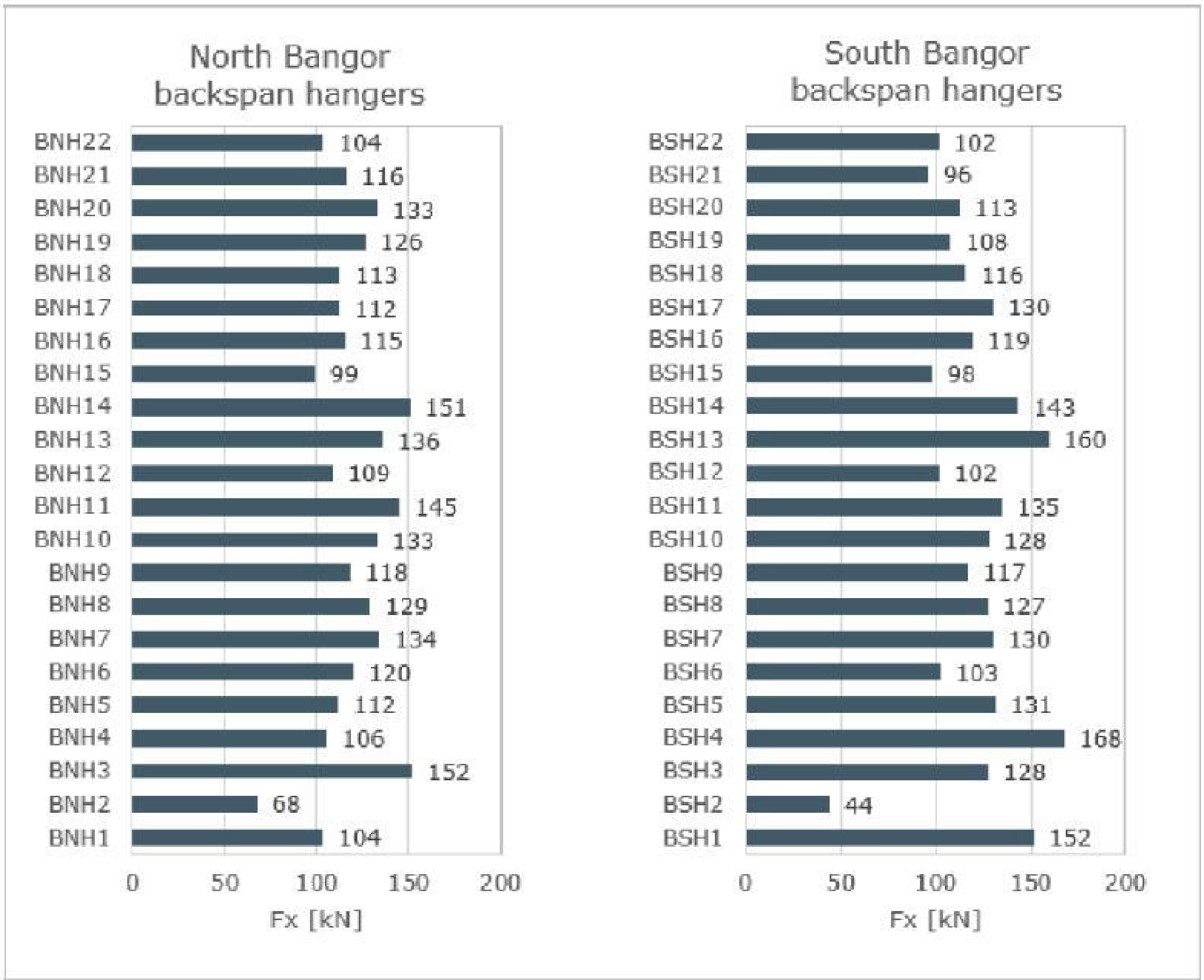


Figure 16: ULS permanent hanger loads: Bangor back span

4.2.3 Sensitivity of permanent hanger forces

The type, scope and accuracy of the survey on which the FE model is based is described in the Mott McDonald report reference *"400757AD-01 Menai Bridge - Topographic Survey Scope of Work - Rev. A"* dated 29th July 2020. The survey was undertaken in accordance with specification "Measured surveys of land, buildings and utilities – RICS guidance note, global – 3rd edition", which includes an accuracy banding table for specific accuracy designations A, B, C etc. The chain pin positions were designated accuracy band C which has a vertical height tolerance of +/- 5mm.

The magnitude of the chain tensions means that even small variations in the chain geometry and inclination have a significant impact on the calculated hanger tensions. A sensitivity check has therefore been undertaken using a Monte Carlo simulation to understand the consequences of adding a measurement error of Δz +/-5mm to the surveyed vertical positions of the chain pins.

The results of this analysis are shown in the plots provided in Appendix B.

With the given accuracy Δz = +/-5mm the range of hanger forces is ±10kN at ULS and ±8kN for SLS, relative to the average values from this analysis.

4.2.4 Comparison versus measured hanger forces

To benchmark our permanent load analysis, we have made comparison against the 1988 hanger force measurements obtained by Laing O'Rourke (LoR); refer Figure 17 and Table 8. Note LoR's measurements are obtained from the January 1989 Appendix A to the Gibb & Partner Load Assessment Report (NMWTRA Ref ER208B-1).

We note the following:

> Mean main span nominal hanger forces are very similar; the COWI values are 2-3% larger than those measured by LoR

> The variability of permanent hanger forces across different hangers, expressed by the sample standard deviation, is similar

> There exists strong correlation between the sets of hanger forces, i.e., those hangers identified as under- or over-tensioned from LoR's measurements are generally identified as such in COWI's analysis also, albeit with some scatter

Given the above, we conclude that the total suspended dead loads calculated by COWI's approach closely resemble reality. We note that since 1988 there have been structural modifications that, whilst not presenting as an overall suspended weight change, could have led to some redistribution of hanger tensions.

Given that field measurements have been obtained that appear to closely resemble those from the nominal analysis, i.e., with no loading partial factors applied, one could make a case for using reduced partial factor values, since the uncertainty in permanent loading has to some extent been reduced through taking such measurements. This has not been done in COWI's current analysis however; CS 454 partial factors, in particular 1.15 on concrete dead load and 1.75 on surfacing, continue to be applied, leading to an overall aggregate partial factor on dead load of 1.14 for ULS (refer Table 6).

Table 8: Comparison of in-situ hanger force statistics

Statistic	Laing O-Rourke, 1988 measurements	COWI, nominal (i.e., from unfactored loads)
Mean – West main span hangers	96.4	99.1
Mean – East main span hangers	96.0	98.9
Mean – All hangers	96.2	99.0
Standard deviation – All main span hangers	20.5	17.7

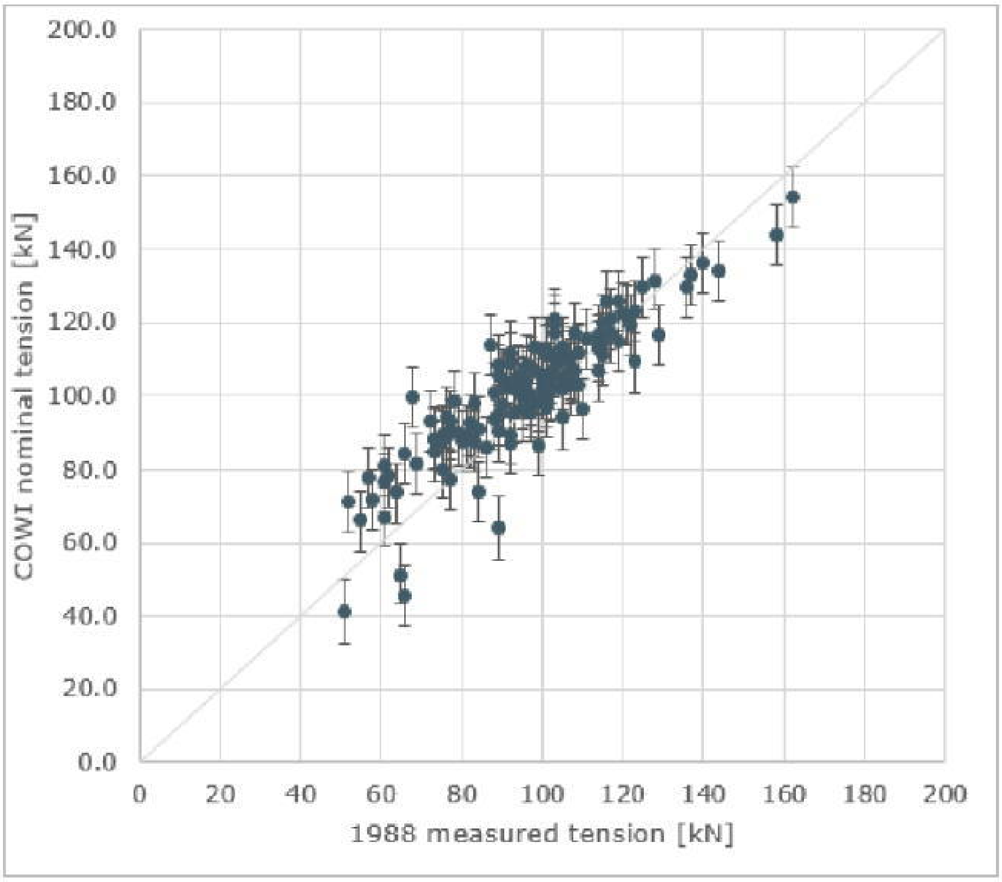


Figure 17: Correlation of Laing O'Rourke's 1988 measurements versus COWI nominal tensions, as obtained from considering unfactored permanent loads and analysing the chain geometry / deviation angles across hanger connections

Vertical error bars relate the precision of the chain geometry survey; these describe the range of permanent load values determined by Monte Carlo simulation as described in Appendix B. The precision of the 1988 tension measurements is not known.

4.3 Variable Load

DMRB CS 454 *(Assessment of Highway Bridges and Structures)* defines the variable loading model(s) for highway bridges. These load models account for:

1 The traffic restriction level to be assessed

2 The influence of the road surface category on the impact effects of vehicles

3 The influence of the traffic flow category on the likelihood of vehicle overloading and lateral bunching

Clause 5.5.2 of CS 454 defines two load models and provides commentary on their derivation, this is reproduced in full in Figure 18 to aid the reader's understanding.

5.5.2 The characteristic traffic actions for normal or restricted traffic should be represented using either one of the following assessment live loading (ALL) models:

1) ALL model 1;
2) ALL model 2.

NOTE 1 *ALL model 1 is suitable for all structures and is based on real vehicles with maximum authorised vehicle weights.*

NOTE 2 *ALL model 2 is suitable for longitudinally spanning bridge decks and is based on nominal Type HA loading as previously included in BD 21 (withdrawn).*

NOTE 3 *ALL model 2 is likely to provide lower effects than ALL model 1 for longer loaded lengths, since it accounts for the reduced probability of the most critical loading effects being experienced on the entire loaded length simultaneously.*

NOTE 4 *ALL model 1 and ALL model 2 both include the effects of road surface category and traffic flow category.*

NOTE 5 *Although they are referred to as characteristic actions, both ALL model 1 and ALL model 2 can be more strictly described as nominal actions that when multiplied by the partial factors in this document provide an assessment load level. The values of the partial factors at SLS and ULS are calibrated on that basis and include an allowance for possible overloading. In contrast, the corresponding characteristic load models in BS EN 1991-2 [Ref 11.I] are higher in magnitude but the associated partial factors in BS EN 1990 [Ref 12.I] are lower.*

Figure 18: ***Vehicle Load Models (from CS 454)***

4.3.1 ALL Model 1

Clause 5.8 of CS 454 defines ALL Model 1 as two separate loading scenarios:

1 A single vehicle in each lane.

2 A convoy of vehicles in each lane.

The weight of these vehicles is determined from the vehicle loads in Appendix B of CS 454 (to correspond with any weight restriction) and modified to account for impact, traffic flow and number of lanes.

For Menai Suspension Bridge this loading model represents either a single 7.5T lorry in each lane (scenario 1) or a convoy of 7.5T lorries at 1m separation over the full length of the span (2). However, as outlined in Section 5 the convoy is not considered a realistic scenario for Menai and the single vehicle does not maximise hanger forces.

4.3.2 ALL Model 2

Clause 5.17 of CS 454 defines ALL Model 2 as consisting of the following loads, applied separately:

1 A combined uniform and knife-edge load;

2 A single axle load.

Again, the single axle load is determined from the vehicle loads in Appendix B (to correspond with any weight restriction).

4.3.3 Selection of Load Model

COWI have adopted ALL Model 2 for this analysis, Section 5.2.2 provides the rationale for this decision.

4.3.4 K Factors

DRMB CS 454 introduces a K factor to modify the vehicle loads to account for road surface quality (i.e., 'good' or 'poor') and traffic volumes ('high', 'medium' or 'low'). The K factor varies with the loaded length of the bridge.

For base lengths L > 50m, Table 5.19c in CS 454 applies; this defines the K-factors for 'Normal' and 7.5t-restricted traffic as 0.91 and 0.40 respectively. For these longer loaded lengths, it should be noted that no distinction is made according to road surface quality or traffic volumes.

It should be noted that the current draft of CS 454 also does not define what K-factors are applicable when considering loads arising from 26t and 18t restricted traffic. K-factors for L>50m for 18t and 26t traffic cannot be reliably interpolated by extrapolation of the L=50m values, since K(L=50) ≠ K(L>50) for Normal and 7.5t restricted traffic.

In our analysis the K-factor for 'Normal' traffic has been conservatively applied when determining loads arising from 18t and 26t restricted traffic using the ALL2 load model. Practically this means that, given L>50m is typically found to be critical when determining the optimal load pattern for almost all hangers, the maximum hanger forces resulting from 18t and 26t restricted traffic are identical to those results from 'Normal' unrestricted traffic.

5 Analysis Methodology & Structural Response

Before considering main span hanger response to the codified live load models, it is instructive to first consider the structural response of the hangers to simpler applied loading scenarios. The following sections of the report review these scenarios and outline COWI's understanding of the bridge's response.

5.1 Live load response of main span hangers to vertical uniform test load

To review the live load response of the main span hangers, a 30kN/m uniformly distributed vertical load has been applied at the deck centreline. Given hangers are at 8ft = 2.44m centres, based on a simple tributary area approach we would expect a change in hanger tension of (30 x 2.44)/2 = 36.6kN.

Having run this load case as a nonlinear analysis, following from the balanced ULS permanent load case, then subtracted results to identify the change in hanger tensions, the results presented in Figure 19 are obtained (the live load response of each hanger).

The following can be noted:

- Response to this symmetric response is broadly North/South symmetric; there are slight differences due to the small differences in main cable profiles
- As expected, near the ends of the bridge the live load is primarily conveyed to the tower piers via the stiffening truss
  - the change in hanger force in the East and West hangers at 1N and 1S is negligible (and in some cases slightly negative)
  - Of the total 5025kN load applied, 90% of this is being carried by the hangers to the main chains, i.e., 10% of the load applied nearest to the ends of the span is being conveyed direct to the abutments via the stiffening truss
- As you move further into the span, changes in hanger force due to this load case become more uniform, with hangers carrying force according to their tributary area
- Between hangers H26 & H27 and H43 & H44 there is an abrupt change in hanger geometry and stiffness, with hangers switching from short wire ropes to longer square bars. This causes a disruption to the pattern of hanger forces, with hangers H29 & H30 and H40 & H41 attracting significantly higher than average changes in tension, hangers H25 & H26 and H44 & H45 attracting significantly lower tension changes, due to the change in stiffness.

These characteristics, which relate to the structural configuration and relative stiffnesses of the bridge superstructure components, can be seen in the ALL1 traffic load case responses also, as presented later in this report.

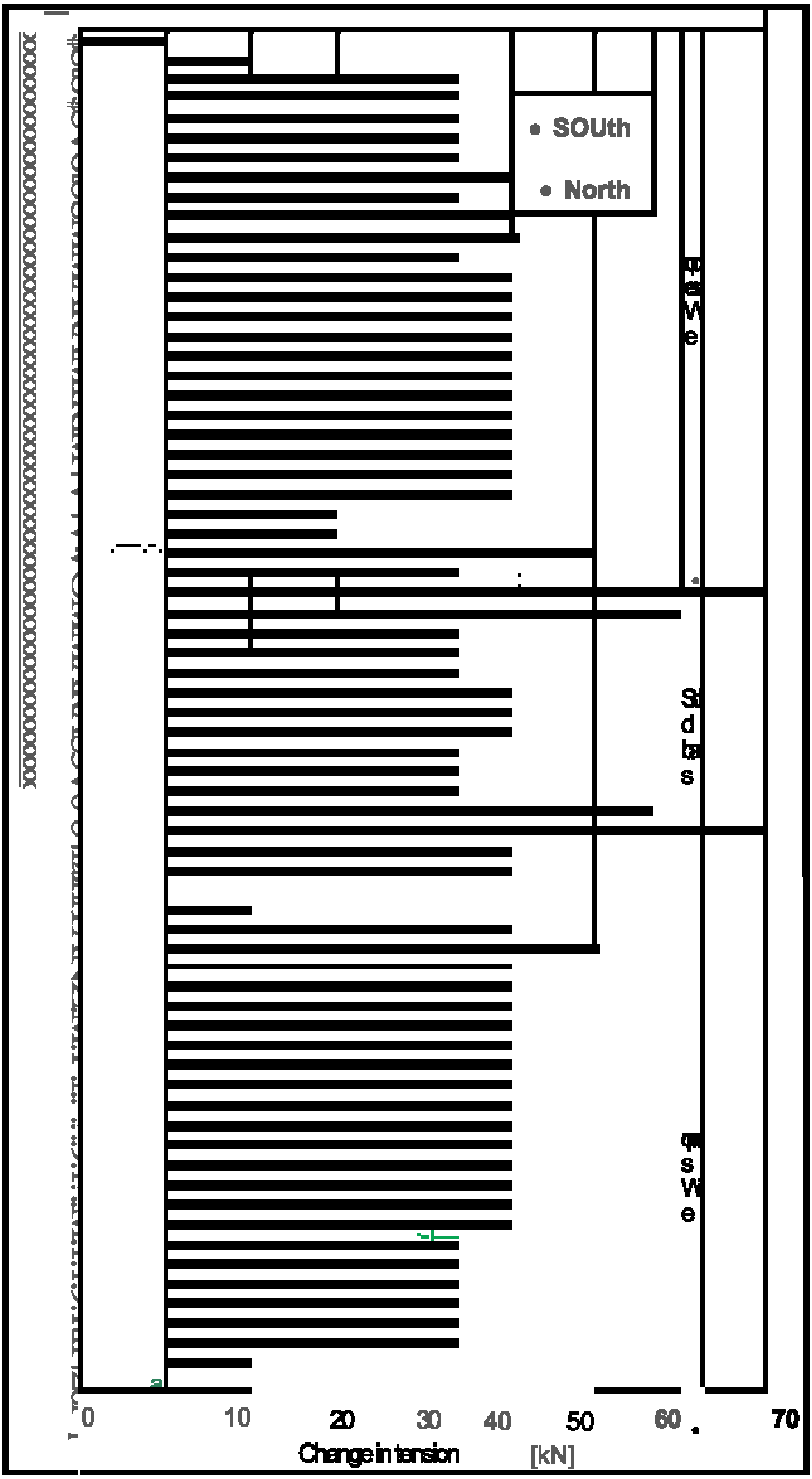


Figure 19: Change in hanger tensions in response to 36kN/m UDL applied at deck centreline

5.2 ULS live load responses of hanger for different traffic restrictions

5.2.1 Linear Influence Surfaces

Linear influence surfaces for the roadway have been calculated for 23No. individual hangers in the SW quadrant, Figure 20 presents contour plots for some of these. These influence surfaces show how changing the tension or length of the hanger in question affects the tension or length of the adjacent hangers.

Due to the deep stiffening truss, these effects are surprisingly global i.e. a change in one hanger affects a significant number of adjacent hangers. For more conventional stiffening truss geometries hanger influence surfaces tend to be more local to the hanger being considered.

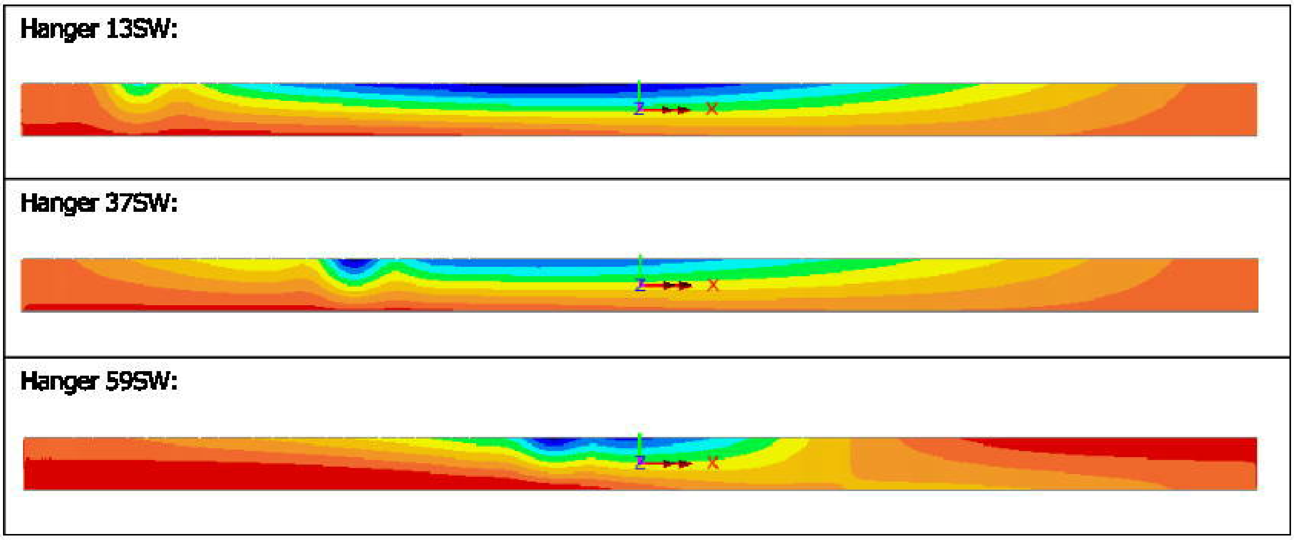


Figure 20: Roadway influence surfaces for axial force in 3 distinct hangers in the SW quadrant, as obtained by reciprocal theorem / unit elongation approach

Contours present vertical displacements, which are all adverse (downward) with the largest negative DZ values coloured blue, with regions of low influence (small negative DZ) coloured orange / red. Influence surfaces for hanger axial forces are surprisingly global, in terms of the extent of the significant adverse areas; this is considered be due to the unusually deep stiffening truss

Using these influence surfaces, optimal (i.e. worst case) live load patterns have been determined, using LUSAS' built-in Vehicle Load Optimisation facility. Optimal live load patterns have been defined according to both the ALL1 and ALL2 load models, for each level of traffic restrictions.

Hanger forces in all hangers (i.e., not just those for which live load has been optimised) have been extracted for use in hanger loss calculations.

5.2.2 Discussion regarding the ALL1 and ALL2 load models

When applying the ALL1 load model, optimal vehicle patterns, given the hanger axial force influence surfaces, have been found to be convoys of self-similar vehicles, of types restricted according to the traffic restriction level being considered.

When seeking to maximise SW quadrant hanger tensions, vehicles are positioned transversely to the +Y side of the carriageway in each notional lane. Remaining area UDLs have been applied in conjunction with these loads as appropriate (i.e., for all traffic restriction levels except 7.5t).

Details of the critical vehicle types identified are provided in Table 9. Given hanger influence lines are in general long (adverse regions typically have length L>50m, often L=167.5m i.e., the entire main span), the critical vehicles are in general those that present the largest effective mass per length; refer Table 9 for details.

Table 9: Details of critical vehicle types as obtained from load optimisation analyses

	Traffic restriction level			
	Normal	26t	18t	7.5t
Critical vehicle type	A	K2	M	N
Gross Vehicle Weight [t]	32	26	18	7.5
Vehicle length, incl. overhangs [m]	8.40	6.72	5.00	4.00
Vehicle length incl overhangs, incl. 1.0m min inter-vehicle spacing [m]	9.40	7.72	6.00	5.00
Effective mass per length [t/m]	3.40	3.37	3.00	1.50
Effective UDL [kN/m per lane]	33.4	33.1	29.4	14.7

It can be seen from Table 9 that the maximum effective mass per length under 26t restricted traffic is only marginally less than under Normal (unrestricted) traffic. It follows that the maximum ULS hanger loads determined according to the ALL1 load model for 26t restricted traffic are on average 97% of those generated by Normal traffic. The 18t and 7.5t restrictions lead to hanger forces that are on average 87% and 41% of Normal respectively. Hence it is only the 7.5t weight limit that offers a significant reduction in maximum ULS hanger forces generated by traffic.

However, COWI have concerns that the ALL1 load model may be unduly conservative, especially when this implies that convoys of similar vehicles should be applied across the full span and in both lanes, in conjunction with remaining area UDL load. In contrast, the ALL2 load model tailors the UDL load intensity to the critical base length as informed by lane influence lines, capturing the reduced probability associated with long convoys of heavy vehicles.; Cl. 5.5.2 NOTE 3 in CS454 discusses this (see Figure 18).

According to the ALL2 load model, for a critical base length of L=167.5m (i.e., loading the whole of the main span), for 'Normal' traffic the unfactored UDL load intensity in Notional Lane 1 is $0.91 \times 36 / (167.5)^{0.1} = 19.6$kN/m. This compares to an effective UDL load of 33.4kN/m according to the ALL1 load model (refer Table 9).

Notwithstanding additional loading arising from the application of the KEL according to the ALL2 load model, plus implementation of the cusp rule as applies when optimising load patterns for some hangers, this means that the ALL2 load model gives rise to hanger forces that are approx. 60% of the hanger forces given by the ALL1 load model. It should be noted however that the ALL2 load model gives rise to hanger forces that exceed those arising from the ALL1 single vehicle load model. Refer to Figure 21 which compares maximum hanger forces calculated according to each of these different load models.

Since the ALL2 load model is considered applicable with reference to Cl. 5.5.2 of CS 454 (including all NOTE clauses) yet leads to lower demands on hangers, maximum hanger loads as obtained using the ALL2 load model have been taken forward for use in both ULS and ALS assessment work.

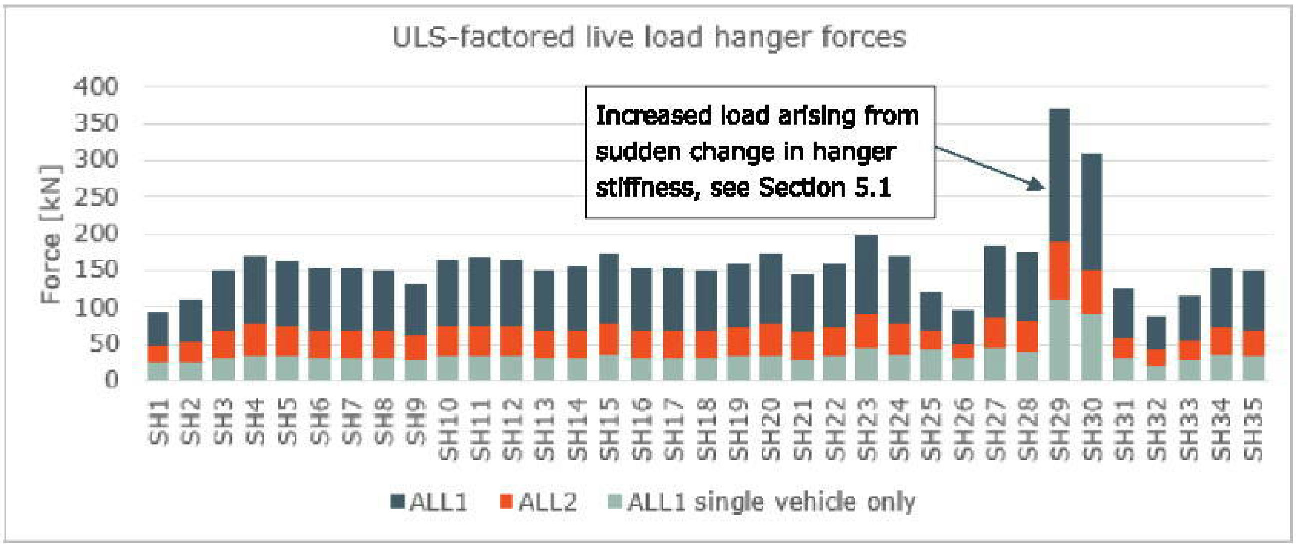


Figure 21: Comparison of SW quadrant ULS-factored live load forces arising from the application of different CS 454 load models

5.3 Consideration of system nonlinearities

The influence surface - based optimised live load analysis (together with the method for accounting for dynamic hanger loss, described in Section 8.1) intrinsically relies on the structural response being linear – or at least sufficiently linear to justify the use of linear superposition.

Suspension bridges are often regarded as highly nonlinear structures, and this is indeed true for some aspects of their structural response (e.g., longitudinal 'swing' displacement response to asymmetric global loads). However, it has been demonstrated by running a subset of the optimised load patterns nonlinearly, following on from the balanced ULS dead load case, that the difference in hanger forces obtained in comparison to a linear approach is not significant, typically less than a few percent.

This has also been shown to hold true when considering live load patterns applied in conjunction with the deactivation of hangers, i.e., when hanger loss is modelled explicitly. Hence, we conclude that tension-stiffening and large displacement effects do not significant alter the structural behaviour, at least the behaviours relevant to this study.

Hence for the remainder of this study a linear approach has been pursued in general. This is considerably more efficient and allows the implications of the various traffic restrictions and hanger loss cases to be more rigorously explored. The latter is important given the variability and uncertainty regarding the permanent hanger loads, coupled with the patterns of replaced hangers, coupled with the non-uniform response of the hangers to applied vertical live load, as discussed in Section 5.1.

6 Hanger Capacities

6.1 Design tension resistance of hangers

Given the material properties of the distinct hanger types, as described in the TAF, the ULS design tension resistance of each of the hanger types has been determined.

N.B. ULS design tension resistances have been used in both ULS and ALS tension resistance verifications.

6.1.1 1938-41 spiral strand hangers

The capacity of the 1938-41 hangers are the primary cause of this study. Testing has shown failure to typically be governed by brittle fracture of the end sockets. In 1991 Gibb & Partner determined the design tension resistance of this type of hangers to be **285kN**; this value has been adopted for use in this assessment.

Refer to Gibb & Partner January 1991 - Load Assessment Report – Additional Testing 1990 (NMWTRA Ref ER-ST-1) for further details on the derivation of this value.

Whilst Gibb & Partners calculated a reduced capacity for the hangers accounting for the brittle nature of the material and some deterioration due to corrosion, they clearly were not satisfied that this offered a long term, robust solution hence their recommendation to replace the hangers. A recommendation that is currently being implemented.

However, COWI also note the method (Barsom-Rolfe Formula) used by Gibb & Partners to adjust for the brittle behaviour may not fully account for the potential rate of loading in some instances. Gibb & Partners assessment considered the effect of a vehicle driving onto the bridge and the arising live load being applied to the socket over a period of 4.6 seconds (as informed by their assessment of the influence surface). In the case of a hanger failing in service, the force fluctuations in adjacent hangers would be expected to be of a considerably faster rate. This may mean that the capacity of the hangers, as stated by Gibb & Partners, is non-conservative for the ALS assessment.

Furthermore, it must be borne in mind that the condition, and thus load bearing capacity of the sockets is likely to deteriorate with time. Repeated loading of the hangers by passing traffic, may lead to crack initiation and propagation and thus to eventual brittle failure of the socket. This deterioration is exacerbated by casting defects and corrosion, both of which are known to affect the sockets.

Of particular concern is the ongoing corrosion of the socket lugs, see Figure 22. This raises three possible issues:

1. Corrosion leading to section loss and thus weakening of the socket

2. Corrosion creating microscopic crack initiators which concentrate stress and eventually lead to the sudden brittle fracture of the socket

3. Corrosion causing delamination (expansion of the socket lug material) which in turn imparts an additional bending force in the lug(s) and thus reduces the capacity of the socket

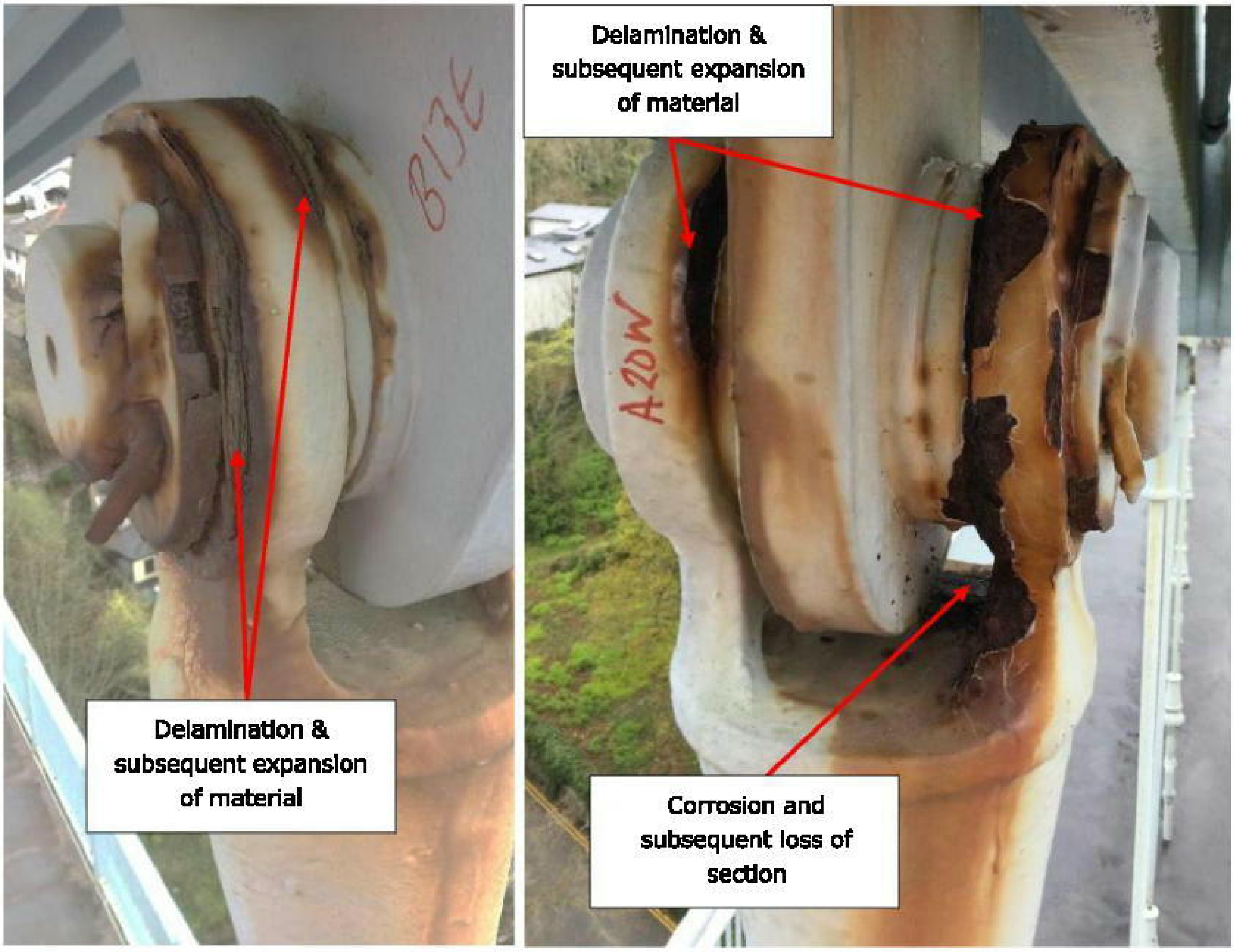


Figure 22: Typical location of delaminating material on sockets

Within the defined scope of this study, we have no means of quantifying the influence the above condition defects may / may not have in degrading the design tension resistance of the hanger load path. Accordingly, no adjustment to the design tension resistances quoted above has been applied. However, the implications of different levels of degradation and other (faster) rates of loading are discussed qualitatively when interpreting assessment results (e.g. utilisation ratios) later in this report.

6.1.2 1988-91 replacement spiral strand hangers

The 1988-91 replacement hangers have an improved end socket design and are fabricated in accordance with modern standards, as listed in the TAF. The design tension resistance has been obtained from the Minimum Breaking Load of 114t, in accordance with Cl. 6.2 of BS EN 1993-1-11:2006 (Eurocode for design of tension components). According to the UK NA, $\gamma_R = 1.00$ should be used, hence design tension resistance = (114 * 9.81) / (1.5 * 1.00) = 746kN.

However, since these replacement hangers connect to the same anchorage rods as the original 1938-41 hangers, which have an assessed design tension resistance of 430kN, the latter limits the resistance, such that the design tension resistance of the replacement spiral strand load path is taken to be **430kN.**

6.1.3 1938-41 solid bar hangers

These are square cross-sections, conservatively assumed to be in mild steel with f_y = 228N/mm^2 (there is some uncertainty from the reference drawings). Given the 2 1/4" cross-section shank dimension, and assuming a BSW thread, the design tension resistance is calculated to be **430kN.**

6.2 Potential causes of hanger failure

Before proceeding to assessment of hangers, it is worth considering the circumstances in which one of the vulnerable 1938-41 spiral strand hangers could fail.

In general terms there are 2 sets of circumstances:

1 Brittle failure of the socket **due to overloading,** where the tension resistance of the hanger is insufficient to sustain the applied tension force arising from the application of permanent loading in conjunction with traffic loading and/or temperature-induced redistribution of forces

1.1 Failure could be caused by the occurrence of an exceptional unprecedented loading event

1.2 Failure could occur due to the progressive reduction of tension resistance, e.g., as attributable to degradation and condition defects

2 Brittle failure at the socket **due to other circumstances**

2.1 Failure initiated by condition defects (as outlined above)

2.2 Failure initiated by road traffic incident

2.3 Direct abnormal loading on hanger sockets, e.g., due to debris impact as arising from an unsecured load

Failure due to overloading (1) is considered through the Ultimate Limit State (ULS) assessment in Section 7. Failure due to other circumstances (2) is considered through the Accidental Limit State (ALS) assessment in Section 8.

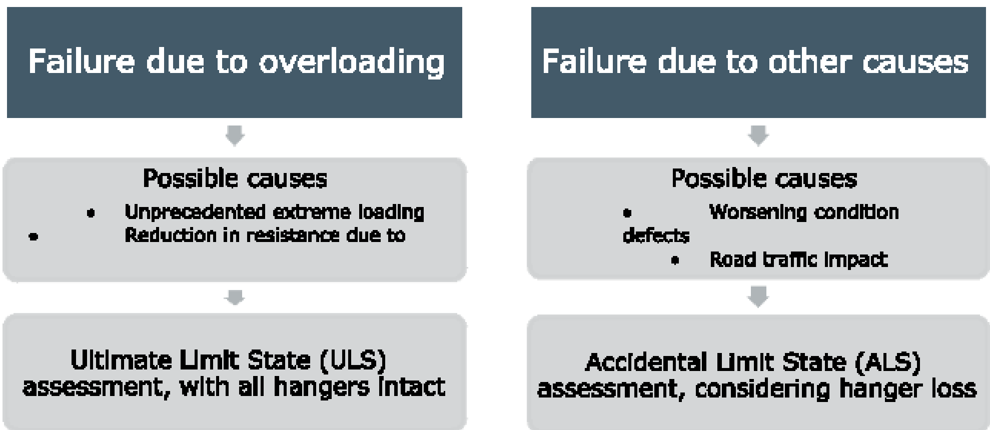


Figure 23: Distinct causes of brittle hanger failure

7 ULS Assessment - No Hanger Loss

Before reviewing the Accidental Limit State (ALS) analyses that consider scenarios involving the loss of individual hangers, it is informative to conduct a baseline ULS assessment with all hangers intact. This assessment considers loading model ALL Model 1 - Combination 1, whereby ULS applied tensions due to permanent, traffic and thermal actions are compared against ULS design tension resistances.

7.1 ULS Results

Having calculated the maximum tension forces in each hanger at ULS, these demands have been divided by the design strengths, assigned according to hanger type (refer Section 6.1), to calculate utilisation ratios.

According to this definition, a utilisation ratio of 1.00 implies that the ULS demand on a given hanger equals its ULS design strength, such that the hanger is operating at its ULS safety limit, where the assessed strength equals assessed applied loading.

In summary, all hangers have been assessed to be adequate at ULS, for all levels of traffic restrictions, including 'Normal' (i.e. unrestricted) traffic. However, it should be noted that this assessment makes no adjustment for the condition of the hangers.

> Figure 24 presents the maximum ULS tension resistance utilisation ratios for hangers grouped according to their type and the level of traffic restrictions.

> Figure 25 presents the ULS axial force in each individual hanger.

> Figure 26 presents ULS tension resistance utilisation ratios for each individual hanger.

The implication of this is that despite having low strength owing to brittle fracture failure mode, the 193841 spiral strand hangers possess sufficient strength to resist the demands placed upon them in normal operating circumstances, providing all hangers are (1) intact and (2) in good condition and thus (3) able to share and distribute the applied loading (both permanent and variable).

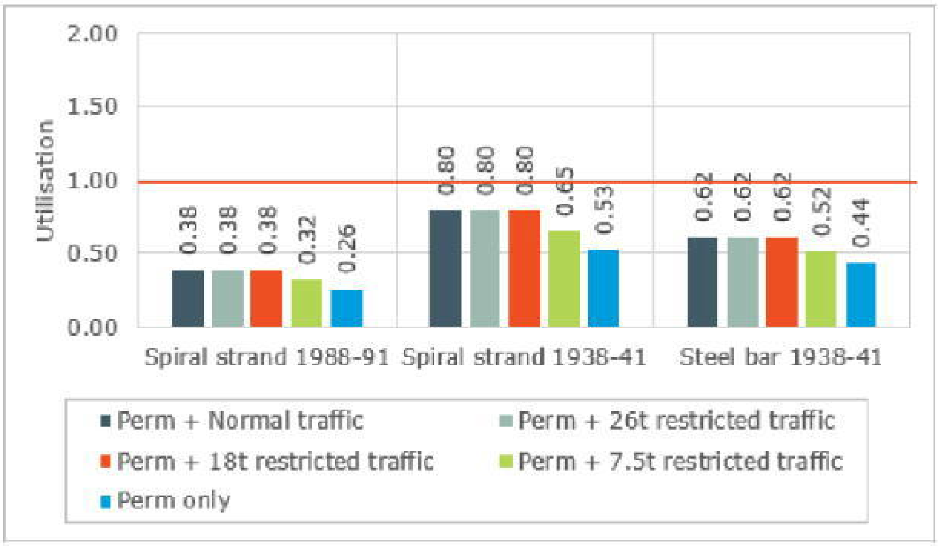


Figure 24: Maximum ULS tension resistance utilisation ratios for hangers of different types; all hangers intact (no hanger loss)

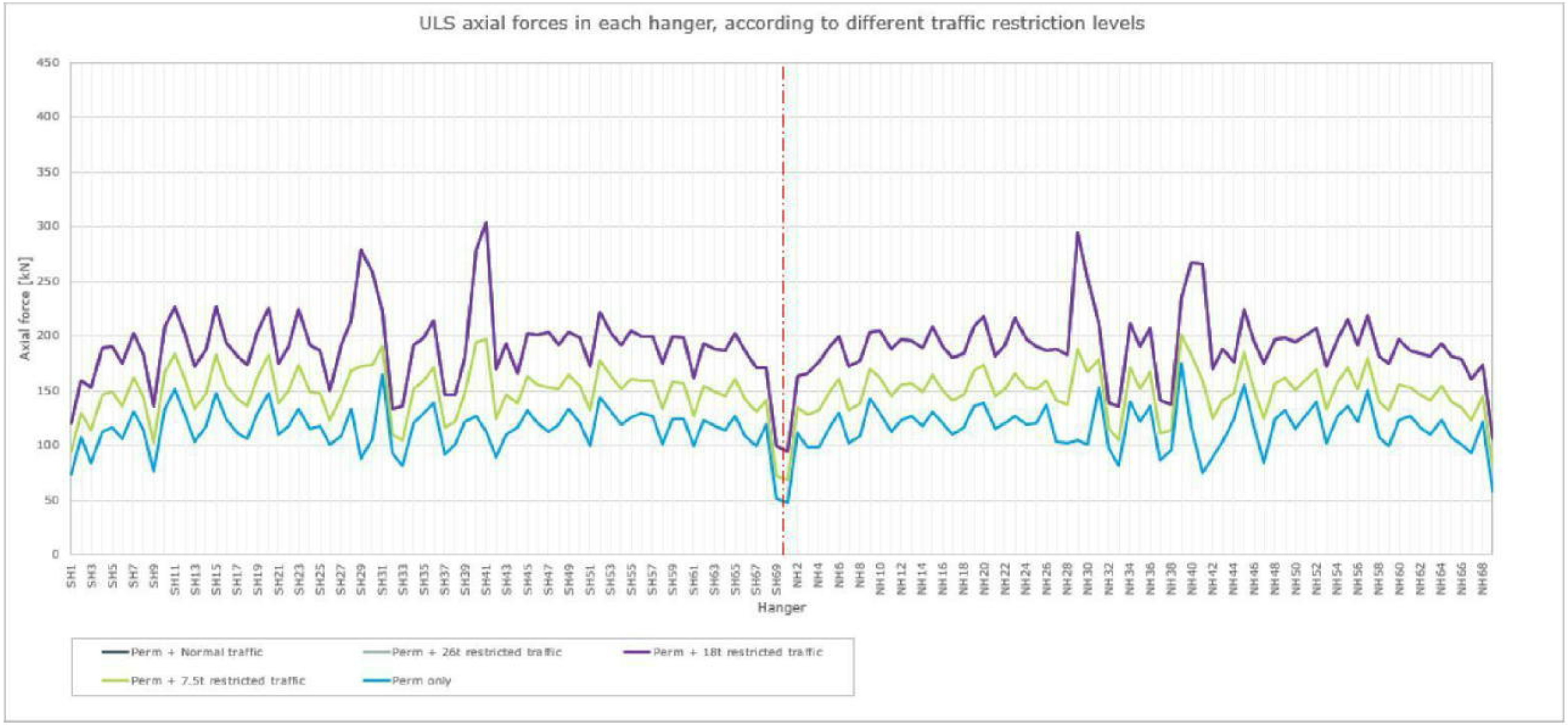


Figure 25: ULS axial force demands in all main span hangers; all hangers intact (no hanger loss).

Curves for 26 and 18t traffic sit behind the purple 'Normal' traffic curve due to similarity of applied K-factors; refer discussion in Section 4.3.4

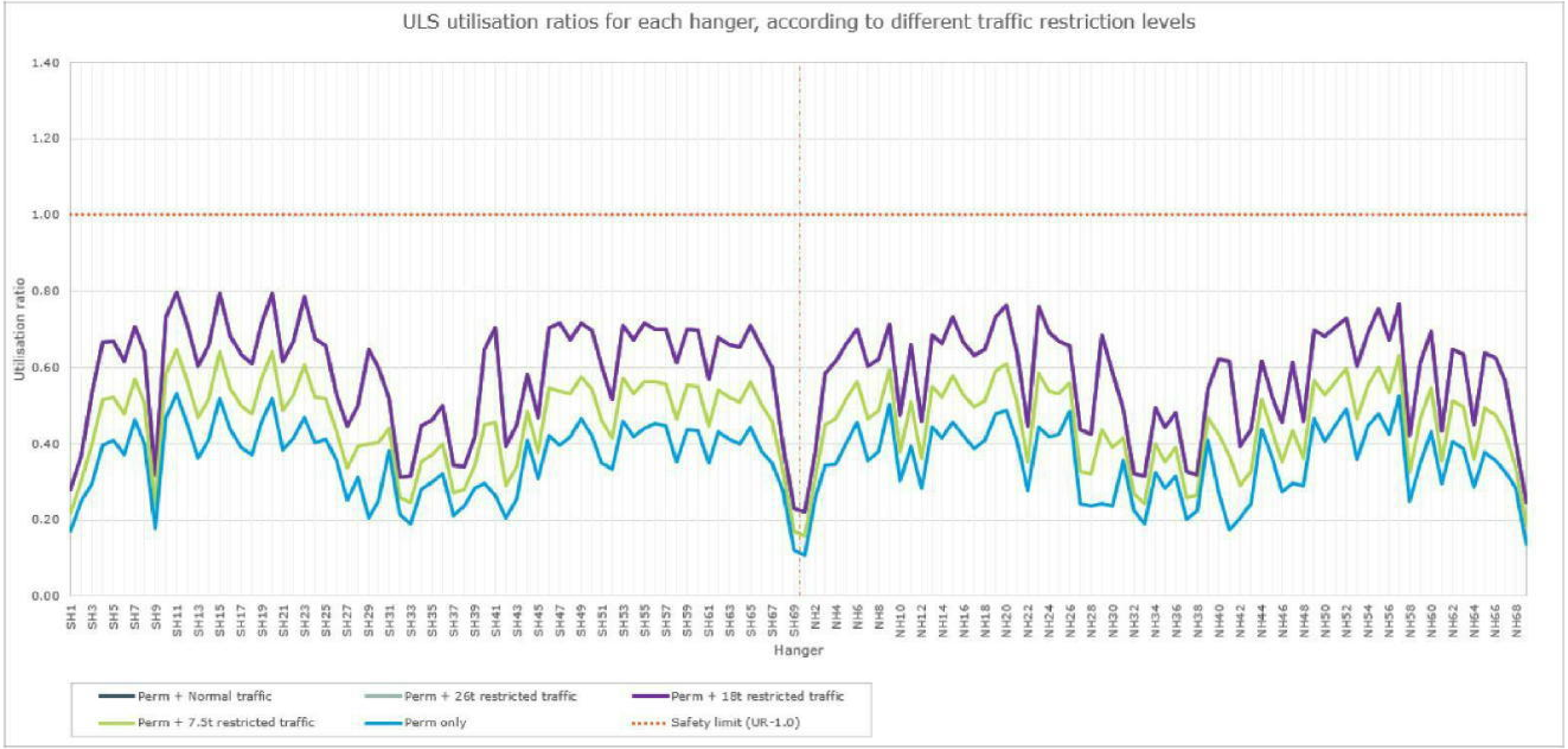


Figure 26: *ULS tension resistance utilisation ratios for in all main span hangers; all hangers intact (no hanger loss)*

Curves for 26 and 18t traffic sit behind the purple 'Normal' traffic curve due to similarity of applied K-factors; refer discussion in Section 4.3.4

8 ALS Assessment – Hanger Loss

8.1 Methodology

An assessment of the main span bridge hangers for the sudden loss of any one main span hanger has been carried out, by applying Cl.2.3.6(2) of BS EN 1993-1-11:2006, incorporating guidance from the UK National Annex.

Practically this involves the following steps:

1 For the chosen hanger *j* being dynamically ruptured, determine the following:

- ALS axial tension being carried under the combined application of unfactored permanent plus SLS-factored ALL1 traffic loading, of chosen traffic restriction level; term this E_{dj1}

  Regarding partial factor on leading variable actions, according to the UK NA to BS EN 1990, γ=1.0 is to be applied. In Eurocodes use of γ=1.0 corresponds to SLS. However, in CS 454 γ=1.20 is to be applied to traffic loading at SLS. CS454 provides no guidance regarding ALS scenarios hence we have chosen to apply SLS partial factor consistent with the intent in Eurocodes. This matches the guidance given in CS 454 cl.5.5.2 Note 5 which provides some background to, and comparison of, the load models and partial factors given in the Eurocodes and in CS 454.

- The live load pattern giving rise to the maximum tension in the hanger being considered

- 'Coincident' ALS-factored tension loads in all hangers (including that being ruptured), including loads arising from permanent loading, when this live load pattern is being applied; term this E_{dk1} for hanger k

- Redistribution coefficients, defining how load from the hanger *j* being ruptured statically redistributes to each other hanger *k*; refer Section 8.2; term these $k_{redist,jk}$

2 ALS applied tension forces $_{E2}$ in all other hangers *k*, accounting for the sudden loss of load from hanger *j*, are evaluated as:

$$E_2 = E_1 + k.\ E.\ k_{,k}$$

where $k_{dynamic=1.5}$ is the dynamic amplification factor given in Cl.2.3.6(2) of EN1993-1-11.

COWI note that the dynamic amplification factor could range from 1.0 to 2.0, therefore 1.5 whilst a reasonable (and codified) estimate could in some cases be non-conservative.

3 Repeat from step 1 for all other hangers

A visualisation of this process, for one of the 138no single hanger loss scenarios considered, is provided in Figure 27.

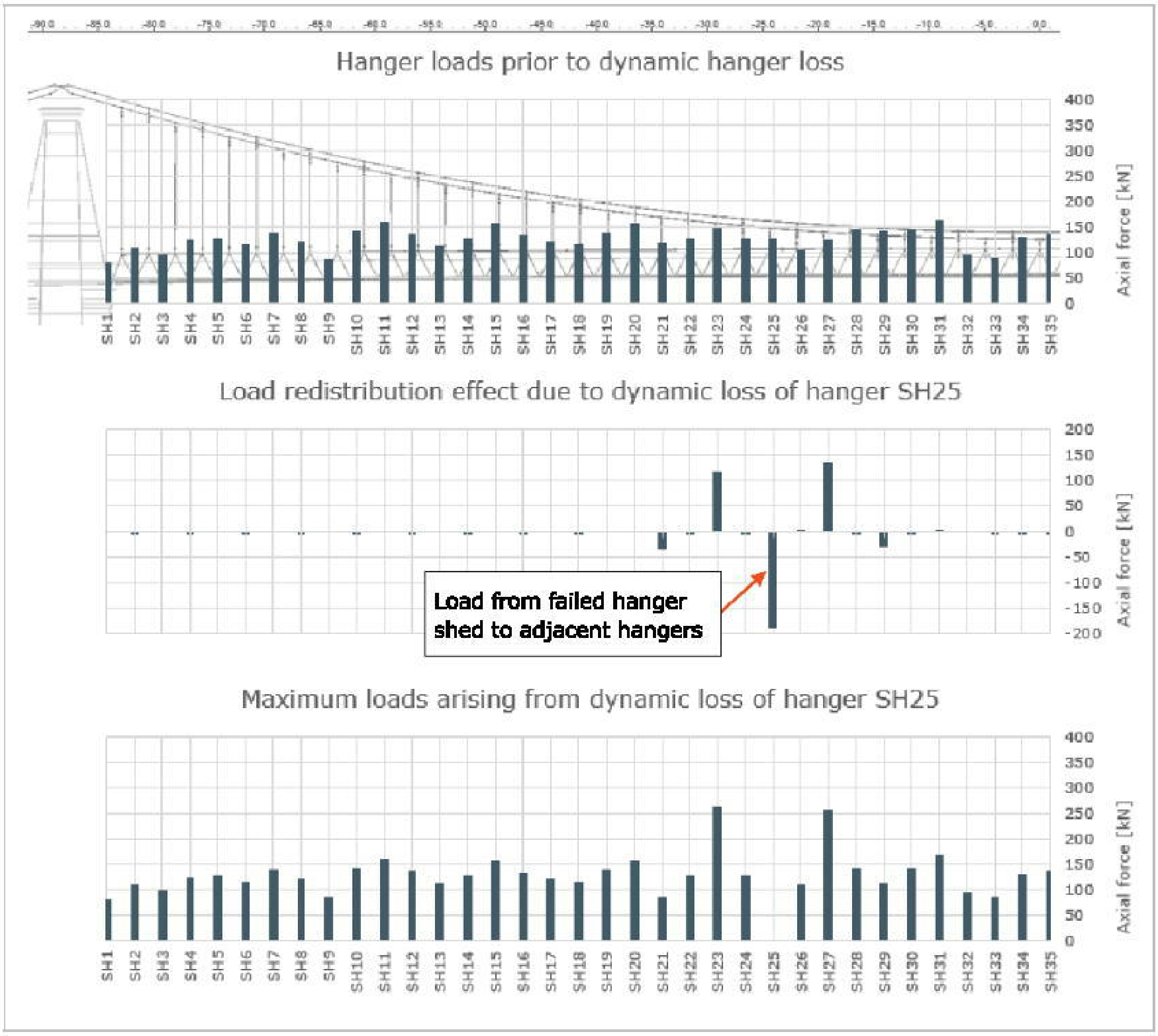


Figure 27: Visualisation for procedure for assessment of dynamic hanger loss

8.2 Hanger loss redistribution coefficients

To determine where load is shed in the event of a hanger failing, a set of linear analyses have been run to obtain redistribution coefficients for each hanger. In each analysis a single hanger is subject to 10°C temperature change. Temperature change has been used since this does not impose external loading on the model and can hence be used to explore the self-stress subspaces of the structural response. Extracting the hanger forces for each loadcase and normalising according to the hanger force in the hanger being relaxed via the temperature change, a matrix of redistribution coefficients is obtained; refer Figure 28 for a partial view of this matrix.

	Relaxed hanger								
Hanger	MS-1SW	MS-3SW	MS-5SW	MS-7SW	MS-9SW	MS-11SW	MS-13SW	MS-15SW	MS-17SW
MS-1SW	100%	-1%	-60%	0%	1%	0%	5%	0%	1%
MS-3SW	0%	100%	0%	-57%	0%	4%	0%	5%	0%
MS-5SW	-60%	0%	100%	-1%	-55%	0%	4%	0%	5%
MS-7SW	0%	-63%	-1%	100%	-1%	-55%	0%	4%	0%
MS-9SW	1%	0%	-61%	-1%	100%	-1%	-56%	0%	5%
MS-11SW	0%	7%	0%	-60%	-1%	100%	-1%	-57%	0%
MS-13SW	6%	-1%	5%	0%	-61%	-1%	100%	-1%	-57%
MS-15SW	0%	9%	0%	5%	0%	-62%	-1%	100%	-1%
MS-17SW	1%	0%	6%	0%	6%	0%	-62%	-1%	100%
MS-19SW	0%	1%	0%	6%	-1%	7%	0%	-63%	-1%
MS-21SW	0%	0%	1%	-1%	6%	-1%	8%	0%	-64%
MS-23SW	0%	0%	0%	1%	0%	6%	-1%	8%	0%
MS-25SW	0%	0%	0%	0%	0%	0%	5%	-1%	10%
MS-27SW	0%	1%	0%	0%	0%	0%	0%	5%	0%
MS-29SW	0%	0%	0%	0%	0%	0%	0%	0%	5%
MS-31SW	0%	1%	0%	0%	0%	0%	0%	0%	0%
MS-33SW	0%	0%	0%	0%	0%	0%	0%	0%	0%
MS-35SW	0%	1%	0%	0%	0%	0%	0%	0%	0%
MS-37SW	0%	0%	0%	0%	0%	0%	0%	0%	0%
MS-39SW	0%	1%	0%	1%	0%	0%	0%	0%	0%
MS-41SW	0%	0%	0%	0%	0%	0%	0%	0%	0%
MS-43SW	0%	1%	0%	0%	0%	0%	0%	0%	0%
MS-45SW	1%	-1%	1%	-1%	1%	-1%	1%	0%	1%
MS-47SW	0%	1%	0%	1%	0%	1%	0%	0%	0%
MS-49SW	0%	0%	0%	0%	-1%	0%	0%	0%	0%
MS-51SW	0%	-1%	0%	-1%	0%	-1%	0%	0%	0%

Figure 28: Hanger force redistribution coefficients as obtained from linear temperature change load cases

This matrix shows how load carried by a given hanger is redistributed to other hangers. Note all diagonal entries are 100% by definition, since these relate to the change in force in the hanger being relaxed-off. The load from a given hanger is generally picked up by a change of load, of opposite sign, in the adjacent hangers connecting to the same main chain; these are physically offset 2No. hanger positions along the bridge.

Totals down each column in general sum to zero, since load redistributed from one hanger is generally picked up entirely by other hangers. The exceptions to this are when hangers close to the tower piers are relaxed-off; in this situation, around 50% the load removed from a given hanger (e.g. MS-1SW) is picked up instead by the vertical support at the pier.

These redistribution coefficients are then used in the hanger loss superposition analysis (refer to Sections 8.3 and 8.4), since these coefficients define how the loads in all other hangers change when a given hanger is removed.

8.3 ALS Results - Dynamic loss of any one hanger

Having calculated hanger loss redistribution coefficients, it is now possible to review the effect of a hanger failing in service on the adjacent hangers using the method outlined in Section 8.1.

For a given level of traffic restrictions, a redistributed set of ALS applied tension forces in all hangers for each hanger loss case has been calculated. Figure 30 presents the results from this analysis for the case of 7.5t restricted traffic loading.

These results have been aggregated to determine the maximum ALS applied tension force and ALS tension resistance utilisation ratios in any other hanger, as different hangers are dynamically lost per this method (refer Figure 31 and Figure 32).

These results have been further aggregated to give the maximum utilisation ratios according to loss of a given hanger type (refer Figure 29) and counts of the number of hanger loss scenarios that result in excessive utilisation ratios in the remaining hangers (refer Table 10).

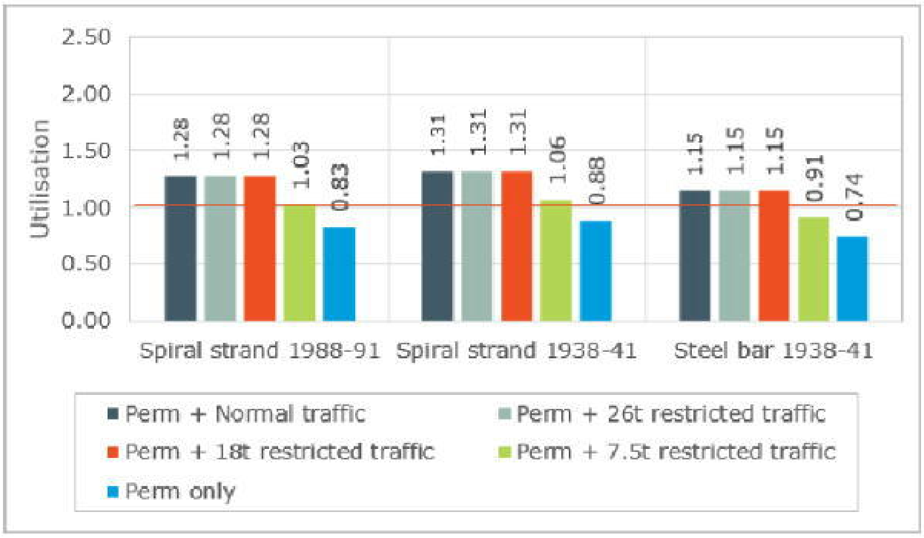


Figure 29: Maximum ALS tension resistance utilisation ratios for hangers of different types, considering all single hanger loss scenarios

		Number of scenarios resulting in subsequent overstressing of one or more remaining hangers				
Hanger type being lost	Total number of scenarios	Normal	26t	18t	7.5t	Permanent load only
Spiral strand 1988-91	20	16	16	16	1	0
Spiral strand 1938-41	84	78	78	78	8	0
Steel bar 1938-41	34	10	10	10	0	0

Table 10: Count of hanger loss scenarios that result in at least one remaining hanger being or becoming overstressed

Redistributed ALS hanger forces : Perm + 7.5t restricted traffic

Removed hanger	SH1	SH2	SH3	SH4	SH5	SH6	SH7	SH8	SH9	SH10
SH1	0.00	0.26	0.53	0.44	0.44	0.41	0.46	0.43	0.20	0.50
SH2	0.19	0.00	0.34	0.82	0.45	0.37	0.49	0.38	0.20	0.50
SH3	0.09	0.28	0.00	0.44	0.70	0.41	0.47	0.44	0.18	0.50
SH4	0.10	0.51	0.34	0.00	0.45	0.81	0.49	0.43	0.20	0.46
SH5	0.18	0.26	0.71	0.44	0.00	0.41	0.90	0.43	0.18	0.51
SH6	0.19	0.24	0.34	0.78	0.45	0.00	0.49	0.89	0.20	0.46
SH7	0.16	0.26	0.31	0.44	0.96	0.41	0.00	0.44	0.51	0.50
SH8	0.19	0.24	0.34	0.41	0.45	0.78	0.49	0.00	0.20	0.91
SH9	0.18	0.26	0.32	0.44	0.42	0.41	0.75	0.44	0.00	0.50
SH10	0.19	0.26	0.34	0.40	0.45	0.37	0.49	0.87	0.20	0.00
SH11	0.18	0.26	0.34	0.44	0.41	0.41	0.43	0.43	0.53	0.51
SH12	0.19	0.26	0.34	0.44	0.45	0.38	0.49	0.31	0.20	0.93
SH13	0.18	0.26	0.34	0.44	0.45	0.41	0.46	0.44	0.37	0.50
SH14	0.19	0.26	0.34	0.44	0.45	0.41	0.49	0.41	0.20	0.45
SH15	0.18	0.26	0.34	0.44	0.45	0.41	0.40	0.44	0.18	0.51
SH16	0.19	0.26	0.34	0.44	0.45	0.41	0.49	0.43	0.20	0.48
SH17	0.18	0.28	0.34	0.44	0.45	0.41	0.49	0.43	0.20	0.50
SH18	0.19	0.26	0.34	0.44	0.45	0.41	0.49	0.43	0.20	0.50
SH19	0.18	0.26	0.34	0.44	0.45	0.41	0.49	0.44	0.20	0.50
SH20	0.19	0.26	0.34	0.44	0.45	0.41	0.49	0.43	0.20	0.50
SH21	0.18	0.26	0.34	0.44	0.45	0.41	0.49	0.43	0.20	0.50
SH22	0.19	0.26	0.34	0.44	0.45	0.41	0.49	0.43	0.20	0.50
SH23	0.18	0.28	0.34	0.44	0.44	0.41	0.49	0.44	0.20	0.50
SH24	0.19	0.28	0.34	0.44	0.45	0.41	0.49	0.43	0.20	0.50
SH25	0.19	0.26	0.34	0.44	0.45	0.41	0.49	0.43	0.20	0.50
SH26	0.18	0.28	0.34	0.44	0.45	0.41	0.49	0.43	0.20	0.50
SH27	0.18	0.26	0.34	0.44	0.45	0.41	0.49	0.43	0.20	0.50
SH28	0.19	0.26	0.34	0.44	0.45	0.41	0.49	0.43	0.20	0.50
SH29	0.18	0.26	0.34	0.44	0.44	0.41	0.49	0.44	0.20	0.50
SH30	0.18	0.26	0.34	0.44	0.45	0.41	0.49	0.43	0.20	0.50
SH31	0.18	0.26	0.34	0.44	0.45	0.41	0.49	0.43	0.20	0.50
SH32	0.19	0.26	0.34	0.44	0.45	0.41	0.49	0.43	0.20	0.50
SH33	0.18	0.26	0.34	0.44	0.45	0.41	0.49	0.43	0.20	0.50
SH34	0.19	0.26	0.34	0.44	0.45	0.41	0.49	0.43	0.20	0.50
SH35	0.18	0.26	0.34	0.44	0.45	0.41	0.49	0.43	0.20	0.50
SH36	0.19	0.28	0.34	0.44	0.45	0.41	0.49	0.43	0.20	0.50
SH37	0.19	0.26	0.34	0.44	0.45	0.41	0.49	0.43	0.20	0.50
SH38	0.18	0.26	0.34	0.44	0.45	0.41	0.49	0.43	0.20	0.50
SH39	0.18	0.26	0.34	0.44	0.45	0.41	0.49	0.43	0.20	0.50
SH40	0.19	0.26	0.34	0.44	0.45	0.41	0.49	0.44	0.20	0.50
SH41	0.19	0.26	0.34	0.44	0.45	0.41	0.49	0.43	0.20	0.50
SH42	0.19	0.26	0.34	0.44	0.45	0.41	0.49	0.43	0.20	0.50
SH43	0.19	0.26	0.34	0.44	0.45	0.41	0.49	0.43	0.20	0.50
SH44	0.19	0.26	0.34	0.44	0.45	0.41	0.49	0.43	0.20	0.50
SH45	0.18	0.26	0.34	0.44	0.44	0.41	0.49	0.43	0.20	0.50
SH46	0.19	0.26	0.34	0.44	0.45	0.41	0.49	0.43	0.20	0.50
SH47	0.18	0.26	0.34	0.44	0.45	0.41	0.49	0.43	0.20	0.50
SH48	0.19	0.26	0.34	0.44	0.45	0.41	0.49	0.43	0.20	0.50
SH49	0.18	0.26	0.34	0.44	0.45	0.41	0.49	0.43	0.20	0.50
SH50	0.19	0.26	0.34	0.44	0.45	0.41	0.49	0.43	0.20	0.50
SH51	0.18	0.26	0.34	0.44	0.45	0.41	0.49	0.43	0.20	0.50
SH52	0.19	0.26	0.34	0.44	0.45	0.41	0.49	0.43	0.20	0.50
SH53	0.18	0.26	0.34	0.44	0.45	0.41	0.49	0.44	0.20	0.50
SH54	0.19	0.26	0.34	0.44	0.45	0.41	0.49	0.43	0.20	0.50

Removed hanger	SH11	SH12	SH13	SH14	SH15	SH16	SH17	SH18	SH19	SH20	SH21	SH22	SH23	SH24	SH25	SH26
SH1	0.56	0.43	0.46	0.45	0.55	0.47	0.43	0.41	0.49	0.55	0.41	0.45	0.52	0.45	0.45	0.37
SH2	0.56	0.43	0.46	0.44	0.55	0.46	0.43	0.40	0.49	0.55	0.42	0.45	0.52	0.44	0.44	0.37
SH3	0.55	0.43	0.46	0.45	0.55	0.47	0.42	0.41	0.49	0.55	0.41	0.45	0.52	0.45	0.45	0.37
SH4	0.56	0.43	0.46	0.45	0.55	0.46	0.43	0.40	0.49	0.55	0.42	0.45	0.52	0.44	0.44	0.37
SH5	0.52	0.43	0.46	0.45	0.55	0.47	0.42	0.41	0.49	0.55	0.41	0.45	0.52	0.45	0.45	0.37
SH6	0.56	0.45	0.46	0.44	0.55	0.47	0.43	0.40	0.49	0.55	0.42	0.45	0.52	0.44	0.44	0.37
SH7	0.50	0.43	0.36	0.45	0.55	0.47	0.43	0.41	0.49	0.55	0.41	0.45	0.52	0.45	0.45	0.37
SH8	0.56	0.43	0.46	0.41	0.55	0.47	0.43	0.41	0.49	0.55	0.42	0.45	0.52	0.44	0.44	0.37
SH9	0.85	0.43	0.36	0.45	0.53	0.47	0.43	0.41	0.49	0.55	0.41	0.45	0.52	0.45	0.45	0.37
SH10	0.56	0.87	0.46	0.37	0.56	0.42	0.43	0.41	0.49	0.55	0.42	0.45	0.52	0.44	0.44	0.37
SH11	0.00	0.43	0.94	0.44	0.45	0.47	0.35	0.41	0.49	0.55	0.41	0.45	0.52	0.45	0.45	0.37
SH12	0.56	0.00	0.46	0.92	0.55	0.38	0.43	0.38	0.49	0.55	0.42	0.45	0.52	0.44	0.44	0.37
SH13	0.96	0.43	0.00	0.45	0.95	0.47	0.34	0.41	0.47	0.55	0.42	0.45	0.52	0.45	0.45	0.37
SH14	0.56	0.88	0.46	0.00	0.56	0.92	0.42	0.31	0.49	0.53	0.42	0.45	0.52	0.44	0.44	0.37
SH15	0.40	0.43	0.90	0.45	0.00	0.47	0.90	0.40	0.36	0.56	0.40	0.45	0.52	0.45	0.45	0.37
SH16	0.56	0.41	0.46	0.87	0.55	0.00	0.43	0.89	0.49	0.43	0.42	0.44	0.52	0.45	0.44	0.37
SH17	0.54	0.43	0.33	0.44	0.94	0.47	0.00	0.41	0.94	0.55	0.29	0.46	0.52	0.45	0.45	0.37
SH18	0.56	0.47	0.46	0.37	0.55	0.95	0.43	0.00	0.49	0.90	0.41	0.33	0.52	0.44	0.44	0.36
SH19	0.56	0.43	0.36	0.45	0.45	0.46	0.93	0.41	0.00	0.56	0.94	0.45	0.36	0.45	0.45	0.37
SH20	0.56	0.43	0.46	0.40	0.56	0.35	0.42	0.93	0.49	0.00	0.42	1.04	0.52	0.37	0.45	0.38
SH21	0.56	0.43	0.46	0.45	0.54	0.47	0.33	0.41	0.99	0.55	0.00	0.45	0.97	0.44	0.30	0.37
SH22	0.56	0.43	0.46	0.45	0.55	0.46	0.43	0.30	0.49	0.99	0.42	0.00	0.53	0.92	0.44	0.29
SH23	0.56	0.43	0.46	0.45	0.55	0.47	0.42	0.41	0.35	0.55	0.92	0.46	0.00	0.45	1.04	0.38
SH24	0.56	0.43	0.46	0.44	0.55	0.47	0.43	0.40	0.49	0.43	0.41	0.85	0.53	0.00	0.44	0.90
SH25	0.56	0.43	0.46	0.45	0.55	0.47	0.43	0.41	0.49	0.55	0.30	0.45	0.93	0.45	0.00	0.33
SH26	0.56	0.43	0.46	0.45	0.55	0.47	0.43	0.41	0.49	0.56	0.43	0.36	0.52	0.79	0.46	0.00
SH27	0.56	0.43	0.46	0.45	0.55	0.47	0.43	0.41	0.49	0.55	0.43	0.46	0.38	0.44	0.90	0.37
SH28	0.56	0.43	0.46	0.44	0.55	0.47	0.43	0.41	0.49	0.55	0.42	0.47	0.53	0.28	0.44	0.90
SH29	0.56	0.43	0.46	0.45	0.55	0.47	0.42	0.41	0.49	0.55	0.41	0.45	0.53	0.45	0.35	0.35
SH30	0.56	0.43	0.46	0.44	0.55	0.46	0.43	0.40	0.49	0.55	0.42	0.45	0.53	0.46	0.43	0.29
SH31	0.56	0.43	0.46	0.45	0.55	0.47	0.43	0.41	0.49	0.55	0.42	0.45	0.52	0.45	0.47	0.38
SH32	0.56	0.43	0.46	0.45	0.55	0.47	0.43	0.41	0.49	0.55	0.42	0.45	0.52	0.45	0.45	0.38
SH33	0.56	0.43	0.46	0.45	0.55	0.47	0.43	0.41	0.49	0.55	0.42	0.45	0.52	0.45	0.44	0.37
SH34	0.56	0.43	0.46	0.44	0.55	0.47	0.43	0.41	0.49	0.55	0.42	0.45	0.52	0.45	0.44	0.38
SH35	0.56	0.43	0.46	0.45	0.55	0.47	0.43	0.41	0.49	0.55	0.41	0.45	0.52	0.45	0.44	0.37
SH36	0.56	0.43	0.46	0.44	0.55	0.47	0.43	0.41	0.49	0.55	0.42	0.45	0.52	0.45	0.45	0.37
SH37	0.56	0.43	0.46	0.45	0.55	0.47	0.43	0.41	0.49	0.55	0.42	0.45	0.52	0.45	0.44	0.37
SH38	0.56	0.43	0.46	0.44	0.55	0.47	0.43	0.41	0.49	0.55	0.42	0.45	0.52	0.45	0.44	0.37
SH39	0.56	0.43	0.46	0.45	0.55	0.47	0.43	0.41	0.49	0.55	0.41	0.45	0.52	0.45	0.43	0.37
SH40	0.56	0.43	0.46	0.45	0.55	0.47	0.43	0.41	0.49	0.55	0.42	0.45	0.52	0.45	0.46	0.38
SH41	0.56	0.43	0.46	0.45	0.55	0.47	0.43	0.41	0.49	0.55	0.42	0.45	0.52	0.45	0.47	0.38
SH42	0.56	0.43	0.46	0.44	0.55	0.47	0.43	0.41	0.49	0.55	0.42	0.45	0.52	0.45	0.45	0.37
SH43	0.56	0.43	0.46	0.45	0.55	0.47	0.43	0.41	0.49	0.55	0.42	0.45	0.52	0.45	0.45	0.37
SH44	0.56	0.43	0.46	0.44	0.55	0.46	0.43	0.40	0.49	0.55	0.42	0.45	0.52	0.45	0.44	0.38
SH45	0.56	0.43	0.46	0.45	0.55	0.47	0.42	0.41	0.49	0.55	0.41	0.45	0.52	0.45	0.42	0.36
SH46	0.56	0.43	0.46	0.44	0.55	0.47	0.43	0.41	0.49	0.55	0.42	0.45	0.52	0.45	0.45	0.37
SH47	0.56	0.43	0.46	0.45	0.55	0.47	0.43	0.41	0.49	0.55	0.42	0.45	0.52	0.45	0.45	0.37
SH48	0.56	0.43	0.46	0.44	0.55	0.47	0.43	0.41	0.49	0.55	0.42	0.45	0.52	0.45	0.45	0.37
SH49	0.56	0.43	0.46	0.45	0.55	0.47	0.43	0.41	0.49	0.55	0.41	0.45	0.52	0.45	0.44	0.37
SH50	0.56	0.43	0.46	0.44	0.55	0.47	0.43	0.41	0.49	0.55	0.42	0.45	0.52	0.45	0.45	0.37
SH51	0.56	0.43	0.46	0.45	0.55	0.47	0.43	0.41	0.49	0.55	0.41	0.45	0.52	0.45	0.44	0.37
SH52	0.56	0.43	0.46	0.44	0.55	0.47	0.43	0.41	0.49	0.55	0.42	0.45	0.52	0.45	0.45	0.37
SH53	0.56	0.43	0.46	0.45	0.55	0.47	0.43	0.41	0.49	0.55	0.41	0.45	0.52	0.45	0.44	0.37
SH54	0.56	0.43	0.46	0.44	0.55	0.47	0.43	0.41	0.49	0.55	0.42	0.45	0.52	0.45	0.45	0.37

Removed hanger	SH27	SH28	SH29	SH30	SH31	SH32	SH33	SH34	SH35	SH36	SH37	SH38	SH39	SH40
SH1	0.29	0.34	0.33	0.34	0.38	0.22	0.21	0.30	0.32	0.34	0.23	0.24	0.29	0.36
SH2	0.29	0.33	0.34	0.33	0.38	0.22	0.21	0.30	0.32	0.34	0.23	0.24	0.30	0.38
SH3	0.29	0.34	0.33	0.34	0.38	0.22	0.21	0.30	0.32	0.34	0.23	0.24	0.29	0.38
SH4	0.29	0.33	0.34	0.33	0.38	0.22	0.21	0.30	0.32	0.34	0.23	0.24	0.30	0.36
SH5	0.28	0.34	0.33	0.34	0.38	0.22	0.21	0.30	0.32	0.34	0.23	0.24	0.29	0.38
SH6	0.29	0.34	0.34	0.33	0.38	0.22	0.21	0.30	0.32	0.34	0.23	0.24	0.30	0.38
SH7	0.28	0.34	0.33	0.34	0.38	0.22	0.21	0.30	0.32	0.34	0.23	0.24	0.29	0.36
SH8	0.29	0.34	0.34	0.33	0.38	0.22	0.21	0.30	0.32	0.34	0.23	0.24	0.30	0.38
SH9	0.29	0.34	0.33	0.34	0.38	0.22	0.21	0.30	0.32	0.34	0.23	0.24	0.29	0.38
SH10	0.29	0.33	0.34	0.33	0.36	0.22	0.21	0.30	0.32	0.34	0.23	0.24	0.30	0.36
SH11	0.28	0.34	0.33	0.34	0.38	0.22	0.21	0.30	0.32	0.34	0.23	0.24	0.29	0.38
SH12	0.29	0.34	0.34	0.33	0.38	0.22	0.21	0.30	0.32	0.34	0.23	0.24	0.30	0.38
SH13	0.29	0.34	0.33	0.34	0.38	0.22	0.21	0.30	0.32	0.34	0.23	0.24	0.29	0.36
SH14	0.29	0.34	0.34	0.33	0.38	0.22	0.21	0.30	0.32	0.34	0.23	0.24	0.30	0.38
SH15	0.29	0.34	0.33	0.34	0.38	0.22	0.21	0.30	0.32	0.34	0.23	0.24	0.29	0.36
SH16	0.29	0.34	0.34	0.33	0.38	0.22	0.21	0.30	0.32	0.34	0.23	0.24	0.30	0.38
SH17	0.29	0.34	0.33	0.34	0.38	0.22	0.21	0.30	0.32	0.34	0.23	0.24	0.29	0.38
SH18	0.29	0.34	0.34	0.34	0.38	0.22	0.21	0.30	0.32	0.34	0.23	0.24	0.29	0.36
SH19	0.29	0.34	0.33	0.34	0.38	0.22	0.21	0.30	0.32	0.34	0.23	0.24	0.29	0.36
SH20	0.29	0.34	0.34	0.33	0.38	0.22	0.21	0.30	0.32	0.34	0.23	0.24	0.30	0.38
SH21	0.30	0.34	0.33	0.34	0.38	0.22	0.21	0.30	0.32	0.34	0.23	0.24	0.29	0.38
SH22	0.29	0.35	0.33	0.34	0.38	0.22	0.21	0.30	0.32	0.34	0.23	0.24	0.29	0.38
SH23	0.14	0.34	0.34	0.34	0.38	0.22	0.21	0.30	0.32	0.34	0.23	0.24	0.29	0.38
SH24	0.29	0.22	0.34	0.35	0.38	0.22	0.21	0.30	0.32	0.34	0.23	0.24	0.29	0.36
SH25	0.80	0.33	0.26	0.33	0.38	0.22	0.20	0.30	0.32	0.34	0.23	0.24	0.29	0.39
SH26	0.29	0.59	0.32	0.29	0.38	0.23	0.21	0.30	0.32	0.34	0.23	0.24	0.29	0.38
SH27	0.00	0.34	0.65	0.34	0.26	0.22	0.24	0.30	0.31	0.34	0.23	0.24	0.29	0.38
SH28	0.29	0.00	0.34	0.71	0.38	0.37	0.21	0.33	0.32	0.33	0.23	0.24	0.29	0.38
SH29	0.59	0.34	0.00	0.36	0.73	0.21	0.00	0.31	0.35	0.34	0.22	0.24	0.30	0.37
SH30	0.29	0.88	0.35	0.00	0.38	0.57	0.21	0.17	0.32	0.37	0.23	0.24	0.30	0.38
SH31	0.80	0.33	0.77	0.34	0.00	0.22	0.63	0.30	0.05	0.35	0.27	0.24	0.29	0.39
SH32	0.23	0.25	0.33	0.53	0.38	0.00	0.21	0.54	0.32	0.25	0.23	0.26	0.29	0.36
SH33	0.31	0.34	0.24	0.34	0.61	0.22	0.00	0.39	0.55	0.34	0.14	0.24	0.31	0.38
SH34	0.25	0.36	0.34	0.21	0.38	0.55	0.21	0.00	0.32	0.62	0.23	0.11	0.30	1.41
SH35	0.28	0.34	0.37	0.34	0.24	0.22	0.56	0.39	0.00	0.35	0.58	0.24	0.15	0.38
SH36	0.29	0.33	0.33	0.37	0.38	0.37	0.20	0.68	0.32	0.00	0.23	0.62	0.29	0.23
SH37	0.20	0.34	0.33	0.34	0.41	0.22	0.11	0.30	0.57	0.34	0.00	0.24	0.55	0.36
SH38	0.29	0.34	0.34	0.34	0.38	0.24	0.21	0.20	0.32	0.61	0.23	0.00	0.30	0.66
SH39	0.28	0.34	0.34	0.34	0.37	0.22	0.24	0.30	0.09	0.34	0.55	0.24	0.00	0.38
SH40	0.29	0.34	0.32	0.33	0.38	0.22	0.21	0.34	0.32	0.19	0.23	0.63	0.29	0.00
SH41	0.29	0.34	0.32	0.33	0.39	0.22	0.20	0.30	0.36	0.35	0.08	0.23	0.69	0.40
SH42	0.29	0.34	0.33	0.34	0.38	0.22	0.21	0.30	0.32	0.37	0.23	0.13	0.29	0.65
SH43	0.29	0.34	0.33	0.34	0.38	0.22	0.21	0.30	0.31	0.34	0.26	0.24	0.17	0.36
SH44	0.29	0.34	0.34	0.35	0.38	0.22	0.21	0.30	0.32	0.34	0.23	0.25	0.30	0.32
SH45	0.26	0.34	0.35	0.35	0.38	0.22	0.21	0.30	0.32	0.34	0.22	0.24	0.31	0.37
SH46	0.29	0.34	0.33	0.34	0.38	0.22	0.21	0.30	0.32	0.34	0.23	0.24	0.29	0.38
SH47	0.29	0.34	0.33	0.34	0.38	0.22	0.21	0.30	0.32	0.34	0.23	0.24	0.29	0.38
SH48	0.29	0.34	0.33	0.34	0.38	0.22	0.21	0.30	0.32	0.34	0.23	0.24	0.29	0.38
SH49	0.29	0.34	0.33	0.34	0.38	0.22	0.21	0.30	0.32	0.34	0.23	0.24	0.29	0.38
SH50	0.29	0.34	0.33	0.34	0.38	0.22	0.21	0.30	0.32	0.34	0.23	0.24	0.29	0.38
SH51	0.29	0.34	0.33	0.34	0.38	0.22	0.21	0.30	0.32	0.34	0.23	0.24	0.29	0.38
SH52	0.29	0.34	0.33	0.34	0.38	0.22	0.21	0.30	0.32	0.34	0.23	0.24	0.29	0.38
SH53	0.29	0.34	0.33	0.34	0.38	0.22	0.21	0.30	0.32	0.34	0.23	0.24	0.29	0.38
SH54	0.29	0.34	0.33	0.34	0.38	0.22	0.21	0.30	0.32	0.34	0.23	0.24	0.29	0.38

Removed hanger	SH41	SH42	SH43	SH44	SH45	SH46	SH47	SH48	SH49	SH50	SH51	SH52	SH53
SH1	0.39	0.25	0.29	0.42	0.32	0.47	0.46	0.45	0.49	0.86	0.39	0.36	0.49
SH2	0.39	0.24	0.29	0.41	0.33	0.47	0.46	0.45	0.50	0.86	0.40	0.35	0.49
SH3	0.39	0.25	0.29	0.42	0.32	0.47	0.46	0.48	0.49	0.87	0.39	0.36	0.49
SH4	0.39	0.24	0.29	0.41	0.33	0.47	0.46	0.45	0.50	0.86	0.40	0.35	0.49
SH5	0.39	0.25	0.29	0.42	0.32	0.47	0.46	0.45	0.49	0.87	0.39	0.36	0.49
SH6	0.39	0.24	0.29	0.41	0.33	0.47	0.46	0.45	0.50	0.86	0.40	0.35	0.49
SH7	0.39	0.25	0.29	0.42	0.32	0.47	0.46	0.45	0.49	0.87	0.39	0.36	0.49
SH8	0.39	0.24	0.29	0.41	0.33	0.47	0.46	0.45	0.50	0.86	0.40	0.35	0.49
SH9	0.39	0.25	0.29	0.42	0.32	0.47	0.46	0.48	0.49	0.87	0.39	0.36	0.49
SH10	0.39	0.24	0.29	0.41	0.33	0.47	0.46	0.45	0.50	0.86	0.40	0.35	0.49
SH11	0.39	0.25	0.29	0.42	0.32	0.47	0.46	0.45	0.49	0.87	0.39	0.36	0.49
SH12	0.39	0.24	0.29	0.41	0.33	0.47	0.46	0.45	0.50	0.86	0.40	0.35	0.49
SH13	0.39	0.25	0.29	0.42	0.32	0.47	0.46	0.45	0.49	0.87	0.39	0.36	0.49
SH14	0.39	0.24	0.29	0.41	0.33	0.47	0.46	0.45	0.50	0.86	0.40	0.35	0.49
SH15	0.39	0.25	0.29	0.42	0.32	0.47	0.46	0.45	0.49	0.87	0.39	0.36	0.49
SH16	0.39	0.24	0.29	0.41	0.33	0.47	0.46	0.45	0.50	0.86	0.40	0.35	0.49
SH17	0.39	0.25	0.29	0.42	0.32	0.47	0.46	0.48	0.49	0.87	0.39	0.36	0.49
SH18	0.38	0.24	0.29	0.41	0.33	0.47	0.46	0.45	0.50	0.86	0.40	0.36	0.49
SH19	0.39	0.25	0.29	0.42	0.32	0.47	0.46	0.45	0.49	0.87	0.39	0.36	0.49
SH20	0.39	0.24	0.29	0.41	0.33	0.47	0.46	0.45	0.50	0.86	0.40	0.35	0.49
SH21	0.39	0.25	0.29	0.42	0.32	0.47	0.46	0.45	0.49	0.87	0.39	0.36	0.49
SH22	0.38	0.24	0.29	0.41	0.33	0.47	0.46	0.45	0.50	0.86	0.40	0.36	0.49
SH23	0.38	0.25	0.29	0.42	0.33	0.47	0.46	0.45	0.49	0.87	0.39	0.36	0.49
SH24	0.38	0.24	0.29	0.42	0.32	0.47	0.46	0.45	0.50	0.86	0.40	0.36	0.49
SH25	0.40	0.25	0.29	0.41	0.31	0.47	0.46	0.48	0.49	0.87	0.39	0.36	0.49
SH26	0.39	0.24	0.29	0.41	0.32	0.47	0.46	0.45	0.50	0.86	0.40	0.35	0.49
SH27	0.38	0.24	0.29	0.42	0.33	0.47	0.46	0.45	0.49	0.87	0.39	0.36	0.49
SH28	0.38	0.24	0.29	0.41	0.33	0.47	0.46	0.45	0.50	0.86	0.40	0.36	0.49
SH29	0.37	0.24	0.29	0.43	0.34	0.47	0.46	0.45	0.50	0.86	0.40	0.36	0.49
SH30	0.38	0.24	0.29	0.43	0.33	0.47	0.46	0.46	0.50	0.87	0.39	0.36	0.49
SH31	0.39	0.25	0.29	0.41	0.32	0.47	0.46	0.48	0.49	0.87	0.39	0.36	0.49
SH32	0.39	0.25	0.29	0.41	0.32	0.47	0.46	0.45	0.50	0.86	0.40	0.36	0.49
SH33	0.38	0.25	0.29	0.42	0.32	0.47	0.46	0.45	0.49	0.86	0.39	0.38	0.49
SH34	0.38	0.24	0.29	0.42	0.33	0.47	0.46	0.45	0.50	0.86	0.40	0.36	0.49
SH35	0.42	0.24	0.28	0.42	0.32	0.47	0.46	0.46	0.49	0.87	0.39	0.36	0.49
SH36	0.39	0.28	0.29	0.41	0.32	0.47	0.46	0.45	0.50	0.86	0.40	0.36	0.49
SH37	0.20	0.25	0.31	0.42	0.32	0.47	0.46	0.46	0.49	0.86	0.39	0.36	0.49
SH38	0.30	0.06	0.29	0.43	0.30	0.47	0.46	0.45	0.50	0.86	0.39	0.36	0.49
SH39	0.72	0.24	0.17	0.42	0.34	0.47	0.46	0.46	0.49	0.86	0.39	0.36	0.49
SH40	0.40	0.68	0.29	0.31	0.32	0.48	0.46	0.45	0.50	0.86	0.40	0.35	0.49
SH41	0.00	0.25	0.64	0.39	0.25	0.48	0.47	0.46	0.49	0.87	0.39	0.36	0.49
SH42	0.39	0.00	0.29	0.80	0.32	0.35	0.46	0.46	0.50	0.86	0.40	0.36	0.49
SH43	0.70	0.25	0.00	0.42	0.83	0.46	0.31	0.46	0.51	0.86	0.39	0.36	0.49
SH44	0.37	0.53	0.29	0.00	0.34	0.95	0.45	0.35	0.50	0.47	0.39	0.36	0.49
SH45	0.31	0.24	0.64	0.44	0.00	0.47	0.91	0.45	0.37	0.87	0.40	0.36	0.49
SH46	0.39	0.12	0.29	0.95	0.32	0.00	0.46	0.91	0.49	0.38	0.40	0.35	0.49
SH47	0.39	0.25	0.16	0.41	0.87	0.47	0.00	0.49	0.94	0.46	0.28	0.36	0.49
SH48	0.39	0.26	0.29	0.25	0.32	0.96	0.46	0.00	0.50	0.91	0.39	0.29	0.49
SH49	0.28	0.25	0.38	0.42	0.21	0.46	0.96	0.48	0.00	0.57	0.87	0.35	0.38
SH50	0.39	0.24	0.29	0.43	0.33	0.32	0.46	0.95	0.50	0.00	0.40	0.65	0.49
SH51	0.30	0.25	0.29	0.42	0.30	0.47	0.33	0.45	0.92	0.47	0.00	0.36	0.00
SH52	0.39	0.24	0.29	0.42	0.32	0.46	0.46	0.30	0.49	1.03	0.40	0.00	0.49
SH53	0.38	0.25	0.29	0.42	0.32	0.47	0.45	0.46	0.38	0.46	0.91	0.36	0.00
SH54	0.39	0.24	0.29	0.42	0.32	0.47	0.46	0.44	0.50	0.35	0.39	0.67	0.49

Figure 30: ***ULS tension resistance utilisations for all hangers (given across columns) in response to dynamic removal of single hangers (given by rows); 7.5 restricted traffic***

Cells are coloured smoothly according to their utilisation ratio, with all utilisation ratios in excess of unity coloured dark red. Removal of a given hanger by definition results in zero utilisation for that hanger (as given by diagonal zero values). Hangers that are of the weaker 1938-41 spiral strand type are indicated by purple text colouring of the row and columns labels and values; at can be seen that it is generally these hangers that have largest utilisation ratios with/without hanger loss

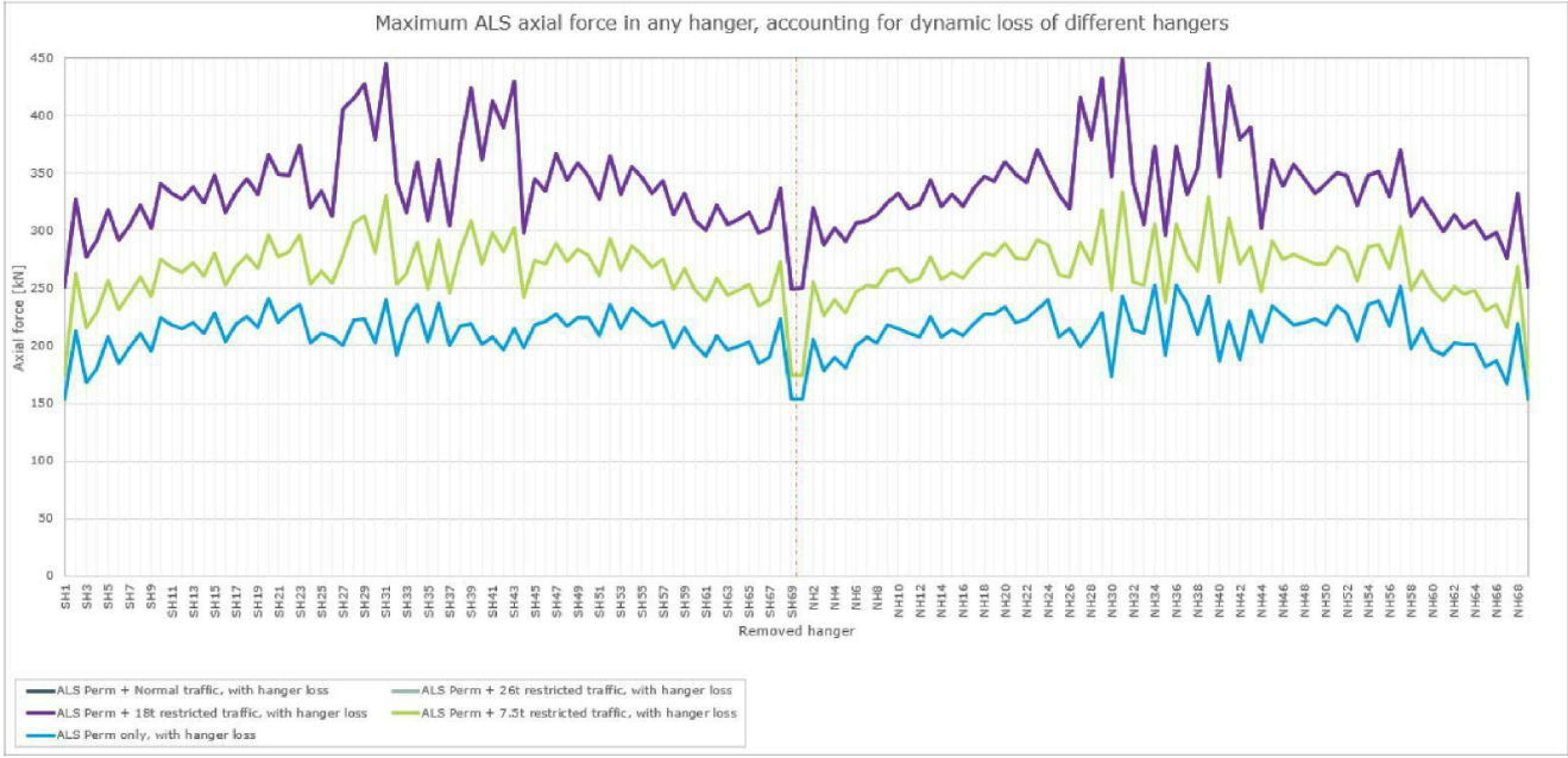


Figure 31: *Maximum ALS applied tension forces arising from dynamic loss of different hangers, for different levels of traffic restrictions*

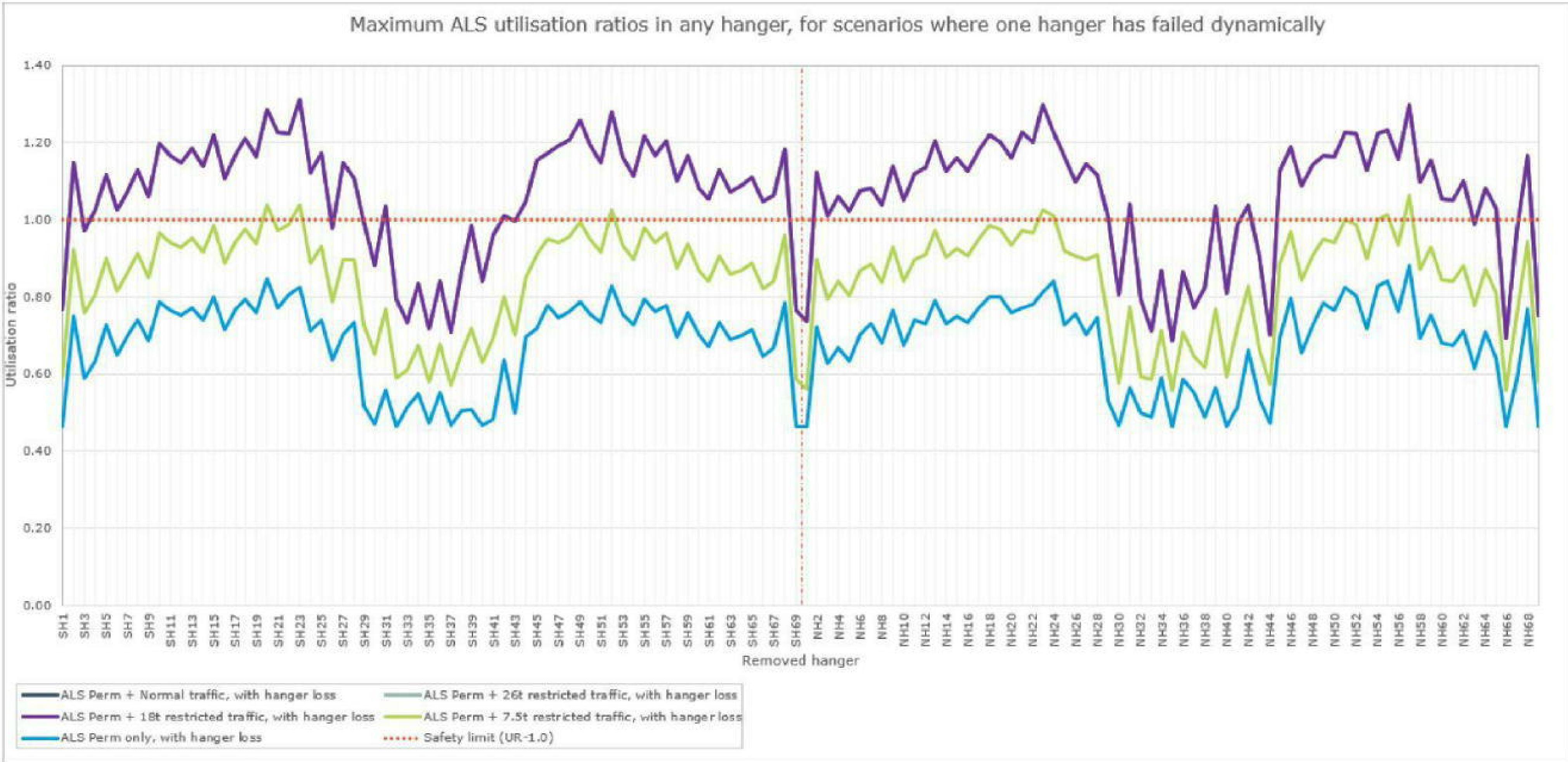


Figure 32: Maximum ALS tension resistance utilisation ratios accounting for dynamic hanger loss, for different levels of traffic restrictions

From the results of the above analyses, the following can be concluded regarding scenarios that consider the abrupt dynamic loss of any one hanger:

> Were loss of a hanger to occur under the action of permanent loads only, all other remaining hangers have utilisation ratios less than unity. This implies that knock-on failures in other adjacent hangers should not occur in these circumstances.

> Were loss of a hanger to occur under the action of normal, 26T or 18T traffic load (assessed to ALL Model 2) 104 out of the 138 hanger loss scenarios considered (72%) result in overstressing of adjacent hangers.

> Were loss of a hanger to occur under the action of 7.5T restricted traffic loading (assessed to ALL Model 2) 9 out of the 138 hanger loss scenarios considered (7%) result in overstressing of adjacent hangers.

Where overstressing of adjacent hangers implies a progressive collapse mechanism i.e., dynamic redistribution of load is predicted to lead to failure of one adjacent hanger, this in turn would lead to further dynamic redistribution of load, which could result in a wider disproportionate collapse / 'unzipping' failure.

Therefore, this analysis demonstrates that there is a credible risk of a disproportionate collapse of the bridge deck if specific hangers fail in service.

Whilst this risk has been reduced by implementing the 7.5T weight restriction, a credible risk remains.

It should be noted that the ULS assessment demonstrates that failure of a hanger by (quasi-static) overloading should not occur; it follows that failure must be initiated by another cause; refer Section 6.2.

Furthermore, when the variability of the hanger dead loads is considered, as quantified via Monte Carlo simulation (Section 4.2.3), the number of 'at risk' hangers increases from 9No to 32No.

When failure is predicted to occur in an adjacent hanger, in each hanger loss scenario, this is either because the hanger being lost dynamically in that scenario was carrying above-average load (which gets redistributed to adjacent hangers) or else because the adjacent hangers are already carrying above-average load, which gets added to by the load redistributed from the failing hanger.

It should be recognised when appraising these results that the current analysis approach implicitly considers elastic redistribution effects only; no account has been made of nonlinear material behaviour, e.g., plastic redistribution effect whereby yielding hangers maintain their failure load. This fully elastic approach is considered a fairly accurate approach when considering sudden failure of one of the 1938-41 hangers, since the critical failure mode for these involves a brittle socket failure i.e., no ductility is available. For other hanger types we would expect a more ductile failure, hence this method may be overly conservative for the modern and bar hangers.

It should be noted that the failure of 2 or more hangers, i.e., consequences of dynamic failure of adjacent hangers following redistribution of load from the first failing hanger, has not been analysed numerically.

8.4 ALS Results – Stability of structure with one lost hanger

A distinct yet related assessment to that presented in Section 8.4 concerns analysis of the structure in its condition following a hanger failure, i.e., with the lost hanger ineffective and unable to participate in the global load path. This assessment relates to the continuing safety of the bridge in the aftermath of an event whereby a dynamic failure of a hanger has occurred and all other hangers appear intact.

The following questions would be of particular interest in such circumstances:

> Is it safe for individuals to access the bridge for inspection and to effect repairs?

> what level (if any) of restricted traffic live load can be safely sustained?

Practically such scenarios have been assessed by following the methodology described in Section 8.1 also. The only change is to not include the dynamic factor of 1.5; rather a value of 1.0 is used. This is justified on the basis that the dynamic hanger loss event has already occurred; this new analysis is simply assessing the structural system that has a missing hanger, with permanent load from the lost hanger having redistributed elastically into other adjacent hangers, i.e., increasing the tension these carry. Similarly loads arising from variable actions (e.g., vehicular traffic) are resisted only by the remaining intact hangers.

Figure 33 presents the results from this assessment. The following conclusions can be made from review of these results:

1 Under permanent loads only, all remaining hangers have been assessed to have adequate strength with a maximum utilisation of 0.73

2 Under the action of permanent plus 7.5t restricted live load, all remaining hangers are assessed to have adequate strength with a maximum utilisation of 0.88

3 Under the action of heavier traffic (18t restricted or greater), out of the 84 scenarios that consider one of the 84No. 1938-41 hangers to have previously failed, in 22No. (26%) of these scenarios one or more of the remaining hangers are assessed to be at risk of becoming overstressed, to a maximum utilisation of 1.08

Whilst conclusions (1) and (2) offer some comfort regarding the capacity of the bridge in the aftermath of a single hanger failure event, this does not negate the findings in Section 8.3 regarding the effect of the dynamic failure of a hanger in service where those most adversely loaded by the failure may also fail in a brittle manner.

Rather the conclusions in this section of the report should only be used to inform the management of the bridge in the event of a single hanger failure which has not led to an immediate progressive collapse.

Notwithstanding the numerical assessment results described above, in the event of a hanger failing the bridge should be considered a vulnerable structure with limited robustness. In no way are we suggesting that the bridge might be kept open/re-opened with one hanger missing. These results should only be used to inform decisions regarding safe access of people and equipment for inspection and remedial actions.

The remaining intact hangers would remain vulnerable to failing due other to other causes, e.g., progressive degradation and direct impact, as discussed in Section 6. And whilst the dynamic loss of a second hanger has not been assessed numerically, we would expect this to result in further knock-on overloading of other remaining hangers and initiate a disproportionate collapse. Furthermore, it should be noted that the loading considered in this section is for the Accidental Limit State, not the Ultimate Limit State, and thus it would not be acceptable to allow the bridge to be left exposed for a prolonged period.

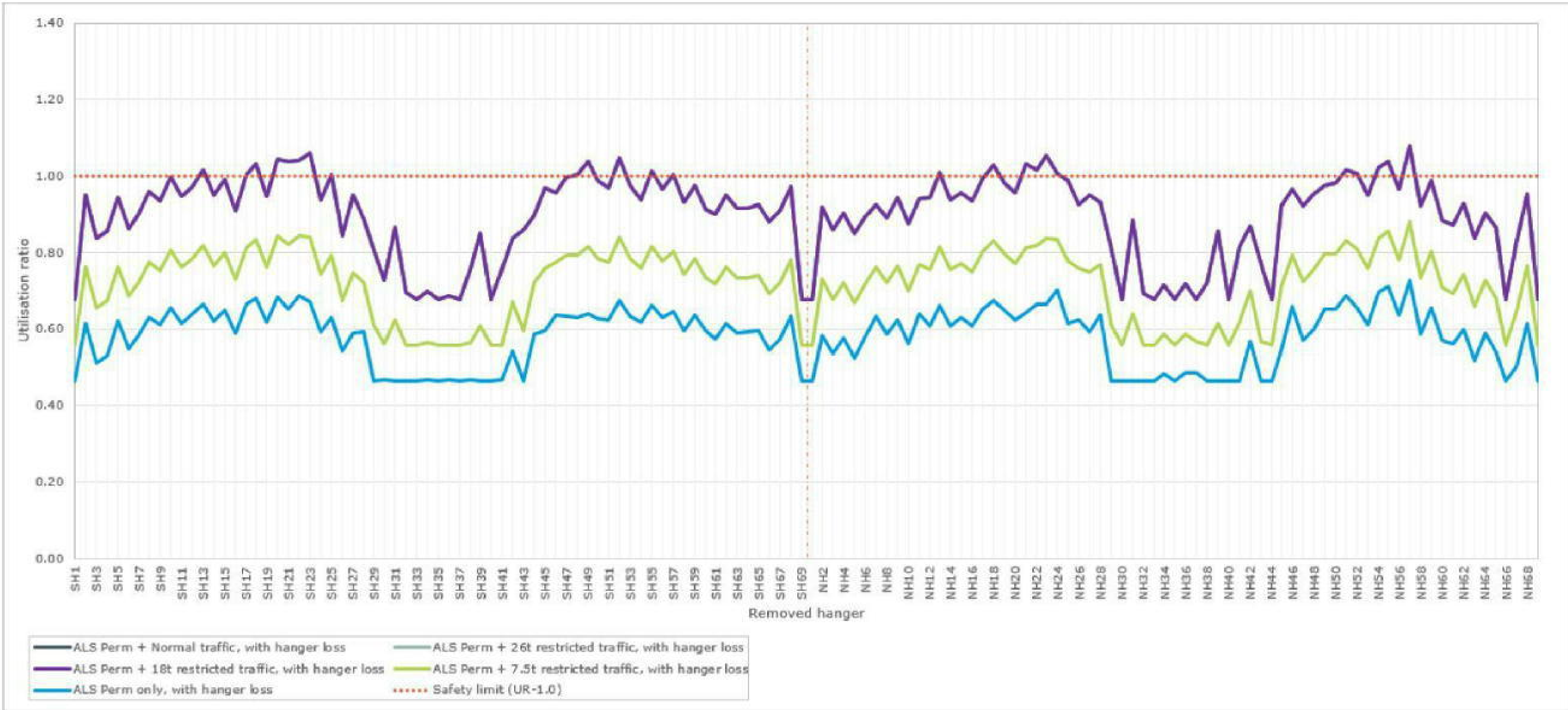


Figure 33: Maximum ALS utilisation ratios in any hanger, for scenarios where one hanger is considered ineffective (i.e., due to having failed previously)

This plot should be compared against related plot in Figure 32; the vertical and horizontal scales are the same. The distinction is that the above summarises the ALS assessment results for the circumstances where the bridge has one ineffective hanger, but virtue of it failing previously. Since there is no dynamic factor relating to the sudden abrupt failure of a hanger, as is accounted for in the analysis that Figure 32 presents the results from, hanger utilisation ratios are reduced.

9 Key Findings

COWI's assessment identifies the following key points:

1 This assessment considers the failure of a hanger on the suspended span with a particular focus on the remaining 1938-41 'original' spiral strand hangers.

2 COWI's assessment determines that Assessment Load Model 1 (ALL1) in DMRB CS 454 results in an unrealistic loading scenario (e.g. 7.5T lorries at 1m spacing in both lanes across the full span). Therefore, COWI have adopted Assessment Load Model 2 (ALL2) from DMRB CS 454. We do not consider this to represent a Departure from the codified guidance, since either ALL1 or ALL2 are applicable.

3 The 1938-41 spiral strand hangers are assessed based on their design tension resistance as calculated by Gibb & Partners in 1991 (285kN).

3.1 COWI note the Gibb & Partners' resistance is based on normal loading rates and does not properly account for the higher rate of loading that would be expected following sudden failure of an adjacent hanger

3.2 COWI also note that the Gibb & Partners' resistance does not account for loss of resistance due to the knock on corrosion defects at the sockets.

3.3 Thus, the 285kN design tension resistance used in this assessment may be non-conservative.

4 COWI have estimated the dead load in each of the hangers from consideration of the applied permanent loads and analysis of the bridge geometry

4.1 COWI's values have been shown to be similar to the site measurements undertaken in 1988 by Laing O'Rourke

4.2 Given tolerance of geometric survey COWI's estimate of hanger forces has an empirical variability of ±8kN for ALS/SLS and ±10kN for ULS. This equates to approximately 10% of the permanent load sustained by each hanger.

5 COWI's ULS assessment determines that the assessed capacity of all hangers exceeds the assessment load effects for all categories of loading e.g. Permanent Only, 7.5T, 18T, 26T and Normal Traffic.

5.1 This implies that failure of a hanger would not normally be expected to occur on the basis of traffic loading only, however, COWI note that the ongoing corrosion of the hanger sockets and the underlying brittle nature of the sockets could trigger failure of the socket at a lower load.

6 COWI's ALS assessment of the consequence of a dynamic hanger failure uses a dynamic amplification factor of 1.5. Whilst this is a reasonable and codified estimate, it should be noted that the upper bound of this value is 2.0 and thus the 1.5 estimate could be non-conservative.

7 COWI's ALS assessment of the dynamic failure of a hanger identifies the following key findings:

7.1 Were loss of a hanger to occur under the action of permanent loads only, all other remaining hangers have utilisation ratios less than unity. This implies that knock-on failures in other adjacent hangers should not occur in these circumstances.

7.2 Were loss of a hanger to occur under the action of normal, 26T or 18T traffic load (assessed to ALL2 Load Model), 100 out of the 138 hanger loss scenarios considered (72%) result in overstressing of adjacent hangers. This implies that knock-on failures in other adjacent hangers could credibly occur.

7.3 Were loss of a hanger to occur under the action of 7.5T restricted traffic loading (assessed to ALL2 Load Model), 9 out of the 138 hanger loss scenarios considered (7%) result in overstressing of adjacent hangers. Accounting for uncertainty in the hanger dead load tensions (Point 6 above), 32 out of 138 hanger loss scenarios considered (23%) result in overstressing of adjacent hangers. This implies that knock-on failures in other adjacent hangers could credibly occur, despite this level of traffic restriction.

Overstressing of adjacent hangers, following sudden failure of the first hanger, implies a progressive and disproportionate collapse mechanism, whereby failure of these second hangers in turn leads to further failures.

7.4 Therefore, COWI's ALS assessment of the dynamic failure of a hanger demonstrates that there is a credible risk of a progressive collapse of the bridge deck if specific hangers were to fail in service. Whilst this risk has been reduced significantly by implementing the 7.5T weight restriction, a credible risk remains.

8 COWI's ALS assessment of the stability of the structure following the loss of one hanger and progressive collapse has not occurred adopts a dynamic amplification factor of 1.0. This is justified on the basis that the dynamic hanger loss event has already occurred i.e., this analysis is simply assessing the structural system that has a missing hanger, with permanent load from the lost hanger having redistributed elastically into other adjacent hangers. From this assessment come the following key findings:

8.1 Under permanent loads only, all remaining hangers have been assessed to have adequate strength.

8.2 Under the action of permanent plus 7.5t restricted live load, all remaining hangers are assessed to have adequate strength.

8.3 Whilst conclusions (8.1) and (8.2) offer some comfort regarding the capacity of the bridge in the aftermath of a hanger failing, this does not negate the findings above regarding the effect of the dynamic failure of a hanger in service. Rather the conclusions in this section of the report should **only be used** to inform the management of the bridge in the event of a hanger failure which does not lead to immediate progressive collapse.

8.4 In the event of a hanger failing the bridge should be considered an extremely vulnerable structure with limited robustness and all people and traffic should be prevented from crossing over or under the bridge (as laid out in the current Emergency Plan) until such time as COWI can attend site and review the condition of the bridge.

10 Recommendations

Our ULS assessment indicates that the hangers are sufficiently strong to carry design loads arising from ULS traffic, in combination with other permanent and variable actions. This implies that one of the possible causes of hanger failure can be ruled out i.e. failure by quasi-static overloading (only) should not occur.

However, were a given hanger to fail in a brittle manner due to a different cause, such as:

- Brittle fracture arising from the effects of corrosion possibly exacerbated by low temperatures
- Direct abnormal loading of the hangers e.g. impact from vehicle or an unsecured load

then the ALS hanger loss analysis shows that such an event has the potential to result in overstresses in adjacent hangers, leading to a progressive and disproportionate collapse of the bridge deck.

This conclusion is strongly influenced by assumptions surrounding the vehicle loading being sustained by the bridge at the time of the failure of the first hanger; were only permanent loads being sustained, no knock-on failures are predicted to occur. However, if traffic levels were approaching the current 7.5T assessment load, conditional on which hanger failed, our assessment indicates that subsequent knock-on failures could credibly occur.

Furthermore, COWI's concerns regarding the loading rate used to determine the hanger capacity suggest that the true capacity of these hangers, if subject to a rapid change in tension due to failure of an adjacent hanger, may be significantly less than the value calculated by Gibb & Partner and adopted in this assessment. A detailed fracture mechanics study could be undertaken to review the derivation of the Gibb & Partner hanger capacity and its applicability to the situations consider in this analysis, but that is beyond the scope of this assessment.

Notwithstanding the above, this analysis identifies up to 32No. hangers as being at particular risk of failure under the 7.5T weight restriction, due to the variability in the dead load hanger tension estimates. This represents 38% of the remaining 1938 hangers and has implications for the ongoing management of this risk. i.e. if only a small number of hangers were affected it may be possible to replace them with temporary hangers until such time as the new hangers can be manufactured and installed. However, given the number of hangers affected this is not considered a viable approach. The risk of progressive collapse will only be reasonably mitigated by removing traffic loading from the structure and by the installation of the replacement hangers at the earliest available opportunity.

COWI have considered reducing the current weight restriction to 3T, however the k-factor associated with the 3T limit is equal to that of the 7.5T model for bridge lengths over 50m, according to the codified Assessment Live Load 2 model. I.e., the load model doesn't offer a reduction in the assessment load if a 3T limit were imposed.

Leaving aside the assessment code issue, COWI note that many 4x4's and SUV's would be over a 3T limit, and our experience at other locations indicates that many drivers would ignore a weight limit to avoid disruption and delay. Therefore, COWI doubt that a 3T limit can be reliably enforced on an unmanned bridge which raises the question of the practicality of any attempt to justify the ongoing use of the bridge by implementing a lower weight restriction.

Due to the inherently unpredictable nature of brittle fracture, it is extremely difficult to accurately determine the probability of such an event, but this analysis shows there is a credible risk of an unzipping failure of the bridge deck. Such an event may lead to significant loss of life and is thus considered an unacceptably high consequence event that could credibly occur.

COWI note that all modern design codes assume ductile behaviour and require any structure to be designed to avoid progressive collapse. Modern suspension bridges are designed to accommodate the sudden loss of one hanger (per the method applied in this study) or the non-dynamic loss of two hangers. Menai Suspension Bridge does not satisfy either requirement and this analysis demonstrates there is a credible risk of progressive collapse even with the 7.5T weight restriction in force.

Therefore, COWI recommend that the bridge should be closed to vehicle traffic until such time as the replacement hangers can be installed.

Whilst this analysis will need to be reviewed in full by the CAT3 Checker, COWI consider the recommendation to close the bridge should be implemented with immediate effect and not delayed until the CAT3 check process can be completed.

In the interim, COWI suggest the CAT 3 Checker is asked to review the key findings and recommendations of this report and comment whether they agree with the recommendations reached, based on the results presented. The full CAT 3 Check of the underlying analysis would then follow this initial review with the risk of collapse mitigated in the interim by the closure of the bridge to vehicle traffic.

Whilst COWI recognise that this analysis contains various unknowns and uncertainties, based on the information available at this time, COWI's recommendation remains that the bridge should be closed to vehicle traffic until such time as the replacement hangers can be installed.

Appendix A Analysis TAF (COWI Ref A238719-TN09-v3.0)

UK HIGHWAYS A55 LTD.

MENAI SUSPENSION BRIDGE – TECHNICAL APPROVAL FORM – ANALYSIS OF HANGER FAILURE

TECHNICAL NOTE NO.9

CONTENTS

PROJECT NO.	DOCUMENT NO.
A238719	TN09

VERSION	DATE OF ISSUE	DESCRIPTION	PREPARED	CHECKED	APPROVED
1.0	17/08/2022	CLIENT DRAFT	[redacted]		
2.0	18/08/2022	CAT3 DRAFT	[redacted]		
3.0	07/09/2022	CAT3 COMMENTS ADDED	[redacted]		

1 NAME OF SCHEME

Menai Suspension Bridge – Hanger Failure Analysis

1.1 Type of Highway

A5 trunk road. Single carriageway comprising one running lane in each direction across the bridge.

1.2 Permitted Traffic Speed (For a bridge give over and/or under)

The permitted traffic speed on the bridge is 30 mph.

2 NAME OF STRUCTURE

Menai Suspension Bridge

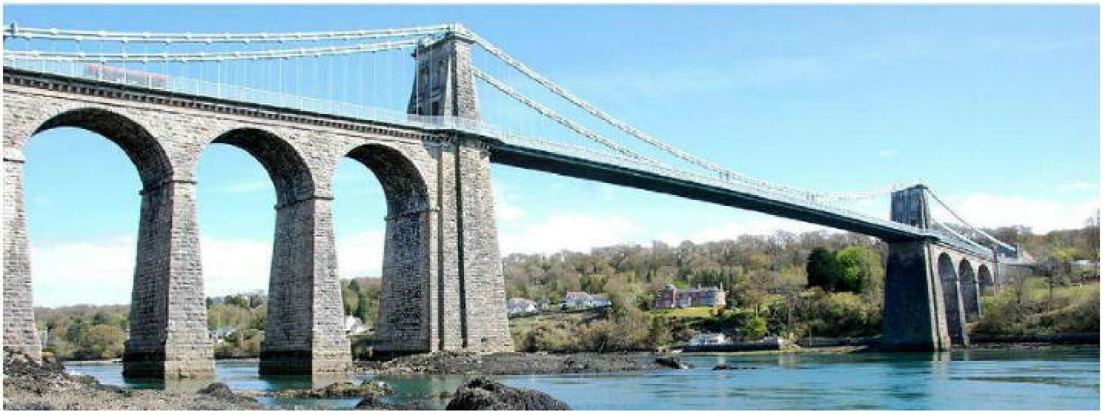


Figure 1 - General view of Menai Suspension Bridge

2.1 Obstacles Crossed

Menai Strait

3 PROPOSED STRUCTURE

3.1 Description of Structure

Menai Suspension Bridge was designed by Thomas Telford and built in 1826. The entire superstructure (chains, saddles, hangers and deck) was replaced between 1938 and 1940. The deck was again replaced in 2000.

The bridge was strengthened between 1988 and 1993. 40No. hangers were replaced, the deviation saddles were reinforced, and additional battens were installed on some bracing members within the longitudinal truss.

The Bridge is a Grade 1 Listed Structure.

The bridge consists of a pair of c.52.7m high masonry towers which support two paired sets of steel link chains across the 176.75m (579′ 10 $^1/_2$″) main span. The approaches are formed of masonry arch viaducts, four spans on the Anglesey side and three spans on the Bangor Mainland side. The chains support spiral strand hangers which are connected to a truss in the main span and are anchored to rods embedded in the masonry viaducts on the approaches. The central section of the main span is supported by 2 $^1/_4$″ square bar hangers.

The deck is 7m wide with a two-lane carriageway and is supported by a pair of 2.6m deep steel plate trusses with a 1.5m wide footway cantilever out on each side. The mastic asphalt surfacing is laid at depth of 38mm +/-3mm and underlain by a 3mm thick waterproofing membrane.

The approach viaducts terminate at masonry retaining walls which curve outwards and decrease in height to match adjacent ground levels. The masonry piers on both sides of the suspended deck are founded on solid rock. On the Anglesey approach all but two of the middle pier bases remain submerged almost permanently, except during Spring Low Tide when all the pier bases are exposed. On the mainland approach the pier bases are submerged at high tide only.

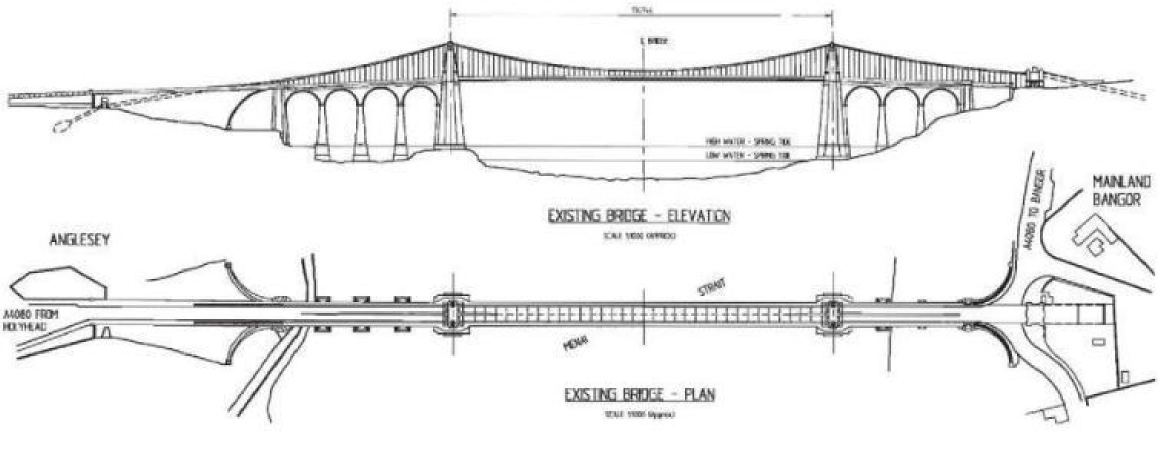


Figure 2 - General Arrangement Drawing

3.2 Structural Type

The main span is a suspended span of 176.75m (579' 10 $^1/_2$") supported by two sets of two steel plate chains. The chains are arranged in vertical pairs with hangers alternating between the upper and lower chains at 2.4m (8' 0") centres i.e. 4.8m on each chain.

The superstructure comprises a lightweight reinforced concrete deck composite with a 9.5mm thick deck plate and rolled steel cross girders at 0.6m (2' 0") centres. These are supported from the lower flanges of the two 2.6m (8' 6") deep longitudinal steel stiffening through trusses.

Footways either side of the deck are carried on tapered steel cantilever members riveted to the bottom chord of the trusses.

The bridge is supported on masonry towers of Penmon Stone on either side and the chains are anchored in tunnels in the rock on either side.

Figure 3 - View at deck level looking toward Bangor

3.3 Foundation Type

The masonry viaducts are founded on solid rock.

3.4 Span Arrangements

The main span is a suspended span of 176.75m (579' 10 $^1/_2$").

3.5 Articulation Arrangements

The longitudinal trusses rest at their ends on bearings on the cast in-situ corbels built into the main towers and are also suspended off the catenary chains. The chains are attached to the saddle bearings located on top of the support towers and continue to the intermediate saddle bearings located on the north approach and the bridge house. The anchor plates continued from the intermediate bearings into the chain tunnels, into which they are fixed.

The operation of the bearings is summarised as follows:

- 4No. Deck Bearings – sliding guided bearings allowing rotation about 3 axes and longitudinal displacement

> 4No. Tower Saddle Bearings – roller bearings allowing longitudinal displacement

> 4No. Deviation Saddle Bearings – roller bearings originally allowing longitudinal displacement. It is noted that these bearings do not move longitudinally and are therefore considered fixed owing to the anchor plates being embedded in concrete or contained within the bridge house.

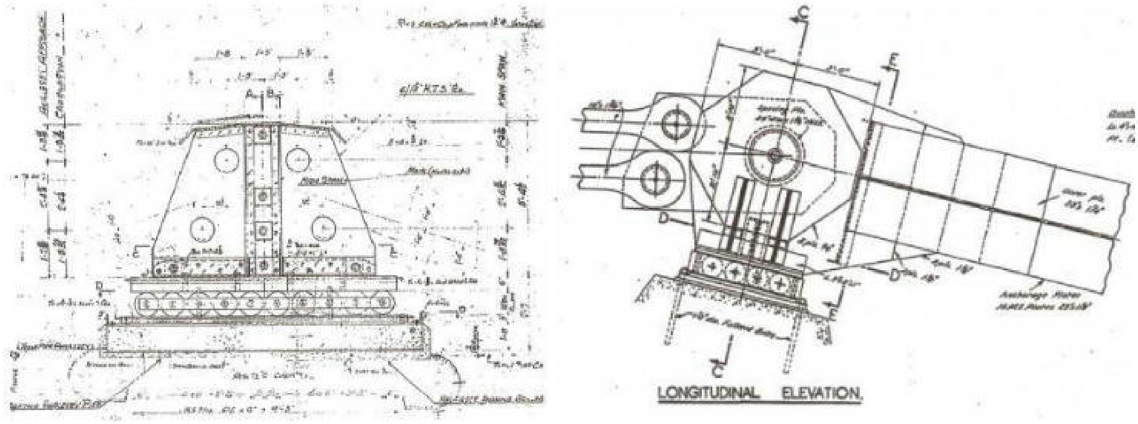


Figure 4 – Tower Saddle Bearings (L) and Deviation Saddle Bearings (R)

In addition to the bearings described above there are chain restraints on both approaches and atop the towers that restrict the relative lateral displacement of the chains and resist the forces generated by the deviation of the chains in plan, see Figure 5.

There are also 8No. 'sway guides' on the suspended span that restrict the relative lateral displacement of the chains and longitudinal trusses, see Figure 6.

Figure 5 - Chain Restraints (Bangor Approach - T, Tower Top - M, Anglesey Approach - B)

Figure 6 - Anglesey Sway Guides

3.6 Parapet Type

As a result of the through truss arrangement of the main span there are no vehicular parapets.

The cantilevered footways and approaches are provided with steel parapets comprising channel section posts, a 'D' section top rail and vertical infill. Parapet posts are installed at 2.4m (8' 0") centres on the suspended span and c.2.75m (9' 0") centres on the approach spans. The parapet on the wing wall is historic and is of historic wrought and cast iron.

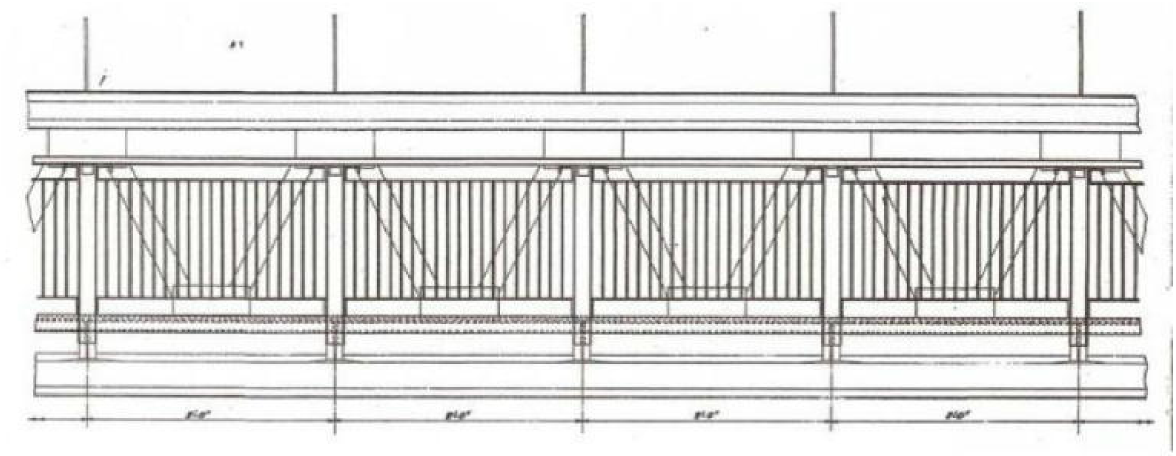


Figure 7 - Part Elevation of Main Span Parapets

3.7 Proposed Arrangements for Inspection & Maintenance

The current inspection and maintenance regime will be unchanged.

3.8 Materials and Finishes

	Location	Date of construction	Material Details
Masonry	Main Towers and Approaches	Original – Constructed 1826	Penmon Stone
Parapets	See 3.6 above		
Concrete	Road Deck	Replaced 1999	C40 LWA (Unit Weight 18.4kN/m^3)
Reinforcement	Road Deck	Replaced 1999	B385 fabric to BS4483 Grade 460 Type 2 Deformed Bars to BS 4449
Steelwork	Chains	Erected 1938-41	HTS to BS548 – Nominal Yield Stress 342N/mm^2
	Hangers – Steel Bar	Erected 1938-41	HTS to BS548 – Nominal Yield Stress 346N/mm^2 *From previous TAFs (e.g. 2005 Repainting TAF), not confirmed from original drawings.*

	Location	Date of construction	Material Details
	Hangers – Spiral Strand	Erected 1938-41	Steel Wire Rope to BS302 N.B. testing in 1991 achieved an actual breaking load of 989kN and 994kN (Nicholson 1996 Paper) for the rope. Socket Castings to BS15 Gibb & Partner testing determined a design strength of 285kN for the sockets.
	Hangers – Spiral Strand 35mm Dia (1x43) galvanised spiral strand 114 tonnes MBL	Replacement of 40No. hangers 1988-1991	Wire BS2763 Socket Castings BS3100 Socketing DINdin3092 Rope ASTM A-603
	Hangers – Spiral Stand	Replacement of 168No. hangers in 2023	TBC as design develops
	Hangers – Lower Eye Bar ($2^1/_4$" dia)	Erected 1938-41	MS to BS15 – Nominal Yield Stress - 228N/mm²
	Anchorage Plates	Erected 1938-41	MS to BS15 – Nominal Yield Stress - 236N/mm²
	Truss Members	Erected 1938-41	Main members: HTS to BS548– Nominal Yield Stress 357 N/mm² (not exceeding 32mm thickness) Diaphragms and tie plates: MS to BS15 – Nominal Yield Stress 247N/mm² (not exceeding 19.5mm thickness)
Corrosion Protection System	Generally, 3/4 coat system based on aluminium pigmented 2 pack epoxy primer plus 2 pack epoxy undercoat and polyurethane fineish i.e. HA painting manual item 115, 116 and 116. Except for soffit of the deck, cross girders and soffit of cantilever footways. The deck soffit is principally a build-up of chlorinated rubber overcoated with 115, 116 and 168 HA system. The cantilevered footways have a moisture curing polyurethane system comprising items 160, 162 and 168.		

4 DESIGN/ASSESSMENT CRITERIA

UK Highways A55 Ltd. intend to replace all remaining original (1938/39) hangers on the bridge to address underlying material deficiencies (N.B. a separate TAF has been prepared to cover the design of the replacement hangers COWI Ref A238719-TN08).

Until this work can be completed a 7.5T weight restriction has been implemented to reduce the risk of one, or more, hangers failing.

The purpose of the assessment work covered by this further TAF is to explore and conclude on the following questions:

> What influence does the abrupt failure of one hanger have on adjacent hangers? Will load redistribution (and associated short-term dynamic effects) cause these to become overloaded?

It should be noted that the assessment work covered by this TAF does not constitute a full assessment of the bridge, rather it is a limited investigation of the effect of a hanger failure.

Loading combinations involving wind loading are assumed not to govern and thus will not be considered, thermal actions will be considered as accompanying variable actions - i.e., load combinations 1 & 3 from CS 454 Table A.1 will be considered, load combinations 2 & 4 will not be considered.

4.1 Live Loading

4.1.1 HA Loading

ALL model 1 (cl.5.8 of DMRB CS 454 [Revision 1]) will be considered to obtain the live load envelope on the hangers.

The analysis will consider a range of *traffic levels* from normal traffic down to the current restricted traffic (7.5T).

The *road surface category* is classified as "good" since the current road surface category is "good" and this is unlikely to deteriorate significantly before the hangers are replaced.

The *traffic flow category* is classified as "high" as defined in Table 5.5N3. Whilst total HGV crossings are relatively low across an entire year, the bridge is the strategic diversion route for when Britannia Bridge is closed due to high winds. Therefore, high HGV traffic flows can be encountered, albeit for limited periods.

4.1.2 HB Loading

Not applicable.

4.1.3 Footway or footbridge live loading

Pedestrian ALL model as defined in cl.5.32 of DMRB CS 454 [Revision 1].

Accidental vehicle loading will not be considered as the footways are protected from vehicular traffic by an effective barrier (cl.5.27 of DMRB CS 454 [Revision 1]).

4.1.4 Provision for exceptional abnormal loads

Not applicable.

4.1.5 Any special loading not covered above

The analysis will apply Cl.2.3.6(2) of BS EN 1993-1-11:2006, incorporating guidance from the UK National Annex, to assess the bridge hangers for the sudden loss of any one hanger. This approach will account for 1) the influence of different levels of assessment live load, 2) elastic re-distribution of hanger forces to other components, 3) the dynamic response.

4.1.6 Departmental heavy or high load route requirements and arrangements being made to preserve route

Not applicable.

4.1.7 Minimum headroom provided

Not applicable.

4.1.8 Authorities consulted and any special conditions required

Authority Consulted	Special Conditions Required
North and Mid Wales Trunk Road Agent (NMWTRA)	UK Highways engaging with DBFO Representative.

4.2 List of relevant documents from the appendix hereto

See Appendices.

4.3 Proposed alternative proposals

Alternative Proposal to enable use of modern DMRB and standards previously submitted by UK Highways A55 Ltd. and accepted by Welsh Government.

5 STRUCTURAL ANALYSIS

5.1 Methods of analysis proposed for superstructure, substructure and foundations

For the purpose of this analysis the main span of the suspension bridge only, together with the tower saddles and back-span chains and hangers is considered. The 4 span north and 3 span arch approach viaducts are assumed to be fixed for the purposes of this analysis.

The superstructure will be analysed using a nonlinear 3D analysis.

For the avoidance of doubt, whilst the analysis will produce loading information for all components, we plan to conduct an assessment (i.e. compare resistance to applied loads) of the hangers only.

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

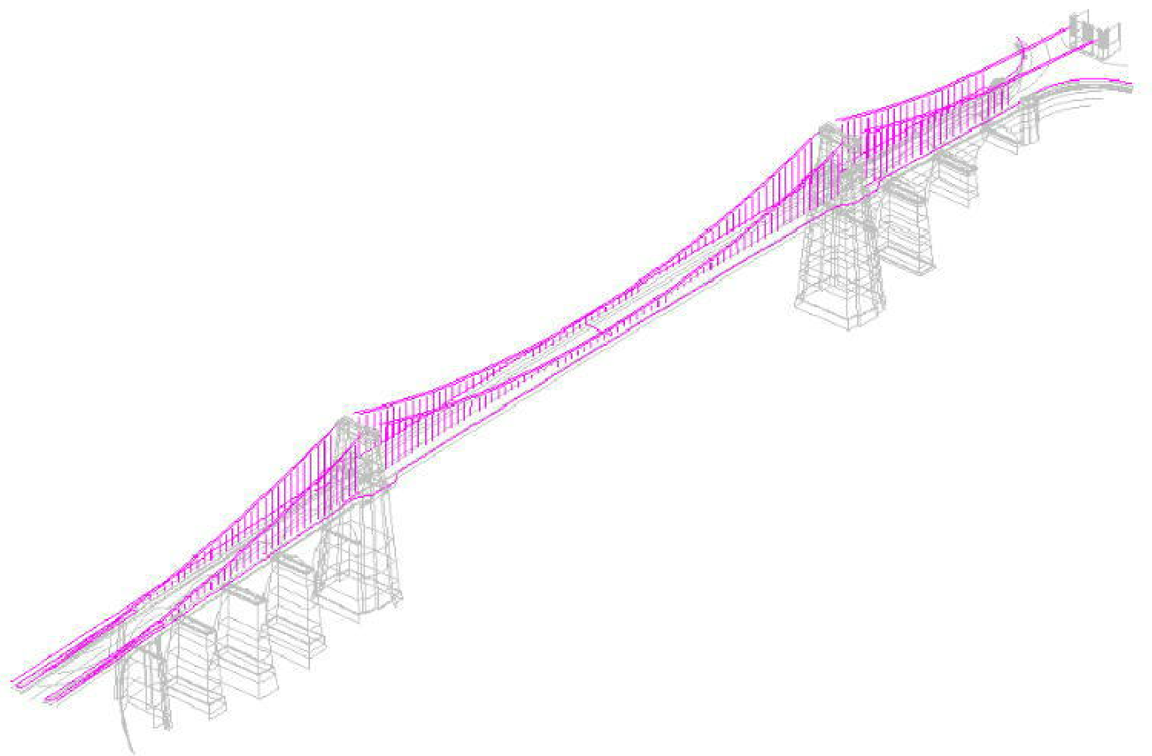


Figure 8 - Idealised structure

> The suspension chains will be idealised as beam elements.
> The spiral strand hangers will be idealised as bar elements.
> The bar hangers will be idealised as beam elements
> All members of the main truss will be idealised as beam elements
> The deck will be modelled as beam and plate elements with composite properties.
> Rigid offsets will be employed to reflect the true geometry at the connection between the hangers and top chord and between the deck cross girders and bottom chord.
> The towers will be modelled as providing rigid supports at saddle level

5.3 Assumptions intended for calculation of structural element stiffness

Gross transformed section properties will be used in the analysis.

The elastic modulus of the relevant materials will be assumed to be:

- Chains – 205,000 N/mm^2
- Bars and steel components of hangers – 205,000 N/mm^2
- Hanger Spiral Strand – 175,000 N/mm^2
- Steel truss members – 205,000 N/mm^2
- Concrete Deck – 35,000 N/mm^2 (assumed uncracked)

5.4 Proposed earth pressure coefficients (k_A, k_0 or k_P) to be used in the design or earth retaining elements

Not applicable.

6 GROUND CONDITIONS

6.1 Acceptance of interpretative recommendations of the soils report to be used in the design and reasons for any proposed departures

Not applicable.

6.2 Describe foundations fully including the reasons for adoption of allowable and proposed bearing pressures/pile loads, strata in which foundations are located, provision for skin friction effects on piles and for lateral pressures due to compression of underlying strata, etc.

Not applicable.

6.3 Differential settlement to be allowed for in design of structure

Not applicable.

6.4 Anticipated ground movements or settlement due to embankment loading, mineral extraction, flowing water, measures proposed to deal with these defects as far as they affect the structure

Not applicable.

6.5 Results of tests of ground water (e.g. pH value, chloride or sulphate content) and any counteracting measures proposed

Not applicable.

7 CHECKING

7.1 Proposed Category of Structure

Category III

7.2 If Category III, Name of Proposed Checker

Mott MacDonald

7.3 Temporary Works for which the DBFO Co. will be required to arrange an independent check listing parts of the structure affected

Not required.

8 DRAWINGS AND DOCUMENTS

8.1 List of Drawings (including numbers) and documents accompanying the submission. To include (without limitation):

8.1.1 Location Plan

General Arrangement – see Appendix B.1

Drg No.	Originator	Title
A238719-DRG-S01-0001-B	COWI	Menai Suspension Bridge – General Arrangement

8.1.2 Preliminary General Arrangement Drawings

Not required.

8.1.3 Relevant Parts of the Ground Investigation Report

Not applicable.

8.1.4 Supporting Documents

1999 DBFO Contract Drawings – see Appendix B.2

Drg No.	Originator	Title
LN00418-001-1	Hyder	North Elevation of Bridge – Hanger Identification
LN00418-002-2	Hyder	South Elevation of Bridge – Hanger Identification
LN00418-003-A	Hyder	General Details

1999 Re-decking Drawings – see Appendix B.3

Drg No.	Originator	Title
GD00280/BR37/D/001-A	Hyder	General Arrangement
GD00280/BR37/D/002-A	Hyder	Composite Deck Arrangement
GD00280/BR37/D/003-A	Hyder	Concrete Deck – Construction Sequence
GD00280/BR37/D/004-A	Hyder	Drainage, waterproofing & expansion joint details
GD00280/BR37/D/005-A	Hyder	Mastic Asphalt – Replacement Surfacing

1938 Reconstruction Drawings – see Appendix B.4

Drg No.	Originator	Title
507/C/02	Gibb & Partners	Record of New Chain Erection
507/C/03	Gibb & Partners	Details of Stiffening Trusses
507/C/05	Gibb & Partners	Main Chains & Anchorages
507/C/06	Gibb & Partners	Details of Saddles
507/C/07	Gibb & Partners	Expansion Joints at Piers
507/C/08	Gibb & Partners	Anglesey Approach (Sheet 1)
507/C/09	Gibb & Partners	Anglesey Approach (Sheet 2)
507/C/10	Gibb & Partners	Caernarvon Approach
507/C/11	Gibb & Partners	Details of Parapet
507/C/12	Gibb & Partners	Stages of Construction
507/C/24	Gibb & Partners	Bridge Master's House - Modifications
507/C/27	Gibb & Partners	Elevations of Main Pier
507/C/30	Gibb & Partners	Concrete & Asphalt Deck
507/C/36	Gibb & Partners	Arrangement of Post Office Cable Boxes & Pipes

9 THE ABOVE DESIGN AND CONSTRUCTION PROPOSALS ARE SUBMITTED FOR REVIEW

Signed: Team Leader, Design Team	
Name:	[redacted]
Engineering Qualifications:	[redacted]
Date:	

Signed: DBFO Co. Representative	
Name:	
Date:	

10 THE ABOVE TAF IS

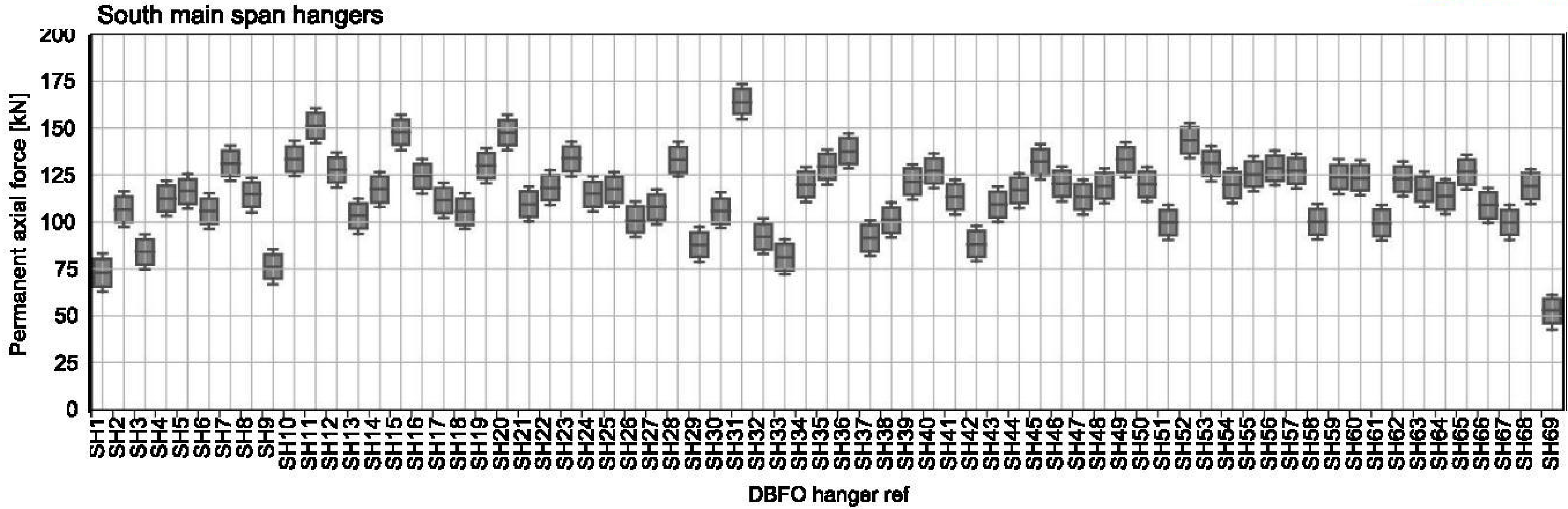
1 received.* 2 received with comments as follows.* 3 returned marked "comments" as follows.* *delete as appropriate.	
Signed:	
Name: For and on behalf of the Secretary of State for Wales	
Position:	
Date:	

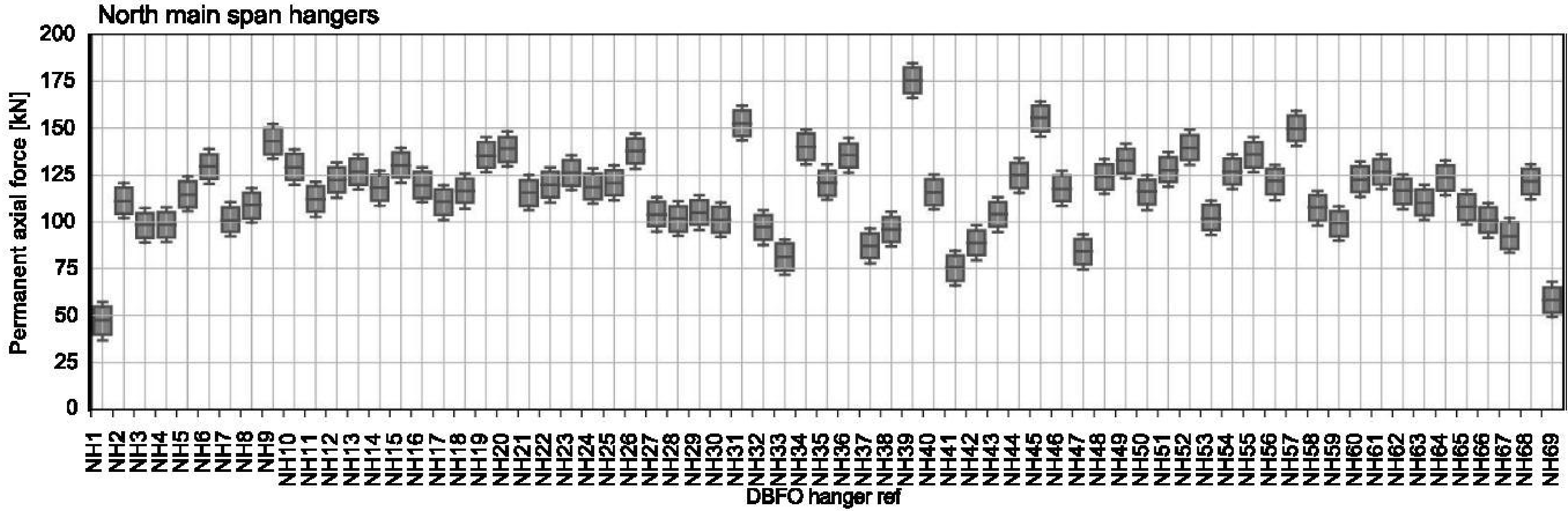
Appendix B Monte Carlo Analysis

Results in this section are like those presented in the main body of this report, except these quantify the uncertainty in assessment outcomes (e.g., utilisation ratios) that directly follow from uncertainty surrounding the dead load tensions sustained by specific hangers.

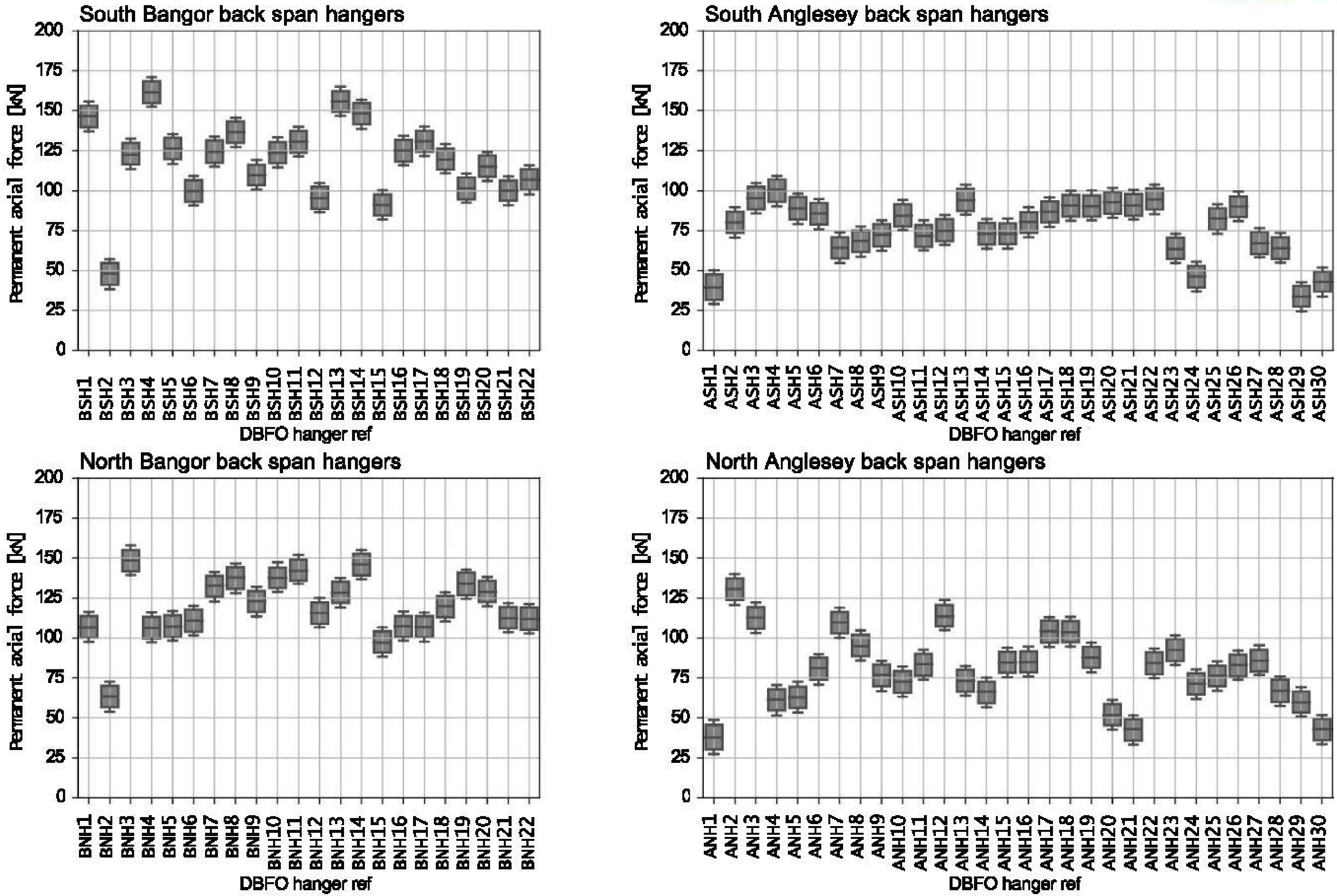
This does **not** constitute a full reliability-based assessment, since other sources of uncertainty / variability are not adequately captured.

Main span hanger forces
from Monte Carlo simulation





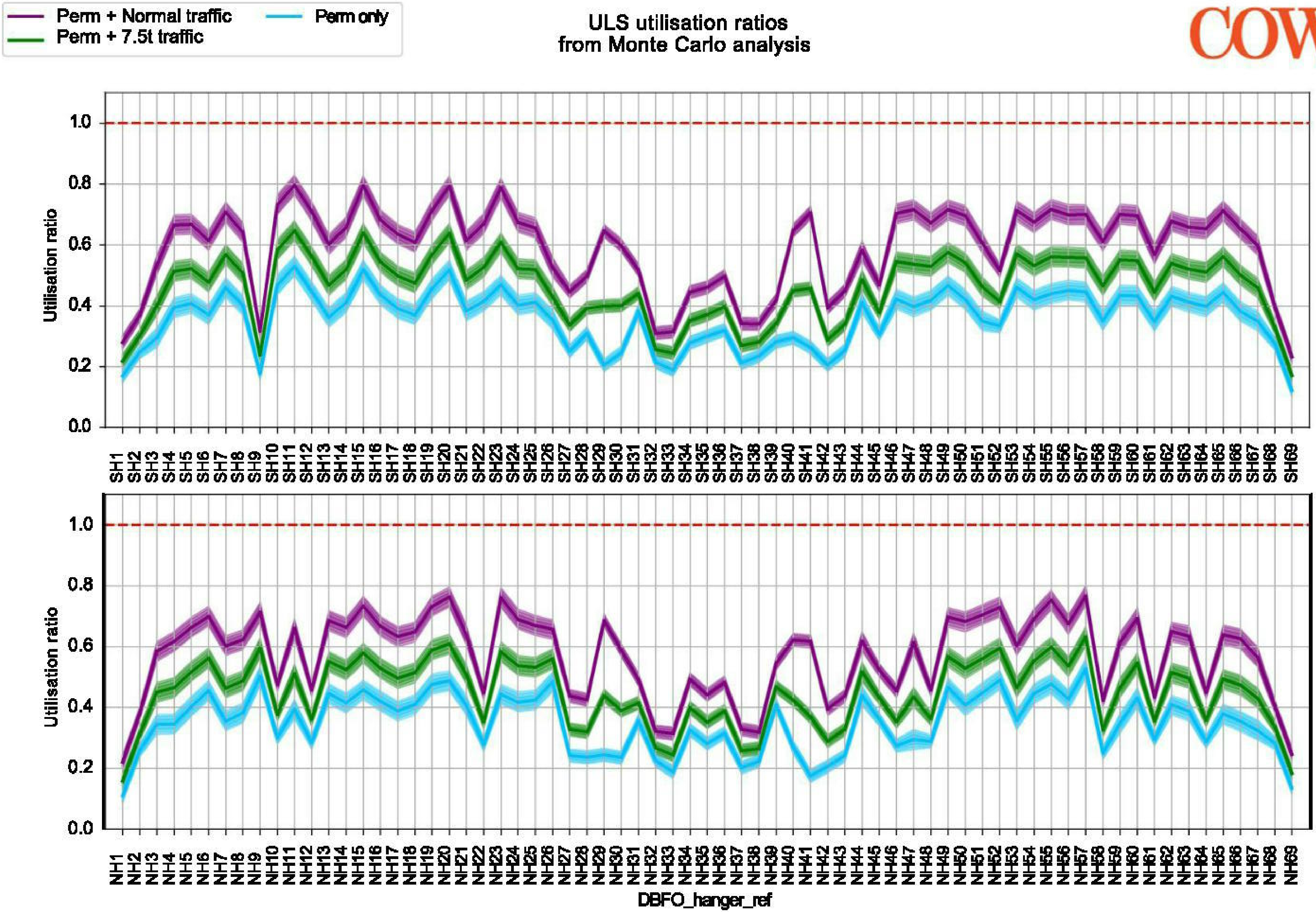
Bangor and Anglesey backspan hanger forces from Monte Carlo simulation

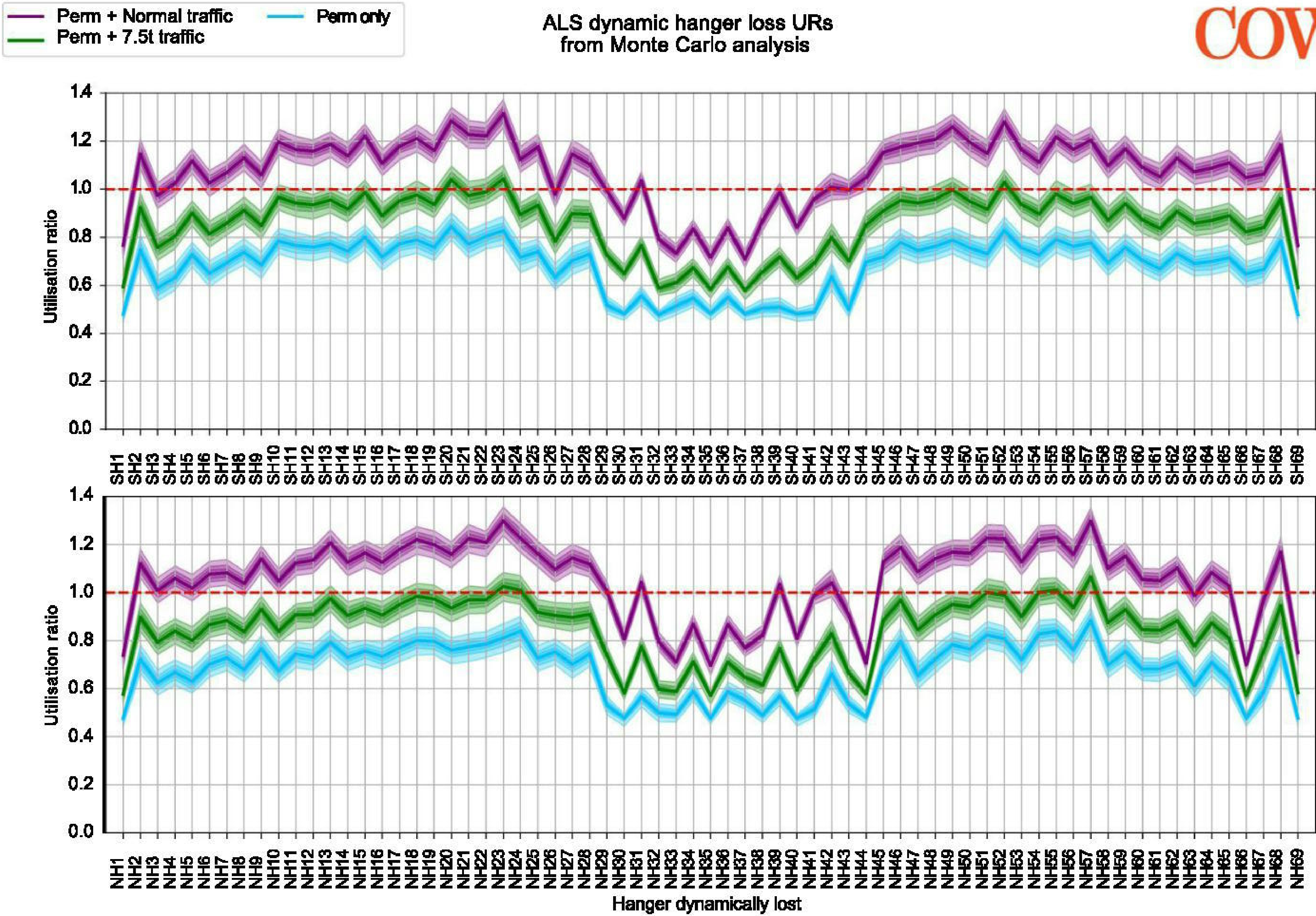


PERM-FX-BACKSPAN

Prepared: 17/10/2022 08:14
Engineer: [redacted]

ULS utilisation ratios
from Monte Carlo analysis




Perm + Normal traffic
Perm + 7.5t traffic
Perm only
ALS dynamic hanger loss URs
from Monte Carlo analysis
Utilisation ratio
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
SH1 SH2 SH3 SH4 SH5 SH6 SH7 SH8 SH9 SH10 SH11 SH12 SH13 SH14 SH15 SH16 SH17 SH18 SH19 SH20 SH21 SH22 SH23 SH24 SH25 SH26 SH27 SH28 SH29 SH30 SH31 SH32 SH33 SH34 SH35 SH36 SH37 SH38 SH39 SH40 SH41 SH42 SH43 SH44 SH45 SH46 SH47 SH48 SH49 SH50 SH51 SH52 SH53 SH54 SH55 SH56 SH57 SH58 SH59 SH60 SH61 SH62 SH63 SH64 SH65 SH66 SH67 SH68 SH69
Utilisation ratio
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
NH1 NH2 NH3 NH4 NH5 NH6 NH7 NH8 NH9 NH10 NH11 NH12 NH13 NH14 NH15 NH16 NH17 NH18 NH19 NH20 NH21 NH22 NH23 NH24 NH25 NH26 NH27 NH28 NH29 NH30 NH31 NH32 NH33 NH34 NH35 NH36 NH37 NH38 NH39 NH40 NH41 NH42 NH43 NH44 NH45 NH46 NH47 NH48 NH49 NH50 NH51 NH52 NH53 NH54 NH55 NH56 NH57 NH58 NH59 NH60 NH61 NH62 NH63 NH64 NH65 NH66 NH67 NH68 NH69
Hanger dynamically lost