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HIGHLANDS AND ISLANDS ENTERPRISE

CAIRNGORM FUNICULAR RAILWAY

ADDENDUM TO VIADUCT APPRAISAL REPORT

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ADDENDUM TO VIADUCT APPRAISAL REPORT

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APPENDICES

1 Introduction

In September 2018, Highlands and Islands Enterprise (HIE) engaged COWI UK Limited to assist in the investigation of the current condition of the Cairngorm funicular railway. In the months leading up to Christmas 2018, COWI undertook an appraisal of the viaduct in its current condition which is documented in report referenced: A116993 Rp01 v2 'Cairngorm Funicular Railway – Viaduct Appraisal Report' dated December 2018. The limited timescale available to complete this work limited the scope and depth of the appraisal.

Following issue of this report, review comments were received from HIE, Highland Council and ADAC Structures. A further scope of works was agreed to develop a concept strengthening scheme to address the shortfalls identified in the appraisal report. A number of areas of the appraisal were identified which would benefit from additional review. The intent of this work was to refine the appraisal results and thus reduce the scope of works requiring intervention and strengthening.

This addendum accompanies the appraisal report and records the additional reviews undertaken and provides commentary on their outcome.

2 Appraisal report review

Following the issue of the appraisal report COWI received comments from HIE, Highlands Council and ADAC Structures. A number of additional reviews of the structure were identified as useful resulting from these comments. COWI also identified other areas in which additional refinement of the appraisal would be likely to be worthwhile. The identified areas for review are summarised below;

- Refined three dimension Finite Element Modelling of the foundation/ground interaction
- Confirmation of the assumed link spacing throughout the length of the longitudinal primary beams
- Concrete strength sensitivity analysis
- A review of torsion effects of the main longitudinal beams
- Review of the implications of serviceability limit state overloads for the operational performance of the funicular carriage system
- Review of design wind speed and application of C_d (drag) factors
- Review of the accidental design load case for the most extreme wind event
- Further analysis to quantify the strength of the in-situ concrete stitches

3 Supplementary reviews

This section describes the supplementary work to the initial structural appraisal that has been undertaken since the most recent version of the appraisal report (A116993_RP01_v2) was issued.

In general, only brief summaries of the various studies are given in this report. A description of why further review was necessary, what review was undertaken and the conclusions are discussed. Reference is made to the appendices of this document for more detailed reports on each topic in the form of Technical Notes.

The last topic is discussed in full within this section and no reference to appendices is made.

3.1 Refined Finite Element Modelling (FEM) foundation analysis

For a complete description of these works, refer to Appendix A.

3.1.1 Purpose of a review

The appraisal report used the traditional effective foundation area method of calculating bearing capacity. As the pad foundations supporting the viaduct piers are embedded in sloping ground, the beneficial effects of lateral loads on the foundation and the contribution to the bearing capacity from backfill were not easy to quantify. Hence a conservative approach was taken.

The findings of the appraisal concluded that under out of service "Accidental" loading, the high value of the calculated eccentricity of reaction force led to either overstressing of the foundation or in extreme cases overturning of the foundation.

A reappraisal of the foundations using 3-dimensional finite element analysis would give a better result than the traditional method, as it would be able to take account of the sloping ground.

3.1.2 Summary of review

Two technical notes were produced.

The first note compares and contrasts the results derived using the two methods of assessment. An analysis of a notional foundation with no sloping ground shows similar results for traditional method and the finite element method, validating the use of the finite element method.

The second note uses the 3-dimensional finite element analysis to quantify bearing resistance, settlement and horizontal deflections for the four piers considered in the appraisal, taking the sloping ground into account. Analyses under in service "Operational" and out of service "Accidental" and "Evacuation" loading were undertaken.

3.1.3 Outcomes and recommendations

The new 3-dimensional finite element analyses give more refined results when compared to the earlier traditional analysis approach. The new results show the foundations are adequate to resist bearing reactions under in service operational and out of service accidental and evacuation loading without need of foundation strengthening.

Furthermore, the magnitude of calculated base settlement and pier head deflections from the new analyses provides an indication of the expected rotation of the piers under the action of dead and live load only.

The benefit of this refined approach in eliminating the need for strengthening of the existing foundations is clear.

3.2 Non-Destructive Testing (NDT) analysis of shear link spacing

For a complete description of these works, refer to Appendix B

3.2.1 Purpose of a review

As-built records made available to COWI showed shear link spacing along the length of all longitudinal beams. A number of drawings were contradictory in the areas adjacent to the scarf joint. An NDT scope of works to provide as-built information on the spacing of shear links focused primarily on this area. Within this scope of works only a single beam, of a single type, was scanned along its full length. These results did not match the anticipated spacing from the as-built information. The spacing measured was greater than expected in some areas and thus this increased spacing was used in the appraisal of the shear capacity of the beams. The critical location was found to be the point at which the shear link spacing transitions from 100mm to 200mm, approximately 2.1m from beam ends. Further NDT was required as part of this refined appraisal to extend the number and types of beams scanned to determine whether appraisal assumptions of shear link spacing along the lengths of all types of beams was valid.

3.2.2 Summary of review

Additional NDT scanning of 8 No. beams covering all beam types, (Type 1, Type 2, and Type 3) was undertaken by Henderson Thomas Associates. As a result, total data available for the transition points between link spacing was available for 18 No. beam ends. This represented a better distribution of investigations to conclude the assumed spacing along the length of all pre-cast longitudinal beams. This data was evaluated.

3.2.3 Outcomes and recommendations

Although there are some differences between the shear link spacing assumed in the structural appraisal and the typical shear link spacing in the surveyed rail support beams, the appraisal results were not be affected. This work has increased confidence in the appraisal findings for this element and no further action is recommended.

3.3 Concrete strength sensitivity analysis

For a complete description of these works, refer to Appendix B.

3.3.1 Purpose of a review

The appraisal used material grades as outlined in the Schedule of Basic Assumptions and recorded on the original design and check certificate. These values are typically the minimum values required in the design. It is common for as-built strengths to exceed the design strengths, which can lead to conservatism in assessment. If the strength of the concrete used in the appraisal of capacity were to be increased, shortfalls could be reduced resulting in less strengthening. Prior to undertaking intrusive investigations, a sensitivity analysis using varying concrete strengths was undertaken. This would determine whether a significant increase in capacity could be gained and whether further investigations would be worthwhile.

3.3.2 Summary of review

Concrete strengths ranging from 40Mpa to 80MPa were analysed at locations where overstress was identified. Shear capacity was determined by both BD 44 and Eurocode standards.

3.3.3 Outcomes and recommendations

A limited amount of benefit was identified if concrete strengths were higher than used in the appraisal. This was most beneficial on the type 2 beams. The level of enhancement is unlikely to eliminate the requirement for strengthening. Any reduction in scope of strengthening would be isolated and thus widespread strengthening of beams would still be required. It was concluded that due to the cost of intrusive investigations, the risks associated with results and the little benefit for reducing extents, that further investigations into as-built concrete strength were not undertaken.

3.4 Analysis of torsional effects

For a complete description of these works, refer to Appendix C.

3.4.1 Purpose of a review

Appraisal assumptions used engineering judgement to determine that torsion did not govern and was therefore ignored. A review of torsion effects was undertaken to determine whether this assumption was valid.

3.4.2 Summary of review

Torsional resistance was determined and analysis options discussed. Commentary is made on the analysis options: (1) line beam theory (2) a grillage model with torsion properties (3) a torsionless grillage. A series of prototypes grillages were analysed and the resultant load effects discussed.

3.4.3 Outcomes and recommendations

Torsion can be ignored if it takes a minor role in the behaviour of the structure and has an alternative load path for transferring lateral loads on the rails to the supports. Beam webs and lateral bracing are determined to be capable of resisting lateral loads and thus an alternative load path exists. It is therefore concluded that it is valid to proceed as if the beams are not acting in torsion and the most appropriate approach for dealing with torsion is the one described in the appraisal report. No further action is recommended.

3.5 Review of SLS deflection and rotation limits

For a complete description of these works, refer to Appendix E.

3.5.1 Purpose of a review

The appraisal report identified shortfalls in the ability of the viaduct to meet the serviceability limit state requirements for funicular railways. These limit 'in service' deflections and rotations of the civil supporting structures to enable adequate functionality of the funicular railway systems that they support. The recommendations in the appraisal report suggested confirmation with the supplier, Doppelmayr, on the implications of these results.

3.5.2 Summary of review

Results of the appraisal were discussed with Doppelmayr and then Garaventa. Application of a draft version of BS EN 13107 at the time of original design, and the subsequent assessment to modern standards was discussed.

3.5.3 Outcomes and recommendations

A review of the draft and current BS EN 13107 standard concluded that no code requirement exists for a check of rotations at piers for the Cairngorm funicular railway.

The calculated beam deflections, although outside the limits of current design standards, were justified as not being of operational concern. This justification used the funicular's operational performance over the life time of the structure with no known operational issues with the funicular systems that they support.

HIE must ensure that any future supplier for new or refurbished funicular systems are satisfied with this approach and the as-built stiffness of the structure as this may have a direct impact on the serviceability of the funicular.

3.6 Review of design wind speed and Drag factors

For a complete description of these works, refer to Appendix E.

3.6.1 Purpose of a review

The design wind speeds used in the structural appraisal were taken from operational requirements as stated in the original Design and Check Certificate. The source document upon which these wind speeds were based (a report by the University of Edinburgh) could not be obtained and therefore the validity of the wind speeds was unknown. An exercise to establish whether the design wind speeds were realistic was intended to validate the load cases used in the appraisal.

A review of Drag factors used in the appraisal was also undertaken as these were questioned as being high and thus severe.

3.6.2 Summary of review

Wind speed data obtained from the weather station on the summit of Cairn Gorm was processed. Data was available from circa 1990. Extreme value analysis methods were used to derive an approximate 1 in 50 year maximum gust speed from the data.

3.6.3 Outcomes and recommendations

The maximum gust speed derived from the data is comparable to the design wind speed at the top of the viaduct for the funicular railway. Limitations with the data acquisition over the years, sample recording method and differences between the local topographies mean that there is a degree of uncertainty in the exact value. Nonetheless, it is recommended that the design wind speeds used in the initial design are reasonable and continue to be used.

Drag factors are a product of structural form and geometry. Although severe, drag factors used in the appraisal are consistent with the relevant standards.

Wind tunnel testing or a study using computational fluid dynamics could be undertaken in an attempt to reduce drag factors. HIE may wish to consider this approach.

3.7 Accidental wind load case sensitivity analysis

For a complete description of these works, refer to Appendix D.

3.7.1 Purpose of a review

Within the structural appraisal, many over-utilised components were found to be governed by the "Accidental" wind load case. This case involves a broken-down carriage clamped to the rails with a coinciding major wind storm. The assessment codes of practice required a quantitative approach to BD 37/01 using load and material factors of safety. This situation has not been experienced by the structure during its 18 years since construction. This load case is a combination of two extreme events, is unlikely to occur and does not endanger life because evacuation would take place before arrival of the extreme storm conditions. A review was proposed to understand the sensitivity to the results of the appraisal if this case is either removed or an unfactored Eurocode approach to an accidental design situation is used. The benefit of this review would be to reduce the extent of shortfalls to the viaduct and hence any strengthening scheme required.

3.7.2 Summary of review

A sensitivity analysis was conducted to compare the extent of strengthening that would be required for three options: (i) keeping the Accidental wind case as a factored load case, (ii) using unfactored load partial factors with the Accidental wind case as per the Eurocodes, and (iii) removing the Accidental wind case altogether.

3.7.3 Outcomes and recommendations

If the Accidental case is treated as Eurocode unfactored for design of any strengthening works, the scope of strengthening would reduce. However, issues with pier crosshead capacity, bearing uplift, and large transverse loads through the guided bearings would still exist.

If the Accidental case is removed entirely, the strengthening required to meet In Operation actions would generally also cover any requirements for Out of Operation actions.

It is discussed that in such an extreme event, no person is put at risk during such a load case and the risk is purely commercial. However there is still an obligation to take measures to avoid disproportionate damage due to accidental actions. This accidental load case is such an event. Removal of this loadcase is a decision that only Highlands and Islands Enterprise can make and would require the agreement of approval bodies.

Discussions with Highlands and Islands Enterprise have indicated thus far that they prefer to keep the Accidental load case. As Eurocodes will be used for the design of the strengthening scheme, it is recommended that the Accidental load case be treated as an Accidental Design Situation and therefore unfactored.

3.8 Beam shear capacity

3.8.1 Scarf joint

Further review has been undertaken with the intention of quantifying the strength of the scarf joints in their current condition. The shear resistance approach of the appraisal used shear enhancement within 3'd' of the support and states that shear need not be assessed within 'd' of the support at all.

The scarf joint falls within this enhancement zone and could be argued to not require assessment within the application of BD 44/15. The appraisal highlights the risk of a failure mechanism not assessed due to the unusual in-situ concrete construction joint.

Typically the assessment codes would treat the hogging zone over a pier support as a continuous monolithic block in this application. Although top reinforcement continuity has been demonstrated, the existence of diagonal construction joints poses some difficulty. The vertical shear links within this joint are not fully effective due to limited continuity and the concrete contribution to the shear capacity is questionable due to the reduced strength at the interface. Also it is known that gaps appear on passage of live load across the diagonal joint at some locations.

Further numerical review of the scarf joint has assumed extreme scenarios using the truss analogy where the in-situ portion of the scarf joint is treated as non-effective. However, for truss analogy to develop the tension component in the vertical links, the bottom reinforcement would need to be continuous above the pier. This is not present as bottom bars are not adequately anchored. Even using optimistic values for the bottom bar anchorage and the optimum shear angle, the shear resistance was not sufficient for the applied loads.

The reliability of any numerical conclusion for the scarf joint would be based on a significant number of assumptions which would be difficult to confirm. The only conclusion that can be drawn from further review would be that any quantification to this element would result in residual risk of an unknown failure mechanism developing. This risk would need to be managed. Mitigation of this risk would be through strengthening or load testing to prove competency.

3.8.2 Beams

Further review of the assessed shear capacity with an applied adjustment factor due to axial load has been undertaken. This adjustment is permitted by BD 44 and is applied to the concrete strength component.

The effect of this review is for some areas shear utilisations increase. The critical utilisation increases from 1.23 to 1.38 in the worse locations. The implications of this require Table 6-6 of the appraisal report v2 and the paragraph following to be updated. For span 56-57 shear resistance is 190 kN/beam resulting in a 38% overstress. The reduced maximum wheel load would be only 40kN. To obtain this wheel load and comply with the appraisal standards, would require an occupancy limitation within the carriage of approximately 10 persons, assuming 80 kg per person. This only affects a small number of beams but will increase the required strengthening work at these limited locations.

4 Conclusions

The supplementary work described in Section 3 alters the conclusions that were presented in the Appraisal Report A116993 Rp01 v2. The original findings regarding the overstress of the pier foundations and the excessive deflections and rotations are no longer applicable. An updated summary of appraisal results (Table 7-1) is presented below in Table 1.

Element	Mode of failure	Result of In Operation load	Result of Out of Operation load**
Main beams	Vertical deflections	ok	not applicable
	Rotations	not applicable	not applicable
	Transverse deflections	ok	ok
	Sag bending	ok	ok
	Hog bending	42% overload	In operation governs
	Shear in span	38% overload	In operation governs
	Shear at scarf joint	Strength cannot be determined	
Bracing	Tension or compression	ok	ok
Bearings	Misalignment	Temperature limited to -3ºC	
	Vertical capacity	65% overload	not applicable
	Lateral capacity	Assume overloaded	
	Uplift capacity	ok	Uplift occurs
Piers	Deflections	ok	ok
	Crosshead links	ok	15% overload
	Column bending	73% overload *	In operation governs
	Column shear	ok	2% overload
	Base slab bending	ok	ok
Pier foundations	Bearing capacity	ok	ok

** Certain columns would also fail in bending under impact load*

*** Based on the Eurocode unfactored Accidental Design Situation*

Table 1 Summary of appraisal results – updated to account for supplementary work

5 Recommendations

The Appraisal Report A116993 Rp01 v2 includes a list of recommended long-term measures for rehabilitation of the viaduct. Following the supplementary work described in Section 3 of this addendum, certain measures are no longer deemed necessary. An updated summary of recommended long-term measures (table 8-1) is presented in below in Table 2.

Element	Mode of failure	Result of In Operation load	Long term measure
Main beams	Deflection	ok	None required
	Rotation	Check not required	None required
	Hog bending	42% overload	Apply permanent load restriction or strengthen
	Shear	38% overload	Apply permanent load restriction or strengthen
Bearings	Misalignment	loss of contact area below +5ºC	Replace bearings
	Vertical overloading	65% overload	Replace bearings
	Lateral overloading	not quantified	Replace bearings
Piers	Column bending	73% overload	Strengthen piers, e.g. by propping and apply permanent protection
Pier foundations	Bearing pressure	ok	None required

Table 2 Recommendations for long term measures – updated to account for supplementary work

Appendix A Refined FEM foundation analysis

HIGHLANDS AND ISLANDS ENTERPRISE

CAIRNGORM FUNICULAR RAILWAY

FOUNDATION ASSESSMENT – COMPARISON OF NUMERICAL MODELLING WITH EFFECTIVE FOUNDATION AREA METHOD IN DETERMINATION OF FOUNDATION BEARING CAPACITY

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CONTENTS

1 Introduction

The preliminary design assessment of the shallow foundations for the Cairngorm Funicular Railway was undertaken using the effective foundation area method of calculating bearing capacity outlined in Annex D of the Eurocode 7 (EC7). As the pad foundations supporting the viaduct piers are embedded in sloping ground, the assumed contribution from the backfill, especially the favourable lateral resistance, on the bearing capacity of the foundation is not easy to quantify.

The findings of the preliminary design assessment reported in the Appraisal Report Ref: A116993 RP01 v2 concluded that under out of service "Accidental" loading representative of high transverse shear and bending moments, the high value of the calculated eccentricity of reaction force led to either overstressing of the foundation or in extreme cases (i.e. at P91) overturning of the foundation prohibiting calculation of a bearing capacity.

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As a consequence, further examination of the lateral resistance to foundation loading provided by the foundation backfill material has been carried out as part of an evaluation of the need for foundation strengthening works.

A 3D finite element (FE) assessment of the foundation behaviour under in service "Operational" and out of service "Accidental" and "Evacuation" loading has been undertaken which shows a significant difference between the two methods of assessment (i.e. the effective foundation area and numerical modelling.

This technical note describes the process undertaken to compare and contrast the results derived using the two methods of assessment. It describes the findings of supplementary 3D FE modelling undertaken on a foundation with simplified geometry (L/B =2) subject to uni-axial loading which is founded at ground level on horizontal ground.

While the results of this assessment are shown to confirm the validity of the effective foundation area method used in the preliminary design assessment it is clear from the findings of the numerical assessment carried out on selected foundations at Piers (P91, P61, P46 and P18) that the simple bearing capacity design assessment approach results in conservative assessment of bearing capacity when the effects of sloping ground and bi-axial loading are modelled explicitly.

2 Foundation model and loads

The shallow foundation modelled in the Plaxis 3D program has the same geometries as pier P91 but is founded at ground surface on horizontal "flat" ground.

The underlying soils are modelled with the same geology, i.e. 1m of weathered rock overlying granite bedrock.

A Mohr-Coulomb constitutive soil model was adopted in the FE assessment.

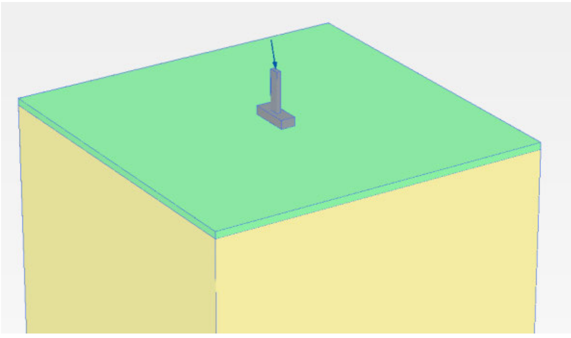
The soils were modelled with the following characteristic parameters.

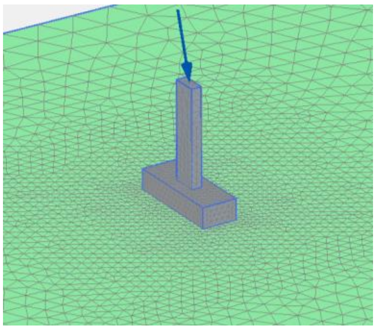
Table 2-1 Characteristic Parameters

Soil	Unit Weight (kN/m^3)	Young's Modulus (MPa)	Poisson Ratio	Internal Angle of Friction (Deg)	Cohesion (kPa)
Weathered Rock	18	50	0.2	42	1
Granite bedrock	26	2000	0.2	50	500

The foundation is modelled as a linear elastic material adopting a cracked concrete stiffness with Young's Modulus equal to 20GPa.

The Plaxis 3D mesh is shown in Figure 1 below.





a) overall mesh *b) detailed local element distribution*

Figure 1 Plaxis 3D FE mesh

The loads applied to the model at bearing level 7.2m above the base of the foundation are:

- Vertical load: 418kN
- Lateral longitudinal load: 139kN and 60.5kN (corresponding to 100% and 50% of SLS load in the longitudinal direction)

For comparison, these loads were applied to the foundation in the EC7 method of bearing capacity calculation.

3 Discussion of Results

3.1 High lateral load of 139kN

Under 100% of the lateral load (applied in the longitudinal (x) direction) , the method of calculation (EC7:2015) showed an eccentricity of more than 1m (i.e. >B/2) and hence the eccentricity lies outside of the area of the base indicative of failure by overturning. The effective area of the foundation is consequently zero prohibiting calculation of bearing capacity.

In the 3D FE assessment, the model failed to converge under the development of high concentration of stress at the edge of the foundation due to the large eccentricity, see Figure 2 below.

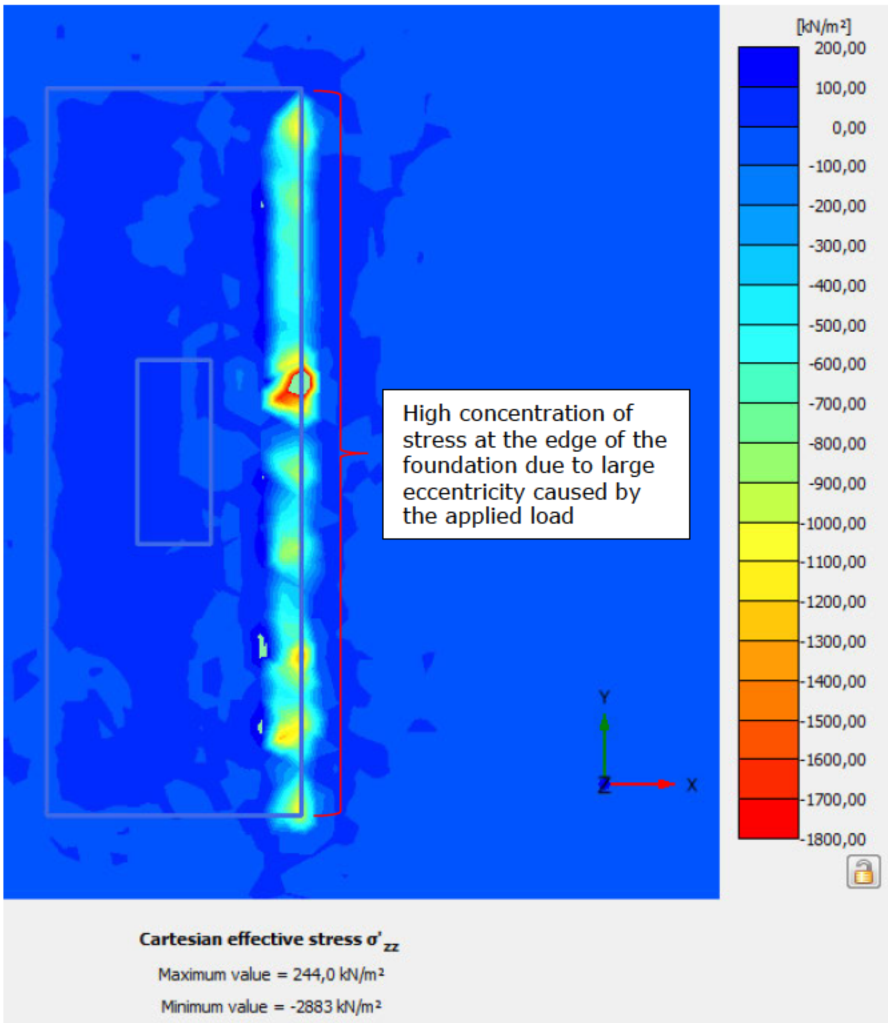


Figure 2 Computed vertical stress beneath the foundation

The finding of the 3D FE confirming the findings of the effective foundation area method.

3.2 Low lateral load of 60.5kN

Under 50% of the lateral load (applied in the longitudinal (x) direction), EC7 method of calculation showed an eccentricity of 0.53m, which lies outside the middle third of the footing. This eccentricity leads to an effective area of the foundation approximating to half the area of the footing. The average stress calculated for the effective area under this load is about 150kPa.

In the 3D FE assessment, the stress distribution on the foundation soils is shown in Figure 2 with a maximum stress of about 140kPa with local peaks slightly in excess of 200kPa.

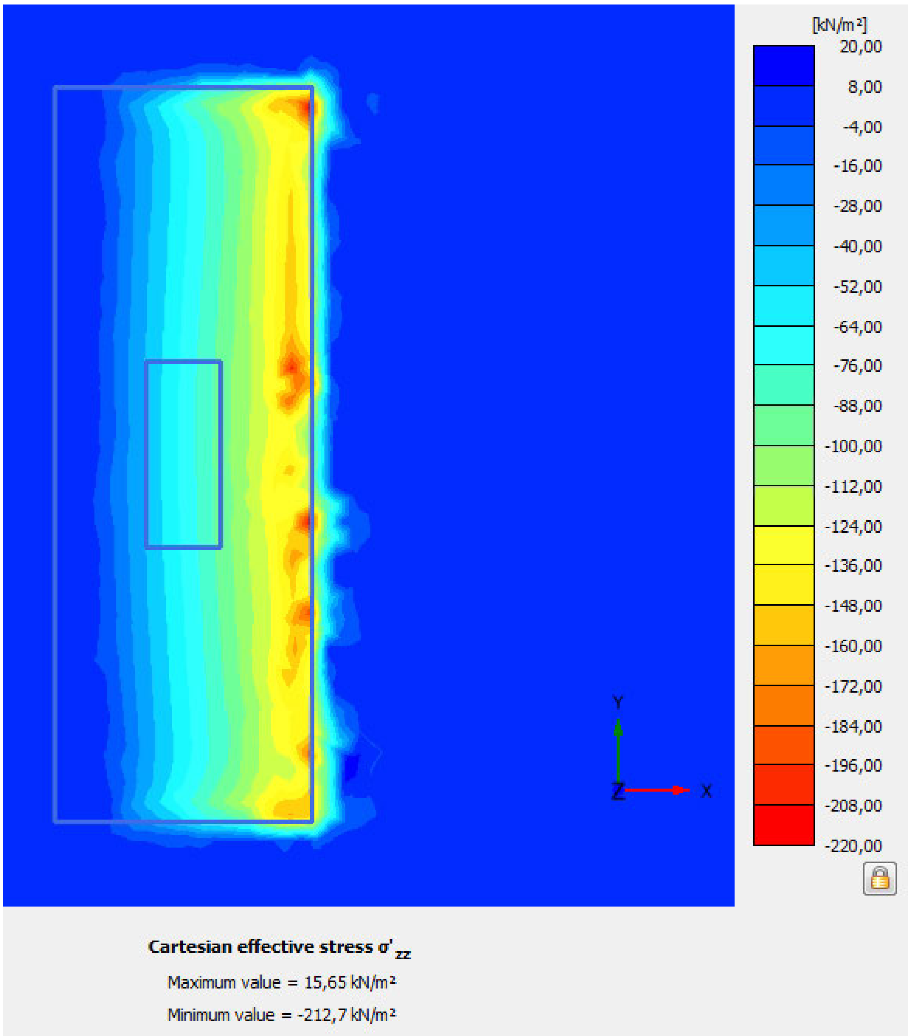


Figure 3 Computed vertical stress beneath the foundation with 50% of the lateral load applied

The finding of the 3D FE confirming the findings of the effective foundation area method.

4 Summary

3D FE assessment of Pier P91 of the Cairngorm Funicular Railway with simplified geometries has been modelled to verify the EC7 bearing capacity calculation approach using the effective area method.

The findings verify the effective area method of bearing capacity calculation of EC7 approach for simple foundation and confirms its approximate nature of the design calculation.

While the results of this assessment are shown to confirm the validity of the effective foundation area method used in the preliminary design assessment it is clear from the findings of the numerical assessment carried out on selected foundations at Piers (P91, P61, P46 and P18) that this simple bearing capacity design assessment approach results in conservative assessment of bearing capacity when the effects of sloping ground and bi-axial loading.

With the complexity introduced by the presence of sloping ground together with the effect of bi-axial loading numerical modelling which can capture the 3D effects is judged to provide a more accurate assessment of the behaviour of the shallow foundations. This is due to the inherent difficulties in determining the contribution of favourable lateral soil pressure and shear resistance from the surrounding fill when adopting the EC7 approach.

For this reason, in any future assessment, the need for foundation strengthening will be based on the results of the numerical modelling rather than the effective area method set out in Annex D of EC7.

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CONTENTS

1 Introduction

The preliminary design assessment of the shallow foundations for the Cairngorm Funicular Railway was undertaken using the effective foundation area method of calculating bearing capacity outlined in Annex D of the Eurocode 7 (EC7).

As the pad foundations supporting the viaduct piers are embedded in sloping ground, the assumed contribution from the backfill, especially the favourable lateral resistance, on the bearing capacity of the foundation is not easy to quantify. As a consequence favourable and unfavourable soil loads were assumed based on commonly adopted theories of soil mechanics.

The findings reported in the Appraisal Report Ref: A116993 RP01 V2 concluded that under out of service "Accidental" loading representative of high transverse

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shear and bending moments, the high value of the calculated eccentricity of reaction force led to either overstressing of the foundation or in extreme cases (i.e. at P91) overturning of the foundation prohibiting calculation of a bearing capacity.

As a consequence, further examination of the lateral resistance to foundation loading provided by the foundation backfill material has been carried out as part of further investigation of the need for foundation strengthening works. This assessment involved more complex numerical analyses where the uncertainties relating to the soil resistances provided by the sloping ground is minimised.

A 3D finite element (FE) assessment of the foundation behaviour under in service "Operational" and out of service "Accidental" and "Evacuation" loading has been undertaken. This technical note describes the findings of 3D FE modelling undertaken on four foundation types in differing ground conditions.

The results of this assessment confirm the adequacy of the foundations to resist bearing reactions under in service operational and out of service accidental and evacuation loading without need of foundation strengthening.

2 FE Modelling and Loadings

2.1 Description of FE model

Three foundation base types founded on each of the four characteristic ground types occurring at the site have been modelled using the Plaxis 3D finite element program.

The 3D analysis captures fully the complexity of soil restraint provided by the sloping backfill.

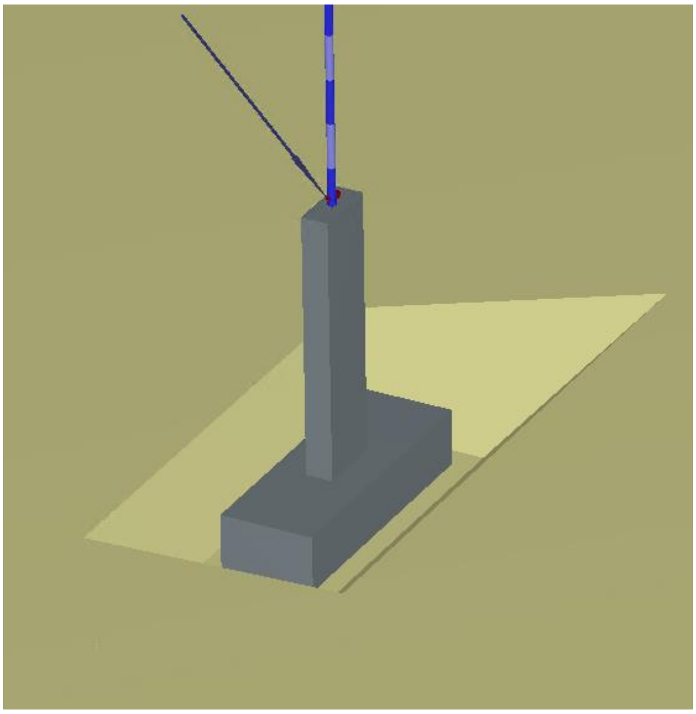


Figure 2-1 General Arrangement of Pier Foundation

A summary of the foundation base types and corresponding ground condition analysed is given in Table 2-1 below.

Table 2-1 Foundation and Ground Types Analysed

Foundation Base Type	Ground Conditions	Ground Inclination	Length (m)	Width (m)	Pier
Base Type 1	Type A	Level	4	2.1	P18
Base Type 6	Type B	10°	6	2.1	P46
Base Type 5	Type C	20°	5.6	2.1	P61
Base Type 6	Type D	20°	6	2.1	P91

The ground condition types identified and reported in TN-3-002 comprise the following.

- Type A- Glacial Deposits (5m thick) overlying weathered/decomposed granite bedrock.
- Type B- Alluvial Deposits (6m thick) overlying weathered/ decomposed granite bedrock.
- Type C- Head Deposits (3m thick) overlying weathered/decomposed granite bedrock.

- Type D- Weathered/decomposed rock (3m thick) overlying granite bedrock.

A linear elastic perfectly plastic Mohr-Coulomb constitutive soil model was adopted in the FE assessment.

The soils were modelled with the following characteristic parameters.

Table 2-2 Characteristic Soil/Rock Parameters Adopted

Soil Type	Unit Weight (kN/m³)	Young's Modulus (MPa)	Poisson's Ratio	Internal Angle of Friction (Deg)	Cohesion (kPa)
Foundation Backfill	18	20	0.2	32	1
Glacial Deposits	18	50	0.2	36	1
Alluvial Deposits	17	20	0.2	32	5
Head Deposits	18	50	0.2	38	1
Weathered Rock	18	100	0.2	42	1
Granite bedrock	26	2000	0.2	50	500

The piers and foundation base are modelled as a linear elastic material adopting a unit weight of 24.5kN/m³ and cracked concrete stiffness with Young's Modulus equal to 20GPa.

An example of a 3D mesh is shown below. The extent of the mesh is 50m by 50m by 50m.

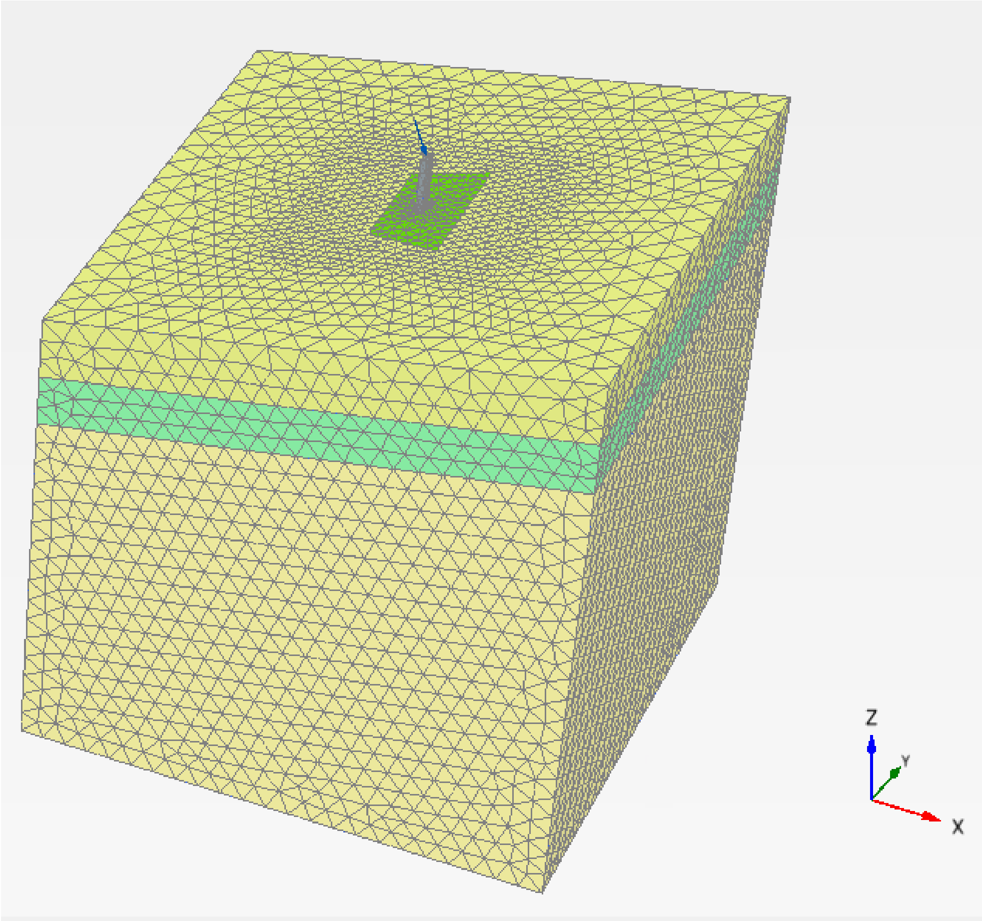


Figure 2-2 Overall Mesh

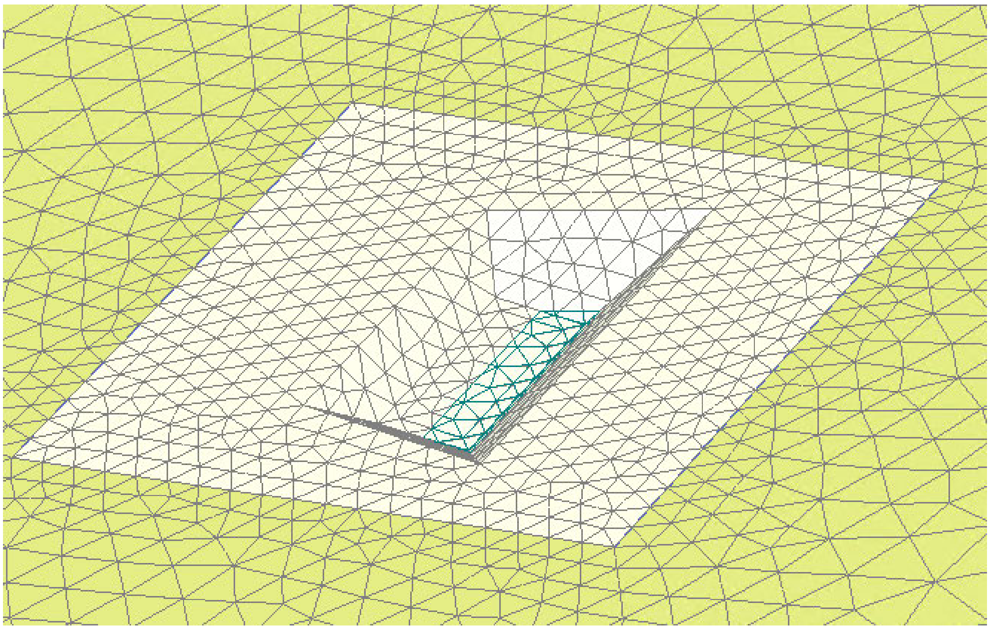


Figure 2-3 Plaxis 3D FE mesh (*detailed local element distribution* around excavation)

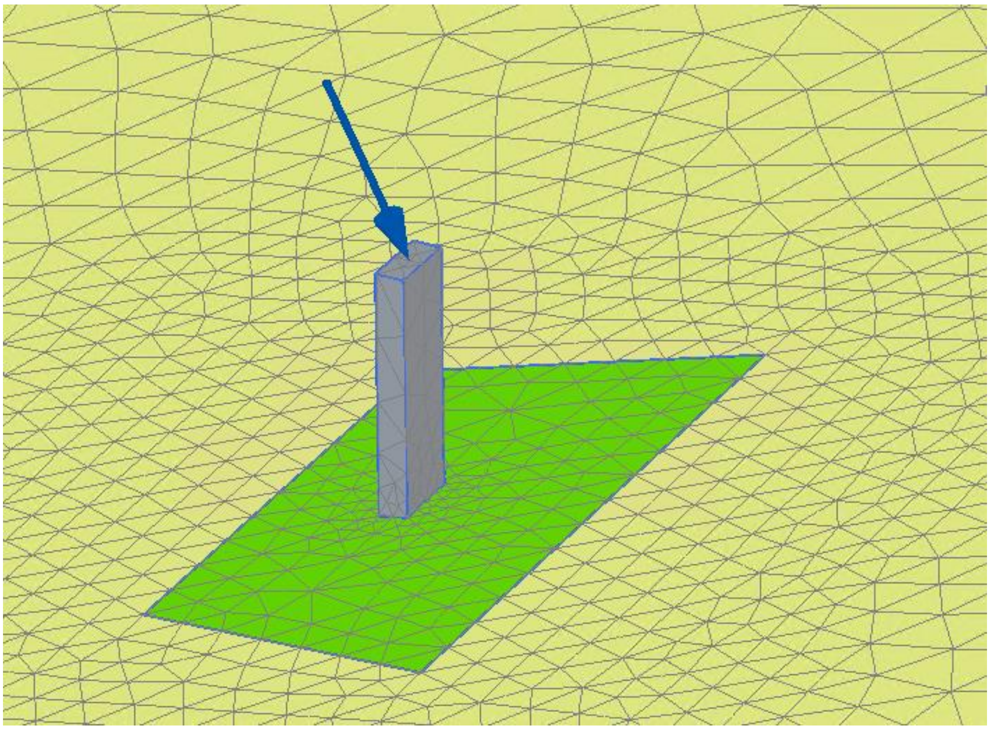


Figure 2-4 *Plaxis 3D FE mesh* (*detailed local element distribution* around foundation and soil backfill)

Groundwater levels have been set at a level equivalent to the underside of the foundation base for SLS loading and equal to 1m below ground surface for ULS loading.

2.2 Applied loadings

In each analysis loads is applied to the model at bearing level at a fixed distance above the foundation base.

The sign convention adopted in the analysis is as follows:

- x-positive (up slope on longitudinal axis)
- y- positive (cross slope on transverse axis)
- z- positive (vertical up)

Partial load and material factors have been applied in accordance with BS EN 1997-1:2015. The partial load and material factors applied in the assessment are listed in the following table:

Table 2-3 Summary of Partial factors

BS EN1997-1:2015	Partial Load Factors Applied to Actions		Partial Material Factors Applied to Soil Strength	
	Dead Load (G_K)	Live Load (Q_k)	Soil Friction	Soil Cohesion
DA1 C1	1.35	1.5	1.0	1.0
DA1 C2	1.0	1.3	1.25	1.25

Notes: *DA1 refers Design Approach 1, C1 and C2 refers to load Combination 1 and Combination 2 respectively*

The following SLS and ULS loads have been applied in each model.

Table 2-4 Pier P18 – Bearing height above foundation base (2.4m)

Pier	Load Case	Fz	Fx	Fy	Mx
P18	SLS	-468kN	41kN	128kN	-190kNm
	ULS Operational (DA1 C1)	-675kN	59kN	192kN	-285kNm
	ULS Operational (DA1 C2)	-554kN	49kN	166kN	-247kNm
	ULS Out of Operation "Accidental" (DA1 C1)	-513kN	46kN	363kN	-477kNm
	Out of Operation "Accidental" (DA1 C2)	-413kN	37kN	314kN	-413kNm
	Out of Operation "Evacuation" (DA1 C1)	-513kN	46kN	272kN	-354kNm
	Out of Operation "Evacuation" (DA1 C2)	-413kN	37kN	235kN	-307kNm

Table 2-5 Pier P46 – Bearing height above foundation base (7.15m)

Pier	Load Case	Fz	Fx	Fy	Mx
P46	SLS	-431kN	106kN	132kN	-187kNm
	ULS Operational (DA1 C1)	-622kN	153kN	198kN	-280kNm
	ULS Operational (DA1 C2)	-568kN	126kN	171kN	-243kNm
	ULS Out of Operation "Accidental" (DA1 C1)	-471kN	115kN	400kN	-503kNm
	Out of Operation "Accidental" (DA1 C2)	-380kN	93kN	347kN	-436kNm
	Out of Operation "Evacuation" (DA1 C1)	-471kN	115kN	278kN	-345kNm
	Out of Operation "Evacuation" (DA1 C2)	-380kN	93kN	240kN	-299kNm

Table 2-6 Pier P61 – Bearing height above foundation base (5.18m)

Pier	Load Case	Fz	Fx	Fy	Mx
P61	SLS	-413kN	147kN	88kN	-108kNm
	ULS Operational (DA1 C1)	-596kN	212kN	132kN	-162kNm
	ULS Operational (DA1 C2)	-490kN	174kN	114kN	-140kNm
	ULS Out of Operation "Accidental" (DA1 C1)	-452kN	161kN	407kN	-510kNm
	Out of Operation "Accidental" (DA1 C2)	-365kN	130kN	352kN	-442kNm
	Out of Operation "Evacuation" (DA1 C1)	-452kN	161kN	241kN	-303kNm
	Out of Operation "Evacuation" (DA1 C2)	-365kN	130kN	209kN	-263kNm

Table 2-7 Pier P91 – Bearing height above foundation base (7.15m)

Pier	Load Case	Fz	Fx	Fy	Mx
P91	SLS	-418kN	139kN	87kN	-109kNm
	ULS Operational (DA1 C1)	-603kN	200kN	131kN	-163kNm
	ULS Operational (DA1 C2)	-495kN	165kN	113kN	-142kNm
	ULS Out of Operation "Accidental" (DA1 C1)	-458kN	152kN	504kN	-639kNm
	Out of Operation "Accidental" (DA1 C2)	-370kN	123kN	439kN	-554kNm
	Out of Operation "Evacuation" (DA1 C1)	-458kN	152kN	240kN	-304kNm
	Out of Operation "Evacuation" (DA1 C2)	-370kN	123kN	208kN	-264kNm

3 Discussion of Analysis Results

3.1 General analyses

The results of the FE analyses are described herein.

The foundations analysed are considered acceptable in terms of the development of adequate bearing capacity on the provision that the analysis models are stable under applied ULS loading with full numerical convergence taking place.

3.1.1 Foundation bearing resistance

All of the foundations/ground types assessed can develop adequate bearing resistance to accommodate Operational and Out of Operation "Accidental" and "Evacuation" loading.

The maximum applied bearing pressures under ULS loading with groundwater level set 1m below ground surface are listed below:

Table 3-1 Maximum Applied ULS Bearing Pressure (DA1C1)

Pier	Base Type	Ground Type	ULS (Operation)	ULS (Out of Operation) "Accidental"	ULS (Out of Operation) "Evacuation"
P18	Base Type 1	Ground Type 1	180kPa	195kPa	180kPa
P46	Base Type 6	Ground Type 2	290kPa	350kPa	290kPa
P61	Base Type 5	Ground Type 3	335kPa	465kPa	345kPa
P91	Base Type 6	Ground Type 4	310kPa	740kPa	320kPa

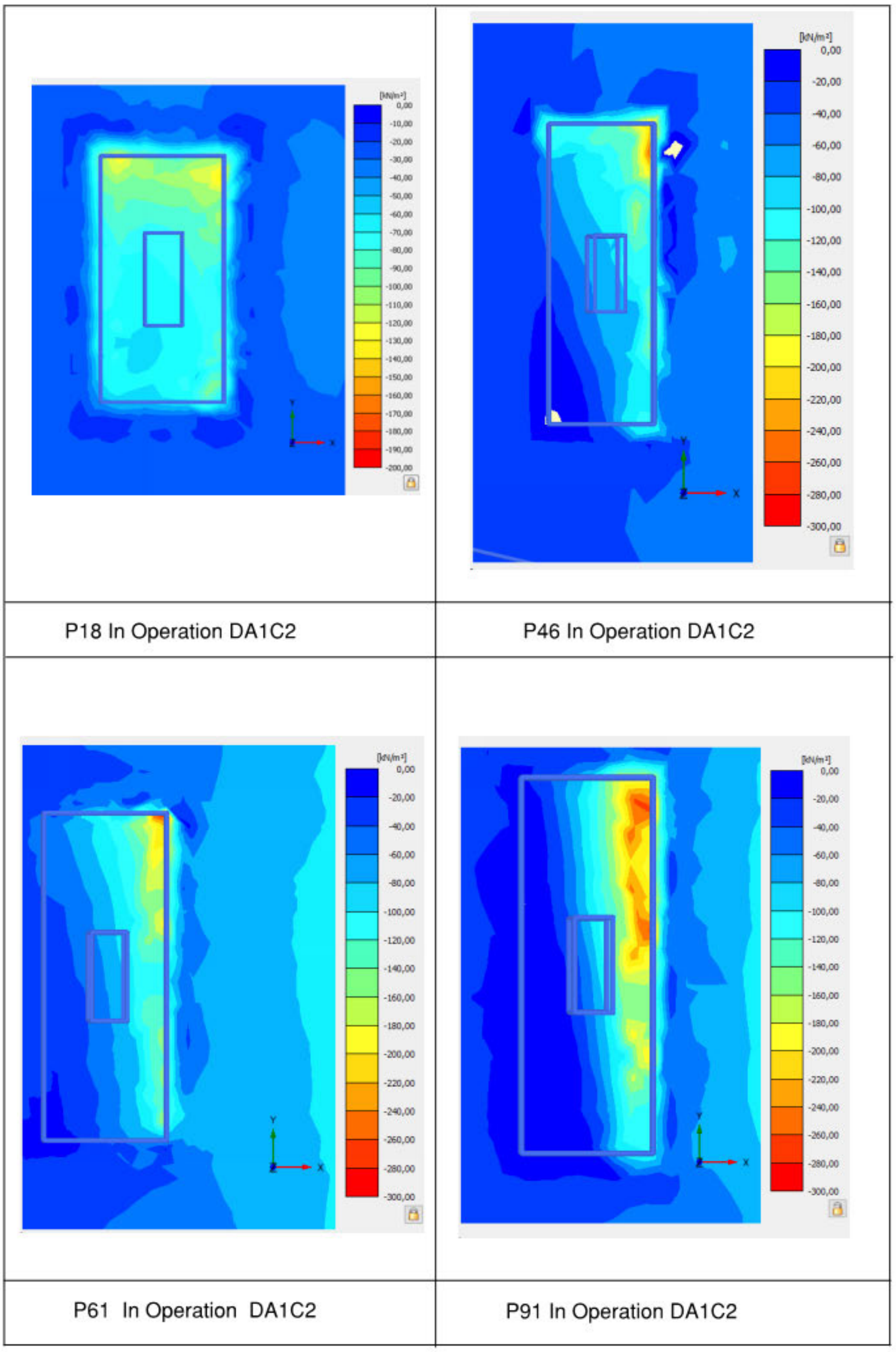
Table 3-2 Maximum Applied ULS Bearing Pressure (DA1C2)

Pier	Base Type	Ground Type	ULS (Operation)	ULS (Out of Operation) "Accidental"	ULS (Out of Operation) "Evacuation"
P18	Base Type 1	Ground Type 1	140kPa	155kPa	140kPa
P46	Base Type 6	Ground Type 2	250kPa	305kPa	255kPa
P61	Base Type 5	Ground Type 3	275kPa	360kPa	285kPa
P91	Base Type 6	Ground Type 4	265kPa	665kPa	275kPa

Due to high levels of transverse shear and moments, in particular for the out of operation "accidental" load case, base reactions are concentrated in the upslope/left corner of the foundation as illustrated in Figure 3-2 to Figure 3-3 below.

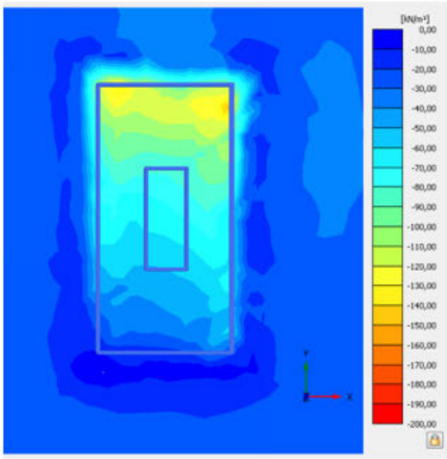
It is worth noting that the reaction of Pier P18 is different to the bases of the other three piers due to much smaller lateral load applied on the foundation

Combination 2 (C2) loading where partial factors are applied to material soil strengths is the governing the case with respect to the geotechnical capacity of the foundation. For this reason contour plots of foundation base pressure are presented for DA1 C2 load cases only.

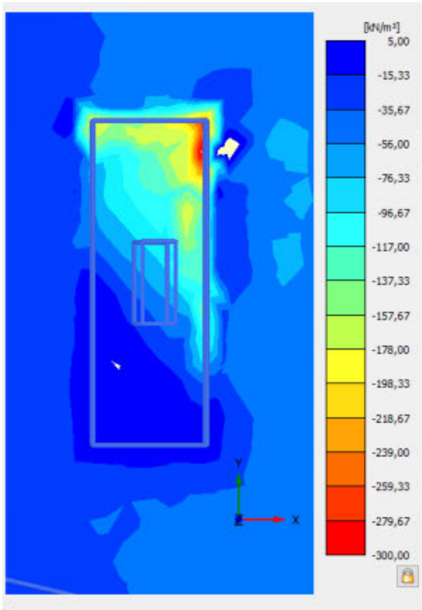


Note: upslope to the right of the above contours plots

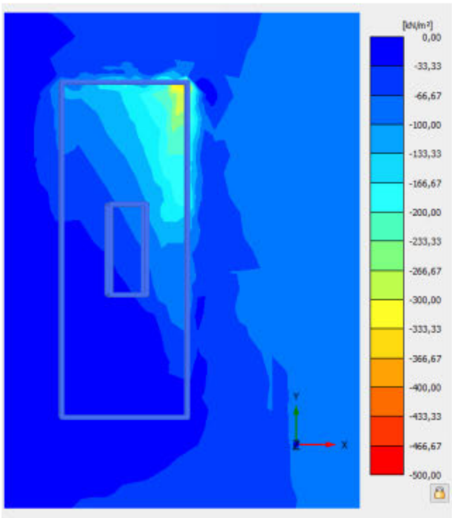
Figure 3-1 Contours of applied bearing pressure for "In Operation" loading (DA1C2)



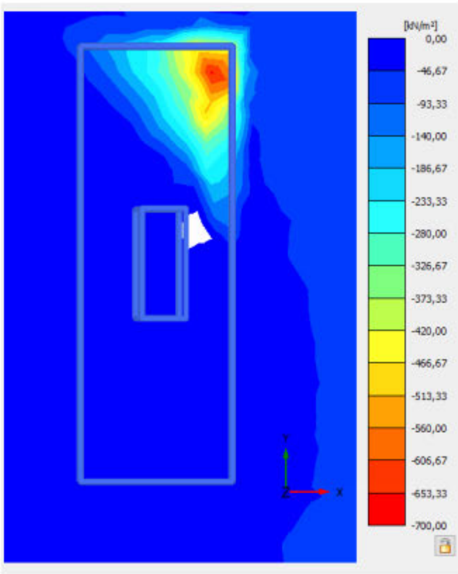
P18 Out of Operation "Accidental" DA1C2



P46 Out of Operation "Accidental" DA1C2



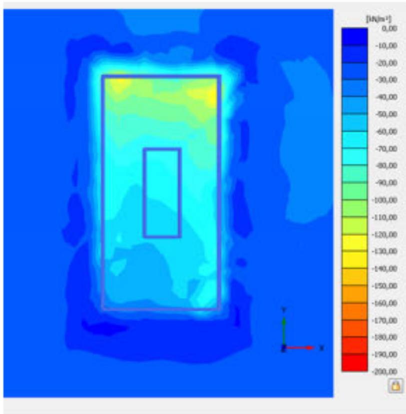
P61 Out of Operation "Accidental" DA1C2



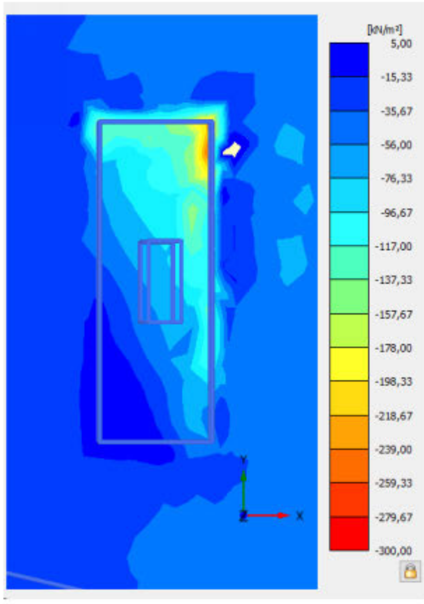
P91 Out of Operation "Accidental" DA1C2

Note: upslope to the right of the above contours plots

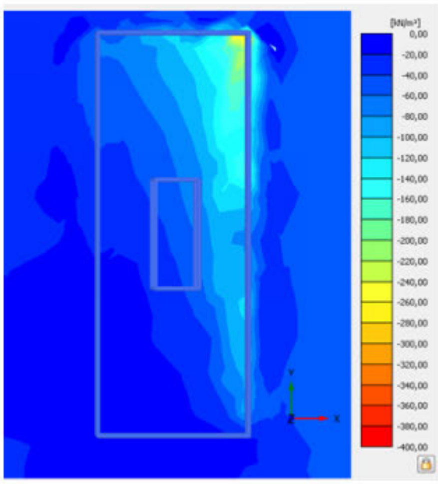
Figure 3-2 Contours of applied bearing pressure for Out of Operation "Accidental" loading (DA1C2)



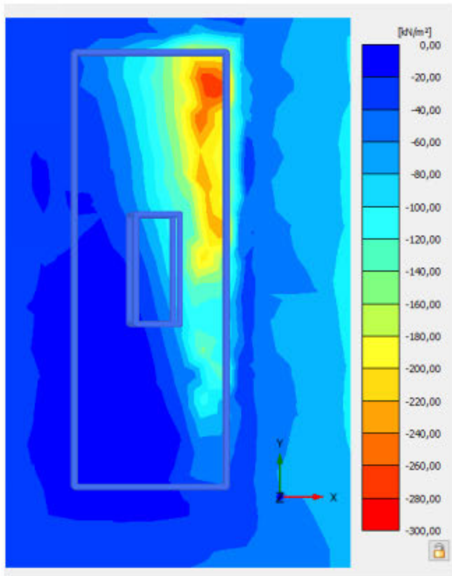
P18 Out of Operation "Evacuation" DA1C2



P46 Out of Operation "Evacuation" DA1C2



P61 Out of Operation "Evacuation" DA1C2



P91 Out of Operation "Evacuation" DA1C2

Figure 3-3 Contours of applied bearing pressure for Out of Operation "Evacuation" loading (DA1C2)

The magnitude of the foundation uplift pressures where present are small in the range 0kPa to 10kPa.

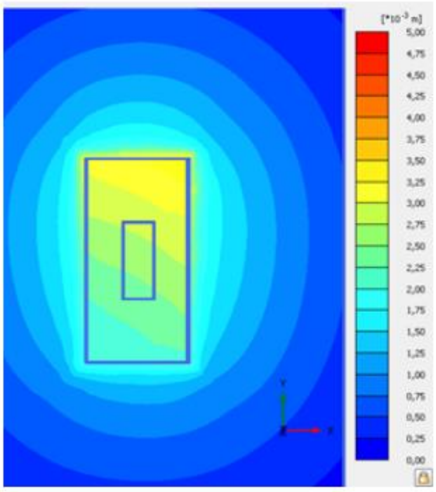
3.1.1 Base settlement

The range of foundation base settlement calculated for each base type/ground type combination under SLS "operational" loading are listed below.

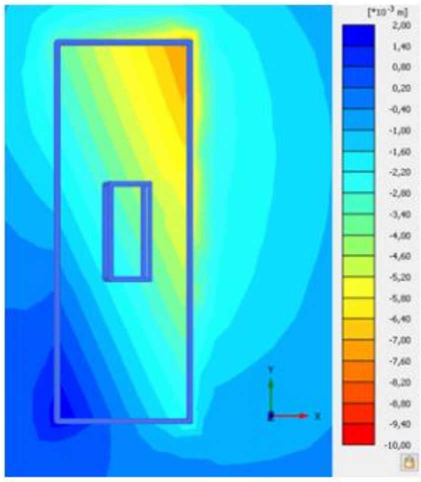
Table 3-3 Summary of base deflections.

Pier	Base Type	Ground Type	SLS (Dead Only)	SLS (Dead Plus Live)
			Delta Z (mm)	Delta Z (mm)
P18	Base Type 1	Ground Type 1	-0.5 to -1.0	-1.5 to -3.0
P46	Base Type 6	Ground Type 2	-0.5 to -1.5	-2.0 to -8.0
P61	Base Type 5	Ground Type 3	-0.1 to -0.5	-0.5 to -2.0
P91	Base Type 6	Ground Type 4	-0.1 to -0.5	-1.5 to -2.5

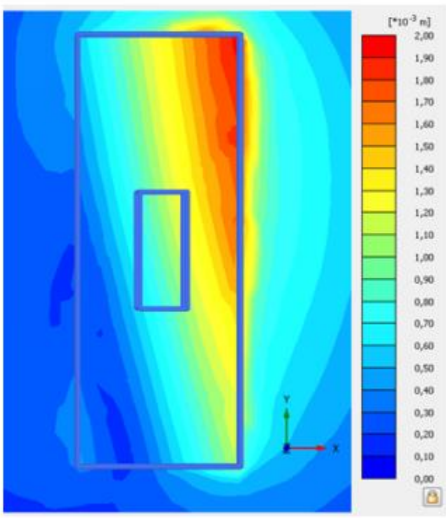
Contour plots of vertical base deflection are presented Figure 3-4 below.



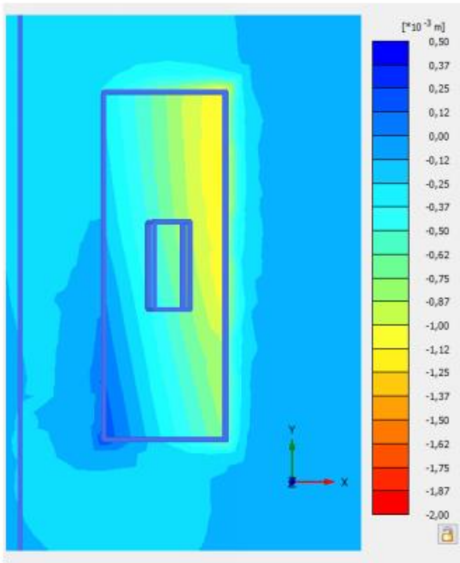
P18 In Operation SLS Delta Z (mm)



P46 In Operation SLS Delta Z (mm)



P61 In Operation SLS Delta Z (mm)



P91 In Operation SLS Delta Z (mm)

Figure 3-4 Contour plots of vertical base deflection under SLS in operation loading

The analysis performed is limited to application of static load and does not considered the effects of cyclic loading on settlement performance.

Over the operational lifetime of the asset, under successive cycles of live loading due to the predominantly granular nature of the foundation backfill material the foundation backfill will have undergone compaction leading to an increase in the stiffness properties. In such circumstances very little additional settlement would be expected following initial settlement resulting from the application of dead load.

However, depending on the fines (clay) content of the foundation backfill at each individual foundation, the stiffness properties may have decreased. At locations where the fines content of the foundation backfill material is high, or at locations where predominantly granular backfill material has been contaminated with cohesive and organic soils, successive cycles of live loading has the potential to lead to degradation in the strength of the foundation backfill material. The effect of this would be an increase in strain under applied loading and a consequential reduction in foundation stiffness leading to additional settlement over time.

3.1.2 Pier head deflection

Pier head deflections are calculated at the centre of the pier adopting an upper bound long term concrete stiffness of 20GPa.

The following sign convention is adopted.

- Delta X- Longitudinal displacement (+ive up slope)
- Delta Y- Transverse Displacement (+ive cross slope left facing upslope)
- Delta Z- Vertical Displacement (-ive downwards)

Table 3-4 Summary of Pier head deflections

Pier	Base Type	Ground Type	SLS (Dead Only)			SLS (Dead + Live)		
			Delta X (mm)	Delta Y (mm)	Delta Z (mm)	Delta X (mm)	Delta Y (mm)	Delta Z (mm)
P18	Base Type 1	Ground Type 1	0.15	0	-0.5	0.5	1	-1.5
P46	Base Type 6	Ground Type 2	9	0	-0.5	30	11	-3
P61	Base Type 5	Ground Type 3	4	0	-0.5	10	2	-1
P91	Base Type 6	Ground Type 4	9	0	-0.5	21	3	-0.5

3.2 Sensitivity analyses

Sensitivity analysis has been carried out on selected models to determine the effect of varying the stiffness properties of the concrete and raising groundwater to ground surface in recognition of the presence of natural groundwater springs located upslope of pier 41 and pier 72.

The results of the sensitivity analyses are presented herein.

3.2.1 Foundation bearing resistance

A check on the sensitivity of the model to change in groundwater level has been carried out for P46 which is founded on the Type B (alluvial deposits) which are considered most likely to be subject to fluctuation in groundwater level.

For the purpose of the check, under ULS loading groundwater level has been set to ground surface.

The calculated foundation bearing pressures are summarised in Table 3-5 below.

Table 3-5

Pier	Base Type	EC7: Load Combination	ULS (Operation)	ULS (Out of Operation) "Accidental"	ULS (Out of Operation) "Evacuation"
P46	Base Type 6	DA1C1	290kPa$^{\Psi}$ 325kPa*	350kPa$^{\Psi}$ 465kPa*	290kPa$^{\Psi}$ 335kPa*
P46	Base Type 6	DA1C2	250kPa$^{\Psi}$ 285kPa*	305kPa$^{\Psi}$ 440kPa*	250kPa$^{\Psi}$ 290kPa*

$^{\Psi}$ Groundwater set 1m below ground surface

* Groundwater set at ground surface

Under the worst credible groundwater pressures foundations bearing on the alluvial deposits are shown to continue to develop adequate bearing resistance to accommodate Operational and Out of Operation "Accidental" and "Evacuation" loading.

3.2.2 Pier head deflection

A check on the sensitivity of the model to change in concrete stiffness has been carried out for pier P46. For the purpose of the check, under SLS loading, groundwater level has been set equal to the underside of the foundation base. The calculated pier head deflections for pier P46 are presented in Table 3-6 below.

Table 3-6

Pier	Concrete Stiffness	SLS (Dead Plus Live)			SLS (Dead Only)		
		Delta X (mm)	Delta Y (mm)	Delta Z (mm)	Delta X (mm)	Delta Y (mm)	Delta Z (mm)
P46	Upper Bound Short Term E=30GPa	25	10	2	7	0	1
P46	Lower Bound Long Term E=15GPa	35	12	5	11	0	2

Pier head deflections are sensitive to deterioration in concrete stiffness over time. Adopting lower bound long term stiffness properties equal to 15GPa results in pier head deflection up to 35mm.

The maximum predicted horizontal displacement (in the X-direction) for piers P61 and P91 are 12mm and 25mm respectively.

3.3 Assessment of the effect of pier strengthening on foundation behaviour

The structural appraisal of the asset confirmed that high piers, P46, P61 and P91 are overstressed under in service operational and out of service accidental loading which is contributing to pier head rotation and bearing misalignment.

The effect on proposals to strengthen the piers through provision of a concrete jacket encasing the original pier have been investigated for piers P46 and P91.

P46 and P91 have been investigated because they represent the tallest piers (at approx. 5.9m) founded on the weakest (Type 2- Alluvium) and strongest (Type 4- Weathered Granite) ground types.

Figure 3-5 illustrates the geometry of the concrete jacket.

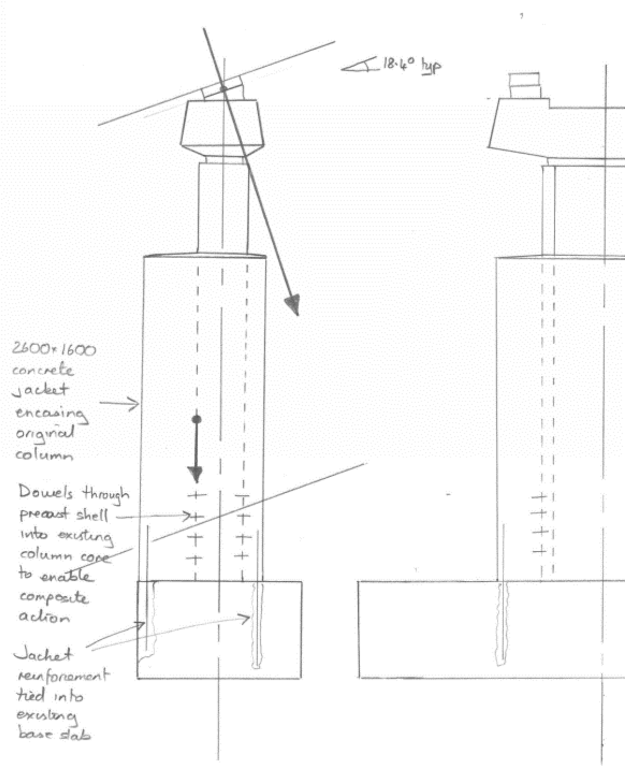


Figure 3-5 Pier Strengthening Proposals- Concrete Jacketing

The concrete jacket has been modelled as a linear elastic shell with dimensions of 1.6m (B) by 2.6m (L). The jacket encases the existing pier to a height set 2m below the level of the pier head.

The results of the analysis are summarised herein.

The maximum applied bearing pressures with ULS groundwater level set at ground surface are summarised in Table 3-7 below.

Table 3-7 Summary of maximum bearing pressures (Jacketed Piers)

Pier	Base Type	Ground Type	EC7: Load Combination	ULS (Operation)	ULS (Out of Operation) "Accidental"	ULS (Out of Operation) "Evacuation"
P46	Base Type 6	Ground Type 2	DA1C1	255kPa	310kPa	260kPa
			DA1C2	215kPa	275kPa	220kPa
P91	Base Type 6	Ground Type 4	DA1C1	325kPa	630kPa	350kPa
			DA1C2	265kPa	510kPa	265kPa

The effect of jacketing on pier head deflection and foundation settlement under in Operation SLS loads (Dead + Live) is shown in Table 3-8 below.

Table 3-8 Pier Head Deflection (Jacketed Piers)

Pier	Base Type	Ground Type	Pier Head Deflection			Foundation Base Settlement (mm)
			Delta X (mm)	Delta Y (mm)	Delta Z (mm)	
P46	Base Type 6	Ground Type 2	18mm	9mm	3mm	2-8mm
P91	Base Type 6	Ground Type 4	6mm	1mm	1mm	0-1mm

Pier head deflections are presented for Lower Bound (LB) long term concrete stiffness equal to E=15GPa.

The range in foundation base settlement reflects the magnitude of settlement on the leading (Upslope) and trailing (Downslope) edge of the foundation.

Contour plots of foundation base pressure are presented in Figure 3-6 to Figure 3-8 below.

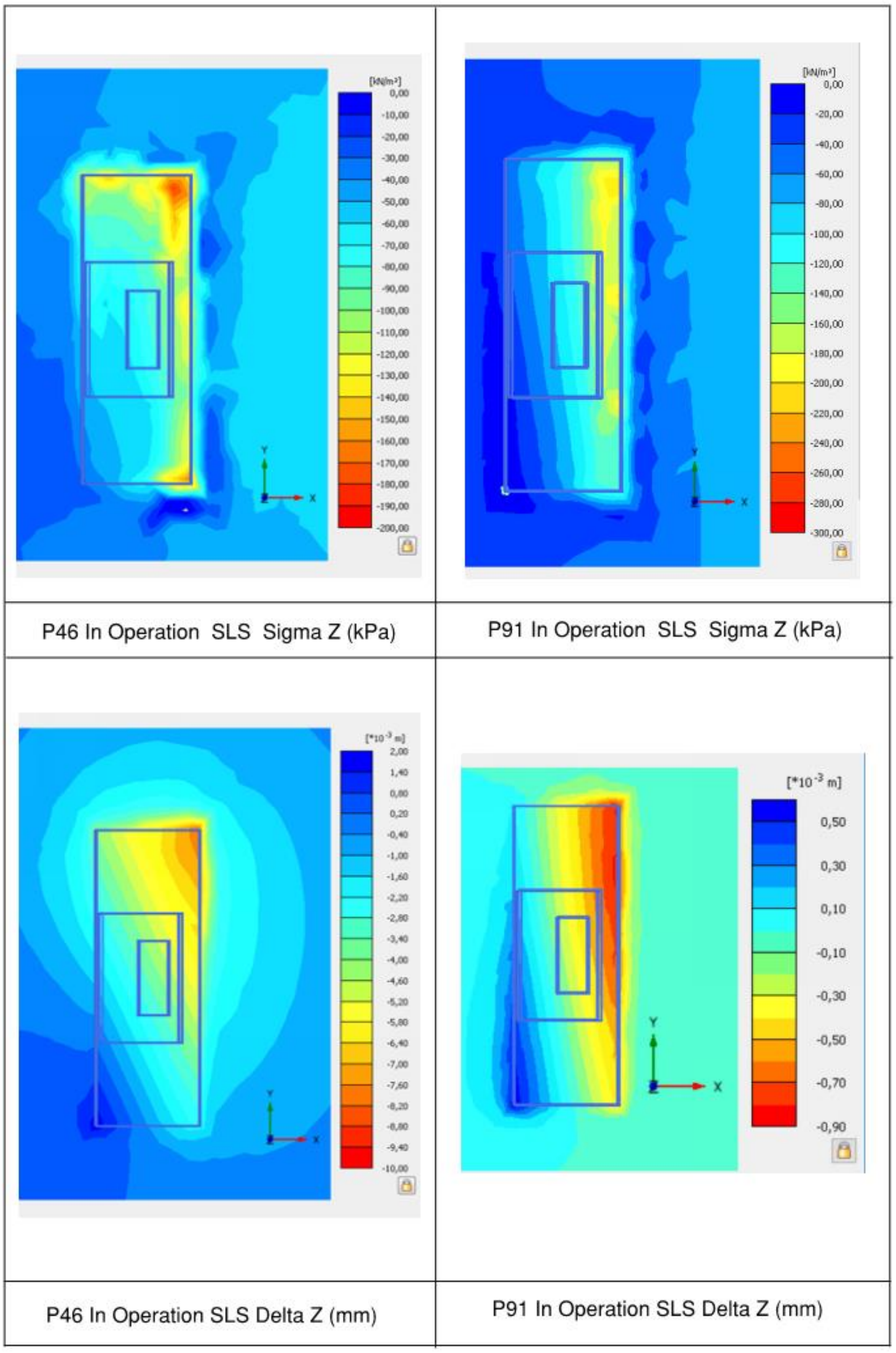


Figure 3-6 Contour Plots P46 & P91 Vertical Bearing Pressure and Settlement under SLS Loading

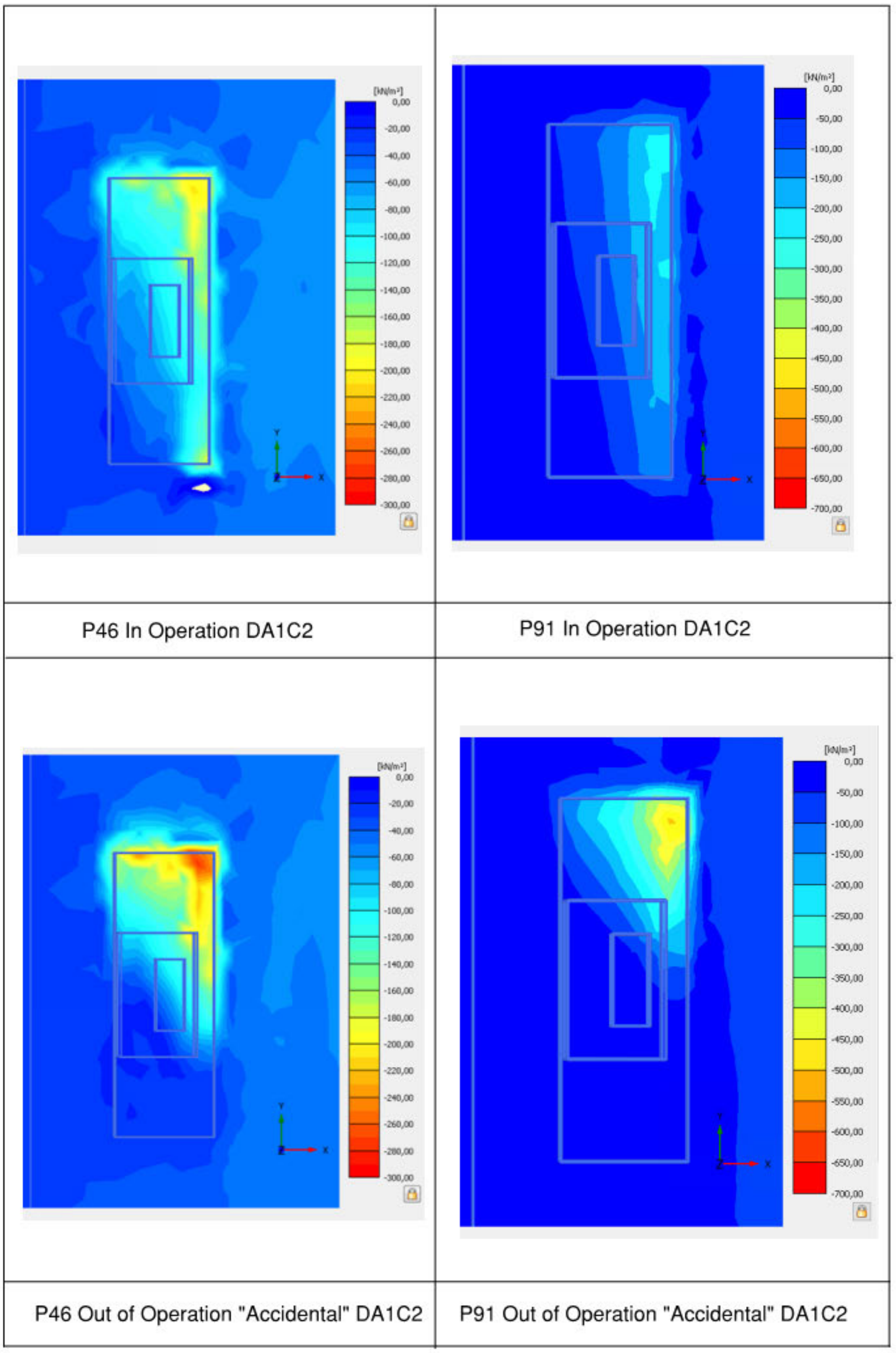
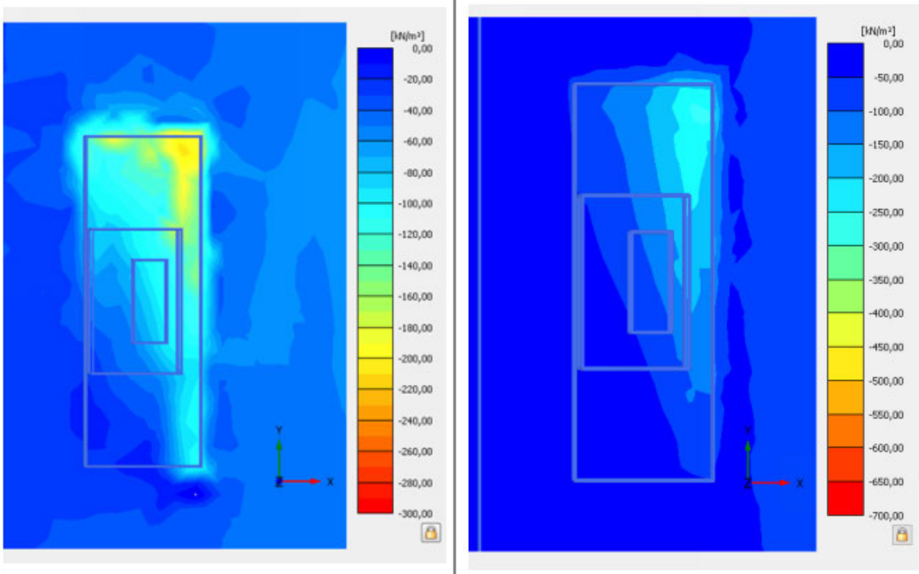


Figure 3-7 Contour Plots P46 & P91 Vertical Bearing Pressure under ULS Loading



P46 Out of Operation "Evacuation" DA1C2	P91 Out of Operation "Evacuation" DA1C2

Figure 3-8 Contour Plots P46 & P91 Vertical Bearing Pressure under ULS Loading (Continued)

The effects of pier strengthening through the provision of concrete jacketing is shown through observation of full numerical convergence of the model to have no detrimental impact on the foundations ability to resist the applied bearing pressure.

4 Summary & Conclusions

The results of this assessment confirms the adequacy of the foundations to resist bearing reactions under in service operational and out of service accidental and evacuation loading without the need for foundation strengthening.

The magnitude of calculated base settlement and pier head deflections provides an indication of the rotation of the piers under the action of dead and live load. However, the calculated values of pier head deflection are significantly less than the measured amount of bearing misalignment as shown in Table 4-1 below.

Table 4-1 Comparison of measured bearing offset with calculated pier head displacement

Pier	Measured Bearing Misalignment (mm)	Calculated Pier Head Deflection (mm)
P46	68mm	30mm
P61	112mm	10mm
P91	116mm	21mm

The difference may in part be due to construction intolerances and exacerbated by softening of the foundation backfill materials with high fines content under successive cycles of bi-directional live loading and seasonal freeze thawing of the soil.

The analysis presented is limited to the application of static loads and takes no account of hysteresis of strain softening/hardening under successive cycles of bi directional loading or seasonal freezing/thawing of the soil. There remains the possibility that at isolated base positions anomalies in the foundation backfill material are present such as peat and clay inclusions which coupled with the effect of solifluction (soil movement) under successive cycles of freezing and thawing may impact on foundation behaviour.

The structural assessment confirms the high piers, P46, P61 and P91 to be overstressed with respect to resisting bending moment under in service operational and out of service accidental loading.

Proposals to strengthen the piers through provision of a concrete jacketing is shown to have no detrimental impact on the foundations ability to resist the applied bearing pressure. The effect of jacketing piers is show to redistribute stresses acting in the base and reduce the concentration of bearing stress acting on the ground which is seen as beneficial to the performance of the foundations.

In summary, on the basis of the available ground investigation data, applied loading and recognising the limitation of the analyses carried out, the foundation bases are considered adequate to resist the bearing stresses under in service operational and out of service accidental and evacuation loading without need of foundation strengthening.

Appendix B NDT analysis of shear link spacing

HIGHLANDS AND ISLANDS ENTERPRISE

CAIRNGORM FUNICULAR RAILWAY

SHEAR LINK SPACING IN RAIL SUPPORT BEAMS

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London
EC3A 7JB

TEL +44 207 9407 600
WWW cowi.com

CONTENTS

1 Background / Summary

The available design drawings for the Cairngorm Funicular Railway show inconsistent shear link details at the ends of the rail support beams and across the in-situ joint-to-precast beam interfaces. COWI recommended that non-destructive investigations be carried out to determine the shear link spacing in these regions (see A116993-SP02). This work was carried out by Henderson Thomas Associates (HTA). The results are described in the HTA report L-1729-2018 dated 29th October 2018. (Additional investigation of longitudinal reinforcement not discussed here was also conducted by HTA.)

The initial survey works specified by COWI were focussed on the beam ends and only requested a single beam be scanned along the entire span. However, the HTA results showed inconsistencies in that span between the as-built condition and the design drawings. Furthermore, COWI's structural appraisal of the Cairngorm Funicular Railway (see COWI Report A116993-RP01 v2) identified

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A116993	TN-03-007

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V1	13-03-2019	For Information	[redacted]	[redacted]	[redacted]

deficiencies in the shear capacity of the rail support beams at the point where the shear link spacing transitions from 100mm to 200mm. As the original HTA survey only showed this information for a single beam, COWI recommended that further investigations be carried out to determine the shear link spacing in several spans with various beam types (see A116993-SP04). HTA again carried out these works, with the results described in their report L-1729-2018-Report 2.

This document discusses the justification of the shear link spacing details assumed by COWI in their structural appraisal.

2 Shear link spacing assumed in structural appraisal

The shear link spacing assumed by COWI for the structural appraisal is also discussed in Section 4.2 of appraisal report A116993-RP01 v2. Refer to that report for further information.

2.1 Available information

2.1.1 Construction photos

Various construction photos have consistently shown 5 No. links protruding from the precast beam ends into the in-situ joint region. The HTA survey discussed below typically only identified 4 No. links in this area, but this is believed to be due to the presence of the diaphragm obscuring the first link.

2.1.2 Design drawings

Drawing CA150/2/88 shows 100mm spacing near the beam ends and across the in-situ joints. A note on Drawing CA150/2/39 states that this link spacing is reduced to75mm at piers 51, 52, 54 and 56. Various drawings show 200mm spacing for the beam cross-sections and throughout the span.

2.1.3 HTA first survey (Report L-1729-2018)

The initial survey identified the shear link spacing for approximately 1200mm on either side of the pier centre for 23 No. beams. An indicative sketch of the information provided by HTA is shown in Figure 1. The beams scanned covered all "types", i.e. Type 1, Type 2, and Type 3 (see dwg CA150/2/49). The average shear link spacing was close to 100mm for all beams, but with poor consistency in the spacing between individual pairs of links (overall average spacing was 106.9mm and standard deviation was 28.4mm). Beams noted on the design drawings as having 75mm spacing did not have noticeably different spacing in the survey. In all cases, no transition to a wider shear link spacing was identified in the first ~1200mm from the pier centres.

The right hand beam at pier 9 (Type 1) was scanned all the way to midspan on both the uphill and downhill side of the pier. It was evident that, at approximately 2100mm from the pier centre, the link spacing transitioned to roughly 200mm. At roughly 5100mm from the pier centre, the link spacing again transitioned, this time to 300mm centres.

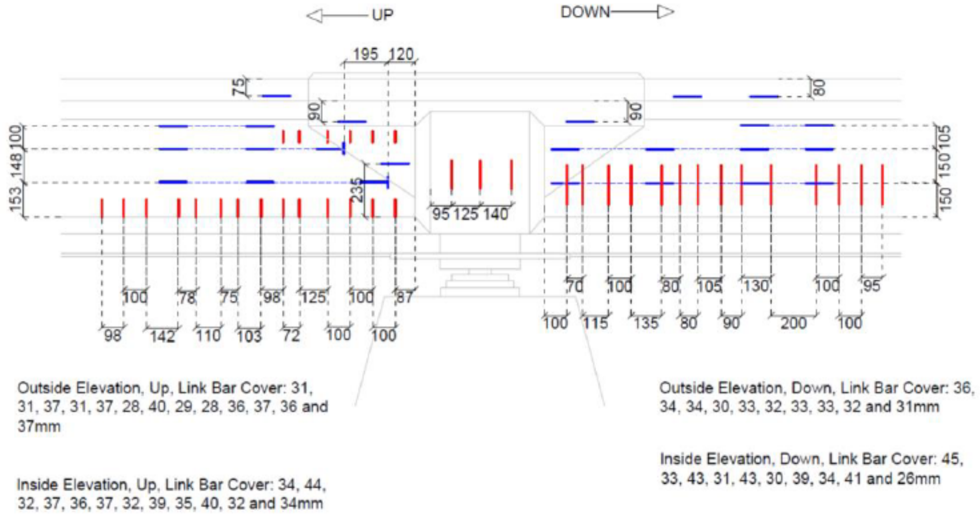

Figure 1 Indicative sketch of shear link spacing information provided by HTA

2.2 Assumed spacing and effect on shear deficiencies

The link spacing assumed in the structural appraisal, illustrated in Figure 2, was based on a combination of the information previously discussed. The shear capacity at the point which the link spacing fully transitioned to 200mm centres was found to be the cause of the majority of shear deficiencies. However, the location of this transition point (2100mm from the pier centre) is not shown in the design drawings and so the assumption was based on the scan of a single beam only. Further investigations were deemed necessary to validate this assumption.

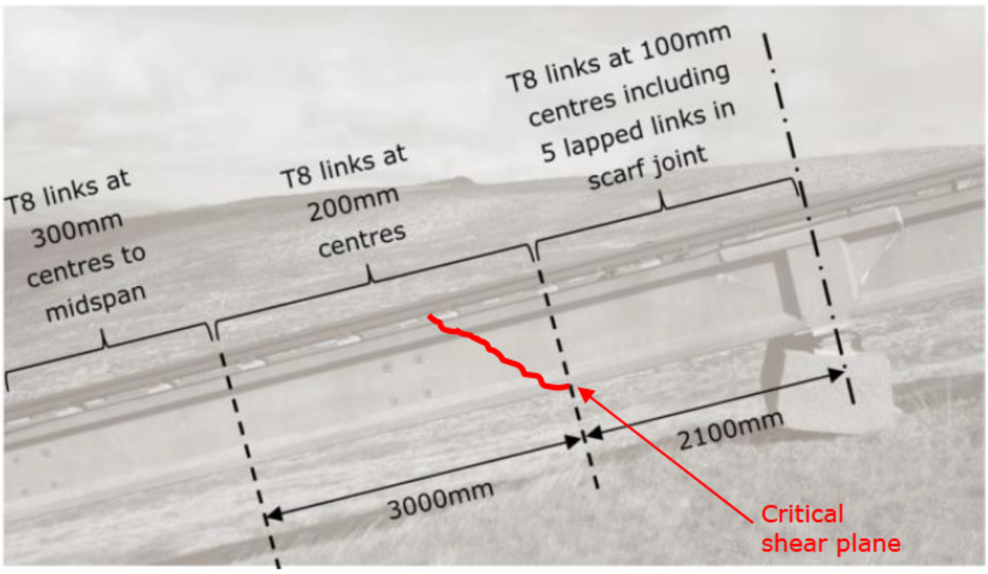

Figure 2 Link spacing assumed in structural appraisal

3 Second HTA survey and data analysis

The second HTA survey scanned the full length of 8 No. beams covering all beam types, i.e. Type 1, Type 2, and Type 3. Total data on the transition points between link spacings were now available for 18 No. beam ends (2 ends x 8 No. beams from second survey + 2 beam ends from first survey). This data was processed to evaluate the validity of the initial assumptions. Note that there was no noticeable difference in link spacing patterns for the various beam types, so all data was processed together.

The progression of link spacing for the 18 No. beam ends is illustrated in Figure 3. There is considerable scatter, but the assumed spacing progression (shown in red) is reasonably representative of the raw data (shown as the multicoloured dots).

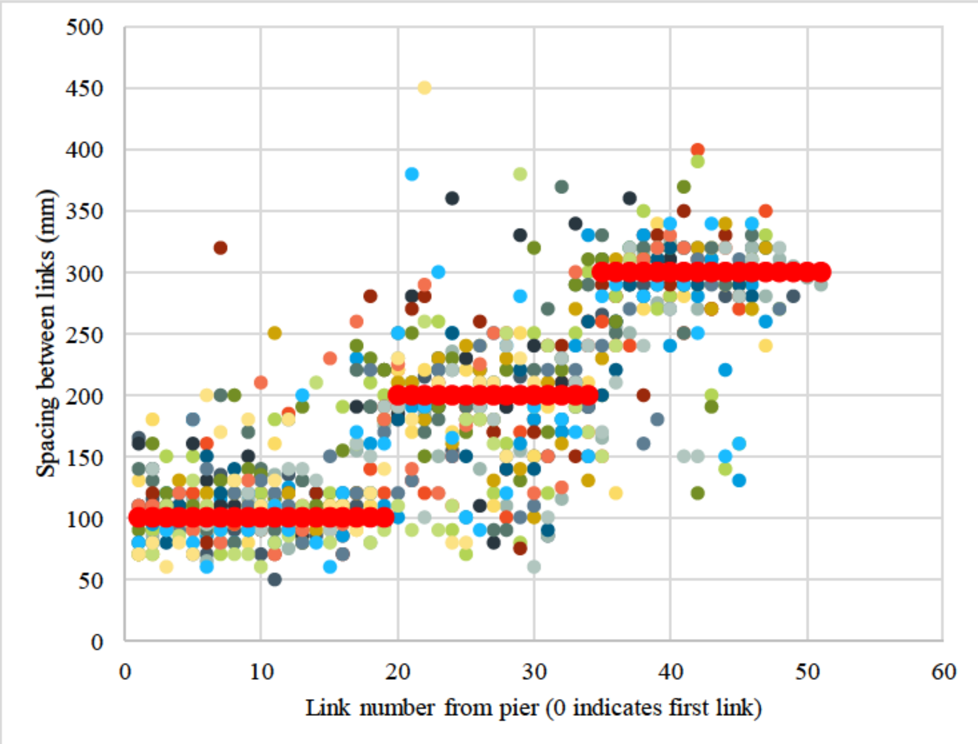

Figure 3 Progression of link spacing from pier centres (red lines are the assumed profile, multicoloured dots are the raw data)

As the link contribution to shear capacity in BD 44 is based on a 45° shear plane, the data was further processed to identify the variation in the number of links crossing such a plane along the lengths of the beams. The assumed profile is compared against (i) the raw data for individual beams (Figure 4), and (ii) the average of the raw data for all beams (Figure 5).

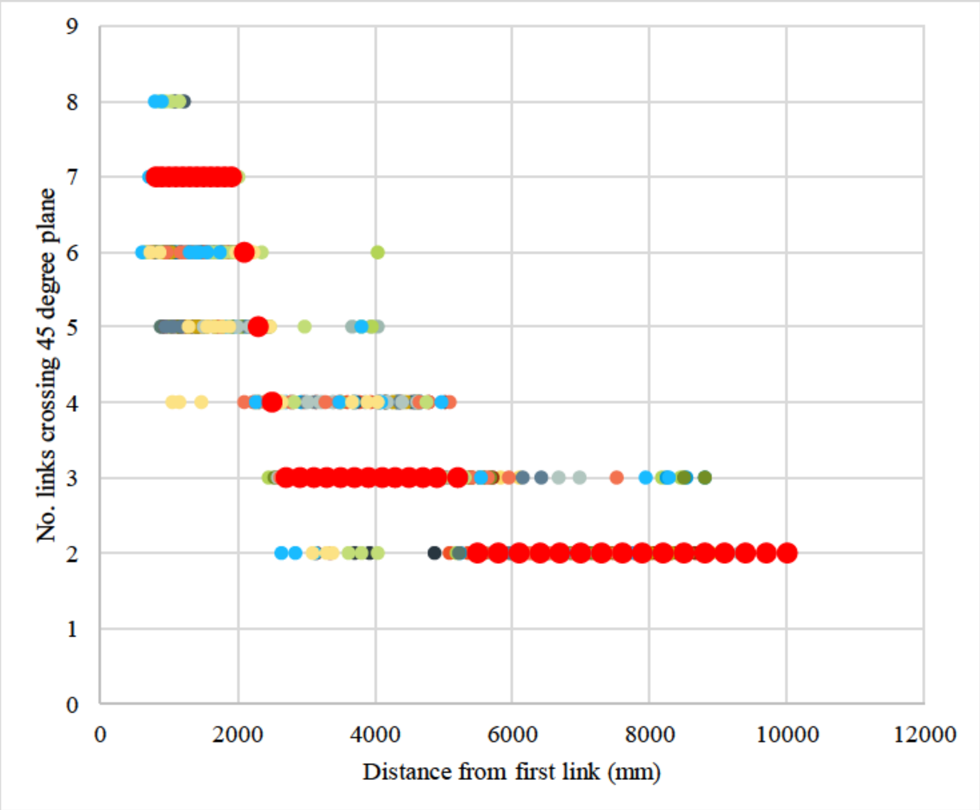

Figure 4 *Progression of the number of links crossing a 45° shear plane (assumed profile shown as red dots, raw data shown as multicoloured dots)*

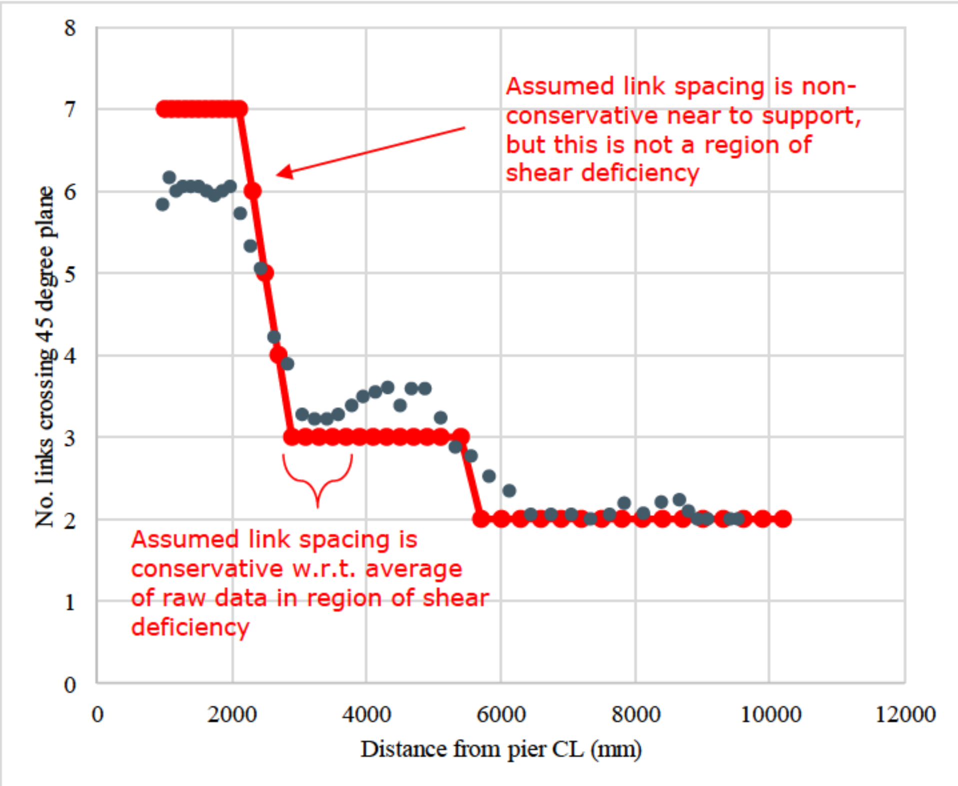

Figure 5 *Progression of the number of links crossing a 45° shear plane (assumed profile shown as red line, average of raw data shown as dark grey dots)*

The assumed link spacing is slightly non-conservative (with respect to the average of the raw data) in the 100mm spacing region, as the actual link average link spacing is approximately 110mm. However, this is not of significant concern as no shear deficiencies were identified in this area.

The assumed link spacing is conservative (with respect to the average of the raw data) in the critical area immediately after transition to 200mm link spacing (i.e. the plane shown in Figure 2).

4 Recommendation

It is recommended that the link spacing profile assumed in the structural appraisal (see Figure 2) can continue to be taken as an acceptable approximation of the as-built link spacing for the following reasons:

- In the critical region of the spans (after the transition to 200mm spacing), the model tends to give a slightly conservative number of links crossing a given 45° shear plane.

- The model is slightly non-conservative in the 100mm spacing region. However, any extra effort to re-calculate shear capacity in this region is considered unnecessary as shear enhancement near the support prevents this region from governing shear utilisations. By inspection, a relatively minor increase in shear link spacing would not affect that finding.

- The model is non-conservative when compared with data from certain individual beams, rather than averages (see Figure 3 and Figure 4). However, it would be over-conservative to assess all beams for the worst-case beam, as variation in shear link spacing due to construction tolerances are typically not considered in design or assessment. Furthermore, all survey data is from NDT scanning, and has not been verified my destructive investigations. It is possible that in some cases shear links may have been present but not identified, resulting in outliers in shear link spacing such as those visible in Figure 3.

Appendix C Concrete strength sensitivity analysis

COWI

HIGHLANDS AND ISLANDS ENTERPRISE

CAIRNGORM FUNICULAR RAILWAY

SENSITIVITY OF CONCRETE STRENGTH GRADES ON STRUCTURE UTILISATIONS

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CONTENTS

1 Introduction

COWI's structural appraisal of the Cairngorm Funicular Railway (see COWI Report A116993-RP01 v2) utilises the material grades given on the Structural Design Check Certificate as outlined in the Schedule of Basic Assumptions (A116993-SBA-Rev01):

- RC40 (f_{cu} = 40MPa) for the in-situ concrete, and
- RC50 (f_{cu} = 50MPa) for the precast beams.

BD 44 allows for concrete strengths to be obtained through destructive testing. If a sufficient number of samples are taken to allow a statistical "worst credible" strength to be obtained, then BD 44 further allows a reduction of the material factor, γ_m.

Theoretically, therefore, were testing to reveal that the concrete used for the Cairngorm Funicular Railway is stronger than assumed, the scope of

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strengthening might be reduced. The cost of testing could potentially be offset by the reduced scope of strengthening with additional cost benefits obtained.

A sensitivity study was conducted to determine what effect increased/decreased concrete strengths would have on the shear and moment capacities previously calculated for the superstructure. This information could then inform the decision whether testing of concrete would be a worthwhile exercise.

2 Sensitivity of material strength on shear capacity

For the assessment of the superstructure, shear capacities were calculated at nine defined locations where changes in geometry, spacing of reinforcement or enhancement factors defined in the assessment standard would result in step changes in the capacity. To optimise the efficiency of the sensitivity study, these locations were reviewed and only locations where overstresses had been calculated in the original assessment were carried forward into the study.

Capacities were then calculated at the two remaining locations (3a and 4) for a range of hypothetical concrete strengths, ranging from 40MPa to 80MPa. This was done both using the standard assessment partial factors defined in BD 44, i.e. where a small sample size of tests indicates an increased characteristic strength, and using reduced partial factors for the "worst credible" strength, i.e. where more extensive testing has been carried out.

In the assessment of the superstructure, shear had been assessed to both the standard BD 44 method and to the alternative method allowed in the standard which follows the approach adopted in the Eurocodes. The higher of the two capacities derived for each location was carried forward.

The standard BD 44 approach calculates the shear strength from a combination of the strength due to the concrete and the strength due to the shear links. As a result, enhancements in concrete strength result in an enhancement of the shear capacity. The Eurocode method, however, considers the shear capacity of the steel shear links in insolation. As such, enhanced concrete strengths do not result in enhanced shear capacities.

Results for the sensitivity study are summarised in Table 1 – Table 6 below. Note that the values include an enhancement for a notional axial compression of 100kN. In reality, this will vary from span to span.

Table 1 Type 1 beam – sensitivity study results – shear capacity – location 3a

	Characteristic Concrete Strength				
Shear Capacity	40MPa	50MPa	60MPa	70MPa	80MPa
BD method	169.2kN	174.4kN	179.9kN	184.7kN	189.2kN
EN method	194.6kN	194.4kN	194.6kN	194.6kN	194.6kN

Governing capacity	194.6kN	194.4kN	194.6kN	194.6kN	194.6kN
Enhancement	none	n/a (existing strength)	none	none	none
	Worst Credible Concrete Strength				
Shear Capacity	40MPa	50MPa	60MPa	70MPa	80MPa
BD method	175.2kN	182.0kN	197.9kN	193.2kN	198.0kN
EN method	194.6kN	194.6kN	194.6kN	194.6kN	194.6kN
Governing capacity	194.6kN	194.6kN	194.6kN	194.6kN	198.0kN
Enhancement	none	none	none	none	2%

Table 2 Type 1 beam – sensitivity study results – shear capacity – location 4

	Characteristic Concrete Strength				
Shear Capacity	40MPa	50MPa	60MPa	70MPa	80MPa
BD method	172.3kN	178.7kN	184.4kN	189.5kN	194.1kN
EN method	196.5kN	196.5kN	196.5kN	196.5kN	196.5kN
Governing capacity	196.5kN	196.5kN	196.5kN	196.5kN	196.5kN
Enhancement	none	n/a (existing strength)	none	none	none
	Worst Credible Concrete Strength				
Shear Capacity	40MPa	50MPa	60MPa	70MPa	80MPa
BD method	179.6kN	186.6kN	192.8kN	198.3kN	203.3kN
EN method	196.5kN	196.5kN	196.5kN	196.5kN	196.5kN

Governing capacity	196.5kN	196.5kN	196.5kN	198.3kN	203.3kN
Enhancement	none	none	none	1%	3%

Table 3 Type 2 beam – sensitivity study results – shear capacity – location 3a

	Characteristic Concrete Strength				
Shear Capacity	40MPa	50MPa	60MPa	70MPa	80MPa
BD method	153.1kN	159.3kN	164.8kN	169.6kN	174.0kN
EN method	162.2kN	162.2kN	162.2kN	162.2kN	162.2kN
Governing capacity	162.2kN	162.2kN	164.8kN	169.6kN	174.0kN
Enhancement	none	n/a (existing strength)	2%	5%	7%
	Worst Credible Concrete Strength				
Shear Capacity	40MPa	50MPa	60MPa	70MPa	80MPa
BD method	160.1kN	166.9kN	172.8kN	178.0kN	182.8kN
EN method	162.2kN	162.2kN	162.2kN	162.2kN	162.2kN
Governing capacity	162.2kN	166.9kN	172.8kN	178.0kN	182.8kN
Enhancement	none	3%	6%	10%	13%

Table 4 Type 2 beam – sensitivity study results – shear capacity – location 4

	Characteristic Concrete Strength				
Shear Capacity	40MPa	50MPa	60MPa	70MPa	80MPa
BD method	157.5kN	164.0kN	169.6kN	174.7kN	179.3kN
EN method	164.4kN	164.4kN	164.4kN	164.4kN	164.4kN

Governing capacity	164.4kN	164.4kN	169.6kN	174.7kN	179.3kN
Enhancement	none	n/a (existing strength)	3%	6%	9%
	Worst Credible Concrete Strength				
Shear Capacity	40MPa	50MPa	60MPa	70MPa	80MPa
BD method	164.8kN	171.8kN	178.0kN	183.4kN	188.4kN
EN method	164.4kN	164.4kN	164.4kN	164.4kN	164.4kN
Governing capacity	164.8kN	171.8kN	178.0kN	183.4kN	188.4kN
Enhancement	none	4%	8%	12%	15%

Table 5 Type 3 beam – sensitivity study results – shear capacity – location 3a

	Characteristic Concrete Strength				
Shear Capacity	40MPa	50MPa	60MPa	70MPa	80MPa
BD method	169.2kN	175.5kN	181.1kN	186.1kN	190.6kN
EN method	193.8kN	193.8kN	193.8kN	193.8kN	193.8kN
Governing capacity	193.8kN	193.8kN	193.8kN	193.8kN	193.8kN
Enhancement	none	n/a (existing strength)	none	none	none
	Worst Credible Concrete Strength				
Shear Capacity	40MPa	50MPa	60MPa	70MPa	80MPa
BD method	176.3kN	183.2kN	189.3kN	194.7kN	199.6kN
EN method	193.8kN	193.8kN	193.8kN	193.8kN	193.8kN

Governing capacity	194.6kN	194.6kN	194.6kN	194.6kN	194.6kN
Enhancement	none	none	none	none	3%

Table 6 Type 3 beam – sensitivity study results – shear capacity – location 4

	Characteristic Concrete Strength				
Shear Capacity	40MPa	50MPa	60MPa	70MPa	80MPa
BD method	171.2kN	177.6kN	183.2kN	188.2kN	192.8kN
EN method	196.5kN	196.5kN	196.5kN	196.5kN	196.5kN
Governing capacity	196.5kN	196.5kN	196.5kN	196.5kN	196.5kN
Enhancement	none	n/a (existing strength)	none	none	none
	Worst Credible Concrete Strength				
Shear Capacity	40MPa	50MPa	60MPa	70MPa	80MPa
BD method	178.4kN	185.4kN	191.5kN	196.9kN	201.9kN
EN method	196.5kN	196.5kN	196.5kN	196.5kN	196.5kN
Governing capacity	196.5kN	196.5kN	196.5kN	196.9kN	201.9kN
Enhancement	none	none	none	none	3%

The sensitivity study shows that a significant enhancement in strength is required before a marginal enhancement in shear capacity is achieved. This is typically the result of the Eurocode shear capacity governing the original assessment.

The Type 2 beams see the largest benefit of increased concrete strength. Shear overstresses in the Type 2 beams were found to range between 6% and 24% in the original assessment. Therefore, testing would need to indicate a significant enhancement in strength for the need for strengthening to be eliminated.

3 Sensitivity of material strength on moment capacity

The assessment of the superstructure was governed by hogging moments experienced at the anchor block locations. Utilisations in the range of 1.17 and 1.42 were calculated for the in-situ concrete with a strength of 40MPa.

Revised utilisations were calculated using FAGUS assuming enhanced characteristic strengths. However, it quickly became apparent that the any realistic enhancements gained by testing would not be sufficient to overcome the lack of capacity and thereby eliminate the need for strengthening. Therefore, the sensitivity study was aborted.

Results obtained from the initial checks are summarised in Table 7.

Table 7 Anchor block hog utilisations – sensitivity study results

	Characteristic Concrete Strength				
Utilisation at anchor block location	40MPa (existing)	50MPa	60MPa	70MPa	80MPa
0	1.42	1.39	1.37	aborted	aborted
14	1.41	1.39	1.37	aborted	aborted
29	1.24	1.20	1.17	aborted	aborted
48	0.84	0.71	0.66	aborted	aborted
65	1.26	1.19	1.15	aborted	aborted
78	1.30	1.23	1.19	aborted	aborted
	Worst Credible Concrete Strength				
	study aborted				

4 Recommendation

The sensitivity study shows that some benefit can be gained by considering an enhanced concrete strength, specifically for the Type 2 beams. However, the level of enhancement is unlikely to eliminate the need for strengthening. While the scope of strengthening may be somewhat reduced, it is anticipated that a large proportion of the cost of the strengthening will be the mobilisation costs of the contractor. Therefore, removing some isolated areas of strengthening is unlikely to offset the cost in sampling and testing of the concrete on site.

It must also be considered that testing with its associated cost is by no means guaranteed to allow the adoption of higher material strengths. The results may indicate that the material is as strong as indicated on the Structural Design Check Certificate or even that a reduction in material strength is necessary.

Appendix D Analysis of torsional effects

HIGHLANDS AND ISLANDS ENTERPRISE

CAIRNGORM FUNICULAR RAILWAY

A RECONSIDERATION OF TORSION IN THE APPRAISAL

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CONTENTS

1 Background / Summary

Shear and bending results for the main beams are reported in Section 6.2 of appraisal report A116993-RP01 v2. Review comments on the report included a query whether torsion stresses are significant. This note discusses the significance of torsion in the structure appraisal and concludes that the most appropriate approach is the one described in the appraisal report.

2 Torsional resistance

BD 44/15 clause 5.3.4.4 gives methods for determining the torsional resistance. The resistance of the beam was calculated accordingly.

A limited amount of torsion can be resisted by the concrete section without contribution from steel reinforcement. However calculations found that this concrete only torsion resistance was small, at approx 4 kNm.

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The torsional resistance was also calculated based on reinforcement. As reinforcement varies along the beam length the calculation considered the critical location for shear, which is approximately 3m from the support, where there are T8 links at 200 centres. The resistance governed by the reinforcement was determined to be 25 kNm. However the links are also needed for shear resistance. If the links are fully utilised in shear resistance they cannot also be used in torsion.

3 Analysis options

A line beam analysis method was used to determine load effects in the structure as described in section 5.1 of appraisal report A116993-RP01 v2. Each beam carries all the loads directly applied to it as if there was no connection to the other beam. This is a conservative approach.

However, the two beams are connected and are subjected to different live loads. Lateral wind load on the carriage, centrifugal effects and nosing loads will all add load to one beam and reduce load on the other beam. As the beams are connected by transverse bracing, the heavier loaded beam will drag down the lighter loaded beam, thus sharing load, i.e. load distribution will take place.

An alternative to the line beam analysis is grillage analysis. A grillage model will account for load distribution. This is also a conservative approach, but often gives more favourable results than the line beam option.

Grillage analyses are usually linear elastic, in which the elastic bending, shear and torsion stiffnesses of uncracked elements are used to determine the degree to which loads are distributed. However, other sets of stiffnesses may be used. One permitted option is to use a torsionless grillage in which elastic bending and shear stiffnesses are used but torsional stiffness is set to zero. According to BD 44/15 clause 5.3.4.2 this is permitted providing "sound engineering judgement has shown that torsion plays only a minor role in the behaviour of the structure".

Hence there are at least three analysis options: (1) line beam (2) grillage with torsion properties (3) torsionless grillage.

The reason these different analyses options are all valid is because they are all "lower bound" methods. Lower bound theory allows the use of any reasonable set of internal forces (i.e. moments, shears and torsions), providing the internal forces are in balance with the external forces. This relies on the assumption that parts of the structure which are overloaded can crack or yield and redistribute those load effects, and effectively the structure finds a way to resist the external loads. As BD 44/15 clause 4.4.3A states "In principle, provided ductility is adequate, any elastic analysis is a safe lower bound solution whatever section properties are used."

4 Test analyses

Tests were made on prototype grillages. Three grillages were analysed, each comprising 5 typical spans. One of the grillages had torsion properties, one had torsionless properties, and one had the bracing made ineffective to represent the line beam result. One load case was applied to produce maximum "in operation" ULS shear at around 3m from the support, as this was the critical result for shear. Another load case was used to produce maximum "out of operation" ULS shear at the same location.

This produced maximum "in operation" torsion of 19 kNm/beam coexistent with maximum shear of 172 kN/beam for the grillage with torsion properties. For the torsionless grillage only local torsions are seen but global shear increases to 186 kN/beam. With ineffective bracing the results are generally similar to that for the torsionless system.

The maximum "out of operation" torsion was 55 kNm/beam coexistent with shear of 144 kN/beam. For the torsionless grillage global shear increases considerably and again the ineffective bracing results are similar.

The "out of operation" result overloads the beam in torsion by itself. By inspection the torsion of 19 kNm in combination with shear of 172 kN/beam will give a higher overstress in the links than the shear arising from the torsionless grillage.

On this basis the optimum result for the structure will be given either by the torsionless grillage or the line beam. Hence the line beam results will be allowed to stand.

5 Local effects

As stated in section 3 above torsion can only be ignored if it takes a minor role in the behaviour of the structure. In this case, given the beams are deemed to be not resisting torsion it is necessary for the structure to have an alternative load path for transferring lateral loads on the rails to the supports.

Section 6.3 of appraisal report A116993-RP01 v2 has demonstrated that the diagonal bracing is capable of resisting the lateral loads. The top flanges of the beams are heavily reinforced and therefore by inspection able to carry the lateral loads to the nearest steel crossbeam. Therefore if the beams had sufficient local strength in the webs to transfer the lateral load from flange to crossbeam then an alternative load path for lateral loads would exist.

A calculation has shown that even with the widest link spacing of 300mm, there are still enough links in the web to transfer the load. Although the links are also utilised in shear, the links are only about 50% utilised in the webs because the shear resistance is governed by the inclined part of the webs in the flanges. Hence the local effect can be resisted by the remaining capacity of the links.

A calculation has shown that the crossbeam end details are also able to transfer the local effects.

Therefore there is sufficient local resistance to provide the alternative load path and allow the beams to be deemed as not acting in torsion.

Appendix E Review of SLS deflection and rotation limits

HIGHLANDS AND ISLANDS ENTERPRISE

CAIRNGORM FUNICULAR RAILWAY

BS EN 13107: REVIEW OF SLS LIMITS FOR OPERATION

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CONTENTS

1 Introduction

COWI UK published "Cairngorm Funicular Railway: Viaduct Appraisal Report" Version 2 in December 2018. Section 6.1 of this document reports Serviceability Limit State (SLS) results over the limiting requirements for the deflections and rotations given in BS EN 13107. Recommendations within section 8 suggested exceedance of these limits are checked with the equipment supplier. Highlands and Islands Enterprise (HIE) requested COWI make contact. In March 2019, COWI contacted Bernd Populorum of Garaventa (+41418591222).

2 BS EN 13107 Requirements

BS EN 13017 – 'Safety requirements for cableway installations designed to carry persons – Civil engineering works' was prepared by Technical Committee

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CEN/TC 242 and was drafted in German. European members had translations into their national language.

The limits for SLS deflections and Rotations as described in the English are;

9.4.4 Deformations

9.4.4.1 The specific vertical deformations of structures or structural elements are expressed by a ratio of the relevant horizontal span L. Variable actions as well as any time dependent deformations due to permanent actions shall be considered. The expressions to be applied according to 9.4.3.3 are given in parentheses below and the following ratios shall be observed:

a)	Buildings (17)	See EN 1992 (all parts) to EN 1996 (all parts) and EN 1999 (all parts);
b)	Bridges of funicular railways (16)	$w \leq L/600$.

9.4.4.2 The deformations of a line support structure at its top in a plane perpendicular to the axis of the line support structure are expressed as a ratio of the relevant height *H*. Variable actions as well as time dependent permanent actions shall be considered; as a general rule snow and avalanche actions as well as ice actions may be neglected.

Favourable self-weight components can be taken into consideration, unfavourable self-weight components can be neglected. Rope actions can be split into permanent and variable parts, whereby the permanent part can be neglected.

Formula (16) applies and the following ratios shall be observed:

a) *"in operation"*

1. support towers $u \leq H/300$;
2. combined support/compression towers $u \leq H/400$;
3. compression towers $u \leq H/500$;

b) *"out of operation"* general: $u \leq H/100$.

9.4.4.3 In a seismic design situation, particular regard shall be paid to EN 1998-6.

9.4.5 Rotations

The rotations of the tops of line support structures shall be verified as angles of rotation. Variable actions as well as time dependent permanent actions shall be considered and formula (16) shall be used. The approximate value of the angle of rotation for the case *"in operation"* shall be 0,003 rad.

Generally speaking, no verifications shall be carried out for the case of *"out of operation"*.

Figure 2-1: BS EN 13107 English requirements

3 COWI interpretation and results

COWI interpreted these requirements as;

- The vertical deflection limit is L/600
- The transverse horizontal deflection limit for piers is H/300 in operation or H/100 out of operation
- The maximum allowable rotation at the pier locations in the direction of the track is 0.003 radians in operation.

COWI reported results of overload for vertical deflections and rotations at supports.

4 Review of rotation requirements

Upon review of the English, French and German versions of BS EN 13107 it became apparent that the intent of the wording for rotations was misrepresented. Sections 9.4.4 clearly states limits for "Bridges of funicular railways" for vertical deflections as noted in 9.4.4.1 (b). However, 9.4.4.2 and 9.4.5 reference "line support structures".

COWI assumed that a line support structure was in this case a pier supporting a funicular railway line and associated cable infrastructure.

A review of the three paragraphs in their respective language by COWI and Garaventa concluded that the rotational requirement for a cable way may have become lost in translation.

9.4.5 Rotations

The rotations of the tops of line support structures shall be verified as angles of rotation. Variable actions as well as time dependent permanent actions shall be considered and formula (16) shall be used. The approximate value of the angle of rotation for the case *"in operation"* shall be 0,003 rad.

Figure 4-1: English text

German

9.4.5 Verdrehungen

Die Verdrehungen der Stützen am Stützenkopf sind als Verdrehungswinkel um die Stützenachse nachzuweisen. Einflüsse infolge veränderlicher Einwirkungen sowie zeitabhängiger ständiger Einwirkungen müssen dabei berücksichtigt und Formel (16) muss angewendet werden. Der Richtwert des Verdrehungswinkels für den Fall « *in Betrieb* » beträgt 0,003 rad.

Figure 4-2: German text

9.4.5 Rotations

Les rotations des supports de ligne au niveau des appuis de câble sont à vérifier comme angle de rotation par rapport à l'axe du pylône. Il faut tenir compte des actions variables ainsi que de toutes les actions permanentes dépendantes du temps. La Formule (16) doit être appliquée. Il est recommandé de limiter l'angle de rotation « *en exploitation* » à 0,003 rad.

Figure 4-3: French text

These translate as;

- English – *"The rotations of the tops of line support structures..."*
- German – *"The rotations of tower supports..."*
- French – *"The rotations of cable support structures..."*

These imply overall rotations of supports of cable way systems rather than the "rotations" assumed at bearing supports of beams under load as typically found in all bridges. Thus, it was agreed that COWI's assumption of a SLS limit for rotations was not a BS EN 13107 requirement and therefore could be ignored.

5 Review of deflection requirements

COWI calculated an overload of 39% against the limiting maximum vertical deflections of BS EN 13107. This equates to a 12mm exceedance of vertical deflection of a beam spanning circa 18m. Vertical deflection is limited to 30mm and 42mm was calculated.

The Cairngorm funicular railway has been in operation since 2001. With respect to SLS vertical deflections only, experience during standard operation over this duration demonstrates that the existing structure functions as required. This calculated SLS limit does not change the evidence that for vertical deflection, the theoretical exceedance of the limit over this time has not adversely affected either the civil structure or the performance of the funicular carriageway.

Design codes of practice detail design requirements for new funicular railways. The limits to which the existing structure was designed to a draft of BS EN 13107 is unknown. Cairngorm funicular is an existing systems with proven operation. This demonstrates that the overload of SLS deflection at its worst SLS load case has not resulted in structural or operational failure. The existing structure exhibits no distress and thus COWI conclude that this overload is of no significant issue.

COWI therefore suggest that the SLS vertical deflection overload as reported in Appraisal Report version 2 is ignored. It is recommended that HIE, as the asset owner, ensure the supplier of any future funicular system that the civil structure shall support is satisfied with this approach.

6 Summary

The results of "Cairngorm Funicular Railway: Viaduct Appraisal Report" Version 2 in December 2018 reported an overload of rotations and vertical deflections of supports and beams of the funicular respectively. Through discussions with funicular system supplier, Garaventa, COWI conclude that;

- A check of SLS rotation limits of supports is not required for funicular structures.

- The justification of operational performance over the life time of the existing structure concludes that the overload of SLS vertical deflections is not an operational concern and can be ignored.

Appendix F Review of design wind speed

COWI

HIGHLANDS AND ISLANDS ENTERPRISE

CAIRNGORM FUNICULAR RAILWAY

REVIEW OF DESIGN WIND SPEED AND DRAG FACTORS

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1 Review of design wind speed

The original design basis for the Cairngorm Railway adopted a 3-second gust wind speed of 75m/s at the highest point on the railway. This memo assesses the validity of this value through analysis of wind speed data collected at the top of Cairngorm Mountain.

The wind speed data has been obtained from:
http://cairngormweather.eps.hw.ac.uk/archive.htm

NOTE:

The weather station is located at the peak of Cairngorm mountain (1245m AMSL).

- Between 1990 and 1995, wind speed is reported in m/s.
- Between 1996 and 2017, wind speed is reported in mph.

In our data analysis, all wind speeds are converted to mph.

Instruments are exposed for ~2.5 minutes every half hour to record data. This was done due to weather conditions at the station and potential for ice build-up.

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A data reading is taken every 2.5 seconds out of these 2.5 minutes (this data not provided - each data point is for a whole 2.5 minute period).

Mean wind speed (MeW) is averaged over a period of 2.5 minutes (150 seconds). Max (MaW) and min (MiW) speeds are highest and lowest readings over the 2.5 minute period.

Wind direction (believed to be an average over the 2.5 minutes) and standard deviation of wind direction are also given. Two temperature measurements (T1 and T2) and (for some years) the temperature inside the weather station hut (TH) are also provided.

Large volumes of data are missing in various periods between 1990 and 2017 and some data is evidently spurious. Completeness of data is summarised in the following table:

Number of data points with non-zero MeW values (% maximum possible)												
Month start day -->	1	32	60	91	121	152	182	213	244	274	305	335
Month end day -->	31	59	90	120	151	181	212	243	273	304	334	365
Year	January	February	March	April	May	June	July	August	September	October	November	December
1990	100%	33%	91%	97%	100%	98%	95%	92%	86%	100%	86%	81%
1991	38%	100%	100%	100%	98%	100%	100%	100%	100%	100%	99%	83%
1992	98%	87%	100%	99%	100%	100%	97%	100%	100%	100%	100%	98%
1993	97%	14%	0%	0%	0%	0%	0%	5%	100%	100%	83%	58%
1994	90%	97%	79%	60%	79%	0%	0%	0%	58%	100%	73%	74%
1995	31%	100%	95%	11%	0%	0%	0%	0%	0%	0%	0%	0%
1996	0%	0%	0%	0%	0%	0%	0%	0%	0%	13%	79%	41%
1997	18%	37%	0%	15%	51%	74%	70%	79%	47%	95%	89%	97%
1998	100%	100%	92%	98%	100%	100%	70%	23%	85%	99%	93%	95%
1999	97%	21%	27%	70%	100%	100%	100%	100%	100%	100%	72%	30%
2000	6%	0%	0%	78%	84%	88%	87%	72%	99%	100%	34%	70%
2001	93%	65%	0%	27%	52%	84%	71%	4%	0%	4%	7%	0%
2002	0%	1%	3%	0%	0%	0%	0%	0%	0%	0%	0%	0%
2003	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%
2004	9%	38%	77%	78%	22%	0%	0%	0%	0%	15%	16%	9%
2005	5%	3%	19%	36%	24%	5%	2%	1%	4%	1%	2%	0%
2006	57%	99%	99%	99%	100%	81%	7%	26%	80%	36%	28%	3%
2007	0%	0%	0%	0%	0%	22%	105%	99%	96%	95%	95%	99%
2008	99%	96%	86%	99%	56%	0%	0%	0%	0%	32%	86%	94%
2009	98%	91%	95%	100%	100%	100%	100%	100%	95%	100%	100%	27%
2010	5%	0%	3%	3%	1%	0%	0%	0%	0%	0%	0%	0%
2011	0%	0%	47%	57%	6%	1%	1%	6%	10%	16%	58%	65%
2012	33%	15%	5%	2%	18%	31%	7%	15%	1%	0%	4%	23%
2013	15%	70%	100%	100%	100%	98%	101%	94%	97%	99%	100%	89%
2014	84%	3%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%
2015	0%	69%	81%	81%	98%	56%	63%	92%	84%	90%	64%	99%
2016	92%	90%	90%	62%	16%	2%	0%	58%	0%	0%	0%	34%

An extreme value analysis of the data has been undertaken based on the modified Jensen & Franck Method (a development of the Gumbel Method that uses daily maxima over a minimum threshold [taken as 50m/s], but rejecting the smaller value of maxima that occur on subsequent days, rather than the annual maxima used in the Gumbel Method). Analysis has been based on wind speed squared, as this has been shown to give better convergence.

Extracting the mode and dispersion from the best fit line in Figure 1 and extrapolating to a return period of 50-years gives:

Return Period =	50	years
Estimated CDF, P =	0.980	
Reduced variate =	3.922	
Mode, U =	20896	
Dispersion, 1/a =	3890	
V^2 =	36152	$(mph)^2$
V =	190	mph
V =	85.0	m/s

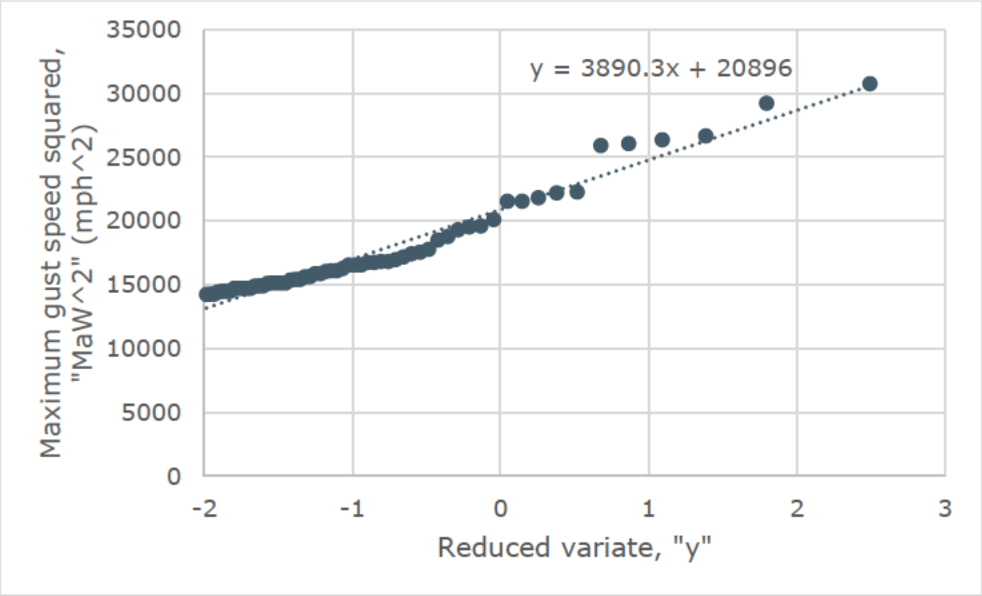

Figure 1 *Extreme value analysis for the square of the maximum gust speed*

Note that the weather station is at the top of Cairngorm Mountain. Summits are subject to vertical compression of airflow causing local acceleration over the crest. The top of the Cairngorm Funicular Railway is at an altitude of approximately 1100m and in a more sheltered location. Accounting for the variation in altitude alone and applying the altitude factor in the NA to BS EN 1991-1-4 gives c_{alt} = 2.245 at the weather station and c_{alt} = 2.100 at the top of the railway. The ratio of these altitude factors is 0.935, giving a predicted gust wind speed of 79.5m/s at the top of the railway.

However:

- The instruments only record for 5-minutes in every hour. It is expected that the prediction of gust wind speed at the top of the railway would increase if data from the remaining 55-minutes in every hour were included (as it is highly likely that faster gust wind speeds have occurred in this unmonitored period).

- The instruments record a 2.5 second gust, which is insignificantly faster (*circa* 1%) than the equivalent 3-second gust.

- The upper exposed part of the railway runs approximately along the WNW/ESE axis. Therefore, winds from the prevailing South Westerly direction will be more-or-less perpendicular to the railway and no benefit can be taken from directional effects (as per the c_{dir} factor in the NA to BS EN 1991-1-4).

- The effect of local acceleration over the crest of the mountain (at the site of the weather station) has not been taken into account but is expected to reduce the prediction of gust wind speed at the top of the funicular railway.

Clearly, the first and last bullet points are compensating factors, but it is not known whether their opposite effects are of similar magnitude.

2 Review of drag factors

Comments received from HIE following issue of the appraisal report questioned the high drag factors used in calculation of the lateral load from wind.

Original comment from client:

"C_D [Drag Factor] values for the concrete beams. These values look high to me but may be derived from a document I do not have access to. As transverse loading is significant overall this may be worth reviewing."

COWI response:

"These values are derived from BD37 and are as a result of the structural geometry of the superstructure. These are high and thus result in very high transverse loads. A review of these is worthwhile. Review of C_D values are only worthwhile if the "Accidental" extreme load case is not removed as this is the governing load case."

Drag factor (or coefficient) is an indication of how aerodynamic an object is.

Furthermore, wind shielding is a function of geometry of both an object and the proximity of other near-by objects. It defines any reduction in load effects on an element of a structure that may not appear to be directly exposed to the full wind load.

Both drag factors and permitted wind shielding have direct effects on the magnitude of lateral loading on the structure as a result of wind. For the Cairngorm funicular railway, the superstructure form is of a pair of concrete 'I' beams separated by a 2m gap. An 'I' section has a drag coefficient at the extreme end of the spectrum. The superstructure geometry defines the amount of wind shielding which can be applied.

The Schedule of Basic Assumptions details the approach to the appraisal of the viaduct. The wind load calculation basis is summarised here:

Wind characteristics on longitudinal beams:

- No shielding is permitted – full transverse load on windward side of both beams – based on BD 37/01 clause 5.3.3.1.2 (c). Although severe this recognises a significant inclination of wind flow likely within the topography adjacent to the mountainous environment. This may result in wind approaching at an angle resulting in both beams being visible to full wind loading.

- Area in elevation (per beam) = 0.8m^2 per m length

- Drag coefficient is taken from BD 37/01 Figure 5 with b/d ratio calculated using b = flange width (varies with beam type), d = total beam depth (800mm)

- Drag coefficients for beam types 1, 2, and 3 are 2.3, 2.4, and 2.75, respectively.

- No additional loading on the structure from vertical wind component has been considered.

Wind characteristics on the carriage:

- Wind load has been applied to windward side of carriage only. No load has been applied on leeward side.

- Side area in elevation = 31.9m^2 (provided by Doppelmayr in original design documentation)

- Drag coefficient = 1.3 (provided by Doppelmayr in original design documentation)

- Centroid of side area is 1.7m above top of rail.

- No additional loading on the structure from vertical wind component has been considered.

For quantification of the Storm/Accidental case, the wind speed in a given "area" is taken as the wind speed interpolated to the highest point of that area.

In summary, although severe the approach of wind shielding and the high drag factors as defined in BD 37/01 is consistent with design intent of the assessment standards. Wind tunnel testing or a study using computational fluid dynamics could be undertaken to determine the actual drag factor which may refine these results. Without site specific data or additional testing to justify any relaxation, it is concluded that the approach in the appraisal is most appropriate method. However, HIE may wish to review the possibility of a departure from standard based on robust justification and permit a different approach to wind shielding and drag factors. Any departure from standard would need approve from a "Technical Authority" and would result in residual risk being passed to HIE.

Appendix G Accidental wind load case sensitivity analysis

HIGHLANDS AND ISLANDS ENTERPRISE

CAIRNGORM FUNICULAR RAILWAY

SENSITIVITY STUDY ON THE EFFECTS OF ACCIDENTIAL WIND LOAD CASES

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1 Current wind load cases

COWI's structural appraisal of the Cairngorm Funicular Railway (see COWI Report A116993-RP01 v2) considered four wind load cases, which were based on information given in the original Structural Design Check Certificate:

1 Operational: Fully laden dynamic carriage + 35 m/s wind.

2 Evacuation: Empty static carriage with 5t kentledge + 50 m/s wind.

3 Storm: No carriage + storm wind loading varying from 56-75 m/s.

4 Accidental: Empty static carriage with 5t kentledge + storm wind loading varying from 56-75 m/s.

In the analysis, either the Operational or Accidental case governed all elements of the viaduct; the Evacuation and Storm cases did not govern in any case.

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2 Effects of removing or modifying the accidental wind case

The Accidental case involves a broken-down carriage clamped to the rails during a major storm. This load case is the combination of two extreme events. As this is unlikely to occur and does not endanger life, it may be possible to remove it or use unfactored loads in accordance with the Eurocode approach to Accidental Design Situations.

Additional analysis was conducted to determine the effects of the various "Out of Operation" wind load cases (i.e. Evacuation, Storm, and Accidental) individually. Analysis on the effects of the Accidental wind load case using partial factors of 1.0 for all loads was also conducted.

The critical effects of various possible "Out of Operation" wind load cases are shown in Table 1. Note that only elements of the viaduct determined to be overstressed are included in Table 1. The three "Out of Operation" permutations considered are:

1. Using the worst case of all wind load cases (as was done in COWI Report A116993-RP01 v2)

2. Using the worst case of the Evacuation, Storm, and *unfactored* Accidental wind load cases

3. Using the worst case of the Evacuation and Storm wind load cases only

For all permutations, only those effects that are worse than those of the "In Operation" case are shown in Table 1. The "In Operation" loading is not affected by this modification of wind load cases.

If an unfactored Accidental case is used (permutation No. 2), there would still be issues with pier crosshead capacity, pier base overturning, bearing uplift, and large transverse loads through the guided bearings.

It the Accidental case is removed entirely (permutation No. 3), "Out of Operation" loads would not cause overstress in any element over and above those from "In Operation" loads, except the transverse loads through the guided bearings.

Table 1 Effect of various wind load cases on viaduct overstress

Parameter	***Out of Operation wind load case permutation***		
	No. 1 – with full Accidental	**No. 2 – with unfactored Accidental**	**No. 3 – no Accidental case**
Beam hogging moment	In operation governs except areas 5 and 6. 3% overstress at piers 92 and 93 and ~10% increase in overstress at anchor blocks 65 and 78 relative to the in operation case.	In operation governs everywhere.	In operation governs everywhere.
Beam shear	In operation governs except areas 5 and 6. 9% overstress at beam 93 and 1% overstress at beam 77.	In operation governs everywhere.	In operation governs everywhere.
Bearing uplift	Occurs at ~50% of piers.	Occurs at ~50% of piers.	Does not occur.
Guided bearing max. transverse reaction	Up to 460kN (ULS).	Up to 380kN (ULS).	Up to 280kN (ULS). Up to 230kN (SLS).
Bearing max. normal reaction	Up to 505kN (ULS).	In operation governs critical ULS bearing load of 475kN.	In operation governs everywhere.
Pier moment capacity	Up to 110% overstress at 50 locations	Up to 70% overstress at 30 locations (similar to in operation case)	Up to 60% overstress at 30 locations (similar to in operation case)
Pier shear capacity	Up to 25% overstress at 12 locations	Up to 2% overstress at 2 locations (could probably be shown to pass with additional analysis).	OK everywhere.

Parameter	***Out of Operation wind load case permutation***		
	No. 1 – with full Accidental	**No. 2 – with unfactored Accidental**	**No. 3 – no Accidental case**
Pier crosshead link capacity	Up to 45% overstress at 40 locations.	Up to 15% overstress at 10 locations.	OK everywhere.
Pier base moment capacity*	Overturning at 24 locations (possibly not of concern based on advanced geotech analysis).	Overturning at 10 locations (possibly not of concern based on advanced geotech analysis).	OK everywhere.
Cross-bracing	OK except possible minor connection overstress in area 6 (not enough information to conduct calculations).	Likely OK everywhere.	Likely OK everywhere.

*Refer to further analysis noted in Addendum to appraisal report

3 Recommendation

For many elements in the structure, the factored accidental load case used in the original design and the appraisal is more severe than all the other load cases. This is particularly true for the bearings and substructure. This means it governs the extent and hence the cost of strengthening.

The accidental load case considered in the original design, i.e. storm winds coinciding with a carriage stranded on the viaduct, is a highly unlikely event. The carriage has never been stranded in the 18 years life of the structure to date, and the probability that the carriages could not be recovered to the end stations prior to a severe storm is very low.

It could be argued that no people are put at risk by this breakdown and wind combination, since no-one is on the mountain in a storm. In this case the risk is purely commercial. The owner could choose to accept the risk of damage in order to reduce the cost of strengthening.

BS EN 13107 clause 7.3.4 lists a number of possible accidental events for funicular railways, but none are similar to this breakdown and wind combination, so there is no code requiring this load case. However, there is an obligation to take measures to avoid disproportionate damage due to accidental actions, hence it seems reasonable to strengthen for this combination if it is a credible occurrence, unless otherwise directed by the owner.

Highways England design standard BD 100/16 clause 1.5 states that Eurocodes are to be used as the basis for strengthening highway structures. This supports the approach of addressing this load combination as a Eurocode Accidental Design Situation, i.e. using unfactored load effects. This provides a codified approach which should be acceptable to the regulatory authorities.

Therefore the Eurocode approach, i.e. an unfactored combination of breakdown and wind, will be proposed for the strengthening, unless otherwise directed by the owner.