DECEMBER 2018
HIGHLANDS AND ISLANDS ENTERPRISE

CAIRNGORM FUNICULAR RAILWAY

VIADUCT APPRAISAL REPORT

COWI

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v2	December 2018	Second issue to incorporate intrusive investigation results and survey data	[redacted]	[redacted]	[redacted]
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APPENDICES

1 Introduction

1.1 Facility Description

CairnGorm Mountain Railway is a funicular railway within the CairnGorm Mountain ski resort near Aviemore, Scotland. It is normally open to passengers every day all year round, subject to weather restrictions.

CairnGorm Mountain Railway has a total length of approximately 1900m horizontally and an elevation gain of approximately 450m up to an altitude of 1090m above sea level. For most of its length the funicular railway is supported on a 94-span viaduct with spans of typically 18m. The top 250m is in a cut and cover tunnel. Construction commenced in 1999 and the facility opened for public use in 2001. The facility is owned by Highlands and Islands Enterprise and is operated by Cairngorm Mountain Limited under a 25 year lease.

Figure 1-1 General view of Cairngorm Funicular Mountain Railway

CairnGorm Mountain Railway comes under the scope of the Cableway Installation Regulations 2018 and EU Directive 2016/424.

In October 2018 Cairngorm Mountain Limited took the decision to suspend operation of the funicular railway temporarily, citing concerns with the structure supporting the track, and to allow investigation works to take place.

1.2 Terms of Engagement

COWI UK Limited were originally engaged by Cairngorm Mountain Limited and Highlands and Islands Enterprise to assist with investigation of the current condition of the viaduct structure supporting the funicular railway, including this appraisal. ADAC Structures are also engaged by Cairngorm Mountain Limited and Highlands and Islands Enterprise as technical advisor and operations support. In late November 2018, Cairngorm Mountain Limited entered administration.

1.3 Scope of this Appraisal

The scope of this study is restricted to an appraisal of the viaduct to determine whether the viaduct in its current condition can support the original design loads using highways assessment standards.

This study includes the following:

- appraisal of the viaduct superstructure, substructure and bearings;
- appraisal of the viaduct foundations.

This study excludes the following:

- appraisal of mechanical and electrical systems including the carriages, track, track fixings, haul cable, drive and control systems;
- appraisal of the top, bottom and intermediate station structures, including the rail support structures in the top and bottom stations;
- appraisal of the tunnel.

2 Structure Details

2.1 Structure Description

The viaduct is predominantly concrete. Most spans are 18m between bearing centres horizontally, but as the gradient of the track varies, the span measured parallel to the track varies.

Figure 2-1 Typical view of structure

The funicular is generally a single track, but with a passing loop at mid-length. There is one intermediate station, just below the passing loop.

Figure 2-2 Passing loop

Each span comprises a pair of precast concrete "I" shaped beams, one under each rail. The precast beams are reinforced but not prestressed. There are three types of beam:

- type 1 is used on straight sections,
- type 2 where the plan curvature is 300m radius,
- type 3 where the plan curvature is 200m radius.

All beams are straight and variations in the track fixings are used to accommodate the track curvature in plan and elevation. For beam types 2 and 3 the track fixings are in transverse channels so they can be offset from the beam centreline.

Most precast beams have scarfed ends with reinforcing bars projecting into cast insitu concrete stitches and diaphragms at each support to form a continuous structure. Straight ends are used at expansion joints.

Figure 2-3 Typical insitu stitch at support

There are expansion joints at approximately 300m intervals breaking the viaduct into six continuous structures as follows:

- "area 1" - piers 0 to 14, adjacent to bottom station,
- "area 2" - piers 14 to 29,
- "area 3" - piers 29 to 48, including the intermediate station,

- "area 4" - piers 48 to 65, including the passing loop,
- "area 5" - piers 65 to 78,
- "area 6" - piers 78 to the tunnel portal.

At the lower end of each continuous length there is a large insitu concrete anchor block, each with combination of rock anchors and shear dowels into the ground.

Figure 2-4 Typical anchor block

Within each span a galvanised steel plan bracing system connects the beams. The bracing consists of transverse "I" beams bolted to both beams at 3.6m centres and diagonal bracing formed from circular hollow sections.

Figure 2-5 Construction photograph showing bracing

At each support a pair of "pot" bearings support the deck. One is free sliding and one is sliding guided providing lateral support to the deck. The bearings are aligned with the slope so that the sliding surfaces are parallel to the rails.

The bearings are supported on a precast concrete crossbeam on concrete piers which comprise a stack of precast outer shell units with solid insitu concrete infill. The crossbeam is post-tensioned to the pier insitu concrete infill using four pre-stressing bars. At the base of each pier is a slab forming a spread foundation which is benched in to the sloping ground.

2.2 Current condition

Over recent years there have been concerns over the condition of the viaduct. The following defects were recorded prior to COWI's involvement or have been discovered during this exercise:

1 Cracks have been observed in the main concrete beams at numerous locations. There are cracks in the bottom flanges of most precast beams up to 0.5mm wide believed to be flexural cracking. There is widespread micro-cracking in the precast web beams around the bolted bracing connections. However, the largest cracks are in the insitu concrete close to the interface between precast and insitu concrete, which in some cases cracks extend through the bottom flange of the precast beam. In many cases attempts have been made to seal cracks, for example by resin injection. At two locations - piers 22 and 56 - the cracks have been seen to widen by approximately 0.7mm under the action of live loads and close up again after the load has passed. [Refer ADAC condition reports Nov 2015, Dec 2016, July 2017, July 2018]

2 Spalling and damage to the main concrete beams has been observed in several areas. This damage has often been attributed to mechanical damage, such as impact from piste machines, or fixings being too close to edges. There are several locations where web reinforcement in precast beams has been exposed. [Refer ADAC condition reports Nov 2015, Dec 2016, July 2017, July 2018]

3 There is evidence of leakage at cracks, especially at the scarfed joints. Many have considerable calcite bleed and rust staining is noted at some of the cracks [Refer ADAC condition reports Nov 2015, Dec 2016, July 2017, July 2018]

4 The grout under bearings is disintegrating at several locations. [Refer ADAC condition reports Nov 2015, Dec 2016, July 2017]

5 Many sliding bearings appear to be misaligned longitudinally. The bearings at several piers, especially in areas 3 and 4 were seen to be near the end of the sliding tracks suggesting that under extreme temperatures the bearings might slide beyond their stainless steel tracks. [Refer ADAC bearing report Aug 2018]

6 One pier - pier 91 - appears to be leaning away from vertical by approximately 1°. [New observation by COWI/ADAC, Nov 2018]

7 Some rock bolts at anchor blocks can be loosened by hand. [New observation by COWI/ADAC, Aug 2018]

The above defects have been accounted for in the appraisal where possible by incorporating appropriate assumptions in the Schedule of Basic Assumptions (refer to Appendix A). Defect 3 indicates corrosion of reinforcement is ongoing in some locations, but the extent of corrosion is unknown. In this appraisal it is assumed no significant reinforcement corrosion has taken place to date as only limited rust staining is visible.

Figure 2-6 Cracking at a pier 9 (refer defect 1)

Figure 2-7 Crack repair at pier 9 (refer defect 1)

Figure 2-8 Typical crack leakage and bearing misalignment (refer defects 3 and 5)

Figure 2-9 Bearing misalignment (refer defect 5)

Figure 2-10 Loosened rock bolt in anchor block (refer defect 7)

2.3 Information available

The available record drawings are listed within the Schedule of Basic Assumptions in Appendix A. These drawings present the following difficulties:

- The available drawings do not provide a clear complete set of as-built information. There are some discrepancies within the drawing set, and in these cases it is not clear which drawings take precedence.
- Some of the drawings are marked "preliminary" but contain details not shown on any other drawing, and therefore cannot be ignored.
- There are some details missing from the available drawing set.
- Some drawings show details that are not consistent with the existing as-built structure.

To corroborate the drawings, information has also been taken from the following sources:

- Construction photographs, where available,
- Site surveys of bearings and concrete reinforcement using non-destructive methods,
- Intrusive investigations of reinforcement details at locations of interest,
- Ground investigations including trial pits,
- Extensive search of third parties involved in design and construction for further information.

A safety report including a limited Health and Safety File has been identified but also contains conflicting information.

3 Appraisal Basis

3.1 Schedule of Basic Assumptions

A Schedule of Basic Assumptions is included in Appendix A.

Highway assessment standards are used for the appraisal of structural elements. Eurocodes are used for the appraisal of the foundations. Reasons for these choices are given in clause 4.5 of the Schedule of Basic Assumptions.

The appraisal is concerned with immediate public safety, as opposed to durability, calculations are generally undertaken at the Ultimate Limit State only, as this is associated with structural collapse. However, deflections and rotations of the superstructure are also considered as rail deformations may affect the ability of the carriage to avoid derailment.

The appraisal uses the loads stated on the original design certificate. Wind speeds are based on a report produced by Edinburgh University which unfortunately is not available, so the validity of these wind speeds cannot be verified.

3.2 Construction sequence

The sequence of construction shown in construction photographs shows that the precast beams were installed as simply supported spans before the insitu concrete at piers was cast. Therefore under dead loads there would initially be sagging moments but no hogging moments. This could change in time due to concrete creep and differential settlement.

Figure 3-1 Precast beam installation

The alignment of the bearings means that substantial axial load builds up towards the anchor blocks at the lower end of each area. It is not clear from construction photographs how precast beams were secured in position until insitu stitches were cast. It is possible that the tang plates at bearings were used to temporarily transfer axial load. The photograph below shows what might be evidence of welding at bottom reinforcement to provide temporary restraint.

Figure 3-2 Construction of insitu splice

However, irrespective of the system used for temporary restraint the axial loads in the final condition will be unaffected. Also, it is assumed that the component of axial load due to the braking load is transferred into the supporting structure by track fixings locally to the carriage position.

The viaduct also supports guide wheels to control the position of the funicular haul cable. The haul cable will therefore impose lateral loads on the viaduct as the track curves in plan, and vertical loads on the viaduct due to cable self weight and where there are crest curves in elevation.

Figure 3-3 General view showing haul cable guide wheels

4 Assumed structural details

4.1 Longitudinal reinforcement in main beams

The main record drawings showing longitudinal reinforcement are:

- Drawing CA150/2/49 rev B marked "preliminary". This shows 3 no T40 bars in bottom of all beam types, 3 no T32 bars in top of beam types 1 and 2, and 3 no T32 + 2 no T25 in top of beam type 3. Bottom bars are not continuous at piers but the middle bar is bent upwards. The top T32 bars are lapped at piers in beam type 1, but are connected using Bartec type B and DB32 grout sleeve couplers in beam types 2 and 3. The drawing does not show whether the T25 bars in beam type 3 are coupled, nor what happens where a type 3 is connected to another beam type.

- Drawing CA150/2/76 rev D marked "contract issue, for construction". This shows details matching those shown on drawing CA150/2/49.

- Drawing CA150/2/79 rev C marked "contract issue, for construction". This shows broadly similar details to those shown on drawing CA150/2/49 but with a DB40 grout sleeve coupler instead of a DB32 and the middle bottom bar reducing from T40 to T32 at span ends.

Other record drawings showing details are:

- Drawing CA150/2/39 rev A mainly shows insitu diaphragm details in the passing loop, but also includes some beam details. It shows that links in the insitu concrete include all 5 top bars.

- Drawing CA150/2/88 rev A mainly shows fixings for passing loop beams, but also includes the beam reinforcement. The longitudinal reinforcement matches drawings CA150/2/76 and CA150/2/79 except for the length of bars projecting beyond the scarf joints.

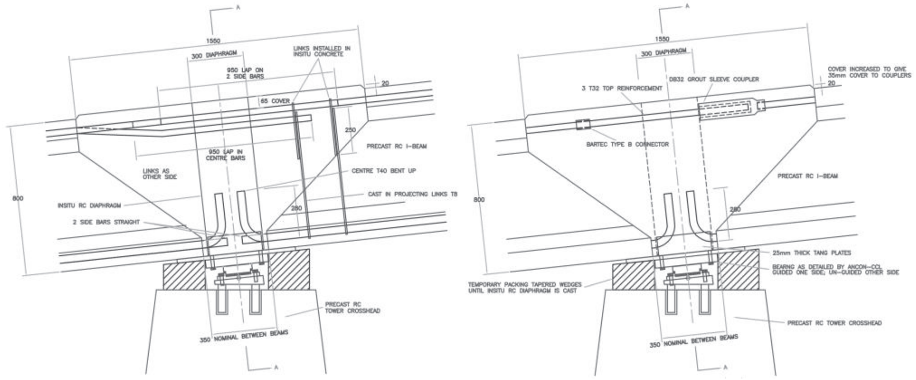


Figure 4-1 Extracts from drawing CA150/2/49

Construction photographs confirm the top and bottom reinforcement projecting as shown on the drawings, but unfortunately there are no photographs showing top steel connections in type 2 or 3 beams, so there is no photographic confirmation of the coupled connection.

Figure 4-2 Beam end details

The intrusive investigation, see Appendix E, has confirmed that there are only the central three T32 bars of the upper reinforcement in the insitu concrete at pier 56, which is within the passing loop and connects type 3 beams. The investigation confirmed that a coupler arrangement using couplers of similar dimensions to a "DB40" were used. Two T25 outer bars terminated in the pre-cast concrete beams and thus are not continuous through the insitu joint for a type 3 to type 3 beam connection. Further non-destructive testing at other locations has also not positively confirmed if couplers have been used, but cover measurements indicate couplers may be used at some piers. Similarly, intrusive investigations at pier 22 which connects type 2 beams confirmed the use of couplers with similar dimensions to "DB40" in the arrangement seen on drawing CA150/2/49. It is also noted that where "HALFEN" channels are located within the proximity of a coupler in the insitu concrete joint a significant section through the coupler has been removed to accommodate the rail fixing detail.

Based on the above it is assumed that longitudinal reinforcement is generally as shown on drawings CA150/2/76 and 79. The use of couplers at all assumed locations has not been proven, but there is no reason to dispute the drawings. Additionally it is assumed that at piers with type 3 beams, including within the passing loop, only the three central T32 bars are connected. It is unknown what happens where type 3 beams are connected to another beam type, but it seems sensible to assume the 3 no T32 bars are coupled and the T25 bars simply stop.

4.2 Shear reinforcement in main beams

The main record drawings showing shear reinforcement are:

- Drawing CA150/2/42 rev C only shows beam type 1 but shows T8 links at 200 centres. Each link appears to be a single bar with an unusual shape.

- Drawing CA150/2/49 rev B marked "preliminary" shows T8 links at 200 centres in all beam types and shows a 250 lap on 2 no links within the scarf joint. Each link is shown as a single bar for beam types 1 and 2, but beam type 3 has an extra link around the top 5 bars.

- Drawing CA150/2/76 rev D marked "contract issue, for construction" shows T8 links in pairs at 200 centres. Two bar marks are shown for each link pair, but there is no detail to show the link shape.

- Drawing CA150/2/79 rev C marked "contract issue, for construction" also shows T8 links in pairs at 200 centres. Three bar marks are shown but there is no detail to show the link shape. A detail shows a 250 lap on 2 no links within the scarf joint.

Other record drawings showing details are:

- Drawing CA150/2/39 rev A mainly shows insitu diaphragm details in the passing loop, but also includes some beam details. A detail shows 5 no T8 links at 100 centres in the scarf joint. The lap length of the links is not shown but appears to be approximately 250mm for 3 of the links, 200mm and 125mm for the remaining 2 links. A revision note says "link spacing reduced to 75mm at towers 51, 52, 54 & 56". Sections suggest each of the links is a single bar, except in the scarf joint where there is a straight lap in the web.

- Drawing CA150/2/47 rev D mainly shows the passing loop, but several sections show links matching the details shown on drawing CA150/2/49.

- Drawing CA150/2/88 rev A mainly shows fixings for passing loop beams, but also includes the beam reinforcement. The shear links match drawings CA150/2/76 and CA150/2/79.

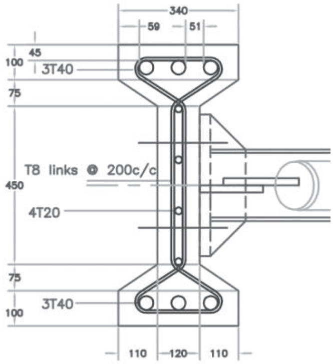


Figure 4-3 Extract from drawing CA150/2/42

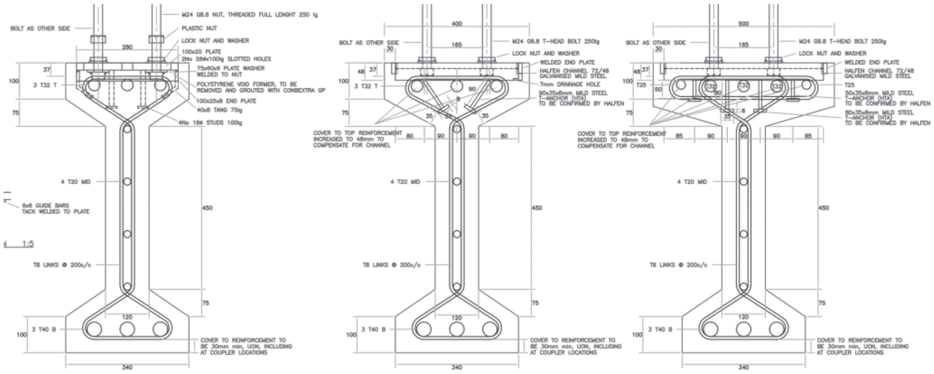


Figure 4-4 Extract from drawing CA150/2/49

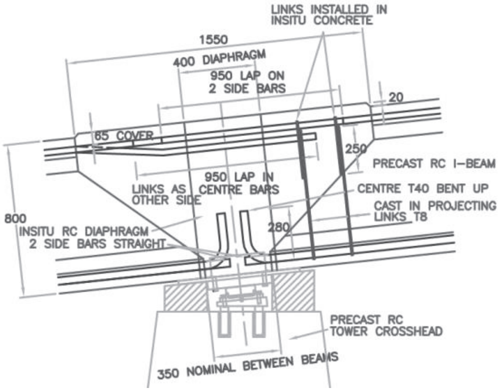


Figure 4-5 Extract from drawing CA150/2/79

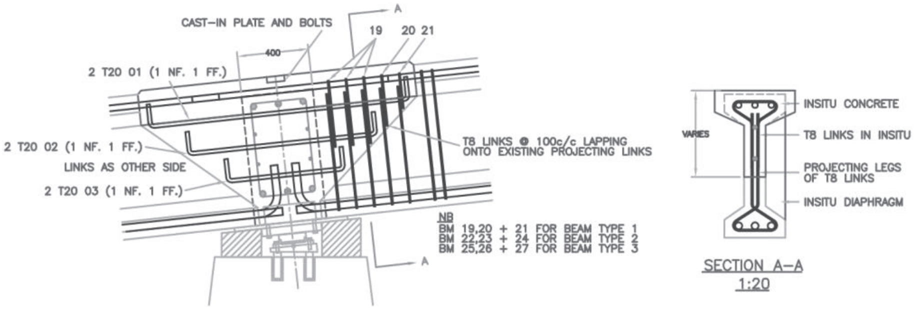


Figure 4-6 Extract from drawing CA150/2/88

Construction photographs clearly show that the shear reinforcement projecting from the precast beams scarf joints appears as shown on drawing CA150/2/39, and not as shown on the other drawings, hence drawing CA150/2/39 is assumed to be correct.

Site non-destructive testing, see Appendix D, has found that the link spacing is quite irregular, but varies in zones along the beam length, unlike that shown on the drawings. Testing at piers 51, 54 and 56 imply that the 75mm link centres stated on drawing CA150/2/39 is not correct.

Based on the site testing the following shear link pattern is assumed for all beams.

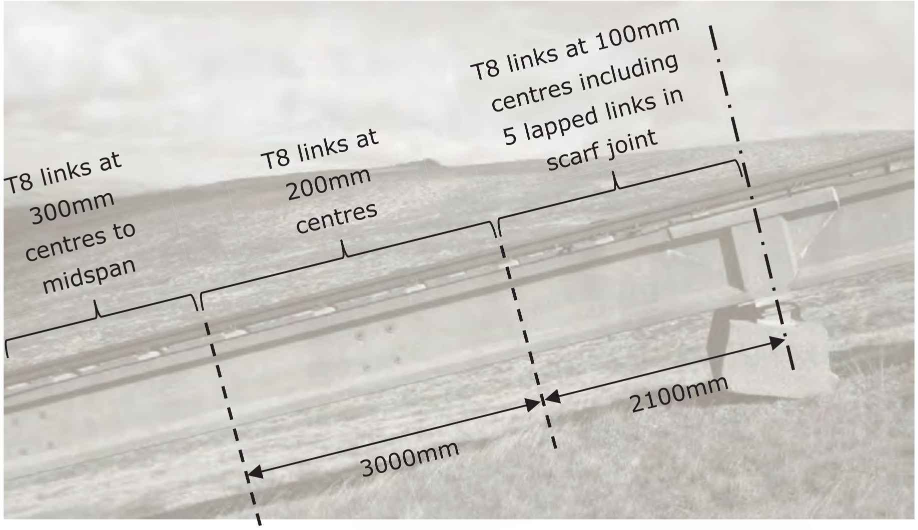


Figure 4-7 Assumed shear link provision

The shear link shape is unclear, as drawings Drawing CA150/2/76, 79 and 88 imply the basic link is formed from at least 2 bars as opposed to the single bar shown elsewhere. However, in the absence of any other shape indicated, the shear link shape for beam types 1 and 2 will assumed to be a single bar as shown on drawings CA150/2/39, 42, 47 and 49, with an extra loop around top steel for beam type 3.

On-site intrusive investigations at piers 22 and 56 shows site alterations not recorded in as-built documentation. At couplers shear links are displaced longitudinally within the in-situ joint and in some areas are only provided around the central T32 bar.

4.3 Bearings

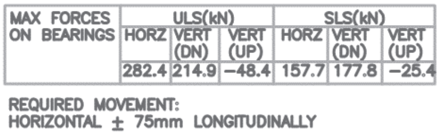
The main record drawings showing bearings are:

- Drawing CA150/2/42 rev C only shows beam type 1 but states "Guided sliding bearing to resist uplift by CCL Systems or equal". A table gives max forces and movements on bearings as shown in Figure 4-8.

- Drawing CA150/2/49 rev B marked "preliminary" refers to Ancon-CCL drawings giving two drawing numbers, but unfortunately these drawings are unavailable. Details of shear plates embedded in the insitu diaphragm and tapered steelwork under the bearings are given.

- Drawing CA150/2/76 rev D marked "contract issue, for construction" shows bearings but gives no details.

- Drawing CA150/2/79 rev C marked "contract issue, for construction" describes bearings for a typical pier with beam types 2 or 3 as "guided one side, un-guided other side". A section at an anchor block seems to show two guided bearings, but this may simply be a drawing error.



MAX FORCES ON BEARINGS	ULS(kN)			SLS(kN)		
	HORZ	VERT (DN)	VERT (UP)	HORZ	VERT (DN)	VERT (UP)
	282.4	214.9	−48.4	157.7	177.8	−25.4

REQUIRED MOVEMENT:
HORIZONTAL ± 75mm LONGITUDINALLY

Figure 4-8 Extract from drawing CA150/2/42

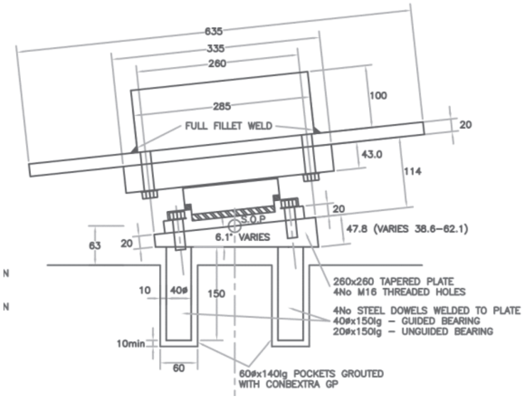


Figure 4-9 Extract from drawing CA150/2/49

Site photographs and inspections do not show any evidence of a hold down mechanism. It is therefore unlikely that uplift bearings have been provided and the table of bearing loads in drawing CA150/2/42 is assumed to be unreliable.

All bearings allow sliding in a direction parallel with the track. The sliding surfaces are low friction, hence the effect of sliding bearings being installed at an inclined angle is that loads imposed by the bearings on the substructure are normal to the track, as shown in Figure 4-10, or horizontal normal to the guides. There is the possibility of a friction force in the direction of the track, but this will be small compared the main force.

These forces resolve into vertical and horizontal components which act at the centre of the bearing contact area at the level of the lower bearing plate. Horizontal and vertical components vary in magnitude relative to imposed loads depending on angular variation in bearing sliding surface.

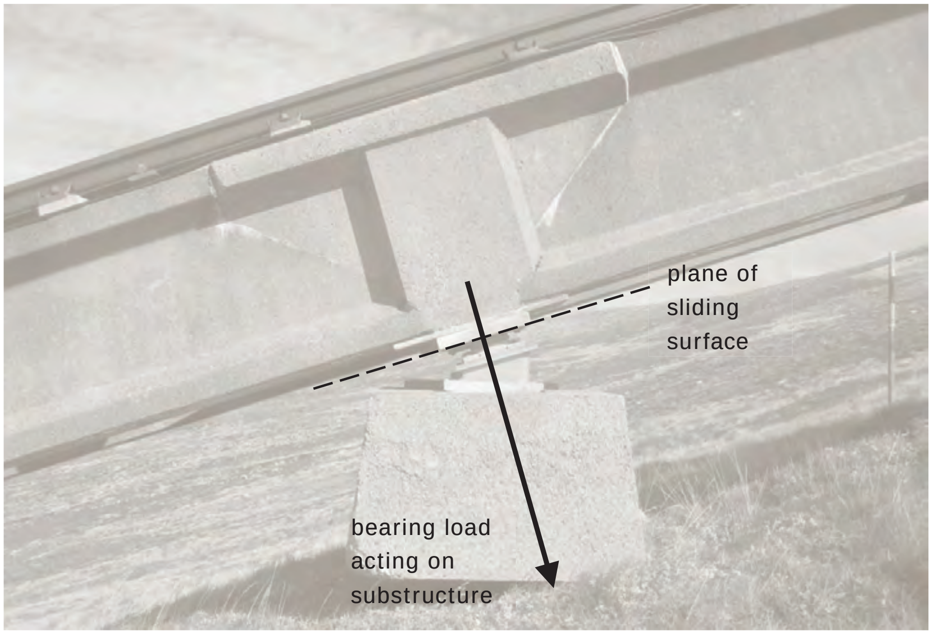


Figure 4-10 Effect of inclined bearings on substructure loads

All bearings are of the "pot" bearing type. Although there are no drawings showing bearing details, the external dimensions have been measured and hence the area of elastomer and PTFE surfaces estimated. The assumed details are that the free sliding bearing has an 80mm diameter elastomeric disc, while the guided bearing has a 140mm diameter disc.

Articulation of any "area" permits longitudinal movement of the main beams along the sliding plane. Thrust blocks provide a fixed connection to enable allowable movement ranges to increase with distance from thrust block. Movement range is quoted at +/- 75mm. Reference made in the Health and Safety file suggest bearings were "pre-set" during construction to account for differing temperatures.

4.4 Substructure reinforcement

Pier crosshead reinforcement is shown on the following drawings:

- Drawing CA150/2/60 rev B shows reinforcement in the precast crossheads for most piers, except for the wider crossheads in the passing loop. The crosshead has 5 no T25 bars in the top face of the bearing corbel and 3 no T8 links at 200 centres along the crosshead length.

- Drawing CA150/2/77 (no rev) shows reinforcement in the widened crosshead at pier 51, and reinforcement details are the same as drawing CA150/2/60.

- Drawing CA150/2/78 (no rev) shows reinforcement in the crossheads at piers 52 and 56 which support 3 main beams but reinforcement details are the same as drawing CA150/2/60.

Pier column reinforcement is shown on the following drawings:

- Drawing CA150/2/57 (no rev) shows pier reinforcement for piers 51, 52 and 56 in the passing loop.
- Drawing CA150/2/67 rev D shows reinforcement for all except the shortest piers. The vertical reinforcement varies from 12 no T32 + 8 no T25 bars in the tallest piers to 18 no T25 bars in the shortest piers. Bars are concentrated on the shortest faces of the columns, i.e. maximising resistance to lateral loads. Shear reinforcement is the same in all columns.
- Drawings CA150/2/68 rev B, CA150/2/69 rev B and CA150/2/70 rev C show reinforcement in the shortest piers, which are all similar to the reinforcement shown on drawing CA150/2/67.

Pier base reinforcement is shown on the following drawings:

- Drawings CA150/2/57 (no rev), CA150/2/68 rev B, CA150/2/69 rev B, CA150/2/70 rev C, CA150/2/71 rev C, CA150/2/72 rev C, CA150/2/73 rev B, CA150/2/74 rev B and CA150/2/75 rev A show reinforcement in the various sized bases. All bases are reinforced with T20 bars at 175 centres top and bottom faces in both directions and vertical side faces, with T8 bars at 240 centres horizontally on side faces.

Anchor block reinforcement is shown on the following drawings:

- Drawing CA150/2/38 (no rev) shows reinforcement in anchor block 48. Drawing CA150/2/63 rev D shows reinforcement in all other anchor blocks. All anchor blocks are reinforced with T16 bars at 200 centres on most faces with T25 bars providing anchorage for the beams into the centre of the block.

There is nothing to verify the above, but there are no conflicting details and there is no reason to doubt the accuracy of these drawings.

4.5 Foundations

Several drawings: CA150/2/38, CA150/2/63 rev D, CA150/2/67 rev D, CA150/2/68 rev B, CA150/2/69 rev B, CA150/2/70 rev C, CA150/2/71 rev C, CA150/2/72 rev C, CA150/2/73 rev B, CA150/2/74 rev B and CA150/2/75 rev A, carry an identical set of notes which includes the following: "Foundation sizes are designed on the basis of a safe bearing capacity of 150kN/sq.m. This must be confirmed before construction commences. Soft spots below the foundations are to be removed and made up in lean mix concrete."

A limited site investigation has been carried out and is included in Appendix C. Assumed soil parameters are described in section 6.8.

5 Analysis

5.1 Global analysis

The independent structures for each of the six "areas" have been analysed as line beams. The line beam model reflects the bearing arrangement such that bearing reactions are normal to the line beam and axial load builds up towards the bottom of each area where it is resisted by the anchor block. For self-weight, hinged joints are used to reflect the construction sequence.

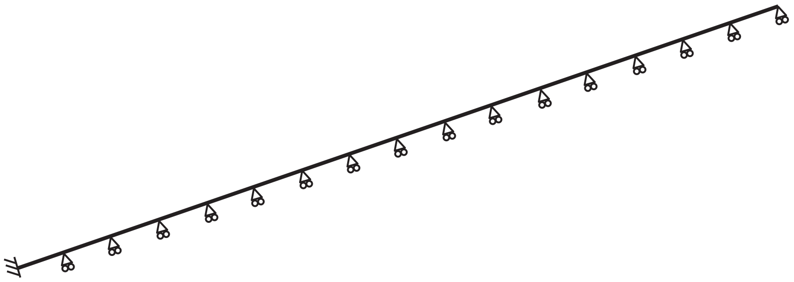


Figure 5-1 Typical line beam analysis

Figure 5-2 Typical line beam bending moment for dead load

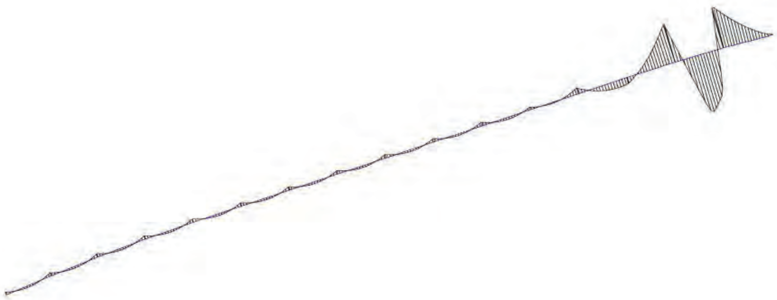


Figure 5-3 Typical line beam bending moment for a typical live load plus superimposed dead load case

A grillage model of half of the passing loop has also been used, as a simple line beam model would not fully capture all the load effects.

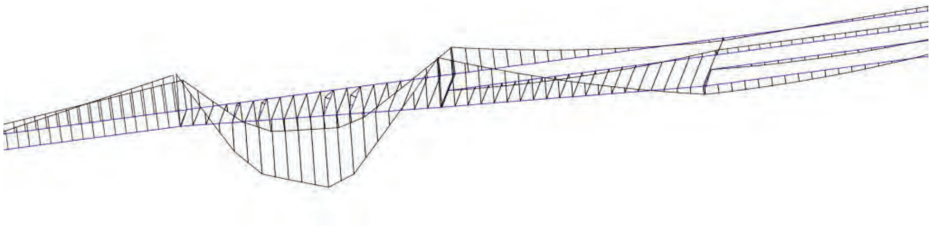


Figure 5-4 Typical grillage analysis at passing loop for a typical live load plus superimposed dead load case

In accordance with BD 44/15 clause 3.4, the analysis for loading at the Ultimate Limit State does not consider concrete creep and differential settlement. Hence in this appraisal, which is predominantly at Ultimate Limit State only, it is assumed there is zero hogging moment at piers due to dead load. Under superimposed and live load the beam is deemed to be continuous at the piers, except as noted below.

The analysis at the Serviceability Limit State is the same as at the Ultimate Limit State but includes an allowance for concrete creep. Differential settlement is not considered as no data is available.

At piers 22 and 56, cracks have been noted at the top of the insitu concrete, which open and close with the passage of the carriage. Intrusive investigations were undertaken to determine the presence of any loss of structural continuity in hogging bending. The observation was the structure is continuous at both piers. These results have been incorporated within the analysis from which conclusions are drawn.

5.2 Wind loads

The load combinations, wind speeds, carriage drag coefficient 'C_D', and carriage dimensions are defined in the Schedule of Basic Assumptions, which in turn is based on statements in the original design certificate.

BD 37/01 calls for four possible combinations of transverse 'P_t', longitudinal 'P_L', and vertical 'P_v' wind loads: (a) P_t alone; (b) P_t in combination with $\pm P_v$; (c) P_L alone; and (d) $0.5P_t$ in combination with $P_L \pm 0.5P_v$. In this appraisal, the effects of P_L have been neglected because (i) the only part of the structure in the longitudinal direction that could attract wind drag forces are the piers and crossheads, and (ii) longitudinal wind loads on the carriage would be transmitted to the haul or counter ropes and have a negligible effect on carriage weight distribution. The effects of P_v have also been neglected due to the structure being open in plan. Only transverse wind loads 'P_t' [combination (a)] are therefore considered in this appraisal.

In calculating the drag coefficient for the transverse wind load on the main rail support beams, the width 'b' was taken as the width of the top flange. The resulting 'b/d' ratios resulted in high C_D values for the rail support beams: 2.3, 2.4, and 2.75 for beam types 1, 2, and 3, respectively. As the gap between the rail support beams is greater than 1.0 m, BD 37/01 does not give any allowance for shielding, and the full transverse wind load must therefore be applied to each beam.

The resulting transverse wind loads, including the appropriate γ_{fL} factor, for the BS EN 13107 'in operation' and 'out of operation' cases are as follows:

EN 13107 classification	Area of structure	Wind speed (m/s)	Carriage total transverse wind load (kN)	Beam transverse wind uniformly distributed load (kN/m per beam)		
				Type 1	Type 2	Type 3
In operation (Principal wind case governs)	All areas	35.0	34.3*	1.5	1.6	1.9
Out of operation (Accidental wind case governs)	Area 1	57.5	92.6	4.1	n/a	n/a
	Area 2	58.5	95.9	4.3	4.4	n/a
	Area 3	61.2	104.7	4.6	4.8	5.0
	Area 4	65.0	118.1	5.3	5.5	6.3
	Area 5	68.5	131.3	5.8	n/a	n/a
	Area 6	72.4	146.6	6.5	n/a	n/a

** Additional transverse carriage loads due to nosing and centrifugal effects apply (not included here)*

Table 5-1 Factored transverse wind loads using EN 13107 classification

5.3 Live loads

The relevant load combinations and carriage weights, axle spacings, and centres of mass are defined in the Schedule of Basic Assumptions, which in turn is based on statements in the original design certificate.

Effects due to acceleration and deceleration were found to be negligible, hence due to symmetry loads on front and rear bogies are assumed to be the same. The bogies also maintain equal load on each axle, hence all the loads down each side of the carriage are the same. Loads might be different on the two sides owing to lateral wind and centrifugal effects.

The requirement for centrifugal effects is governed by EN 13796 which specifies transverse acceleration to be taken as 0.1g. This is more onerous than would be obtained using acceleration=V^2/R using the design speed of 10m/s. This requirement also means the same centrifugal effects apply to all curve radii.

The dynamic amplification factor is included in load combinations where the carriage is moving, but is excluded where the carriage is static.

The appropriate γ_{fL} factors are included in all loads tabulated below.

The load combinations and maximum calculated wheel loads are:

BD 37/ 01 combination	Descriptive name	Carriage condition	Area of structure	Wind speed (m/s)	Wheel load (kN)	
					SLS	ULS
comb 1	Live only	Full payload and moving	All areas	0 *	43.0 or 47.9 †	54.7 or 61.0 †
comb 2	Principal	Full payload and moving	All areas	35	45.8 or 50.3 †	54.2 or 59.7 †
	Emergency	5t kentledge only and static	All areas	50	38.1	44.3
	Storm	Not present	Area 1	57.5	n/a	n/a
			Area 2	58.5	n/a	n/a
			Area 3	61.2	n/a	n/a
			Area 4	65.0	n/a	n/a
			Area 5	68.5	n/a	n/a
			Area 6	72.4	n/a	n/a
	Accidental	5t kentledge only and static (clamped to tracks)	Area 1	57.5	42.5	49.2
			Area 2	58.5	43.2	49.9
			Area 3	61.2	44.9	51.8
			Area 4	65.0	47.5	54.7
			Area 5	68.5	50.1	57.6
			Area 6	72.4	53.1	60.8

** No wind loads are included in BD 37/01 combination 1*

† Higher loads are due to centrifugal loads, hence these apply to beam types 2 and 3 only

Table 5-2 BD 37/01 Load combinations and maximum wheel loads

BS EN 13107 divides variable loads into "in operation" and "out of operation" loads. Using this the above table is simplified into the following:

EN 13107 classification	Area of structure	Wheel load (kN)	
		SLS	ULS
In operation	Areas 1, 5 and 6	45.8	54.7
	Areas 2, 3 and 4	45.8 or 50.3 †	54.7 or 61.0 †
Out of operation	Area 1	42.5	49.2
	Area 2	43.2	49.9
	Area 3	44.9	51.8
	Area 4	47.5	54.7
	Area 5	50.1	57.6
	Area 6	53.1	60.8

† Higher loads are due to centrifugal loads, hence these apply to curved track. Curves are supported on beam types 2 and 3 only and are found in areas 2, 3 and 4 only.

Table 5-3 Maximum wheel loads using EN 13107 classification

In most cases wheel loads are normal to the track, as longitudinal loads are balanced by the haul ropes as shown in Figure 5-5. However, braking and clamping can produce loads longitudinally as shown in Figure 5-6. It is assumed in all the above load cases except the accidental case that the loads are normal to the track, and for the accidental case the loads are vertical, as illustrated below.

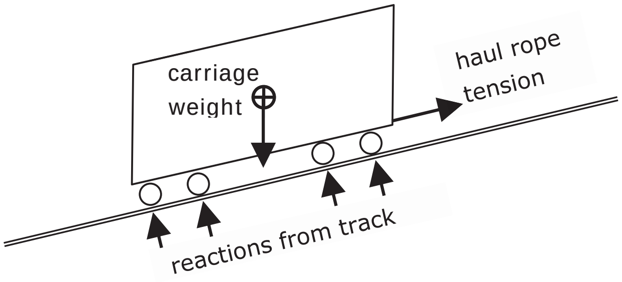


Figure 5-5 Load diagrams for all load cases except accidental

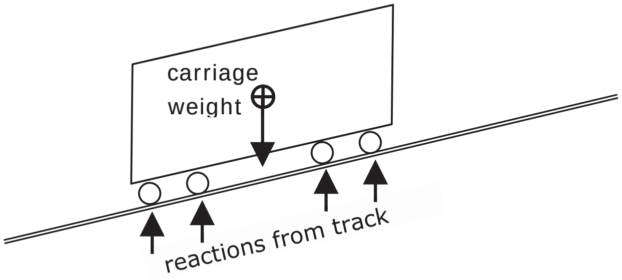


Figure 5-6 Load diagrams for all accidental load case

The accidental load case therefore introduces an axial load into the track. This will eventually be transferred into the structure via the rail fixings, but this transfer may not take place until downhill of the carriage. Hence the additional axial load caused by the accidental load case is not included where it is beneficial to structural strength.

6 Appraisal Results

6.1 Deflections and rotations

Limiting deflections and rotations are given in BS EN 13107 at the Serviceability Limit State. Deformation is limited for variable actions plus any time dependent deformations due to permanent actions, i.e. live load plus creep only.

The vertical deflection limit is L/600 where L=horizontal span. It is not stated whether this applies in operation or out of operation, but it is assumed to apply only to the in operation case (note that the maximum vertical deflections in the out of operation case are not necessarily higher).

Selected results, including the most critical location, are as follows:

Span between piers	Vertical deflection	Limiting deflection	Result
In operation			
22 to 23 (typical curve)	27.9 mm	30 mm	ok
52 to 53 (passing loop)	30.4 mm	30 mm	1% overload
above 93 (top span)	41.8 mm	30 mm	39% overload

Table 6-1 Vertical deflection results

The transverse horizontal deflection limit for piers is H/300 in operation or H/100 out of operation, where H is the "relevant height". This term is not defined, but in this appraisal H is taken as the height of the pier bearings above the top of the base slab. Selected results, including the most critical location, are as follows:

At piers	Transverse deflection	Limiting deflection	Result
In operation			
46 (approx. 6m high)	4.0 mm	20.3 mm	ok
51 (approx. 5m high)	2.7 mm	17.4 mm	ok
91 (approx 6m high)	2.6 mm	20.1 mm	ok
Out of operation			
46 (approx 6m high)	8.1 mm	60.9 mm	ok
51 (approx 5m high)	6.3 mm	52.2 mm	ok
91 (approx 6m high)	9.9 mm	60.3 mm	ok

Table 6-2 Transverse deflection results in operation

The maximum allowable rotation at pier locations in the direction of the track is 0.003 radians in operation, with no limit out of operation.

Selected results, including the most critical location, are as follows:

At piers	Rotation	Limiting deflection	Result
In operation			
40 (typical span)	0.0032 radians	0.003 radians	6% overload
53 (passing loop)	0.0039 radians	0.003 radians	30% overload
93 (top span)	0.0042 radians	0.003 radians	41% overload

Table 6-3 Rotation results in operation

Note that the greatest beam deflections and rotations occur at the span immediately below each anchor block, where the top connection is at a movement joint and thus effectively pinned rather than continuous (e.g. the span above pier 93). Typical spans with continuous supports on both ends (e.g. the span above pier 40) in some cases still fail to meet the BS EN 13107 rotation criteria, but with reduced overload percentages.

6.2 Main beam bending and shear

Bending and shear criteria are given in highway bridge assessment standard BD 44/15. Only Ultimate Limit State criteria are considered in this appraisal. All bending and shear limits apply both in operation and out of operation. Criteria are given for pure bending, pure shear, and combined bending and shear.

Results are quoted following the convention of highway bridge assessment standard BD 21/01, in which S_A* represents the load effects including γ_{fL} and γ_{f3}, and R_A* represents the resistance including γ_m and accounting for the current condition. In this structure the condition factor is taken as 1.0 for all elements.

Axial thrust in the beams will enhance the bending and shear resistance. This means that sections at the lower end of each continuous structure near the anchor block will have higher resistance than sections at the upper end of each continuous structure near the expansion joints. This enhancement is included in some of the results as discussed below.

Bending has been considered at midspan and at supports. Although there are laps and splices meaning that the resistance will vary along the beam length, the laps are by inspection well away from the critical locations, so only sag at midspan and hog at piers need be considered.

Selected results for sag bending, including the most critical location, are as follows:

Span between piers	Sag moment S_A*	Resistance R_A*	Result
In operation			
22 to 23 (typical curve)	720 kNm/beam	1140 kNm/beam†	ok
51 to 52 (passing loop)	880 kNm/beam	1120 kNm/beam†	ok
above 93 (top span)	780 kNm/beam	1100 kNm/beam†	ok
Worst case of In Operation or Out of operation			
22 to 23 (typical curve)	In operation governs		n/a
51 to 52 (passing loop)	In operation governs		n/a
above 93 (top span)	840 kNm/beam	1100 kNm/beam†	ok

† *Resistance without axial load. Resistances could be increased due to axial load, but this has not been necessary.*

Table 6-4 Sag bending results

Note that spans in the tapering parts of the passing loop experience greater sag moments because some beams carry not only one rail but a proportion of the opposite rail.

In hog, it was found that the lap lengths of the top reinforcement at the piers are critical. The lap length is 950mm but since laps are less than 150mm clear distance apart they should be 1550mm for full strength. Hog resistance has therefore been reduced accordingly. This only affects type 1 beams, since type 2 and type 3 beams are connected using couplers.

It was also found that the connections to the anchor blocks are critical. The beams are connected to the anchor blocks by a rigid fixed ended connection. The bars in the top of the beam ends project into the anchor block, but the anchor block does not contain a matching area of reinforcement to lap with them, nor is the lap length of a sufficient length. Hog resistance has therefore been reduced accordingly.

Selected results for hog bending, including the most critical location at anchor blocks (above anchor block 0) and at piers (pier 93), are as follows:

At piers	Hog moment S_A*	Resistance R_A*	Result
In operation			
0 (anchor block)	630 kNm/beam	440 kNm/beam	42% overload
52 (passing loop)	480 kNm/beam	680 kNm/beam	ok
93 (top pier)	420 kNm/beam	450 kNm/beam	ok
Worst case of In Operation or Out of operation			
0 (anchor block)	In operation governs		n/a
52 (passing loop)	In operation governs		n/a
93 (top pier)	460 kNm/beam	450 kNm/beam	3% overload

Table 6-5 Hog bending results

Hog bending resistance at all piers was found to be sufficient in the 'in operation' case. In the 'out of operation' cases, hog bending was found to be overloaded at the pier 93 only as shown above.

As well as at anchor block 0, the spans above anchor blocks 14, 29, 65 and 78 were also found to be overstressed, but generally the anchor blocks higher up are less critical since they have greater axial loads enhancing the resistance. The span above anchor block 48 is much shorter and is not overloaded.

Moment redistribution was considered for the spans above anchor blocks, since these have surplus sag resistance, but unfortunately BD 44/15 does not permit moment redistribution in this case.

The structure therefore has insufficient resistance for hog bending in the spans above anchor blocks. To comply with BD 44/15 in the critical 'in operation' span (above pier 0), the factored wheel loads would have to be reduced to approximately 38 kN. To obtain this wheel load and comply with BD 37/01 combination 1 would require an occupancy limitation within the carriage of approximately 30 persons, assuming 80 kg per person.

Shear has been considered throughout the length of the beam. According to BD 44/15 the shear resistance may be based on either of 2 methods: a method derived from BS 5400-4 in which a component of resistance due to concrete shear is added to a component of resistance due to shear reinforcement, or the method used by Eurocodes in which the resistance is derived entirely from shear reinforcement with a variable compression strut angle. The resistances presented here are based on whichever method gives the greater resistance at that location as permitted by BD 44/15.

In determining the shear resistance it has been found that the shear reinforcement has the following weaknesses:

- The links are not vertical for the full height of the section. In the truss analogy used in reinforcement design, the vertical shear must be carried between the top and bottom of the section (see Figure 6-1). Hence the ability of the link to carry vertical shear is reduced by sinΦ where Φ is the angle of the link to the vertical in the cross section. Shear resistance has therefore been reduced accordingly.

- In the scarf joint the link effectiveness is reduced because short vertical laps are used. As shown in Figure 4-6 the five bars in the joint have limited lap lengths. Examination of photographs such as Figure 4-2 it is estimated that two bar laps are around 250mm, the third is 150mm, the forth 100mm and the last is negligible. Since laps are less than 150mm clear distance apart they should be 280mm for full strength. In addition at type 2 and type 3 beams some lapped links are displaced and inclined in order to fit around couplers. Shear resistance has therefore been reduced accordingly.

- In the scarf joint the bend in the upper half of the lapped link is not anchored around a bar, see Figure 4-6, and therefore has a tendency to tear through the concrete section. This would limit the strength in the bar, but it was found that the above limits are more critical, so this weakness does not govern the shear resistance.

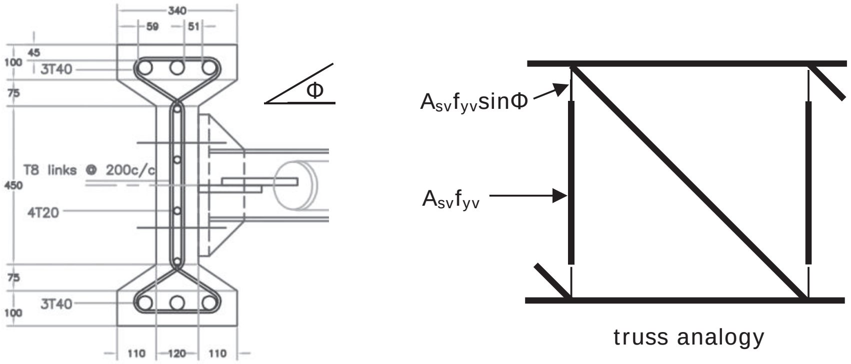


Figure 6-1 Effect of link shape on shear resistance

The scarf joint itself poses some difficulty with determining shear resistance. The existence of a diagonal construction joint, which in some cases is cracked, is highly unusual and is not something that is addressed by BD 44/15. Hence there is a risk that there is a potential shear failure mechanism which is not taken into account in the calculated shear resistance.

Shear resistance has been calculated including shear enhancement. According to BD 44/15 this may be applied to the component of resistance due to concrete shear up to 3d from the support, where 'd' is the effective beam depth.
BD 44/15 also states that sections need not be assessed for shear within d of a support, hence only the upper limit due to web crushing applies. Taking account

of the assumed spacing of links shown in Figure 4-10, the profile of shear resistance along a half span is as follows:

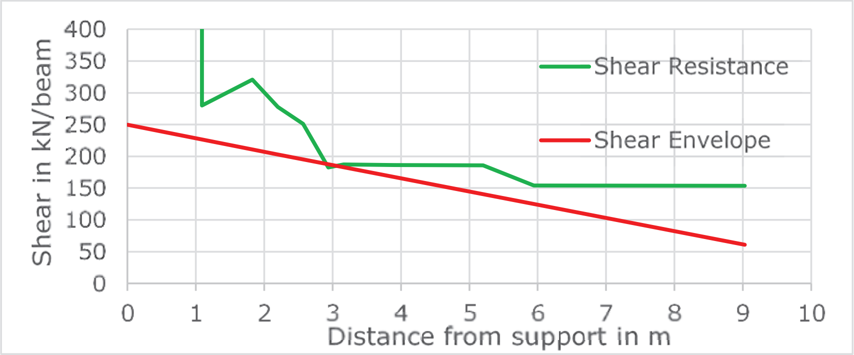


Figure 6-2 Typical profile of shear resistance against applied shear envelope

This shows that shear resistance dips at around 1m from the support. This is due to the shortcomings in the shear reinforcement at the scarf joint. However shear enhancement close to the support boosts the resistance in this zone. Shear resistance dips again at 2.9m and 6m from the support - both locations are distance 'd' beyond positions where shear link spacing changes. From the above it is clear that the critical position for shear in the span is at 2.9m from the support.

Selected results for shear, including the most critical location, are as follows:

Span between piers	Shear near pier	Shear load effect S_A*	Shear resistance R_A*	Result
In operation				
22 to 23 (typical curve)	22	200 kN/beam	160 kN/beam	23% overload
56 to 57 (passing loop)	56	270 kN/beam	220 kN/beam	23% overload
93 to tunnel (top span)	93	195 kN/beam	195 kN/beam	ok
Worst case of In Operation or Out of operation				
22 to 23 (typical curve)	22	In operation governs		n/a
55 to 56 (passing loop)	55	In operation governs		n/a
93 to tunnel (top span)	93	213 kN/beam	195 kN/beam	9% overload

Table 6-6 Shear results at 2.9m from support

Shear 2.9m from supports therefore limits the strength of the structure. To comply with BD 44/15 in the critical 'in operation' spans the wheel loads would have to be reduced to approximately 47 kN. To obtain this wheel load and comply with the appraisal standards, would require an occupancy limitation within the carriage of approximately 50 persons, assuming 80 kg per person. It is noted that the critical spans for shear are where there are curves and in the passing loop where there are higher loads per beam.

Combined shear and bending requires an additional area of longitudinal reinforcement to carry half the shear load in addition to that required for bending. However, this is limited to the peak reinforcement required for bending, and hence this will not give a worse condition than for bending alone.

6.3 Main beam bracing and diaphragms

The bracing between the main rail support beams is made of structural steel. Strength criteria are therefore given in highway bridge assessment standard BD 56/10, which refers to BS 5400-3 for most clauses. The diaphragms are reinforced concrete and strength criteria are therefore given in highway bridge assessment standard BD 44/15. Only Ultimate Limit State criteria are considered in this appraisal. All bracing and diaphragm capacity limits apply both in operation and out of operation.

Bracing loads are governed by the transverse wind loads applied to the structure in a given span (Figure 6-3). The bracing acts as truss web members to distribute the transverse loads into the diaphragms and guided bearings at the piers. The main rail support beams act as the truss chord members.

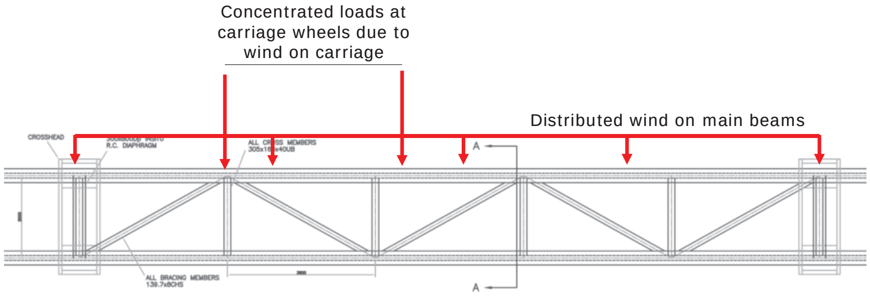


Figure 6-3 Plan view of typical span with applied transverse wind loads

The cross-bracing was analysed for one critical load case: the accidental wind case in area 6 (where wind speeds are highest), with the maximum span length of 18.4 m. Critical axial loads of 394 kN in the diagonals and 97 kN in the cross-members were obtained (critical loads can be either tension or compression depending on the direction of the wind).

The diagonals are 139.7x8 CHS sections and were assessed as having a compressive capacity of 496 kN, giving a utilisation S_A*/R_A* of 0.79. The cross-members are 305x165x40 UB sections and were assessed as having a compressive capacity of 900 kN, giving a utilisation S_A*/R_A* of 0.11.

The cross-bracing connection capacities were also assessed where possible, but limited information on the connection details is provided in the design drawings. The drawings show 4 No. M24 8.8 bolts used at each connection, which were found to be sufficient in all cases. Possible overloads of the connection plate welds or net sections were identified, but these required estimates of the dimensions and details to be made. As the overloads are uncertain, relatively minor, and only apply to the accidental wind case in the upper areas of the structure, this is not considered to be an area of significant concern.

The bolted connections for the UB cross-members were identified as having slotted holes in the direction of load, but bolt preload was not specified in the design drawings. It is therefore possible that these connections are unable to transmit any significant load without slippage. However, the UB cross-members are not essential load-carrying members, as the loads in the diagonals do not change if the cross-members are removed and the rail support beams have adequate stability to span between diagonal connection points.

In typical spans, the diaphragms act as tension or compression members to carry transverse loads to the guided bearing. Critical loads in the diaphragms in typical spans are therefore governed by the same accidental wind case that governs the cross-bracing. However, in the passing loop, the diaphragms at piers 52 and 53 also act in bending to resist the axial thrust of the additional rail support beams that are terminated at those piers. In all cases, the diaphragms were found to have adequate strength to resist the applied loadings.

6.4 Bearings

According to the Schedule of Basic Assumptions it is assumed that the bearings have sufficient load capacity. However, it is noted that the high wind loads in the accidental combination (out of operation) may lead to uplift on the upwind bearing, and it is assumed the bearing has no uplift capacity.

Manufacturing details or original load ratings for either the free or guided sliding bearings are not available and these particular bearing models are no longer manufactured. However based on measurements and assumptions, bearing resistance has been calculated to BS 5400-9. The compression resistance is governed by limiting stresses on PTFE and elastomer at the Serviceability Limit State as follows:

Free sliding Bearing	Utilization Ratio at SLS (BS5400-9)
PTFE stress limit	1.47 (47% overstress)
Elastomer pad stress limit	1.65 (65% overstress)

Table 6-7 Free sliding bearing utilization ratios

Lateral bearing resistance has not been determined, but it is known that guide bearings of this type generally require a significant vertical load in order to resist large lateral loads - typically the lateral load should not exceed 25% of the coexistent vertical load. In this case the guide bearing could be subjected to a low vertical load and large lateral load, hence it is assumed the bearings are overloaded by lateral loads.

The bearings are approximately seventeen years old. BS EN 13107 states a design life of bearings as twenty years. Numerous bearings exhibit significant wear of PTFE sliding surface and in some locations no visible PTFE sliding pad was noted.

Numerous bearings exhibit signs of longitudinal movement approaching and exceeding their allowable limits. In colder temperatures contraction of the main longitudinal beams result in bearing contact surfaces exceeding their support limit. This increases stress on the bearing components and risks introducing additional horizontal actions when thermal expansion occurs.

An appraisal of bearing movements has been carried out based on observations of bearing positions noted by ADAC Structures at reference temperatures. These observations were monitored on site by video monitoring equipment and further confirmed by inspections. Appendix B reviews the monitoring of bearing movement and finds that the thermal movements at the bearings are broadly in line with what would be expected, indicating that the bearing articulation is acceptable.

Results from site surveys and measurements suggest bearings absolute and relative positions vary widely along the viaduct's length. A representation of distance from theoretical centre of relative positions at piers is shown in figure 6-4.

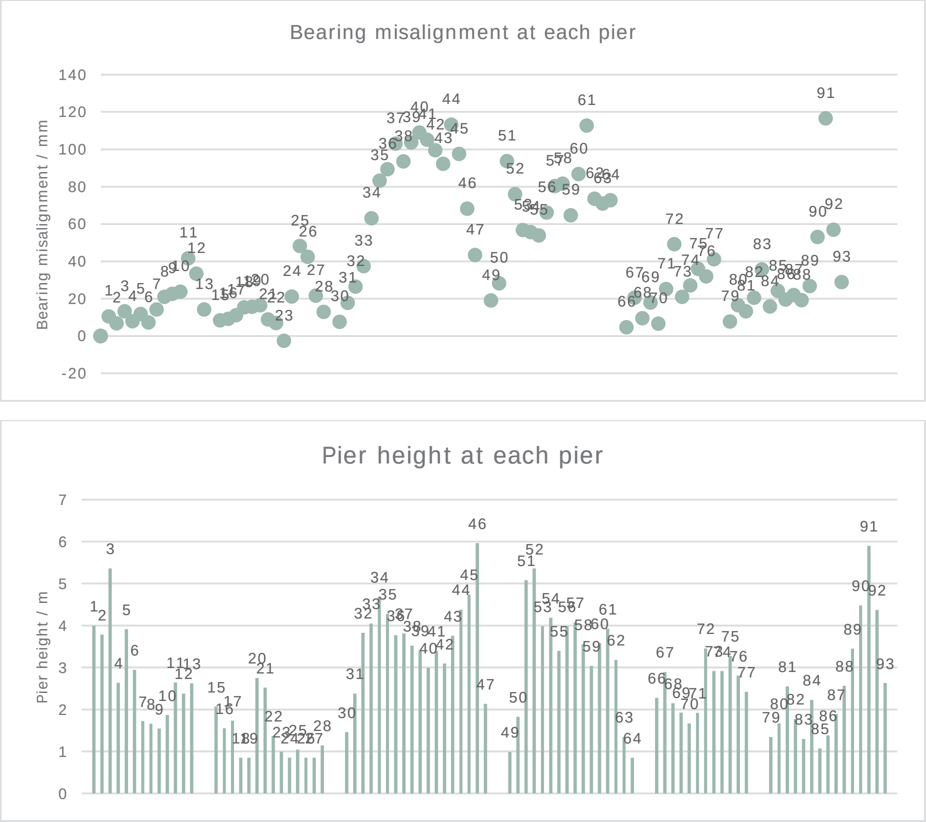


Figure 6-4 Longitudinal displacement of bearings along viaduct length at a reference temperature of 0°C and pier heights for comparison.

Due to bearing displacements relative to supporting main beams, the allowable movement range at some pier locations is severely reduced. This bearings displacement is consistently towards the slope of the mountain and thus affects the allowable movement range during contraction of the main beams in cooler weather.

Review of ADAC's bearing report, Aug 2018, and bearing monitoring confirms that some bearings exhibit partial loss of contact at temperatures as high as +5ºC. At lower temperatures many bearing locations experience some degree of contact surface overhang which typical increases with correlation to pier height. Figure 6-5 shows the correlation between measured and theoretical bearing positions relative to pier height. Monitoring has confirmed that although broadly in line with expected, measured results are consistently less than theoretical predictions.

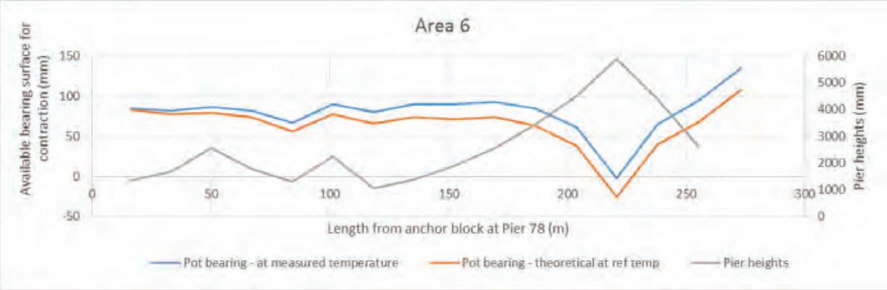


Figure 6-5 Longitudinal displacement of bearings, measured (at ambient temperatures of 8.9 to 20.8°C) and theoretical (at reference temperature 0ºC).

COWI undertook a review of survey information of the lower half of the viaduct. Absolute level information was available for pier top and bearing bottom plate. Key observations from this review are noted below;

- Transversely, the pier crosshead levels exhibit significant differences.
- The bearings are more level than the pier tops. However a difference in level of 20mm between bearings is not uncommon.
- The difference between intended and actual top of pier levels varies generally by a range of 60mm suggesting a typical as built tolerance of +/-30mm in pier top levels.
- Deducting the effect of the intended vertical alignment curvature from an interpolated straight line shows a typical bearing level tolerance of +/-10mm up to pier 32, but +/-20mm above pier 32.

It is assumed the grout in the bearing pack was used to regulate differences in levels across the pier tops and thus the grouted bearing pads vary in depth along the viaducts length. Data was interrogated for possible differential settlement between piers though it has not been possible to confirm any evidence of this. Any potential settlement in the order of 10mm would be lost in the construction tolerances assumed and permitted on a structure of this nature.

Operational tolerance requirements for the funicular rail track and cableway were likely regulated by the rail plinth grouting at rail support points. Due to this possibility no judgement about whether any ground movements have occurred based on this data would be accurate. If the track was laid to a smooth alignment, data on the current track alignment is the only way to determine if ground movements have occurred.

6.5 Pier crossheads

Strength criteria are given in highway bridge assessment standard BD 44/15. Only Ultimate Limit State criteria are considered in this appraisal.

The pier crossheads are subject to bearing loads and have to transfer the bearing loads to the pier columns. The pier crossheads therefore act as corbels and have been analysed using strut and tie systems.

The guide bearings impose lateral loads on the crossheads, but due to the inclination of the bearings, longitudinal loads as well as vertical loads are imposed on the crossheads. The crossheads have large bars with good strength to resist vertical and lateral loads but it is the longitudinal loads that govern, because the resistance is limited by relatively small shear links.

Selected results for the links, including the most critical location, are as follows:

At piers	Hog moment S_A*	Hog resistance R_A*	Result
In operation			
56	21 kN/bar	22 kN/bar	ok
77	22 kN/bar	22 kN/bar	ok
79	21 kN/bar	22 kN/bar	ok
Worst case of In Operation or Out of operation			
56	24 kN/bar	22 kN/bar	11% overload
77	30 kN/bar	22 kN/bar	36% overload
79	32 kN/bar	22 kN/bar	44% overload

Table 6-8 Results of strut and tie analysis for crosshead links

In fact most of the pier crossheads in the upper half of the structure fail under out of operation loads, due to the inclination of the bearings.

The crossheads are fixed down to the columns solely by prestressed bars. The prestress gives sufficient shear resistance against lateral and longitudinal bearing loads, but is critical in bending due to the lateral bending moments generated by lateral bearing loads and unequal vertical bearing loads, and longitudinal moments generated by longitudinal bearing loads.

Selected results for bending on the prestressed interface, including the most critical location, are as follows:

Pier	Minor axis bending		Major axis bending		Combined biaxial factor	Result
	S_A*	R_A*	S_A*	R_A*		
In operation						
56	144 kNm	360 kNm	366 kNm	1140 kNm	0.65	ok
77	170 kNm	360 kNm	193 kNm	1140 kNm	0.61	ok
79	170 kNm	360 kNm	193 kNm	1140 kNm	0.60	ok
Worst case of In Operation or Out of operation						
91	116 kNm	360 kNm	789 kNm	1140 kNm	0.87	ok
90	137 kNm	360 kNm	737 kNm	1140 kNm	0.89	ok
75	137 kNm	360 kNm	824 kNm	1140 kNm	0.95	ok

Table 6-9 Crosshead fixing biaxial bending results

Hence the crosshead fixings are adequate.

6.6 Pier columns

Bending and shear criteria are given in highway bridge assessment standard BD 44/15. Only Ultimate Limit State criteria are considered in this appraisal. All bending and shear limits apply both in operation and out of operation.

Bending and shear will occur about both axes. The inclination of the bearings produces bending and shear in the direction of the track, see Figure 6-6. Lateral bending and shear is generated by lateral wind loads, track nosing loads, and loads arising from track curvature.

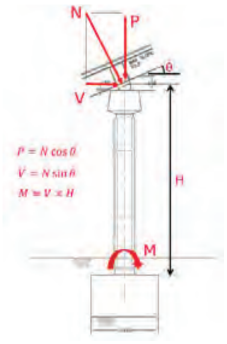


Figure 6-6 Calculation of pier minor axis (i.e. in the longitudinal direction of the track) loads as a result of normal reactions on the bearings

Results are quoted following the convention of highway bridge assessment standard BD 21/01, in which S_A* represents the load effects including γ_{fL} and γ_{f3}, and R_A* represents the resistance including γ_m and accounting for the current condition. In this structure the condition factor is taken as 1.0 for all elements.

Selected results for bending, including the most critical location for piers with base types 4, 5, and 6, are given in Table 6-10. Pier bending results presented here use a linear combination of the minor axis bending utilisation (i.e. S_A*/R_A*) and major axis bending utilisation to determine the combined biaxial factor. The enhancement of bending resistance due to axial load is neglected in the results given in Table 6-10. The overload percentages presented are therefore conservative. However, the effects of full biaxial moment resistance and axial load interaction have been evaluated separately, e.g. as shown in Figure 6-7. The increased resistance of the actual resistance envelope is, in the majority of cases, insufficient to significantly lower the overload percentage, relative to what would be calculated using the simplified resistance envelope.

Pier	Minor axis bending		Major axis bending		Combined biaxial factor	Result
	S_A*	R_A*	S_A*	R_A*		
In operation						
91	1205kNm	809 kNm	755 kNm	3200kNm	1.73	73% overload
90	901 kNm	741 kNm	604 kNm	2817kNm	1.43	43% overload
75	748 kNm	634 kNm	472 kNm	2303kNm	1.38	38% overload
Worst case of In Operation or Out of operation						
91	975 kNm	809 kNm	2970kNm	3200kNm	2.13	113% overload
90	729 kNm	741 kNm	2392kNm	2817kNm	1.83	83% overload
75	603 kNm	634 kNm	1690kNm	2303kNm	1.68	68% overload

Table 6-10 Pier biaxial bending results

Only taller piers (over 2500mm from base top to crosshead beam top) with sufficient bearing inclination (greater than 11°) are overloaded in the 'in operation' case. All 25 piers overloaded in the 'in operation' case therefore occur at or above pier 39, as the lower sections of the railway have less inclination than the upper sections.

Operational limits that would allow the critical 'in operation' pier (pier 91) to comply with BD 44/15 cannot be obtained while still complying with loading criteria from BD 37/01. The mass of the empty carriage (14,900 kg) is sufficient to cause a minor axis bending overload of approximately 25% in Pier 91.

M_y-M_z interaction diagram – Pier 91

——— Simplified resistance envelope – linear combination with no axial load

——— Actual resistance envelope

(axial load corresponding to out of operation load case)

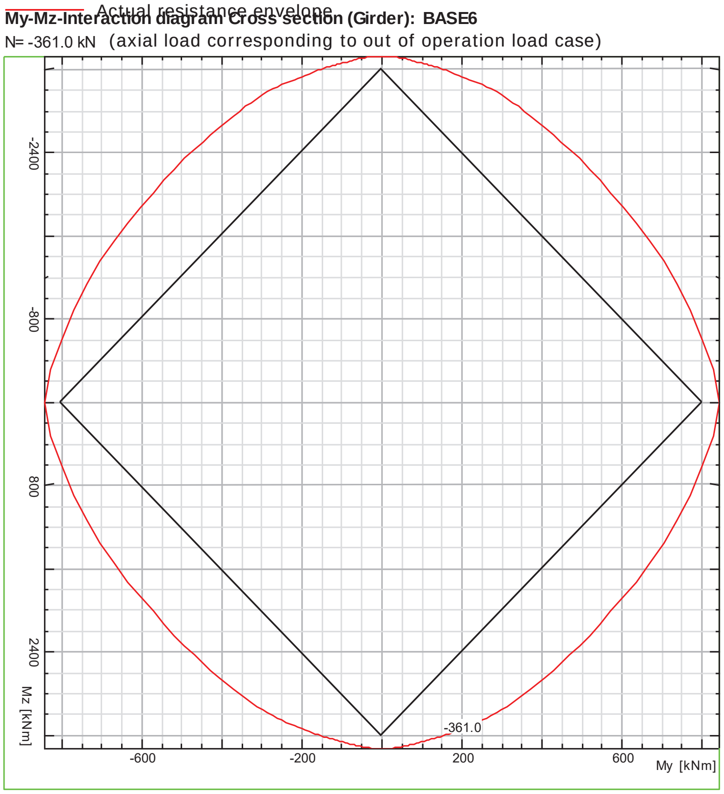


Figure 6-7 Simplified and actual pier biaxial bending resistance envelopes for Pier 91

Pier shear resistances were calculated including the enhancing effects of the coincident axial load and short pier heights (where the pier stem is less than 3 times the effective depth to tension reinforcement 'd' in height). Selected results for pier shear, including the most critical location, are as follows:

Pier	Minor axis shear		Major axis shear		Combined biaxial factor	Result
	S_A*	R_A*	S_A*	R_A*		
In operation						
54	169 kN	528 kN	160 kN	444 kN	0.68	ok
51	161 kN	555 kN	180 kN	473 kN	0.67	ok
57	180 kN	514 kN	141 kN	455 kN	0.66	ok
Worst case of In Operation or Out of operation						
90	145 kN	518 kN	406 kN	423 kN	1.24	24% overload
92	136 kN	504 kN	407 kN	423 kN	1.22	22% overload
91	145 kN	537 kN	406 kN	467 kN	1.14	14% overload

Table 6-11 Pier biaxial shear results

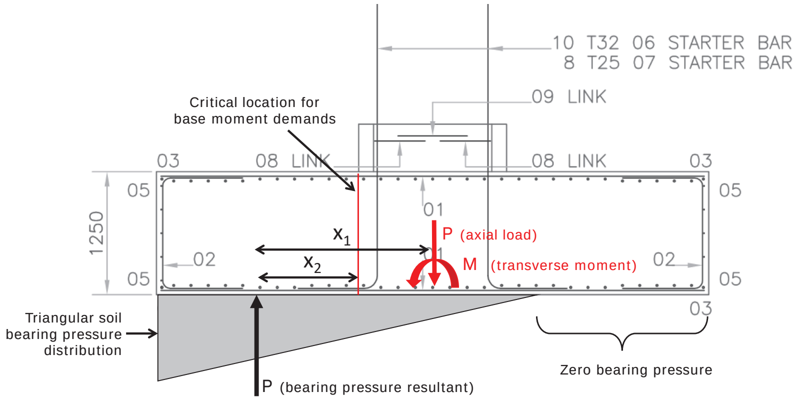
Note that pier shear demand in individual axes is also governed by the impact load case, calculated as per the Schedule of Basic Assumptions, with a design value S_A* = 415kN. Taking into account the applicable height of the impact loading and the enhancement of the pier shear resistance near the support, all piers have sufficient capacity to resist this loading in both axes. The bending induced as a result of the impact load has a maximum value of S_A* = 1080kNm. All piers have sufficient capacity to resist this bending moment if applied in the major axis. However, the impact load is sufficient to cause minor axis bending overload in all piers tall enough to permit the full impact bending to be applied. The overloads due to minor axis bending can be as high as 70%, but it is noted that this load case requires impact on the piers in the longitudinal direction of the railway.

6.7 Pier base slab

Bending and shear criteria are given in highway bridge assessment standard BD 44/15. Only Ultimate Limit State criteria are considered in this appraisal. All bending and shear limits apply both in operation and out of operation.

Results are quoted following the convention of highway bridge assessment standard BD 21/01, in which S_A* represents the load effects including γ_{fL} and γ_{f3}, and R_A* represents the resistance including γ_m and accounting for the current condition. In this structure the condition factor is taken as 1.0 for all elements.

Base slabs are generally narrow in the direction of the track but wide transversely. Since reinforcement is the same in both directions, by inspection bending in the base slab is critical in the transverse direction. A trapezoidal (or triangular where applicable) bearing pressure diagram under the base slab has been assumed and hence bending moments have been determined at the face of the pier, see Figure 6-8. For the out of operation case, base overturning failure in the transverse direction was determined to occur at piers 81, 90, 91, 92, and 93. Bending moments at the pier face are therefore not provided at these piers, but they should be considered as overloaded in the out of operation case.



$$M = P \times x_1$$
$$M_{base} = P \times x_2$$

Figure 6-8 Base bending moment calculation procedure

Selected results for base bending, including the most critical location, are as follows:

At piers	Bending moment S_A*	Bending resistance R_A*	Result
In operation			
46	963 kNm	1875 kNm	ok
51	945 kNm	1875 kNm	ok
91	782 kNm	1875 kNm	ok
Worst case of In Operation or Out of operation			
51	2253 kNm	1875 kNm	20% overload
46	1989 kNm	1875 kNm	6% overload
57	1776 kNm	1875 kNm	ok

Table 6-12 Base slab bending results

The minimum possible base shear resistance (R_A*) was determined to be over 1200kN for the most critical base type (type 6). The maximum possible shear loading on the base (S_A*) is equal to the maximum vertical load transferred from the base to the underlying soil, which was determined to be less than 1200kN in all load cases (both in and out of operation). The bases therefore were assessed as having sufficient shear capacity at all piers.

6.8 Pier foundations

The foundation assessment has been carried out for In Operation and Out of Operation load cases. For both load cases, foundation pressures have been derived and checked against ultimate bearing capacity at 4 locations along the length of the viaduct structure.

Piers 22, 44, 61 and 91 have been selected on the basis that they represent the variation in the ground conditions encountered along the length of the viaduct as described in Technical Note Ref: TN-3-002 Ground Investigation Report, see Appendix C. A summary of the prevailing ground conditions and characteristic soil strength properties adopted in the foundation assessment are summarised in the following table:

Pier	Foundation Subgrade	Characteristic Bulk Unit Soil Weight (kN/m^3)	Characteristic Internal Soil Friction (Degrees)	Characteristic Soil Cohesion (kN/m^2)
22	Glacial Deposits	18	35	0
44	Alluvium	17	32	5

61	Head Deposits	18	38	0
91	Weathered Granite	19	42	5

Table 6-13 Summary of Foundation Subgrade

From the 2018 trial pit investigation the foundation backfill material has been assessed as loose to medium dense granular fill comprising sand and gravel with trace silt and clay and varying amounts of cobbles and boulders. A summary of the characteristic soil strength properties of the backfill material adopted in the foundation assessment are summarised in the following table:

Foundation Backfill	Characteristic Bulk Unit Soil Weight (kN/m^3)	Characteristic Internal Soil Friction (Degrees)	Characteristic Soil Cohesion (kN/m^2)
Granular Fill	18	30-34	1

Table 6-14 Summary of Foundation Backfill

The guidelines set out in BS EN1997-1:2015 (Eurocode 7-Geotechnical Design) have been adopted in the foundation assessment. In keeping with UK practice Design Approach 1 has been adopted which requires two separate combinations of partial factors on actions and soil strength properties to be applied when checking that the ultimate limit state is satisfied.

The partial load and material factors applied in the assessment are listed in the following table:

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

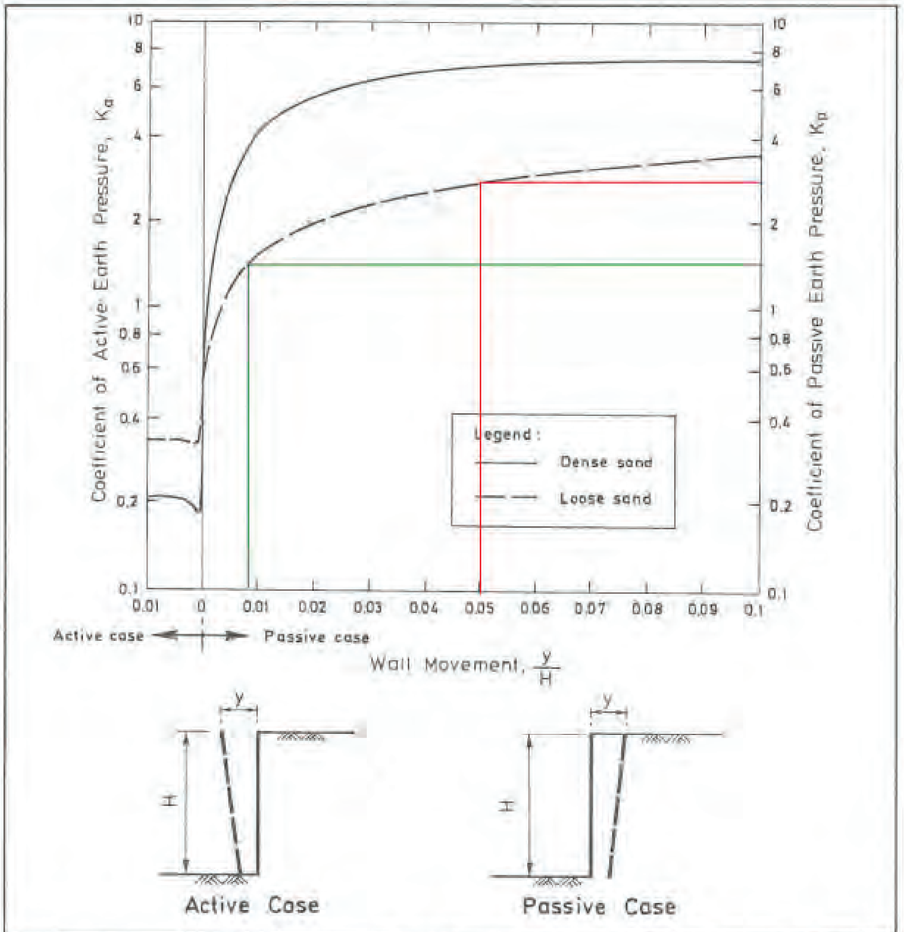
Notes: DA1 means Design Approach 1

C1 and C2 mean Combination 1 and Combination 2 respectively

Table 6-15 Summary of Partial Load and Material Factors Applied

In the derivation of foundation pressure under operational loads it is assumed that only 50% of the maximum available passive soil resistance is mobilised on any given side of the foundation pad.

As illustrated in Figure 6-9, in the case of loose sands, (representative of the backfill material) to mobilize full passive resistance of the soil placed against the foundation pad would require the foundation (depth 1.25m) to displace by approximately 60mm (i.e. y=0.05*1.25=0.062m).



Soil Type	Movement for the Development of Failure Conditions, $\frac{y}{H}$	
	Active Case	Passive Case
Dense sand	0.001	0.02 - 0.06
Loose sand	0.005	0.06 - 0.35
Stiff clay	0.01	0.02
Soft clay	0.02	0.04

Notes : (1) Figure based on data given by Canadian Geotechnical Society (1985) and Rowe & Peaker (1965).
(2) Figure only applicable for rigid walls and soil with K_o less than unity.

Figure 6-9 Approximate values of retaining wall movements for the development of Active and Passive failure in soils

In comparison, only 10mm of displacement would be required to mobilise 50% of the maximum available passive soil resistance which is considered reasonable given the magnitude of the operational loads.

Under accidental loading where high transverse shear and moments about the longitudinal axis occur, larger displacements would be expected justifying the adoption of full passive soil resistance in the derivation of the base pressures.

The results of the foundation assessment are described herein. Checks on foundation overturning, bearing capacity and sliding have been carried out.

The results of the bearing capacity checks are expressed in terms of bearing capacity utilisation ratio (UR) defined as the ultimate bearing pressure/ultimate bearing capacity. On the basis that the value of UR listed in the summary table is less than 1.0 the limit state is satisfied. In certain cases (Pier 91) where the eccentricity of the base reaction lies outside of the footprint of the foundation it has not possible to calculate a value of UR for bearing capacity. In these circumstances overturning of the foundation is the governing failure mechanism.

A summary of the results of the foundation assessment are listed below.

Pier	BS:EN 1997-1: 2015	Foundation Pressure (kN/m²)	Ultimate Bearing Capacity (kN/m²)	Utilisation ratio UR	Result
In Operation					
22	DA1 C1	201	1469	0.14	ok
	DA1 C2	151	694	0.22	ok
44	DA1 C1	289	757	0.38	ok
	DA1 C2	257	372	0.69	ok
61	DA1 C1	507	1101	0.46	ok
	DA1 C2	591	460	1.28	28% overload
91	DA1 C1	853	2323	0.36	ok
	DA1 C2	1588	914	1.74	74% overload
Worst case of In Operation or Out of Operation					
22	DA1 C1	274	315	0.87	ok
	DA1 C2	255	71	3.60	260% overload
44	DA1 C1	338	731	0.46	ok
	DA1 C2	360	266	1.35	35% overload
61	DA1 C1	665	1027	0.65	ok
	DA1 C2	1035	257	4.02	302% overload
91	DA1 C1	N/A	N/A	N/A	overturning
	DA1 C2	N/A	N/A	N/A	overturning

Notes: *DA1 means Design Approach 1*

C1 and C2 mean Combination 1 and Combination 2 respectively

N/A indicates overturning of the foundation is governing.

Table 6-16 Summary of foundation assessment expressed in terms of utilisation of ultimate bearing capacity.

In conclusion, at the location of P22 and P44 the foundations satisfy conditions of ultimate limit state as defined by BS EN1997-1:2015 under normal operational loading conditions.

At the location of P61 and P91 the foundations are overstressed under normal operational loading conditions.

At the location of P22, P44 and P61 the foundations are overstressed under accidental storm force loading.

At the location of P91 under accidental storm force loading, overturning of the foundation is the governing failure mechanism.

Remedial works to strengthen the foundations will be required to address these defects.

7 Conclusions

7.1 Summary of Results

The structure has failed to meet the appraisal requirements both for In Operation and Out of Operation loading.

The structure does not comply with the deformation limits of BS EN 13107. Note that in making this determination conservative assumptions have been made including that deformations due to concrete creep have occurred (this makes up much of the vertical deformation) and that the track rails do not contribute to structure stiffness.

The superstructure does not comply with strength criteria of BD 44/15. Failures are noted in hog bending at anchorages, and shear in numerous spans in the structure both for In Operation and Out of Operation loading. The hog bending failure is slightly more critical than the shear failure - to comply with BD 44/15 the carriage load would have to be limited to 30 persons to avoid bending failure which is more onerous than the limit of 50 persons to avoid shear failure.

In making this determination a number of assumptions about the superstructure reinforcement have had to be made based on limited investigations. Based on the investigation it has also been assumed the structure is not weakened by corrosion.

Although there is little information about the bearings, it appears that the bearings are overloaded both for vertical load and lateral load according to the original design standard BS 5400-9, and more severely overloaded according to the current standard BS EN 1337. In particular the lateral guide bearings are unsuited for the combination of low vertical load and high lateral load which could occur under strong wind from the south or west. There is also the possibility of uplift under Out of Operation loads. In addition, many of the bearings in areas 3, 4 and also pier 91 appear to be displaced significantly in the uphill direction and will therefore slide beyond their limits under low temperatures. This will further overload elements within the bearing to stress levels in excess of their assumed capacity.

Many of the piers do not comply with strength criteria of BD 44/15. Failures are noted due to bending and shear in the pier columns both for In Operation and Out of Operation loading. The piers that fail with the highest utilisations are the taller piers. The degree of failure for the tallest piers is so severe that it fails to comply under the weight of an empty carriage.

Based on a sample of four pier foundations, many of the pier foundations do not satisfy bearing pressure limited determined to BS EN 1997. Failures occur in the piers in the upper parts of the structure for In Operation loading, and more extensively for Out of Operation loading. In addition, the uppermost pier is considered failed by overturning for Out of Operation loading.

The results reported above are summarised in the table below:

Element	Mode of failure	Result of In Operation load	Result of Out of Operation load
Main beams	Vertical deflections	39% overload	not applicable
	Rotations	41% overload	not applicable
	Transverse deflections	ok	ok
	Sag bending	ok	ok
	Hog bending	42% overload	In operation governs
	Shear †	23% overload	In operation governs
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	44% overload
	Column bending	73% overload *	113% overload
	Column shear	ok	24% overload
	Base slab bending	ok	20% overload
Pier foundations	Bearing capacity	74% overload	Piers overturn

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

† Potential shear failure at scarf joints has not been taken into account

Table 7-1 Summary of results

7.2 Commentary

A number of non-compliances have been found in this appraisal.

Deformation limits are Serviceability Limit State criteria. Although the reasons for deformation limits are not given in BS EN 13107, the criteria are thought to be related to proper functioning of the funicular including avoiding derailment, passenger comfort, and to appear robust to users and the public. In this case the funicular has been in regular use until recently and unless there are concerns with any of the above, it is suggested that the failure to comply with deformation limits may not need to be addressed but that this should be reviewed by a suitably qualified organisation to ensure that safety is not compromised.

The main beams show cracks, including some large cracks around piers, and there is evidence of rust staining. Cracking is a Serviceability Limit State criterion but can lead to reinforcement corrosion. Intrusive investigations have not found any bar failures or significant corrosion to date, but corrosion of main reinforcement or shear links leading to loss of bending or shear strength could occur in the future.

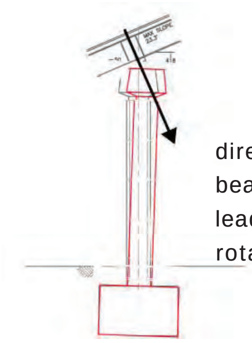
Structural strength criteria such as bending and shear limits are Ultimate Limit State requirements aimed at avoiding structural collapse with a suitable safety margin. However failures are not always sudden - some modes of failure are ductile and generally signs of distress can be noted before fracture occurs. Other modes of failure can be brittle with little or no warning signs. On this structure the bending failures should be ductile, but shear failures might be brittle, hence arguably the shear failures are of more serious concern than the bending failures.

In addition to the identified shear failures there is a risk that there is a failure mechanism in shear at the scarf joints which is not addressed by the standards, and hence has not been quantified in this appraisal.

The biggest and most imminent problem with the bearings is that many are expected to slide beyond the ends of their sliding tracks at moderately low temperatures. The bearings are thought to be significantly overloaded with the full contact area but if bearings slide off their tracks then the overload becomes considerably more acute. The eccentric loading could lead to the elastomeric disc popping out or the bearing seizing. Thus there is a risk of damage to the bearing itself, but should not lead to damage to the structure unless complete seizure occurred.

The overloading of foundations suggests that some piers are liable to overturn due to In Operation loads. The piers most likely to fail are the taller piers with high track inclination.

Although investigations have not definitively identified the reason for bearing misalignments there is a correlation between the most overloaded pier foundations and the observed misalignments of bearings in areas 3 and 4 and pier 91. Based on current knowledge it is thought the most likely reason for the bearing misalignment is that some pier foundations have failed and rotated in towards the slope due to the inclination of the bearings as shown in Figure 7-1. All the bearing misalignments are in the uphill direction, which supports this theory.



direction of bearing load, leading to rotation of pier

Figure 7-1 Rotation of pier leading to bearing misalignment

8 Recommendations

8.1 General

The appraisal has found the structure does not comply with the standards as outlined in the Schedule of Basic Assumptions. That does not necessarily mean the structure is in imminent danger, but it does indicate that there is a lower margin of safety than desirable. It is therefore recommended that measures are taken to address the failures prior to any resumption of the funicular railway operation.

Highways standard BD 79/13 offers a method for addressing the management of deficiencies in substandard structures on the highways network. This is considered best practice in the industry so it is recommended that a similar approach is taken for this structure.

Based on the results of this appraisal the structure would be classed as an Immediate Risk Structure to BD 79/13, governed by the substructure findings. According to the processes in BD 79/13 interim measures should be put in place. It is noted that the funicular is currently closed, hence it is recommended that the interim measures are implemented before the funicular is opened to passengers. The interim measures are discussed below as short term measures.

For the funicular to remain open, strengthening is recommended. Suggestions are discussed below as long term measures.

8.2 Short term (Interim) measures

Before the funicular is to be put into service, the following measures are recommended in the short term:

› Either accept that the structure cannot be put into service at low temperatures, or install jacks to temporarily support the deck at misaligned bearing positions. The jacks should incorporate sliding surfaces and could use the existing sliding surfaces on the downhill side of existing bearings - this would also have the advantage of introducing a restoring moment onto the pier foundations, see below. The existing bearings would not be removed, hence guide bearings would continue to provide lateral support.

› Reduce loading so that shear and bending overstress in the superstructure is avoided. This means restrictions in the maximum number of persons to be carried at one time.

› Regularly monitor the structure for signs of further deterioration, for example visual checks on bearings or superstructure crack measurements.

› Protect any piers that may be subject to accidental collision loading in the track direction.

Note that it is not proposed that piers are strengthened or propped. It is believed that any further signs of distress would appear gradually over time, and therefore it is sufficient to monitor these elements at this stage.

The short term measures are compared to the appraisal findings as follows:

Element	Mode of failure	Result of In Operation load	Consequence of failure	Short term measure
Main beams	Deflection	39% overload	See 7.2	No action
	Rotation	41% overload	See 7.2	No action
	Hog bending	42% overload	Assumed ductile failure	Apply load restriction
	Shear	23% overload	Possible brittle failure	Apply load restriction
Bearings	Misalignment	loss of contact area below +5ºC	Bearing damage	Apply temperature restriction or install jacks. Monitor
	Vertical overloading	65% overload	Bearing damage	Monitor (load restriction will help)
	Lateral overloading	not quantified	Bearing damage	Monitor (load restriction will help)
Piers	Column bending	73% overload	Assumed ductile failure	Monitor (load restriction and jacks will help), protect from impact
Pier foundations	Bearing pressure	74% overload	Pier rotation	Monitor (load restriction and jacks will help)

Table 8-1 Short term measures

8.3 Long term measures

To keep the funicular in service the following measures are recommended in the long term:

- Piers could be strengthened to provide better resistance to bending and foundation overturning. A possible arrangement would be to install diagonal props. Preloading such props might allow the existing piers to be pushed back towards their original positions to some degree.

- Replacement of all bearings. Bearings have finite service life and will need to be replaced at some stage. New bearings should have a higher load capacity and adequate movement capacity. They should also be specifically designed for a combination of low vertical load and high horizontal load where appropriate.

- Unless permanent load restrictions are acceptable, strengthen main beams where necessary to provide sufficient bending and shear resistance. Excess cracking may lead to reinforcement corroding in the future. A main beam strengthening scheme that reduces cracking would be an advantage, as would one that intuitively reduces the risk of shear failure along the line of the scarf joint.

- For the avoidance of doubt, the deformations should be checked with the equipment supplier, Doppelmayr, to verify the mechanical equipment is compatible with the calculated deflections and rotations.

- Reconsider the "accidental" load case in which a broken down carriage is clamped to the tracks in a storm. This is an extreme situation, does not result in persons at risk, and therefore there is little safety value strengthening the structure to meet this criterion.

- Consider permanent protection to any piers that may be subject to accidental collision loading in the track direction (may be combined with propping).

The long term measures are compared to the appraisal findings as follows:

Element	Mode of failure	Result of In Operation load	Short term measure
Main beams	Deflection	39% overload	Confirm with supplier
	Rotation	41% overload	Confirm with supplier
	Hog bending	42% overload	Apply permanent load restriction or strengthen
	Shear	23% 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	74% overload	Strengthen foundations, e.g. by propping or anchors

Table 8-2 Long term measures

Appendix A Schedule of Basic Assumptions

CAIRNGORM MOUNTAIN LIMITED / HIGHLANDS AND ISLANDS ENTERPRISE

CAIRNGORM FUNICULAR RAILWAY

Schedule of Basic Assumptions for Structural Appraisal

Working Document

A116993-SBA-Rev01.docx

November 2018

Document Control Sheet

Rev.	Status	Date	By	Check	Approved
01	First Issue	Nov. 2018	[redacted]	[redacted]	[redacted]

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Contents

Name of Bridge or Structure: Cairngorm Funicular Railway Viaduct

1 STRUCTURE DETAILS

1.1 Type of installation

Existing funicular mountain railway for transport of foot passengers and skiers up and down Cairn Gorm Mountain. The railway has a total length, measured horizontally in plan, of approximately 1900m, and a net elevation gain of approximately 450m.

The structure is a funicular railway. It comes under the scope of the Cableway Installation Regulations 2018 and EU Directive 2016/424.

1.2 Permitted traffic speed

The maximum permitted carriage speed is 10m/s.

1.3 Existing restrictions

None.

2 SITE DETAILS

2.1 Obstacles crossed

The railway passes through the ski resort on Cairn Gorm Mountain, but crosses no public roads. One private access road is crossed approximately 850m along the railway from the base station.

3 PROPOSED STRUCTURE

3.1 Description of structure and design working life

The railway consists of a 1650m long 93-span viaduct followed by a 250m long tunnel. This appraisal covers the viaduct section of the railway only. The tunnel section of the railway is outside the scope of this appraisal. The railway is a single track (track gauge = 2.0m) except at a passing loop situated near the midpoint of the viaduct. The viaduct has curved and straight alignment in both plan and elevation. A selection of the original design drawings showing an overview of the viaduct geometry is given in Appendix B.

The original design working life of the viaduct is unknown. In the absence of this information, the original design working life is assumed to have been 120 years as per BS 5400-1:1988. BS 5400 is believed to be the standard upon which the original design was based, according to information provided in the Structural Design Check Certificate. The original design life of the bearings, trackway, and other mechanical and electrical elements is likely to have been considerable less than 120 years. BS EN 13107:2015 states that renewable components and parts of structures that absorb actions induced by ropes should have design working lives of 20 and 30 years, respectively. Design life for funicular structures is stated as 50 years. The original design was based on a draft of this standard.

3.2 Structural type

The viaduct structural form is two continuous beams supported on intermediate pier supports. All structural elements are reinforced concrete except structural steel lateral bracing between the two beams. Six anchor blocks are located throughout the viaduct at approximately equal spacing to restrain the viaduct longitudinally. Movement joints at the anchor blocks effectively separate the viaduct into six independent structures.

3.3 Foundation Type

All pier foundations are in-situ reinforced concrete pad footings that bear directly onto the underlying ground. The in-situ reinforced concrete anchor blocks bear onto the underlying ground and are assumed to be anchored to bedrock with rock bolts and/or shear dowels.

3.4 Span Arrangements

The superstructure of the viaduct consists of two precast concrete beams, one supporting each rail of the track. The precast beams are supported on reinforced concrete piers and made continuous over the supports via in-situ stitch joints. The longest and shortest span lengths between pier centres, measured horizontally in plan, are approximately 18m and 12m, respectively. Span lengths vary throughout the viaduct, with most spans being over 16m long. Structural steelwork provides lateral bracing between the two precast beams. Rails are supported on the top flange of the concrete beams and held in position with "HALFEN" channels embedded in the pre-cast beams. Rails do not necessarily follow the central line of the beam, especially in areas of curved alignment in plan.

3.5 Articulation Arrangements

The two rail support beams are connected to the piers via low friction PTFE–stainless steel sliding bearings that permit the beams to expand and contract longitudinally. One bearing at each pier is guided, restricting lateral movement. The bearings are inclined to match the inclination of the railway and vary from 4° to 23°. Bearing sliding surfaces are parallel with the soffit of the beams.

The precast beams are rigidly connected to the up-mountain side of the anchor blocks. Movement joints on the down-mountain side of the anchor blocks separate the viaduct into six independently articulating segments. The individual segments vary in length between 220 to 330m.

The track rail is continuously welded for each continuous structure length, with expansion joints coinciding with the movement joints at the anchor blocks.

3.6 Road restraint systems requirements

There is no footpath or guardrails on the railway.

3.7 Inspection for assessment

3.7.1 Traffic management

All rail traffic will be halted prior to major investigative works.

3.7.2 Access arrangements to structure

An access road runs alongside a portion of the viaduct. The remaining areas of the viaduct can be accessed by foot. Pier and superstructure investigative works will require ladders or alternate means of access to heights. The tallest area of the viaduct is approximately 6m above ground level.

3.7.3 Intrusive or further investigations proposed

The following investigative works are proposed:

1. Trial pitting adjacent to selected piers.
2. Survey of bearing positions at all piers.
3. Non-destructive investigation of reinforcement layouts within selected precast beam and in-situ stitch joint elements.
4. Possible intrusive investigation to identify cause of cracks at in-situ/precast concrete connection above piers.

Future additional intrusive works may be proposed if deemed necessary based on the structural appraisal or a review of the investigative works listed here.

3.8 Environment and sustainability

Investigative works will be conducted in such a manner as to avoid negative impact to flora and fauna or contamination of watercourses.

3.9 Materials strengths assumed and basis of assumptions

3.9.1 Basis of assumptions

Limited documentation is available. Assumptions of material strengths and the relevant material standards are based on a combination of two sources:

1. A Structural Design Check Certificate dated 28-10-02. Design Engineer listed as A. F. Cruden Associates and Checking Engineer listed as Bullen Consultants
2. Notes on the original design drawing set by A. F. Cruden Associates (drawings listed in Section 8.2 of this report).

If material strength testing is undertaken then values for characteristic or worst credible strength based on test results may be used instead.

3.9.2 Precast concrete

Assumed to be grade RC50 (f_{cu} = 50MPa) to BS5328-1:1997. This information is given in the Structural Design Check Certificate.

3.9.3 In-situ concrete

Assumed to be grade RC40 (f_{cu} = 40MPa) to BS5328-1:1997. This information is given in the Structural Design Check Certificate.

3.9.4 Reinforcing steel

Assumed to be grade 460 (f_y = 460MPa) deformed bars to BS 4449:1997. This information is given in the Structural Design Check Certificate.

3.9.5 Post-tensioning bars

Several A. F. Cruden Associates drawings show the pier crossheads connected to the crosshead beams by "T32 Macalloy bars", but it is understood that the bars used in construction were not supplied by Macalloy. It will be assumed the bars are post-tensioned "Macalloy-type" bars 32mm diameter with breaking strength 1030MPa initially stressed to 70% of breaking strength. This assumed strength may be altered if more information on the post-tensioning bars is obtained.

3.9.6 Rock bolts and dowels

Assumed to have a safe working load of 30T, based on information given in A. F. Cruden Associates drawing No. CA150/2/63 Rev. D. This assumed safe working load may be altered if more information on the rock bolts is obtained.

The A. F. Cruden Associates drawings also specify the use of T40 "dowels" at several anchor blocks. In the absence of other information, the dowels will be assumed to have the same strength as the reinforcing steel (f_y = 460MPa).

Site observations have identified loose nuts at the rock bolt / dowel and anchor block interfaces. The rock bolts and dowels will therefore be assumed to be unstressed.

3.9.7 Structural steel

Assumed to be grade S275 to BS EN 10025:1990 + A1:1993 or BS EN 10210-1:1994 (for hollow sections). This information is given in the Structural Design Check Certificate.

The yield strength f_y will be calculated as a function of thickness as per ES 10025 Table 5 or EN 10210 Table A.3. For thicknesses less than or equal to 16mm, f_y = 275MPa.

3.10 Risks and hazards considered for design, execution, maintenance and demolition. Consultation and agreement from the CDM co-ordinator

This appraisal does not include any design work and therefore CDM does not apply.

3.11 Year of construction

Construction of the railway was completed circa 2001.

3.12 Reason for assessment

Annual inspections of the viaduct have identified a number of areas of structural distress. These include:

1. Deterioration of concrete elements in the superstructure.
2. Limited remaining available travel on bearings.
3. Deterioration of bearing wearing plates.

These reports led to the owner of the funicular railway requesting that a review of the current structural condition and capacity be undertaken.

3.13 Part of structure to be assessed

A quantitative appraisal will be carried out on the viaduct substructure and superstructure, from foundations to top of rail support beams. Appraisal of the rail, rail clips, and rail support plinths is outside the scope of this work. Appraisal of any cableway machinery or related installations along the length of the viaduct, including the cables, cableway rollers, and roller support brackets, is also outside the scope of this work.

The structure and foundations will be appraised under Ultimate Limit State conditions only, as the Ultimate Limit State is associated with structural collapse, which affects public safety. However, the deflections and rotations of the superstructure will be appraised under Serviceability Limit State conditions, as excessive superstructure deformations could cause instability of the rail carriages and endanger public safety.

In the absence of a bearing schedule or bearing drawings, the load capacity of the bearings cannot be determined. As there is no widespread evidence that the bearings are distressed by overloading it will be assumed the bearings have sufficient load capacity. However there is evidence that bearings are misaligned along the structure length, hence the appraisal of the bearings will involve a determination of bearing movements, and a consideration of the consequences where predicted bearing movements exceed the extents of the sliding surfaces.

4 ASSESSMENT CRITERIA

4.1 Actions

The following actions are intended to be representative of the actions that the original design was based on.

4.1.1 Permanent Actions

Permanent actions shall be determined in accordance with BD 21/01.

Dead Loads

Nominal dead loads shall be calculated using material weights from Table 4.1 of BD 21/01:

Reinforced concrete	2400 kg/m^3
Plain concrete	2300 kg/m^3
Steel	7850 kg/m^3

Superimposed Dead Loads

The track shall be treated as a superimposed dead load. The track is believed to be formed from S33 rail, which has a weight of 33 kg per metre. A nominal superimposed dead load of 50 kg per metre of each rail will be used to account for the weight of the rail, track fixings, cables, cableway rollers, and roller support brackets.

Support Settlement, Creep, and Shrinkage

BD 44/15 clause 3.4 states "The effects of creep and shrinkage of concrete, temperature difference and differential settlement need not be considered at the ultimate limit state". Hence these will not be considered in the appraisal of structural elements.

However for the appraisal of the bearings, bearing movements due to shrinkage and creep will be considered.

4.1.2 Snow, Wind and Thermal Actions

Snow

Normally, in an assessment of a bridge to BD 21/01, snow loads would not be considered. However, in view of this structure's location, snow loads will be considered as follows:

When the funicular is operational, the snow depth will be limited to top of rail level, as the carriage is fitted with snow plough blades. This is assumed to be 160mm above the top of the rail support beams. Snow density will be assumed as 4 kN/m^3 as given in BS EN13107:2015. Where the funicular is not operational, drifting snow will be considered.

In the absence of other guidance snow will be included in all load combinations where it has an adverse effect with γ_{fL} = 1.5 at the ultimate limit state.

Wind

Normally, in an assessment of a bridge to BD 21/01, wind loads would not be considered. However, as this structure is exposed and in use in high winds, wind loads will be considered as follows:

Wind loads will be determined and combined with other loads in accordance with BD 37/01.

Wind load cases will try to match the original design criteria. The cases considered will therefore be taken from the Structural Design Check Certificate as follows:

1. Principal operational load: Carriage fully laden + wind at 35 m/s.
2. Emergency evacuation load: Empty static carriage + 5000 kg mass + wind at 50 m/s.
3. Storm load: No carriage + wind at 75 m/s max. (at top station), 56 m/s min. (at bottom station). A reduced partial load factor for wind of γ_{fL} = 1.1 will be used with this load case.
4. Accidental case: Empty static carriage clamped to rails + 5000 kg mass + wind at 75 m/s max. (at top station), 56 m/s min. (at bottom station).

For wind load cases 3 and 4 it is assumed that the wind speeds at intermediate positions can be determined by linear interpolation according to altitude.

The wind speeds listed in cases 1 to 4 will be assumed to be Maximum Wind Gust Speeds (V_d), based on information provided in the Structural Design Check Certificate. The derivation in the Structural Design Check Certificate already accounts for return period, direction, fetch, topography, terrain, and altitude. Note that the wind speeds in cases 1 and 2 are not believed to come from code-based derivations, but instead are operational limits.

The drag factor C_D for wind load on the carriage will be taken as 1.30, the side area of the carriage exposed to wind will be taken as 31.93m^2, and the centre of the side area will be taken as 1.722m above the top of rails, as stated in the Doppelmayr Operations and Maintenance Manual.

Thermal

Normally, in an assessment of a bridge to BD 21/01, thermal effects would not be considered. However, for the appraisal of the bearings, bearing movements due to thermal effects will be considered.

Thermal effects will be determined and combined with other loads in accordance with BD 37/01.

Minimum and maximum shade air temperatures for the appraisal are taken from BD 37/01 isotherm maps (-24.0°C and 33.0°C for the site location, respectively) and modified to account for the site elevation. The minimum shade air temperature is modified to account for an elevation of 1000 m above mean sea level (the approximate elevation of the upper end of the viaduct). The maximum shade air temperature is modified to account for an elevation of 600 m above mean sea level (the approximate elevation of the lower end of the viaduct). The corresponding values are as follows:

• Minimum shade air temperature:	-29.0°C
• Maximum shade air temperature:	27.0°C

The viaduct superstructure is assumed to be in construction type Group 4. No adjustment to the effective bridge temperature for surfacing is applicable. The maximum effective bridge temperature is taken from BD 37/01 Table 11. The minimum effective bridge temperature is extrapolated from BD 37/01 Table 10 as the minimum shade air temperature + 10.0°C. (Extrapolation is necessary as the minimum shade air temperature in BD 37/01 Table 10 is -24.0°C, which corresponds to a Group 4 minimum effective bridge temperature of -14.0°C, or the minimum shade air temperature + 10.0°C.) The corresponding values are as follows:

Minimum effective bridge temperature:	-19.0°C
Maximum effective bridge temperature:	29.0°C

The effects of differential temperature will not be considered.

4.1.3 Live Loads

Nominal carriage live loads will be based on the information provided by Doppelmayr in the Operations and Maintenance Manual:

Empty carriage:	14,900 kg
Fully laden carriage (120 persons at 80 kg per person):	24,500 kg

Cairngorm Mountain Limited currently limits the number of passengers to 100.

The carriage is supported on two bogies spaced at 6.20 m, each with two axles spaced at 1.35 m (see Figure 1). It is assumed that in the absence of acceleration or braking the load is equally distributed between the four axles.

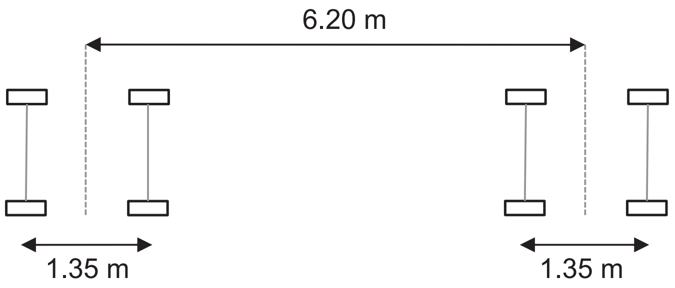

Figure 1. Carriage axle spacing

The dynamic amplification factor (ϕ) will be taken as 0.3 based on Clause 7.3.3.4 of BS EN 13107:2015.

There are two carriages operating on the railway. Carriage load cases will be based on a single carriage except at the passing loop, where both carriages will be considered.

A maintenance trolley of length 2.0 m may be attached to either carriage. The trolley weights will be taken as those given by Doppelmayr: 350 kg empty, 800 kg fully laden.

The axle spacing and connection details to the carriage are unknown. In the absence of other information, the axle spacing will be assumed to be 1.5 m, with the lead axle acting 2.0 m behind the rear axle of the carriage.

Acceleration, deceleration, emergency braking, and centrifugal loads will be considered. Acceleration and deceleration will be assumed to be 0.35 m/s^2. Emergency braking will be assumed to be 25% of the axle load on 3 of the 4 axles.

Nosing forces will be taken as 25% of the wheel load, as per BS EN 13107:2015.

In the absence of other information, the live load partial factors γ_{fL} will be taken as those used for type RL loading to BD 37/01.

The carriage is guided by one doubly flanged wheel per axle, the opposite wheel being without flanges, so that all lateral loads are carried on one of the two rails.

The centre of mass of the empty carriage is 1.3m above top of rail based on the information provided by Doppelmayr in the Operations and Maintenance Manual. In the absence of definitive information, the centre of mass of the fully laden carriage will be taken as 1.5m above top of rail.

4.1.4 Loading relating to normal traffic under the Road Vehicles (Authorised Weight) 1998 (AW) regulations and The Construction & Use (C & U) regulations 1996

N/A.

4.1.5 Loading relating to General Order Traffic under STGO regulations

N/A.

4.1.6 Footway or footbridge live loading

N/A.

4.1.7 Loading relating to Special Order Traffic, provision for exceptional abnormal indivisible loads including location of vehicle track on deck cross section

N/A.

4.1.8 Accidental Actions

Accidental actions due to vehicle impact with the viaduct will be considered in accordance with BS EN 13107:2015 and BS EN 1991-1-7. The lowest categories of impact forces will be used due to the expected low speeds and light weights of any vehicles operating near the viaduct. Equivalent static forces due to vehicle impact with the piers will be taken from Table NA.1 of the UK National Annex to BS EN 1991-1-7, assuming the category "Bridges over roads: minimum forces for robustness". Equivalent static forces due to vehicle impact with the superstructure will be taken from Table NA.9 and Table NA.10 of the UK National Annex to BS EN 1991-1-7, assuming the category "Courtyards and parking garages".

Avalanche loads will not be considered due to the shallow slope (<25°) and the fact that the railway is not believed to lie in an avalanche run-out zone.

Derailment loads are not applicable, as there are no elements (e.g. deck plate, walkway) that would be loaded by a derailed carriage.

4.1.9 Actions during construction

N/A.

4.1.10 Any special action not covered above

Horizontal actions induced by the carriage haul ropes where the viaduct is curved in plan will be considered. The maximum tension in the rope will be taken as 135kN, as per the Doppelmayr Operations and Maintenance Manual.

Frictional forces at the sliding bearings between the rail support beams and piers will be considered. The coefficient of friction between PTFE and stainless steel will be taken from Table 3 of BS 5400-9.1:1983.

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

N/A.

4.3 Minimum head room provided

The only object crossed is the private access road. The minimum head room for the private access road is unknown.

4.4 Authorities consulted and any special conditions required

None.

4.5 Standards and documents listed in the Technical Approval Schedule

The appraisal of structural elements will be undertaken to highways assessment standards, because (1) these standards have been explicitly devised to be appropriate for assessing existing UK bridge structures and (2) these standards are closely aligned to BS 5400 which was used as the original design standard (as per the Structural Design Check Certificate). The appraisal of foundations will be undertaken to Eurocodes, because (1) there are no highways assessment standards for foundations (2) the original foundations would have been designed to BS 8004 which was a working stress code and using a limit state code is thought more appropriate for assessing an existing structure.

In order to conduct the appraisal to a consistent standard, partial factors taken from highways assessment standards will be used in preference to those given in cableway installation standard BS EN 13107:2015.

For a list of standards see the Technical Approval Schedule in Appendix A.

4.6 Proposed Departures relating to Standards given in 4.5

For appraisal of the longitudinal shear on the scarf joints at each rail support beam at each pier, resistance will be calculated to Eurocodes instead of BD 44/15 Clause 7.4.2.3 because BD 44 does not take into the account the significance of normal loading across the interface.

No further departures are anticipated at this time.

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

Where an aspect not covered by highways assessment standards is encountered, structural Eurocodes or other international standards will be used.

5 STRUCTURAL ANALYSIS

5.1 Methods of analysis proposed for superstructure, substructure and foundations

The bridge superstructure will be analysed as either a one-dimensional line element or a two-dimensional grillage, as deemed necessary. Analysis will be conducted using

COWI's in-house analysis program NODLE. First order linear elastic analysis will be used.

The substructure and foundations will be analysed using first principles, using input loads into the substructure determined from the superstructure analysis.

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

Analysis of the superstructure will be undertaken using an idealized model of each anchor block-to-movement joint segment, as shown in Figure 2. The rail support beams will be taken as continuous between the anchor block and movement joint, but the effects of dead loads due to the self-weight of the precast beams will be determined assuming each span is simply supported, due to the method of construction.

The inspection reports listed in Section 8.4 have given rise to concerns about the continuity of top flange longitudinal reinforcement through the in-situ joints over the piers. Depending on the outcome of investigations, superstructure analysis may therefore also be undertaken assuming moment redistribution or even a simply supported condition at some piers.

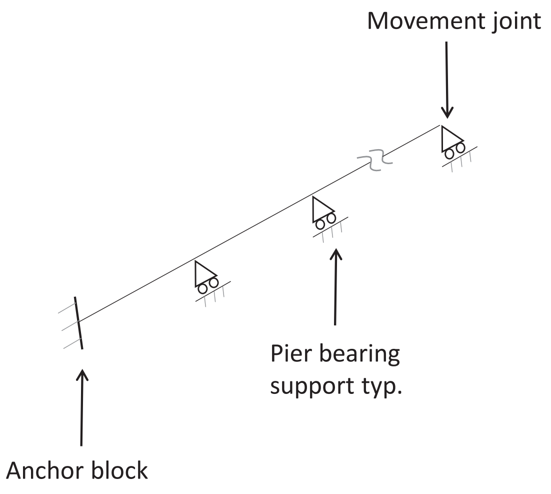

Figure 2. Idealized model for superstructure analysis

Substructure analysis will assume the piers have rigid foundations.

5.3 Assumptions intended for calculation of structural element stiffness

Initially gross (un-cracked) concrete section properties will be assumed in the global analysis. Where this is found to be inappropriate, reduced stiffness values based on a net transformed section may be used, in accordance with BD 44/15.

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

There are no earth retaining structures as part of the railway viaduct.

6 GEOTECHNICAL CONDITIONS

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

To the best of COWI's knowledge a Geotechnical Design Report did not form part of the original design submission prepared by A.F Cruden Associates.

Notwithstanding limited geotechnical information is presented on the design drawings and extracts from the original Design and Check certificates held in the project Health and Safety file.

Indicative rock head levels are shown on the longitudinal sections Sheet 1-7. (DWG-CA150/2/11 to CA150/2/17)

The design drawings indicate that foundation sizes have been designed on the basis of a safe bearing capacity of 150kN/m². It is stated that this must be confirmed on site before construction of the base commences. Moreover, where soft spots below the foundation are encountered they shall be removed and made up in lean mix concrete.

The drawings indicate that the filling around the foundations shall be executed in 250mm lifts. Backfill material shall clean granular material.

Design information listed on the certificate of compliance states the soils to typically comprise weathered granite bedrock overlain with dense to very dense natural gravelly sand with cobbles under gritty topsoil. An allowable bearing capacity of 400kN/m² is quoted based upon the findings of SPT N values recorded during a site investigation campaign undertaken by HTS Associates on behalf of the Highland Council.

In view of the ambiguity relating to bearing capacity terminology referenced in the design documentation and uncertainty over the ground conditions encountered along the length of the structure, COWI undertook a site walkover and desk study investigation of existing information that was used to scope a trial pit investigation to better establish the characteristics of foundation soils.

The findings of a preliminary desk study identified three distinct areas/types of surface geology along the length of the structure.

Area 1, located between CH0+000 and CH0+600m is characterised by glacial sands, gravels and boulders. This area is located at the toe of south facing tallus slopes and is bounded to the south by a natural watercourse.

Area 2, located between CH0+600 and CH0+950m is characterised by alluvial deposits of sand, silt and clay. In this area the surface sediments are likely to be thickest containing a higher percentage of silt and clay sized particles together with peat and organic deposits. Vegetation was observed to be better established in this area compared with areas 1 and 3 indicative of the soils ability to retain water to a greater extent. A natural spring was observed to discharge at surface above pier No.41 at CH 0+785m.

Area 3, located between CH1+000 and Ch1+700m is characterised by glacial head deposits. This area corresponds with the steepest down slope ground profile ranging between 15˚ to 22˚. In this area it is anticipated that surface sediments will be thinnest. A natural spring was observed to discharge at surface above pier No.72 at CH 1+310m. Details of the scope of trial pitting work is set out in TN-03-001.

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

The findings of the desk study, trial pit investigation and subsequent laboratory testing will be used to develop ground models at specific locations along the length of the structure.

These ground models will form the basis for a geotechnical assessment to determine the level of bearing capacity utilisation at specific foundations along the length of the structure.

Predictions of total foundation settlement will be made under serviceable dead and live loading together with estimates of differential settlement between adjacent piers.

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

Differential settlement between adjacent piers may occur as a result of variation in the founding strata. Such variations are not uncommon, particularly in glacial deposits where lenses of clay may be found in predominantly sandy material or vice versa. In areas of water laid deposits of sands and gravels (Alluvium) in-situ densities can vary widely as can areas with irregular bedrock surface where part of the structure may be founded on shallow rock whereas parts maybe funded on weathered rock with greater compressibility. An assessment value for differential settlement will be derived on completion of the studies described in 6.2.

7 CHECK

7.1 Proposed Category

This appraisal is not a formal assessment and the checking procedures of BD 2/12 do not apply.

7.2 If Category 3, name of proposed independent Checker

N/A.

8 DRAWINGS AND DOCUMENTS

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

Appendix A:	Technical Approval Schedule
Appendix B:	Selected drawings
Appendix C:	Doppelmayr Operations and Maintenance Manual
Appendix D:	Structural Design Check Certificate
Appendix E:	Relevant photos from Previous Inspection Reports

8.2 List of construction and record drawings (including numbers) to be used in the assessment

Construction or as-built drawings are not available. The appraisal will use the original design drawings by A. F. Cruden Associates listed in Table 1 where appropriate, but it is noted that the drawings listed contain inconsistencies and are in some cases marked as preliminary.

Table 1. A. F. Cruden Associates original design drawings

Drawing No.	Revision	Title
CA150/2/01	E	SITE PLAN SHEET 1 OF 7 CHAINAGE 0 TO 340
CA150/2/02	D	SITE PLAN SHEET 2 OF 7 CHAINAGE 360 TO 700
CA150/2/03	D	SITE PLAN SHEET 3 OF 7 CHAINAGE 700 TO 1040
CA150/2/04	D	SITE PLAN SHEET 4 OF 7 CHAINAGE 920 – 1260
CA150/2/05	D	SITE PLAN SHEET 5 OF 7 CHAINAGE 1220 – 1560
CA150/2/06	D	SITE PLAN SHEET 6 OF 7 CHAINAGE 1460 – 1800
CA150/2/08	C	FUNICULAR – HORIZONTAL GEOMETRY
CA150/2/11	F	LONGITUDINAL SECTION SHEET 1 OF 7 CHAINAGE 0 – 340
CA150/2/12	F	LONGITUDINAL SECTION SHEET 2 OF 7 CHAINAGE 360 – 700
CA150/2/13	F	LONGITUDINAL SECTION SHEET 3 OF 7 CHAINAGE 700 – 1040
CA150/2/14	F	LONGITUDINAL SECTION SHEET 4 OF 7 CHAINAGE 920 – 1260
CA150/2/15	F	LONGITUDINAL SECTION SHEET 5 OF 7 CHAINAGE 1220 – 1560
CA150/2/16	F	LONGITUDINAL SECTION SHEET 6 OF 7 CHAINAGE 1460 – 1800
CA150/2/17	F	LONGITUDINAL SECTION SHEET 7 OF 7 CHAINAGE 1660 – 1920
CA150/2/18	B	FUNICULAR – VERTICAL GEOMETRY
CA150/2/31	B	FUNICULAR PLAN CHAINAGE 0 – 720 SHEET 1 OF 3
CA150/2/32	B	FUNICULAR PLAN CHAINAGE 580 – 1480 SHEET 2 OF 3
CA150/2/33	B	FUNICULAR PLAN CHAINAGE 1220 – 1920 SHEET 3 OF 3
CA150/2/34	N/A	SITE PLAN OF TUNNEL CHAINAGE 1680 – 1920
CA150/2/35	N/A	TUNNEL CATCHPIT DETAIL

CA150/2/36	A	BEARING DETAIL AT TUNNEL ANCHOR BLOCK
CA150/2/37	N/A	BEARING DETAIL: BEAMS UPHILL OF ANCHOR BLOCKS
CA150/2/38	N/A	ANCHOR BLOCK 48 – RC DETAILS
CA150/2/39	A	INSITU DIAPHRAGM DETAILS
CA150/2/40	B	SUPPORT TOWER DETAILS
CA150/2/42	C	STEELWORK SUPERSTRUCTURE DETAILS
CA150/2/44	G	TOWER ELEVATIONS SHEET 1 OF 2
CA150/2/45	F	TOWER ELEVATIONS SHEET 2 OF 2
CA150/2/47	D	STEELWORK SUPERSTRUCTURE OF PASSING LOOP
CA150/2/49	B	RAIL BOLT / BEAM SUPPORT / AND BEARING DETAILS
CA150/2/50	C	TUNNEL GENERAL ARRANGEMENT
CA150/2/51	H	TUNNEL DETAILS (SHEET 1 OF 2)
CA150/2/52	G	TUNNEL DETAILS (SHEET 2 OF 2)
CA150/2/53	N/A	TUNNEL CROSS SECTIONS LOCATED NEAR EXISTING CHAIRLIFT BASES
CA150/2/54	B	TUNNEL DETAILS PTARMIGAN STATION
CA150/2/55	A	EARTHWORKS / LANDSCAPING AT TUNNEL ENTRANCE
CA150/2/56	A	TUNNEL REINFORCEMENT DETAILS SHEET 1 OF 2
CA150/2/57	N/A	ELEVATIONS OF TOWERS 48 TO 58 (LOCATED AT THE PASSING LOOP)
CA150/2/60	B	R-C DETAILS CROSSHEAD 1 91 No. REQUIRED THUS
CA150/2/61	C	ANCHOR BLOCK SETTING OUT DETAILS
CA150/2/63	D	ANCHOR BLOCK TYPE 3A – TYPICAL DETAIL R-C DETAILS
CA150/2/67	D	R-C DETAILS TOWERS
CA150/2/68	B	R-C DETAILS – 4m LONG BASE TYPE 1A 5 No. REQUIRED THUS
CA150/2/69	B	R-C DETAILS – 4.5m LONG BASE TYPE 2C 5 No. REQUIRED THUS
CA150/2/70	C	R-C DETAILS – 4.5m LONG BASE TYPE 2A 14 No. REQUIRED THUS
CA150/2/71	C	R-C DETAILS – 4.5m LONG BASE TYPE 2B 4 No. REQUIRED THUS
CA150/2/72	C	R-C DETAILS – 4.8m LONG BASE TYPE 3 18 No. REQUIRED THUS
CA150/2/73	B	R-C DETAILS – 5.15m LONG BASE TYPE 4 14 No. REQUIRED THUS
CA150/2/74	B	R-C DETAILS – 5.6m LONG BASE TYPE 5 24 No. REQUIRED THUS
CA150/2/75	A	R-C DETAILS – 6.0m LONG BASE TYPE 6 5 No. REQUIRED THUS
CA150/2/76	D	PRECAST CONCRETE BEAM DETAIL 1 OF 2
CA150/2/77	N/A	R-C DETAILS CROSSHEAD 51 1 No. REQUIRED
CA150/2/78	A	R-C DETAILS CROSSHEAD 52 + 56 2 No. REQUIRED
CA150/2/79	C	PRECAST CONCRETE BEAM DETAIL 2 OF 2
CA150/2/80	A	DETAILED PLAN OF THE FUNICULAR RAILWAY 1 OF 7 CHAINAGE 0 – 280
CA150/2/82	A	DETAILED PLAN OF THE FUNICULAR RAILWAY 3 OF 7 CHAINAGE 520 – 780
CA150/2/83	B	DETAILED PLAN OF THE FUNICULAR RAILWAY 4 OF 7 CHAINAGE 780 – 1060
CA150/2/84	A	DETAILED PLAN OF THE FUNICULAR RAILWAY 5 OF 7 CHAINAGE 1060 – 1460
CA150/2/85	A	DETAILED PLAN OF THE FUNICULAR RAILWAY 6 OF 7 CHAINAGE 1460 – 1860
CA150/2/86	N/A	DETAILED PLAN OF THE FUNICULAR RAILWAY 7 OF 7 CHAINAGE 1840 – 1920
CA150/2/88	A	PRECAST CONCRETE BEAMS CONCERNED WITH CROSS MEMBERS
CA150/2/89	N/A	TOWERS 6, 7, 18 REMEDIAL WORK FOR MISALIGNMENT
CA150/2/90	N/A	TYPICAL SECTION THROUGH CONC. RAIL SUPPORT SHOWING STEELWORK CONNECTION DETAILS
CA150/2/91	N/A	TYPICAL SECTION THROUGH CONC. RAIL SUPPORT AT PASSING LOOP SHOWING STEELWORK CONNECTION DETAILS
CA150/2/92	N/A	STEELWORK SUPERSTRUCTURE OF PASSING LOOP ENLARGED PART SETOUT
CA150/2/93	N/A	STEELWORK SUPERSTRUCTURE OF PASSING LOOP ENLARGED PART SETOUT
CA150/2/94	N/A	STEELWORK SUPERSTRUCTURE OF PASSING LOOP ENLARGED PART SETOUT

CA150/2/95	N/A	STEELWORK SUPERSTRUCTURE OF PASSING LOOP ENLARGED PART SETOUT
CA150/2/96	N/A	LONGITUDINAL SECTION SHEET 7 OF 7 CHAINAGE 1660 – 1920

8.3 List of pile driving or other construction records

No construction records are available. The inspection reports listed in Section 8.4 of this report contain information on recent maintenance works conducted.

8.4 List of previous inspection and assessment reports

Annual inspections of the funicular railway have been carried out since at least 2010. Table 2 lists the available inspection reports.

Table 2. Past inspection reports

Date	Revision	Author	Title
05-09-2018	A	ADAC Structures	Cairngorm funicular railway scope for comprehensive structural review
05-09-2018	B	ADAC Structures	Factual report on the sliding bearings to the funicular railway
24-07-2018	A	ADAC Structures	Funicular railway inspection report - 2018
09-08-2017	D	ADAC Structures	Funicular railway beam 51/R
28-07-2017	A	ADAC Structures	Condition report – funicular railway, Cairngorm Mountain
06-01-2017	A	ADAC Structures	Condition report – funicular railway, Cairngorm Mountain
16-11-2015	A	ADAC Structures	Condition report into concrete support structures for funicular railway
June 2014	N/A	A. F. Cruden Associates	Cairngorm mountain funicular & tows condition survey 2014
June 2013	N/A	A. F. Cruden Associates	Cairngorm mountain funicular & tows condition survey 2013
June 2012	N/A	A. F. Cruden Associates	Cairngorm mountain funicular & tows condition survey 2012
June 2011	N/A	A. F. Cruden Associates	Cairngorm lifts & tows – Condition report 2011
Feb. 2011	N/A	A. F. Cruden Associates	Cairngorm lifts & tows – Progress report 2010
Nov. 2010	N/A	A. F. Cruden Associates	Cairngorm lifts & tows – Progress report 2010

9 THE ABOVE IS SUBMITTED FOR ACCEPTANCE

Signed

Name [redacted]

Engineering Qualifications [redacted]

Name of Organisation COWI UK Limited

Date

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

Signed

Name

Position held

Engineering Qualifications

TAA

Date

Appendix A Technical Approval Schedule

Schedule of Documents to be Used in Appraisal

Standard	Title	Amendment / Corrigenda
The Design Manual for Roads and Bridges (DMRB)		
BD 2/12	Technical Approval of Highway Structures	
BD 21/01	The Assessment of Highway Bridges and Structures	
BD 37/01	Loads for Highway Bridges	
BD 44/15	The Assessment of Concrete Highway Bridges and Structures	
BD 56/10	The Assessment of Steel Highway Bridges and Structures	
Eurocodes		
BS EN 1991-1-3:2003 +A1:2015	Eurocode 1: Actions on structures. General Actions. Snow loads	+A1:2015 Incorporating corrigenda Dec 2004 & Mar 2009
NA to BS EN 1991-1-3:2003+A1:2015	UK National Annex to Eurocode 1: Actions on structures. General Actions. Snow loads	+A1:2015 Incorporating corrigendum No.1
BS EN 1991-1-7:2006 +A1:2014	Eurocode 1: Actions on structures. General Actions. Accidental actions	+A1:2014 Incorporating corrigendum February 2010
NA to BS EN 1991-1-7:2006+A1:2014	UK National Annex to Eurocode 1: Actions on structures. General Actions. Accidental actions	+A1:2014 Incorporating corrigenda August 2014 & November 2015
BS EN 1992-1-1:2004 + A1:2014	Eurocode 2: Design of concrete structures– Part 1-1: General rules and rules for buildings	Incorporating corrigenda January 2008, November 2010 & January 2014
NA + A2:2014 to BS EN 1992-1-1:2004 + A1:2014	UK National Annex to Eurocode 2: Design of concrete structures – Part 1-1: General rules and rules for buildings	
BS EN 1997-1:2004+A1:2013	Eurocode 7: Geotechnical design – Part 1 General rules	+A1:2013 Incorporating corrigendum February 2009
NA+A1 to BS EN 1997-1:2004+A1:2013	UK National Annex to Eurocode 7: Geotechnical design – Part 1 General rules	+A1:2013 Incorporating corrigendum No.1
BS EN 1997-2:2007	Eurocode 7: Geotechnical design – Part 2 Ground investigation and testing	Incorporating corrigendum June 2010
NA to BS EN 1997-2:2007	UK National Annex to Eurocode 7: Geotechnical design – Part 2 Ground investigation and testing	
Other European Standards		
BS EN 10025:1990 + A1:1993	Hot rolled products of non-alloy structural steels – Technical delivery conditions	+A1:1993
BS EN 10210-1:1994	Hot finished structural hollow sections of non-alloy and fine grain structural steels – Part 1: Technical delivery requirements	
BS EN 13107:2015	Safety requirements for cableway installations designed to carry persons – Civil engineering works	Incorporating corrigendum July 2016
BS EN 1709:2004	Safety requirements for cableway installations designed to carry persons – Precommissioning inspection, maintenance, operational inspection and checks	
BS EN 12929-1:2015	Safety requirements for cableway installations designed to carry persons – General requirements – Part 1: Requirements for all installations	
British Standards		
BS 5328-1:1997	Concrete – Part 1: Guide to specifying concrete	Incorporating Amendments Nos. 1 & 2 and corrigendum No. 1

BS 4449:1997	Carbon steel bars for the reinforcement of concrete	Incorporating Amendment No. 1
BS 5400-9.1:1983	Steel, concrete and composite bridges – Part 9: Bridge bearings – Section 9.1 Code of practice for design of bridge bearings	

Appendix B Selected drawings

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Drawing
LONGITUDINAL SECTION
SHEET 1 OF 7
CHAINAGE 0 – 340

Drawing No. CA150/2/11
Drawn By [redacted]
REV. F
Date 08/02/99
Scale 1:500

Revisions **Date** **By**

A 26/2/99 [redacted]
Column chainages and top rail levels added.
Column bases added.

B 15/4/99 [redacted]
Ref pt. A & Bottom Station altered.
Borehole and trial pit information added.
Tower types shown.
Table amended.

C 17/5/99 [redacted]
Ref pt. A moved to elevation 637m. Table amended.

D 15/10/99 [redacted]
Towers 0–4 have been altered due to existing features.
Table amended for revised Vertical alignment.
Designed steel sections added.
Extent of Bottom Station has been updated.
Supports from new station are added.

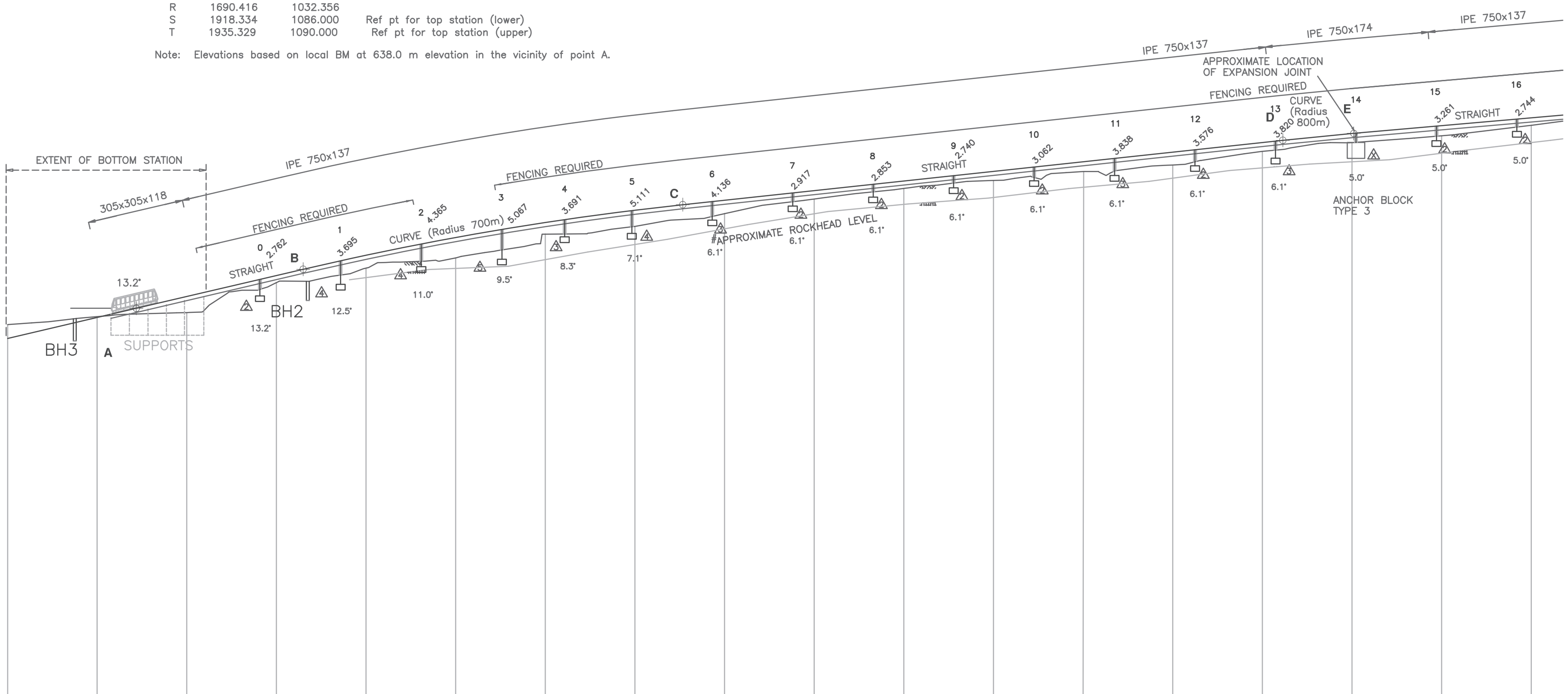
E 24/3/00 [redacted]
Angle of brackets added.
Locations of required pedestrian walkway and fencing are added.
Foundation levels on towers 0–4 have been amended.

F 9/11/00 [redacted]
Pedestrian walkway removed.

Point	Chainage	Elevation	Remarks
A	28.716	637.000	Ref pt for bottom station
B	65.956	645.709	
C	150.668	660.103	
D	284.566	674.471	
E	300.409	676.012	
F	435.580	687.802	
G	452.087	689.070	
H	898.677	761.500	Ref pt for middle station (lower)
J	912.057	765.000	Ref pt for middle station (upper)
K	999.208	789.752	Mid–point of line
L	1165.781	846.479	
M	1191.462	855.691	
N	1408.929	939.087	
O	1471.287	962.795	
P	1620.826	1012.618	
R	1690.416	1032.356	
S	1918.334	1086.000	Ref pt for top station (lower)
T	1935.329	1090.000	Ref pt for top station (upper)

Note: Elevations based on local BM at 638.0 m elevation in the vicinity of point A.

NOTES
BASED ON GROUND RADAR INFORMATION
3.820 HEIGHT OF RAIL LEVEL ABOVE TOP OF FOUNDATION
△ COLUMN TYPE(SEE DRG. CA150/2/40–41)



CHAINAGE	EXISTING GROUND LEVEL	COLUMN CHAINAGE	TOP RAIL LEVEL	TOP OF FOUNDATION LEVEL
000.00	633.37			
20.00	635.14			
40.00	636.04			
		56.316	643.455	639.761
60.00	642.73			
		74.316	647.610	642.325
80.00	645.95			
		92.316	651.340	646.277
100.00	648.22			
		110.316	654.581	647.942
120.00	653.53			
		124.316	656.769	652.929
140.00	655.494	139.299	658.789	653.678
	655.26			
	657.386	157.248	660.809	656.673
160.00	658.10			
	660.861	175.204	662.736	659.819
180.00	661.37			
	662.839	193.160	664.663	661.810
200.00	663.72			
	664.984	211.116	666.590	663.850
220.00	665.50			
	666.214	229.072	668.516	665.454
240.00	667.50			
	667.592	247.028	670.443	666.605
260.00	668.97			
	669.701	264.984	672.370	668.794
280.00	672.01			
	671.919	282.940	674.297	670.477
300.00	674.146			
	300.896	300.896	676.055	
320.00	675.401	318.852	677.621	674.360
	675.47			
	677.409	336.808	679.187	676.443
340.00	677.83			

Drawing LONGITUDINAL SECTION 1 OF 7
CHAINAGE 0 – 340
Drawing No. CA150/2/11
Revision. F

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Drawing
LONGITUDINAL SECTION
SHEET 2 OF 7
CHAINAGE 360 – 700

Drawing No. CA150/2/12
Drawn By [redacted]
REV. F
Date 8/2/'99
Scale 1:500

Point	Chainage	Elevation	Remarks
A	28.716	637.000	Ref pt for bottom station
B	65.956	645.709	
C	150.668	660.103	
D	284.566	674.471	
E	300.409	676.012	
F	435.580	687.802	
G	452.087	689.070	
H	898.677	761.500	Ref pt for middle station (lower)
J	912.057	765.000	Ref pt for middle station (upper)
K	999.208	789.752	Mid–point of line
L	1165.781	846.479	
M	1191.462	855.691	
N	1408.929	939.087	
O	1471.287	962.795	
P	1620.826	1012.618	
R	1690.416	1032.356	
S	1918.334	1086.000	Ref pt for top station (lower)
T	1935.329	1090.000	Ref pt for top station (upper)

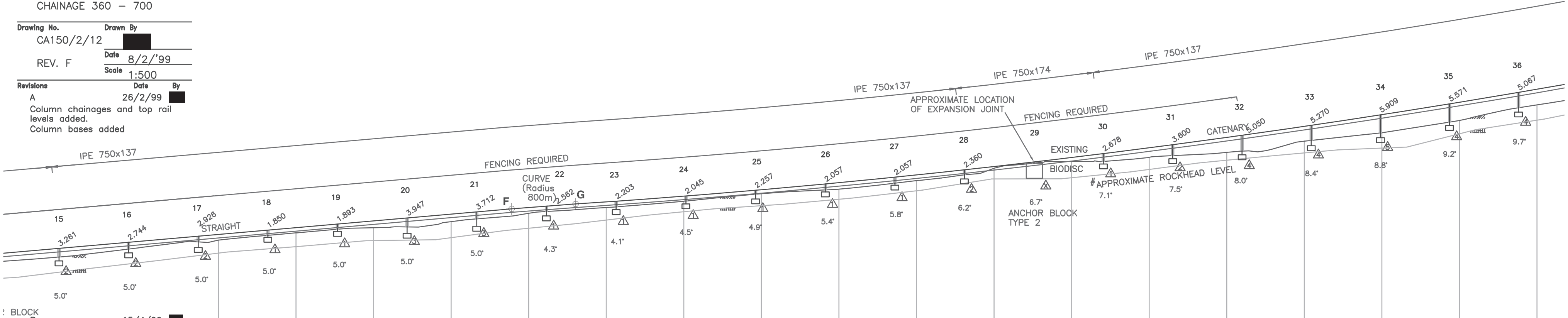
Note: Elevations based on local BM at 638.0 m elevation in the vicinity of point A.

NOTES

BASED ON GROUND RADAR INFORMATION

3.820 HEIGHT OF RAIL LEVEL ABOVE TOP OF FOUNDATION

△ COLUMN TYPE(SEE DRG. CA150/2/40–41)



Revisions

	Date	By
A Column chainages and top rail levels added. Column bases added	26/2/99	[redacted]
B Borehole and trial pit information added. Tower types shown. Table amended.	15/4/99	[redacted]
C Table amended	17/5/99	[redacted]
D Table amended for revised Vertical alignment. Designed steel sections added.	15/10/99	[redacted]
E Angle of brackets added. Locations of required pedestrian walkway and fencing are added.	9/11/99	[redacted]
F Pedestrian walkway removed.	9/11/00	[redacted]

DATUM 590.00m

CHAINAGE		360.00		380.00		400.00		420.00		440.00		460.00		480.00			500.00		520.00		540.00		560.00		580.00		600.00		620.00		640.00			660.00		680.00		700.00
EXISTING GROUND LEVEL	679.445	679.75	681.203	681.62	682.592	682.94	682.885	681.52	685.256	686.65	686.963	688.15	688.338	689.77	689.697	690.955	691.71	692.799	692.90	694.525	695.10	696.172	698.26	699.506	700.00	699.980	701.11	701.669	702.37	702.622	705.26	705.372	707.379	707.44	710.241	710.60	713.478	714.64
COLUMN CHAINAGE	354.765		372.720		390.674		408.630		426.585		444.557		462.551		480.536	498.511		516.474		534.424		552.361		570.283		588.189		606.078		623.950		641.804	659.639		677.453		695.246	
TOP RAIL LEVEL	680.753		682.319		683.885		685.451		687.017		688.534		689.788		691.131	692.612		694.229		695.982		697.871		699.897		702.058		704.353		706.784		709.349	712.048		714.880		717.845	
TOP OF FOUNDATION LEVEL	677.827		680.469		681.992		681.504		683.305		685.972		687.585		689.086	690.355		692.172		693.925		695.511				699.380		700.753		701.734		704.079	706.139		709.309		712.778	

Drawing
LONGITUDINAL SECTION 2 OF 7
CHAINAGE 360 – 700
Drawing No. CA150/2/12
Revision. F

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Drawing

LONGITUDINAL SECTION
SHEET 3 OF 7
CHAINAGE 700 – 1040

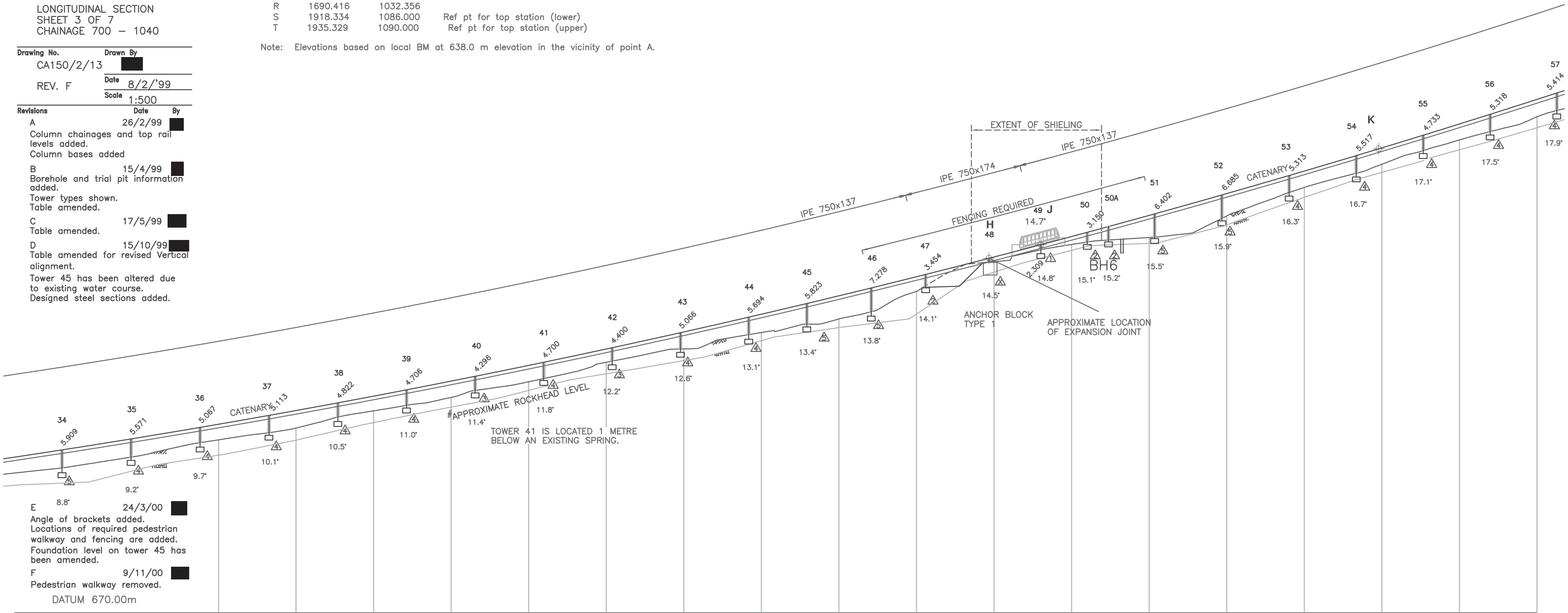
Drawing No. CA150/2/13
Drawn By [redacted]
REV. F
Date 8/2/'99
Scale 1:500

Revisions	Date	By
A Column chainages and top rail levels added. Column bases added	26/2/99	[redacted]
B Borehole and trial pit information added. Tower types shown. Table amended.	15/4/99	[redacted]
C Table amended.	17/5/99	[redacted]
D Table amended for revised Vertical alignment. Tower 45 has been altered due to existing water course. Designed steel sections added.	15/10/99	[redacted]
E Angle of brackets added. Locations of required pedestrian walkway and fencing are added. Foundation level on tower 45 has been amended.	24/3/00	[redacted]
F Pedestrian walkway removed.	9/11/00	[redacted]

Point	Chainage	Elevation	Remarks
A	28.716	637.000	Ref pt for bottom station
B	65.956	645.709	
C	150.668	660.103	
D	284.566	674.471	
E	300.409	676.012	
F	435.580	687.802	
G	452.087	689.070	
H	898.677	761.500	Ref pt for middle station (lower)
J	912.057	765.000	Ref pt for middle station (upper)
K	999.208	789.752	Mid–point of line
L	1165.781	846.479	
M	1191.462	855.691	
N	1408.929	939.087	
O	1471.287	962.795	
P	1620.826	1012.618	
R	1690.416	1032.356	
S	1918.334	1086.000	Ref pt for top station (lower)
T	1935.329	1090.000	Ref pt for top station (upper)

Note: Elevations based on local BM at 638.0 m elevation in the vicinity of point A.

NOTES
\# BASED ON GROUND RADAR INFORMATION
3.820 HEIGHT OF RAIL LEVEL ABOVE TOP OF FOUNDATION
△ COLUMN TYPE(SEE DRG. CA150/2/40–41)



DATUM 670.00m

CHAINAGE	700.00	720.00	740.00	760.00	780.00	800.00	820.00	840.00	860.00	880.00	900.00	920.00	940.00	960.00	980.00	1000.00	1020.00	1040.00																			
EXISTING GROUND LEVEL	714.64	716.814	717.66	720.269	722.12	723.428	725.59	727.331	729.64	730.550	734.68	734.603	738.059	738.11	741.300	742.35	745.86	747.234	752.93	754.612	761.073	760.63	763.704	765.06	765.727	766.87	767.213	771.760	772.41	778.148	779.69	783.083	785.66	789.563	791.95	794.201	797.93
COLUMN CHAINAGE	713.017	730.765	748.488	766.188	783.861	801.510	819.132	836.726	851.816	868.326	882.332	898.677	912.057	924.046	929.509	941.400	958.751	976.095	993.405	1010.680	1027.919																
TOP RAIL LEVEL	720.943	724.173	727.534	731.027	734.650	738.403	742.285	746.296	749.823	753.840	757.323	761.500	765.000	768.206	769.688	772.960	777.845	782.861	788.000	793.261	798.644																
TOP OF FOUNDATION LEVEL	715.830	719.351	722.828	726.731	729.950	734.003	737.219	740.602	743.770	746.562	753.869		762.691	765.056		766.558	771.160	777.548	782.483	788.528	793.326																

Drawing LONGITUDINAL SECTION 3 OF 7
CHAINAGE 700 – 1040
Drawing No. CA150/2/13
Revision. F

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Drawing
LONGITUDINAL SECTION
SHEET 4 OF 7
CHAINAGE 920 – 1260

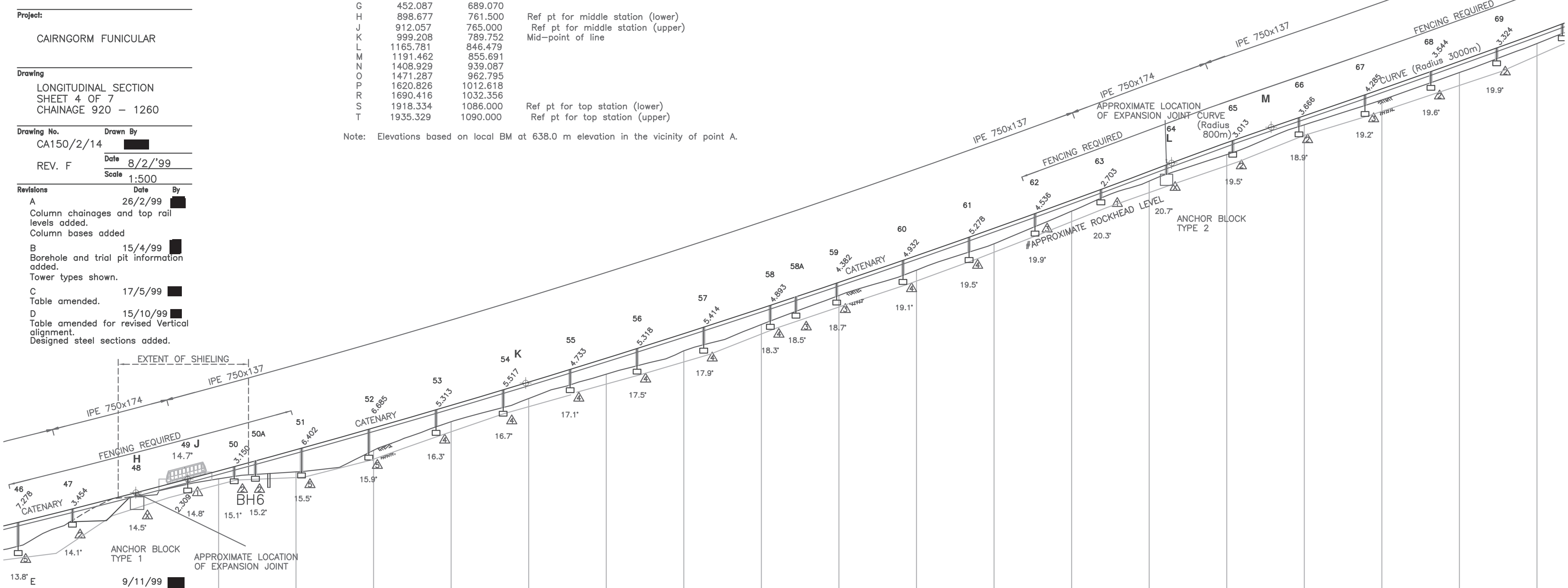
Drawing No. CA150/2/14
Drawn By [redacted]
REV. F
Date 8/2/'99
Scale 1:500

Revisions **Date** **By**
A 26/2/99 [redacted]
Column chainages and top rail levels added.
Column bases added
B 15/4/99 [redacted]
Borehole and trial pit information added.
Tower types shown.
C 17/5/99 [redacted]
Table amended.
D 15/10/99 [redacted]
Table amended for revised Vertical alignment.
Designed steel sections added.
E 9/11/99 [redacted]
Angle of brackets added.
Locations of required pedestrian walkway and fencing are added.
F 9/11/00 [redacted]
Pedestrian walkway removed.

Point	Chainage	Elevation	Remarks
A	28.716	637.000	Ref pt for bottom station
B	65.956	645.709	
C	150.668	660.103	
D	284.566	674.471	
E	300.409	676.012	
F	435.580	687.802	
G	452.087	689.070	
H	898.677	761.500	Ref pt for middle station (lower)
J	912.057	765.000	Ref pt for middle station (upper)
K	999.208	789.752	Mid–point of line
L	1165.781	846.479	
M	1191.462	855.691	
N	1408.929	939.087	
O	1471.287	962.795	
P	1620.826	1012.618	
R	1690.416	1032.356	
S	1918.334	1086.000	Ref pt for top station (lower)
T	1935.329	1090.000	Ref pt for top station (upper)

Note: Elevations based on local BM at 638.0 m elevation in the vicinity of point A.

NOTES
BASED ON GROUND RADAR INFORMATION
3.820 HEIGHT OF RAIL LEVEL ABOVE TOP OF FOUNDATION
⚠ COLUMN TYPE(SEE DRG. CA150/2/40–41)



DATUM 710.00m

CHAINAGE	920.00	940.00	960.00	980.00	1000.00	1020.00	1040.00	1060.00	1080.00	1100.00	1120.00	1140.00	1160.00	1180.00	1200.00	1220.00	1240.00	1260.00
EXISTING GROUND LEVEL	765.59	765.727 766.87	767.213 771.760	772.41 778.148 779.69	783.083 785.66	789.563 791.95	794.201 797.93	799.368 804.72	805.802 812.100	812.21 817.282 818.73	823.065 825.53	829.889 833.95	837.891 841.68	844.636 848.55	849.839 855.362	856.21 860.907 862.61	867.092 869.67	873.586 877.13
COLUMN CHAINAGE		924.046 929.509	941.400 958.751	976.095	993.405	1010.680	1027.919	1045.122	1062.288 1068.907 1079.416	1096.506	1113.557	1130.569	1147.541	1164.472	1181.551 1198.621	1215.653	1232.650	1249.613
TOP RAIL LEVEL		768.206 769.688	772.960 777.845	782.861	788.000	793.261	798.644	804.147	809.770 811.974 815.513	821.373	827.352	833.447	839.658	845.984	852.252 858.132	864.021	870.012	876.107
TOP OF FOUNDATION LEVEL		765.056	766.558 771.160	777.548	782.483	788.528	793.326	798.733	804.877 811.131	816.441	822.074	828.911	836.955		849.239 854.466	859.736	866.468	872.783

Drawing
LONGITUDINAL SECTION 4 OF 7
CHAINAGE 920 – 1260
Drawing No. CA150/2/14
Revision. F

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Drawing

LONGITUDINAL SECTION
SHEET 5 OF 7
CHAINAGE 1220 – 1560

Drawing No. CA150/2/15
Drawn By [redacted]

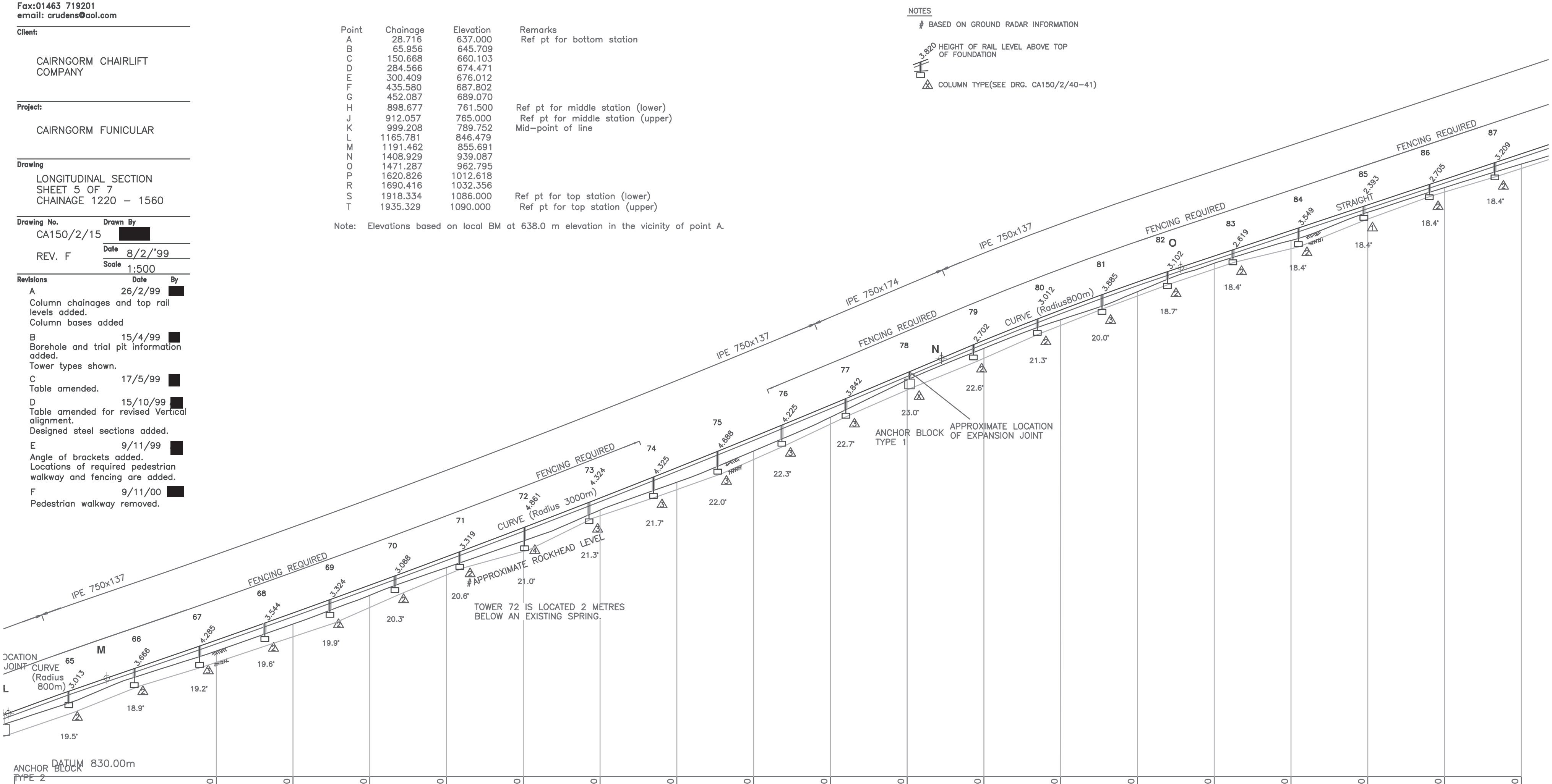
REV. F
Date 8/2/'99
Scale 1:500

Revisions — **Date** — **By**

	Date	By
A Column chainages and top rail levels added. Column bases added	26/2/99	[redacted]
B Borehole and trial pit information added. Tower types shown.	15/4/99	[redacted]
C Table amended.	17/5/99	[redacted]
D Table amended for revised Vertical alignment. Designed steel sections added.	15/10/99	[redacted]
E Angle of brackets added. Locations of required pedestrian walkway and fencing are added.	9/11/99	[redacted]
F Pedestrian walkway removed.	9/11/00	[redacted]

Point	Chainage	Elevation	Remarks
A	28.716	637.000	Ref pt for bottom station
B	65.956	645.709	
C	150.668	660.103	
D	284.566	674.471	
E	300.409	676.012	
F	435.580	687.802	
G	452.087	689.070	
H	898.677	761.500	Ref pt for middle station (lower)
J	912.057	765.000	Ref pt for middle station (upper)
K	999.208	789.752	Mid–point of line
L	1165.781	846.479	
M	1191.462	855.691	
N	1408.929	939.087	
O	1471.287	962.795	
P	1620.826	1012.618	
R	1690.416	1032.356	
S	1918.334	1086.000	Ref pt for top station (lower)
T	1935.329	1090.000	Ref pt for top station (upper)

Note: Elevations based on local BM at 638.0 m elevation in the vicinity of point A.



CHAINAGE	EXISTING GROUND LEVEL	COLUMN CHAINAGE	TOP RAIL LEVEL	TOP OF FOUNDATION LEVEL
1220.00	862.61			
	867.092	1232.650	870.012	866.468
1240.00	869.67			
	873.586	1249.613	876.107	872.783
1260.00	877.13			
	880.018	1266.541	882.303	879.235
1280.00	884.41			
	886.285	1283.434	888.603	885.284
1300.00	891.33			
	890.891	1300.291	895.004	890.143
	898.050	1317.112	901.508	897.184
1320.00	899.36			
	904.427	1333.896	908.114	903.789
1340.00	906.64			
	910.734	1350.644	914.822	910.134
1360.00	914.76			
	918.094	1367.354	921.632	917.407
1380.00	923.53			
	925.302	1384.027	928.544	924.702
1400.00	933.24			
	933.504	1400.641	935.549	
	940.499	1417.254	942.601	939.899
1420.00	941.53			
	947.033	1433.910	949.302	946.290
1440.00	949.00			
	952.714	1450.767	955.648	951.763
1460.00	956.29			
	959.226	1467.797	961.624	958.522
1480.00	963.74			
	965.310	1484.862	967.318	964.699
1500.00	949.68			
	970.329	1501.927	973.004	969.455
	976.896	1518.992	978.689	976.296
1520.00	977.02			
	982.359	1536.057	984.375	981.670
1540.00	983.47			
	987.603	1553.122	990.061	986.852
1560.00	989.59			

Drawing LONGITUDINAL SECTION 5 OF 7
CHAINAGE 1220 – 1560
Drawing No. CA150/2/15
Revision. F

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Drawing
LONGITUDINAL SECTION
SHEET 6 OF 7
CHAINAGE 1460 – 1800

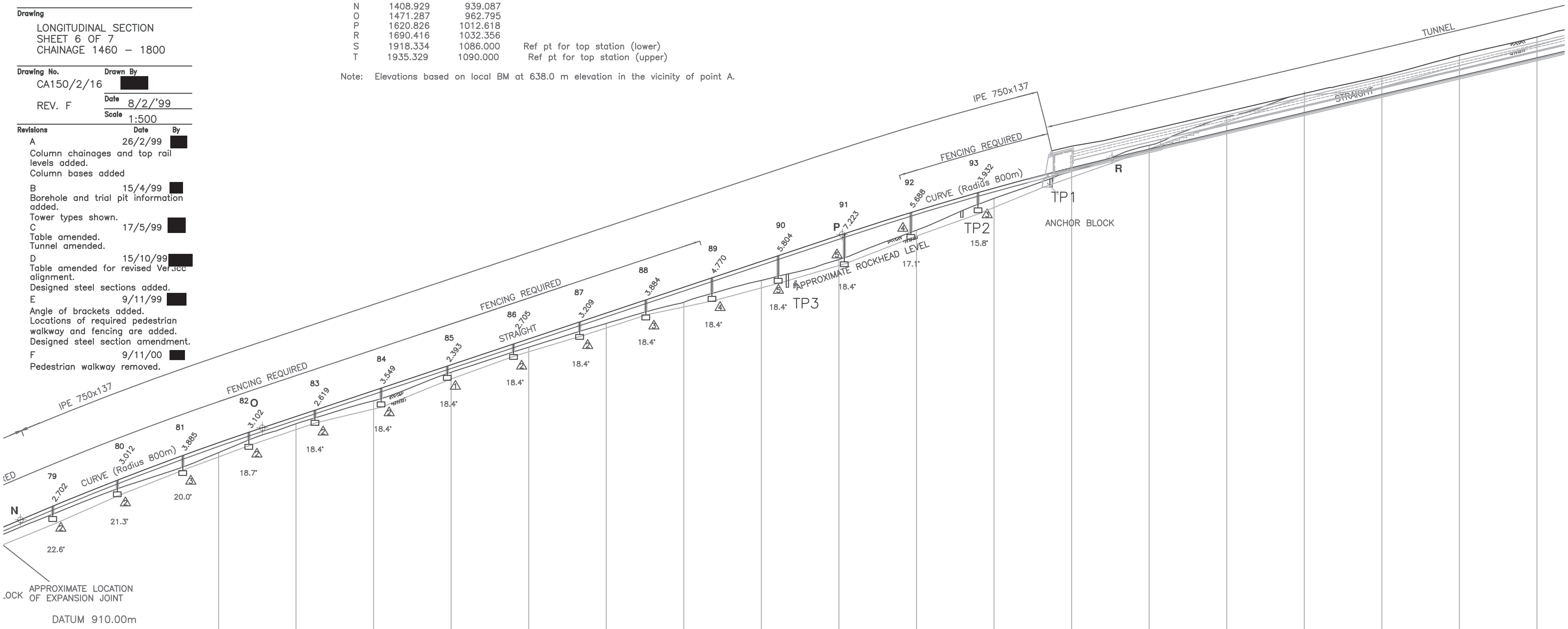
Drawing No. CA150/2/16
Drawn By [redacted]
REV. F
Date 8/2/'99
Scale 1:500

Revisions	Date	By
A Column chainages and top rail levels added. Column bases added	26/2/99	[redacted]
B Borehole and trial pit information added. Tower types shown.	15/4/99	[redacted]
C Table amended. Tunnel amended.	17/5/99	[redacted]
D Table amended for revised Vertcc alignment. Designed steel sections added.	15/10/99	[redacted]
E Angle of brackets added. Locations of required pedestrian walkway and fencing are added. Designed steel section amendment.	9/11/99	[redacted]
F Pedestrian walkway removed.	9/11/00	[redacted]

Point	Chainage	Elevation	Remarks
A	28.716	637.000	Ref pt for bottom station
B	65.956	645.709	
C	150.668	660.103	
D	284.566	674.471	
E	300.409	676.012	
F	435.580	687.802	
G	452.087	689.070	
H	898.677	761.500	Ref pt for middle station (lower)
J	912.057	765.000	Ref pt for middle station (upper)
K	999.208	789.752	Mid–point of line
L	1165.781	846.479	
M	1191.462	855.691	
N	1408.929	939.087	
O	1471.287	962.795	
P	1620.826	1012.618	
R	1690.416	1032.356	
S	1918.334	1086.000	Ref pt for top station (lower)
T	1935.329	1090.000	Ref pt for top station (upper)

Note: Elevations based on local BM at 638.0 m elevation in the vicinity of point A.

NOTES
\# BASED ON GROUND RADAR INFORMATION
3.820 HEIGHT OF RAIL LEVEL ABOVE TOP OF FOUNDATION
△ COLUMN TYPE(SEE DRG. CA150/2/40–41)



CHAINAGE	1460.00		1480.00		1500.00			1520.00		1540.00		1560.00		1580.00	
EXISTING GROUND LEVEL	956.29	959.226	963.74	965.310	969.68	970.329	976.896	977.02	982.359	983.47	987.603	989.59	992.677	995.03	997.380
COLUMN CHAINAGE		1467.797		1484.862		1501.927	1518.992		1536.057		1553.122		1570.187		1587.251
TOP RAIL LEVEL		961.624		967.318		973.004	978.689		984.375		990.061		995.746		1001.432
TOP OF FOUNDATION LEVEL		958.522		964.699		969.455	976.296		981.670		986.852		991.862		996.662

CHAINAGE	1600.00		1620.00			1640.00		1660.00	1680.00	1700.00	1720.00	1740.00	1760.00	1780.00	1800.00
EXISTING GROUND LEVEL	1000.72	1001.944	1006.09	1006.355	1013.468	1013.60	1020.144	1021.81	1028.79	1035.53	1042.86	1049.56	1053.33	1058.71	1063.44
COLUMN CHAINAGE		1604.316		1621.381	1638.536		1655.852								
TOP RAIL LEVEL		1007.117		1012.803	1018.290		1023.403		1029.831	1034.612	1039.319	1044.027	1048.734	1053.441	1058.149
TOP OF FOUNDATION LEVEL		1001.313		1005.580	1012.602		1019.471								

Drawing
LONGITUDINAL SECTION 6 OF 7
CHAINAGE 1460 – 1800
Drawing No. CA150/2/16
Revision. F

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Inverness IV1 1QU
Tel: 01463 719200
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email: crudens@aol.com

Client:
CAIRNGORM CHAIRLIFT
COMPANY

Project:
CAIRNGORM FUNICULAR

Drawing
LONGITUDINAL SECTION
SHEET 7 OF 7
CHAINAGE 1660 – 1920

Drawing No. CA150/2/17
Drawn By [redacted]
REV. F
Date 8/2/'99
Scale 1:500

Revisions **Date** **By**

A 26/2/99 [redacted]
Column chainages and top rail levels added.
Column bases added

B 15/4/99 [redacted]
Borehole and trial pit information added.
Tower types shown.

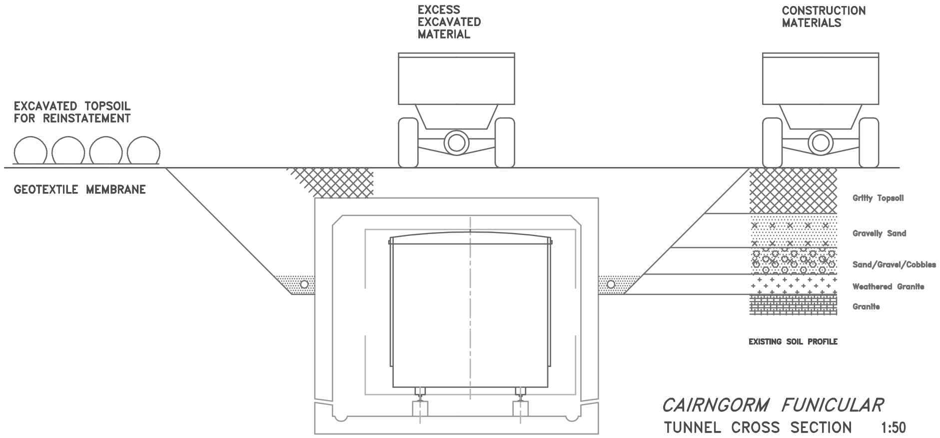
C 17/5/99 [redacted]
Table amended.
Tunnel amended.

D 15/10/99 [redacted]
Table amended for revised Vertical alignment.
Designed steel sections added.
Extent of Upper Station has been updated.
Supports from new station are added.

E 9/11/99 [redacted]
Angle of brackets added.
Locations of required pedestrian walkway and fencing are added.
Designed steel section amendment.

F 9/11/00 [redacted]
Pedestrian walkway removed.

DATUM 970.00m

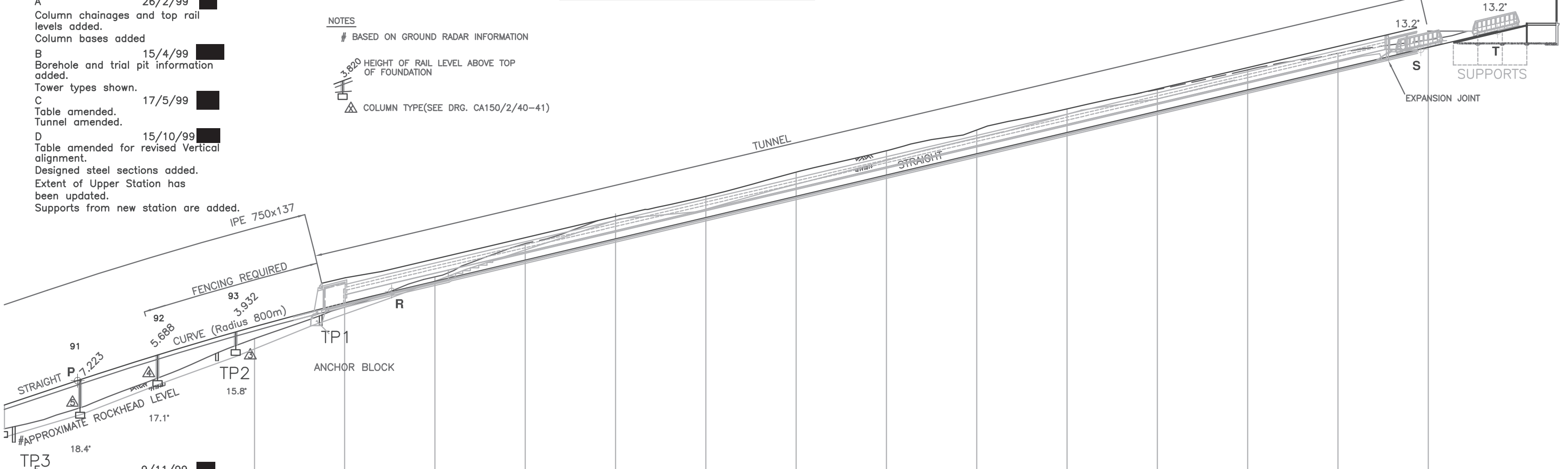


CAIRNGORM FUNICULAR
TUNNEL CROSS SECTION 1:50

Point	Chainage	Elevation	Remarks
A	28.716	637.000	Ref pt for bottom station
B	65.956	645.709	
C	150.668	660.103	
D	284.566	674.471	
E	300.409	676.012	
F	435.580	687.802	
G	452.087	689.070	
H	898.677	761.500	Ref pt for middle station (lower)
J	912.057	765.000	Ref pt for middle station (upper)
K	999.208	789.752	Mid–point of line
L	1165.781	846.479	
M	1191.462	855.691	
N	1408.929	939.087	
O	1471.287	962.795	
P	1620.826	1012.618	
R	1690.416	1032.356	
S	1918.334	1086.000	Ref pt for top station (lower)
T	1935.329	1090.000	Ref pt for top station (upper)

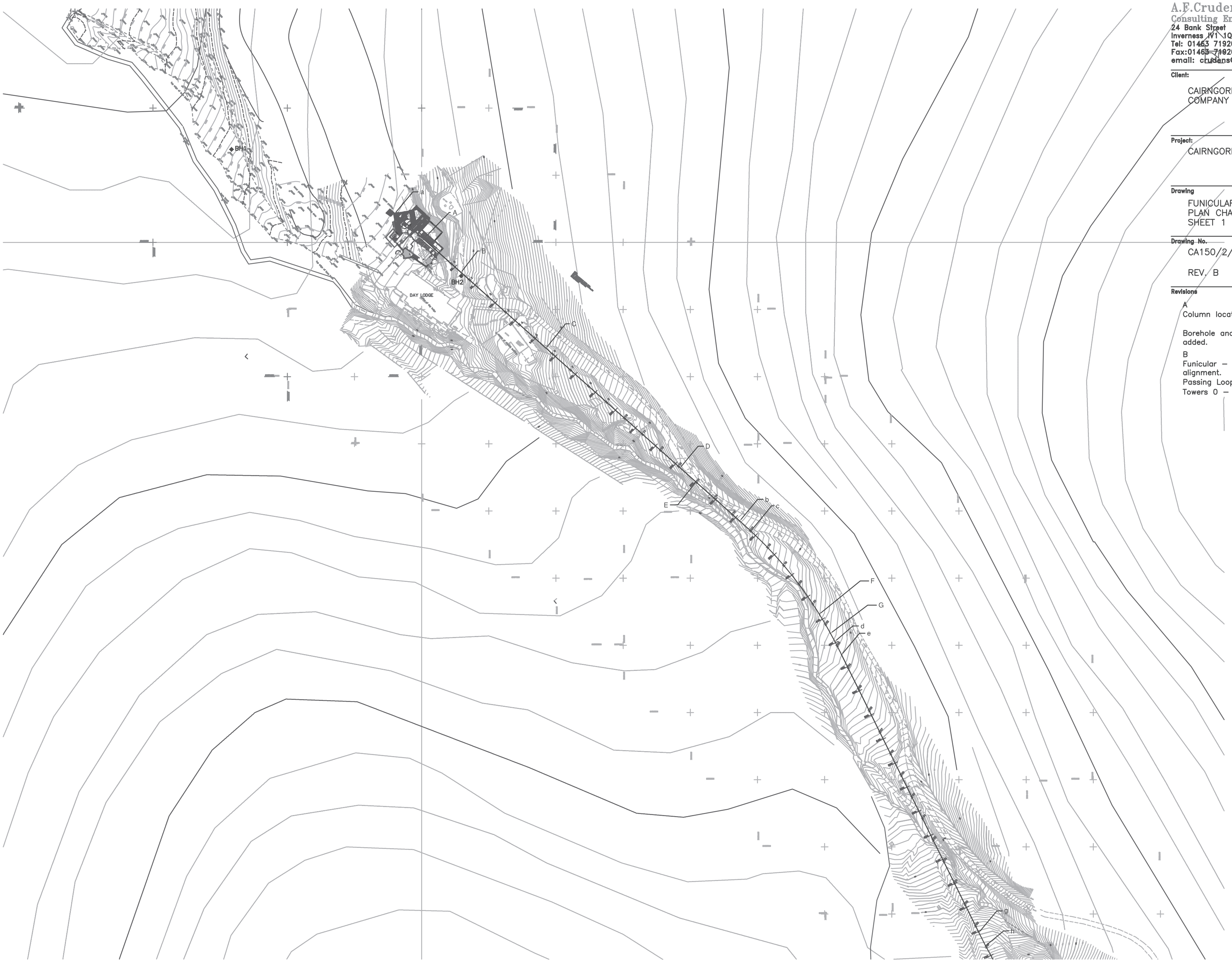
Note: Elevations based on local BM at 638.0 m elevation in the vicinity of point A.

NOTES
BASED ON GROUND RADAR INFORMATION
3.820 HEIGHT OF RAIL LEVEL ABOVE TOP OF FOUNDATION
△ COLUMN TYPE(SEE DRG. CA150/2/40–41)



CHAINAGE	1660.00	1680.00	1700.00	1720.00	1740.00	1760.00	1780.00	1800.00	1820.00	1840.00	1860.00	1880.00	1900.00	1920.00				
EXISTING GROUND LEVEL	1021.81	1028.79	1035.53	1042.86	1049.56	1053.33	1058.71	1063.44	1068.01	1072.72	1077.00	1081.37	1085.37	1087.99				
COLUMN CHAINAGE																		
TOP RAIL LEVEL		1029.831	1034.612	1039.319	1044.027	1048.734	1053.441	1058.149	1062.856	1067.563	1072.270	1076.978	1081.685	1086.392				
TOP OF FOUNDATION LEVEL																		

Drawing
LONGITUDINAL SECTION 7 OF 7
CHAINAGE 1660 – 1920
Drawing No. CA150/2/17
Revision. F



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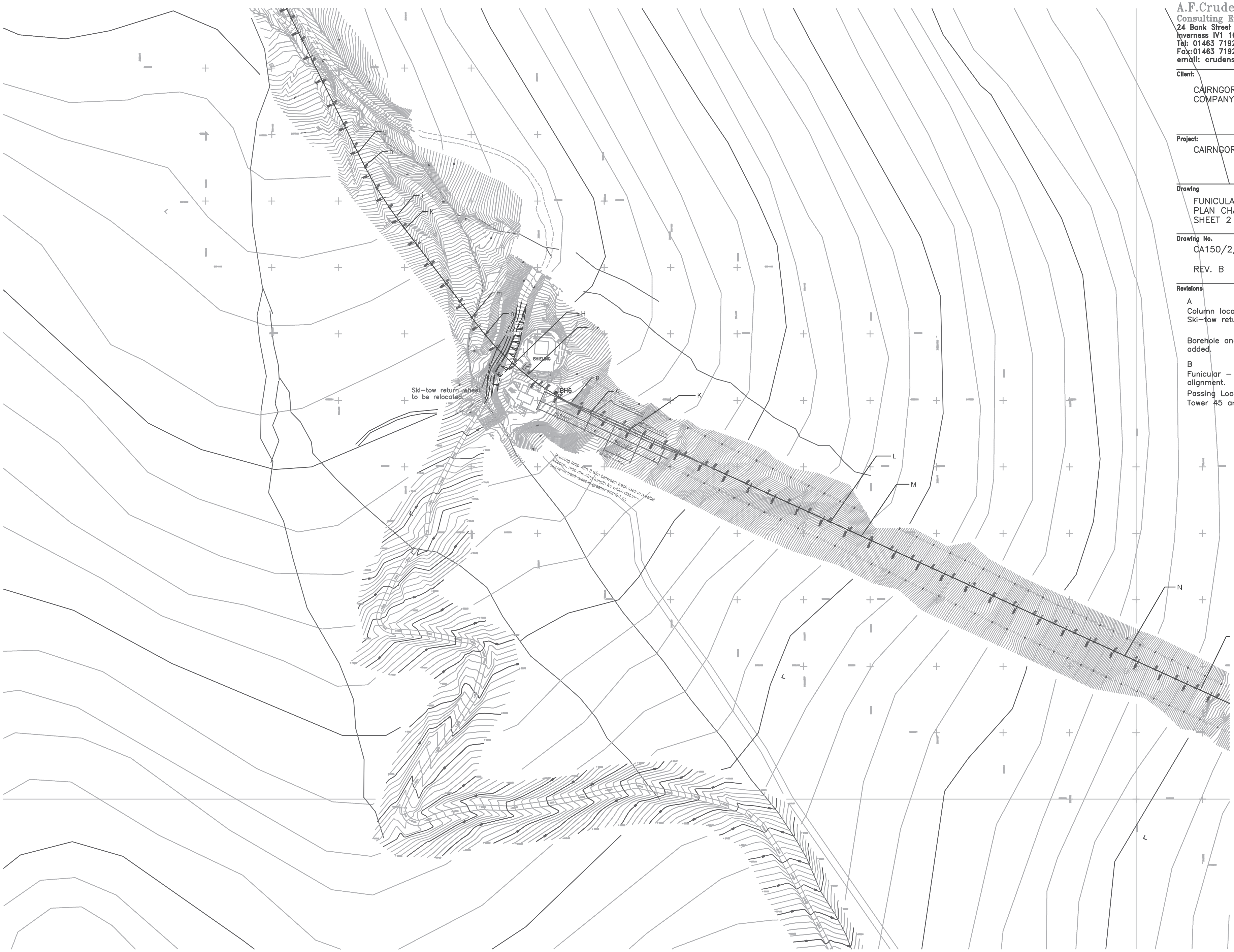
Client:
CAIRNGORM CHAIRLIFT
COMPANY

Project:
CAIRNGORM FUNICULAR

Drawing
FUNICULAR
PLAN CHAINAGE 0–720
SHEET 1 OF 3

Drawing No. CA150/2/31
Drawn By [redacted]
REV. B
Date FEB 1999
Scale 1:1250

Revisions	**Date**	**By**
A Column locations indicated.	2/3/'99	[redacted]
Borehole and Trial pit locations added.	2/4/'99	[redacted]
B Funicular – Horizontal geometry alignment. Passing Loop added. Towers 0 – 4 amended.	8/10/99	[redacted]



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Client:
CAIRNGORM CHAIRLIFT COMPANY

Project:
CAIRNGORM FUNICULAR

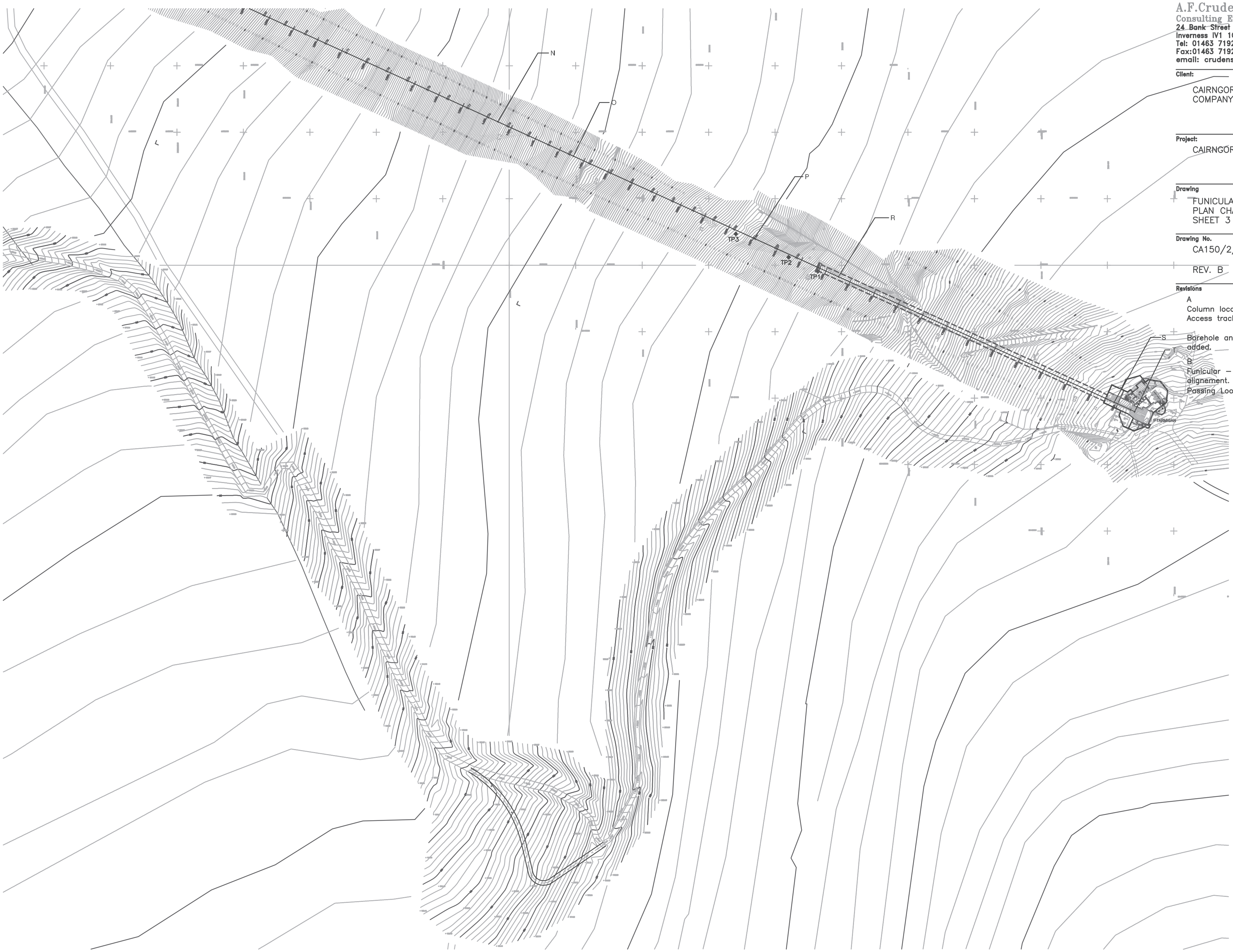
Drawing
FUNICULAR
PLAN CHAINAGE 580–1480
SHEET 2 OF 3

Drawing No.	**Drawn By**
CA150/2/32	[redacted]
REV. B	**Date** FEB 1999
	Scale 1:1250

Revisions **Date** **By**

A 2/3/'99 [redacted]
Column locations indicated.
Ski–tow return wheel indicated.
2/4/'99 [redacted]
Borehole and Trial pit locations added.

B 8/10/99 [redacted]
Funicular – Horizontal geometry alignment.
Passing Loop added.
Tower 45 amended.



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email: crudens@aol.com

Client:
CAIRNGORM CHAIRLIFT
COMPANY

Project:
CAIRNGORM FUNICULAR

Drawing
FUNICULAR
PLAN CHAINAGE 1220–1920
SHEET 3 OF 3

Drawing No. CA150/2/33
Drawn By [redacted]
Date FEB 1999
REV. B
Scale 1:1250

Revisions	**Date**	**By**
A Column locations indicated. Access track realigned	2/3/'99	[redacted]
Borehole and Trialpit locations added.	2/4/'99	[redacted]
B Funicular – Horizontal geometry alignement. Passing Loop added.	8/10/99	[redacted]

Appendix C Doppelmayr Operation and Maintenance Manual

C TECHNICAL MAIN DATA AND PERFORMANCE

1 SYSTEM

Funicular with two vehicles, in shuttle operation and a single track with passing loop type "Abt".

The hourly capacity in one direction with one intermediate stop and a speed of 10 m/s is 1200 passengers per hour.

The friction drive is at the top station.

The system is stabilized with a counter rope, tensioning at the bottom station with a hydraulic tensioning device.

1.1 Notation of vehicles

(Looking uphill)

- Left side of passing loop: No. 1
- Right side of passing loop: No. 2

2 Track

2.1 Topographical data

Length of rails: 1984.5

Length of line (travel distance):..................................1969.4 m

Horizontal length of line: ....1906.6 m

Bottom station (elevation at ref. pt. A):........................637,0 masl.

Intermediate station

  Elevation at ref. pt. H (lower): ....761,5 masl.

  Elevation at ref pt. J (upper): ....765,0 masl.

Top station

  Elevation at ref. pt. S (lower stop):.........1086 masl. (Summer stop)

  Elevation at ref. pt. T (upper stop): ....1090 masl. (Winter stop)

Vertical rise (pt. A to pt. T): ....453,0 m

Slope angle (° DEG)

  Maximum: .... 23.15° (~ 42,8%)

  Minimum: .... 3.81° (~ 8,4 %)

  At bottom station:................................ 13,2 ° (~ 23,5%)

  At intermediate station:......................... 14,7° (~ 26.2%)

  At top station:....................................... 13.2° (~ 23.5%)

2.2 Mechanical line equipment

Type of rails ....S 33

Rail fixation ....Tranosa SKL-1

Type of passing loop..Abt

Length of passing loop ca.:.......................................130 m

Gauge of track (midline – midline of rails):..................2000 mm

Distance midline of track to midline of rope: ....± 100 mm

Pedestrian walkway (looking uphill): ....no walkway

Number of straight line sheaves: ....(2 x 112) = 224 pieces

Number of tilted line sheaves:...................................(2 x 66) = 132 pieces

Nominal diameter of line sheaves:	400 mm
Lining of line sheaves:	
Straight sheaves:	insulating rubber
Tilted sheaves:	groove bottom: insulating rubber side: insulating polymer plastic
Bedding of line sheaves:	insulated

2.3 Ropes

Upper haul rope:

Supplier:	Fatzer AG, Switzerland
Diameter:	39 mm
Construction:	6X25 Filler Wire, compacted, PP-core, long lay rightwards
Unit weight:	5.96 kg/m
Tensile strength:	1860 N/mm^2
Metallic cross section:	684.7 mm^2
Minimum braking force:	1108 kN
Supply length:	2100 m + 9 m EMPA
Surface of wire	galvanized
Modulus of elasticity:	70 kN/mm^2
Thermal expansion coefficient	1,17E-05 1/K

Lower haul rope (counter rope)

Supplier:	Fatzer AG, Switzerland
Diameter:	24 mm
Construction:	6X19 Seale, compacted, PP-core, long lay rightwards
Unit weight:	2.24 kg/m
Tensile strength:	1770 N/mm^2
Metallic cross section:	256.8 mm^2
Minimum braking force:	404 kN
Supply length:	2070 m + 9 m EMPA
Surface of wire	galvanized
Modulus of elasticity:	70 kN/mm^2
Thermal expansion coefficient	1,17E-05 1/K

3 Drive

3.1 Main drive (mechanical)

System: Twin AC motor drive, controlled by a frequency converter. The serial connected AC motors are coupled by a clutch to the gearbox, which is also coupled by a clutch to the bull wheel. Each AC motor is capable of driving the system at half speed with full load.

Location:	top station (Ptarmigan)
Installation level	1088 masl.
Required nominal power (according to travel diagram):	410 kW
Required maximal power (static operation):	936 kW
Required maximal power (acceleration):	1248 kW

Motor:..ABB M2BA 400LKC 6 B3

Installed motor power:

- Nominal: 2 x 500 kW
- Peak: 2 x 880 kW

Rotation speed of motor (speed of 10 m/s):................1553.3 rpm

Gearbox: ....three-stage bevel-spur gear

Supplier ....Flender

Type:..B3 SH18 A,

Ratio: 25.936

Lubrication and cooling system:.................................circulated lubrication by gear pump

Couplings:

Motor – motor:..elastic bolt coupling
RUPEX RWS 400

Motor – gearbox:..elastic bolt coupling
RUPEX RWS 400

Gearbox – bull wheel (drive shaft): ....elastic bolt coupling, electrically
isolated RUPEX RWS 1120

3.2 Main drive (electrical):

Type of electrical Motor:..ABB M2BA 400 LKC 6 B3

Nominal power:..1000 kW (2x500 kW)

Peak power:..1660 kW

Maximal current: ....880 A

Voltage:..3 x 400 V

Nominal frequency:..50 Hz

3.3 Rope drive

System: Adhesion drive at top station with a double groove bull wheel, a double groove counter wheel and a single groove deflection wheel. Hydraulic tensioning device at bottom station.

Bull wheel:

Nominal diameter (groove bottom):	3150 mm
Active diameter (centre of rope):	3189 mm
Contact arc of rope:	2,20 x π (396° deg)
Rotation speed (10 m/s travel speed)	59,89 rpm.
Lining:	double groove, Becorit
Mass	5210 kg

Counter wheel:

Nominal diameter (groove bottom):	3150 mm
Active diameter (centre of rope):	3189 mm
Lining:	double groove, polymer plastic
Mass	4200 kg

Deflection wheel:

Nominal diameter (groove bottom):	3150 mm
Active diameter (centre of rope):	3189 m
Lining:	single groove, polymer plastic
Mass	2100 kg

Deflection wheels (bottom station) for counter rope:

Nominal diameter (groove bottom):	2240 mm
Active diameter (centre of rope):	2264 mm
Contact arc of rope:	1 x π
Lining:	single groove, polymer plastic
Mass	700 kg
Number of deflection wheels:	3 pieces

3.4 Auxiliary drive

System: Diesel – hydraulic drive, connected to the main shaft by means of a switch coupling.

Diesel engine:	IVECO aifo, water cooled
Type:	8210 SI 15
Engine power:	210 kW
Rotation speed (diesel):	2100 rpm
Hydraulic pump:	A4 VG 180
Hydraulic motor:	A2 FM 160
Alternator	20 kVA
Travel speed:	1,3 – 1.5 m/s
Installation level:	1088 masl

4 Brakes

4.1 Service brake

System: The service brake unit acts on a brake disc, mounted on the high-speed shaft of the gearbox. The deceleration of the service brake is controlled.

Type:	DMCH 10kN spring-loaded disc brake, hydraulically opened.
Deceleration (controlled):	≈ 0.5 m/s^2
Maximum brake force (at brake diameter)	10 kN
Diameter of brake disc:	1000 mm
Active brake disk diameter	445 mm
Thickness of brake disc:	60 mm
Number of brake units:	2 x 2 pieces
Number of brake discs:	2 pieces

4.2 Emergency brake

System: The emergency brake unit acts on the rims (machined lateral surfaces) of the bull wheel. The deceleration of the service brake is not controlled.

Type:	DMCH 60 kN spring-loaded brake unit, hydraulically opened.
Deceleration:	not controlled
Maximum brake force (at linings)	60 kN
Active brake diameter:	3040 mm
Number of brake units:	2 pieces
Number of over speed centrifugal switches:	2 mounted on bull wheel

Supplier of hydraulic brake unit: Rexroth AG, Switzerland

5 Rope tensioning device

5.1 Haul rope tensioning

System: Hydraulic tensioning of the counter rope by a hydraulic piston, attached to a tensioning bogie.

Nominal tension force: 49 kN

Maximum admissible force: 260 kN

Approximate nominal hydraulic pressure: 70 bar

Safety valve set pressure: 190 bar

Maximum travel of tensioning bogie: 6 m

Maximum piston speed: 15 cm/s

Nominal electric motor power: 15 kW

Supplier of hydraulic unit and tension cylinder: Rexroth AG, Switzerland

6 Carriages

6.1 Carriage body:

Supplier: .... Gangloff AG, Switzerland

Number of vehicles: 2

Load volume: .... 120 +1 persons per carriage

Number of compartments per carriage: 4

Number of doors per carriage: .... 2 (one each side)

Wideness of doors: .... 920 mm

Door drive: .... electro-pneumatic

Flight line of stairs: .... 1000 mm (empty carriage)

Dimensions:

Width over all:	3200 mm
Length over all:	10736 mm
Height (top of rails to roof):	3300 mm

Side area exposed to the wind: 31.93 m2

Air drag coefficient: 1,30

Centre of side area:* .... 1722 mm

Centre of gravity of empty carrier *: .... 1300 mm

Inclination of floor (compared to rails): .... 13,5°

Standing surface: .... 0,2 m^2 per person

Electricity supply: .... by contact rails in the terminals (top and bottom station) and pantographs mounted on the roof of the carriages.

* measured from top of rails

6.2 Bogies and track brake

Bogies:

System: DMCH bogies 2000

Number of bogies: 2 per carriage

Distance between the bogies: 6200 mm

Flanged-wheel:
- Number per bogie: 2 pieces
- Nominal diameter: 400 mm
- External diameter: 450 mm

Flat-wheel:
- Number per bogie: 2 pieces
- Nominal diameter: 400 mm

Wheelbase of bogie: 1350 mm

Track brakes (Onboard brakes):

System: spring-loaded rail brake,
hydraulically opened.

Number of track brakes
- On uphill bogie 2 pieces
- On downhill bogie 1 piece

6.3 Rope fixings

Haul rope fixing:

System	rope anchorage drum
Diameter of rope fixing drum:	880 mm
Contact arc of rope	1290°
Maximum static tension of rope:	125 kN
Minimum static tension of rope:	42 kN
Slack rope release:	5 kN
Load measuring device:	no

Counter rope fixing:

System	rope anchorage drum
Diameter of rope fixing drum:	580 mm
Contact arc of rope	1290°
Nominal tension due to hydraulic tensioning device:	22 kN
Maximum tension of rope (rail brake acting)	135 kN
Maximum static tension of rope:	32 kN
Slack rope release:	10 kN
Load measuring device:	no

6.4 Masses of carriages:

	Calculated weight [kg]		Weighted weight [kg]	
	Car. 1	Car. 2	Car. 1	Car. 2
Carriage body, supplier Gangloff AG	7800	7800		
Glass roof	330	330		
Compressor for door drive	160	160		
Battery	150	150		
Battery charger	100	100		
Hydraulic equipment	150	150		
Electric equipment	350	350		
Power pick-up fingers	10	10		
Cross beam of bogie (downhill)	250	250		
Cross beam of bogie (uphill)	250	250		
Cross beam of rope fixing	390	390		
Rope fixing drum of haul rope incl. rope	320	320		
Rope fixing drum of counter rope incl. rope	180	180		
Extra weight (tools, ladders, other)	50	50		
Carriage body	**10490**	**10490**		
Bogie downhill	1650	1650		
Bogie uphill	1650	1650		
One track brake (onboard brake), downhill	290	290		
Two track brakes (onboard brake), uphill	580	580		
Rope saddle downhill	60	60		
Rope saddle uphill	60	60		
Rail greasing device	40	40		
Rolling stock	**4330**	**4330**		
Carriage attendant	80	80		
Dead weight of carriage	**14900**	**14900**	**14840**	excl. attendant
Payload (120 persons * 80kg)	**9600**	**9600**		
Full loaded carriage	**24500**	**24500**		

6.5 Maintenance Trolley

System	Trolley with two flanged wheels and two flat wheels, with unmistakable coupling for attachment to the correct carriage
Dead weight:	350 kg
Payload:	450 kg
Wheel diameter:	200 mm
Length:	2000 mm
Width:	2600 mm
Height (without lifting aid):	1270 mm

7 Electrical Drive-, control-, and supervision system

Supplier.. [redacted]

For information refer to manuals of [redacted]

8 Low voltage control system

All low voltage control system equipment incl. remote control, safety circuits and telephone installations are provided by Frey AG. For technical data and details refer to manuals of Frey AG.

9 Operating and performance data

Nominal travel speed: 10,0 m/s

Acceleration: 0,35 m/s^2

Deceleration (controlled by frequency converter): 0,35 m/s^2

Speed within stations (creep speed): 0,80 m/s

Timing:

Travel time *: 295 s

Electrical tests: 5 s

Embarking, debarking and door operation: 60 s

Cycle time: 360 s

Number of cycles per hour: 10

Capacity in one direction per hour **: 1200 pph

* calculated with de- and acceleration and a stop of 30 s at the intermediate station.

** incl. stop at the intermediate station

Appendix D Structural Design Check Certificate

A. F. Cruden Associates
Consulting Engineers

Cairngorm Funicular Railway

A. F. Cruden Associates.

Certificate of Compliance with HM Railway Inspectorate Requirements and Other Relevant Standards)

(Part of the Works.)

Scope: **Civil / Structural Design of Funicular Structure**
Certificate No:AFC D/02

---

1. Location of the works/plant/equipment.

Funicular structure from Base Station to the tunnel portal.

---

2. Description of the proposal.

Civil / structural design for structure comprising:

a. Anchor blocks (including rock anchors).
b. Towers (including reinforced concrete base, reinforced concrete column, precast column sections, precast concrete cross heads).
c. Bearings.
d. Precast concrete beams.
e. Structural steel beams, bracing, maintenance / deicing walkway & passing loop steelwork.

---

3. Details of the supporting documentation.

Drawings:	Refer to attached pages.
Calculations:	Refer to Category 3 Check Certificate and Design Information Sheet for design parameters.
Safety Principles:	Refer to attached pages.
Risk Assessment:	Refer to attached pages.
Certificates	Cat 3 check for funicular structure. Ancon Design Certificate AN D/01

---

4. A list of the principal standards used in the design, construction/installation and use of the works, plant or equipment.

Refer to Category 3 Check Certificate and Design Information Sheet.

---

5. A complete list of all deviations, if any, from HM Railway Inspectorate's requirements and other relevant standards for which dispensation is required. (Where there are no such deviations the certificate must state this clearly.)

Design Deviation Request 001.

I confirm that these proposals have been designed in accordance with relevant HM Railway Inspectorate Requirements and the standards listed at 4 above apart from the exceptions set out at 5 above.

Organisation: A. F. Cruden Associates.

Name: [redacted] **Position:** [redacted]

Signature: [redacted] **Date:** 21st November 2001.

---

3. Drawings.

Drawing No	Rev	Drawing Title
Towers		
CA150/2/44	G	Tower Elevations Sheet 1 of 2
CA150/2/45	F	Tower Elevations Sheet 2 of 2
CA150/2/67	D	R-C Details - Towers
CA150/2/68	B	R-C Details :- 4m Long Base Type 1A
CA150/2/69	B	R-C Details :- 4.5m Long Base Type 2C
CA150/2/70	C	R-C Details :- 4.5m Long Base Type 2A
CA150/2/71	C	R-C Details :- 4.5m Long Base Type 2B
CA150/2/72	C	R-C Details :- 4.8m Long Base Type 3
CA150/2/73	B	R-C Details :- 5.15m Long Base Type 4
CA150/2/74	B	R-C Details :- 5.6m Long Base Type 5
CA150/2/75	A	R-C Details :- 6.0m Long Base Type 6
CA150/2/60	B	R-C Details Crosshead 1 - 91No. required
CA150/2/64	-	R-C Details Crosshead 2 - 2No. required
CA150/2/77	-	R-C Details : Crosshead 51
CA150/2/78	A	R-C Details : Crosshead 52+56
Anchor Blocks		
CA150/2/38	-	Anchor Block 48 - RC Details
CA150/2/63	D	Anchor Block R.C. Details - Type 3
Precast Beams		
CA150/2/76	D	Precast Concrete Beam Detail 1 of 2
CA150/2/79	C	Precast Concrete Beam Detail 2 of 2
CA150/2/39	A	Insitu Diaphragm Details
Steelwork		
CA150/2/92	-	Passing loop, enlarged part section sheet 1
CA/150/2/47	D	Steelwork Superstructure of Passing Loop
CA150/2/49	B	Rail Bolt / Beam Support / and Bearing Details
CA150/2/93	-	Passing loop, enlarged part section sheet 2
CA150/2/94	-	Passing loop, enlarged part section sheet 3
CA150/2/95	-	Passing loop, enlarged part section sheet 4
CA150/4/18	-	Railway support beams Bottom Station-anchor block 0

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3. Safety Principles

a. No 3 Infrastructure – Protection of the Railway.
b. No 4 Infrastructure – Clearances for People.
c. No 5 Infrastructure – The Track.
d. No 6 Infrastructure – Clearances for Track.
e. No 7 Infrastructure – Earthworks and Structures Under the Track.
f. No 8 Infrastructure – Earthworks and Structures Over the Track.

3. Risk Assessment

1. Hazard Analysis and Risk Assessment:
   i. Funicular Structure.

A.F. Cruden Associates
24 Bank Street
Inverness IV1 1QU
Tel 01463 719200
Fax 01463 719201
E-mail: crudens@aol.com

Cairngorm Funicular

Sub-section

TRACK SUPPORT BEAMS AND PIERS

Design Information

Job ref.	CA150
Sheet No.	2/01C
Made by	[redacted]
Date	**Sep 2001**
Checked by	

~~Client~~ ~~Cairngorm Chairlift Company~~
Architect Unwin Jones Partnership

~~Engineer responsible~~
[redacted]

Building Regulation Authority or other

Highland Council: Badenoch & Strathspey District.
HMRI.

Relevant Building Regulations and Design Codes

BS6399 Part 2: Code of Practice for Wind Loads.
BS5950 Part 1: The Structural Use of Steel in Buildings
BS5400: Steel, Concrete and Composite Bridges
BS 8110 Parts 1 & 2: Structural use of concrete
BS8004: Foundations
Draft prEN 13107: Safety requirements for passenger transportation by rope - Civil Engineering Works

Intended Use of Structure & description

Support to funicular railway carriage, comprising pairs of steel beams latticed together, radiussed and cambered to suit the required track profile, spanning 18 metres between reinforced concrete support piers. Sliding bearings at each column cross-head, fixed bearings at expansion joints, at nominally 300 metre centres along track.

Fire Resistance requirements

None

General loading conditions

Carriage (8 wheel) *weights based on information from supplier (Doppelmayr)*
Total unladen weight:- **130** kN.
Total laden weight:- **220** kN.

Principal Load Cases:
1. Carriage fully laden + wind at **50**m/s
2. Carriage unladen + wind at **50**m/s
3. No carriage + wind at 75m/s
4. Carriage unladen + wind at 75m/s (short term)

Wind loading conditions

Speed
Normal operational wind speed: 35m/s. Maximum operational wind speed: **50**m/s.
Maximum design speed 75m/s.
Basic wind speed Vb = 23m/s. Site wind speed Vs = 44.9m/s (BS6399)

Factors

Altitude factor	$S_a = 2.1$	Fetch factor	$S_f = 1.02$
Directional factor	$S_d = 0.93$	Turbulence factor	$S_t = 0.178$
Seasonal factor	$S_s = 1.0$	Gust peak factor	$g_t = 3.16$
Probability factors	$S_p = 1.0$ (cases 1 to 3)	Topographic location factor	$s = 0$
	$S_p = 0.77$ (case 4)	Topographic increment	$S_h = 0$
		Terrain & building factor	$S_b = 1.59$

Subsoil conditions

Weathered granite bedrock overlain with dense to very dense natural gravelly sand with cobbles, under gritty topsoil.
N values for gravelly sand > 50. Allowable bearing pressure 400 kN/m^2.

Foundation type

Insitu reinforced concrete pad foundations to all support piers, designed for vertical, lateral and overturning loads applied at bearings. Foundations bear onto gravelly sand material.
Insitu reinforced concrete anchor blocks at expansion joint locations, bearing onto rockhead, tied back with rock anchors.

Material data

Concrete
All concrete grade RC40 to BS5328. (28 day strength 40N/mm^2).
Exposed concrete to be air-entrained.
Exposed aggregate finish to piers.

Reinforcement
High yield deformed bars to BS4449.
Cover to be **45**mm (external faces)

Other relevant information

Track support beams resting on sliding bearings supported off a precast reinforced concrete cross-head member at the top of each pier.
RC piers and cross-heads to comprise 90mm thick precast concrete outer shell as permanent shutter, with insitu reinforced concrete core.

STRUCTURAL DESIGN CHECK CERTIFICATE

Date: 28-10-02

1.0 Name of Scheme: Cairngorm Funicular Railway

1.1 Design Engineer:
A F Cruden Associates
24 Bank Street
Inverness
IV1 1QU

1.2 Checking Engineer:
Bullen Consultants
Kimberly House
169 Elderslie Street
Glasgow
G3 7JR

2.0 Brief Description of Structure:

2.1 Structure Type: Support structure to funicular railway carriage, comprising pairs of precast reinforced concrete beams with steel lattice horizontal bracing, spanning 18 metres between reinforced concrete support piers with precast concrete cross-heads. Beams tied at supports via an insitu R.C. diaphragm, providing full continuity. Piers comprise 90mm thick precast concrete outer shall as permanent shutter, with insitu r.c. core. Sliding bearings at each column cross-head, fixed connections at expansion joint anchor blocks positioned at nominally 300m intervals along length of track.

2.2 Foundation Type: Insitu reinforced concrete pad foundations to all support piers, designed for vertical, lateral and overturning loads applied at bearings. Foundations bear onto gravelly sand material. Insitu reinforced concrete anchor blocks at expansion joint locations, either bearing onto rockhead and tied back with rock anchors, or onto gravelly sand stratum with additional rock anchors/piles.

2.3 Materials and Finishes

2.3.1 Concrete: Insitu concrete grade RC40 to BS5328. Precast concrete grade RC50. All exposed concrete air entrained. Precast concrete cross-heads and pier shutters to have exposed aggregate finish.

2.3.2 Reinforcement: High yield deformed bars to BS4449. Cover to be 45mm (external faces)

2.3.3 Steelwork: Grade S275 to BS EN 10025 and BS EN 10210. Shot blasted and galvanised to 85 micron thickness.

3.0 Design Criteria:

3.1 Loading:

3.1.1 Wind load cases:

1. Principal operational load: Carriage fully laden and wind at 35m/sec. Dynamic factor = 1.3
2. Emergency evacuation load: Carriage + 5.0T Kentledge + wind at 50m/sec. Dynamic factor excluded.
3. Storm load: No carriage + wind at 75 m/sec max. (at top station), 56 m/sec min. (at Bottom Station). Reduced partial load factor $\gamma_{fL} = 1.1$
4. Accidental case: Carriage clamped to rails + 5.0T kentledge + wind at 75m/sec max., 56 m/sec. min. Reduced partial load factor $\gamma_{fL} = 1.1$

3.1.2 Carriage loads:

i) Unladen weight 130 k/N total
ii) Laden weight 220 k/N total
iii) Braking loads
iv) Acceleration loads
v) Centrifugal loads at curves.
vi) Lateral loads due to cable tension at curves
All loads detailed by carriage manufacturer (Doppelmayr)

4.0 Standards:

4.1 Wind loads derived from:

i) Study Report "The Prediction of Wind Speed in Coire Cas for the Cairngorm Chairlift Company", by the University of Edinburgh, commissioned to assess wind speeds by computer modelling.

ii) BS6399 : Part 3 : "Code of Practice for Wind Loads", using the following data:

Basic wind speed V_b = 23m/s
Site wind Speed V_s = 44.9m/s
Altitude factor S_a = 2.1
Directional factor S_d = 0.93
Seasonal factor S_s = 1.0
Probability factor S_p = 1.0
Fetch factor S_f = 1.02
Turbulence factor S_t = 0.178
Gust peak factor gt = 3.16
Topographic location factor S = 0
Topographic increment = 0
Terrain and building factor = 1.59

4.2 Design Standards:

BS 5400 : Part 2 : Specification for Loads
BS 5400 : Part 3 : Code of Practice for design of Steel Buildings
BS 5400 : Part 4 : 1990 : Code of Practice for design of Concrete Bridges.
BS 8004 : Foundations
Draft prEN 13107 : Safety Requirements for passenger transportation by rope - Civil Engineering Works.

4.3 Departures from Standards:

Draft prEN 13107 adopted for serviceability criteria viz. deflection of concrete beams. (Departure from BS 5400 : Part 4).

BS 5400 : Part 2 used for load combination factors only, except as stated in 3.1.1. Loads obtained from funicular manufacturer, or derived otherwise viz. wind loads.

5.0 Structural Analysis:

5.1 Description of idealised structure to be used for analysis, and methods of analysis used.

5.1.1 Idealised structure: 3-span pair of continuous beams, pinned supports one end, all other supports roller. Lattice bracing all pin-ended.

5.1.2 Method of analysis: Linear elastic theory

5.2 Computer software used.

5.2.1 **Original** Design:
CADS Analyse 3D v1.78 (Build 406) - 3-dimensional analysis
CADS Steelwork member designer v1.16 (Build 135)
Microsoft Excel 5.0:
SCALE (Structural Calculations Ensemble)

5.2.2 **Check** Design:
Microsoft Excel 5.0:
SCALE (Structural Calculations Ensemble)
LEAP5 v 6.2.2
SAM v 4.50

6.0 Ground Conditions:

6.1 Soils investigation report.

Site investigation report carried out by HTS Associates for Highland Council.

Soils typically comprise weathered granite bedrock overlain with dense to very dense natural gravelly sand with cobbles, under gritty topsoil. N values for gravelly sand greater than 50, allowable bearing pressure 400 kN/m^2

7.0 List of drawings and documents on which the design check was based:

7.1 Drawings:

CA150/2/1D	Site Plan Sheet 1 of 7	Chainage 0 to 340
2C	Site Plan Sheet 2 of 7	Chainage 360 to 700
3C	Site Plan Sheet 3 of 7	Chainage 700 to 1040
4C	Site Plan Sheet 4 of 7	Chainage 920 to 1260
5C	Site Plan Sheet 5 of 7	Chainage 1220 to 1560
6C	Site Plan Sheet 6 of 7	Chainage 1460 to 1800
7C	Site Plan Sheet 7 of 7	Chainage 1660 to 1920
8B	Funicular - Horizontal Geometry	
11D	Longitudinal Section Sheet 1 of 7	Chainage 0 to 340
12D	Longitudinal Section Sheet 2 of 7	Chainage 360 to 700
13D	Longitudinal Section Sheet 3 of 7	Chainage 700 to 1040
14D	Longitudinal Section Sheet 4 of 7	Chainage 920 to 1260
15D	Longitudinal Section Sheet 5 of 7	Chainage 1220 to 1560
16D	Longitudinal Section Sheet 6 of 7	Chainage 1460 to 1880
17D	Longitudinal Section Sheet 7 of 7	Chainage 1660 to 1920
18A	Funicular - Vertical geometry	
31A	Funicular Plan Chainage 0 - 720	Sheet 1 of 3
32A	Funicular Plan Chainage 580 - 1400	Sheet 2 of 3
33A	Funicular Plan Chainage 1220-1900	Sheet 3 of 3
34	Site Plan of Tunnel - Chainage 1680 - 1920	
39A	Insitu Diaphragm details	
40B	Support Tower details	

44B	Tower elevations - Sheet 1 of 2
45B	Tower elevations - Sheet 2 of 2
47D	Steelwork superstructure of passing loop
49A	Rail bolt/beam support and bearing details.
57	Elevations of tower 48 to 58 (located at the passing loop)
60B	R.C. details Crosshead 1 : 91 No. required
63A	Anchor Block R.C. details - Type 3
67A	R.C.Details Towers
68A	R.C. Details 4m Long Base Type 1A
69B	R.C. Details 4.5m Long Base Type 2C
70B	R.C. Details 4.5m Long Base Type 2A
71C	R.C. Details 4.5m Long Base Type 2B
72C	R.C. Details 4.8m Long Base Type 3
73B	R.C. Details 5.15m Long Base Type 4
74B	R.C. Details 5.6m Long Base Type 5
75A	R.C. Details 6.0m Long Base Type 6
76	Precast Concrete beam detail 1 of 2
77	R.C. Details - Crosshead 51
78	R.C. Details - Crossheads 52 and 56
88A	Precast Concrete beams/cross members
92	Passing Loop, enlarged part section sheet 1
93	Passing Loop, enlarged part section sheet 2
94	Passing Loop, enlarged part section sheet 3
95	Passing Loop, enlarged part section sheet 4
CB 03	Concrete Beam Structure.

7.2 <u>Calculations</u>: Doppelmayr sheets D1 to D9 for wheel loads.

7.3 <u>Reports:</u>

- HTS Associates Site Investigation report

- University of Edinburgh report "The Prediction of Wind Speed in Coire Cas for the Cairngodm Chairlift Company".

8.0 Scope of the Design Check

8.1 Structural elements to be checked for ultimate and serviceability limit states:

Concrete beams, diaphragms and steel bracing;
reinforced concrete cross-heads;
support piers;
foundations and anchor blocks.

8.2 Design parameters:

Derivation of wind loads
Permissible bearing pressures.

Design Engineer:

Signed [redacted]

Name [redacted]

Qualifications [redacted] Team Leader

Name of Organisation

A F Cruden Associates
24 Bank Street
Inverness
IV1 1QU

Date 1st Nov. 2002.

Checking Engineer:

Signed ......[redacted]...

Name ......[redacted]......................

Qualifications......[redacted]..................... Team Leader

Name of Organisation

Bullen Consultants
Kimberley House
169 Elderslie Street
Glasgow
G3 7JR

Date 30 OCTOBER 2002.

Appendix E Relevant photos from Previous Inspection Reports

	From: ADAC Structures – Factual report on the sliding bearings to the funicular railway – 05-09-2018 Showing: Sliding pot bearing reaching the end of the stainless steel wearing plate at Pier 91.
	From: ADAC Structures – Factual report on the sliding bearings to the funicular railway – 05-09-2018 Showing: Guided pot bearing reaching the end of the stainless steel wearing track at Pier 61.

	From: ADAC Structures – Funicular railway inspection report – 2018 – 24-07-2018 Showing: Cracking of a precast beam bottom flange where it meets the bearing plate.
	From: ADAC Structures – Funicular railway inspection report – 2018 – 24-07-2018 Showing: In-situ joint at Pier 9 in poor condition. Cracking noted at the in-situ / precast interface and within the in-situ mass.

Appendix B Bearing Monitoring Report

CAIRNGORM MOUNTAIN LTD / HIGHLANDS AND ISLANDS ENTERPRISE

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Bevis Marks House
24 Bevis Marks
London
EC3A 7JB

TEL +44 207 9407 600
WWW cowi.com

CAIRNGORM FUNICULAR RAILWAY- BEARING MONITORING

TECHNICAL NOTE: REVIEW OF VIDEO MONITORING

CONTENTS

1 Introduction

Previous inspection reports prepared by ADAC structures highlighted bearing movements close to the limit of their articulation. Monitoring of these bearings would confirm structural articulation was similar to theoretical predictions.

Video monitoring has been undertaken by ADAC Structures at three piers, 44, 61 & 91. This technical note covers COWI's review of these videos.

The purpose of this monitoring is to establish whether the structure is articulating as intended and to correlate the theoretical movement against actual observations. This will permit a review of permissible movement ranges and associated temperature limits.

2 Summary of Data

- **P44 –** 5no. sets of data, covering a continuous period from 3/10/2018 through to 9/11/18.

PROJECT NO.	DOCUMENT NO.
A116993	TN-03-003

VERSION	DATE OF ISSUE	DESCRIPTION	PREPARED	CHECKED	APPROVED
R1	Nov 2018	For Client Review	[redacted]	[redacted]	[redacted]

- **P61** – 4no. sets of data, covering a period from 3/10/2018 through to 9/11/18. Gap from 12/10/18 through until 25/10/18 due to technical issues.

- **P91 –** 2no. sets of usable data, covering a period from 3/10/2018 through until 25/10/18. Data after this point was not usable due to adverse weather conditions.

3 Results

3.1 Pier 44

Results at Pier 44 are shown in Figure 1.

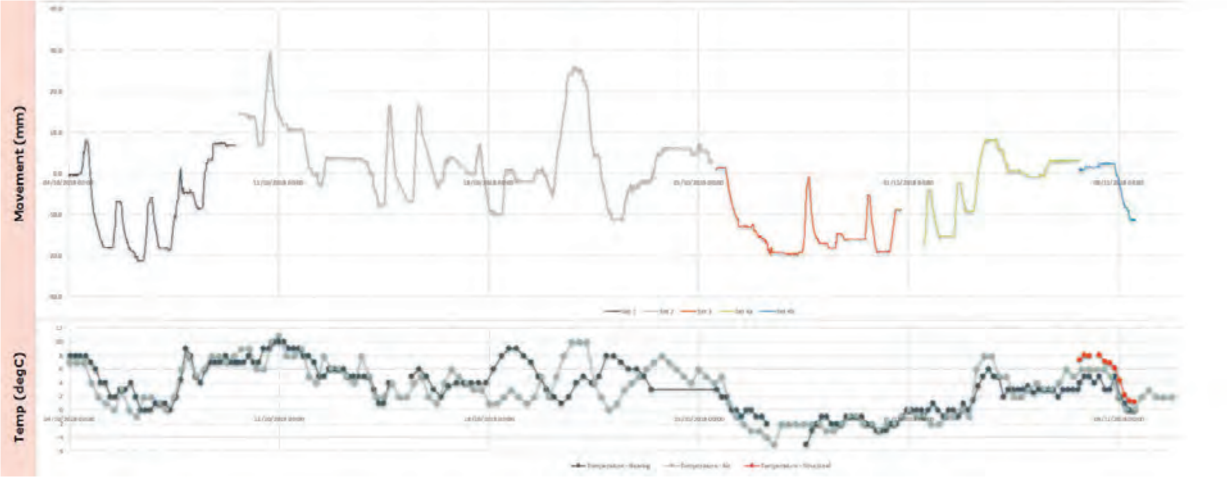


Figure 1 - P44

3.1.1 Movement Range

An approximate movement range of -20mm (contraction) and +25mm (expansion) was seen from the zero point (taken as the start of movement).

The corresponding temperature change was approximately 15 degrees C. This corresponds to a movement of 3mm per degree.

This pier is 260m from the point of fixity and would experience a theoretical movement of up to 3.1mm per degree. These results are considered similar and within the accuracy of this method.

3.1.2 Temperature Behaviour

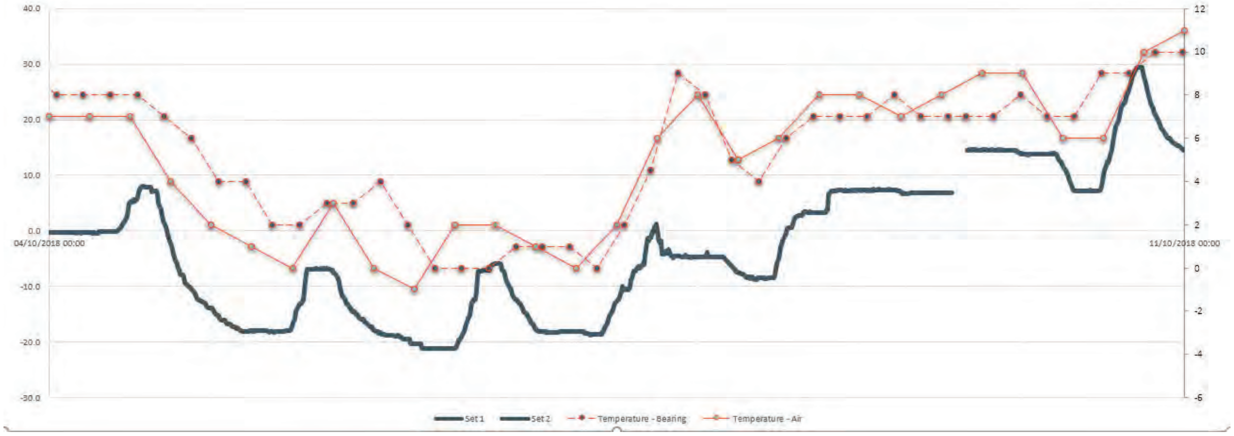


Figure 2 – Zoomed in temperature movement at P44. Temperature (RH Axis) and Movement (LH axis)

Figure 2 shows a section of the data available at Pier 44. Temperature (orange) is plotted against movement (blue).

In general, reasonable agreement is seen. Temperature profiles of Cairngorm, where nights can be hotter than days, means that it is difficult to make accurate conclusions. In general, the maximum expansion of the structure is seen at around 5pm. Maximum contraction seems to be more gradual and has generally occurred by around 10am. Owing to these profiles it is difficult to deduce a thermal lag.

Good agreement is seen between air temperature (Cairngorm Monitoring Station) and local structure temperature. Air temperature and structural temperature can therefore be considered analogous at the low temperatures seen.

3.1.3 Relative Bearing Position

Site measurements from ADAC structures suggest that the bearing had approximately 10mm of available movement range at 11degC structural temperature.

Based on the movement range seen, the bearing exceeded the available sliding surface by between 10-15mm at the extremity of its movement range, which occurred at -1degC. This agrees with results measured from the tracking video, and subsequent further site measurements by ADAC structures/COWI, although it is noted that there was no movement beyond this value despite lower temperatures.

These results would suggest that the bearing exceeds the available sliding surface at a temperature of approximately 5degC.

Figure 3 - Pier 44, extremity of movement

3.2 Pier 61

Results at Pier 61 were affected by a camera outage of approximate 13 days. However, there is still sufficient information available to allow conclusions to be drawn.

3.2.1 Movement Range

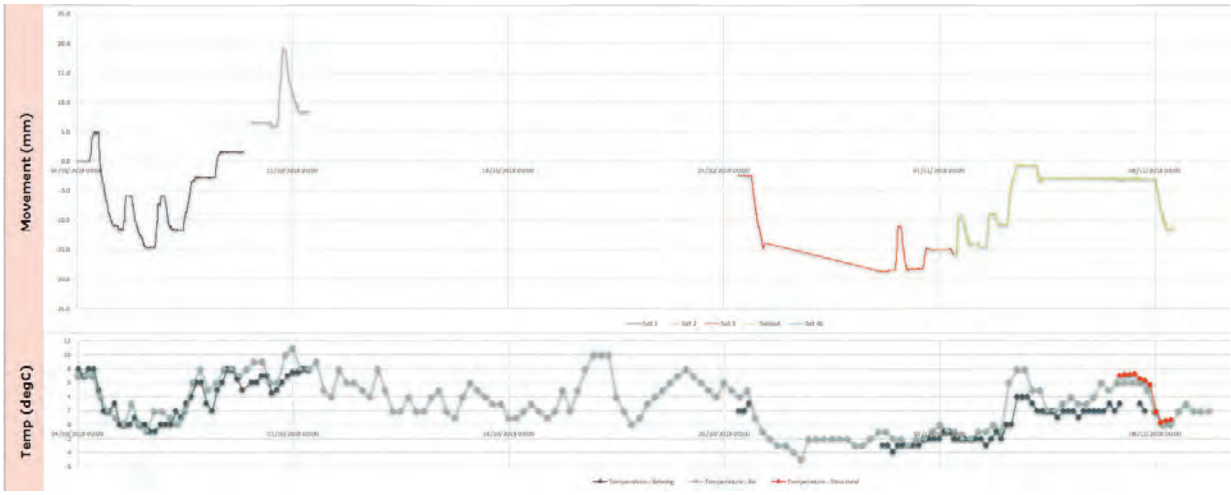


Figure 4 - Movement Range - P61

An approximate movement range of -20mm (contraction) and +20mm (expansion) was seen from the zero point (taken as the start of movement).

The corresponding temperature range was approximately 13 degrees C. This corresponds to a movement of 3.1mm per degree.

This pier is 214m from the point of fixity and would experience a theoretical movement of up to 2.6mm per degree. These results are considered similar and within the accuracy of this method.

3.2.2 Temperature Behaviour

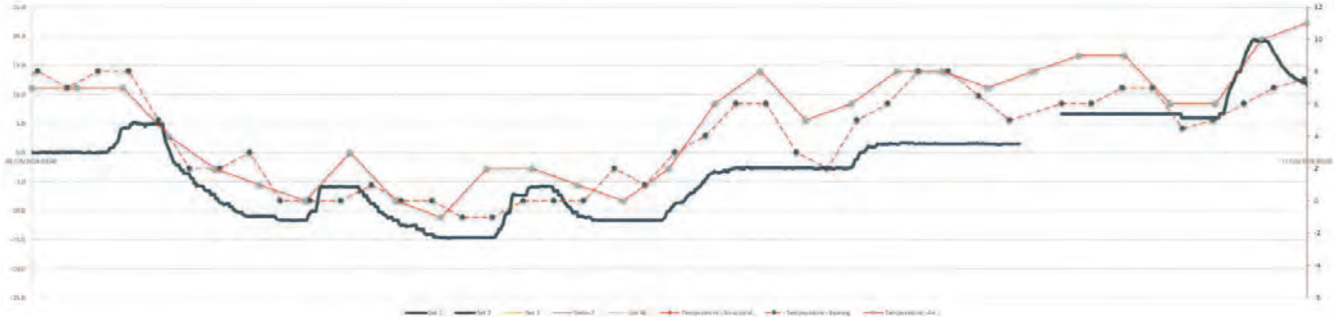


Figure 5 - Zoomed in on first week of P61 data. Temperature (RH Axis) and Movement (LH axis)

Figure 4 shows a section of the data available at Pier 61. Temperature (orange) is plotted against movement (blue).

In general, reasonable agreement is seen. Temperature profiles of Cairngorm, where nights can be hotter than days, means that it is difficult to make accurate conclusions. In general, the maximum expansion of the structure is seen at around 5pm, although it is noted that the maximum expansion often “plateaus” in the afternoon / evening. Maximum contraction seems to be more gradual and has generally occurred by around midday.

Owing to these profiles it is difficult to deduce a thermal lag. The structure would appear to behave in tandem with the temperature. Where it is present however, the lag appears to be around 4 hours.

Relatively good agreement is seen between air temperature (Cairngorm Monitoring Station) and local structure temperature.

3.2.3 Relative Bearing Position

Site measurements from ADAC structures suggest that the bearing had approximately 10mm of available movement range at 13degC structural temperature.

Based on the movement range seen, the bearing exceeded the available sliding surface by around 15mm at the extremity of its movement range, which occurred at -3degC. This broadly agrees with measurements taken manually from the camera and those measured on-site by ADAC Structures/COWI though are less than theoretical. This suggests that the bearing will exceed its allowable sliding surface at 5degC.

Figure 6 - Pier 61, extremity of movement

3.3 Pier 91

Results at Pier 91 were affected by storms which blocked the camera lens from the 25/10 onwards.

3.3.1 Movement Range

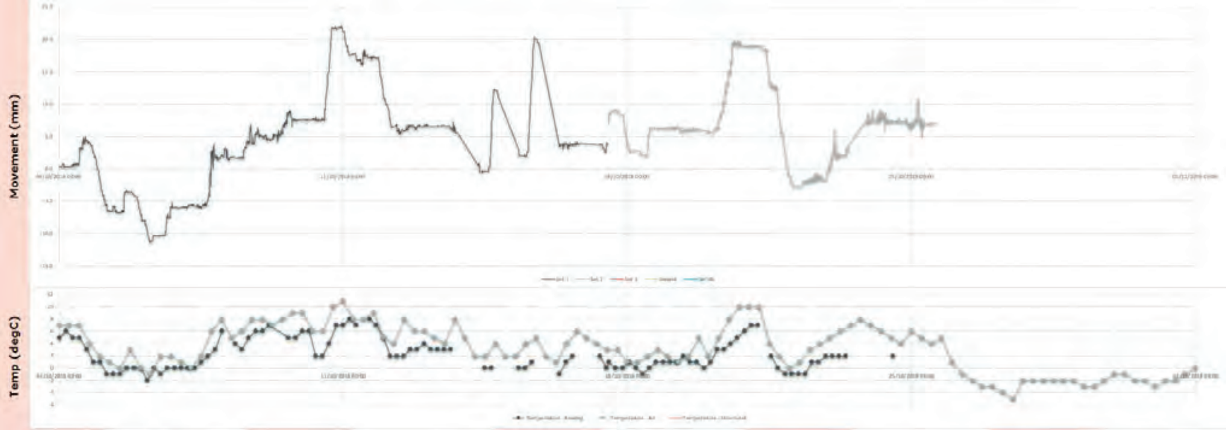


Figure 7 - Movement Range – P91

An approximate movement range of -12mm (contraction) and +22mm (expansion) was seen from the zero point (taken as the start of movement).

The corresponding air temperature range was approximately 12 degrees C. This corresponds to a movement of 2.8mm per degree.

This pier is 220m from the point of fixity, which would suggest a theoretical movement of up to 2.6mm per degree. These results are considered similar and within the accuracy of this method. The results agree with those at Piers 44 and 61.

3.3.2 Temperature Behaviour

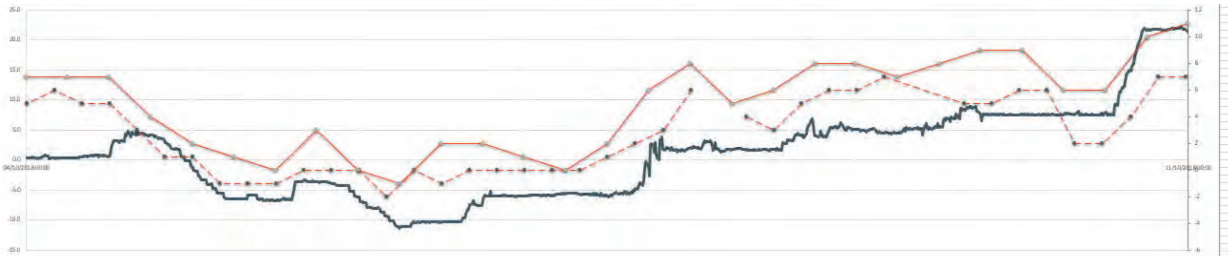


Figure 8 - Zoomed in on first week of P91 data. Temperature (RH Axis) and Movement (LH axis)

Figure 4 shows a section of the data available at Pier 91. Temperature (orange) is plotted against movement (blue).

The movement behaviour at Pier 91 broadly correlates with temperature, however the day-to-day temperature behaviour is erratic, with sudden rises/falls being apparent.

Owing to these profiles it is difficult to deduce a thermal. Where it is present however, the lag appears to be around 4 hours.

Air temperature would appear to be around 2degC greater than measured temperatures.

3.3.3 Relative Bearing Position

Measurements from ADAC structures suggest that the bearing was at the limits of its movement range at approximately 10degC air temperature.

Based on the movement range seen, the bearing exceeded the available sliding surface by around 12mm at the extremity of its movement range, which occurred at 0degC air temperature. This result agrees with measurements taken on site by ADAC structures/COWI though is less than theoretical. It is however noted that the limited monitoring period for this bearing did not include days where the temperature was colder (below 0degC). As such the bearing is likely to have exceeded its sliding surface by more than this value.

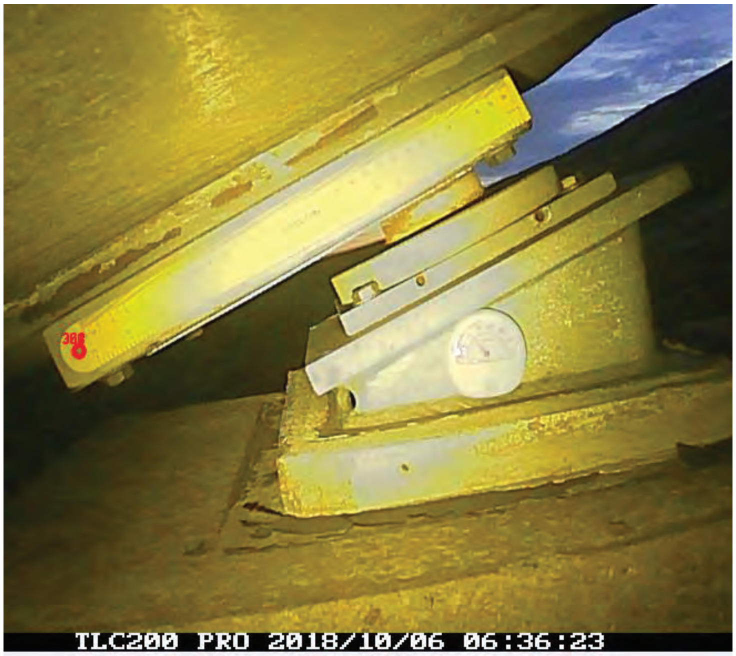


Figure 9 - Pier 91, extremity of movement

4 Conclusions

- The bearings at Cairngorm Funicular Railway are moving as anticipated and experiencing the expected movement range.
- This movement broadly tracks with temperature. The unusual weather patterns experienced at Cairngorm, where it is regularly hotter overnight than during the day mean it is difficult to draw conclusions about the day-to-day behaviour of the structure. This is likely limited to the "shoulder" seasons where temperature inversions are experienced within mountainous topography.
- The bearings monitored all theoretically exceeded their maximum sliding distance significantly during the monitoring period. This was verified by on-site measurements.
- Bearings exceed contact surface area at relatively high temperatures. Bearings will have some degree of loss of contact area at +5degrees.

5 Recommendations

- Continue monitoring to gain additional data at lower temperatures.
- Investigate reasons for bearing misalignment – See Appraisal Report.
- Apply temperature restriction to structure.
- Review lower limits and verify movements at lower temperatures with on-site measurements.

Appendix C Ground Investigation Report

CAIRNGORM MOUNTAIN LIMITED / HIGHLANDS AND ISLANDS ENTERPRISE

CAIRNGORM FUNICULAR MOUNTAIN RAILWAY - STRUCTURAL INSPECTION

GROUND INVESTIGATION REPORT

CONTENTS

1 Introduction

This technical note summarises the findings of a desk study and subsequent trial pitting investigation carried out to characterise the ground conditions along the route of the funicular mountain railway.

The technical note comprises the ground investigation report prepared in support of the ongoing appraisal being carried out by COWI of the viaduct structure.

A desk study review of the available archive data, published geological maps, memoirs and historical ground investigation data held on the British Geological Society (BGS) database has been carried out. The findings of the desk study review are described in Section 2.

PROJECT NO.	DOCUMENT NO.
A116993	TN-03-002

VERSION	DATE OF ISSUE	DESCRIPTION	PREPARED	CHECKED	APPROVED
01	04-11-18	For Client Review	[redacted]	[redacted]	[redacted]

Between 10th and 19th October 2018 a site investigation comprising excavation of 12No trial pits was carried out to establish the ground conditions as a selected number of foundations supporting the viaduct piers. The scope of this trail pit investigation is described in TN-03-001.

The findings of the 2018 trial pit investigation are described in Section 3.

Pictorial trial pit logs and photographs are reproduced in Appendix A.

Soil samples recovered during the 2018 investigation were scheduled for laboratory testing. Testing has provided geotechnical data on soil classification and strength.

The test results are summarised in Section 3 and reproduced in Appendix B.

2 Background Information

2.1 General description

The funicular mountain railway viaduct is approximately 1700m in length. It rises 440m in elevation from the Cairngorm Ski Center base station at approximately 630mOD up to the tunnel portal on the approach to the summit station at 1070mOD.

The viaduct structure can be separated into 6 structural units. Each unit comprising a downslope thrust block and variable number of concrete piers supporting precast concrete cross heads (transverse beams). Precast concrete longitudinal beams made continuous through provision of a cast in-situ concrete stitch support the running rails. These longitudinal beams are supported on the cross head beams by free and guided pot bearings.

The piers are founded on shallow gravity base foundations of various size. Seven foundation base types are indicated on the A.F. Cruden Associates drawings as being present.

The base types range in size from 4m by 2m (type 1) to 8m by 2m (type 7). The drawings indicate that the foundation bases are 1.25m deep with the top surface of the foundation buried by 500mm (minimum) cover, i.e. on the downslope side of the pier.

2.2 Desk study

2.2.1 Archive data

Archive information comprising extracts from the project Health and Safety file indicate the gravity base foundations to be founded on either weathered granite rock or a matrix of granular deposits comprising sand and gravel deposits.

2.2.2 Published geological data

Geological data published by the BGS indicate the underlying geology at the site is Granite Biotite of Silurian age.

Drift (near surface) sediment mapping indicates the route of the railway passes through various sediment types of Quaternary age comprising, glacial, alluvial and blanket head deposits as illustrated in Figure 2-1 below.

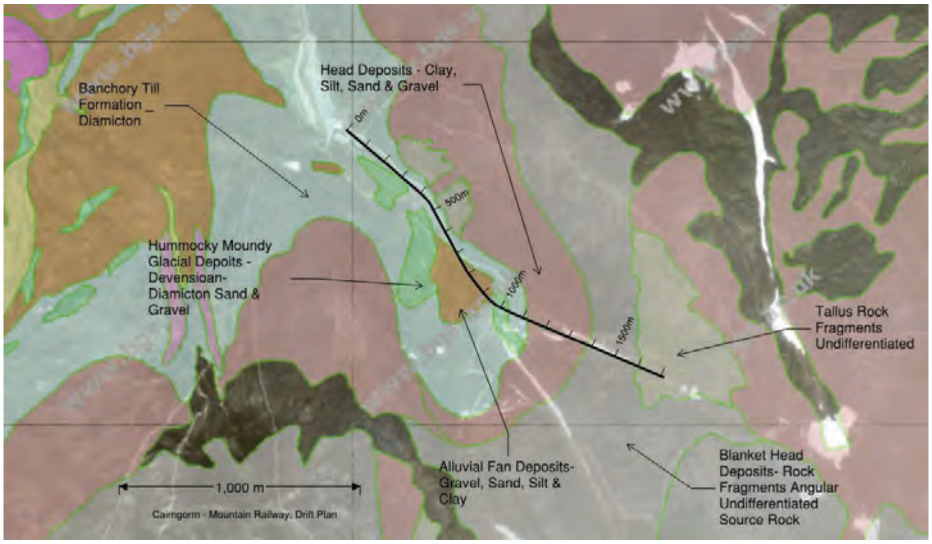


Figure 2-1 Extract BGS Drift Map showing variation in near surface sediment type along the route of the funicular mountain railway

Three distinct areas/types of surface geology are indicated to be present.

- Area 1A, located between CH0+000 and CH0+600m is characterised by glacial sands, gravels and boulders. This area is located at the toe of south west facing tallus slopes. The down slope ground profile in plane with the structure varies between 4˚to 8˚ in this area.

- Area 1B, located between CH0+950 and CH1+100m is characterised by glacial sands, gravels and boulders. This area is located above the Sheiling crossing loop. The down slope ground profile in plane with the structure varies between 15˚to 18˚ in this area.

- Area 2, located between CH0+600 and CH0+950m is characterised by alluvial deposits of sand, silt and clay. In this area the surface sediments are likely to be thickest containing a higher percentage of silt and clay sized particles together with peat and organic deposits. Vegetation was observed to be better established in this area compared with areas 1 and 3 indicative of the soils ability to retain water to a greater extent. The down slope ground profile in plane with the structure varies between 8˚to 15˚ in this area.

  A natural spring was observed to discharge at surface above pier No.41 at CH 0+785m.

- Area 3, located between CH1+100 and Ch1+700m is characterised by head deposits. This area corresponds with the steepest down slope ground profile ranging between 15˚ to 22˚. In this area it is anticipated that surface sediments will be thinnest. A natural spring was observed to discharge at surface above pier No.72 at CH 1+310m.

2.2.3 Historical ground investigation data

Two phases of historical ground investigation are known to have been carried out by Grampian Soil Survey Ltd of Aberdeen on behalf of Cairngorm Chairlift Company Ltd (1994) and HTS Associates on behalf of Highlands Council (1999) prior to construction of the funicular mountain railway.

Phase 1 1994 Investigation

The first phase of investigation carried out in 1994 comprised excavation of three trial pits located upslope of the Sheiling in proximity to Piers 90, 93 and the tunnel portal

The 1994 historical ground investigation data indicates the presence of peaty topsoil overlying quaternary deposits of silty gravelly sands and gravels of glacial origin overlying weathered granite. At only one location (TP1 in proximity to tunnel portal) was bedrock comprising weak, highly to moderately weathered granite definitively encountered at a depth of 2.2m below existing ground level.

Elsewhere the trial pits were terminated in dense sands and gravel with many cobbles and boulders of broken rock at depths below 1.7m (TP2) and 3.4m (TP3) respectively. This material may is described as very dense and is interpreted to be representative of the weathered rock head profile.

The data indicates the likelihood that the pad foundations constructed on the upper slopes in proximity to the trial pits are founded either directly on weathered bedrock or on a thin layer of granular material comprising silty sand and gravel.

The sands and gravels are described as medium dense with angular sand particles and sub rounded to sub angular gravel particles indicative of the soil possessing an internal friction angle in the range 34-36 degrees.

Groundwater seepages were encountered as the contact between the topsoil layer and Quaternary deposits and within the quaternary deposits at depths varying between 1.5m to 3m indicative of the founding soils being partially saturated.

1998-99 Investigation

The second phase of investigation carried out in 1998-99 proposed 6No cable percussion boreholes and 8No trial pits. Due to the winter timing of the investigation fieldwork, boreholes were located near the Cairngorm Ski Centre base station (Day Lodge) and Sheiling as indicated in Figure 2-2 below.

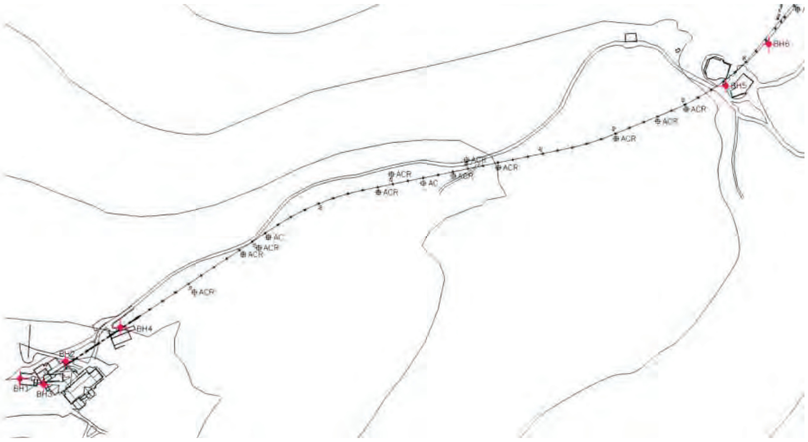


Figure 2-2 1998-99 Ground investigation borehole location plan.

Boreholes were advanced using shell and auger boring equipment and 200mm diameter casing. The boreholes recovered between 4m and 5m of Quaternary granular deposits of glacial origin before terminating on weathered bedrock.

Boreholes BH4 & BH5 are understood to have been abandoned. Boreholes BH7 and BH8 were drilled in July 1999 as possible replacements to BH4 & BH5. The exact location of boreholes BH7 and BH8 is unknown.

The exact location of the 8No trial pits excavated during the 1998-99 investigation is unknown. However the description of the location given on the trial pit logs suggest they may have been excavated alongside the access track.

The 1998-99 investigation data indicate the site in proximity the base station (BH1, BH2, BH3) the thickness of the quaternary deposits may exceed 5m.

Only in BH 2 was weathered rock definitively encountered at a depth of 5m. Elsewhere the boreholes were advanced through sand and gravels containing many cobbles and boulders of broken rock on occasion weathered to residual soil. These quaternary deposits are described as dense to very dense based on Standard Penetration test data, although this data is likely to be influenced by the presence of the cobble and boulder obstructions.

The data indicates the likelihood that the pad foundations constructed on the lower slopes in proximity to the base station are founded on a 1-2m thick layer of granular material comprising silty sand and gravel overlying a 1m thick layer of disintegrated rock bound in a sand and gravel matrix.

The sands and gravels are described as dense with angular sand particles and sub angular to angular gravel particles which is indicative of the soil possessing an internal friction angle in the range 36-38 degrees.

In proximity to the Sheiling, the data indicates the pad foundation are likely to be constructed on weathered rock. In BH6 weathered rock was encountered at a depth of 1m.

The location of BH7 and BH8 are unknown. The logs indicate the presence of soft and loose mix of peat, silt and sand with numerous cobbles and boulders to depth of 1.5-2m underlain by Quaternary deposits of coarse sand and gravel of glacial origin with many boulders of broken rock weathered to a residual soil.

Rock head was proven in BH7 at a depth of 4.3m.

3 2018 Ground Investigation

3.1 Scope of works

12No trial pits were excavated at the site by McGowan's Civil Engineering Ltd between 10th and 19th October 2018.

The trial pit investigation was supervised by representatives from Cairngorm Mountain Ltd and observed by geotechnical specialists from COWI and structural specialist from ADAC Structures.

The trial pits were excavated in two phases using different excavators to manage the variable terrain encountered downslope and upslope of the Shieling station.

The first phase of the trial pit investigation was carried out downslope of the Shieling station at CH 0+900m.

- 6No pits were excavated between 10th and 11th October at pier locations 42,41,40 and 24,23,22 with a light weight (5 ton) Volvo ECR58D tracked excavator.

The second phase of the trial pit investigation was carried out upslope of the Shieling station.

- 6No pits were excavated between 17th and 19th October at pier locations 91,72,70 and 57,56,55 with a (10 ton) Menzie Muck A91 wheeled excavator.
- Trial pit 72 is located in proximity to a groundwater spring discharging from the hillside approximately 10m upslope of the pier location. The backfill material surrounding the foundation base was saturated and despite efforts to control water flow the excavation was abandoned due to collapse of unstable side walls without exposing the underside of the foundation base.

All trial pits were excavated alongside the pier foundations with the excavator orientated perpendicular to the longitudinal axis of the viaduct structure.

This orientation was selected to expose the full breadth of the pad foundation without disturbing backfill material placed against the upslope and downslope face of the foundation which provides some passive resistance to lateral loadings.

The location of all 12No trial pit are shown in Figure 3-1 below.

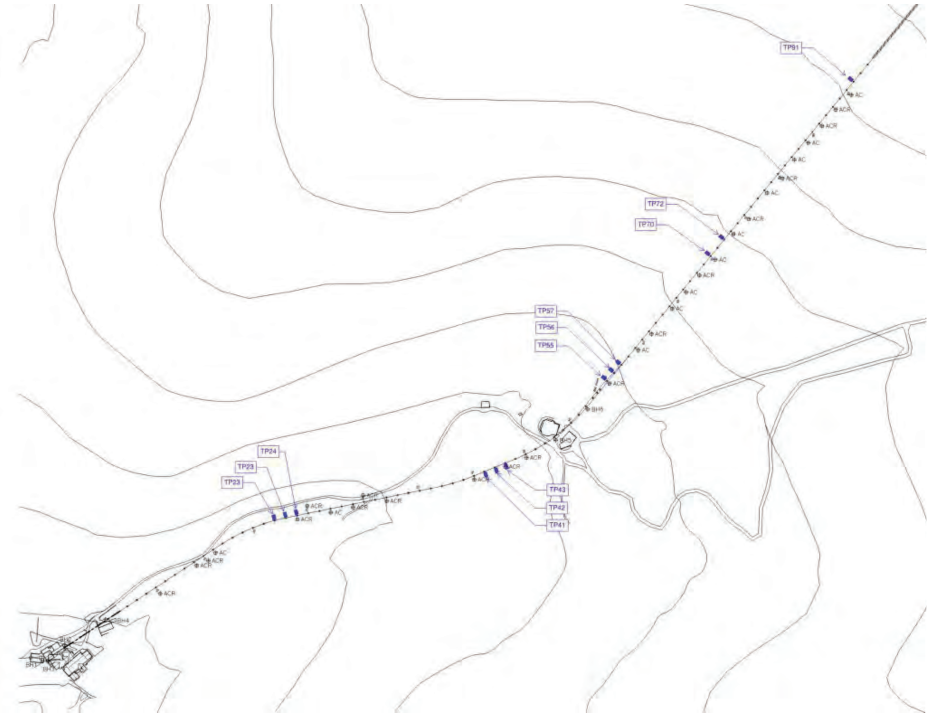


Figure 3-1 2018 Ground investigation trial pit location plan

3.2 Sampling and laboratory testing

Bulk and disturbed samples recovered from selected trial pits were scheduled for laboratory testing.

Soil classification testing in accordance with BS1377-2:1990 and total stress strength testing in accordance with BS1377-7:1990 was carried out by MAT test Ltd in Glasgow.

Details of the laboratory test results provided by MAT test Ltd are given in Appendix B.

3.3 Investigation findings

The trial pit investigation established the foundation geometry and characteristics of the foundation subgrade soils at 5 locations along the route of the funicular railway.

The 5 locations investigated reflect the three areas/types of surface geology identified during the desk study investigation.

The reference datum adopted for measurement of foundation depth, bearing strata and sample recording in all cases is the top surface of the foundation pad.

Description of the foundation geometry, backfill and foundation subgrade at each of the five investigation locations are given below.

Pictorial logs and photographs for each of the 12No trial pits excavated along the route of the viaduct are reproduced in Appendix A.

3.3.1 Area 1A (Piers 22, 23 & 24)

The depth to the top surface of the foundation below present ground level varied between 0.45m and 0.75m. Generally, the foundation overburden consisted of a thin layer of organic peat, silt and clay topsoil 0.3m in thickness overlying sandy silty topsoil containing many granite boulders.

All three of the foundations exposed were 2.1m in breadth. The offset measured between the foundation edge and leading edge of the pier varied between 0.85m and 1.0m.

The thickness depth of the foundation pads varied between 1.5m to 1.65m. Typically, a cold joint was exposed at 1.25m depth which was interpreted to mark the interface between blinding concrete and the reinforced section of the foundation.

Foundation backfill comprised sandy fine to coarse gravel with many cobbles and boulders of granite, some decomposed to a residual soil. The deposit was assessed as loose to medium dense with an internal friction angle between 32 and 34 degrees.

The foundation subgrade comprised sand of glacial origin with variable silt and gravel content. The deposit was assessed as damp and medium dense with an internal friction angle between 34 and 36 degrees.

Groundwater flow was encountered in TP23 and TP24 entering from the side wall of the excavation at depth varying between 0.4m and 1.0m below datum indicative of the foundation subgrade being saturated.

The results of the liquid limit and particle size distribution tests performed on bulk samples confirmed the foundation stratum to comprise a non-plastic silty sand with trace clay (<10%) and variable gravel content (10-30%).

The results of direct shear box testing confirmed that the material when subjected to light compaction can possess an internal friction angle of 36 degrees.

3.3.2 Area 2 (Piers 40, 41 & 42)

The depth to the top surface of the foundation below present ground level varied between 0.3m and 0.6m. Generally, the foundation overburden consisted of a thin layer of organic peat, silt and clay topsoil 0.5m in thickness overlying sandy silty topsoil containing many granite boulders.

All three of the foundations exposed were 2.1m in breadth. The offset measured between the foundation edge and leading edge of the pier varied between 1.75m and 1.95m.

The thickness depth of the foundation pads is 1.3m. Typically a cold joint was exposed at 1.25m depth which was interpreted to mark the interface between blinding concrete and the reinforced section of the foundation.

Foundation backfill comprised sandy fine to coarse gravel with many cobbles and boulders of granite, some decomposed to a residual soil. The deposit was assessed as loose to medium dense with an internal friction angle between 32 and 34 degrees.

A 0.2m thick layer of made ground was present beneath the concrete blinding layer.

The foundation subgrade comprised interlayered alluvial sediments of sandy silt and silty sand with variable gravel content. The deposit was assessed as damp, loose in density and firm in terms of shear strength with an internal friction angle between 30 degrees and 32 degrees and effective cohesion of 5kPa.

Groundwater flow was encountered in all three trial pits entering from the side wall of the excavation at depth varying between 1.25m and 1.7m below datum indicative of the foundation subgrade being saturated.

The results of the particle size distribution tests performed on bulk samples confirmed the foundation stratum to comprise silty sand with trace clay (<10%) with variable gravel content (10-40%).

The results of liquid limit tests carried out on disturbed silt samples confirmed the silt horizons are of intermediate plasticity.

The results of direct shear box testing confirmed that the silty sand material when subjected to light compaction can possess an internal friction angle of 36-38 degrees while the sandy silt and silt materials have an internal friction angle of 32 degrees.

3.3.3 Area 1B (Piers 55, 56 & 57)

The depth to the top surface of the foundation below present ground level varied between 0.3m and 0.4m. Generally, the foundation overburden consisted of a thin layer of fibrous peat, and soft organic silt and clay topsoil 0.2-0.25m in thickness overlying sandy silty loamy topsoil containing many granite boulders.

All three foundation pads exposed were 2.1m in breadth. The offset measured between the foundation edge and leading edge of the pier varied between 1.2m and 2.15m.

The thickness depth of the foundation pad foundations varied between 1.3m and 1.4m. Typically a cold joint was exposed between 1.05m and 1.25m depth which was interpreted to mark the interface between blinding concrete and the reinforced section of the foundation.

Foundation backfill comprised sandy fine to coarse gravel with many cobbles and boulders of granite. The deposit was assessed as loose to medium dense with an internal friction angle between 32 and 34 degrees.

The foundation subgrade comprised slightly clayey fine to coarse sand and fine to coarse angular to sub-angular gravel. The deposit was assessed as medium dense with an internal friction angle between 34 and 36 degrees.

The foundation backfill and subgrade were either dry or damp indicative of the subgrade soils being subject to variable levels of saturation.

The results of the particle size distribution tests performed on bulk samples confirmed the foundation stratum to comprise fine to coarse sand and gravels with trace silt (<10%).

3.3.4 Area 3 (Pier 70)

The depth to the top surface of the foundation below present ground level varied between 0.4m and 0.7m. Generally, the foundation overburden consisted of a thin layer of fibrous peat, and soft organic sillty topsoil up to 0.2m in thickness overlying organic loamy silty fine to coarse SAND and fine to medium gravel with occasional cobbles and boulders of granite.

The foundations pad measured 2.1m in breadth. The offset between the foundation edge and leading edge of the pier measured 1.5m.

The thickness depth of the foundation pad measured 1.4m. A cold joint was exposed at a 1.2m depth which was interpreted to mark the interface between blinding concrete and the reinforced section of the foundation.

Foundation backfill comprised sandy fine to coarse gravel with many cobbles and boulders of granite and traces of silty organic sand. The deposit was assessed as loose to medium dense with an internal friction angle between 32 and 34 degrees.

A 0.45m thick layer of made ground was present beneath the concrete blinding layer.

The foundation subgrade comprised slightly silty fine to coarse sand and fine to coarse angular to sub-angular gravel characteristic of head deposits. The deposit was assessed as medium dense to dense with an internal friction angle between 34 and 38 degrees.

The foundation subgrade was damp.

The results of the particle size distribution tests performed on bulk samples confirmed the foundation stratum to comprise fine to coarse sand and gravels with trace silt (<10%).

3.3.5 Area 3 (Pier 91)

The depth to the top surface of the foundation below present ground level varied between 0.6m and 0.8m. Generally, the foundation overburden consisted of a layer of fibrous peat, and soft organic silty topsoil 0.6m in thickness overlying organic loamy silty fine to coarse SAND and fine to medium gravel with occasional cobbles and boulders of granite.

The foundations pad measured 2.1m in breadth. The offset between the foundation edge and leading edge of the pier measured 1.8m.

The thickness depth of the foundation pad measured 1.45m. A cold joint was exposed at a 1.25m depth which was interpreted to mark the interface between blinding concrete and the reinforced section of the foundation.

Foundation backfill comprised sandy fine to coarse gravel with many cobbles and boulders of granite and traces of silty organic sand. The deposit was assessed as loose to medium dense with an internal friction angle between 32 and 34 degrees.

The foundation subgrade comprised highly weathered granite bedrock recovered as broken cobbles and boulders bound in a coarse sand matrix. The deposit was assessed as to dense to very dense with an internal friction angle between 38 and 42 degrees.

The foundation subgrade was dry.

The results of the particle size distribution tests performed on bulk samples confirmed the foundation stratum to comprise fine to coarse sand and gravels with trace silt (<10%).

4 Conclusions

The 2018 ground investigation data supports the findings of the desk study investigation.

The composition and strength of the foundation subgrade varies across the site.

Weathered rock head was only encountered in TP91 at pier 91.

Drained shear strength parameters of 42 degrees (Friction) and 5kpa (Cohesion) is assigned to the weathered rock.

Elsewhere the foundation subgrade comprised medium dense to dense sands with varying silt and gravel content of glacial origin (Area 1A/B and 3) and interlayered sediments of silt and sand with variable gravel content of alluvial origin (Area 2).

Bulk unit weight of the foundation subgrade is assessed as 18-19kN/m³ for glacial sands/gravels and 17kN/m³ for the interlayered alluvial sediments.

Based on the laboratory test data drained shear strength parameters are assessed as 34-38 degrees for sands/gravels and 32 degrees and 5kPa (cohesion) for the interlayered alluvial sediments.

The thickness of the foundation subgrade overlying weathered rock remains unknown over the length of the viaduct structure but is estimated to vary between 1-3m in Area 1A/B, 5-10m in Area 2 and 1-3m in Area 3.

As-built foundation geometry was observed to broadly comply with the data presented on the A.F Cruden design drawings. The thickness depth of the foundation pad measured between 1.3m and 1.65m. Typically, a cold joint was exposed at a 1.25m depth which was interpreted to mark the interface between blinding concrete and the reinforced section of the foundation.

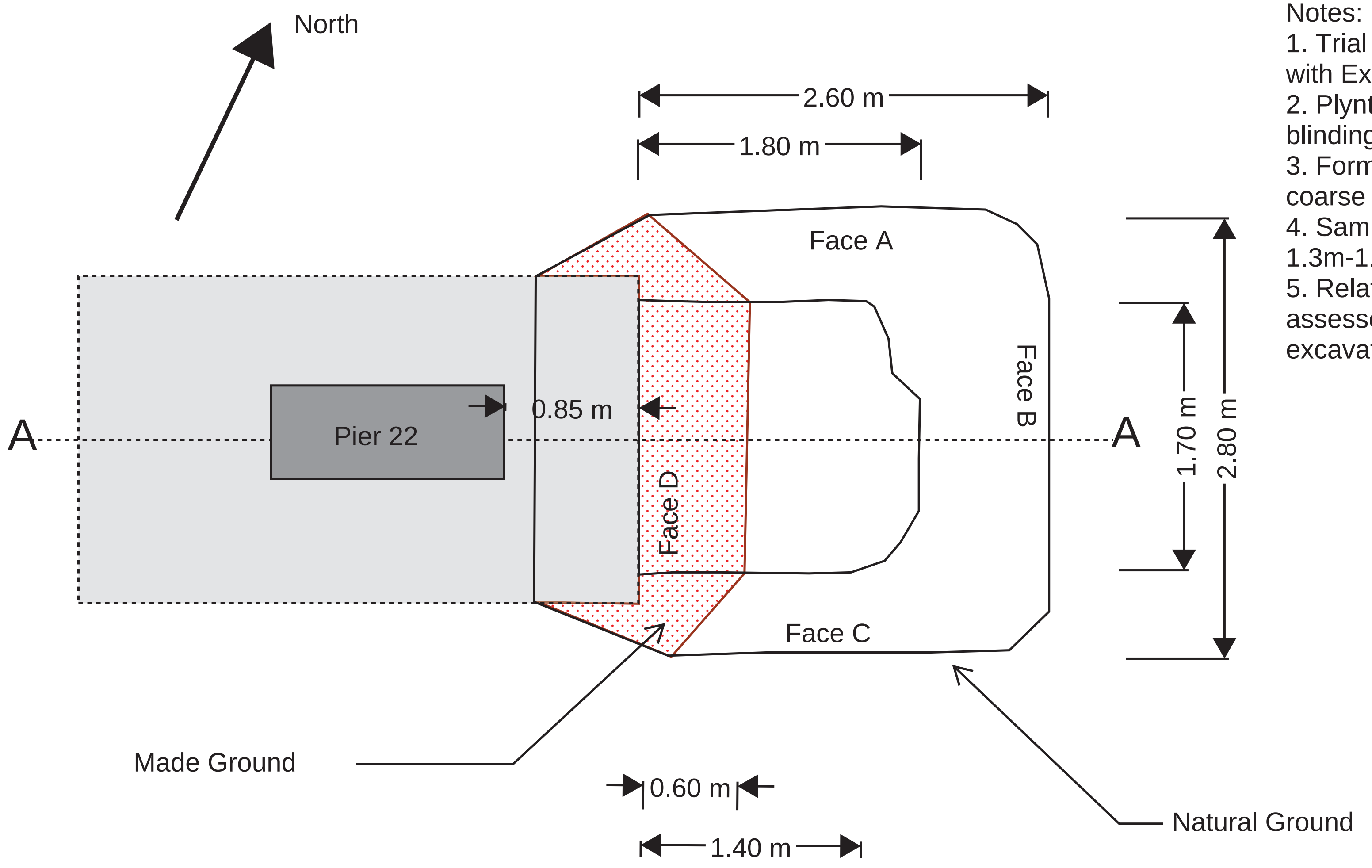
Foundation backfill comprised sandy fine to coarse gravel with many cobbles and boulders of granite with trace organic silt and clay. The deposit was assessed as loose to medium dense with bulk unit weight assessed at 17-19kN/m³ and friction angle between 32 and 34 degrees.

Foundation subgrade soils were typically observed to be damp or saturated indicative of groundwater being at or above formation level.

CAIRNGORM FUNICULAR MOUNTAIN RAILWAY - STRUCTURAL INSPECTION

GROUND INVESTIGATION REPORT

Appendix A - Trial Pit Logs



Notes:
1. Trial Pit Excavated on 11-10-18 with Excavator Volvo ECR58D
2. Plynth Thickness 1.25m with 0.25m blinding concrete
3. Formation slightly silty medium to coarse Sand, assessed as medium dense.
4. Sampling: D1 - 1.3m-1.4m, B1 - 1.3m-1.4m and B2 1.6m-1.7m.
5. Relative density of granular deposits assessed on basis of difficulty of excavation.

COWI

Sketch Title:

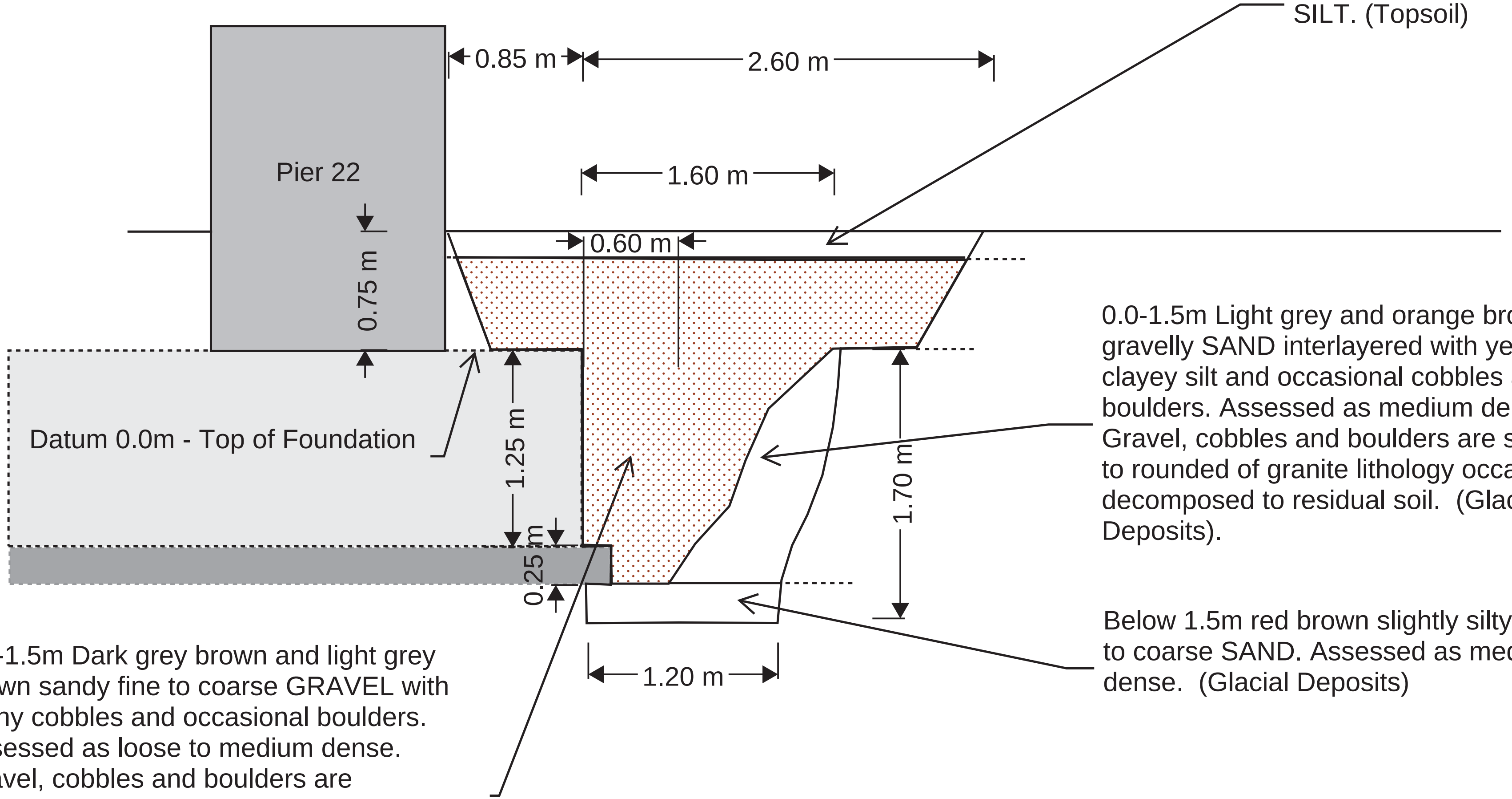
Cairngorm Funicular Mountain Railway - Trial Pit Investigation - TP22 Layout

[redacted]

Sketch No: TN-03-002 -001

Rev: 01

Date: 18-10-18



Fibrous peat and soft black organic SILT. (Topsoil)

0.0-1.5m Light grey and orange brown gravelly SAND interlayered with yellow brown clayey silt and occasional cobbles and boulders. Assessed as medium dense. Gravel, cobbles and boulders are sub-angular to rounded of granite lithology occasionally decomposed to residual soil. (Glacial Deposits).

Below 1.5m red brown slightly silty medium to coarse SAND. Assessed as medium dense. (Glacial Deposits)

0.0-1.5m Dark grey brown and light grey brown sandy fine to coarse GRAVEL with many cobbles and occasional boulders. Assessed as loose to medium dense. Gravel, cobbles and boulders are sub-angular angular to sub-rounded of granite lithology. (Made Ground)

Excavation Dry

COWI

Sketch Title: Cairngorm Funicular Mountain Railway - Trial Pit Investigation - TP22 Section A-A

[redacted]

Sketch No: TN-03-002 -001

Rev: 01

Date: 18-10-18

Face A

Face B

COWI

Sketch Title: Cairngorm Funicular Mountain Railway -
Trial Pit Investigation - TP22 Photographs

[redacted]

Sketch No: TN-03-002 -001

Rev: 01

Date: 18-10-18

Face C

Face D

COWI

Sketch Title: Cairngorm Funicular Mountain Railway -
Trial Pit Investigation - TP22 Photographs

[redacted]

Sketch No: TN-03-002 -001
Rev: 01
Date: 18-10-18

0.0-1.5m Light grey and orange brown gravelly SAND interlayered with yellow brown clayey silt and occasional cobbles and boulders. Assessed as medium dense. Gravel, cobbles and boulders are sub-angular to rounded of granite lithology occasionally decomposed to residual soil. (Glacial Deposits).

Below 1.5m red brown slightly silty medium to coarse SAND. Assessed as medium dense. (Glacial Deposits)

Sample D1 & B1 -1.3m-1.4m

COWI

Sketch Title:

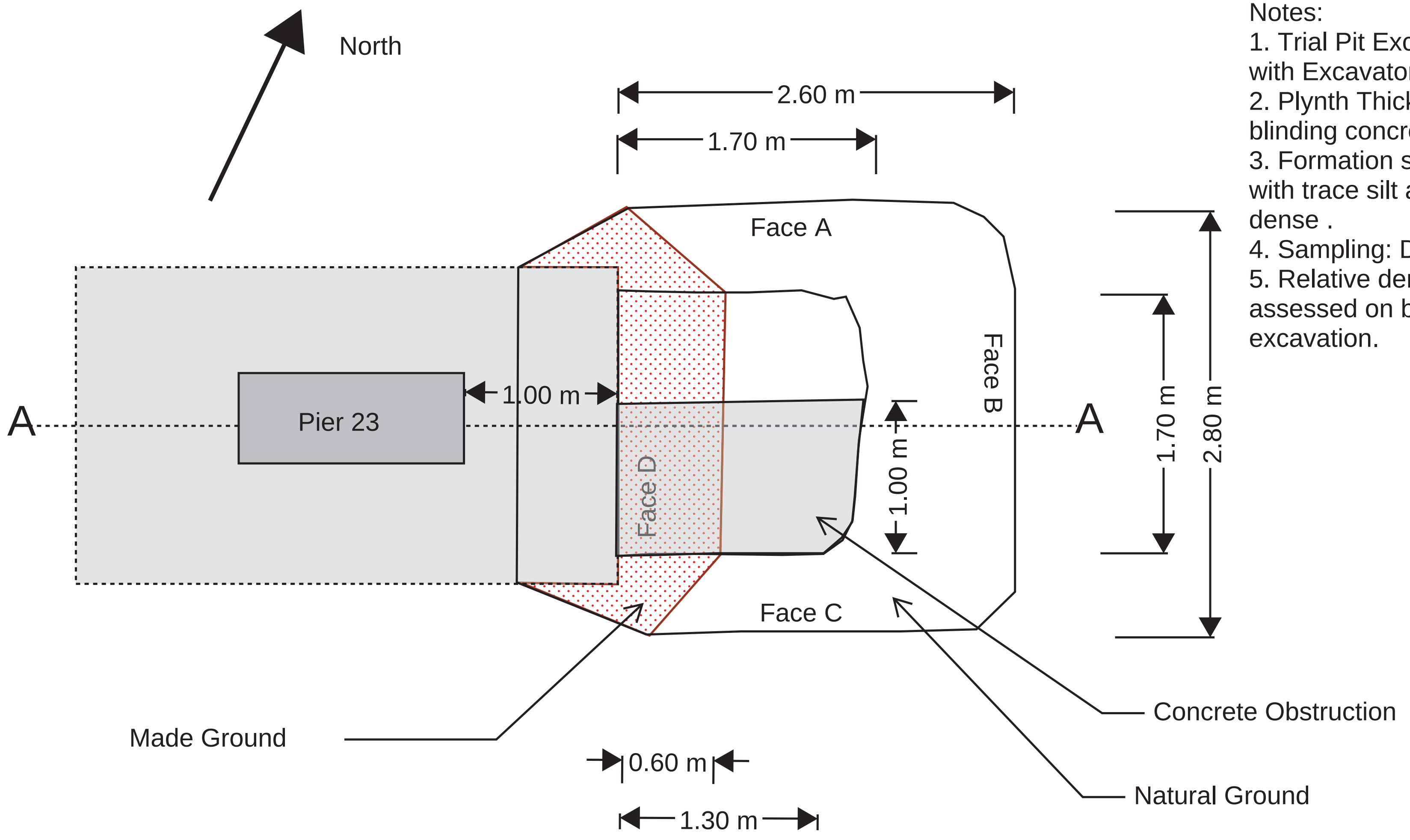
Cairngorm Funicular Mountain Railway -
Trial Pit Investigation - TP22 Photographs

[redacted]

Sketch No: TN-03-002 -001

Rev: 01

Date: 18-10-18



Notes:
1. Trial Pit Excavated on 11-10-18 with Excavator Volvo ECR58D
2. Plynth Thickness 1.25m with 0.25m blinding concrete
3. Formation slightly gravelly Sand , with trace silt assessed as medium dense .
4. Sampling: D1 - 1.5m-1.6m
5. Relative density of granular deposits assessed on basis of difficulty of excavation.

COWI

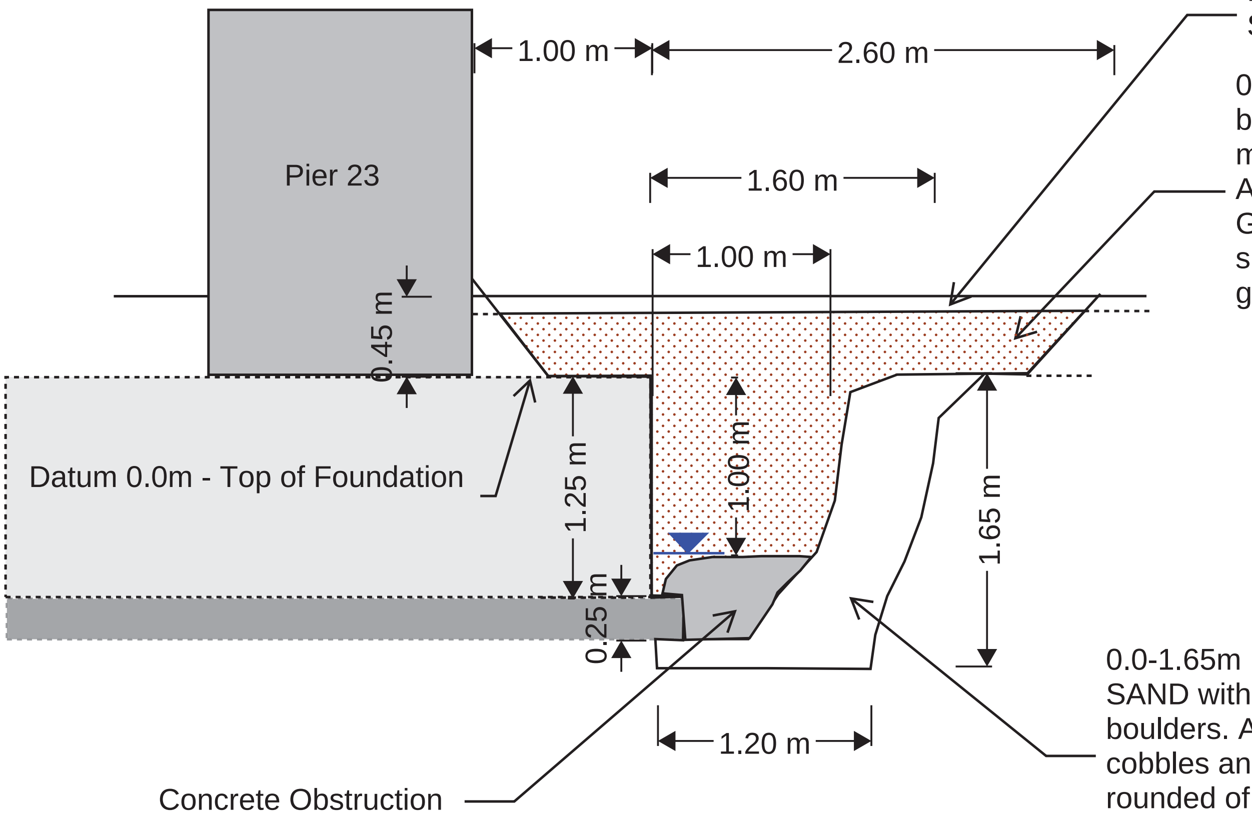
Sketch Title: Cairngorm Funicular Mountain Railway - Trial Pit Investigation - TP23 Layout

[redacted]

Sketch No: TN-03-002 -002

Rev: 01

Date: 18-10-18



Fibrous peat and soft black organic SILT. (Topsoil)

0.0-1.5m Dark grey brown and light grey brown sandy fine to coarse GRAVEL with many cobbles and occasional boulders. Assessed as loose to medium dense. Gravel, cobbles and boulders are sub-angular angular to sub-rounded of granite lithology. (Made Ground)

0.0-1.65m Light grey and orange brown gravelly SAND with trace silt and occasional cobbles and boulders. Assessed as medium dense. Gravel, cobbles and boulders are sub-angular to sub rounded of granite lithology occasionally decomposed to residual soil. (Glacial Deposits).

Seepage- low <1l/min @1m at interface between made and natural ground

COWI

Sketch Title: Cairngorm Funicular Mountain Railway - Trial Pit Investigation - TP23 Section A-A

Sketch No: TN-03-002 -002

Rev: 01

Date: 18-10-18

Face A

Face B

COWI

Sketch Title: Cairngorm Funicular Mountain Railway - Trial Pit Investigation - TP23 Photographs

[redacted]

Sketch No: TN-03-002 -002

Rev: 01

Date: 18-10-18

Face C

Face D

Sketch Title: Cairngorm Funicular Mountain Railway -
Trial Pit Investigation - TP23 Photographs

[redacted]

Sketch No: TN-03-002 -002

Rev: 01

Date: 18-10-18

Formation - gravelly
SAND with trace silt

0.0-1.65m Light grey and orange/red brown gravelly SAND with trace silt and occasional cobbles and boulders. Assessed as medium dense. Gravel, cobbles and boulders are sub-angular to rounded of granite lithology occasionally decomposed to residual soil. (Glacial Deposits).

COWI

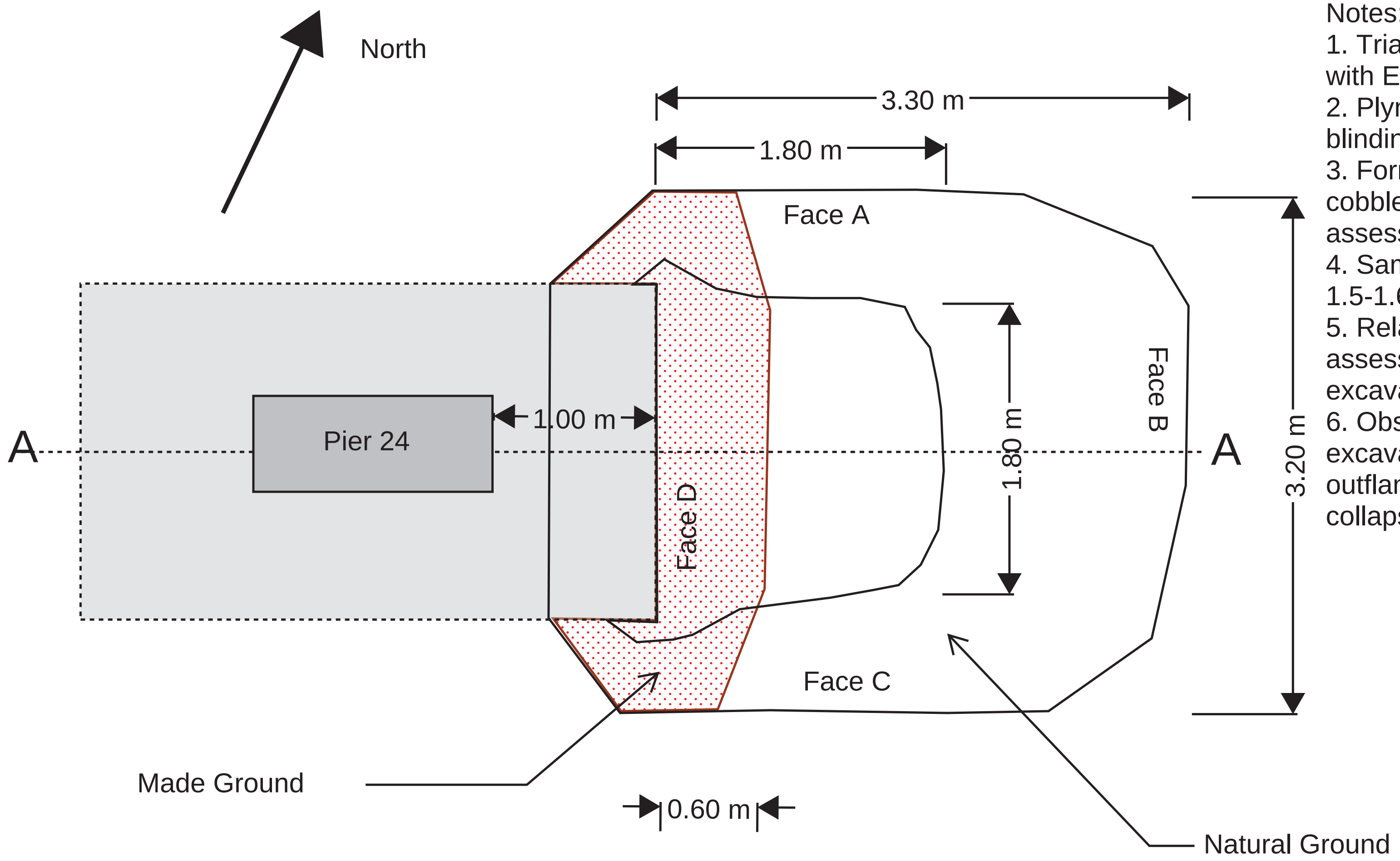
Sketch Title: Cairngorm Funicular Mountain Railway -
Trial Pit Investigation - TP23 Photographs

[redacted]

Sketch No: TN-03-002 -002

Rev: 01

Date: 18-10-18



Notes:
1. Trial Pit Excavated on 11-10-18 with Excavator Volvo ECR58D
2. Plynth Thickness 1.3m with 0.3m blinding concrete
3. Formation slightly gravelly sand with cobbles interlayered with sandy silt, assessed as medium dense .
4. Sampling: D1 - 0.9m-1.0m & D2 1.5-1.6m
5. Relative density of granular deposits assessed on basis of difficulty of excavation.
6. Obstruction encountered at 1.6m, excavation aborted to prevent further outflanking of the foundation caused by collapse of the excavtion sidewalls.

COWI

Sketch Title:

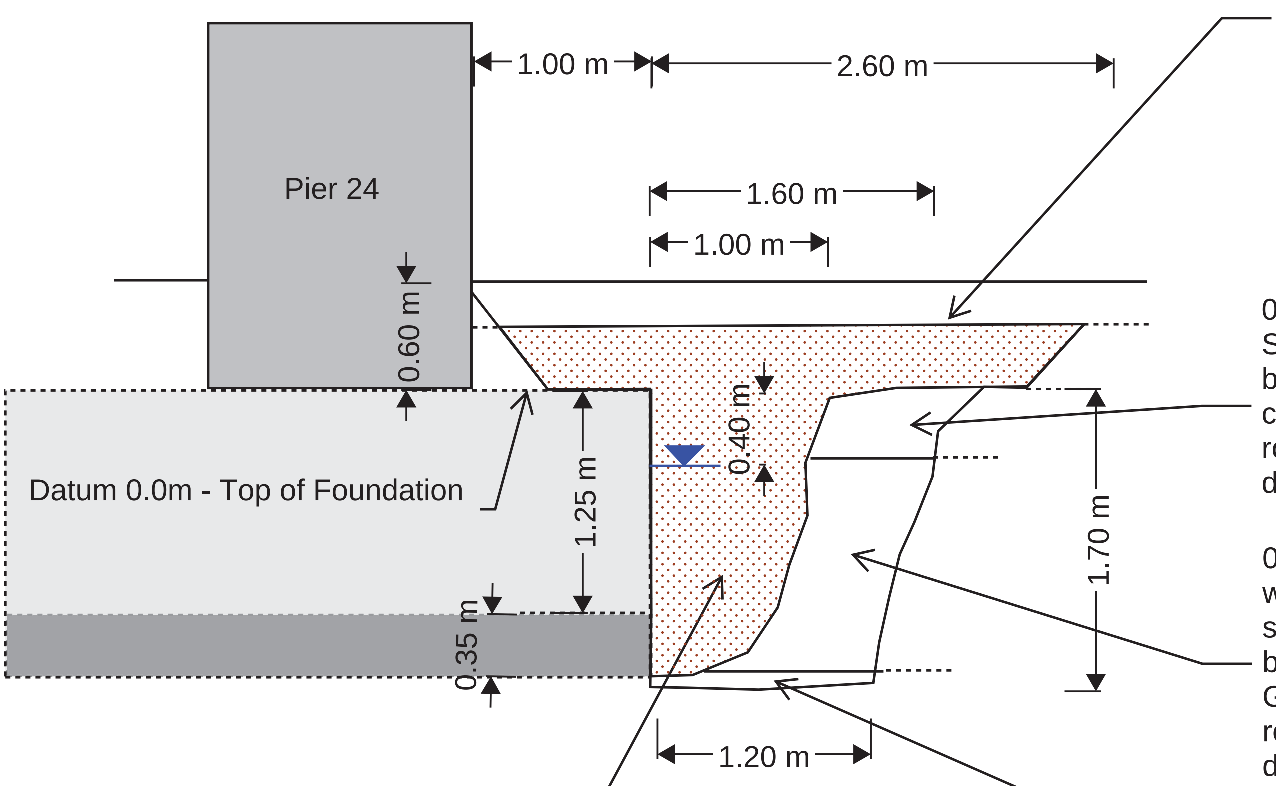
Cairngorm Funicular Mountain Railway -
Trial Pit Investigation - TP24 Layout

[redacted]

Sketch No: TN-03-002 -003

Rev: 01

Date: 18-10-18



Fibrous peat and soft black organic SILT. (Topsoil)

0.0-0.4m Light grey and orange brown gravelly SAND with trace silt and some cobbles and boulders. Assessed as medium dense. Gravel, cobbles and boulders are sub-angular to rounded of granite lithology occasionally decomposed to residual soil. (Glacial Deposits).

0.4-1.6m red and orange brown gravelly SAND with interlayered yellow brown and light grey sandy SILT and occasional cobbles and boulders. Damp. Assessed as medium dense. Gravel, cobbles and boulders are sub-angular to rounded of granite lithology occasionally decomposed to residual soil. (Glacial Deposits).

Below 1.6m . Hard obstruction granite boulder (Glacial Deposits).

0.0-1.6m Dark grey brown and light grey brown sandy fine to coarse GRAVEL with many cobbles and occasional boulders. Assessed as loose to medium dense. Gravel, cobbles and boulders are sub-angular angular to sub-rounded of granite lithology. (Made Ground)

Seepage- moderate <5l/min @0.4m at interface between made and natural ground

COWI

Sketch Title: Cairngorm Funicular Mountain Railway - Trial Pit Investigation - TP23 Section A-A

[redacted]

Sketch No: TN-03-002 -002

Rev: 01

Date: 18-10-18

Face A

Face B

COWI

Sketch Title: Cairngorm Funicular Mountain Railway - Trial Pit Investigation - TP24 Photographs

[redacted]

Sketch No: TN-03-002 -003

Rev: 01

Date: 18-10-18

Face C

Face D

COWI

Sketch Title: Cairngorm Funicular Mountain Railway - Trial Pit Investigation - TP24 Photographs

[redacted]

Sketch No: TN-03-002 -003

Rev: 01

Date: 18-10-18

Red and orange brown gravelly SAND with interlayered yellow brown and light grey sandy SILT and occasional cobbles and boulders. Assessed as medium dense. Gravel, cobbles and boulders are sub-angular to rounded of granite lithology occasionally decomposed to residual soil. (Glacial Deposits).

Formation

Sample D01 1.5-1.6m

COWI

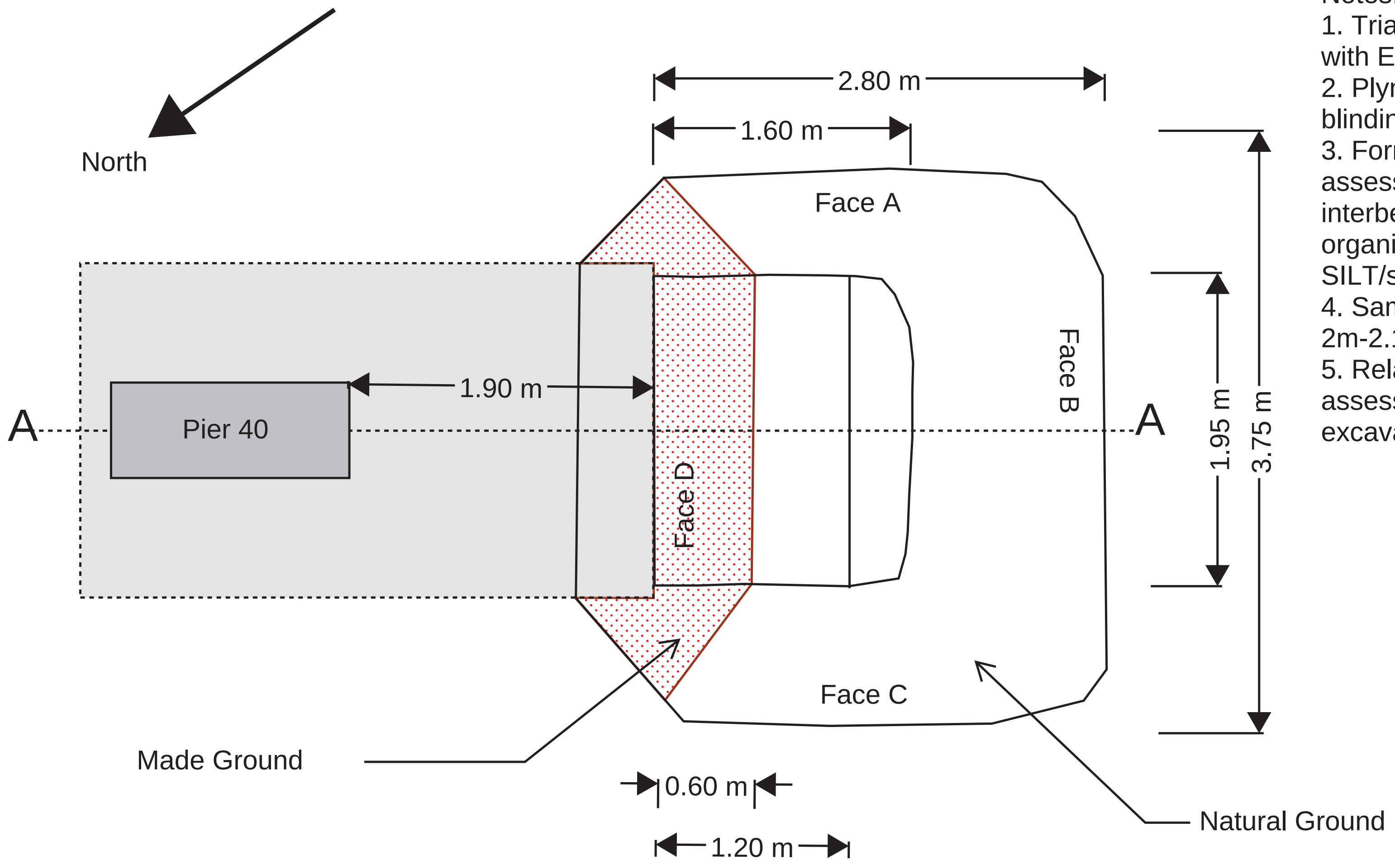
Sketch Title: Cairngorm Funicular Mountain Railway -
Trial Pit Investigation - TP24 Photographs

[redacted]

Sketch No: TN-03-002 -003

Rev: 01

Date: 18-10-18



Notes:
1. Trial Pit Excavated on 10-10-18 with Excavator Volvo ECR58D
2. Plynth Thickness 1.25m with 0.05m blinding concrete
3. Formation 0.2m granular fill assessed as medium dense with interbedded layers of firm silt and soft organic clay overlying firm sandy SILT/silty SAND assessed as loose.
4. Sampling: D1 - 1.6m-1.8m, B1 - 2m-2.1m.
5. Relative density of granular deposits assessed on basis of difficulty of excavation.

COWI

Sketch Title:

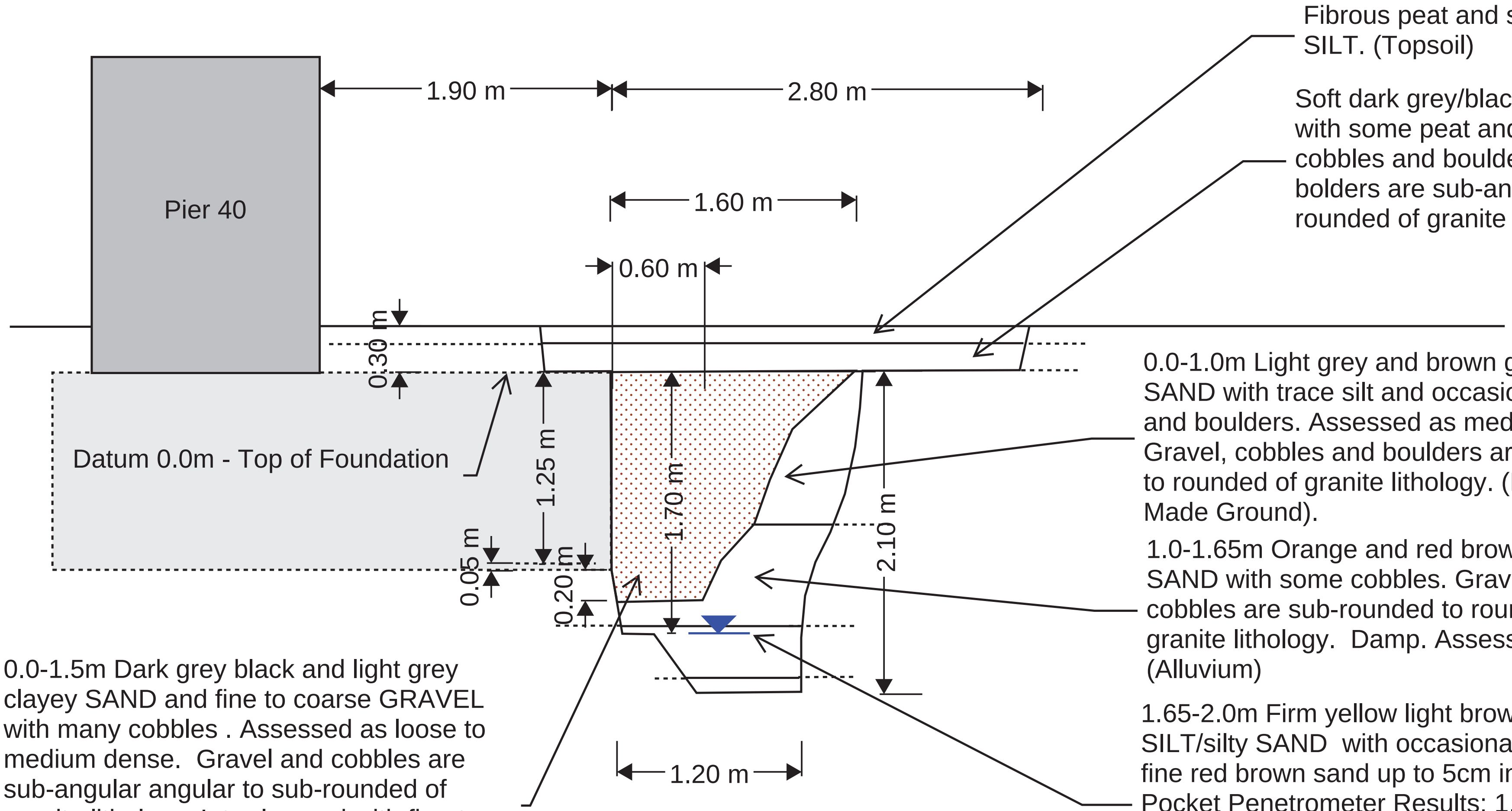
Cairngorm Funicular Mountain Railway - Trial Pit Investigation - TP40 Layout

[redacted]

Sketch No: TN-03-002 -004

Rev: Rev 01

Date: 18-10-18



Fibrous peat and soft black organic SILT. (Topsoil)

Soft dark grey/black clayey SILT with some peat and occassional cobbles and boulders. Cobbles and bolders are sub-angular to sub rounded of granite lithology.

0.0-1.0m Light grey and brown gravelly SAND with trace silt and occasional cobbles and boulders. Assessed as medium dense. Gravel, cobbles and boulders are sub-angular to rounded of granite lithology. (Possible Made Ground).

1.0-1.65m Orange and red brown gravelly SAND with some cobbles. Gravel and cobbles are sub-rounded to rounded of granite lithology. Damp. Assessed as loose (Alluvium)

1.65-2.0m Firm yellow light brown sandy SILT/silty SAND with occasional lenses of fine red brown sand up to 5cm in thickness. Pocket Penetrometer Results: 1.5 kg/cm2 Seepage- low <1l/min @1.7m. (Alluvium)

0.0-1.5m Dark grey black and light grey clayey SAND and fine to coarse GRAVEL with many cobbles . Assessed as loose to medium dense. Gravel and cobbles are sub-angular angular to sub-rounded of granite lithology. Inter-layered with firm to stiff clayey SILT with traces of soft black silt and clay below foundation base, Damp. (Made Ground)

COWI

Sketch Title:

Cairngorm Funicular Mountain Railway - Trial Pit Investigation - TP40 Section A-A

[redacted]

Sketch No: TN-03-002 -004

Rev: 01

Date: 18-10-18

Face A

Face B

COWI

Sketch Title: Cairngorm Funicular Mountain Railway -
Trial Pit Investigation - TP40 Photographs

[redacted]

Sketch No: TN-03-002 -004

Rev: 01

Date: 18-10-18

Face C

Face D

[redacted]

COWI

Sketch Title: Cairngorm Funicular Mountain Railway -
Trial Pit Investigation - TP40 Photographs

Sketch No: TN-03-002 -004

Rev: 01

Date: 18-10-18

Formation: 200mm thick layer Dark grey black and light grey clayey SAND and fine to coarse GRAVEL with many cobbles . Assessed as medium dense. Gravel and cobbles are sub-angular angular to sub-rounded of granite lithology. Interlayered with firm to stiff clayey SILT with traces of soft black silt and clay below foundation base. (Made Ground)

Formation

Below 2.0m red brown slightly silty fine to medium SAND. Assessed as medium dense. (Alluvium)

Sample B1 -2m-2.1m

COWI

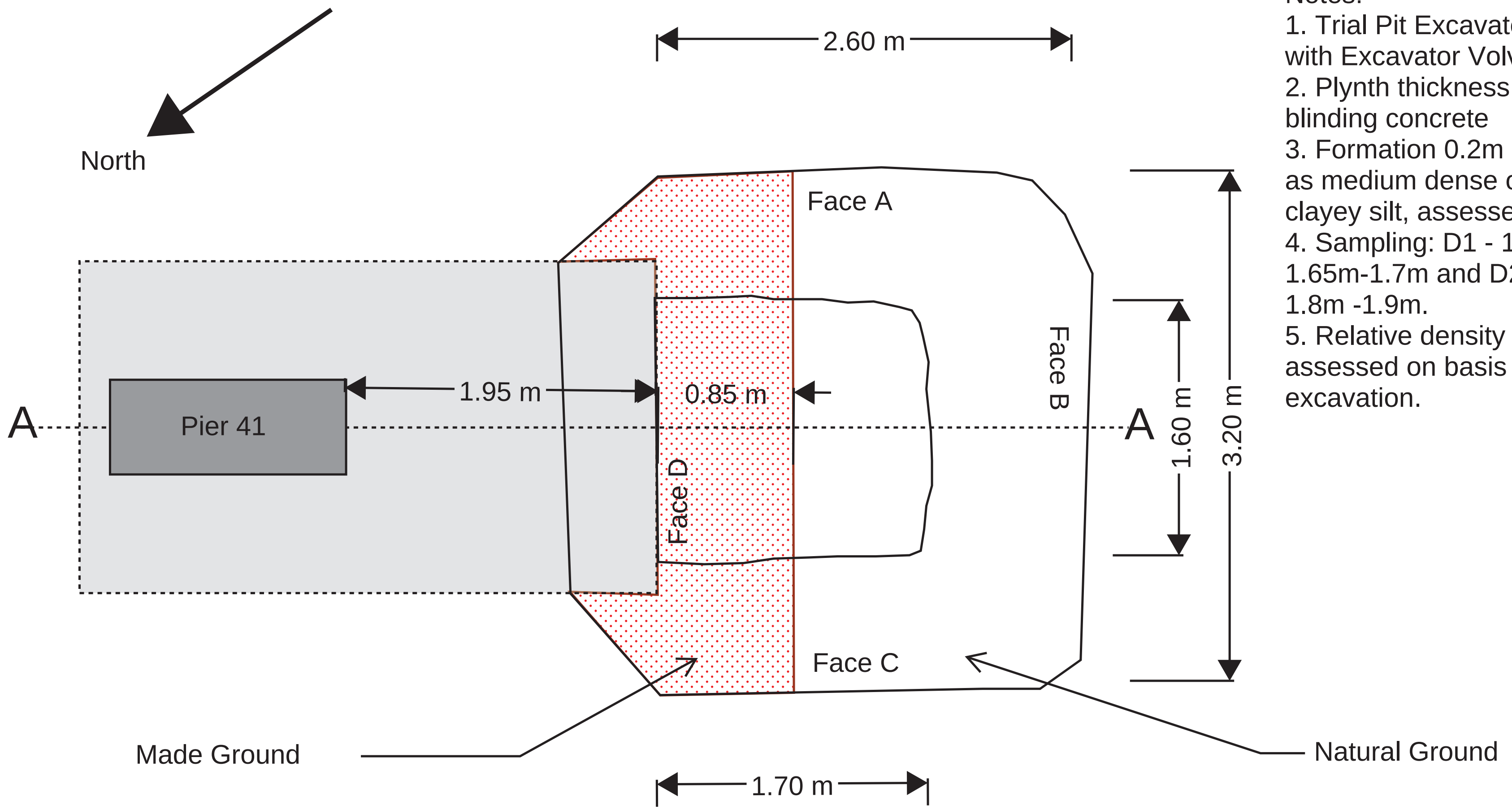
Sketch Title: Cairngorm Funicular Mountain Railway - Trial Pit Investigation - TP40 Photographs

[redacted]

Sketch No: TN-03-002 -004

Rev: 01

Date: 18-10-18



Notes:
1. Trial Pit Excavated on 10-10-18 with Excavator Volvo ECR58D
2. Plynth thickness 1.25m with 0.05m blinding concrete
3. Formation 0.2m granular fill, assessed as medium dense overlying slightly clayey silt, assessed as firm.
4. Sampling: D1 - 1.65m-1.7m, B1 - 1.65m-1.7m and D2 - 1.8m-1.9m, B2 - 1.8m -1.9m.
5. Relative density of granular deposits assessed on basis of difficulty of excavation.

COWI

Sketch Title:

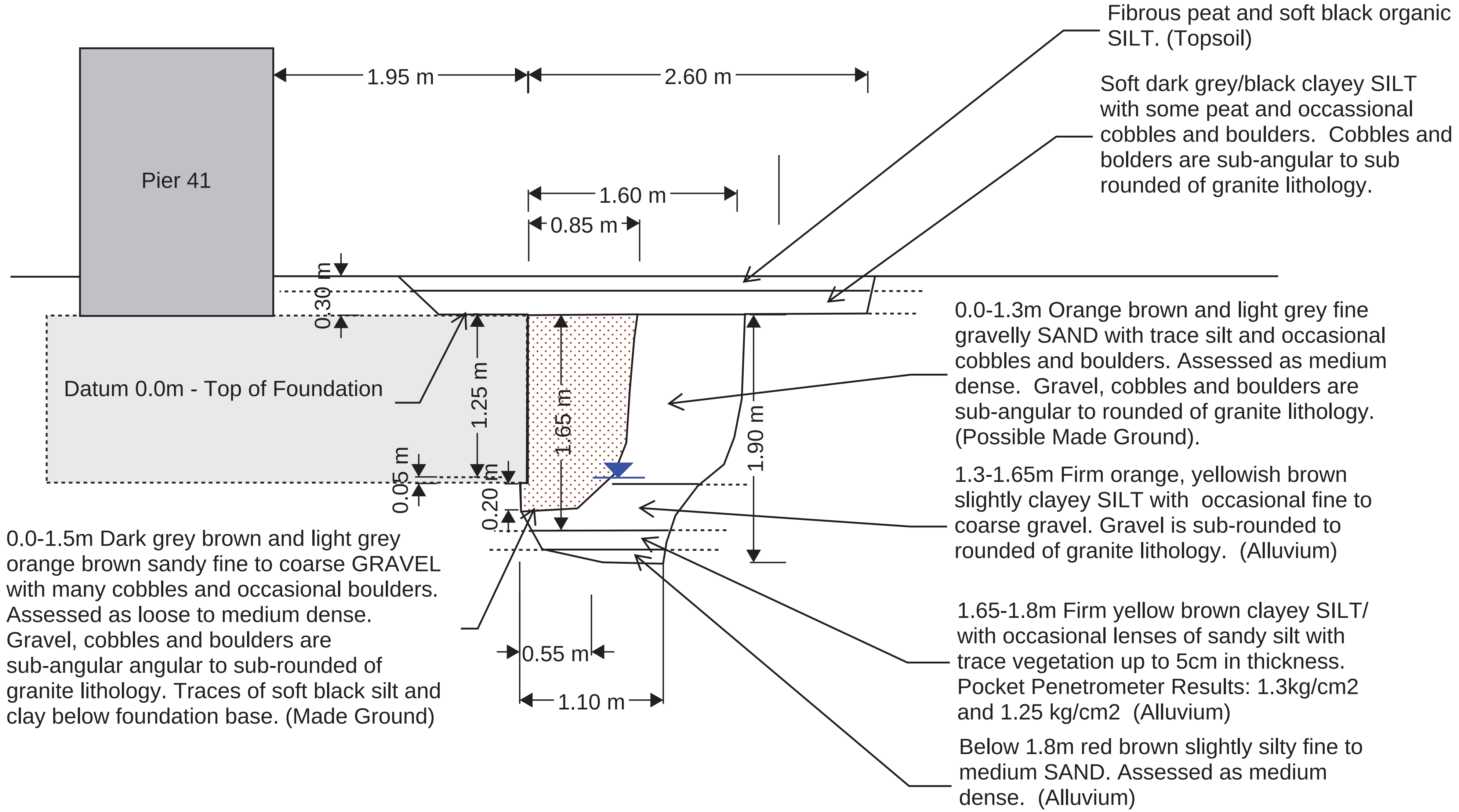
Cairngorm Funicular Mountain Railway -
Trial Pit Investigation - TP41 Layout

[redacted]

Sketch No: TN-03-002 -002

Rev: 01

Date: 17-10-18



Fibrous peat and soft black organic SILT. (Topsoil)

Soft dark grey/black clayey SILT with some peat and occassional cobbles and boulders. Cobbles and bolders are sub-angular to sub rounded of granite lithology.

0.0-1.3m Orange brown and light grey fine gravelly SAND with trace silt and occasional cobbles and boulders. Assessed as medium dense. Gravel, cobbles and boulders are sub-angular to rounded of granite lithology. (Possible Made Ground).

1.3-1.65m Firm orange, yellowish brown slightly clayey SILT with occasional fine to coarse gravel. Gravel is sub-rounded to rounded of granite lithology. (Alluvium)

1.65-1.8m Firm yellow brown clayey SILT/ with occasional lenses of sandy silt with trace vegetation up to 5cm in thickness. Pocket Penetrometer Results: 1.3kg/cm2 and 1.25 kg/cm2 (Alluvium)

Below 1.8m red brown slightly silty fine to medium SAND. Assessed as medium dense. (Alluvium)

0.0-1.5m Dark grey brown and light grey orange brown sandy fine to coarse GRAVEL with many cobbles and occasional boulders. Assessed as loose to medium dense. Gravel, cobbles and boulders are sub-angular angular to sub-rounded of granite lithology. Traces of soft black silt and clay below foundation base. (Made Ground)

COWI

Sketch Title: Cairngorm Funicular Mountain Railway - Trial Pit Investigation - TP41 Section A-A

[redacted]

Sketch No: TN-03-002 -005

Rev: 01

Date: 17-10-18

Face A

Face B

COWI

Sketch Title: Cairngorm Funicular Mountain Railway - Trial Pit Investigation - TP41 Photographs

[redacted]

Sketch No: TN-03-002 -005

Rev: 01

Date: 17-10-18

Face C

Face D

COWI

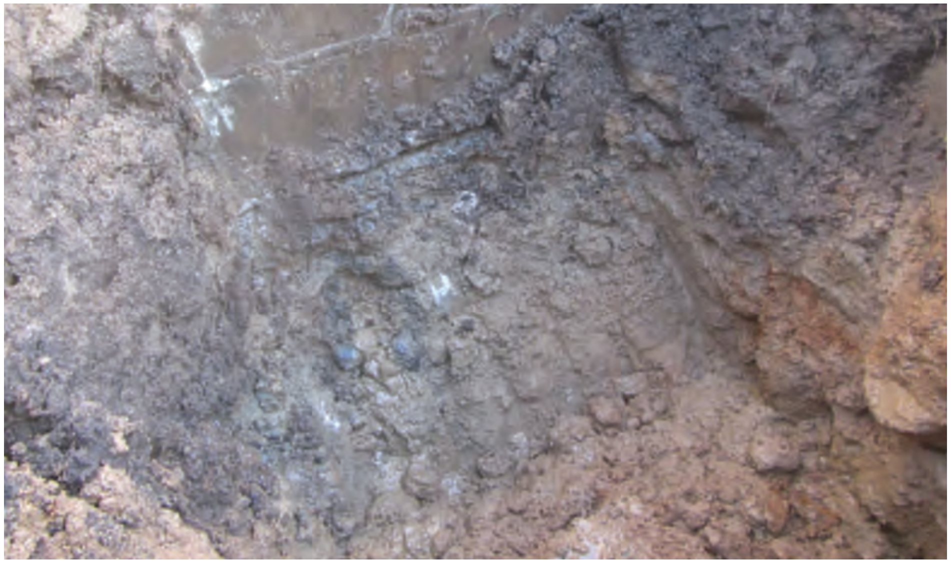
Sketch Title: Cairngorm Funicular Mountain Railway - Trial Pit Investigation - TP41 Photographs

[redacted]

Sketch No: TN-03-002 -005

Rev: 01

Date: 17-10-18



1.65-1.8m Firm yellow brown clayey SILT/ with occasional lenses of sandy silt with trace vegetation up to 5cm in thickness. Pocket Penetrometer Results: 1.3kg/cm2 and 1.25 kg/cm2 (Alluvium)

Samples: D1 - 1.65m-1.7m, B1 - 1.65m-1.7m

Below 1.8m red brown slightly silty fine to medium SAND. Assessed as loose. (Alluvium)

Samples: D2 - 1.8m-1.9m, B2 - 1.8m-1.9m

COWI

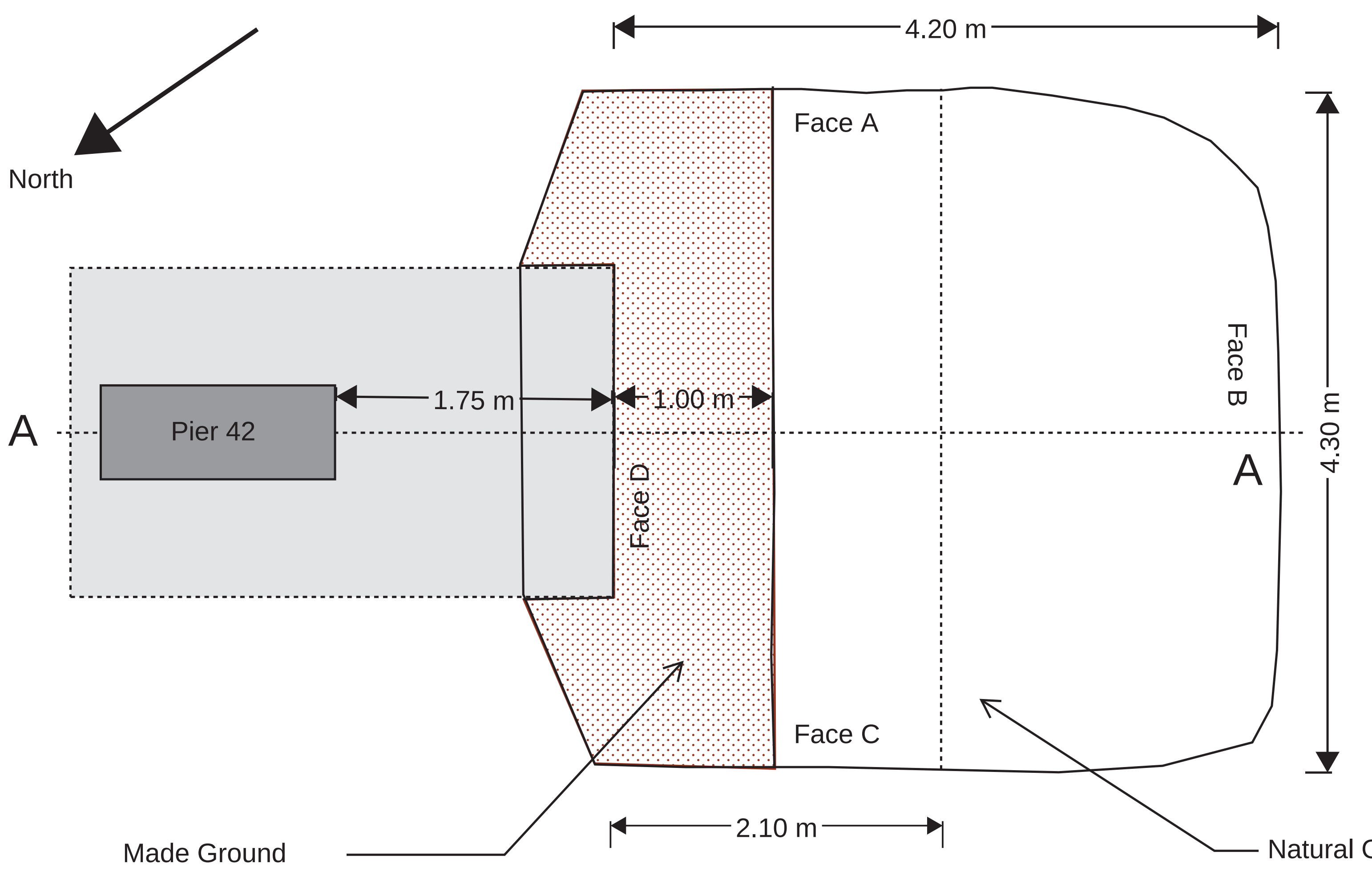
Sketch Title: Cairngorm Funicular Mountain Railway - Trial Pit Investigation - TP41 Photographs

[redacted]

Sketch No: TN-03-002 -005

Rev: 01

Date: 17-10-18



Notes:
1. Trial Pit Excavated on 10-10-18 with Excavator Volvo ECR58D
2. Plynth Thickness 1.25m with 0.05m blinding concrete
3. Formation 0.4m granular fill, assessed as medium dense overlying slightly silty Sand, assessed as loose.
4. Sampling: D1 - 1.8m-2.0m, B1 - 1.8m-2.0m
5. Relative density of granular deposits assessed on basis of difficulty of excavation.

COWI

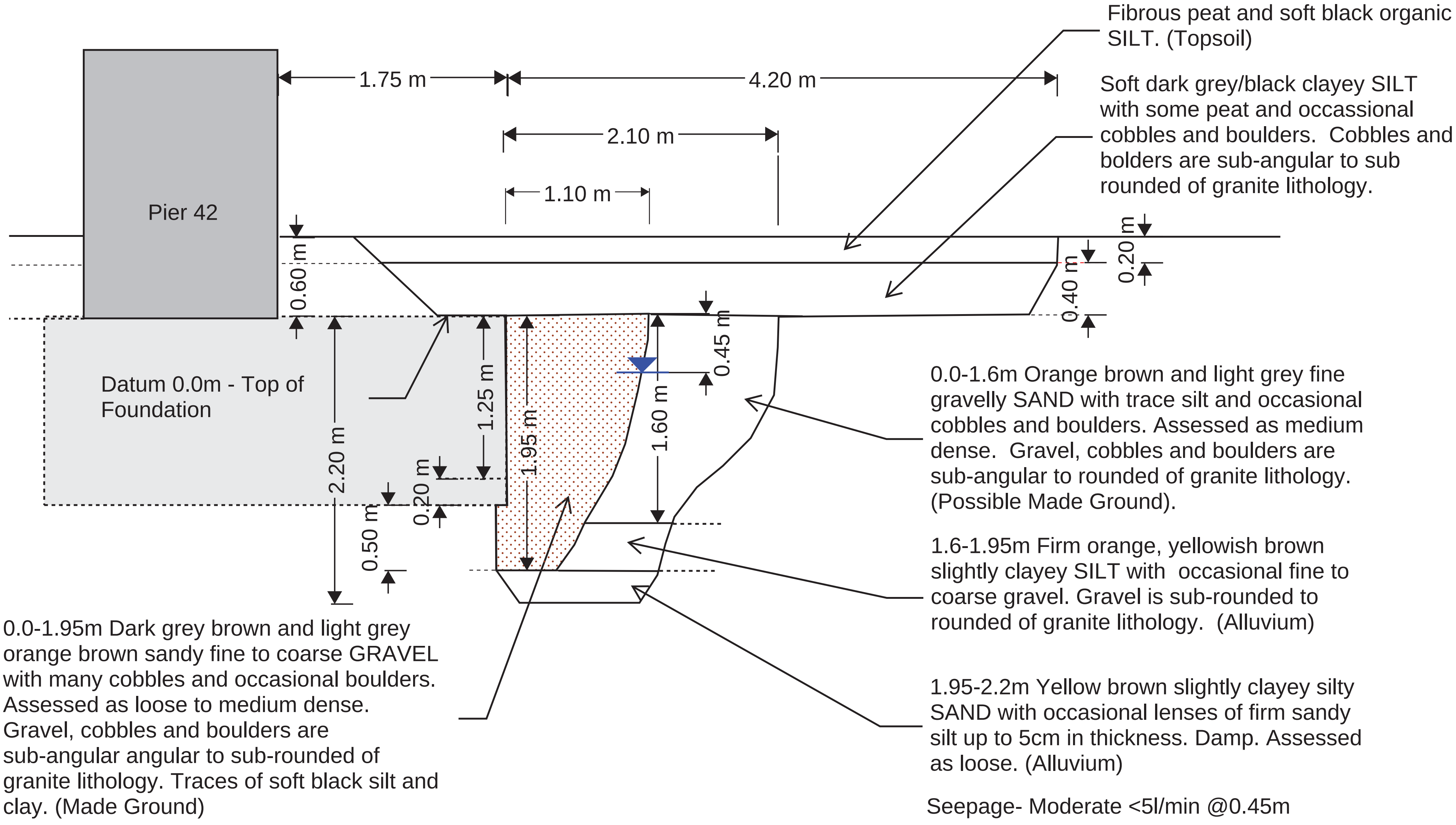
Sketch Title:

Cairngorm Funicular Mountain Railway -
Trial Pit Investigation - TP42 Layout

[redacted]

Sketch No: TN-03-002 -006

Rev: 01
Date: 17-10-18



Fibrous peat and soft black organic SILT. (Topsoil)

Soft dark grey/black clayey SILT with some peat and occassional cobbles and boulders. Cobbles and bolders are sub-angular to sub rounded of granite lithology.

0.0-1.6m Orange brown and light grey fine gravelly SAND with trace silt and occasional cobbles and boulders. Assessed as medium dense. Gravel, cobbles and boulders are sub-angular to rounded of granite lithology. (Possible Made Ground).

1.6-1.95m Firm orange, yellowish brown slightly clayey SILT with occasional fine to coarse gravel. Gravel is sub-rounded to rounded of granite lithology. (Alluvium)

1.95-2.2m Yellow brown slightly clayey silty SAND with occasional lenses of firm sandy silt up to 5cm in thickness. Damp. Assessed as loose. (Alluvium)

Seepage- Moderate <5l/min @0.45m

0.0-1.95m Dark grey brown and light grey orange brown sandy fine to coarse GRAVEL with many cobbles and occasional boulders. Assessed as loose to medium dense. Gravel, cobbles and boulders are sub-angular angular to sub-rounded of granite lithology. Traces of soft black silt and clay. (Made Ground)

COWI

Sketch Title: Cairngorm Funicular Mountain Railway - Trial Pit Investigation - TP42 Section A-A

[redacted]

Sketch No: TN-03-002 -006

Rev: 01

Date: 17-10-18

Face A

Face B

Sketch Title: Cairngorm Funicular Mountain Railway - Trial Pit Investigation - TP42 Photographs

[redacted]

Sketch No: TN-03-002 -006

Rev: 01

Date: 17-10-18

Face C

Face D

COWI

Sketch Title: Cairngorm Funicular Mountain Railway -
Trial Pit Investigation - TP42 Photographs

[redacted]

Sketch No: TN-03-002 -006

Rev: 01

Date: 17-10-18

1.6-1.95m Firm orange, yellowish brown slightly clayey SILT with occasional fine to coarse gravel. Gravel is sub-rounded to rounded of granite lithology. (Alluvium)

Samples: D1 - 1.8m-2.0m

1.95-2.2m Yellow brown slightly clayey silty SAND with occasional lenses of firm sandy silt up to 5cm in thickness. Assessed as loose. (Alluvium)

Samples: B1 - 1.8m-2.0m

COWI

Sketch Title:

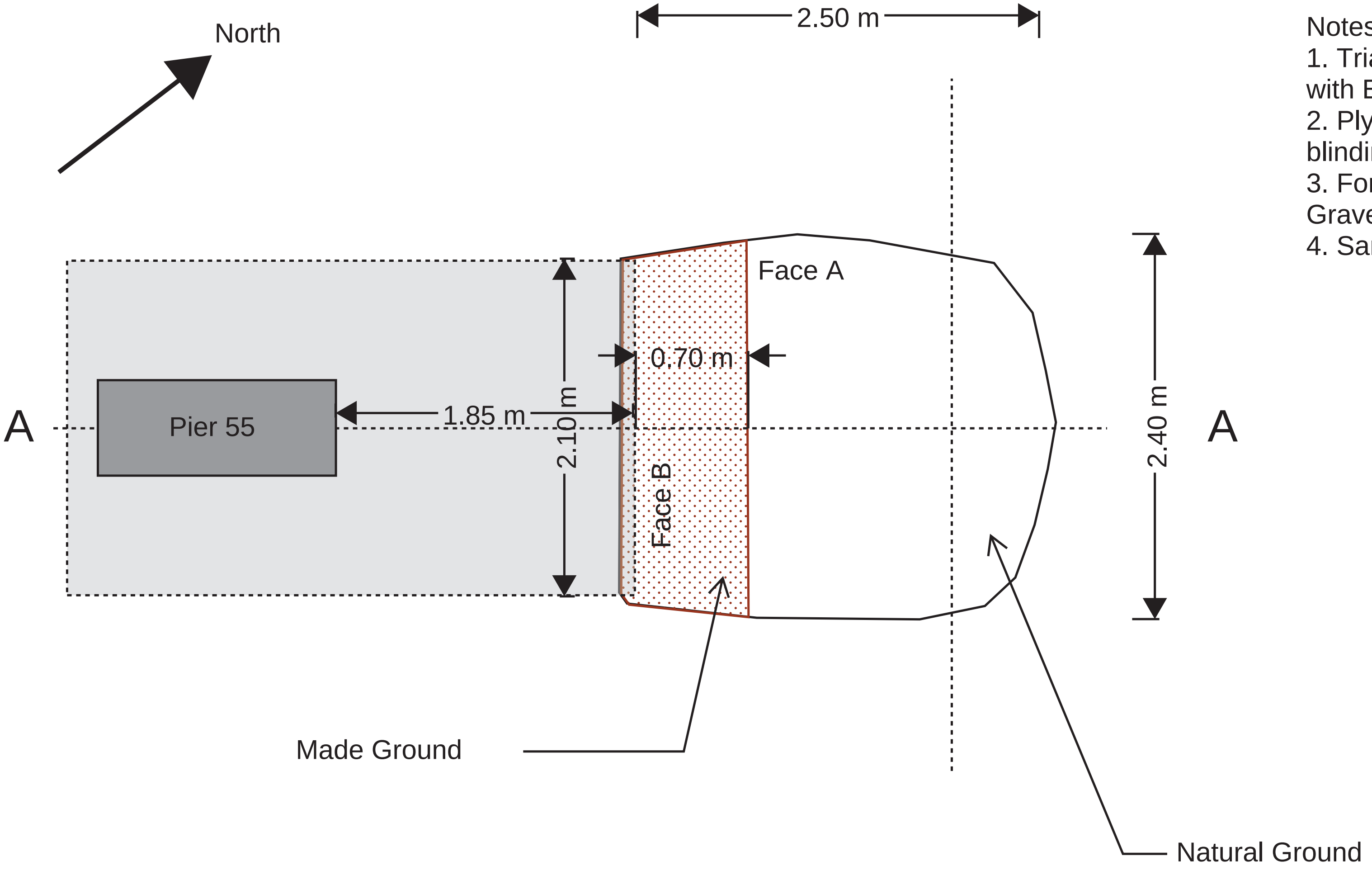
Cairngorm Funicular Mountain Railway -
Trial Pit Investigation - TP42 Photographs

[redacted]

Sketch No: TN-03-002 -006

Rev: 01

Date: 17-10-18



Notes:
1. Trial Pit Excavated on 19-10-18 with Excavator Menzi Muck A91
2. Plynth Thickness 1.05m with 0.25m blinding concrete
3. Formation medium dense Sand & Gravel
4. Sampling: B1 - 1.4-1.5m.

COWI

Sketch Title:

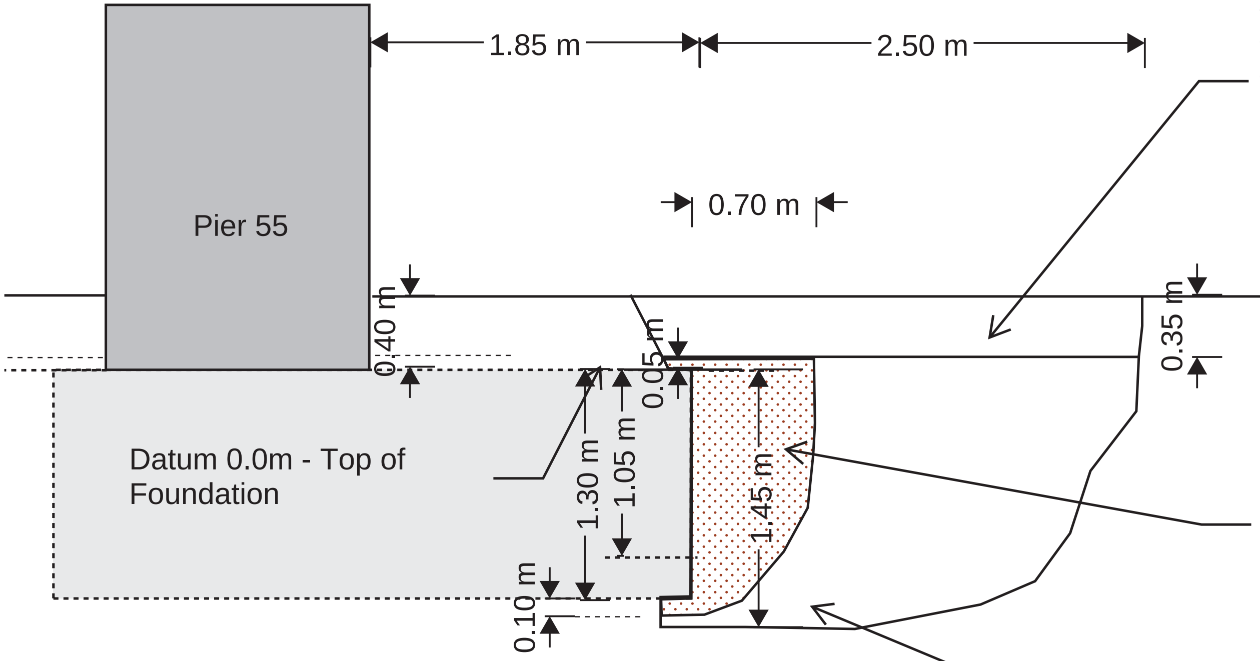
Cairngorm Funicular Mountain Railway -
Trial Pit Investigation - TP55 Layout

[redacted]

Sketch No: TN-03-002 -007

Rev: 01

Date: 22-10-18



350mm Fibrous peat (Topsoil) over soft dark grey/black clayey SILT with some peat and occasional cobbles and boulders. Cobbles and boulders are sub-angular to sub rounded of granite lithology.

-0.05-1.40m Dark grey brown damp silty clayey fine to coarse SAND with fine to medium angular to sub angular GRAVEL occasional sub-angular to sub-rounded cobbles and boulders of granite lithology. Assessed as loose to medium dense. Traces of soft black loamy organic silt. (Made Ground) UCS 1Kgf/cm2

-0.05-1.45m Light orangey pinkish brown dry slightly clayey silty fine to coarse SAND with fine to coarse angular to sub angular GRAVEL occasional sub-angular to sub-rounded cobbles and boulders of granite lithology. Damp. Assessed as medium dense. UCS 1.5Kgf/cm2

Seepage- None

COWI

Sketch Title:

Cairngorm Funicular Mountain Railway - Trial Pit Investigation - TP55 Section A-A

[redacted]

Sketch No: TN-03-002 -007

Rev: 01

Date: 22-10-18

Face A

Face B

COWI

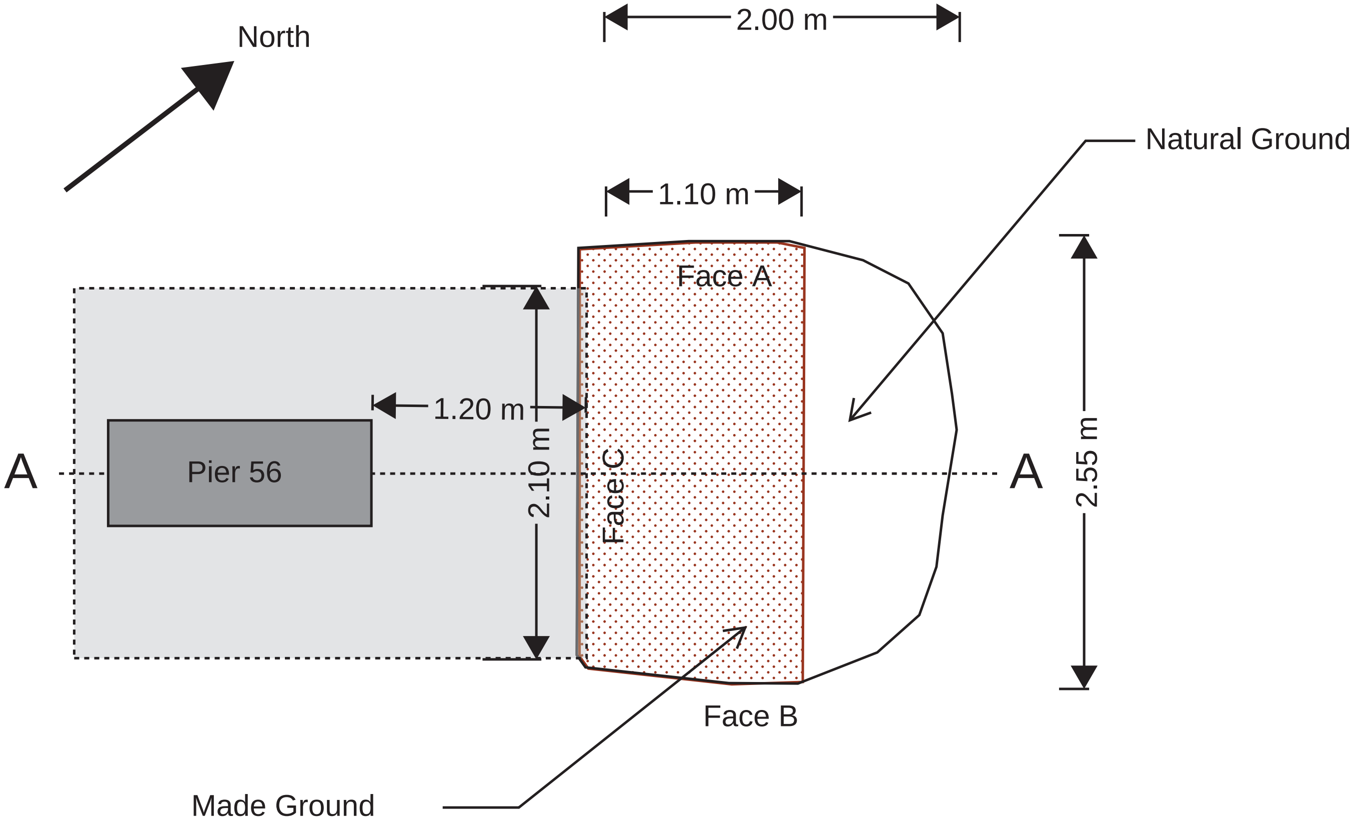
Sketch Title: Cairngorm Funicular Mountain Railway - Trial Pit Investigation - TP55 Photographs

[redacted]

Sketch No: TN-03-002 -007

Rev: 01

Date: 22-10-18



Notes:
1. Trial Pit Excavated on 19-10-18 with Excavator Menzi Muck A91
2. Plynth Thickness 1.25m with 0.02m blinding concrete
3. Formation medium dense Sand & Gravel
4. Sampling: B1 - 1.45m.

COWI

Sketch Title:

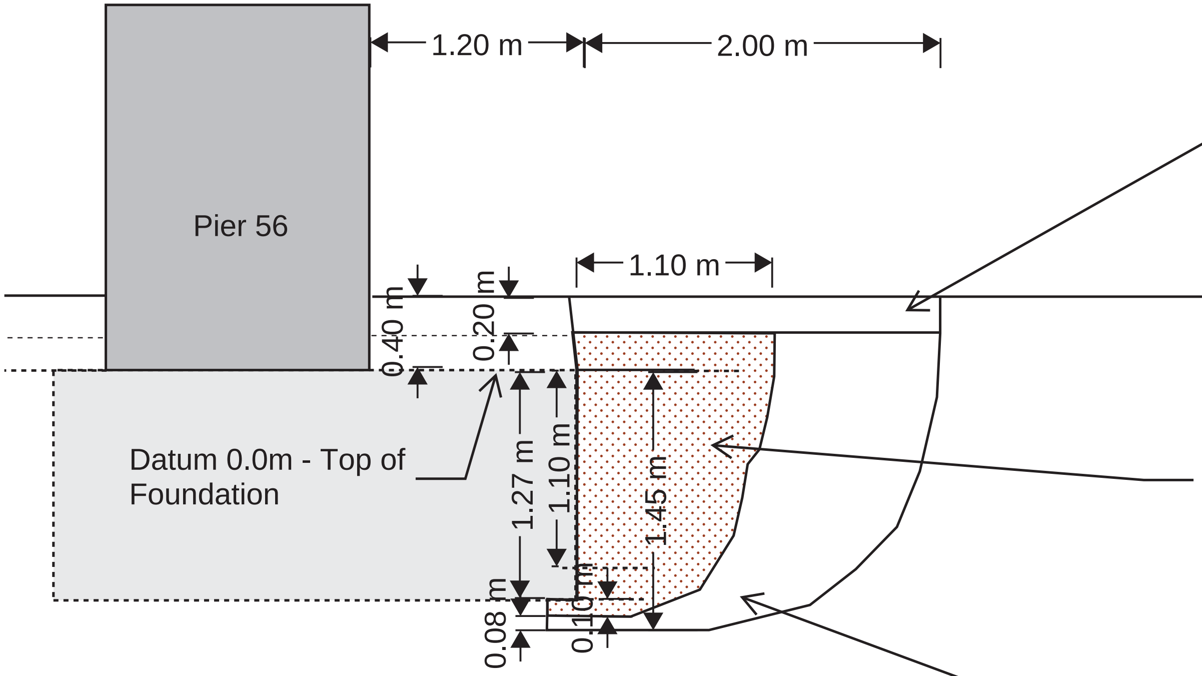
Cairngorm Funicular Mountain Railway - Trial Pit Investigation - TP56 Layout

[redacted]

Sketch No: TN-03-002 -008

Rev: 01

Date: 22-10-18



200mm Fibrous peat (Topsoil) over soft dark grey/black clayey SILT with some peat and occasional cobbles and boulders. Cobbles and boulders are sub-angular to sub rounded of granite lithology.

-0.2-1.37m Light orangey greyish brown damp clayey silty fine to coarse SAND with fine to medium angular to sub angular GRAVEL occasional sub-angular to sub-rounded cobbles and boulders of granite lithology. Assessed as loose. Traces of soft black loamy organic silt. (Made Ground)

-0.2-1.45m Light orangey pinkish brown dry organic silty fine to coarse SAND with fine to coarse angular to sub angular GRAVEL occasional sub-angular to sub-rounded cobbles and boulders of granite lithology. Assessed as medium dense.

Seepage- None, dry

COWI

Sketch Title:

Cairngorm Funicular Mountain Railway - Trial Pit Investigation - TP56 Section A-A

[redacted]

Sketch No: TN-03-002 -008

Rev: 01

Date: 22-10-18

Face A

Face B

COWI

Sketch Title:

Cairngorm Funicular Mountain Railway -
Trial Pit Investigation - TP56 Photographs

[redacted]

Sketch No: TN-03-002 -008

Rev: 01

Date: 22-10-18

Face C

COWI

Sketch Title:

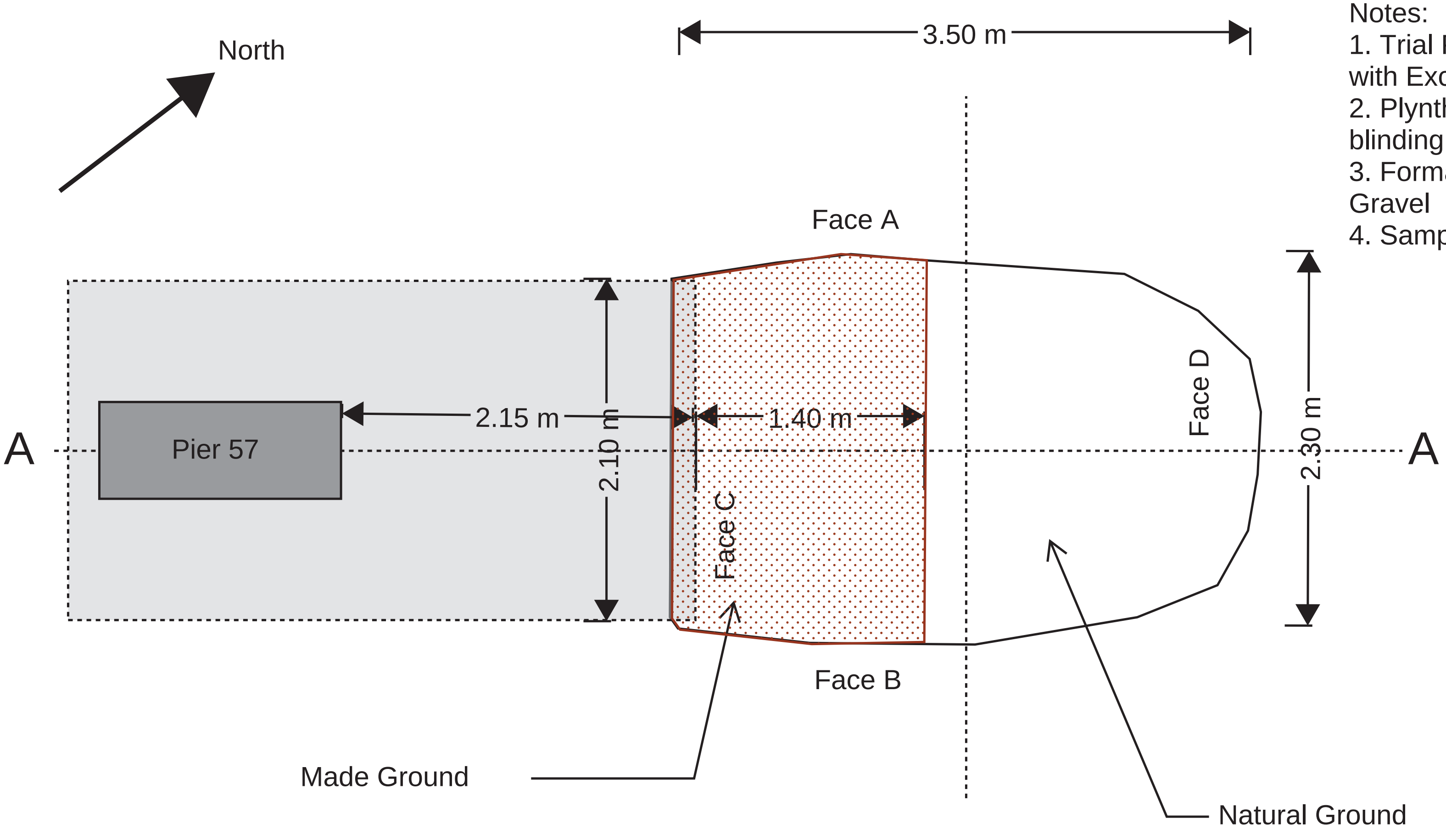
Cairngorm Funicular Mountain Railway -
Trial Pit Investigation - TP56 Photographs

[redacted]

Sketch No: TN-03-002 -008

Rev: 01

Date: 22-10-18



Notes:
1. Trial Pit Excavated on 19-10-18 with Excavator Menzi Muck A91
2. Plynth Thickness 1.25m with 0.2m blinding concrete
3. Formation medium dense Sand & Gravel
4. Sampling: B1 - 1.5m.

COWI

Sketch Title:

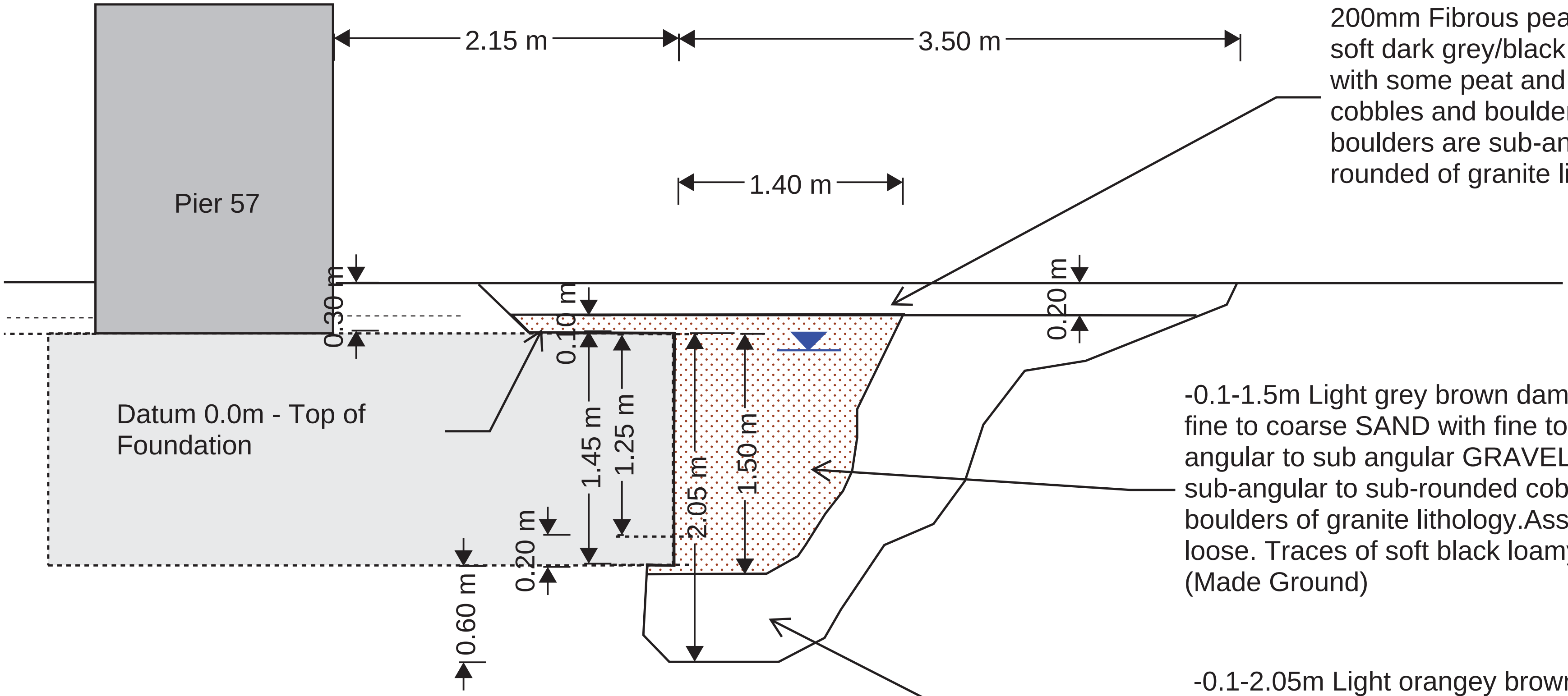
Cairngorm Funicular Mountain Railway - Trial Pit Investigation - TP57 Layout

[redacted]

Sketch No: TN-03-002 -009

Rev: 01

Date: 22-10-18



200mm Fibrous peat (Topsoil) over soft dark grey/black clayey SILT with some peat and occasional cobbles and boulders. Cobbles and boulders are sub-angular to sub rounded of granite lithology.

-0.1-1.5m Light grey brown damp clayey silty fine to coarse SAND with fine to medium angular to sub angular GRAVEL occasional sub-angular to sub-rounded cobbles and boulders of granite lithology.Assessed as loose. Traces of soft black loamy organic silt. (Made Ground)

-0.1-2.05m Light orangey brown dry organic loamy silty fine to coarse SAND with fine to coarse angular to sub angular GRAVEL occasional sub-angular to sub-rounded cobbles and boulders of granite lithology.Assessed as loose to medium dense.

Seepage- Slight <1l/min @0.2m

Sketch Title:

Cairngorm Funicular Mountain Railway - Trial Pit Investigation - TP57 Section A-A

[redacted]

TN-03-002 -009

Rev: 01

Date: 22-10-18

Face A

Face B

COWI

Sketch Title: Cairngorm Funicular Mountain Railway -
Trial Pit Investigation - TP57 Photographs

[redacted]

Sketch No: TN-03-002 -009

Rev: 01

Date: 22-10-18

Face C

Face D

COWI

Sketch Title:

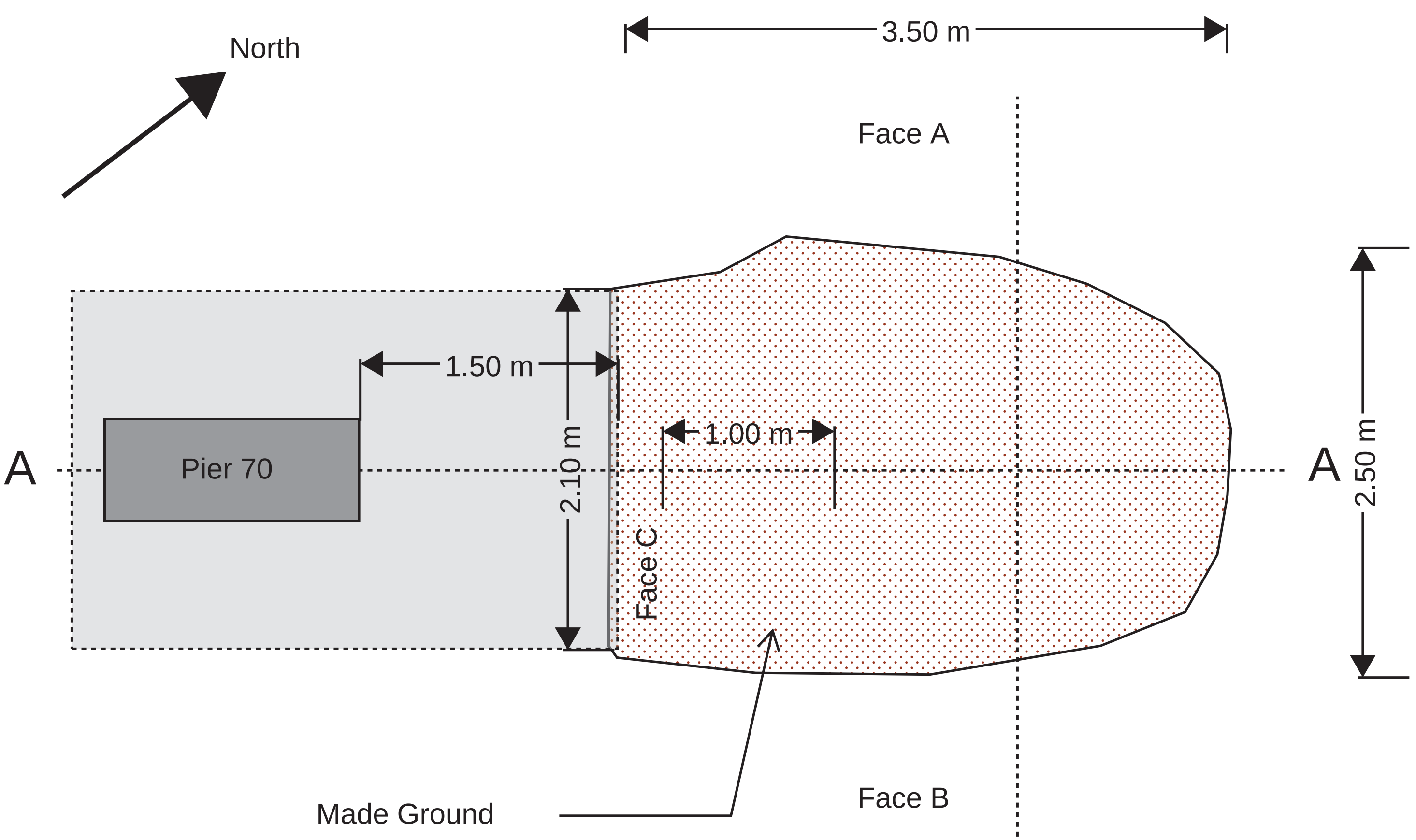
Cairngorm Funicular Mountain Railway -
Trial Pit Investigation - TP57 Photographs

[redacted]

Sketch No: TN-03-002 -009

Rev: 01

Date: 22-10-18



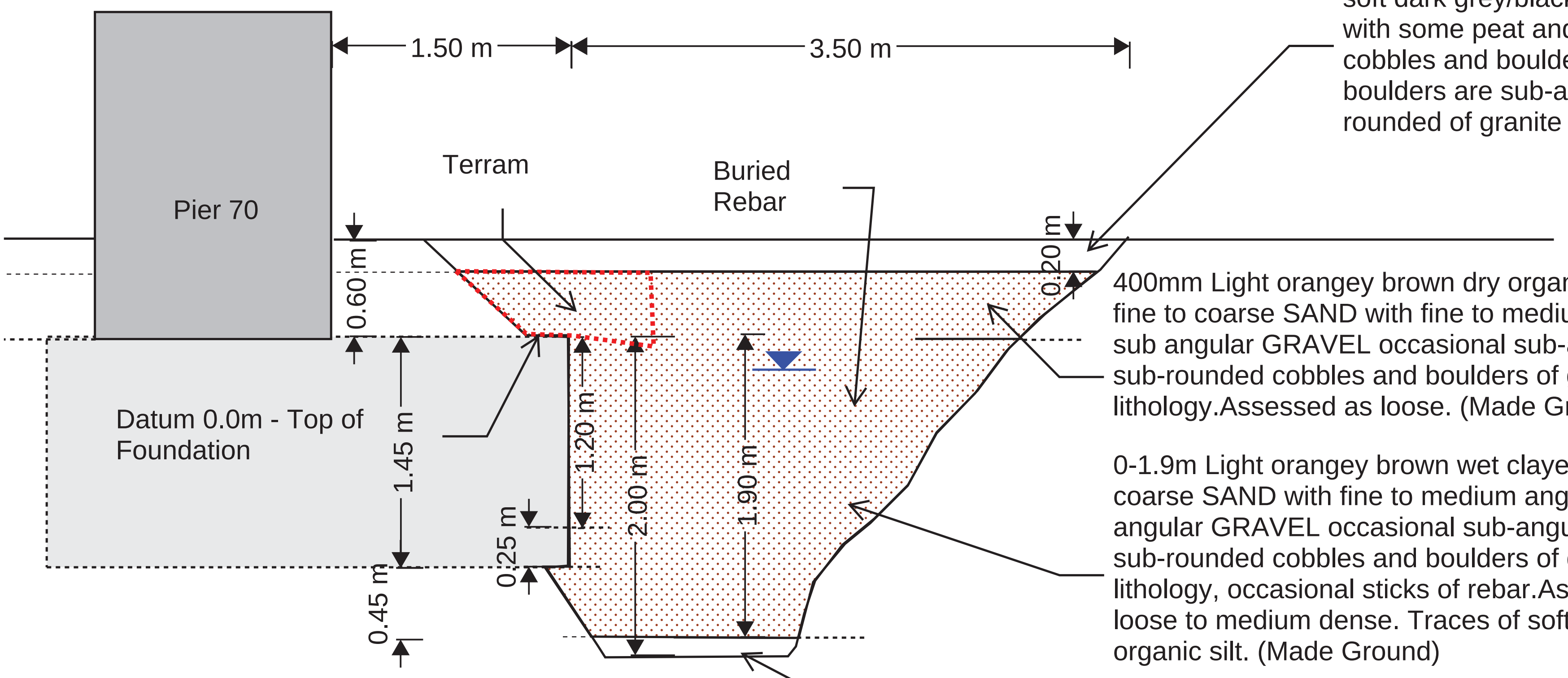
Notes:
1. Trial Pit Excavated on 18-10-18 with Excavator Menzi Muck A91
2. Plynth Thickness 1.2m with 0.25m blinding concrete
3. Formation medium dense to dense Sand & Gravel
4. Sampling: B1 - 1.8m. B2 - 2.0m

COWI

Sketch Title: Cairngorm Funicular Mountain Railway - Trial Pit Investigation - TP70 Layout

[redacted]

Sketch No: TN-03-002 -010
Rev: 01
Date: 22-10-18



200mm Fibrous peat (Topsoil) over soft dark grey/black clayey SILT with some peat and occasional cobbles and boulders. Cobbles and boulders are sub-angular to sub rounded of granite lithology.

400mm Light orangey brown dry organic loamy silty fine to coarse SAND with fine to medium angular to sub angular GRAVEL occasional sub-angular to sub-rounded cobbles and boulders of granite lithology.Assessed as loose. (Made Ground)

0-1.9m Light orangey brown wet clayey silty fine to coarse SAND with fine to medium angular to sub angular GRAVEL occasional sub-angular to sub-rounded cobbles and boulders of granite lithology, occasional sticks of rebar.Assessed as loose to medium dense. Traces of soft black loamy organic silt. (Made Ground)

1.9-2m Light pinkish brown damp silty fine to coarse SAND with fine to medium angular to sub angular GRAVEL occasional angular to sub-angular cobbles and boulders of granite lithology. Assessed as medium dense to dense.

Seepage- Moderate <5l/min @0.2m

COWI

Sketch Title: Cairngorm Funicular Mountain Railway - Trial Pit Investigation - TP70 Section A-A

[redacted]

Sketch No: TN-03-002 -010

Rev: 01

Date: 22-10-18

Face A

Face B

[redacted]

Sketch Title: Cairngorm Funicular Mountain Railway - Trial Pit Investigation - TP70 Photographs

Sketch No: TN-03-002 -010

Rev: 01

Date: 22-10-18

Face C

1.9-2.0m Light pinkish brown damp silty fine to coarse SAND with fine to medium angular to sub angular GRAVEL occasional angular to sub-angular cobbles and boulders of granite lithology

COWI

Sketch Title:

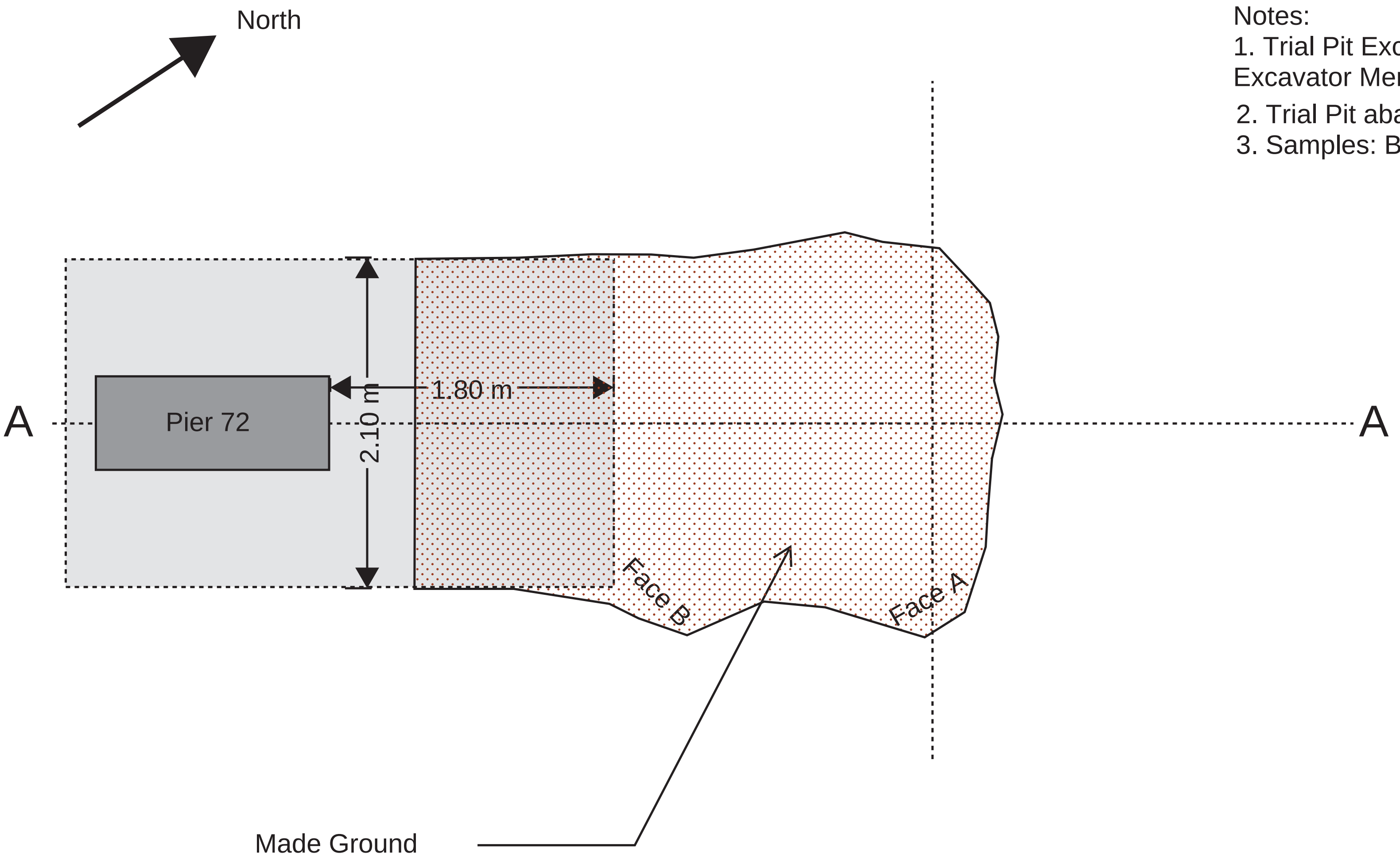
Cairngorm Funicular Mountain Railway -
Trial Pit Investigation - TP70 Photographs

[redacted]

TN-03-002 -010

Rev: 01

Date: 22-10-18



Notes:
1. Trial Pit Excavated on 17-10-18
Excavator Menzi Muck A91
2. Trial Pit abandoned due to water ingress
3. Samples: B1 & T1 - 1m

COWI

Sketch Title:

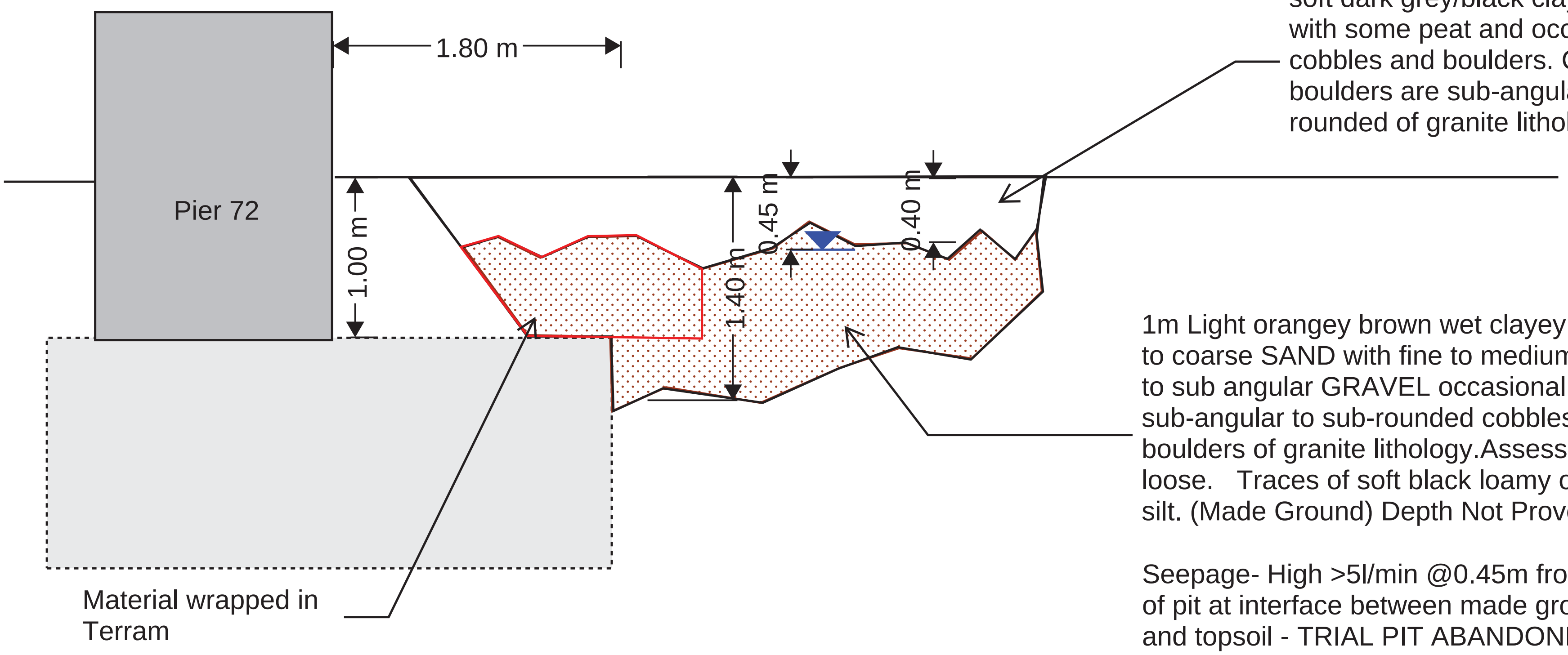
Cairngorm Funicular Mountain Railway -
Trial Pit Investigation - TP72 Layout

[redacted]

Sketch No: TN-03-002 -011

Rev: 01

Date: 22-10-18



400mm Fibrous peat (Topsoil) over soft dark grey/black clayey SILT with some peat and occasional cobbles and boulders. Cobbles and boulders are sub-angular to sub rounded of granite lithology.

1m Light orangey brown wet clayey silty fine to coarse SAND with fine to medium angular to sub angular GRAVEL occasional sub-angular to sub-rounded cobbles and boulders of granite lithology.Assessed as loose. Traces of soft black loamy organic silt. (Made Ground) Depth Not Proven

Seepage- High >5l/min @0.45m from top of pit at interface between made ground and topsoil - TRIAL PIT ABANDONED

Material wrapped in Terram

COWI

Sketch Title: Cairngorm Funicular Mountain Railway - Trial Pit Investigation - TP72 Section A-A

[redacted]

Sketch No: TN-03-002 -011

Rev: 01

Date: 22-10-18

Face A

Face B

COWI

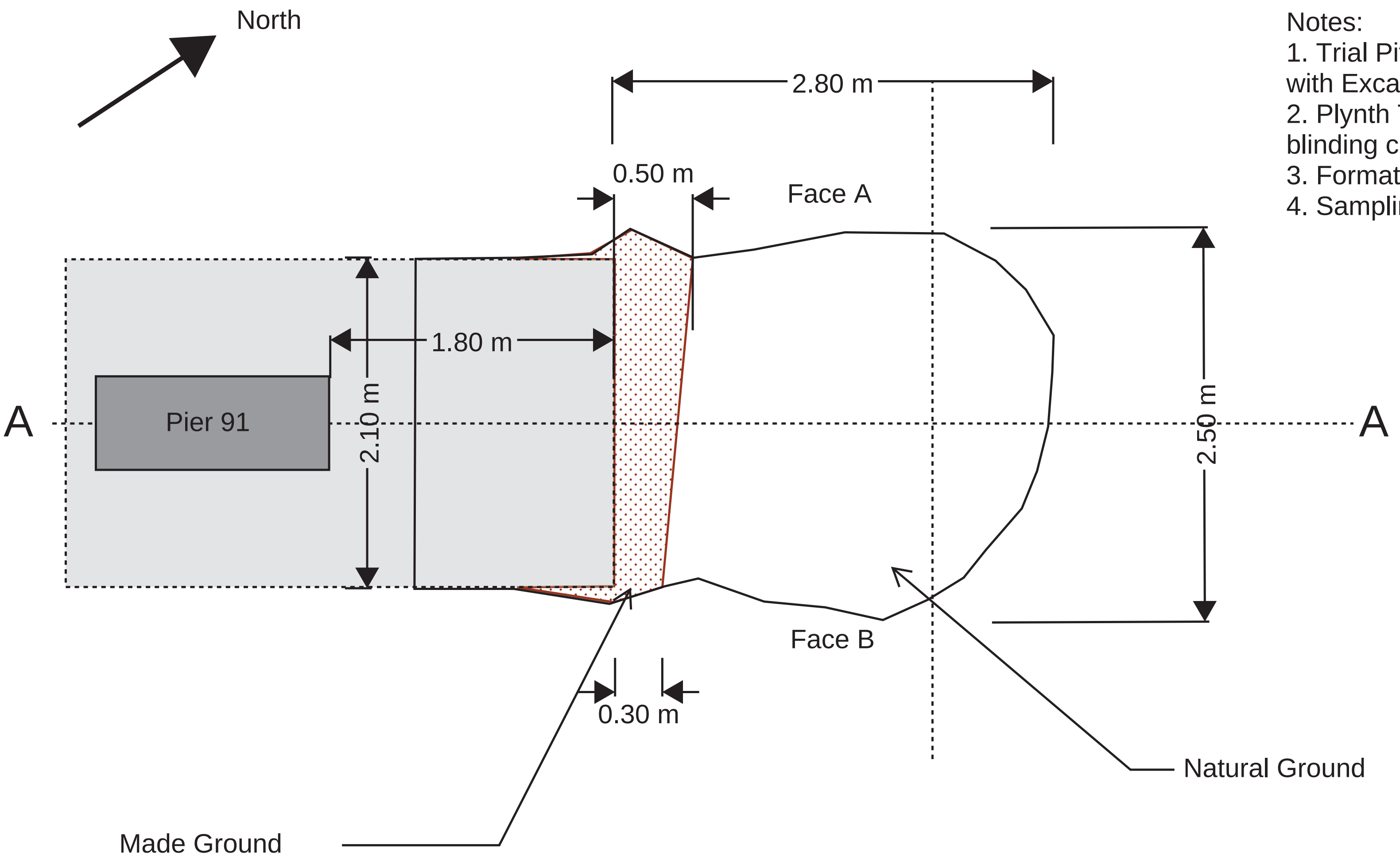
Sketch Title: Cairngorm Funicular Mountain Railway - Trial Pit Investigation - TP72 Photographs

[redacted]

Sketch No: TN-03-002 -011

Rev: 01

Date: 22-10-18

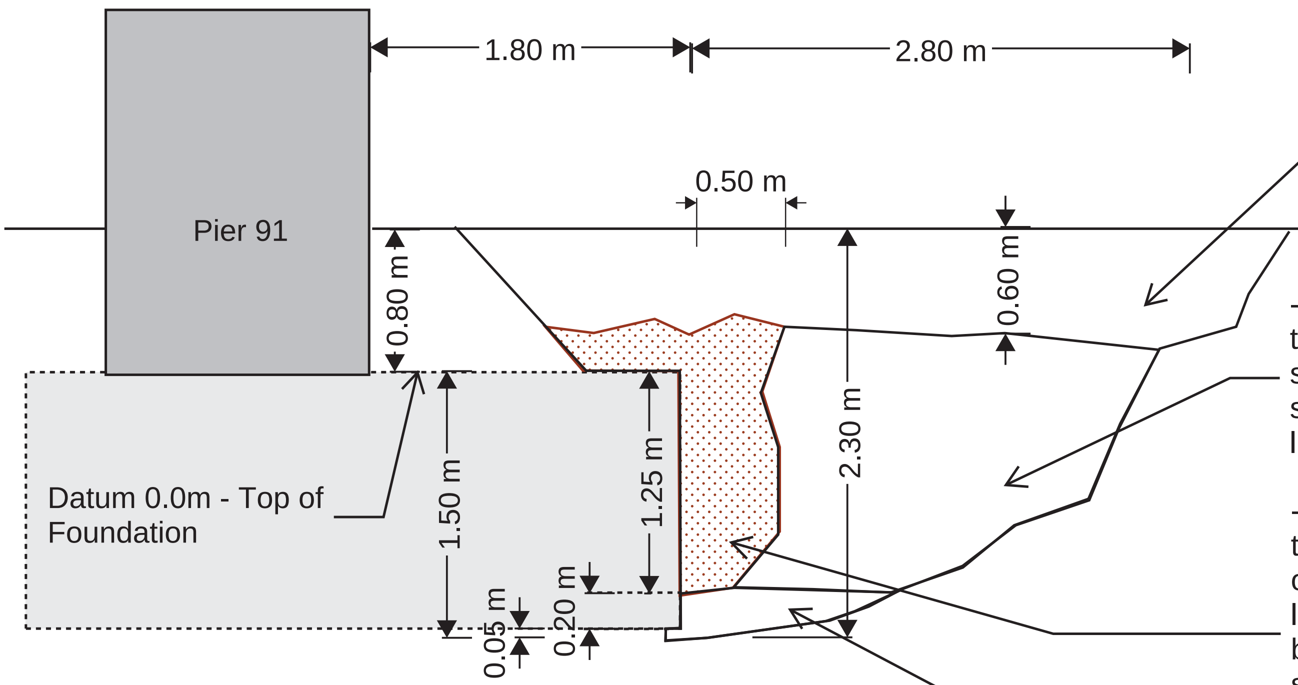


Notes:
1. Trial Pit Excavated on 17-10-18
with Excavator Menzi Muck A91
2. Plynth Thickness 1.25m with 0.2m
blinding concrete
3. Formation weathered granite bedrock.
4. Sampling: B1 - 1.2m.

[redacted]

Sketch Title:
Cairngorm Funicular Mountain Railway -
Trial Pit Investigation - TP91 Layout

Sketch No: TN-03-002 -012
Rev: 01
Date: 22-10-18



600mm Fibrous peat (Topsoil) over soft dark grey/black clayey SILT with some peat and occasional cobbles and boulders. Cobbles and boulders are sub-angular to sub rounded of granite lithology.

-0.2-1.25m Pinkish reddy light brown silty fine to coarse SAND with fine to coarse angular to sub angular gravel and abundant angular to sub angular cobbles and boulders of Granite lithology. Assessed as medium dense.

-0.2-1.25m Light grey brown silty sandy fine to coarse GRAVEL with many cobbles and occasional boulders. Assessed as loose/medium dense. Gravel, cobbles and boulders are sub-angular angular to sub-rounded of granite lithology. Traces of soft black loamy organic silt. Damp. (Made Ground)

1.25-1.5m Assessed as Granite bedrock (from limited recovered scrapings of broken rock bound in coarse sand matrix).

Dry Excavation

COWI

Sketch Title: Cairngorm Funicular Mountain Railway - Trial Pit Investigation - TP91 Section A-A

[redacted]

Sketch No: TN-03-002 -012

Rev: 01

Date: 22-10-18

Face A

Face B

COWI

Sketch Title: Cairngorm Funicular Mountain Railway -
Trial Pit Investigation - TP91 Photographs

[redacted]

Sketch No: TN-03-002 -012

Rev: 01

Date: 22-10-18

COWI

CAIRNGORM FUNICULAR MOUNTAIN RAILWAY - STRUCTURAL INSPECTION

GROUND INVESTIGATION REPORT

Appendix B - Laboratory Test Results (MatTest Ltd)

MATtest Limited
materials testing & consultancy

10 Queenslie Point
Queenslie Industrial Estate
120 Stepps Road
Glasgow
G33 3NQ

Tel: 0141 774 4032

email: info@mattest.org
Website: www.mattest.org

LABORATORY TEST CERTIFICATE

Certificate No : **18/1442 - 01**

To : [redacted]

Client : **ADAC-structures Ltd.**
Dalchully
Laggan
Inverness-shire
PH20 1BU

Dear Sirs,

LABORATORY TESTING OF SOIL

Introduction

We refer to samples taken from Cairn Gorm Funicular Mountain Railway and delivered to our laboratory on 05th November 2018.

Material & Source

Sample Reference : See Report Plates
Sampled By : Client
Sampling Certificate : Not Supplied
Location : See Report Plates
Description : See Page 2
Date Sampled : Not Supplied
Date Tested : 05th November 2018 Onwards
Source : Cairn Gorm Funicular Mountain Railway

Test Results;

As Detailed On Page 2 to Page 29 inclusive

Comments;

Opinions and interpretations expressed herein are outside the scope of UKAS accreditation
This report should not be reproduced except in full without the written approval of the laboratory
All remaining samples for this project will be disposed of 28 days after issue of this test certificate

Remarks;

Approved for Issue

[redacted] Date 21/11/2018

UKAS TESTING 2643

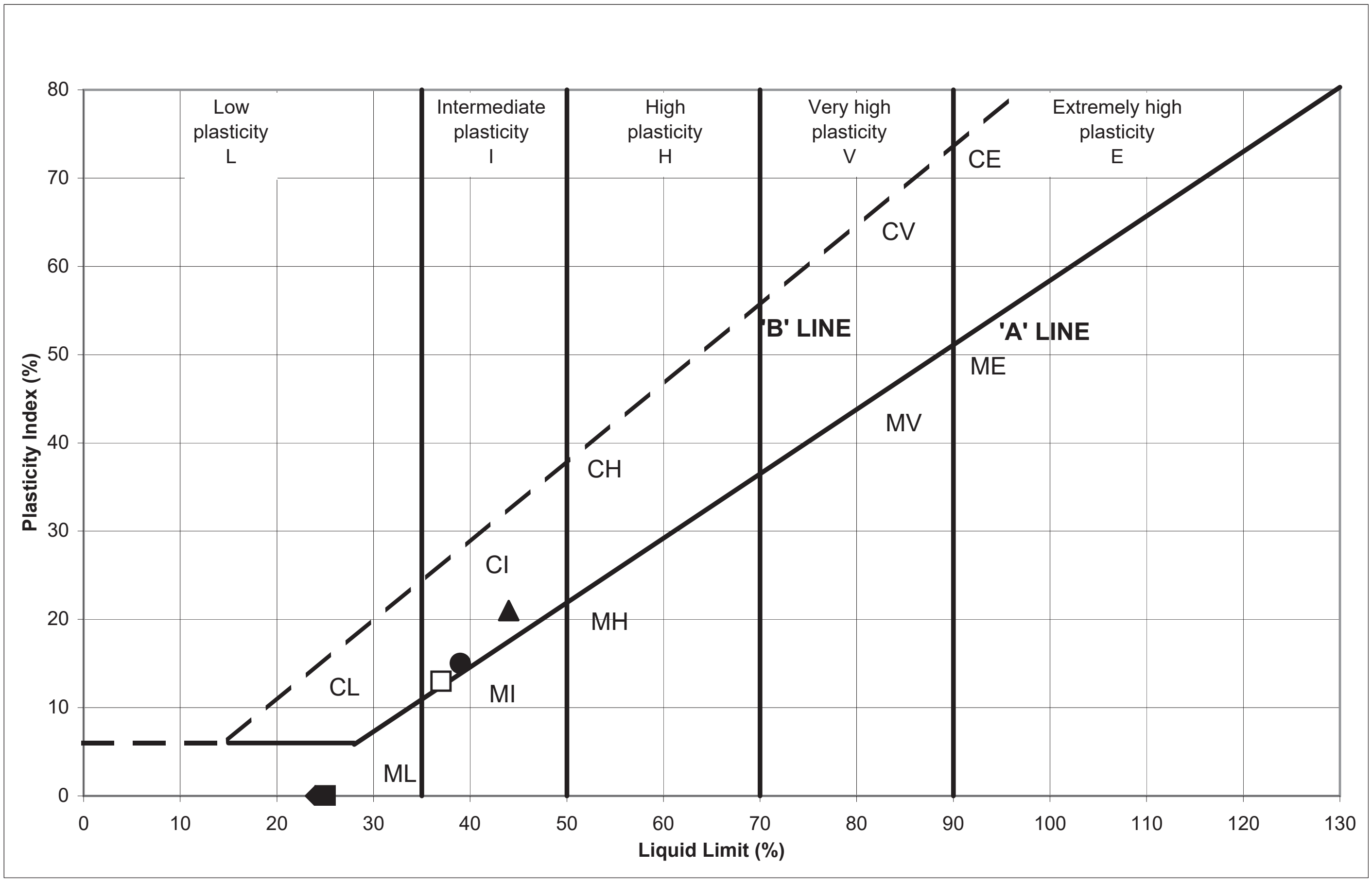
TRIAL PIT	SAMPLE	DEPTH (m)	SAMPLE DESCRIPTION
TP22	D1	1.30-1.40	Brown very silty fine to coarse SAND.
TP22	B1	1.30-1.40	Brown clayey very silty fine to coarse SAND and GRAVEL with cobbles.
TP22	B2	1.60-1.70	Brown slightly gravelly clayey very silty fine to coarse SAND. Gravel is fine to coarse.
TP23	D1	1.50-1.60	Reddish brown / grey slightly clayey very silty fine to coarse SAND.
TP40	D1	1.60-1.80	Brown slightly silty very sandy CLAY.
TP40	B1	2.00-2.10	Brown slightly clayey very silty fine to coarse SAND and GRAVEL.
TP41	D1	1.65-1.70	Brown very sandy clayey SILT.
TP41	B1	1.65-1.70	Brown very gravelly very sandy clayey SILT. Gravel is fine to coarse.
TP41	B2	1.80-1.90	Brown gravelly very clayey very silty fine to coarse SAND. Gravel is fine to coarse.
TP42	D1	1.80-2.00	Brown very clayey very sandy SILT.
TP42	B1	1.80-2.00	Brown gravelly very clayey very sandy SILT. Gravel is fine to coarse.
TP55	B1	1.40-1.50	Brown slightly silty fine to coarse SAND and GRAVEL with cobbles.
TP57	B1	1.50	Brown silty fine to coarse SAND and GRAVEL with cobbles.
TP70	B2	2.00	Light brown silty fine to coarse SAND and GRAVEL with cobbles.
TP91	B1	1.20	Light grey slightly silty sandy fine to coarse GRAVEL with cobbles.

SUMMARY OF SAMPLE DESCRIPTIONS

TRIAL PIT	SAMPLE	DEPTH (m)	MOISTURE CONTENT (%)
TP22	D1	1.30-1.40	11
TP22	B2	1.60-1.70	12
TP23	D1	1.50-1.60	9.2
TP40	D1	1.60-1.80	21
TP40	B1	2.00-2.10	5.3
TP41	D1	1.65-1.70	20
TP41	B1	1.65-1.70	12
TP41	B2	1.80-1.90	14
TP42	D1	1.80-2.00	20

Tested in accordance with BS 1377: Part 2: 1990: Clause 3

SUMMARY OF MOISTURE CONTENT TEST RESULTS

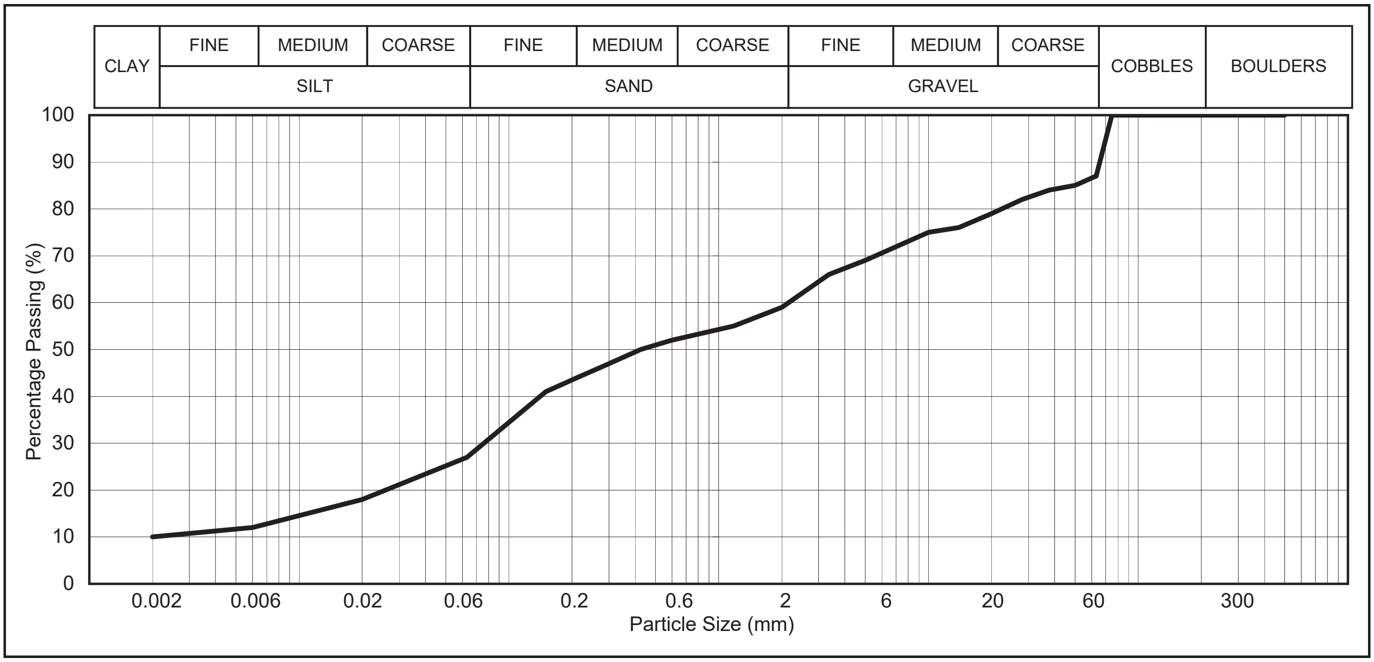


Symbol	Trial Pit	Sample	Depth	Moisture Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	% Passing 0.425mm Sieve	Remarks
■	TP22	D1	1.30-1.40	11	25	Non Plastic	Non Plastic	48	
◆	TP23	D1	1.50-1.60	9.2	24	Non Plastic	Non Plastic	55	
▲	TP40	D1	1.60-1.80	21	44	23	21	78	Clay with intermediate plasticity
●	TP41	D1	1.65-1.70	20	39	24	15	76	Clay with intermediate plasticity
□	TP42	D1	1.80-2.00	20	37	24	13	56	Clay with intermediate plasticity
◇									
△									
○									
✕									
✳									

All samples were tested in accordance with BS 1377 : Part 2 : 1990 Clause 4.3, 5.3 and 5.4.
All samples were washed on a 0.425mm test sieve prior to test.

SUMMARY OF ATTERBERG LIMITS TEST RESULTS

Borehole	TP22
Sample	B1
Depth (m)	1.30-1.40



SIEVING

Sieve Size (mm)	Percentage Passing (%)	Specification	
		Not Applicable	
		Lower %	Upper %
500.0	100	-	-
300.0	100	-	-
125.0	100	-	-
90.0	100	-	-
75.0	100	-	-
63.0	87	-	-
50.0	85	-	-
37.5	84	-	-
28.0	82	-	-
20.0	79	-	-
14.0	76	-	-
10.0	75	-	-
6.30	71	-	-
5.00	69	-	-
3.35	66	-	-
2.00	59	-	-
1.18	55	-	-
0.600	52	-	-
0.425	50	-	-
0.300	47	-	-
0.212	44	-	-
0.150	41	-	-
0.063	27	-	-

SEDIMENTATION

Particle Size (mm)	Percentage Passing (%)
0.020	18
0.006	12
0.002	10

GRADING CLASSIFICATION (SHW TABLE 6/2)

-

Grading classification proves the material has met the relevant grading requirements only. Further testing may be required to assess compliance with SHW.

PERCENTAGE SOIL TYPES

CLAY	SILT Ŧ	SAND	GRAVEL	COBBLES
10	17	32	28	13

UNIFORMITY COEFFICIENT (SHW TABLE 6/1 NOTE 5)

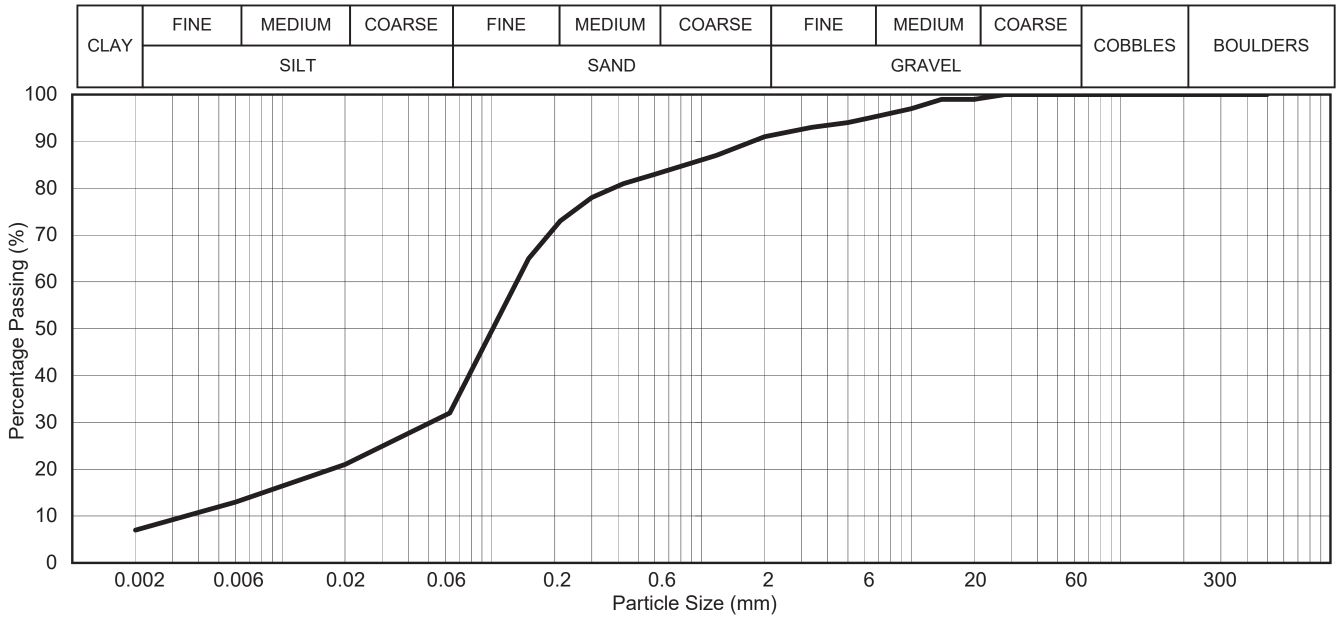
D10	D60	Specification
-	-	
UNIFORMITY COEFFICIENT	-	-

Remarks

Ŧ Where a sedimentation test was not carried out, this figure represents total fines, i.e., particles of diameter less than 63 microns

Sample does not meet minimum mass requirement for material type

Borehole	TP22
Sample	B2
Depth (m)	1.60-1.70



SIEVING

Sieve Size (mm)	Percentage Passing (%)	Specification Not Applicable	
		Lower %	Upper %
500.0	100	-	-
300.0	100	-	-
125.0	100	-	-
90.0	100	-	-
75.0	100	-	-
63.0	100	-	-
50.0	100	-	-
37.5	100	-	-
28.0	100	-	-
20.0	99	-	-
14.0	99	-	-
10.0	97	-	-
6.30	95	-	-
5.00	94	-	-
3.35	93	-	-
2.00	91	-	-
1.18	87	-	-
0.600	83	-	-
0.425	81	-	-
0.300	78	-	-
0.212	73	-	-
0.150	65	-	-
0.063	32	-	-

SEDIMENTATION

Particle Size (mm)	Percentage Passing (%)
0.020	21
0.006	13
0.002	7

GRADING CLASSIFICATION (SHW TABLE 6/2)

-

Grading classification proves the material has met the relevant grading requirements only. Further testing may be required to assess compliance with SHW.

PERCENTAGE SOIL TYPES

CLAY	SILT Ŧ	SAND	GRAVEL	COBBLES
7	25	59	9	0

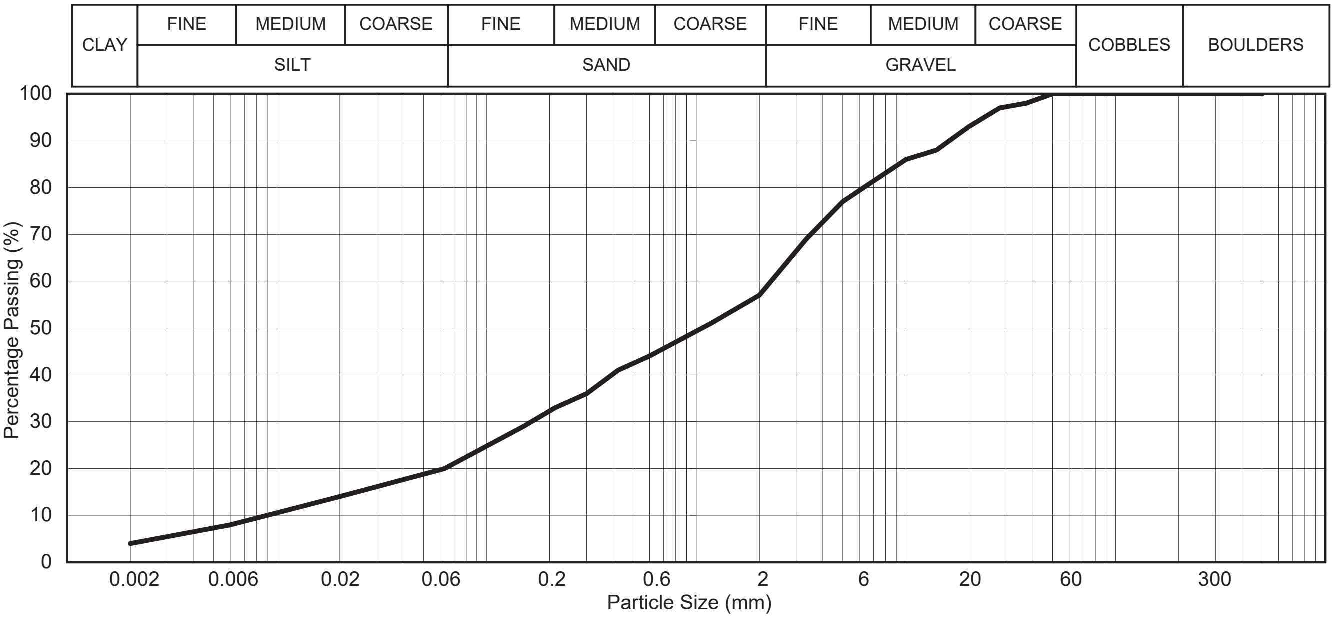
UNIFORMITY COEFFICIENT (SHW TABLE 6/1 NOTE 5)

D10	D60	Specification
-	-	
UNIFORMITY COEFFICIENT	-	-

Remarks

Ŧ Where a sedimentation test was not carried out, this figure represents total fines, i.e., particles of diameter less than 63 microns

Borehole	TP40
Sample	B1
Depth (m)	2.00-2.10



SIEVING

Sieve Size (mm)	Percentage Passing (%)	Specification Not Applicable Lower %	Upper %
500.0	100	-	-
300.0	100	-	-
125.0	100	-	-
90.0	100	-	-
75.0	100	-	-
63.0	100	-	-
50.0	100	-	-
37.5	98	-	-
28.0	97	-	-
20.0	93	-	-
14.0	88	-	-
10.0	86	-	-
6.30	80	-	-
5.00	77	-	-
3.35	69	-	-
2.00	57	-	-
1.18	51	-	-
0.600	44	-	-
0.425	41	-	-
0.300	36	-	-
0.212	33	-	-
0.150	29	-	-
0.063	20	-	-

SEDIMENTATION

Particle Size (mm)	Percentage Passing (%)
0.020	14
0.006	8
0.002	4

GRADING CLASSIFICATION (SHW TABLE 6/2)

-

Grading classification proves the material has met the relevant grading requirements only. Further testing may be required to assess compliance with SHW.

PERCENTAGE SOIL TYPES

CLAY	SILT Ŧ	SAND	GRAVEL	COBBLES
4	16	37	43	0

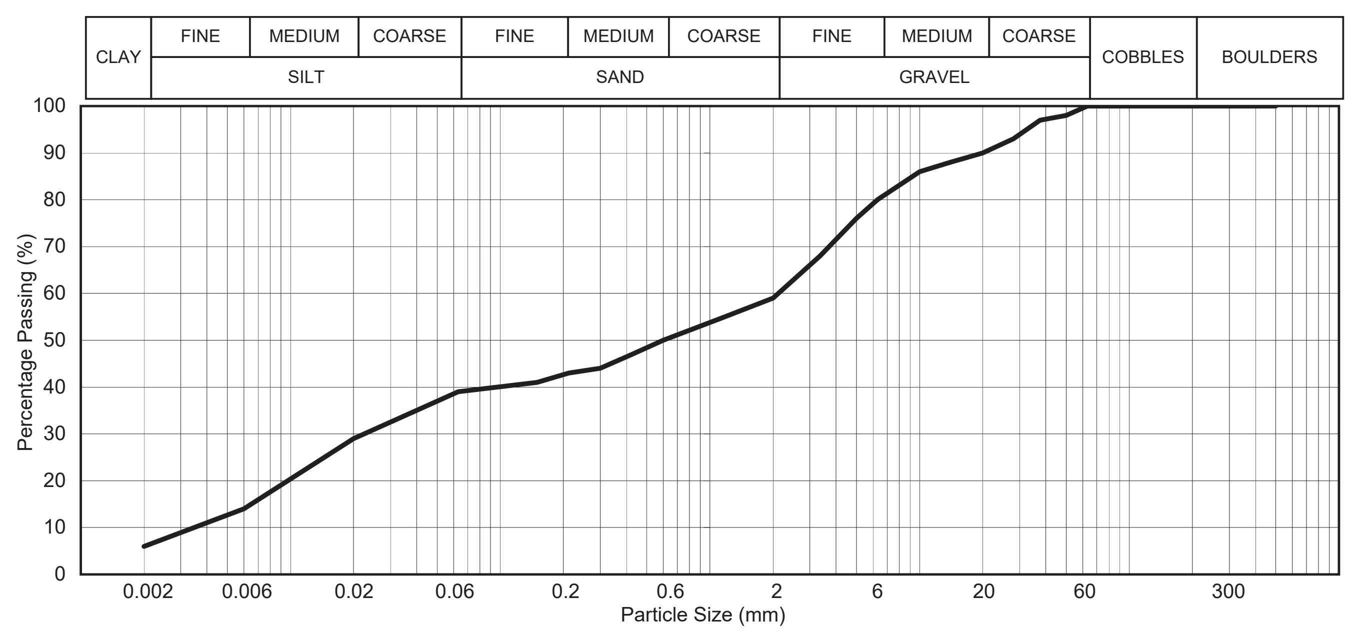
UNIFORMITY COEFFICIENT (SHW TABLE 6/1 NOTE 5)

D10	D60	Specification
-	-	
UNIFORMITY COEFFICIENT	-	-

Remarks

Ŧ Where a sedimentation test was not carried out, this figure represents total fines, i.e., particles of diameter less than 63 microns

Borehole	TP41
Sample	B1
Depth (m)	1.65-1.70



SIEVING

Sieve Size (mm)	Percentage Passing (%)	Specification Not Applicable	
		Lower %	Upper %
500.0	100	-	-
300.0	100	-	-
125.0	100	-	-
90.0	100	-	-
75.0	100	-	-
63.0	100	-	-
50.0	98	-	-
37.5	97	-	-
28.0	93	-	-
20.0	90	-	-
14.0	88	-	-
10.0	86	-	-
6.30	80	-	-
5.00	76	-	-
3.35	68	-	-
2.00	59	-	-
1.18	55	-	-
0.600	50	-	-
0.425	47	-	-
0.300	44	-	-
0.212	43	-	-
0.150	41	-	-
0.063	39	-	-

SEDIMENTATION

Particle Size (mm)	Percentage Passing (%)
0.020	29
0.006	14
0.002	6

GRADING CLASSIFICATION (SHW TABLE 6/2)

-

Grading classification proves the material has met the relevant grading requirements only. Further testing may be required to assess compliance with SHW.

PERCENTAGE SOIL TYPES

CLAY	SILT Ŧ	SAND	GRAVEL	COBBLES
6	33	20	41	0

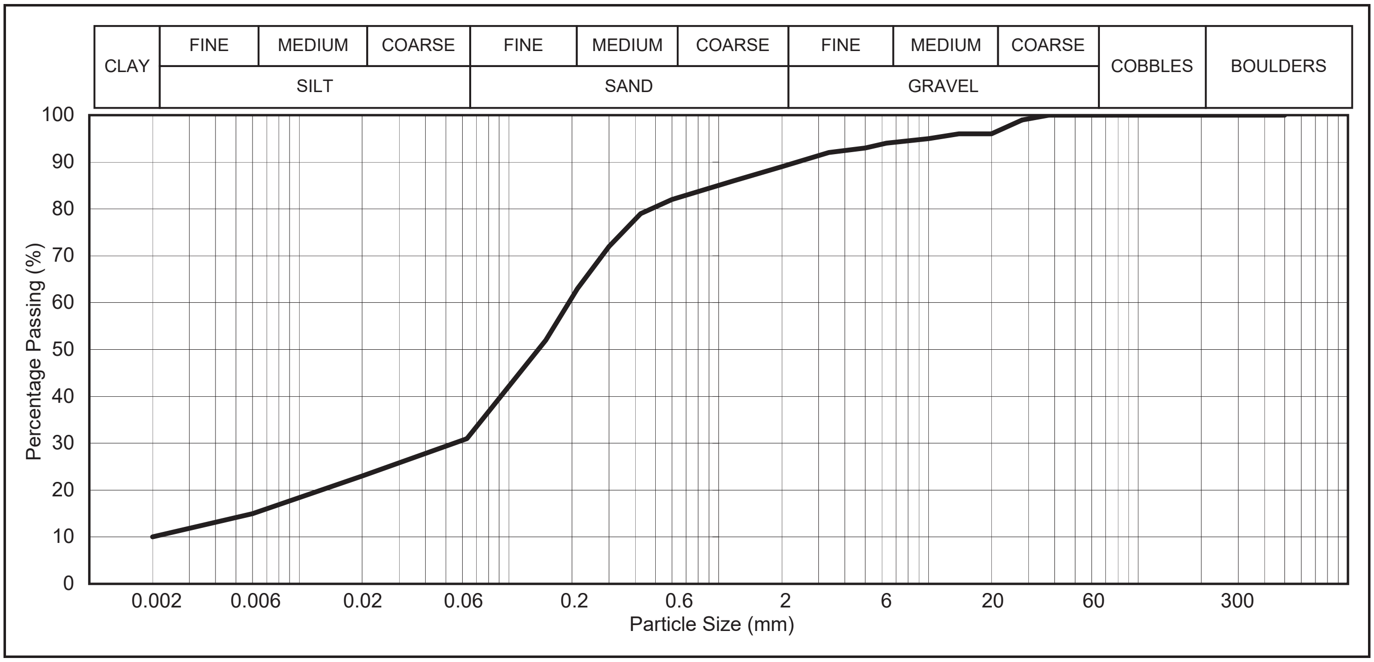
UNIFORMITY COEFFICIENT (SHW TABLE 6/1 NOTE 5)

D10	D60	Specification
-	-	
UNIFORMITY COEFFICIENT	-	-

Remarks

Ŧ Where a sedimentation test was not carried out, this figure represents total fines, i.e., particles of diameter less than 63 microns

Borehole	TP41
Sample	B2
Depth (m)	1.80-1.90



SIEVING				SEDIMENTATION	
Sieve Size (mm)	Percentage Passing (%)	Specification Not Applicable		Particle Size (mm)	Percentage Passing (%)
		Lower %	Upper %		
500.0	100	-	-	0.020	23
300.0	100	-	-	0.006	15
125.0	100	-	-	0.002	10
90.0	100	-	-		
75.0	100	-	-		
63.0	100	-	-		
50.0	100	-	-		
37.5	100	-	-		
28.0	99	-	-		
20.0	96	-	-		
14.0	96	-	-		
10.0	95	-	-		
6.30	94	-	-		
5.00	93	-	-		
3.35	92	-	-		
2.00	89	-	-		
1.18	86	-	-		
0.600	82	-	-		
0.425	79	-	-		
0.300	72	-	-		
0.212	63	-	-		
0.150	52	-	-		
0.063	31	-	-		

GRADING CLASSIFICATION (SHW TABLE 6/2)

-

Grading classification proves the material has met the relevant grading requirements only. Further testing may be required to assess compliance with SHW.

PERCENTAGE SOIL TYPES

CLAY	SILT Ŧ	SAND	GRAVEL	COBBLES
10	21	58	11	0

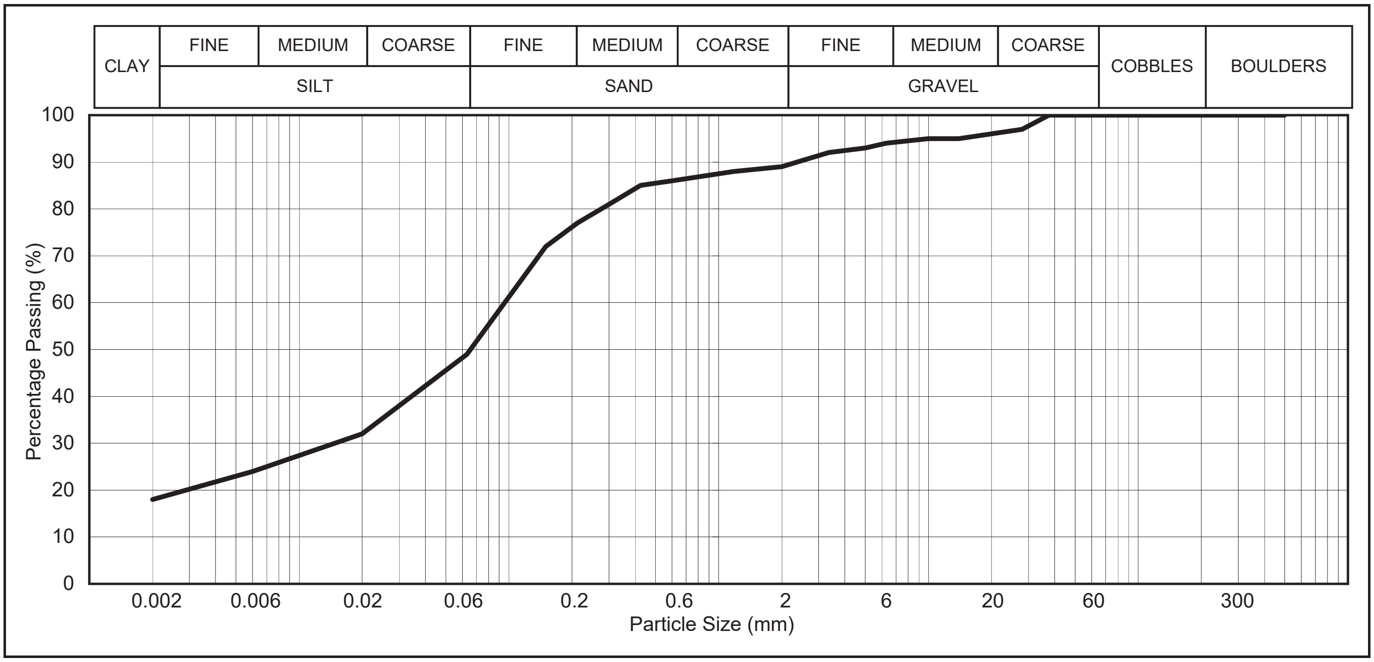
UNIFORMITY COEFFICIENT (SHW TABLE 6/1 NOTE 5)

D10	D60	Specification
-	-	
UNIFORMITY COEFFICIENT	-	-

Remarks

Ŧ Where a sedimentation test was not carried out, this figure represents total fines, i.e., particles of diameter less than 63 microns

Borehole	TP42
Sample	B1
Depth (m)	1.80-2.00



SIEVING			
Sieve Size (mm)	Percentage Passing (%)	Specification Not Applicable	
		Lower %	Upper %
500.0	100	-	-
300.0	100	-	-
125.0	100	-	-
90.0	100	-	-
75.0	100	-	-
63.0	100	-	-
50.0	100	-	-
37.5	100	-	-
28.0	97	-	-
20.0	96	-	-
14.0	95	-	-
10.0	95	-	-
6.30	94	-	-
5.00	93	-	-
3.35	92	-	-
2.00	89	-	-
1.18	88	-	-
0.600	86	-	-
0.425	85	-	-
0.300	81	-	-
0.212	77	-	-
0.150	72	-	-
0.063	49	-	-

SEDIMENTATION	
Particle Size (mm)	Percentage Passing (%)
0.020	32
0.006	24
0.002	18

GRADING CLASSIFICATION (SHW TABLE 6/2)

-

Grading classification proves the material has met the relevant grading requirements only. Further testing may be required to assess compliance with SHW.

PERCENTAGE SOIL TYPES

CLAY	SILT Ŧ	SAND	GRAVEL	COBBLES
18	31	40	11	0

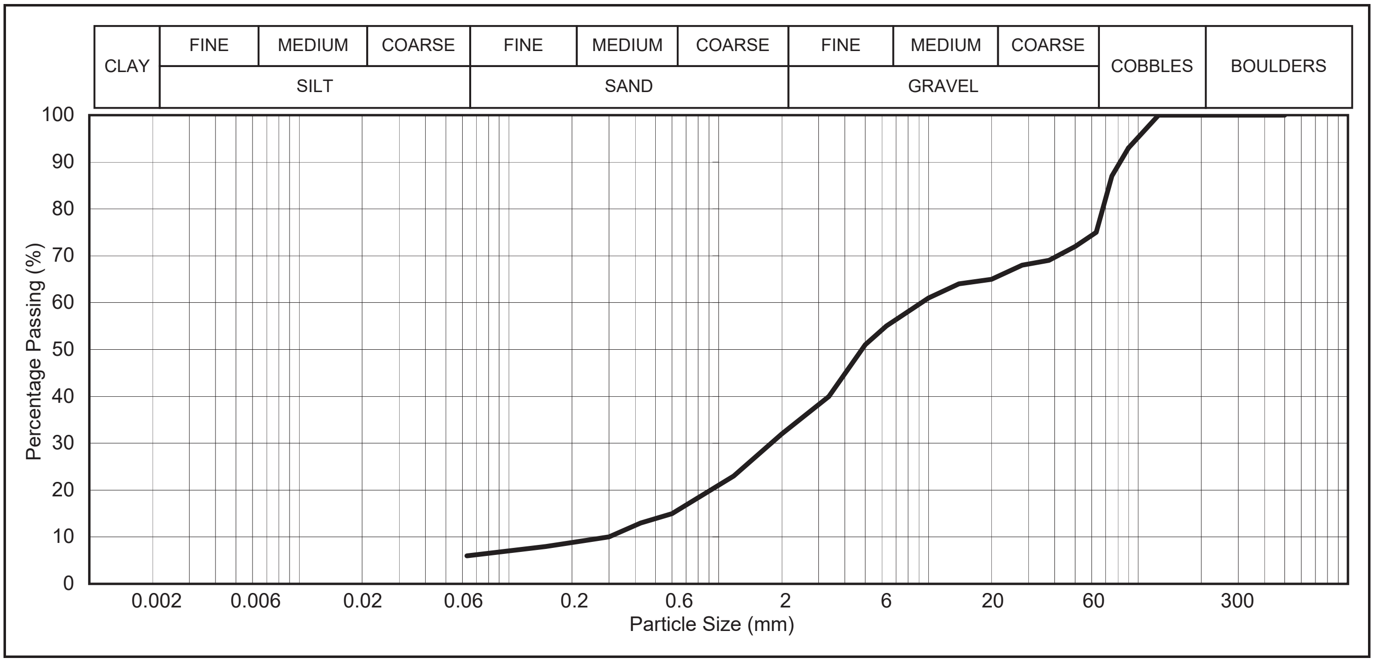
UNIFORMITY COEFFICIENT (SHW TABLE 6/1 NOTE 5)

D10	D60	Specification
-	-	
UNIFORMITY COEFFICIENT	-	-

Remarks

Ŧ Where a sedimentation test was not carried out, this figure represents total fines, i.e., particles of diameter less than 63 microns

Borehole	TP55
Sample	B1
Depth (m)	1.40-1.50



SIEVING

Sieve Size (mm)	Percentage Passing (%)	Specification Not Applicable	
		Lower %	Upper %
500.0	100	-	-
300.0	100	-	-
125.0	100	-	-
90.0	93	-	-
75.0	87	-	-
63.0	75	-	-
50.0	72	-	-
37.5	69	-	-
28.0	68	-	-
20.0	65	-	-
14.0	64	-	-
10.0	61	-	-
6.30	55	-	-
5.00	51	-	-
3.35	40	-	-
2.00	32	-	-
1.18	23	-	-
0.600	15	-	-
0.425	13	-	-
0.300	10	-	-
0.212	9	-	-
0.150	8	-	-
0.063	6	-	-

SEDIMENTATION

Particle Size (mm)	Percentage Passing (%)
0.020	
0.006	
0.002	

GRADING CLASSIFICATION (SHW TABLE 6/2)

-

Grading classification proves the material has met the relevant grading requirements only. Further testing may be required to assess compliance with SHW.

PERCENTAGE SOIL TYPES

CLAY	SILT Ŧ	SAND	GRAVEL	COBBLES
/	6	26	43	25

UNIFORMITY COEFFICIENT (SHW TABLE 6/1 NOTE 5)

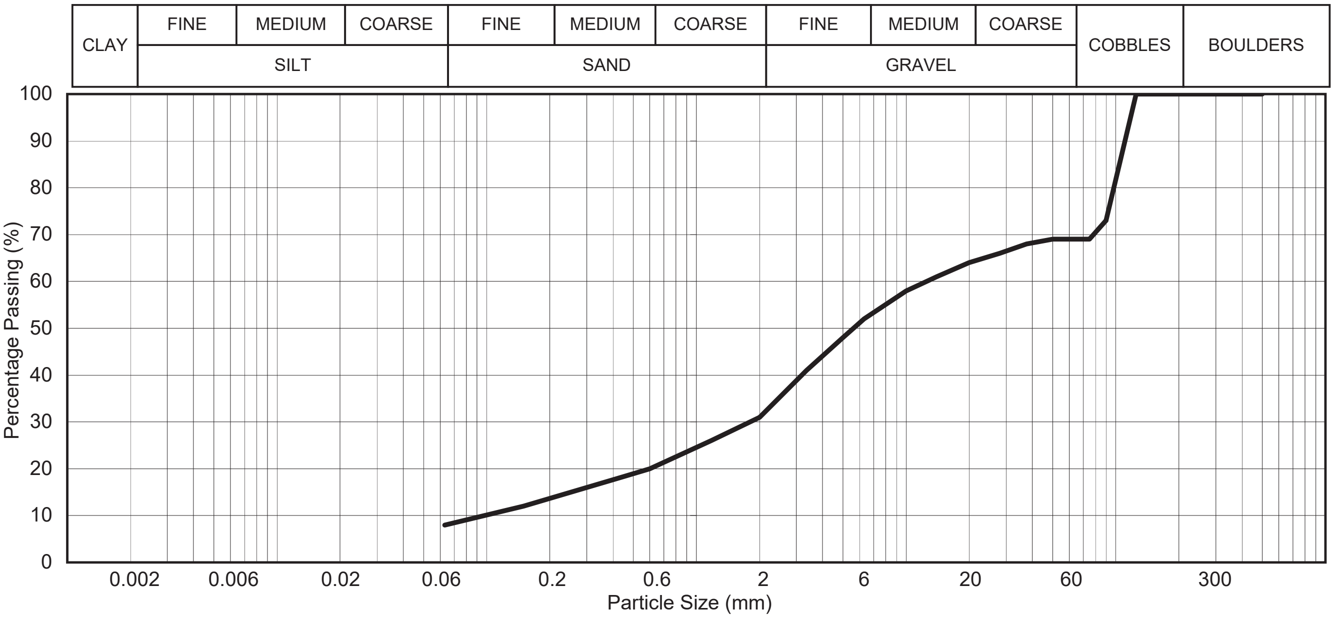
D10	D60	Specification
-	-	
UNIFORMITY COEFFICIENT	-	-

Remarks

Ŧ Where a sedimentation test was not carried out, this figure represents total fines, i.e., particles of diameter less than 63 microns

Sample does not meet minimum mass requirement for material type

Borehole	TP57
Sample	B1
Depth (m)	1.50



SIEVING

Sieve Size (mm)	Percentage Passing (%)	Specification Not Applicable	
		Lower %	Upper %
500.0	100	-	-
300.0	100	-	-
125.0	100	-	-
90.0	73	-	-
75.0	69	-	-
63.0	69	-	-
50.0	69	-	-
37.5	68	-	-
28.0	66	-	-
20.0	64	-	-
14.0	61	-	-
10.0	58	-	-
6.30	52	-	-
5.00	48	-	-
3.35	41	-	-
2.00	31	-	-
1.18	26	-	-
0.600	20	-	-
0.425	18	-	-
0.300	16	-	-
0.212	14	-	-
0.150	12	-	-
0.063	8	-	-

SEDIMENTATION

Particle Size (mm)	Percentage Passing (%)
0.020	
0.006	
0.002	

GRADING CLASSIFICATION (SHW TABLE 6/2)

-

Grading classification proves the material has met the relevant grading requirements only. Further testing may be required to assess compliance with SHW.

PERCENTAGE SOIL TYPES

CLAY	**SILT Ŧ**	**SAND**	**GRAVEL**	**COBBLES**
/	8	23	38	31

UNIFORMITY COEFFICIENT (SHW TABLE 6/1 NOTE 5)

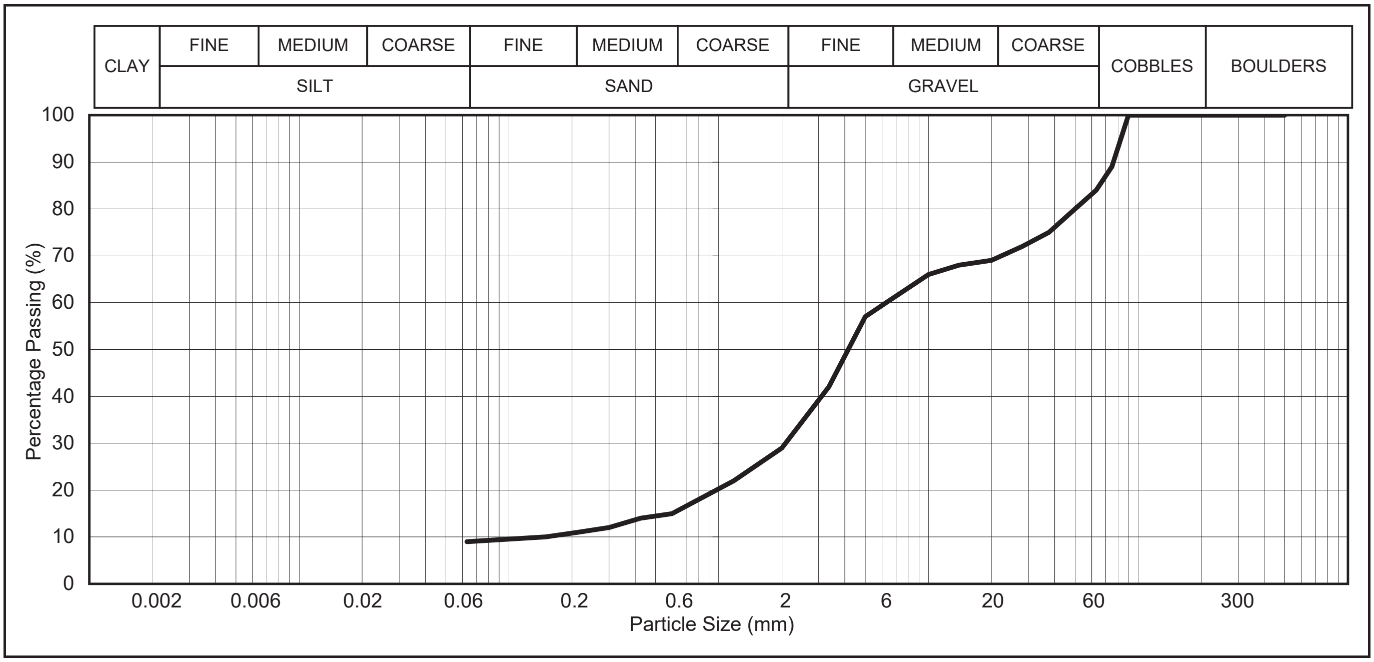
D10	**D60**	**Specification**
-	-	
UNIFORMITY COEFFICIENT	-	-

Remarks

Ŧ Where a sedimentation test was not carried out, this figure represents total fines, i.e., particles of diameter less than 63 microns

Sample does not meet minimum mass requirement for material type

Borehole	TP70
Sample	B2
Depth (m)	2.00



SIEVING				SEDIMENTATION	
Sieve Size (mm)	Percentage Passing (%)	Specification Not Applicable		Particle Size (mm)	Percentage Passing (%)
		Lower %	Upper %		
500.0	100	-	-	0.020	
300.0	100	-	-	0.006	
125.0	100	-	-	0.002	
90.0	100	-	-		
75.0	89	-	-		
63.0	84	-	-		
50.0	80	-	-		
37.5	75	-	-		
28.0	72	-	-		
20.0	69	-	-		
14.0	68	-	-		
10.0	66	-	-		
6.30	60	-	-		
5.00	57	-	-		
3.35	42	-	-		
2.00	29	-	-		
1.18	22	-	-		
0.600	15	-	-		
0.425	14	-	-		
0.300	12	-	-		
0.212	11	-	-		
0.150	10	-	-		
0.063	9	-	-		

GRADING CLASSIFICATION (SHW TABLE 6/2)

-

Grading classification proves the material has met the relevant grading requirements only. Further testing may be required to assess compliance with SHW.

PERCENTAGE SOIL TYPES

CLAY	SILT Ŧ	SAND	GRAVEL	COBBLES
/	9	20	55	16

UNIFORMITY COEFFICIENT (SHW TABLE 6/1 NOTE 5)

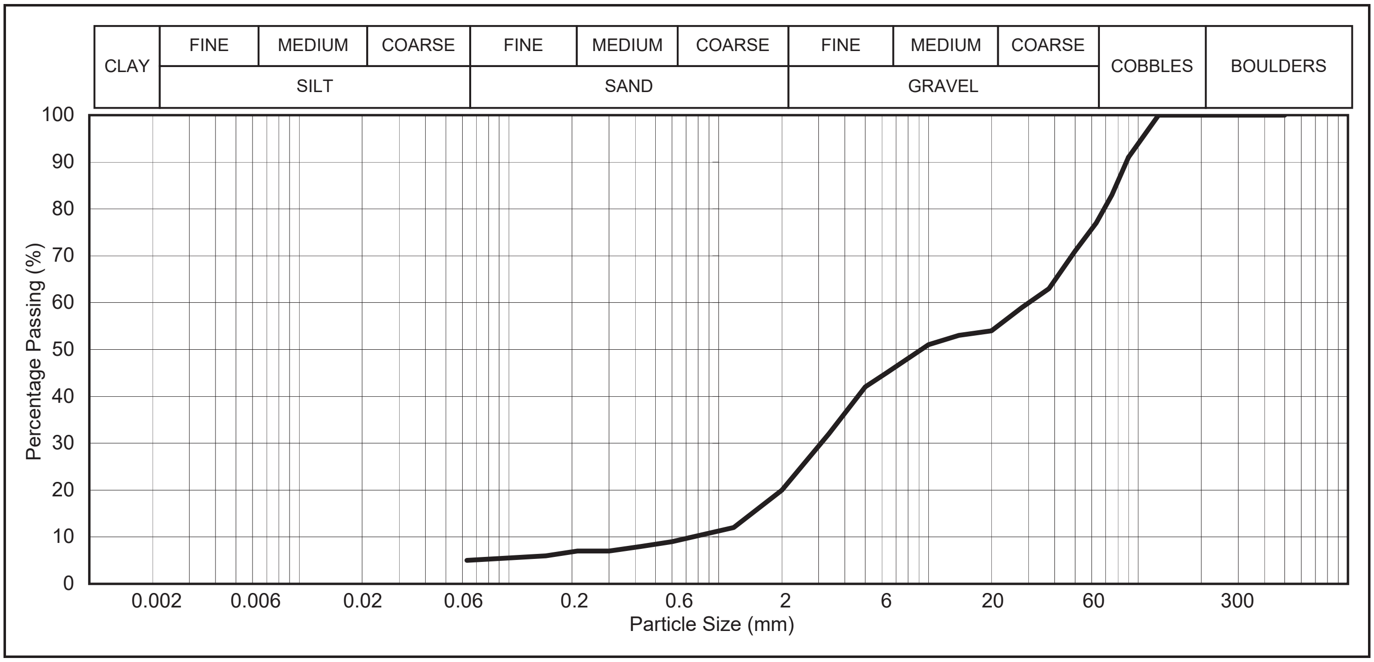
D10	D60	Specification
-	-	
UNIFORMITY COEFFICIENT	-	-

Remarks

Ŧ Where a sedimentation test was not carried out, this figure represents total fines, i.e., particles of diameter less than 63 microns

Sample does not meet minimum mass requirement for material type

Borehole	TP91
Sample	B1
Depth (m)	1.20



SIEVING				SEDIMENTATION	
Sieve Size (mm)	Percentage Passing (%)	Specification Not Applicable		Particle Size (mm)	Percentage Passing (%)
		Lower %	Upper %		
500.0	100	-	-	0.020	
300.0	100	-	-	0.006	
125.0	100	-	-	0.002	
90.0	91	-	-		
75.0	83	-	-		
63.0	77	-	-		
50.0	71	-	-		
37.5	63	-	-		
28.0	59	-	-		
20.0	54	-	-		
14.0	53	-	-		
10.0	51	-	-		
6.30	45	-	-		
5.00	42	-	-		
3.35	32	-	-		
2.00	20	-	-		
1.18	12	-	-		
0.600	9	-	-		
0.425	8	-	-		
0.300	7	-	-		
0.212	7	-	-		
0.150	6	-	-		
0.063	5	-	-		

GRADING CLASSIFICATION (SHW TABLE 6/2)

-

Grading classification proves the material has met the relevant grading requirements only. Further testing may be required to assess compliance with SHW.

PERCENTAGE SOIL TYPES

CLAY	SILT Ŧ	SAND	GRAVEL	COBBLES
/	5	15	57	23

UNIFORMITY COEFFICIENT (SHW TABLE 6/1 NOTE 5)

D10	D60	Specification
-	-	
UNIFORMITY COEFFICIENT	-	-

Remarks

Ŧ Where a sedimentation test was not carried out, this figure represents total fines, i.e., particles of diameter less than 63 microns

Sample does not meet minimum mass requirement for material type

SIEVE ANALYSIS - BS 1377 : Part 2 : 1990 : CLAUSE 9.2

Borehole	TP22
Sample	B2
Depth (m)	1.60-1.70

Specimen Details

Particle Density (assumed)	(Mg/m3)	2.65

Specimen Number		**1**	**2**	**3**
Length	mm	100.02	100.02	100.02
Width	mm	100.11	100.11	100.11
Height	mm	24.88	24.90	24.93
Initial Moisture Content	%	12	12	12
Initial Bulk Density	Mg/m3	2.08	2.08	2.07
Initial Dry Density	Mg/m3	1.86	1.86	1.86
Optimum Moisture Content	%	-		
Maximum Dry Density	Mg/m3	-		

Shearing Stage

Normal Pressure	kPa	70	140	280

Peak Conditions

Rate of horizontal displacement	mm/min	0.125	0.125	0.125
Peak shear stress	kPa	51.633	102.267	204.634
Horizontal displacement at peak	mm	2.21	2.89	3.91

Residual Conditions

Rate of horizontal displacement	mm/min	-	-	-
Residual shear stress	kPa	-	-	-
Final cumulative displacement	mm	-	-	-
Total traverses		-	-	-
Method of reversal		-	-	-

Final Moisture Content	%	15	15	15

Shear Strength Parameters

Peak Condition

Apparent Cohesion	kPa	0.4
Angle of Shearing Resistance	o	36.0

Residual Condition

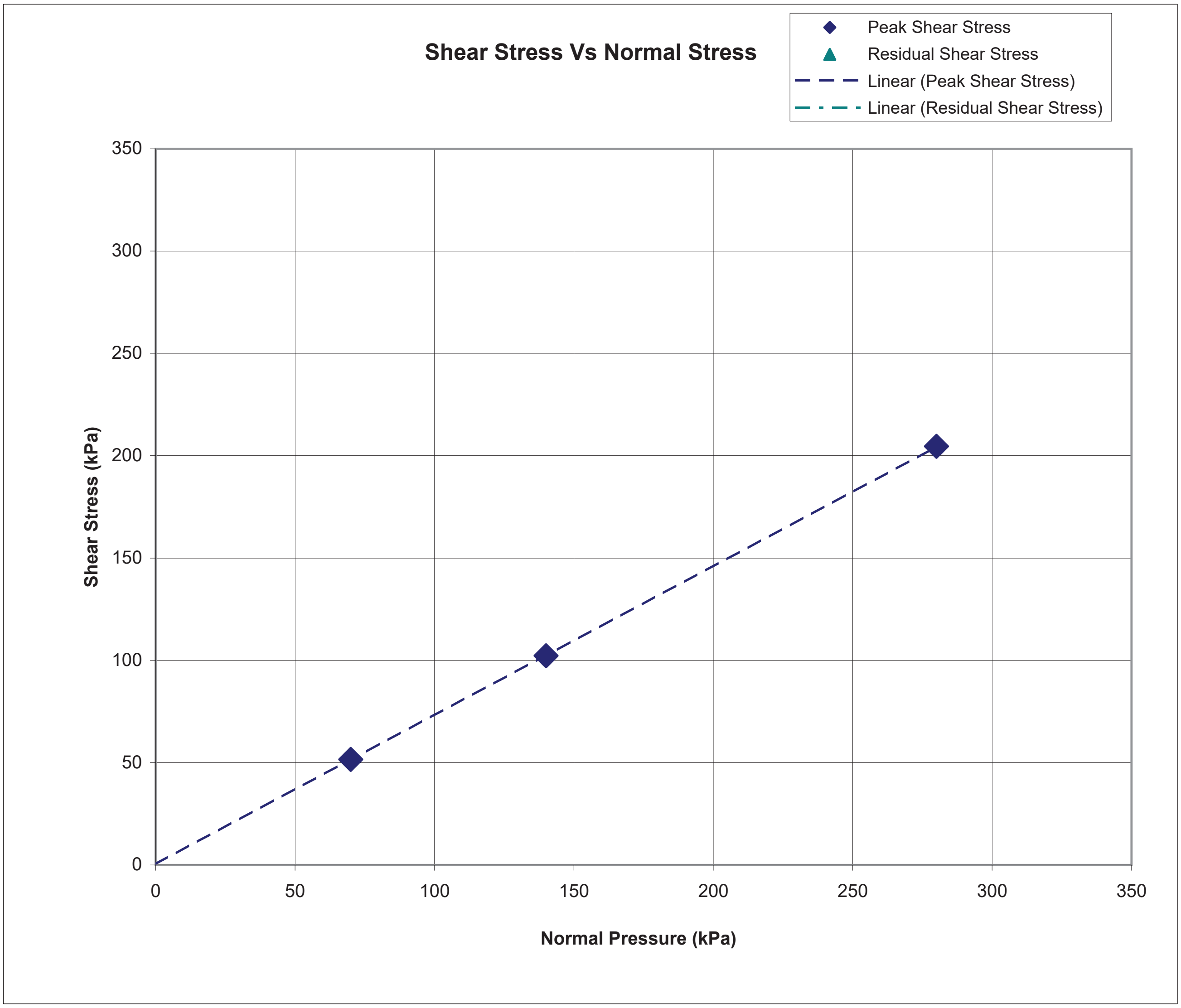
Apparent Cohesion	kPa	-
Angle of Shearing Resistance	o	-

Test Notes

Preparation - <2mm material prepared in accordance with BS 1377 : Part 7 : 1990 : Clause 4.4.3
Test condition - Submerged
Test specimen remoulded at natural moisture content using a 2.5kg rammer

DETERMINATION OF SHEAR STRENGTH BY DIRECT SHEAR - SMALL SHEARBOX APPARATUS
Tested in accordance with BS 1377 : Part 7 : 1990 : Clause 4.5.4

Borehole	TP22
Sample	B2
Depth (m)	1.60-1.70



Peak Conditions

Apparent Cohesion	kPa	0.4
Angle of Shearing Resistance	o	36.0

Normal Pressure	kPa	70	140	280
Peak shear stress	kPa	51.633	102.267	204.634

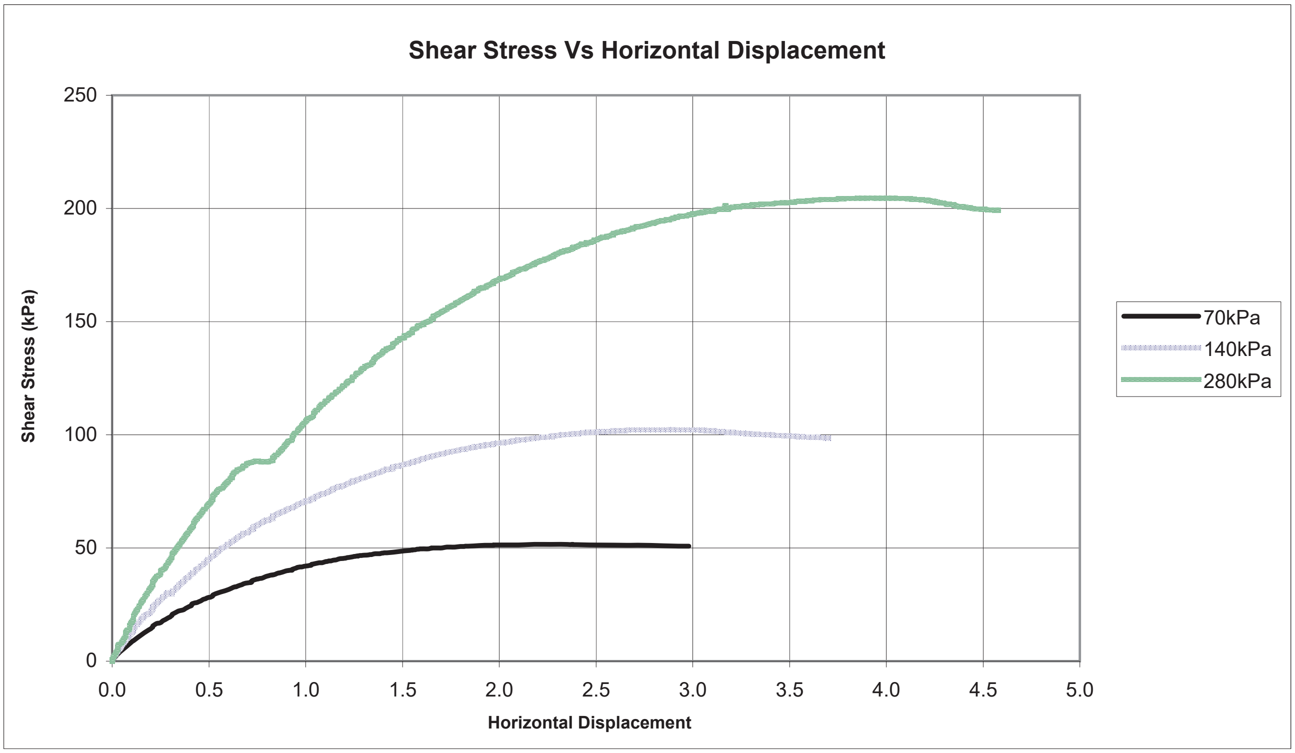
Residual Conditions

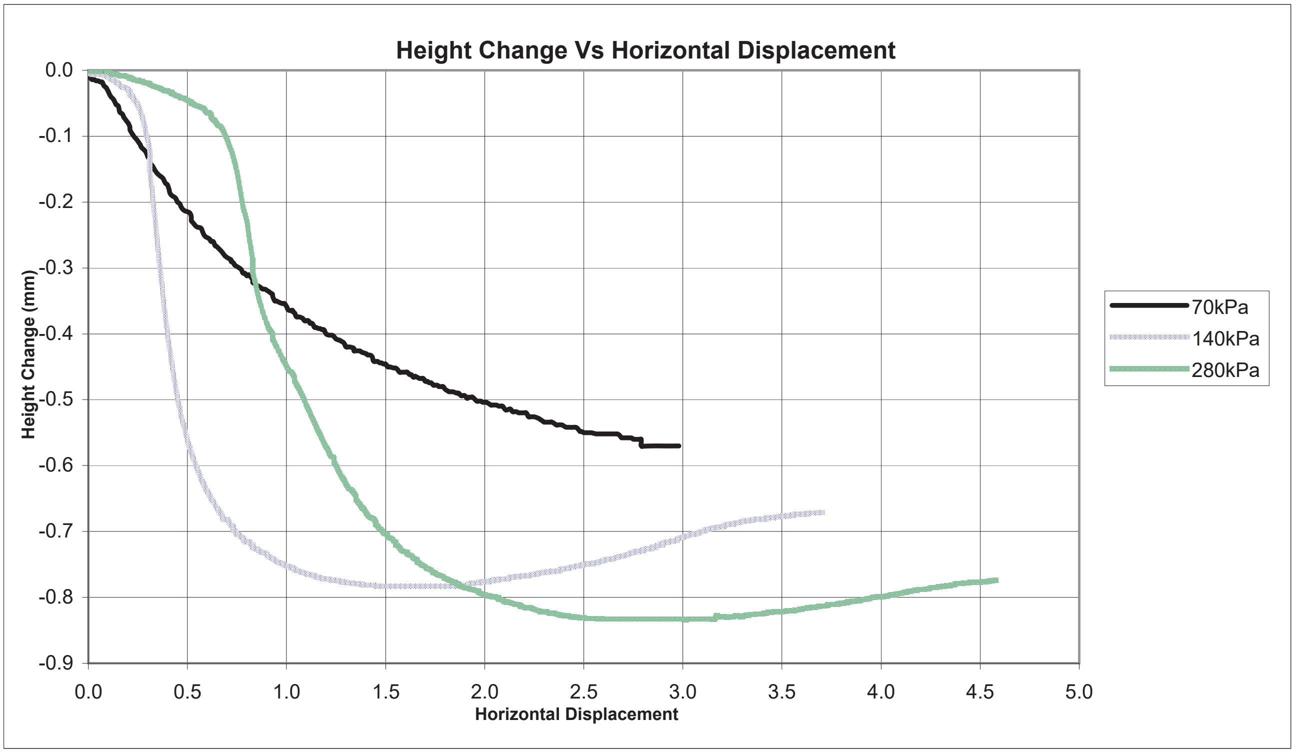
Apparent Cohesion	kPa	-
Angle of Shearing Resistance	o	-

Normal Pressure	kPa	-	-	-
Residual shear stress	kPa	-	-	-

DETERMINATION OF SHEAR STRENGTH BY DIRECT SHEAR - SMALL SHEARBOX APPARATUS
Tested in accordance with BS 1377 : Part 7 : 1990 : Clause 4.5.4

Borehole	TP22
Sample	B2
Depth (m)	1.60-1.70





DETERMINATION OF SHEAR STRENGTH BY DIRECT SHEAR - SMALL SHEARBOX APPARATUS
Tested in accordance with BS 1377 : Part 7 : 1990 : Clause 4.5.4

Borehole	TP40
Sample	B1
Depth (m)	2.00-2.10

Specimen Details

Particle Density (assumed)	(Mg/m3)	2.65

Specimen Number		**1**	**2**	**3**
Length	mm	100.02	100.02	100.02
Width	mm	100.11	100.11	100.11
Height	mm	25.20	25.30	25.30
Initial Moisture Content	%	5.5	5.4	5.4
Initial Bulk Density	Mg/m3	1.84	1.83	1.83
Initial Dry Density	Mg/m3	1.74	1.73	1.73
Optimum Moisture Content	%	-		
Maximum Dry Density	Mg/m3	-		

Shearing Stage

Normal Pressure	kPa	100	200	400

Peak Conditions

Rate of horizontal displacement	mm/min	0.125	0.125	0.125
Peak shear stress	kPa	68.211	140.218	277.140
Horizontal displacement at peak	mm	5.68	9.23	9.53

Residual Conditions

Rate of horizontal displacement	mm/min	-	-	-
Residual shear stress	kPa	-	-	-
Final cumulative displacement	mm	-	-	-
Total traverses		-	-	-
Method of reversal		-	-	-

Final Moisture Content	%	16	15	15

Shear Strength Parameters

Peak Condition

Apparent Cohesion	kPa	0.0
Angle of Shearing Resistance	°	35.0

Residual Condition

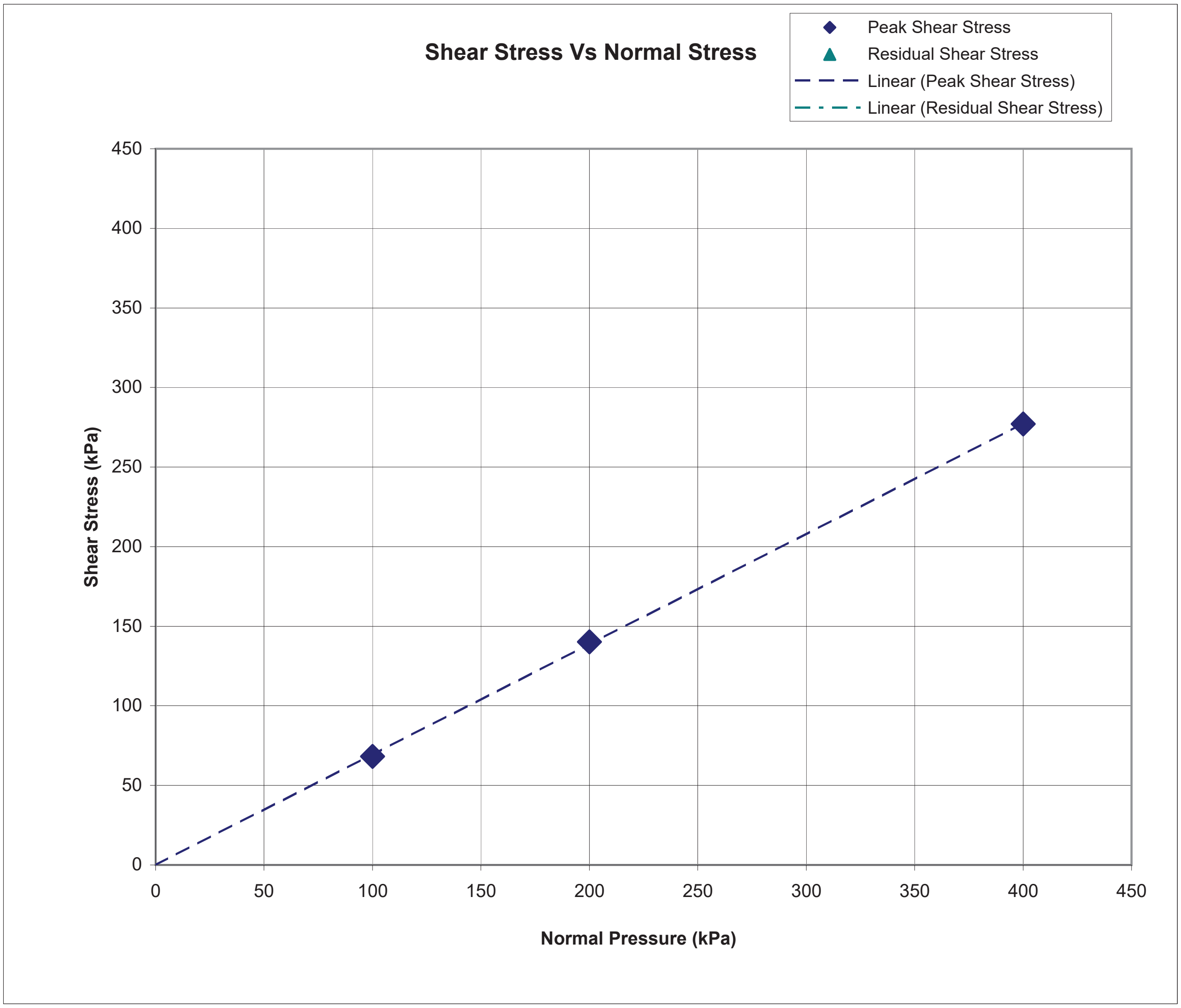
Apparent Cohesion	kPa	-
Angle of Shearing Resistance	°	-

Test Notes
Preparation - <2mm material prepared in accordance with BS 1377 : Part 7 : 1990 : Clause 4.4.3
Test condition - Submerged
Test specimen remoulded at natural moisture content using a 2.5kg rammer.

DETERMINATION OF SHEAR STRENGTH BY DIRECT SHEAR - SMALL SHEARBOX APPARATUS

Tested in accordance with BS 1377 : Part 7 : 1990 : Clause 4.5.4

Borehole	TP40
Sample	B1
Depth (m)	2.00-2.10



Peak Conditions

Apparent Cohesion	kPa	0.0
Angle of Shearing Resistance	o	35.0

Normal Pressure	kPa	100	200	400
Peak shear stress	kPa	68.211	140.218	277.140

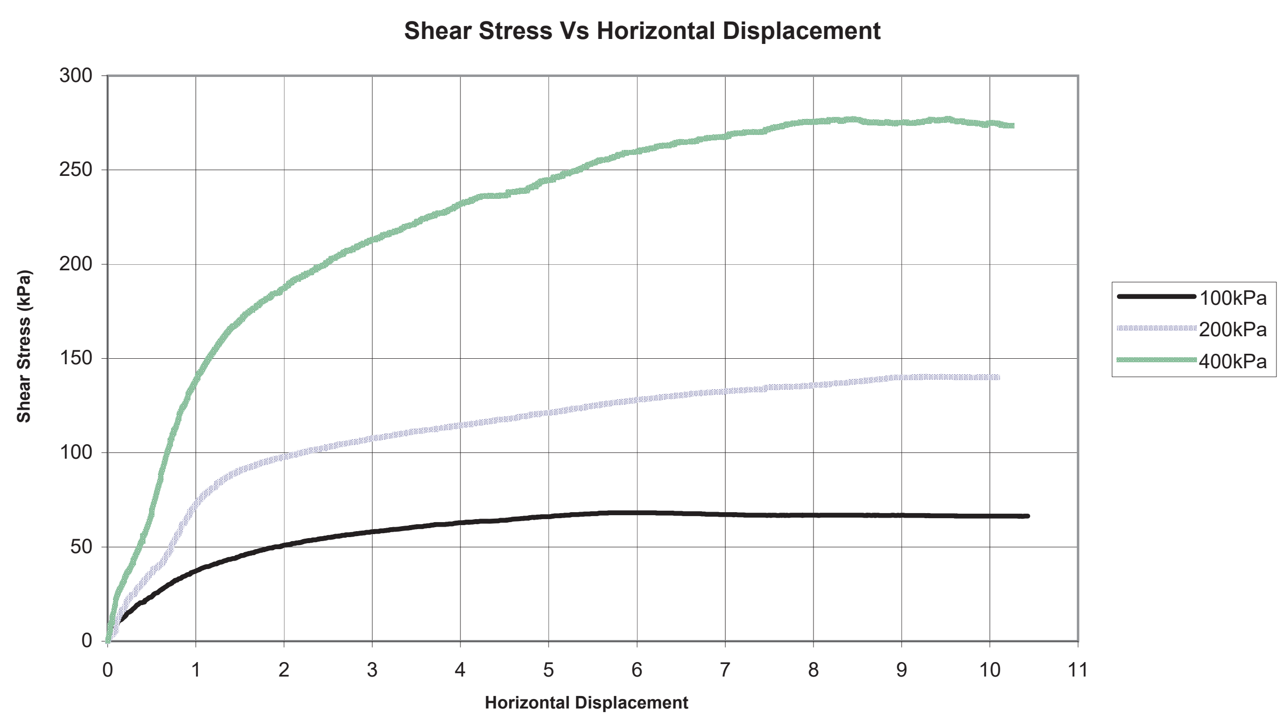
Residual Conditions

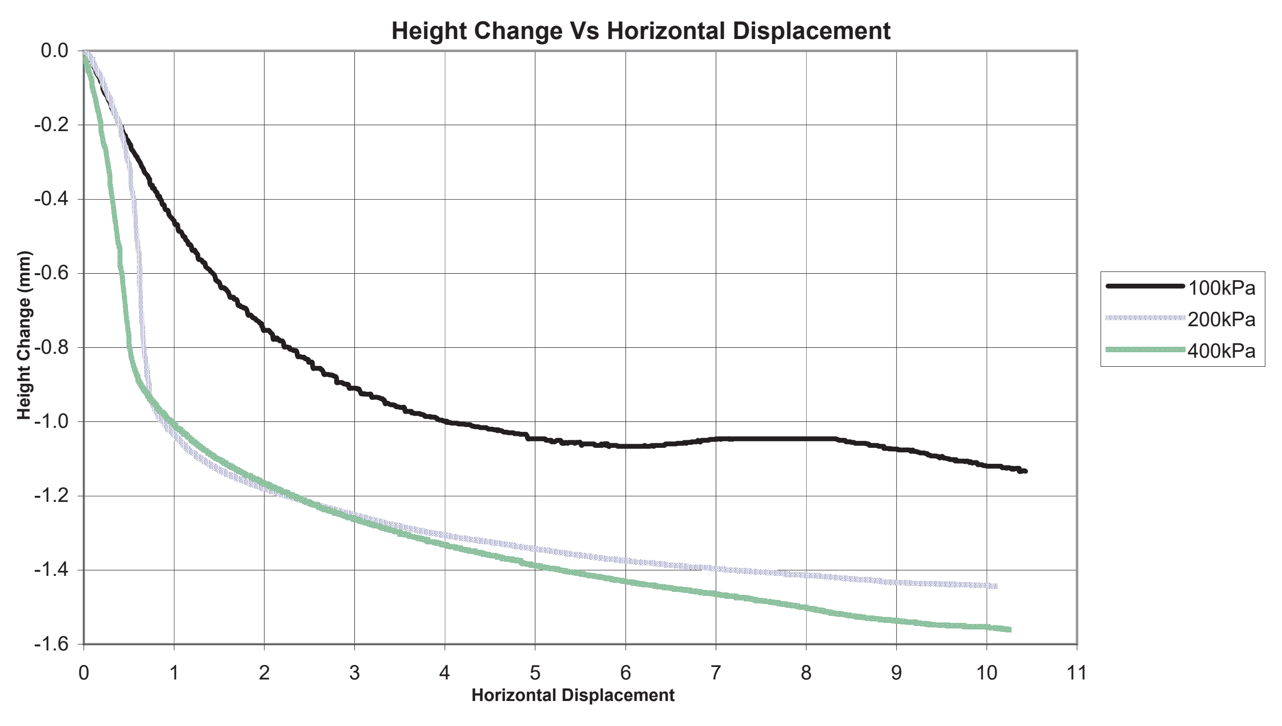
Apparent Cohesion	kPa	-
Angle of Shearing Resistance	o	-

Normal Pressure	kPa	-	-	-
Residual shear stress	kPa	-	-	-

DETERMINATION OF SHEAR STRENGTH BY DIRECT SHEAR - SMALL SHEARBOX APPARATUS
Tested in accordance with BS 1377 : Part 7 : 1990 : Clause 4.5.4

Borehole	TP40
Sample	B1
Depth (m)	2.00-2.10





DETERMINATION OF SHEAR STRENGTH BY DIRECT SHEAR - SMALL SHEARBOX APPARATUS

Tested in accordance with BS 1377 : Part 7 : 1990 : Clause 4.5.4

Borehole	TP41
Sample	B1
Depth (m)	1.65-1.70

Specimen Details

Particle Density (assumed)	(Mg/m3)	2.65

Specimen Number		**1**	**2**	**3**
Length	mm	100.02	100.02	100.02
Width	mm	100.11	100.11	100.11
Height	mm	25.00	24.98	24.98
Initial Moisture Content	%	12	12	12
Initial Bulk Density	Mg/m3	1.88	1.88	1.88
Initial Dry Density	Mg/m3	1.67	1.67	1.67
Optimum Moisture Content	%	-		
Maximum Dry Density	Mg/m3	-		

Shearing Stage

Normal Pressure	kPa	100	200	400

Peak Conditions

Rate of horizontal displacement	mm/min	0.125	0.125	0.125
Peak shear stress	kPa	63.118	127.934	253.171
Horizontal displacement at peak	mm	5.52	8.44	7.40

Residual Conditions

Rate of horizontal displacement	mm/min	-	-	-
Residual shear stress	kPa	-	-	-
Final cumulative displacement	mm	-	-	-
Total traverses		-	-	-
Method of reversal		-	-	-

Final Moisture Content	%	14	13	13

Shear Strength Parameters

Peak Condition

Apparent Cohesion	kPa	0.5
Angle of Shearing Resistance	o	32.5

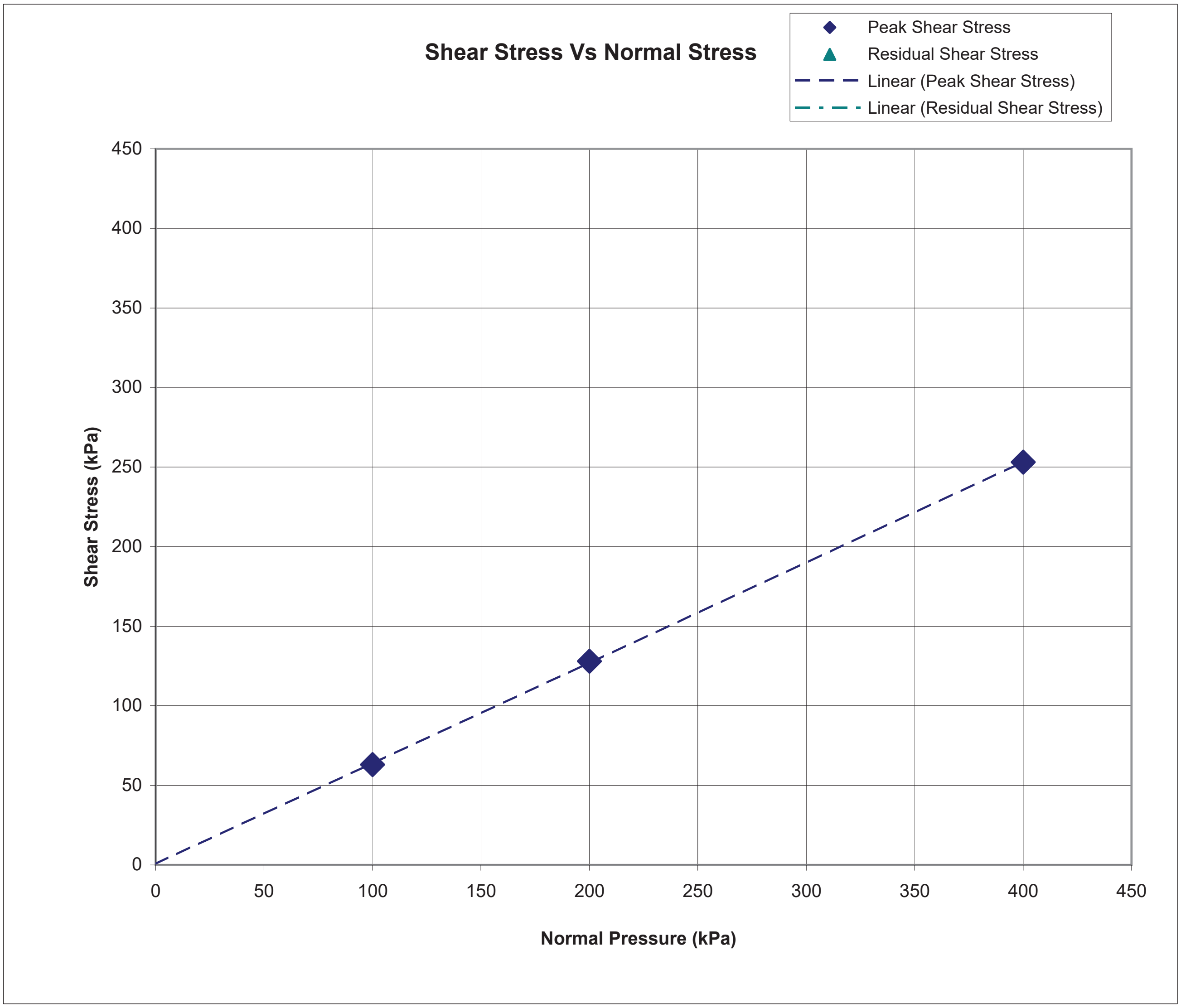
Residual Condition

Apparent Cohesion	kPa	-
Angle of Shearing Resistance	o	-

Test Notes
Preparation - <2mm material prepared in accordance with BS 1377 : Part 7 : 1990 : Clause 4.4.3
Test condition - Submerged
Test specimen remoulded at natural moisture content using a 2.5kg rammer

DETERMINATION OF SHEAR STRENGTH BY DIRECT SHEAR - SMALL SHEARBOX APPARATUS
Tested in accordance with BS 1377 : Part 7 : 1990 : Clause 4.5.4

Borehole	TP41
Sample	B1
Depth (m)	1.65-1.70



Peak Conditions

Apparent Cohesion	kPa	0.5		
Angle of Shearing Resistance	o	32.5		

Normal Pressure	kPa	100	200	400
Peak shear stress	kPa	63.118	127.934	253.171

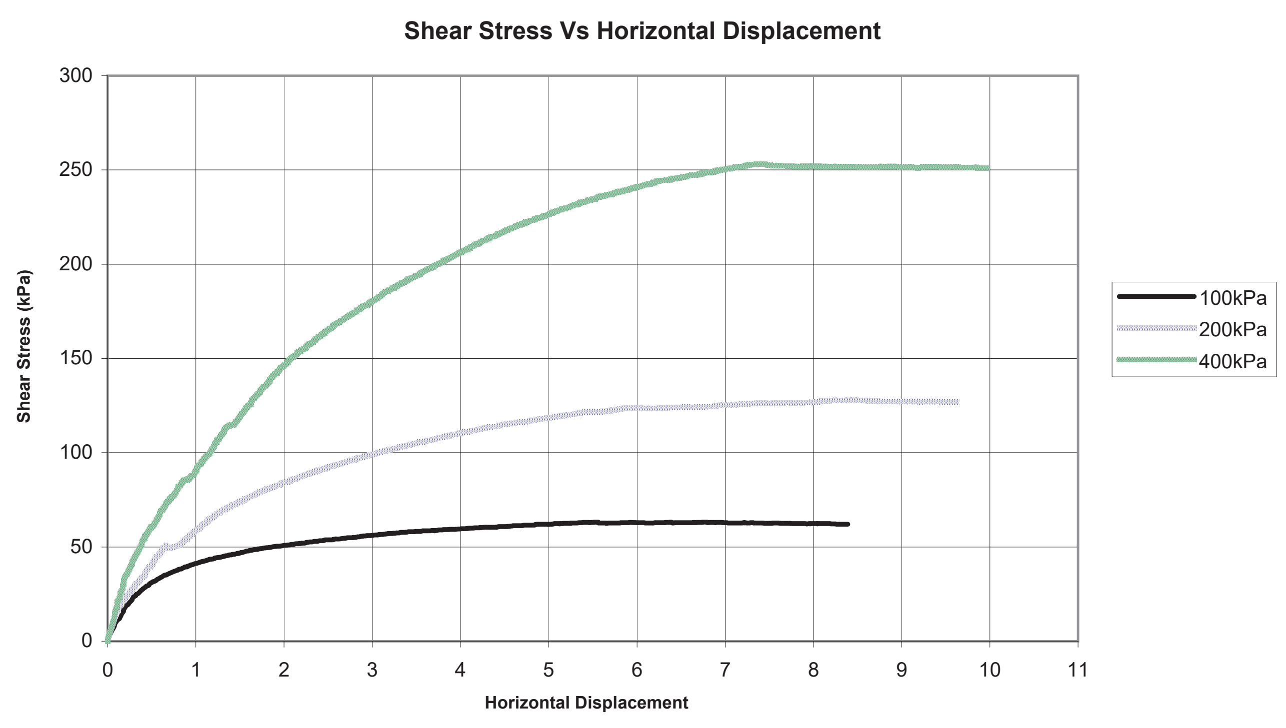
Residual Conditions

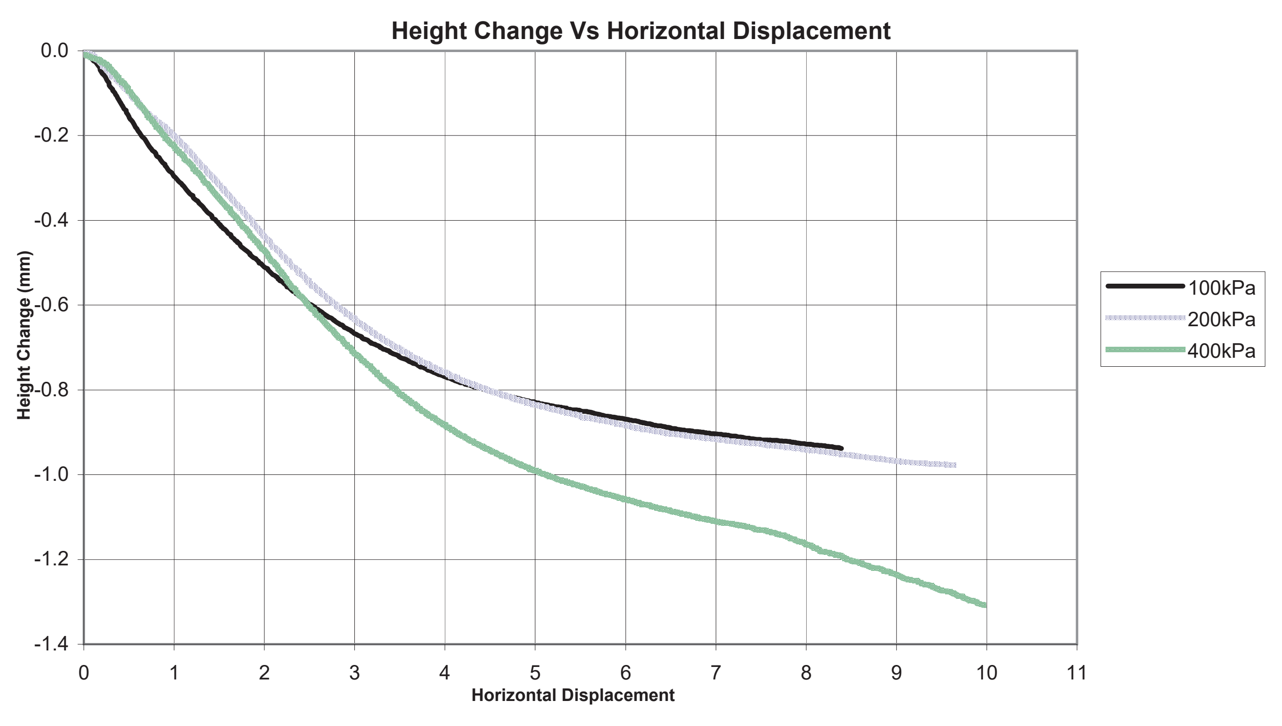
Apparent Cohesion	kPa	-		
Angle of Shearing Resistance	o	-		

Normal Pressure	kPa	-	-	-
Residual shear stress	kPa	-	-	-

DETERMINATION OF SHEAR STRENGTH BY DIRECT SHEAR - SMALL SHEARBOX APPARATUS
Tested in accordance with BS 1377 : Part 7 : 1990 : Clause 4.5.4

Borehole	TP41
Sample	B1
Depth (m)	1.65-1.70





DETERMINATION OF SHEAR STRENGTH BY DIRECT SHEAR - SMALL SHEARBOX APPARATUS
Tested in accordance with BS 1377 : Part 7 : 1990 : Clause 4.5.4

Borehole	TP41
Sample	B2
Depth (m)	1.80-1.90

Specimen Details

Particle Density (assumed)	(Mg/m3)	2.65

Specimen Number		**1**	**2**	**3**
Length	mm	100.02	100.02	100.02
Width	mm	100.11	100.11	100.11
Height	mm	24.88	24.65	24.67
Initial Moisture Content	%	14	14	14
Initial Bulk Density	Mg/m3	2.11	2.13	2.13
Initial Dry Density	Mg/m3	1.85	1.86	1.86
Optimum Moisture Content	%	-		
Maximum Dry Density	Mg/m3	-		

Shearing Stage

Normal Pressure	kPa	100	200	400

Peak Conditions

Rate of horizontal displacement	mm/min	0.125	0.125	0.125
Peak shear stress	kPa	76.401	155.198	310.496
Horizontal displacement at peak	mm	3.34	3.95	6.00

Residual Conditions

Rate of horizontal displacement	mm/min	-	-	-
Residual shear stress	kPa	-	-	-
Final cumulative displacement	mm	-	-	-
Total traverses		-	-	-
Method of reversal		-	-	-

Final Moisture Content	%	14	13	12

Shear Strength Parameters

Peak Condition

Apparent Cohesion	kPa	0.0
Angle of Shearing Resistance	o	38.0

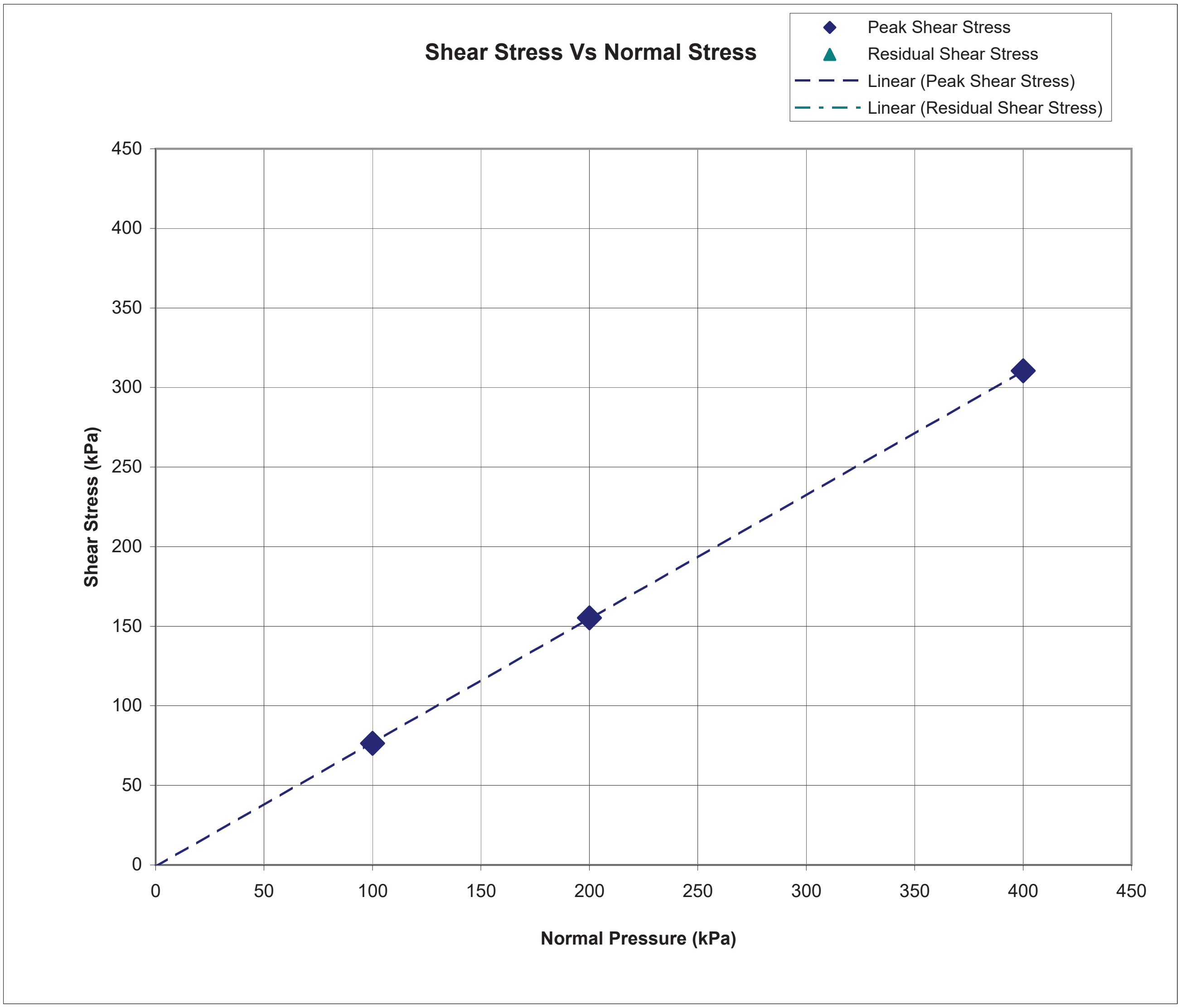
Residual Condition

Apparent Cohesion	kPa	-
Angle of Shearing Resistance	o	-

Test Notes
Preparation - <2mm material prepared in accordance with BS 1377 : Part 7 : 1990 : Clause 4.4.3
Test condition - Submerged
Test specimen remoulded at natural moisture content using a 2.5kg rammer.

DETERMINATION OF SHEAR STRENGTH BY DIRECT SHEAR - SMALL SHEARBOX APPARATUS
Tested in accordance with BS 1377 : Part 7 : 1990 : Clause 4.5.4

Borehole	TP41
Sample	B2
Depth (m)	1.80-1.90



Peak Conditions

Apparent Cohesion	kPa	0.0
Angle of Shearing Resistance	°	38.0

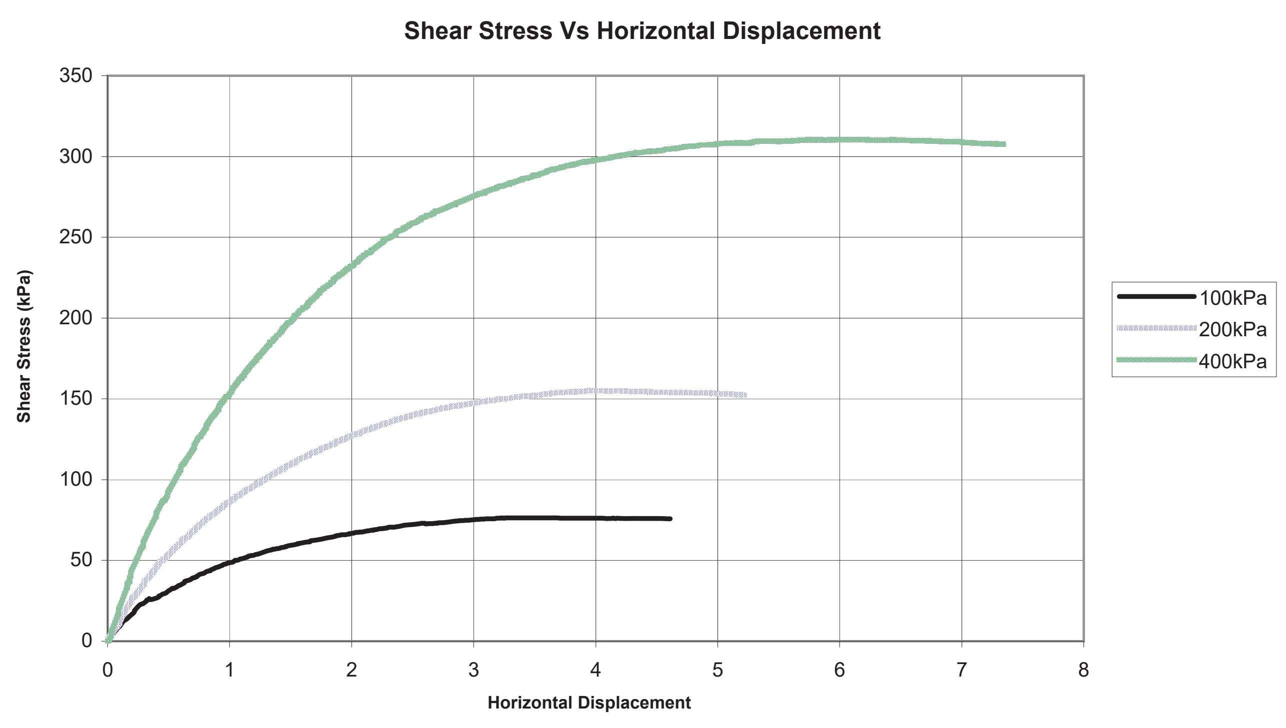
Normal Pressure	kPa	100	200	400
Peak shear stress	kPa	76.401	155.198	310.496

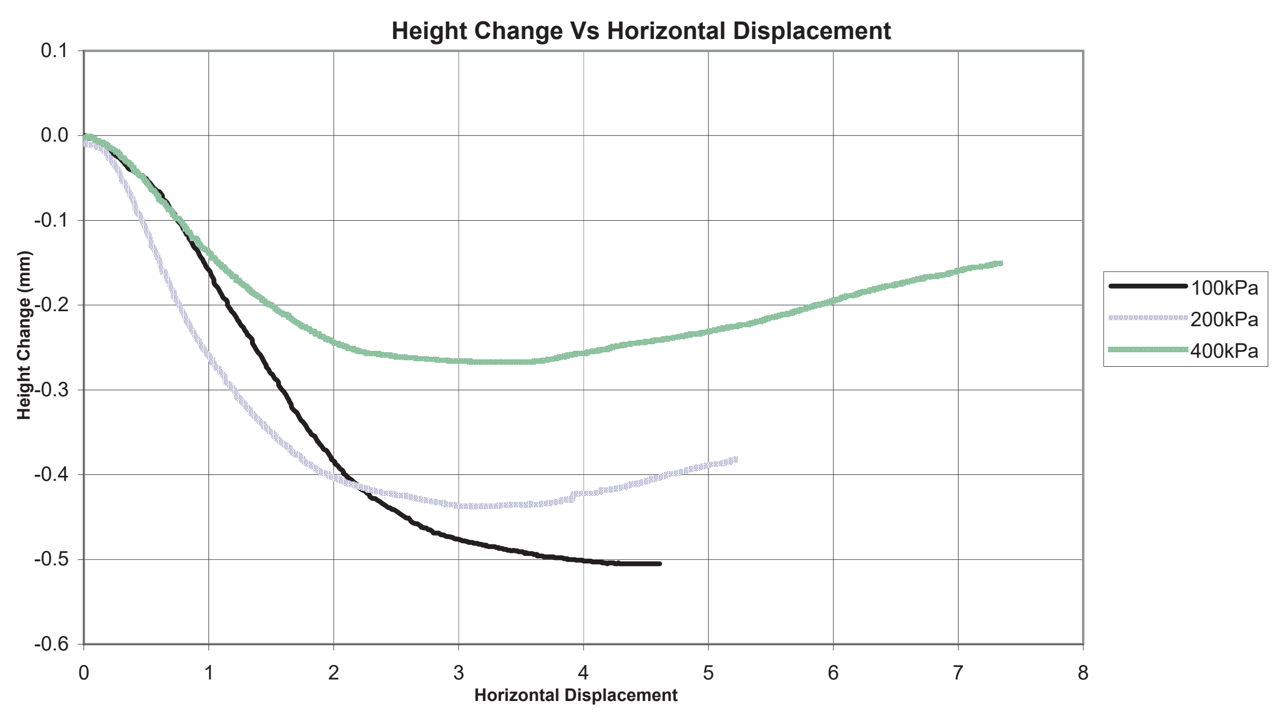
Residual Conditions

Apparent Cohesion	kPa	-
Angle of Shearing Resistance	°	-

Normal Pressure	kPa	-	-	-
Residual shear stress	kPa	-	-	-

Borehole	TP41
Sample	B2
Depth (m)	1.80-1.90





DETERMINATION OF SHEAR STRENGTH BY DIRECT SHEAR - SMALL SHEARBOX APPARATUS

Tested in accordance with BS 1377 : Part 7 : 1990 : Clause 4.5.4

Borehole	TP42
Sample	B1
Depth (m)	1.80-2.00

Specimen Details

Particle Density (assumed)	(Mg/m3)	2.65

Specimen Number		1	2	3
Length	mm	99.59	99.59	99.59
Width	mm	99.46	99.46	99.46
Height	mm	25.01	25.01	25.01
Initial Moisture Content	%	13	14	13
Initial Bulk Density	Mg/m3	2.00	2.01	2.00
Initial Dry Density	Mg/m3	1.77	1.77	1.76
Optimum Moisture Content	%	-		
Maximum Dry Density	Mg/m3	-		

Shearing Stage

Normal Pressure	kPa	100	200	400

Peak Conditions

Rate of horizontal displacement	mm/min	0.125	0.125	0.125
Peak shear stress	kPa	63.098	130.032	255.118
Horizontal displacement at peak	mm	7.93	6.95	8.44

Residual Conditions

Rate of horizontal displacement	mm/min	-	-	-
Residual shear stress	kPa	-	-	-
Final cumulative displacement	mm	-	-	-
Total traverses		-	-	-
Method of reversal		-	-	-

Final Moisture Content	%	16	15	15

Shear Strength Parameters

Peak Condition

Apparent Cohesion	kPa	0.6
Angle of Shearing Resistance	o	32.5

Residual Condition

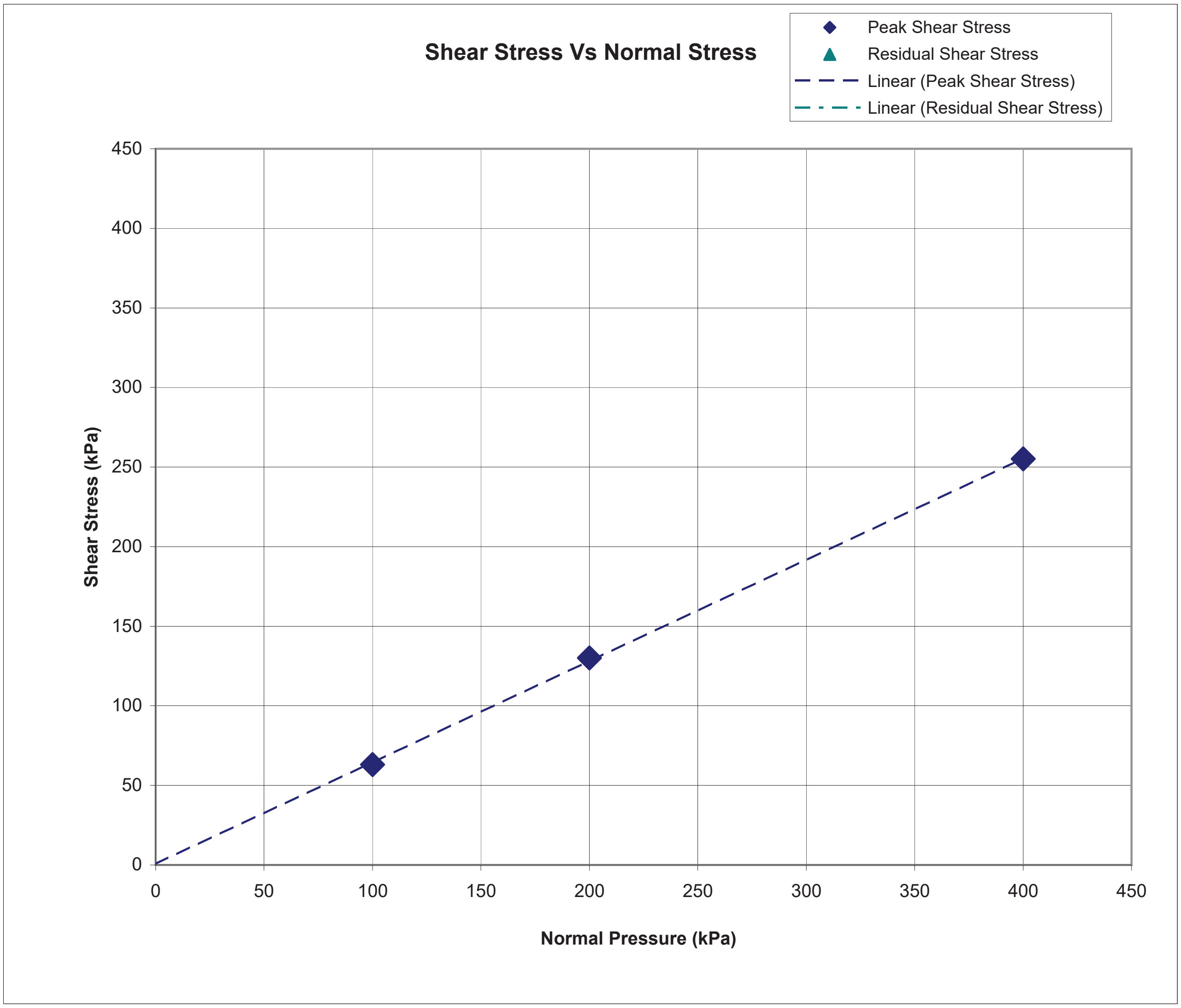
Apparent Cohesion	kPa	-
Angle of Shearing Resistance	o	-

Test Notes

Preparation - <2mm material prepared in accordance with BS 1377 : Part 7 : 1990 : Clause 4.4.3
Test condition - Submerged
Test specimen remoulded at natural moisture content using a 2.5kg rammer.

DETERMINATION OF SHEAR STRENGTH BY DIRECT SHEAR - SMALL SHEARBOX APPARATUS
Tested in accordance with BS 1377 : Part 7 : 1990 : Clause 4.5.4

Borehole	TP42
Sample	B1
Depth (m)	1.80-2.00



Peak Conditions

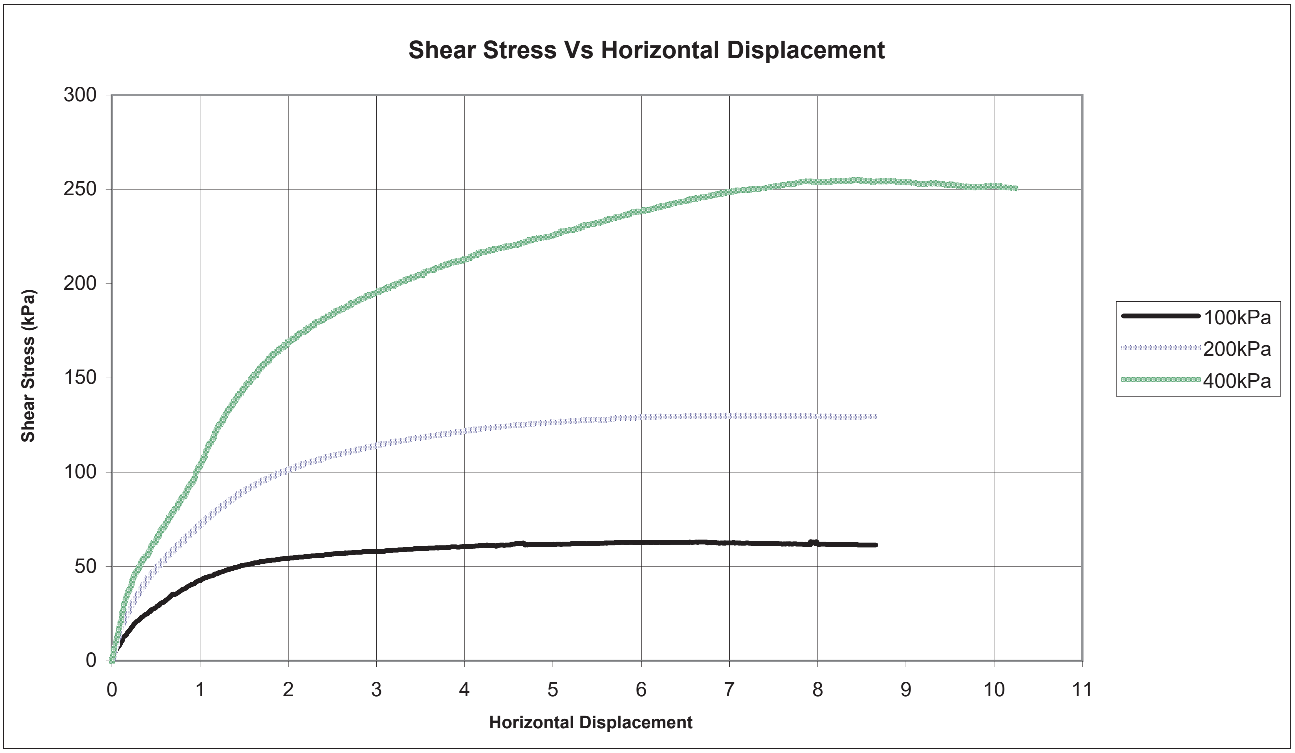
Apparent Cohesion	kPa	0.6		
Angle of Shearing Resistance	o	32.5		
Normal Pressure	kPa	100	200	400
Peak shear stress	kPa	63.098	130.032	255.118

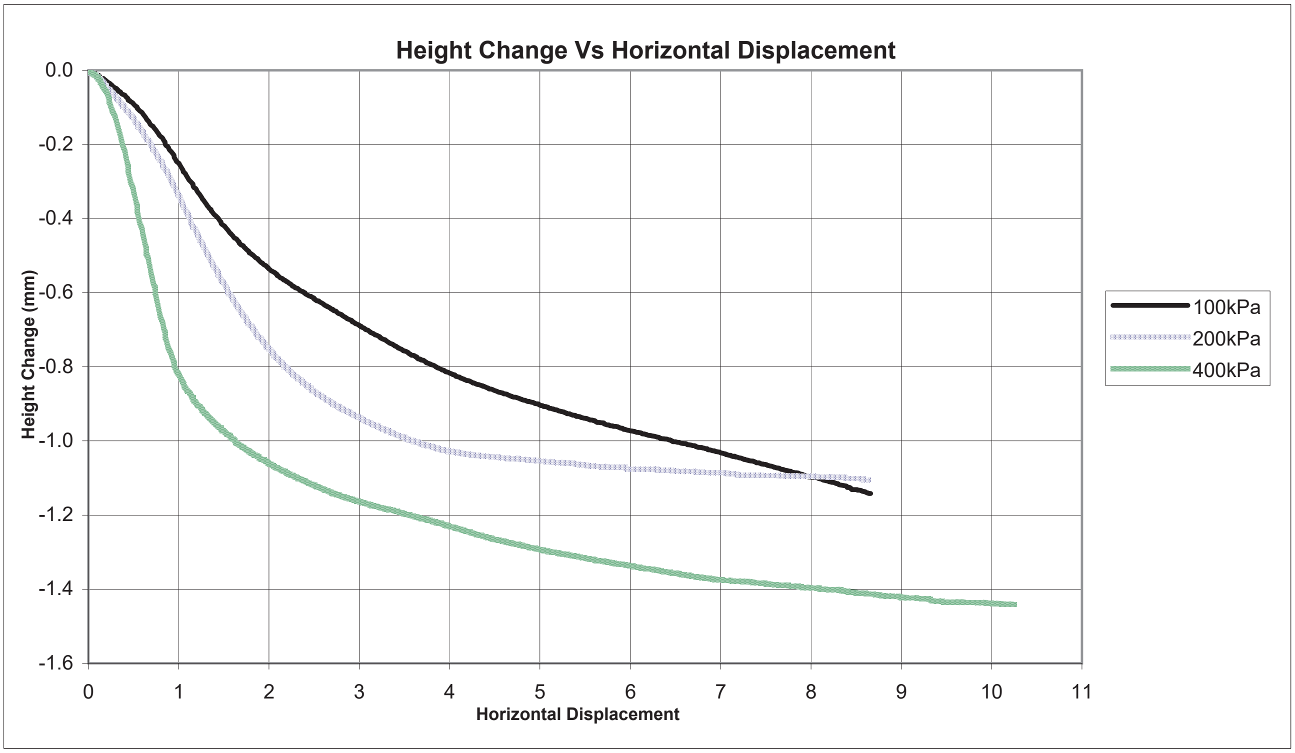
Residual Conditions

Apparent Cohesion	kPa	-		
Angle of Shearing Resistance	o	-		
Normal Pressure	kPa	-	-	-
Residual shear stress	kPa	-	-	-

DETERMINATION OF SHEAR STRENGTH BY DIRECT SHEAR - SMALL SHEARBOX APPARATUS
Tested in accordance with BS 1377 : Part 7 : 1990 : Clause 4.5.4

Borehole	TP42
Sample	B1
Depth (m)	1.80-2.00





DETERMINATION OF SHEAR STRENGTH BY DIRECT SHEAR - SMALL SHEARBOX APPARATUS
Tested in accordance with BS 1377 : Part 7 : 1990 : Clause 4.5.4

Appendix D Non-Destructive Testing Report

HENDERSON THOMAS ASSOCIATES
STRUCTURAL INVESTIGATION EXPERTS

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CAIRNGORM MOUNTAIN LTD
CAIRGORMS NATIONAL PARK
AVIEMORE
PH22 1RB
SCOTLAND, UK

Date: - 29th October 2018

FACTUAL REPORT ON THE NON DESTRUCTIVE TESTING SURVEY TO CAIRNGORM FUNICULAR MOUNTAIN RAILWAY

L-1729-2018

Prepared and Approved By ..[redacted]

[redacted]

For Henderson Thomas Associates limited

CONTENTS

1.0 INTRODUCTION

Further to your order, Henderson Thomas Associates Limited (HTA) carried out non-intrusive Ground Penetrating Radar (GPR) and Cover meter survey to selected elements of the Cairngorms Funicular Railway, Scotland. The main purpose of the investigation was to determine the shear link presence / spacing's and the longitudinal reinforcement locations protruding from the pre-cast beams.

The Cairngorm funicular railway is a viaduct supported on ninety three piers, running for approximately 1.7km from its base station towards the top of Cairngorm Mountain. The Funicular railway is used for the transportation of public visitors up the mountain throughout the year. The viaduct mainly consists of two precast reinforced concrete rail support beams supported on cast in-situ reinforced concrete piers. Structural steelwork is used as lateral bracing between the two pre-cast beams along the length of the viaduct. The precast beams are made continuous over the supports by a cast in-situ reinforced concrete joint. In addition a cast in-situ diaphragm connects the two stitch joints at each pier.

Following the 2018 annual inspection undertaken by COWI UK Ltd a number of areas of structural distress to the superstructure of the viaduct was noted and in particular over the supports. In turn, the findings of the inspection concluded that an assessment of viaduct be undertaken in particular the pre-cast beams and cast in-situ joints over the support.

Due to the lack of as-built drawings and consistency of the original design details it was recommended to undertake investigations works to facilitate the assessment of the elements showing visible signs of distress.

This factual report presents on site non-intrusive investigation findings only.

2.0 ON SITE NON-INTRUSIVE INVESTIGATIONS

Following on from the findings of the COWI UK Ltd report referenced '*Cairngorm Funicular Railway – Structural Inspection*' and recommendations outlined within the report. Cairngorms Mountain Ltd. appointed HTA to undertake non-intrusive investigations to a number of selected elements.

The site works were undertaken during daytime weekday working hours from Wednesday 17th October to Friday 26th October 2018.

Prior to attending HTA issued risk assessment and method statements for approval by the client. In addition prior to commencing work HTA's on site personnel attended a site specific induction given by the client.

All site investigation works detailed in this report were undertaken by HTA. The site works were undertaken in general accordance with the works specification as outlined within the afore mentioned COWI UK Ltd report and on-site instructions of [redacted] [redacted] of ADAC Structures Ltd.

The works specifications outlined the required non-intrusive investigation locations and were as follows:-

- Pier 9
- Pier 22
- Pier 46* (Amended on site to Pier 49)
- Pier 51
- Pier 53
- Pier 54
- Pier 56
- Pier 69
- Pier 80

Access was via the mountain access road to the nearest location using a four wheel drive vehicle and then by foot to the selected pier. At a number of the selected piers the beams could be accessed from ground level, although at a number of piers ladders were required to facilitate the works to the beams. The client provided ladders on site for HTA's use.

At pier 46, HTA were unable to facilitate the works at high level due the weather conditions on the day it was agreed on site with ADAC / COWI that the works should be moved to Pier 49 which could be accessed from ground level given the weather conditions. At a number of locations HTA was unable to safely access the top flange to facilitate scanning due to the nature of these specific piers reference is given to these within the figures.

At all locations the beams were referenced in relation to the Pier and handed (i.e. left / right looking up the Mountain) and if it was the upper or lower beam in relation to the pier.

2.1 GROUND PENETRATING RADAR SURVEY

Impulse radar scanning was carried out to all pre-cast concrete beam side elevations at all of the afore mentioned piers using GSSI's Structure Scan Mini GPR. The impulse radar scanning was undertaken in an attempt to ascertain the following:-

- The location & spacing's of shear link reinforcement protruding from the precast beams into the in-situ joints.
- The location & spacing's of shear link reinforcement within the in-situ joints and the lap length with the pre-cast beam shear links.
- The location & spacing's of shear links within the span of the precast beams up to 1000mm from the in-situ joints.
- The location & presence of longitudinal reinforcement protruding from the webs of the precast beams into the cast in-situ joints and presence within the webs of the precast beams up to 1000mm from the in-situ joints.
- The location of the longitudinal top flange reinforcement within the in-situ joints, and attempt to ascertain the presence of couplers.

The impulse radar scanning was undertaken on the external side elevations of all beams as outlined in the scope of works and the site findings were marked on the concrete surface.

During the site works it became apartment that the impulse radar equipment was unable to fit beneath the running rails of the railway to scan the top face of the top flange and HTA were only able to scan the side elevations. This was discussed on site with ADAC and confirmed that HTA should scan the side elevations in an attempt to locate the presence of the longitudinal reinforcement protruding from the pre-cast beams into the in-situ joints. Although with the exception of pier 54 and 56 where the impulse radar equipment was able to fit beneath the running rail and the pre-cast beam to facilitate scanning of the top flange.

In addition, HTA discussed the location the shear link laps from the in-situ joint and the pre-cast beams and that there was uncertainty to the readings gathered on site as it appeared when scanned that the shear link spacing were the same in both and no visible difference in the data gathered. Although at a Pier 9 & Pier 22 right hand beam upper & lower the shear link spacing did not align when measured which suggests there is links present in the in-situ joint.

The impulse radar scan results identifying the located reinforcement are presented in the figures in Appendix A.

Photographs of each beam showing the marked concrete surface is presented in Appendix B.

2.2 COVER METER SURVEY

In addition to the impulse radar scanning undertaken cover meter scans were undertaken using Proceq's Profoscope Cover meter at all of the afore mentioned beam locations to ascertain as follows:-

- The depth of concrete cover to the shear link reinforcement on both faces of the beam web.
- The location of the longitudinal top flange reinforcement within the in-situ joints, and attempt to ascertain the presence of couplers.

During the site works it became apartment that the cover meter was unable to fit to be utilized beneath the running rails of the railway to scan the top face of the top flange due to the readings being affected by the rail. Although HTA scanned the side elevations on all of the afore mentioned top flanges within the in-situ joint in an attempt to ascertain the presence of couplers.

The cover meter readings were taken using an assumed reinforcing bar diameter as presented in the design drawings provided by COWI. Main longitudinal reinforcement in the top flange was assumed to be 32mm diameter and the shear link reinforcement was assumed to be 8mm diameter.

At none of the test locations was the cover meter checked against the actual depth of cover due to the scope of works being non-destructive only.

The cover meter results identifying the located reinforcement depth of concrete cover are presented in the figures in Appendix A.

3.0 RECOMMENDATIONS

Following the onsite NDT testing undertaken to date, HTA would recommend the carrying out of intrusive investigations to confirm the findings outlined within this report to the selected elements. As with all non-destructive testing methods HTA would recommended that NDT testing is undertaken in conjunction with intrusive findings to calibrate the testing undertaken.

The intrusive investigations would confirm the following:-

1) The presence of shear link reinforcement within the in-situ joints and the lap length with the pre-cast beam shear links if any.
2) Ascertain if in fact the longitudinal reinforcement within the webs of the precast beams stops within the pre-cast beam and not infact protruding into the in-situ joint.
3) Confirm the presence of couplers to the top flange reinforcement within the in-situ joint if any.

On completion of the above intrusive investigation works HTA believe this would provide the client with the required information to undertake an assessment of the structure.

4.0 QUALITY STATEMENT

HTA Ltd. confirm that all reasonable skill and care has been exercised in the production of this report, however all comments relate only to the location at which data was acquired and no inference can or should be made to any other part of the structure.

No part of this report should be reproduced without the written consent of HTA Ltd and it is intended for the sole use of the named client only.

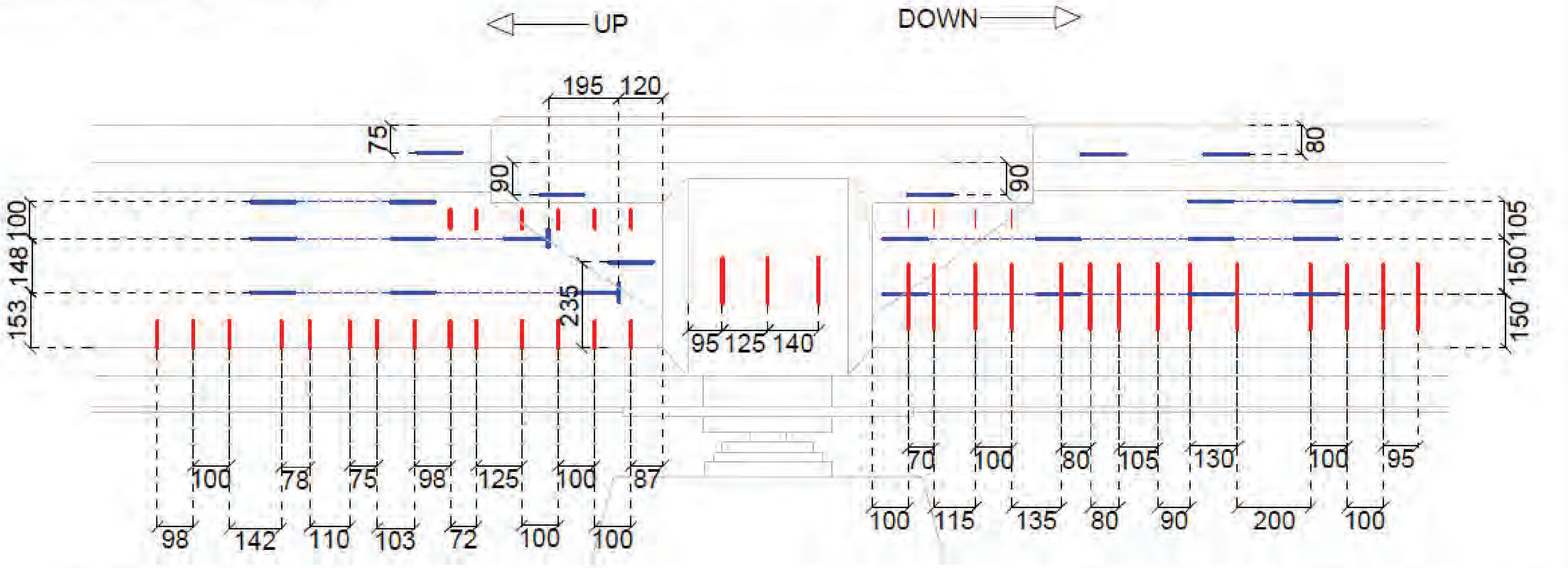
APPENDIX A

Figures

All figures are not to scale

FIGURE 1

CAIRNGORM FUNICULAR RAILWAY



Outside Elevation, Up, Link Bar Cover: 31, 31, 37, 31, 37, 28, 40, 29, 28, 36, 37, 36 and 37mm

Inside Elevation, Up, Link Bar Cover: 34, 44, 32, 37, 36, 37, 32, 39, 35, 40, 32 and 34mm

Outside Elevation, Down, Link Bar Cover: 36, 34, 34, 30, 33, 32, 33, 33, 32 and 31mm

Inside Elevation, Down, Link Bar Cover: 45, 33, 43, 31, 43, 30, 39, 34, 41 and 26mm

Note: Not to Scale. Dimensions in (mm)

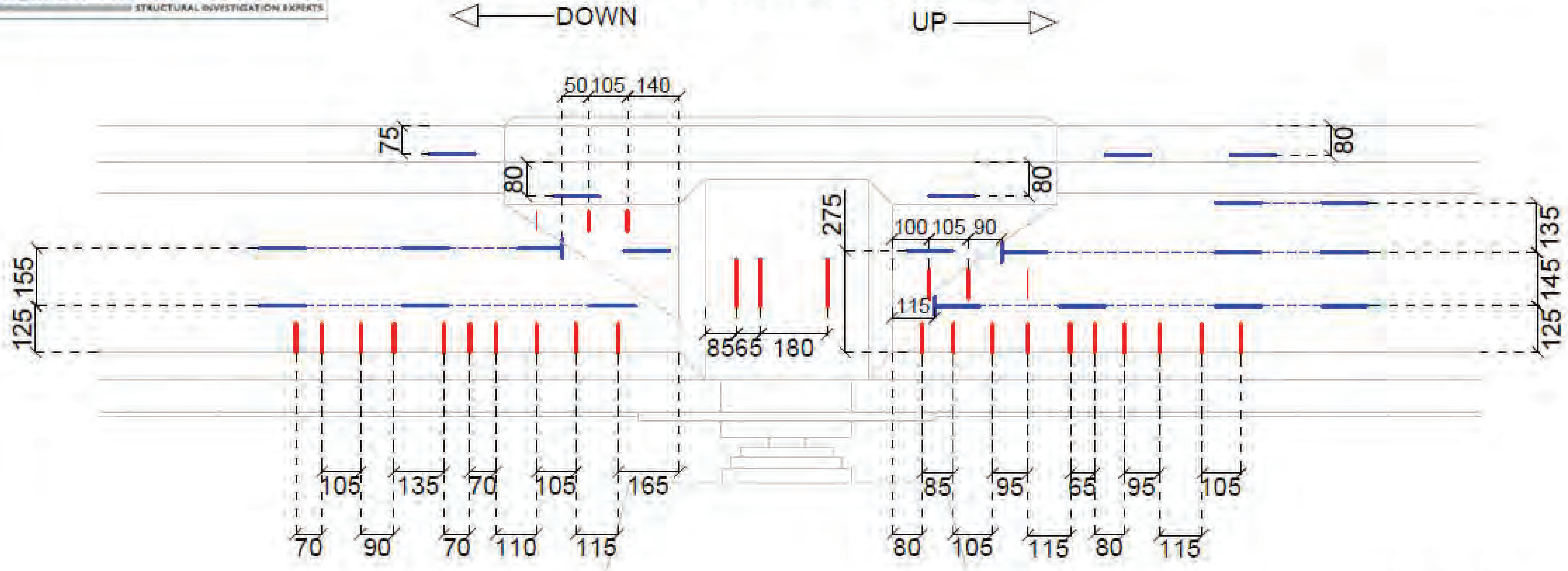
Note :-
Impulse radar equipment was unable to fit beneath the running rail at this location to scan the top flange.

PIER 9 LEFT BEAM OUTSIDE ELEVATION

ADAC/CML	CAIRNGORM FUNICULAR
File No: L/1728/18	Date: 25/10/2018

FIGURE 2

CAIRNGORM FUNICULAR RAILWAY



Outside Elevation, Up, Link Bar Cover: 40, 40, 46, 35, 36, 37, 37, 32, 36, 34, 33, 28, 31, 33, 31, 34 and 29mm

Inside Elevation, Up, Link Bar Cover: 35, 37, 34, 33, 31, 29, 31, 29, 31, 30, 38, 27, 39, 30, 38, 30, 35, 30, 37, 28, 36 and 29mm

Outside Elevation, Up, Link Bar Spacing from 10th link: 50,135, 100, 85, 100, 100, 110, 100, 95, 180, 210, 215, 190, 155, 230, 220, 200, 190, 215, 150, 85, 170, 190, 150, 265, 280, 290, 290, 310, 310, 295, 300, 310, 280, 290, 320, 300, 300, 280, 300mm

Note :-
Impulse radar equipment was unable to fit beneath the running rail at this location to scan the top flange.

Outside Elevation, Down, Link Bar Spacing from 10th link: 135, 75, 130,105, 90, 110, 155, 80, 175, 230, 195, 185, 150, 235, 100, 140, 170, 110, 155, 180, 85, 115, 195, 190, 165, 255, 250, 315, 275, 325, 320, 270, 310, 300 305, 320, 280, 310, 305, 295, 290mm

PIER 9 RIGHT BEAM OUTSIDE ELEVATION

Outside Elevation, Down, Link Bar Cover: 32, 26, 25, 26, 25, 29, 25, 25, 22, 20, 26, 24, 21, 21, 21, 21, 33, 21, 22, 20, 36, 22, 24 and 24mm

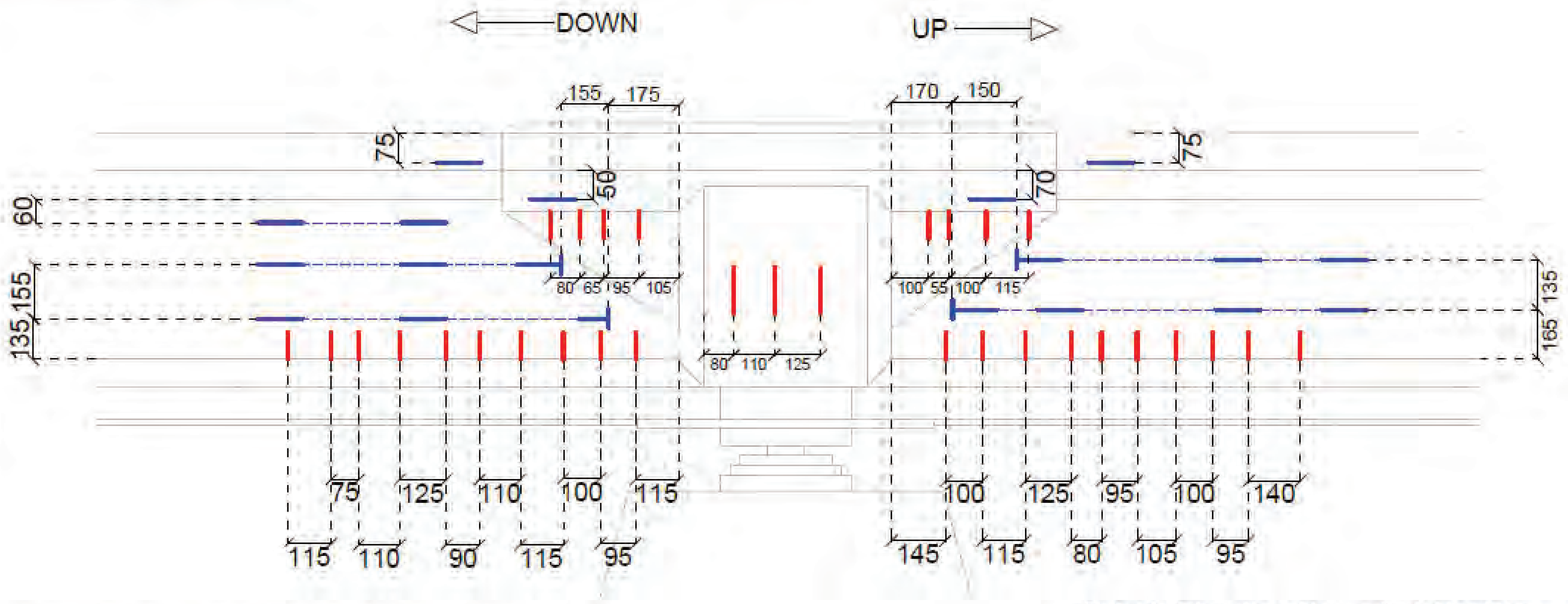
Inside Elevation, Down, Link Bar Cover: 40, 48, 40, 48, 40, 48, 35, 39, 35, 37, 50, 36, 45, 35, 47, 39, 37, 51, 39, 44, 40, 51 and 39mm

Note: Not to Scale. Dimensions in (mm)

ADAC/CML	CAIRNGORM FUNICULAR
File No: L/1728/18	Date: 25/10/2018

FIGURE 3

CAIRNGORM FUNICULAR RAILWAY



Outside Elevation, Up, Link Bar Cover: 29, 31, 21, 32, 33, 30, 34, 30, 32 and 31mm

Outside Elevation, Down, Link Bar Cover: 30, 34, 34, 33, 32, 32, 33, 34, 36 and 42mm

Inside Elevation, Up, Link Bar Cover: 50, 49, 40, 50, 39, 48, 35, 40, 36, 35 and 35mm

Inside Elevation, Down, Link Bar Cover: 31, 46, 32, 47, 31, 31, 47, 30, 45, 29, 31, 46, 35, 42, 39 and 39mm

Note: Not to Scale. Dimensions in (mm)

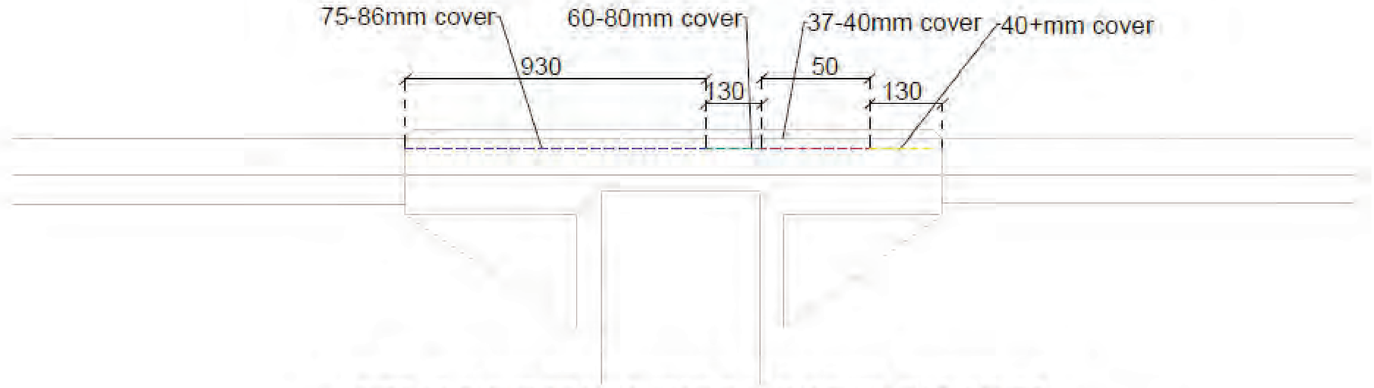
Note :-
Impulse radar equipment was unable to fit beneath the running rail at this location to scan the top flange.

PIER 22 RIGHT BEAM OUTSIDE ELEVATION

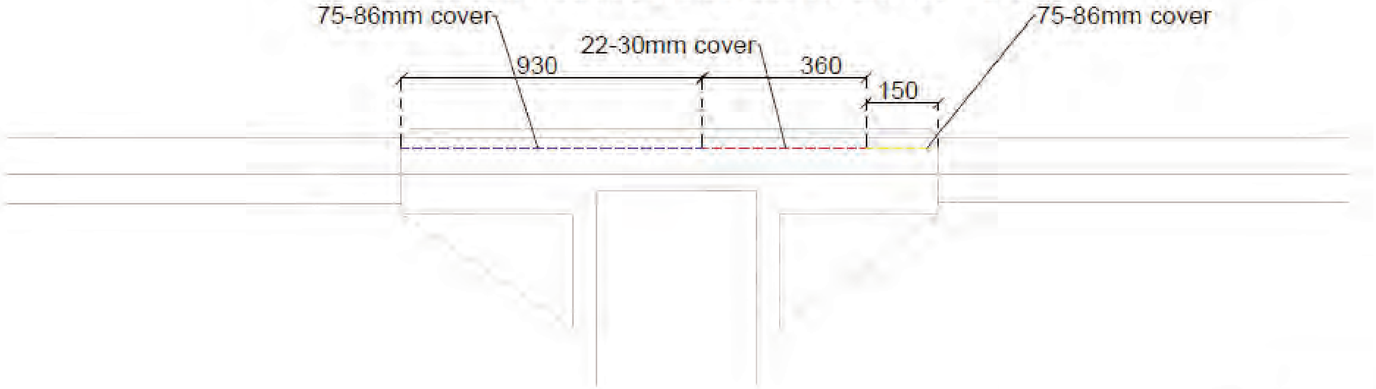
ADAC/CML	CAIRNGORM FUNICULAR
File No: L/1728/18	Date: 25/10/2018

FIGURE 4

CAIRNGORM FUNICULAR RAILWAY



PIER 22 RIGHT BEAM OUTSIDE ELEVATION- TOP COVER



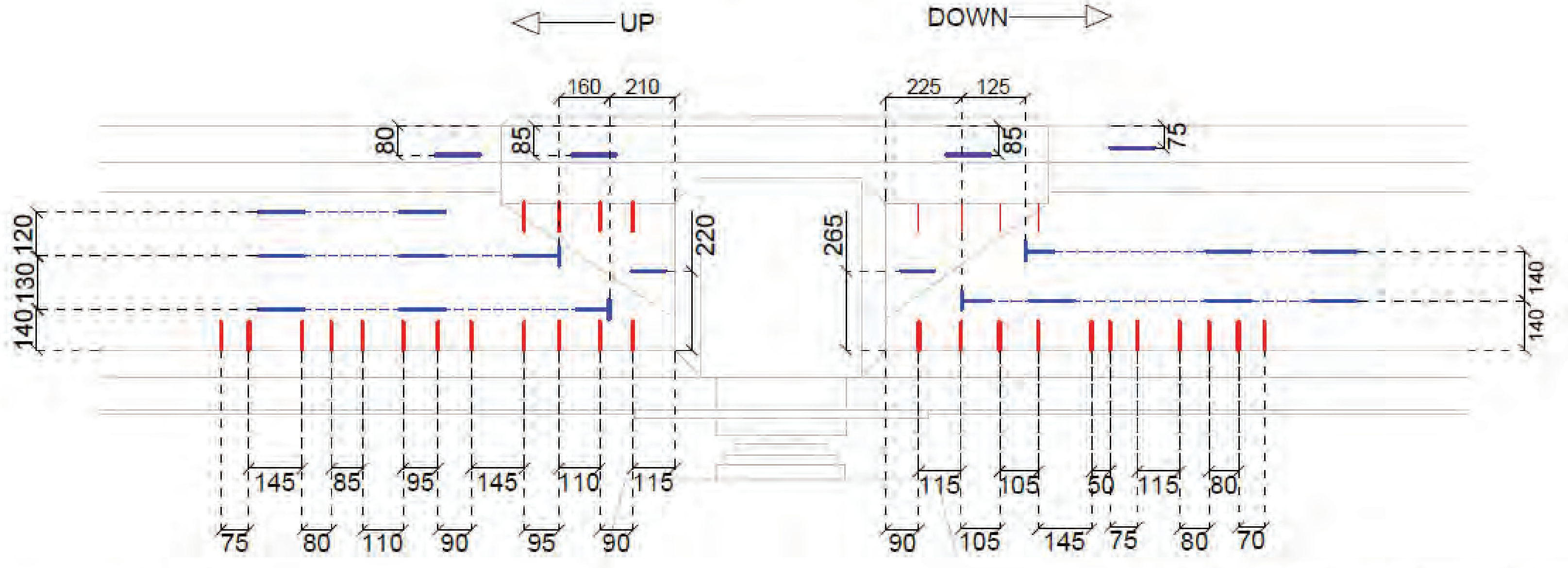
PIER 22 RIGHT BEAM INSIDE ELEVATION- TOP COVER

Note: Not to Scale. Dimensions in (mm)

ADAC/CML	CAIRNGORM FUNICULAR
File No: L/1728/18	Date: 25/10/2018

FIGURE 5

CAIRNGORM FUNICULAR RAILWAY



Outside Elevation, Up, Link Bar Cover: 34, 36, 46, 34, 38, 33, 35, 35, 42, 34, 41, 35, 35, 46, 36, 36, 34 and 41mm

Inside Elevation, Up, Link Bar Cover: 27, 41, 26, 38, 33, 40, 33, 27, 23, 46 and 29mm

Outside Elevation, Down, Link Bar Cover: 49, 52, 40, 39, 38, 37, 38, 38, 37, 35 and 36mm

Inside Elevation, Down, Link Bar Cover: 47, 30, 35, 40, 35, 38, 33, 43, 32, 38, 30, 47, 25, 43, 32 and 41mm

Note: Not to Scale. Dimensions in (mm)

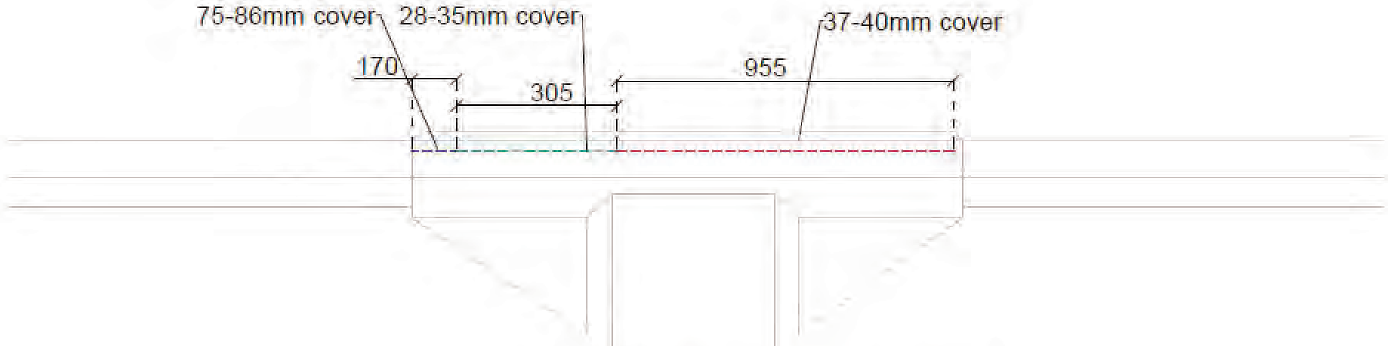
Note :-
Impulse radar equipment was unable to fit beneath the running rail at this location to scan the top flange.

PIER 22 LEFT BEAM OUTSIDE ELEVATION

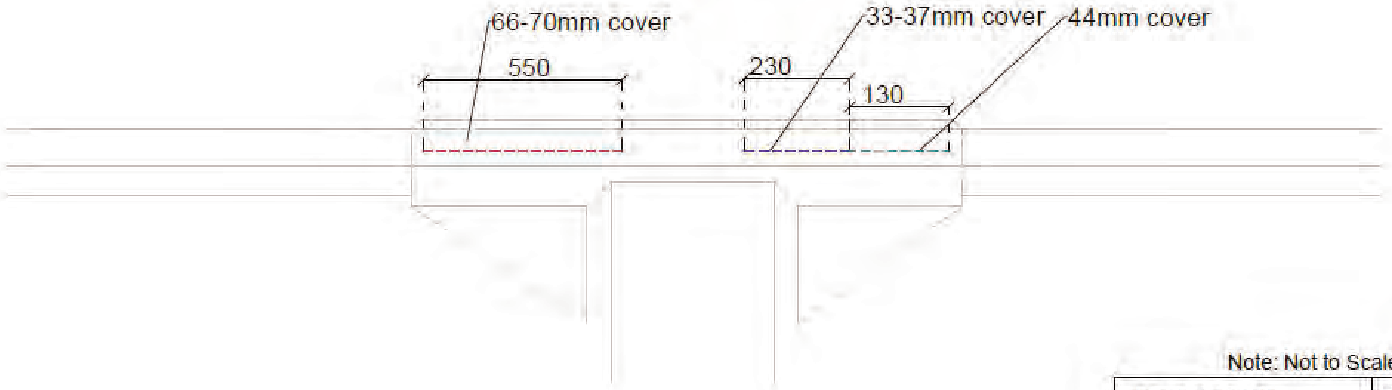
ADAC/CML	CAIRNGORM FUNICULAR
File No: L/1728/18	Date: 25/10/2018

FIGURE 6

CAIRNGORM FUNICULAR RAILWAY



PIER 22 LEFT BEAM OUTSIDE ELEVATION



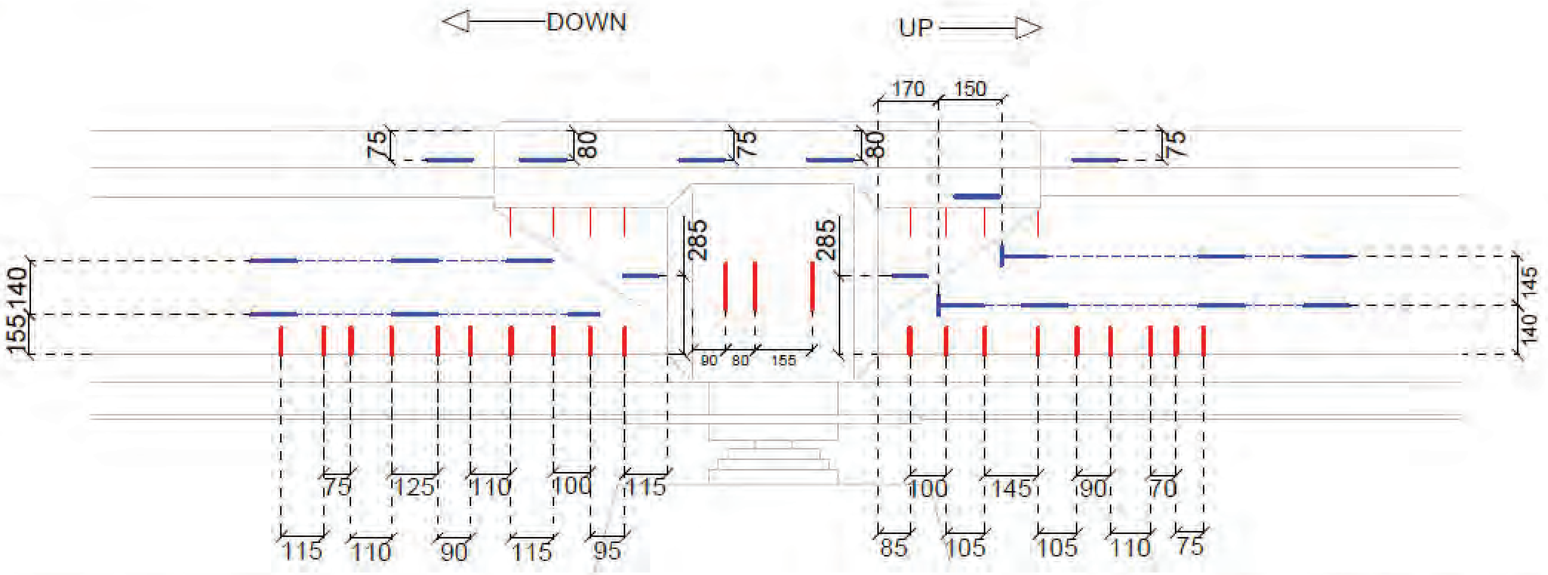
PIER 22 LEFT BEAM INSIDE ELEVATION

Note: Not to Scale. Dimensions in (mm)

ADAC/CML	CAIRNGORM FUNICULAR
File No: L/1728/18	Date: 25/10/2018

FIGURE 7

CAIRNGORM FUNICULAR RAILWAY



Outside Elevation, Up, Link Bar Cover: 51, 35, 44, 37, 47, 38, 45, 31, 26, 46, 25 and 32mm

Inside Elevation, Up, Link Bar Cover: 42, 37, 41, 35, 41, 32, 38, 30, 35, 24, 37, 26, 33, 27, 30, 26 and 38mm

Outside Elevation, Down, Link Bar Cover: 37, 33, 37, 32, 42, 41, 30, 38, 34, 41, 32 and 29mm

Inside Elevation, Down, Link Bar Cover: 30, 43, 39, 43, 42, 48, 44, 53 and 42mm

Note: Not to Scale. Dimensions in (mm)

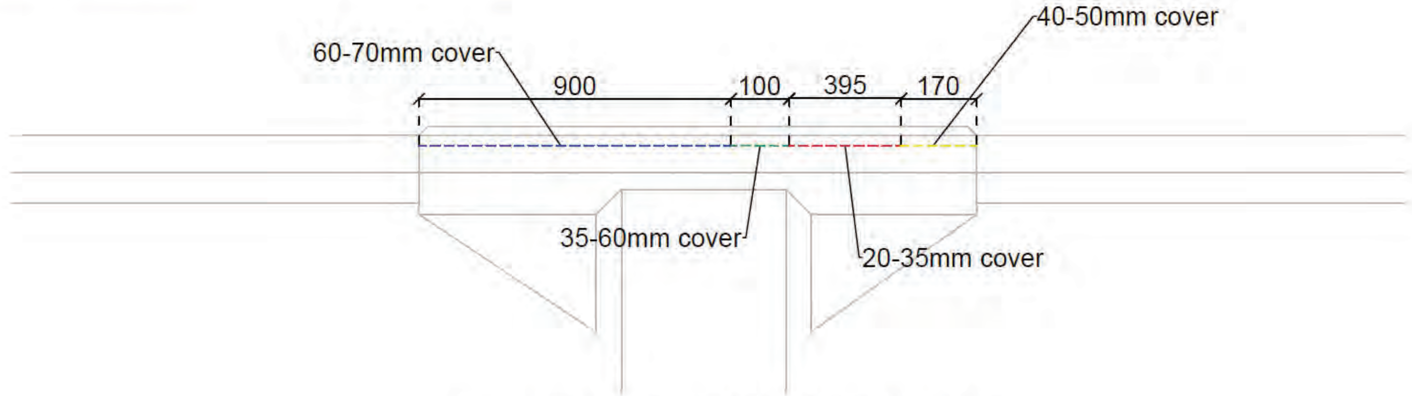
Note :-
Impulse radar equipment was unable to fit beneath the running rail at this location to scan the top flange.

PIER 49 RIGHT BEAM OUTSIDE ELEVATION

ADAC/CML	CAIRNGORM FUNICULAR
File No: L/1728/18	Date: 25/10/2018

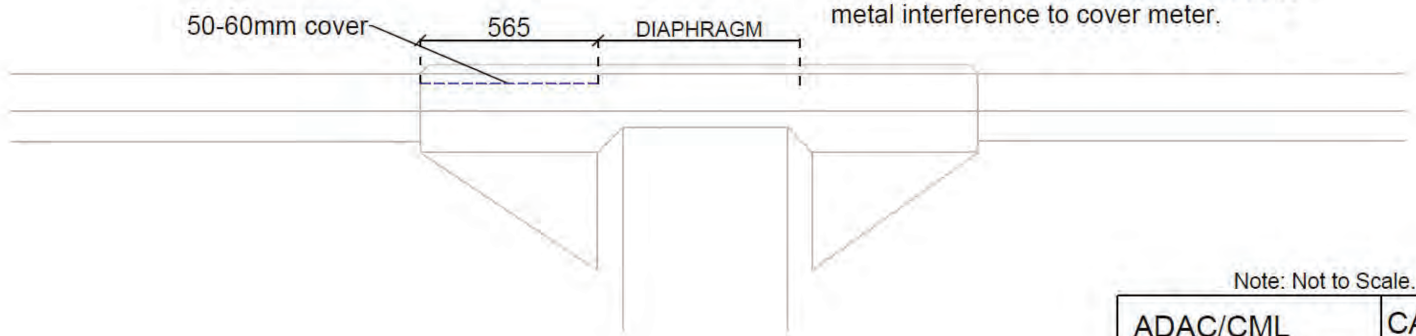
FIGURE 8

CAIRNGORM FUNICULAR RAILWAY



PIER 49 RIGHT BEAM OUTSIDE ELEVATION

Inside Face (Upside) inaccessible due to metal interference to cover meter.



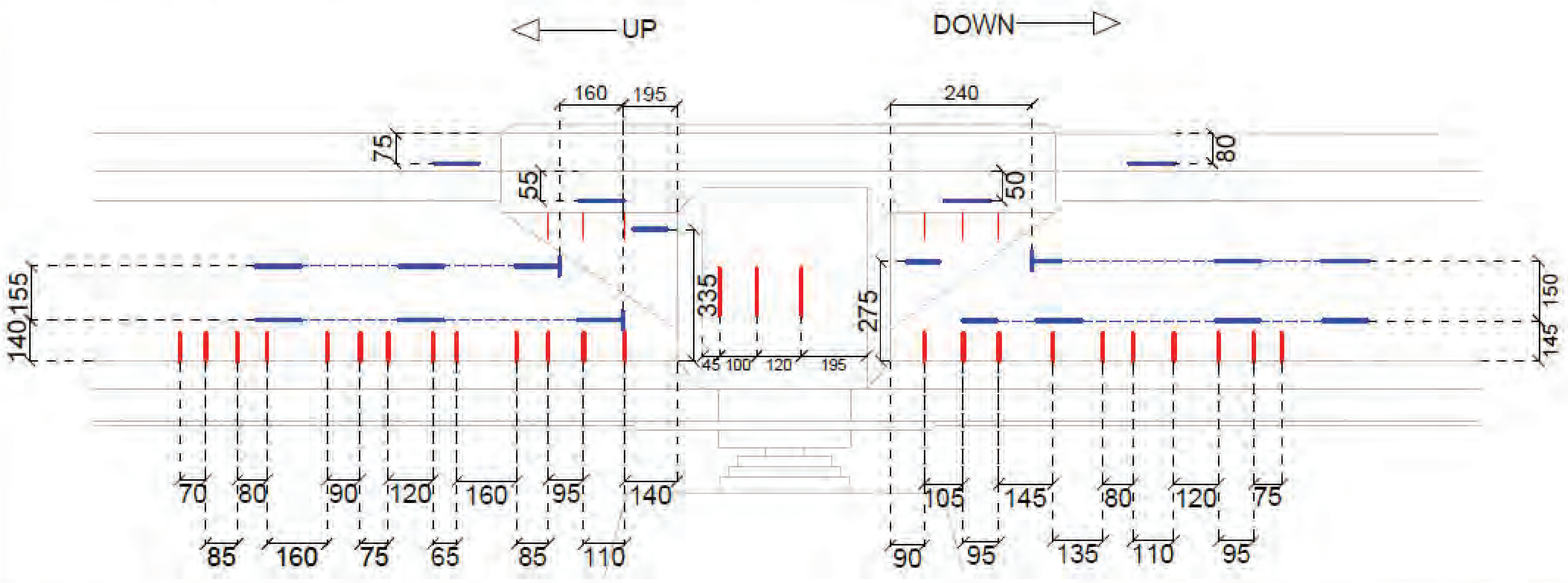
PIER 49 RIGHT BEAM INSIDE ELEVATION

Note: Not to Scale. Dimensions in (mm)

ADAC/CML	CAIRNGORM FUNICULAR
File No: L/1728/18	Date: 08/11/2018

FIGURE 9

CAIRNGORM FUNICULAR RAILWAY



Outside Elevation, Up, Link Bar Cover: 40, 29, 35, 31, 47, 29, 31, 27, 51, 39, 45, 43, 49, 54 and 44mm

Inside Elevation, Up, Link Bar Cover: 44, 32, 36, 31, 36, 30, 42, 34, 34, 44, 33, 35, 32 and 41mm

Outside Elevation, Down, Link Bar Cover: 35, 40, 35, 40, 33, 40, 32, 36, 34, 38, 32, 36, 27, 31 and 27mm

Inside Elevation, Down, Link Bar Cover: 42, 41, 44, 39, 44, 38 and 44mm

Note: Not to Scale. Dimensions in (mm)

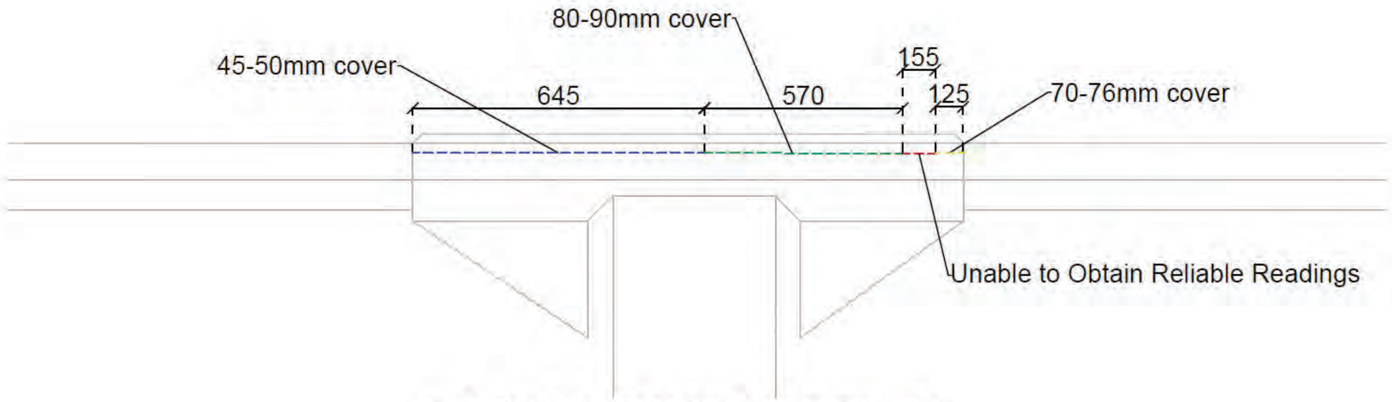
Note :-
Impulse radar equipment was unable to fit beneath the running rail at this location to scan the top flange.

PIER 49 LEFT BEAM OUTSIDE ELEVATION

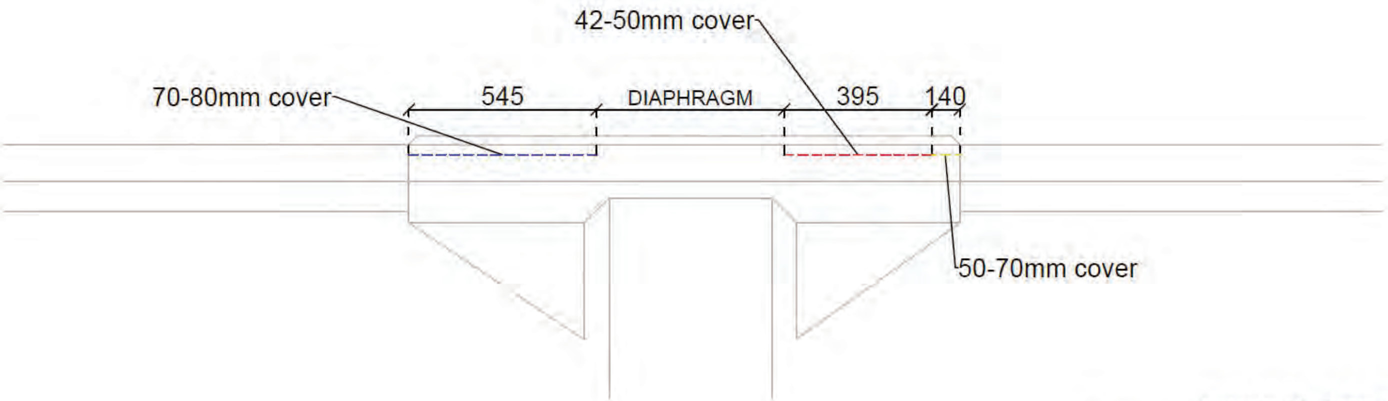
ADAC/CML	CAIRNGORM FUNICULAR
File No: L/1728/18	Date: 25/10/2018

FIGURE 10

CAIRNGORM FUNICULAR RAILWAY



PIER 49 LEFT BEAM OUTSIDE ELEVATION



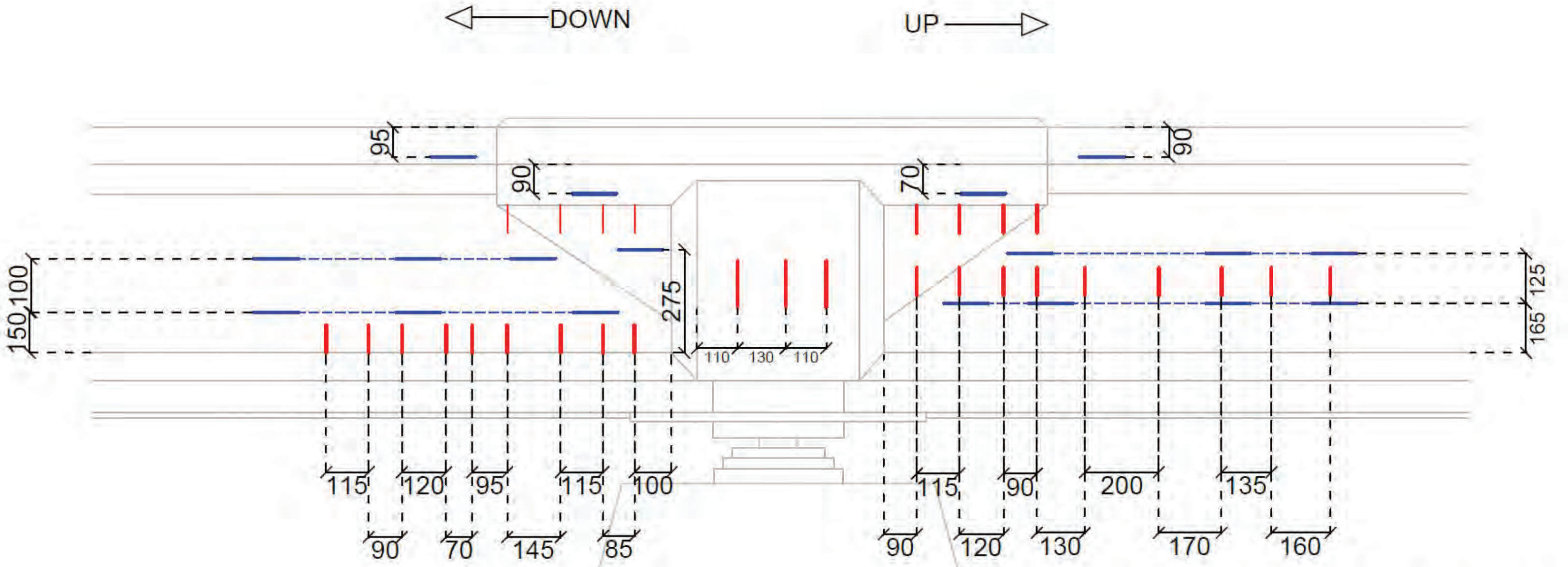
PIER 49 LEFT BEAM INSIDE ELEVATION

Note: Not to Scale. Dimensions in (mm)

ADAC/CML	CAIRNGORM FUNICULAR
File No: L/1728/18	Date: 08/11/2018

FIGURE 11

CAIRNGORM FUNICULAR RAILWAY



Outside Elevation, Up, Link Bar Cover: 33, 30, 34, 32, 32, 38, 32 and 38mm

Outside Elevation, Down, Link Bar Cover: 38, 38, 33, 34 and 31mm

Inside Elevation, Up, Link Bar Cover: Unable to determine due to presence of cross beams.

Inside Elevation, Down, Link Bar Cover: Unable to determine due to presence of cross beams.

Note :-
Impulse radar equipment was unable to fit beneath the running rail at this location to scan the top flange.

Note: Bolts on both up and down side prevent location of a number of link bars and termination points of longitudinal bars.

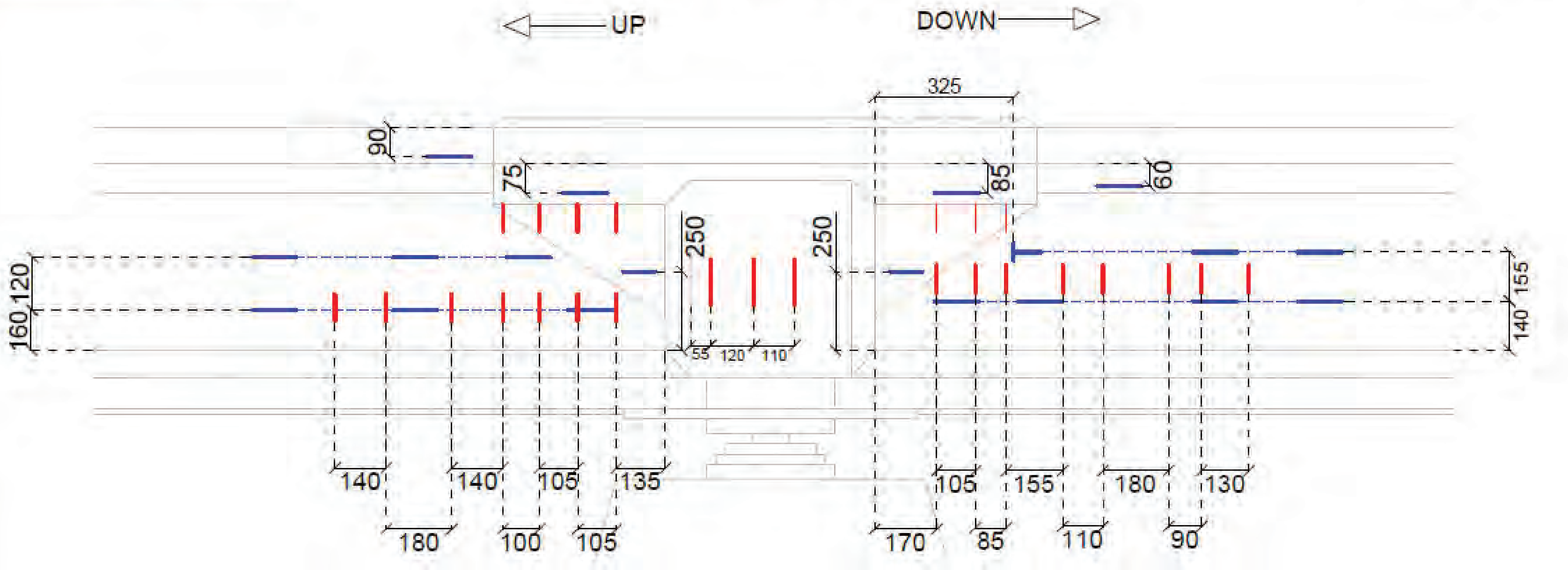
PIER 51 RIGHT BEAM OUTSIDE ELEVATION

Note: Not to Scale. Dimensions in (mm)

ADAC/CML	CAIRNGORM FUNICULAR
File No: L/1728/18	Date: 08/11/2018

CAIRNGORM FUNICULAR RAILWAY

FIGURE 12



Outside Elevation, Up, Link Bar Cover: 41, 38, 30, 40, 47 and 26mm

Outside Elevation, Down, Link Bar Cover: 31, 31, 30, 38 and 38mm

Inside Elevation, Up, Link Bar Cover: Unable to determine due to presence of cross beams.

Inside Elevation, Down, Link Bar Cover: Unable to determine due to presence of cross beams.

Note :-
Impulse radar equipment was unable to fit beneath the running rail at this location to scan the top flange.

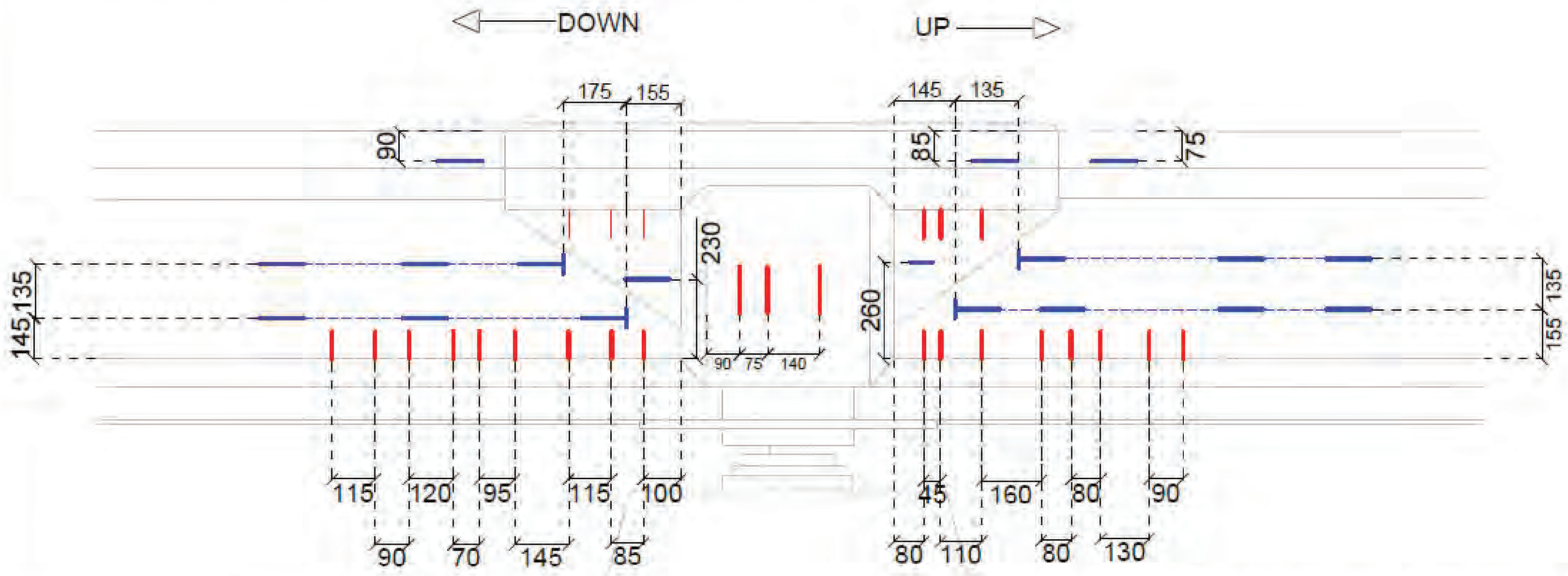
PIER 51 LEFT BEAM OUTSIDE ELEVATION

Note: Not to Scale. Dimensions in (mm)

ADAC/CML	CAIRNGORM FUNICULAR
File No: L/1728/18	Date: 25/10/2018

FIGURE 13

CAIRNGORM FUNICULAR RAILWAY



Outside Elevation, Up, Link Bar Cover: 32, 33, 30, 33, 34, 35 and 36mm

Inside Elevation, Up, Link Bar Cover: 47, 40, 45, 40, 49, 31, 35, 31 and 39mm

Outside Elevation, Down, Link Bar Cover: 38, 38, 37, 35, 34, 33, 33, 34, 34 and 34mm

Inside Elevation, Down, Link Bar Cover: 33, 36, 44, 33, 41, 30, 46, 33, 28, 45, 36 and 47mm

Note: Not to Scale. Dimensions in (mm)

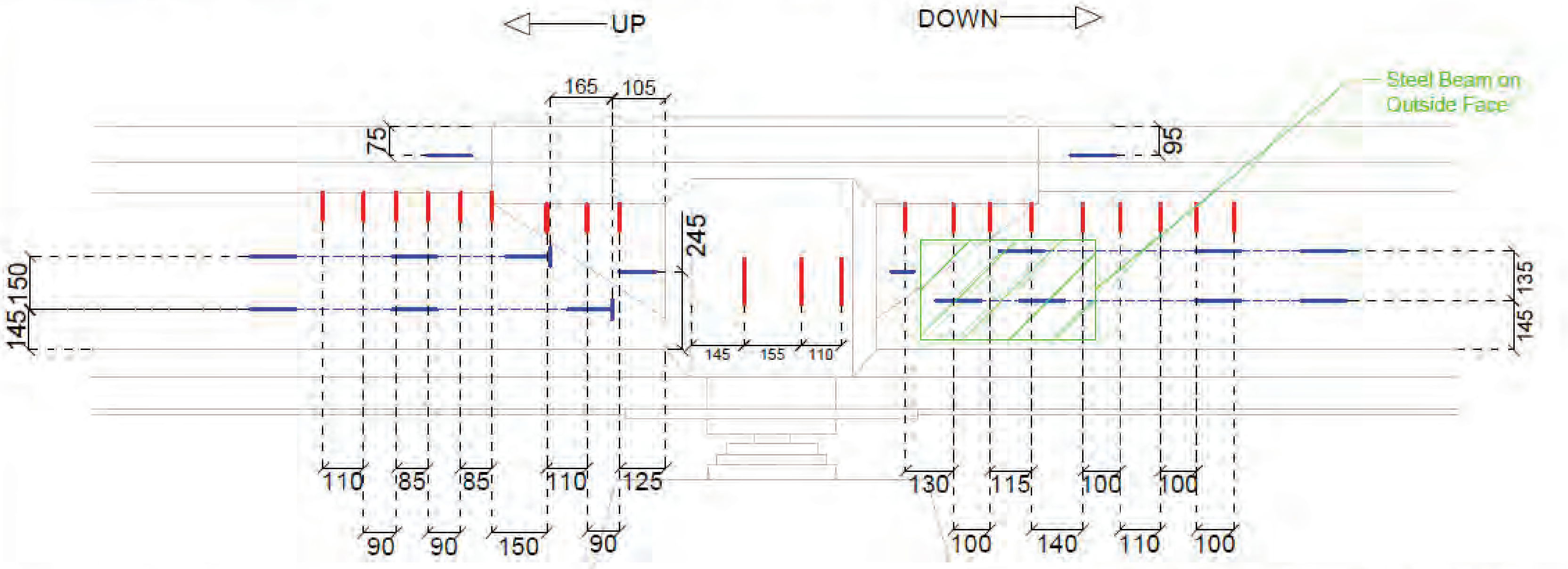
Note :-
Impulse radar equipment was unable to fit beneath the running rail at this location to scan the top flange.

PIER 53 RIGHT, RIGHT BEAM OUTSIDE ELEVATION

ADAC/CML	CAIRNGORM FUNICULAR
File No: L/1728/18	Date: 25/10/2018

FIGURE 14

CAIRNGORM FUNICULAR RAILWAY



Outside Elevation, Up, Link Bar Cover: 40, 28, 21, 41, 40, 40, 40, 40 and 40mm

Outside Elevation, Down, Link Bar Cover: Unable to scan due to steel beam.

Inside Elevation, Up, Link Bar Cover: 34, 45, 28, 25, 41, 23, 38, 37 and 25mm

Inside Elevation, Down, Link Bar Cover: 40, 26, 30, 34, 36, 30 and 38mm

No top bars located with covermeter

Note :-
Impulse radar equipment was unable to fit beneath the running rail at this location to scan the top flange.

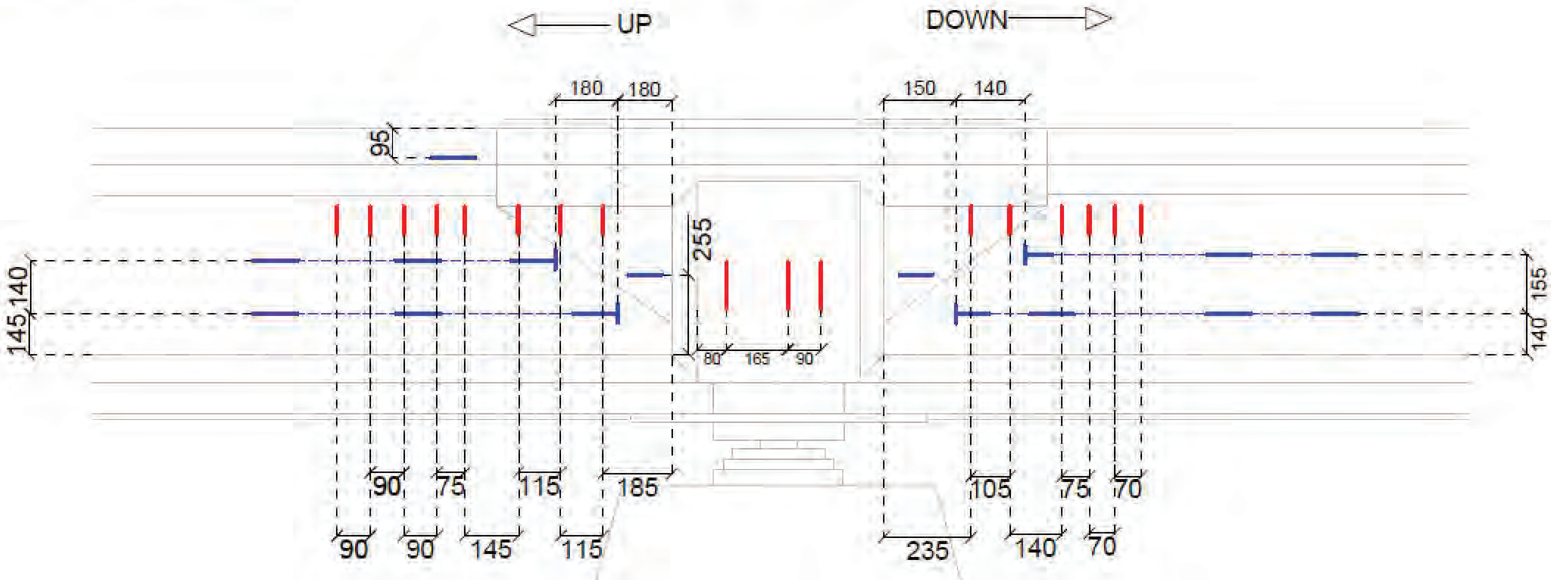
PIER 53 RIGHT, LEFT BEAM OUTSIDE ELEVATION

Note: Not to Scale. Dimensions in (mm)

ADAC/CML	CAIRNGORM FUNICULAR
File No: L/1728/18	Date: 25/10/2018

FIGURE 15

CAIRNGORM FUNICULAR RAILWAY



Outside Elevation, Up, Link Bar Cover: 33, 34, 34, 40, 34, 34, 33 and 33mm

Outside Elevation, Down, Link Bar Cover: 31, 33, 36, 35, 38 and 38mm

Inside Elevation, Up, Link Bar Cover: 39, 33, 46, 33, 31, 31, 40, 40, 32, 40, 30, 30 and 47mm

Inside Elevation, Down, Link Bar Cover: 43, 47, 30, 32, 45, 35, 44, 31 and 30mm

Note :-
Impulse radar equipment was unable to fit beneath the running rail at this location to scan the top flange.

Note: Not to Scale. Dimensions in (mm)

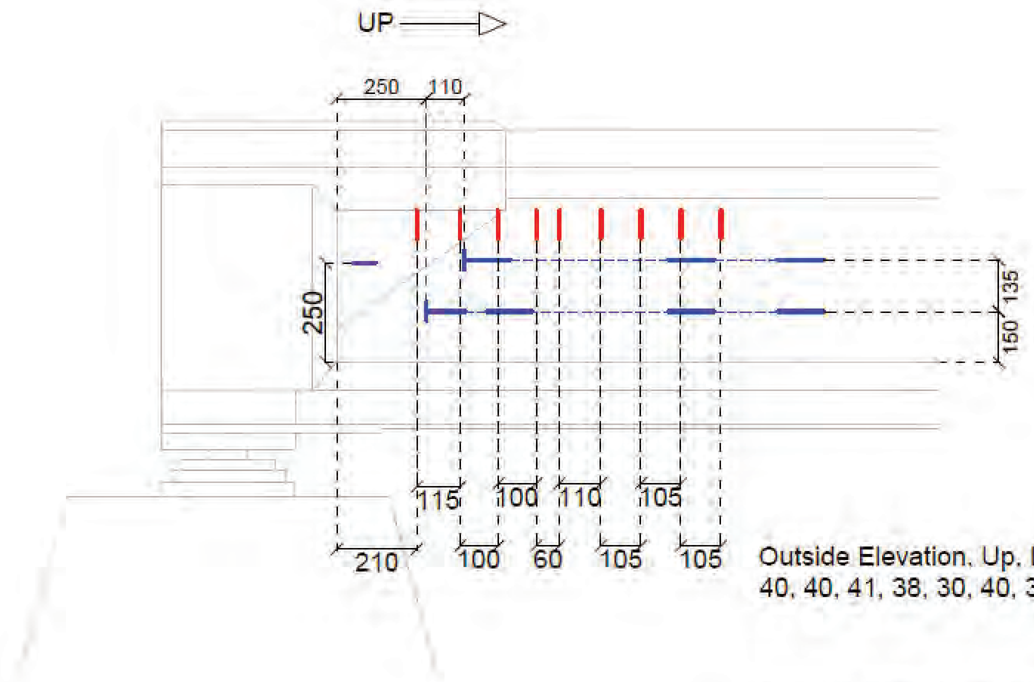
Note: Bolts in beam prevent further location of reinforcement

PIER 53 LEFT, LEFT BEAM OUTSIDE ELEVATION

ADAC/CML	CAIRNGORM FUNICULAR
File No: L/1728/18	Date: 25/10/2018

FIGURE 16

CAIRNGORM FUNICULAR RAILWAY



Outside Elevation, Up, Link Bar Cover: 42, 40, 40, 41, 38, 30, 40, 35 and 35mm

Inside Elevation, Up, Link Bar Cover: 38, 29, 40, 30, 39, 30, 43, 35, 41, 33 and 44mm

Note :-
Impulse radar equipment was unable to fit beneath the running rail at this location to scan the top flange.

Note :-
Top flange over supoort was unable to be safely accessed therfore reinforcement cover and location was not confirmed at the time of survey

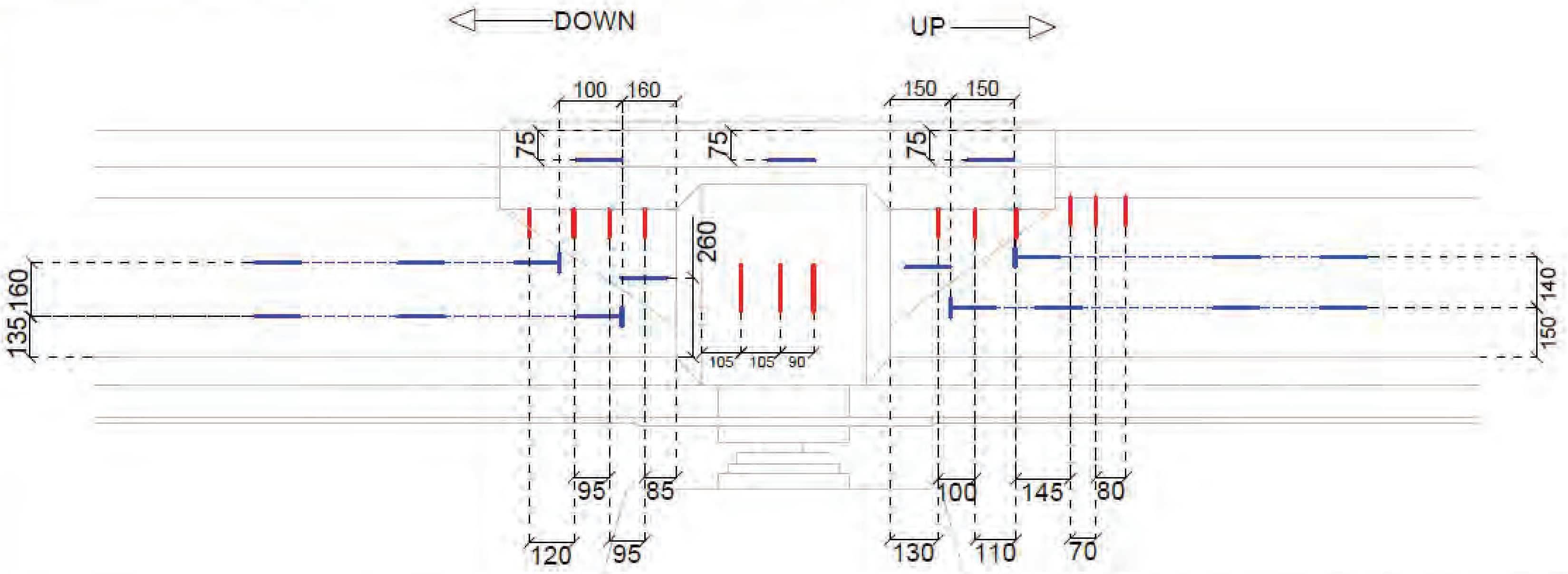
PIER 53 LEFT, RIGHT BEAM OUTSIDE ELEVATION

Note: Not to Scale. Dimensions in (mm)

ADAC/CML	CAIRNGORM FUNICULAR
File No: L/1728/18	Date: 25/10/2018

FIGURE 17

CAIRNGORM FUNICULAR RAILWAY



Outside Elevation, Up, Link Bar Cover: 35, 39, 33, 31, 29 and 32mm

Outside Elevation, Down, Link Bar Cover: 36, 38, 40 and 42mm

Inside Elevation, Up, Link Bar Cover: 41, 37, 34, 31, 44, 30 and 29mm

Inside Elevation, Down, Link Bar Cover: 48, 21, 31, 22, 27, 22 and 32mm

Note: Not to Scale. Dimensions in (mm)

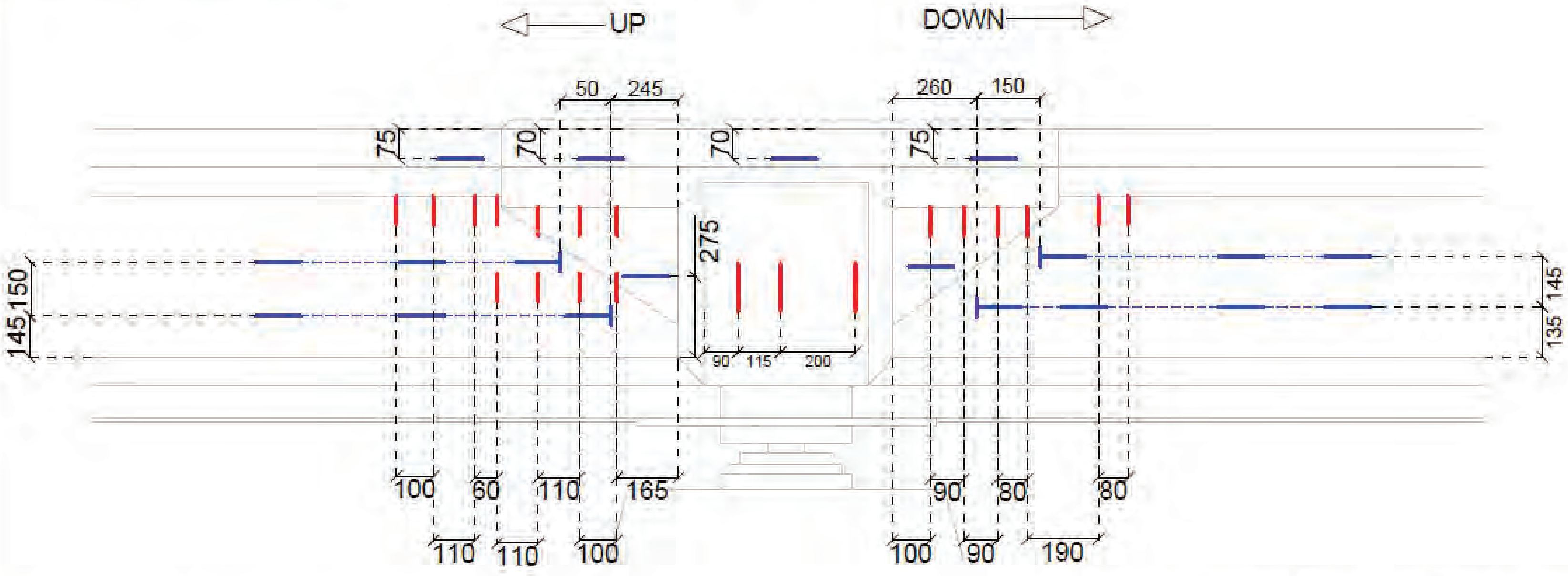
PIER 54 RIGHT, RIGHT BEAM OUTSIDE ELEVATION

Note :-
Impulse radar equipment was unable to fit beneath the running rail at this location to scan the top flange.

ADAC/CML	CAIRNGORM FUNICULAR
File No: L/1728/18	Date: 25/10/2018

FIGURE 18

CAIRNGORM FUNICULAR RAILWAY



Outside Elevation, Up, Link Bar Cover: 35, 33, 33, 35, 35, 36 and 37mm

Inside Elevation, Up, Link Bar Cover: 38, 39, 31, 29, 30, 32, 29, 42, 38 and 43mm

Outside Elevation, Down, Link Bar Cover: 41, 40, 40, 40, 39 and 45mm

Inside Elevation, Down, Link Bar Cover: 42, 34, 48, 31, 41, 32, 42, 32 and 30mm

Note: Not to Scale. Dimensions in (mm)

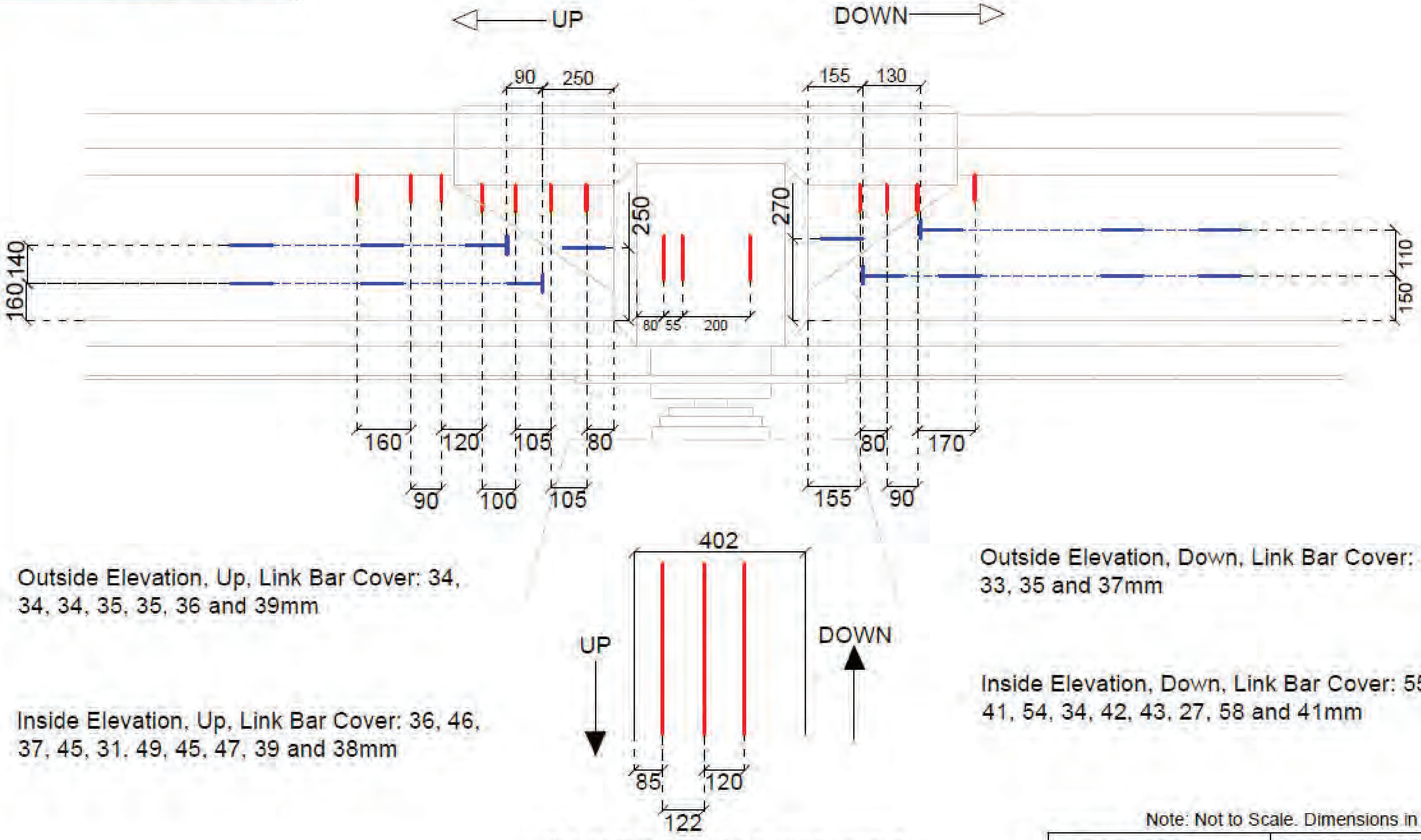
Note :-
Impulse radar equipment was unable to fit beneath the running rail at this location to scan the top flange.

PIER 54 RIGHT, LEFT BEAM OUTSIDE ELEVATION

ADAC/CML	CAIRNGORM FUNICULAR
File No: L/1728/18	Date: 25/10/2018

FIGURE 19

CAIRNGORM FUNICULAR RAILWAY



Outside Elevation, Up, Link Bar Cover: 34, 34, 34, 35, 35, 36 and 39mm

Inside Elevation, Up, Link Bar Cover: 36, 46, 37, 45, 31, 49, 45, 47, 39 and 38mm

Outside Elevation, Down, Link Bar Cover: 35, 33, 35 and 37mm

Inside Elevation, Down, Link Bar Cover: 55, 41, 54, 34, 42, 43, 27, 58 and 41mm

Pre-cast Beam Top Flange Layout
Cover: 47, 43 and 48mm

Note: Not to Scale. Dimensions in (mm)

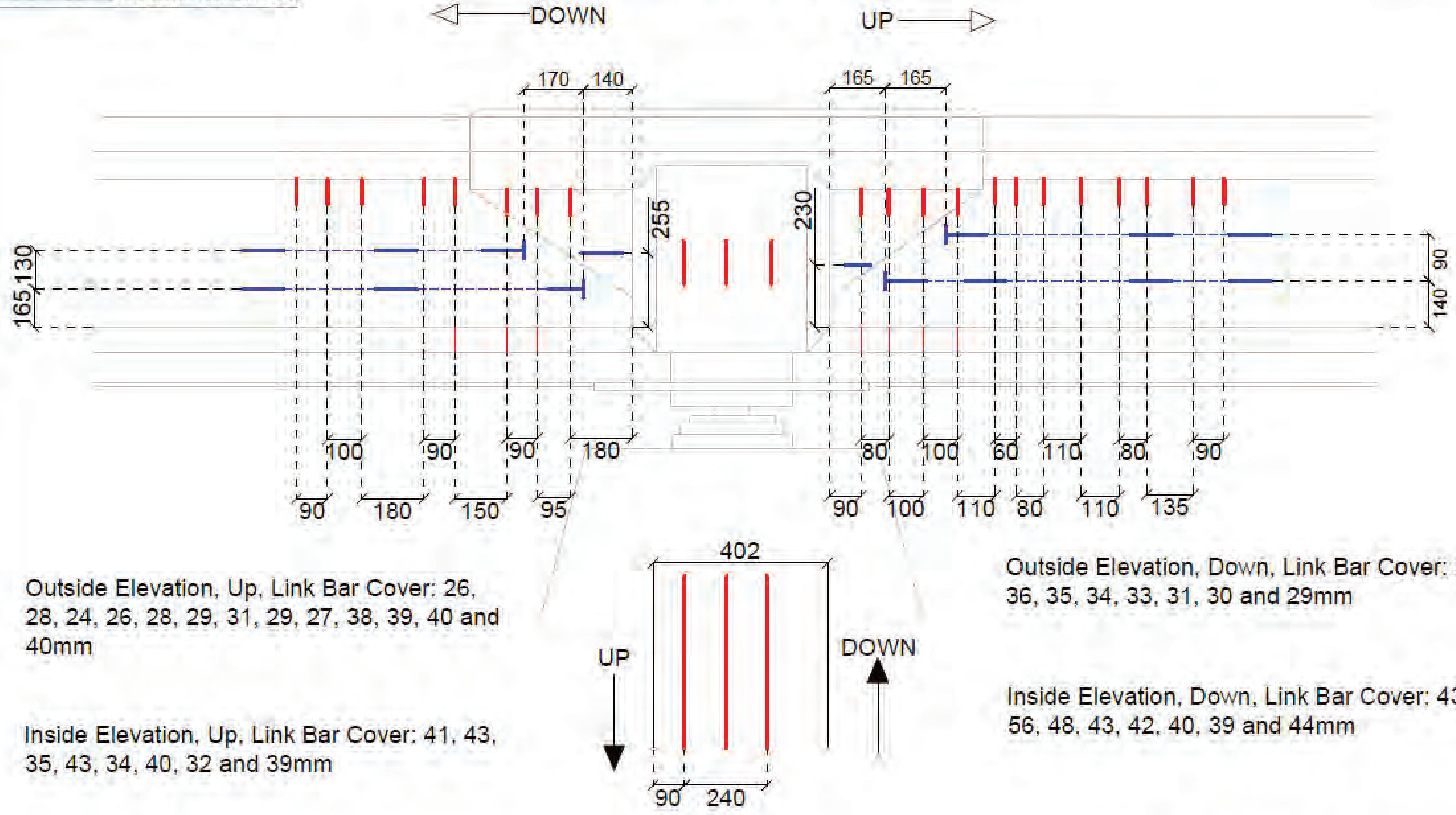
Note :-
Top flange over supoort was unable to be safely accessed therfore reinforcement cover and location was not confirmed at the time of survey

PIER 54 LEFT, LEFT BEAM OUTSIDE ELEVATION

ADAC/CML	CAIRNGORM FUNICULAR
File No: L/1728/18	Date: 25/10/2018

FIGURE 20

CAIRNGORM FUNICULAR RAILWAY



Outside Elevation, Up, Link Bar Cover: 26, 28, 24, 26, 28, 29, 31, 29, 27, 38, 39, 40 and 40mm

Inside Elevation, Up, Link Bar Cover: 41, 43, 35, 43, 34, 40, 32 and 39mm

Outside Elevation, Down, Link Bar Cover: 37, 36, 35, 34, 33, 31, 30 and 29mm

Inside Elevation, Down, Link Bar Cover: 43, 56, 48, 43, 42, 40, 39 and 44mm

Pre-cast Beam Top Flange Layout

Note: Not to Scale. Dimensions in (mm)

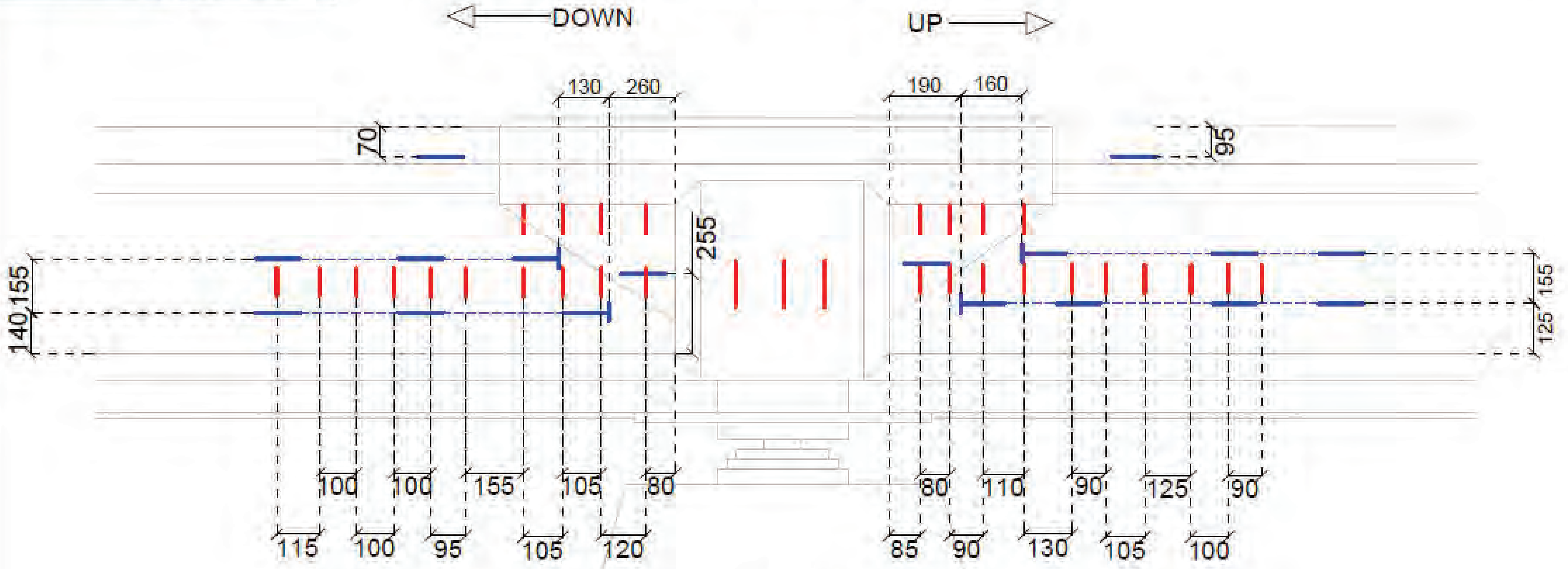
Note :-
Top flange over supoort was unable to be safely accessed therfore reinforcement cover and location was not confirmed at the time of survey

PIER 54 LEFT, RIGHT BEAM OUTSIDE ELEVATION

ADAC/CML	CAIRNGORM FUNICULAR
File No: L/1728/18	Date: 25/10/2018

FIGURE 21

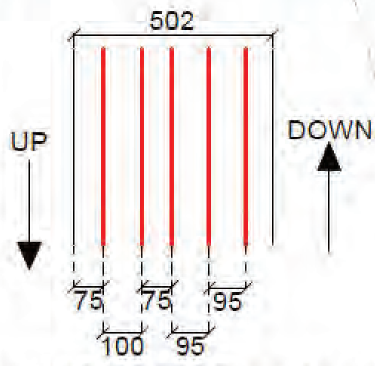
CAIRNGORM FUNICULAR RAILWAY



Outside Elevation, Up, Link Bar Cover: 27, 32, 30, 25, 23, 39, 41 and 42mm

Inside Elevation, Up, Link Bar Cover: 50, 46, 34, 33, 22, 29 and 29mm

No top bars located with covermeter



Pre-cast Beam Top Flange Layout
Cover: 54, 44, 44, 47 and 54mm

Outside Elevation, Down, Link Bar Cover: 34, 32, 36, 35, 36, 36, 35, 35, 36, 37 and 44mm

Inside Elevation, Down, Link Bar Cover: 30, 47, 25, 41, 29, 30, 43, 29, 33, 49, 35, 44, 37 and 38mm

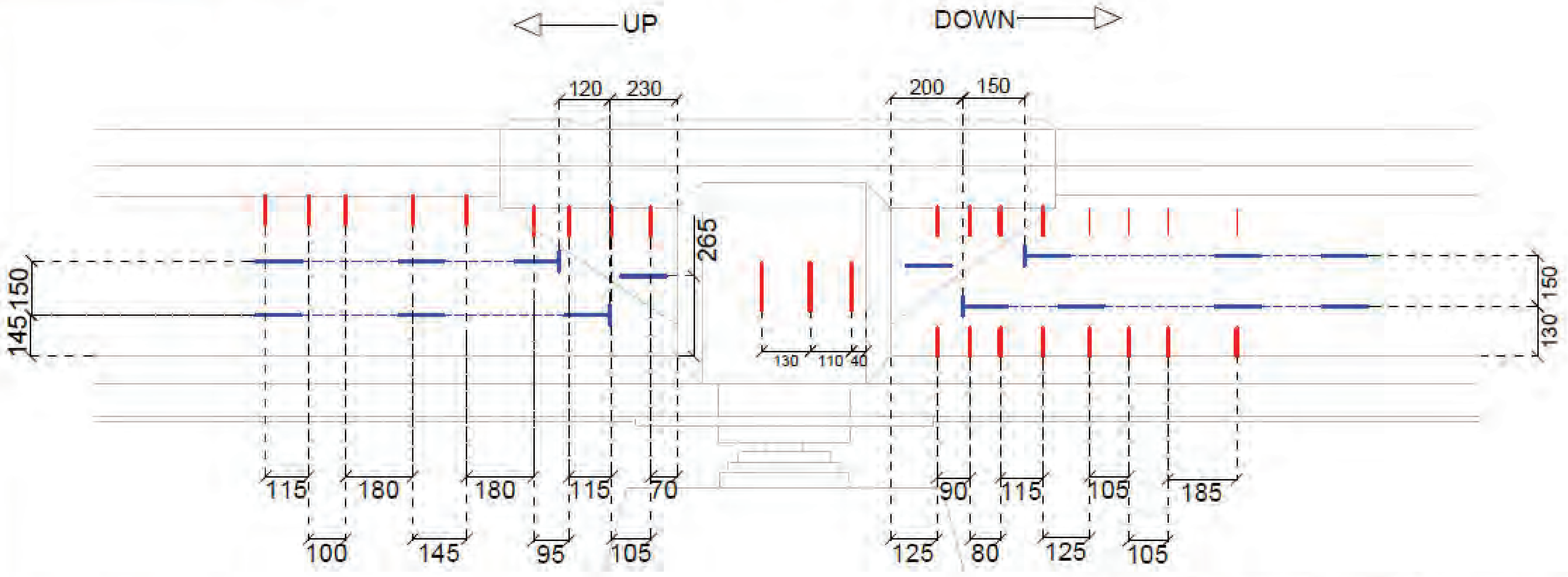
Note: Not to Scale. Dimensions in (mm)

PIER 56 RIGHT BEAM OUTSIDE ELEVATION

ADAC/CML	CAIRNGORM FUNICULAR
File No: L/1728/18	Date: 25/10/2018

FIGURE 22

CAIRNGORM FUNICULAR RAILWAY



Outside Elevation, Up, Link Bar Cover: 30, 33, 29, 28, 30, 29, 27 and 27mm

Inside Elevation, Up, Link Bar Cover: 46, 41, 42, 48, 34, 43, 35 and 52mm

No top bars located with covermeter

Note :-
Impulse radar equipment was unable to fit beneath the running rail at this location to scan the top flange.

Outside Elevation, Down, Link Bar Cover: 31, 30, 35, 34, 36, 39, 40 and 33mm

Inside Elevation, Down, Link Bar Cover: 35, 52, 46, 41, 42, 44 and 47mm

Note :-
Top flange over supoort was unable to be safely accessed therfore reinforcement cover and location was not confirmed at the time of survey

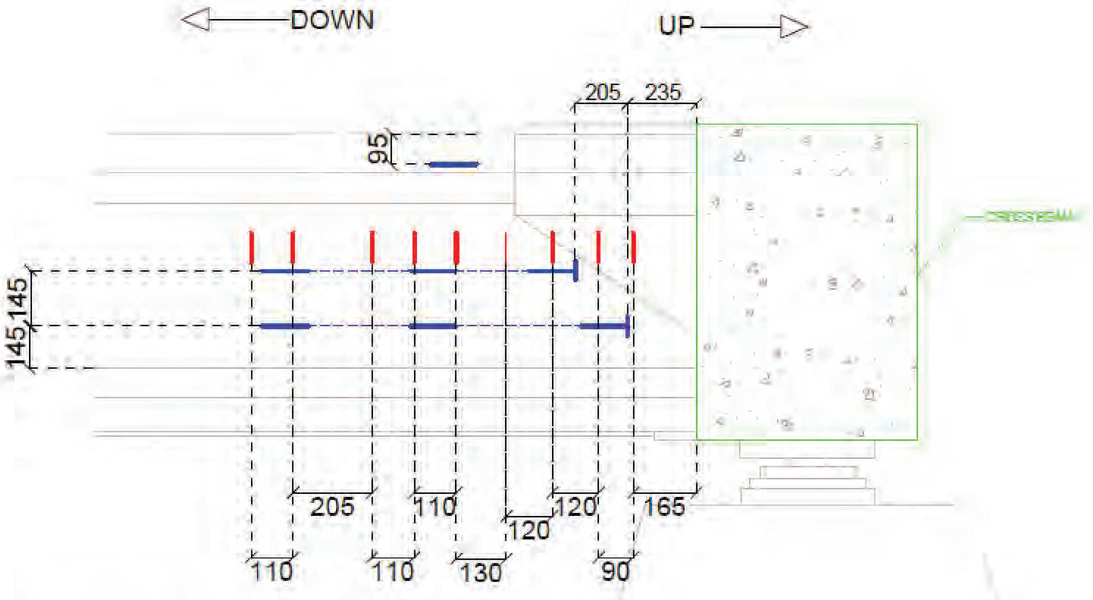
PIER 56 LEFT BEAM OUTSIDE ELEVATION
no top bar located

Note: Not to Scale. Dimensions in (mm)

ADAC/CML	CAIRNGORM FUNICULAR
File No: L/1728/18	Date: 25/10/2018

FIGURE 23

CAIRNGORM FUNICULAR RAILWAY



Link Cover Looking From 56R: 41, 32, 39, 36, 36, 30, 27, 25 and 37mm

Link Cover Looking From 56L: 42, 42, 37, 42, 35, 42, 36, 44 and 26mm

No top bars located with covermeter

Note :-
Impulse radar equipment was unable to fit beneath the running rail at this location to scan the top flange.

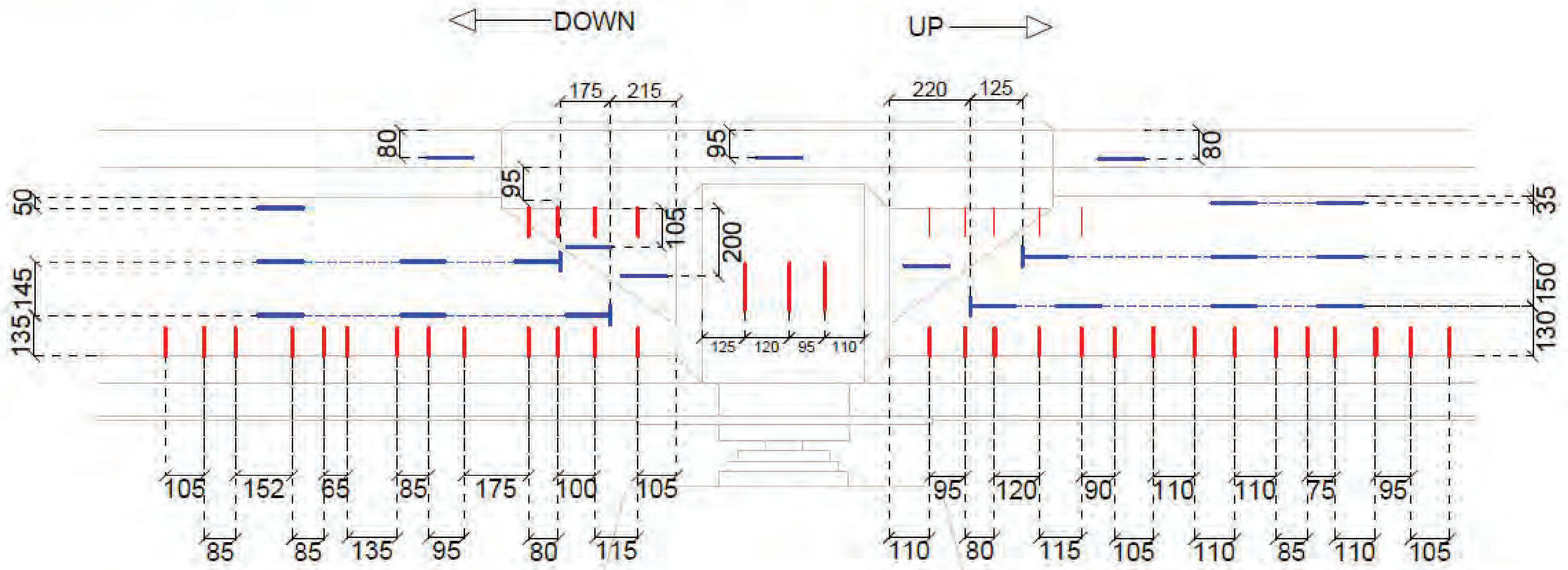
PIER 56 CENTRE BEAM ELEVATION FROM 56R

Note: Not to Scale. Dimensions in (mm)

ADAC/CML	CAIRNGORM FUNICULAR
File No: L/1728/18	Date: 25/10/2018

FIGURE 24

CAIRNGORM FUNICULAR RAILWAY



Outside Elevation, Up, Link Bar Cover: 35, 34, 35, 35, 34, 34, 34, 31, 32, 34, 35, 35, 34, 35, 33 and 34mm

Inside Elevation, Up, Link Bar Cover: 48, 38, 39, 33, 34, 35, 45, 36 and 39mm

Outside Elevation, Down, Link Bar Cover: 22, 27, 23, 25, 29, 26, 29, 24, 29, 37 and 37mm

Inside Elevation, Down, Link Bar Cover: 36, 38, 47, 36, 36, 42, 35 and 35mm

Note: Not to Scale. Dimensions in (mm)

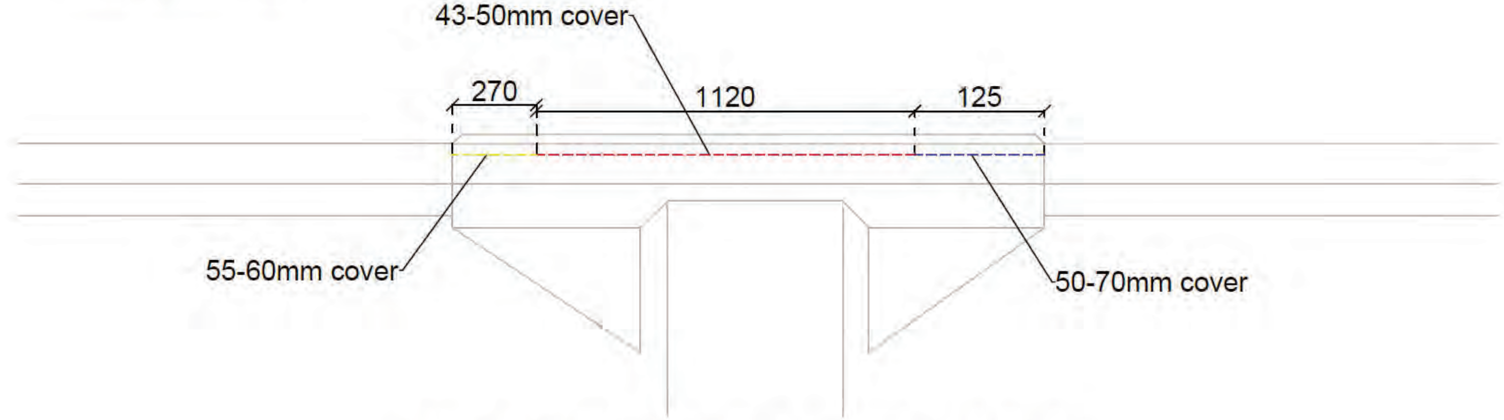
Note :-
Impulse radar equipment was unable to fit beneath the running rail at this location to scan the top flange.

PIER 69 RIGHT BEAM OUTSIDE ELEVATION

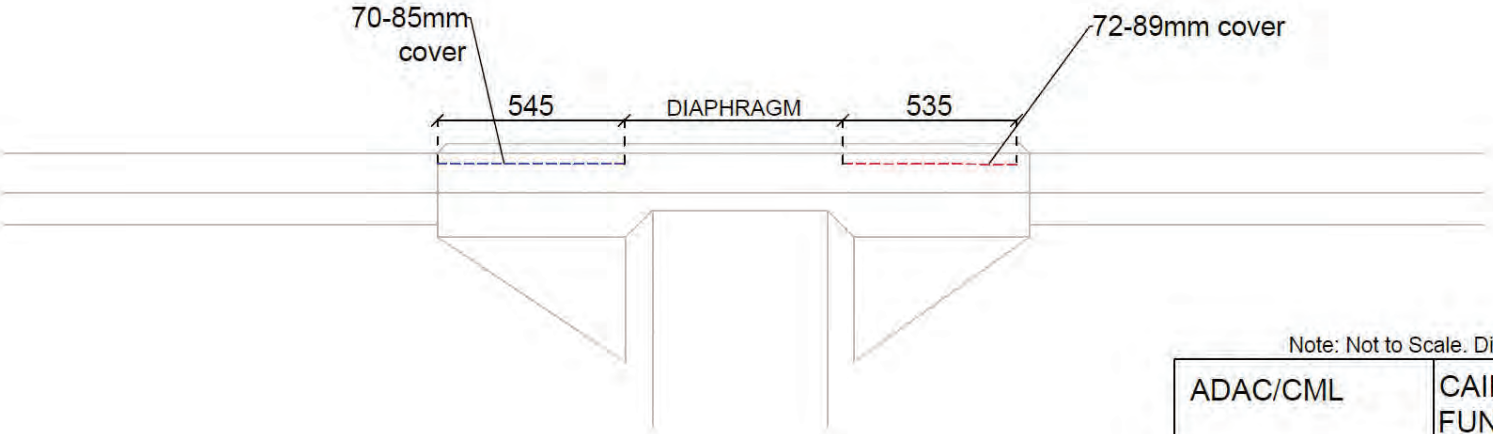
ADAC/CML	CAIRNGORM FUNICULAR
File No: L/1728/18	Date: 25/10/2018

FIGURE 25

CAIRNGORM FUNICULAR RAILWAY



PIER 69 RIGHT BEAM OUTSIDE ELEVATION- TOP COVER



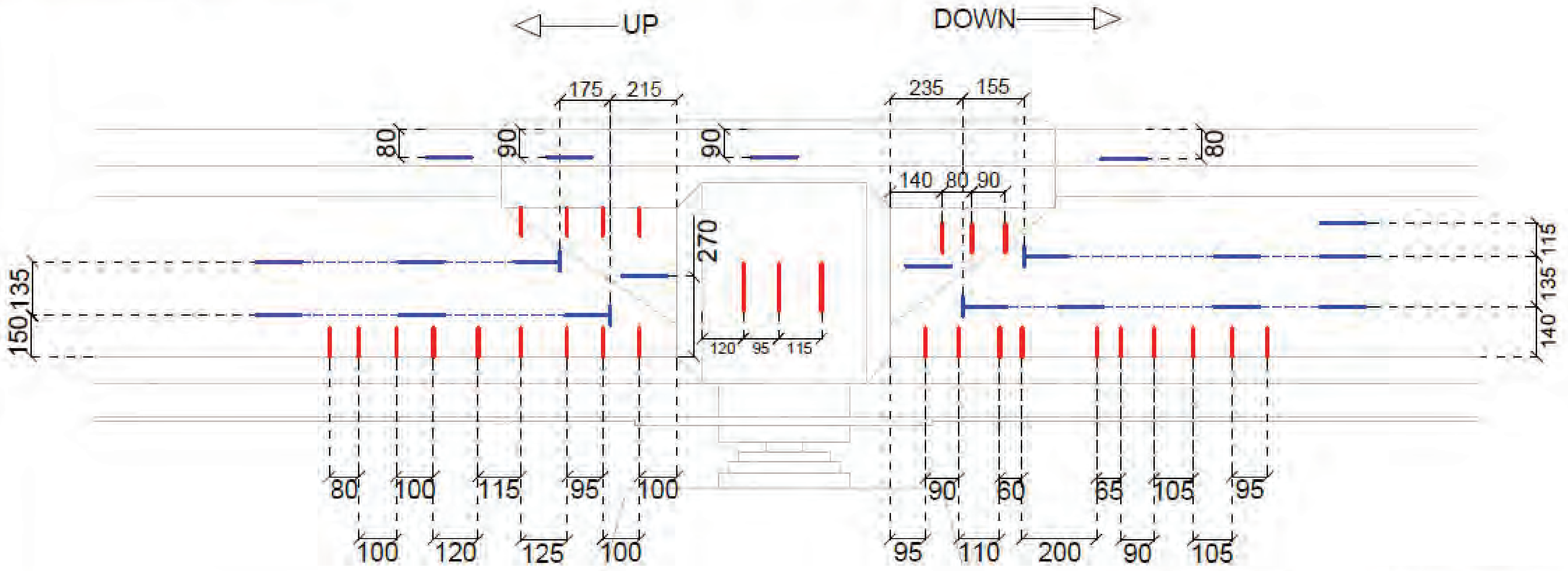
PIER 69 RIGHT BEAM INSIDE ELEVATION- TOP COVER

Note: Not to Scale. Dimensions in (mm)

ADAC/CML	CAIRNGORM FUNICULAR
File No: L/1728/18	Date: 08/11/2018

FIGURE 26

CAIRNGORM FUNICULAR RAILWAY



Outside Elevation, Up, Link Bar Cover: 32, 31, 29, 28, 24, 34, 34, 34 and 35mm

Inside Elevation, Up, Link Bar Cover: 27, 40, 27, 30, 29, 33, 32 and 38mm

Outside Elevation, Down, Link Bar Cover: 46, 40, 46, 38, 25, 30, 34, 31, 35 and 31mm

Inside Elevation, Down, Link Bar Cover: 42, 31, 38, 30, 30, 29, 36 and 27mm

Note: Not to Scale. Dimensions in (mm)

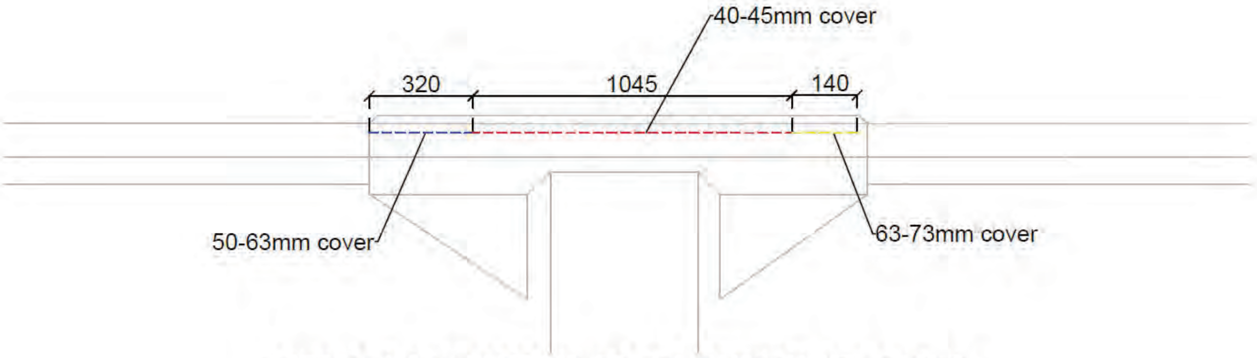
Note :-
Impulse radar equipment was unable to fit beneath the running rail at this location to scan the top flange.

PIER 69 LEFT BEAM OUTSIDE ELEVATION

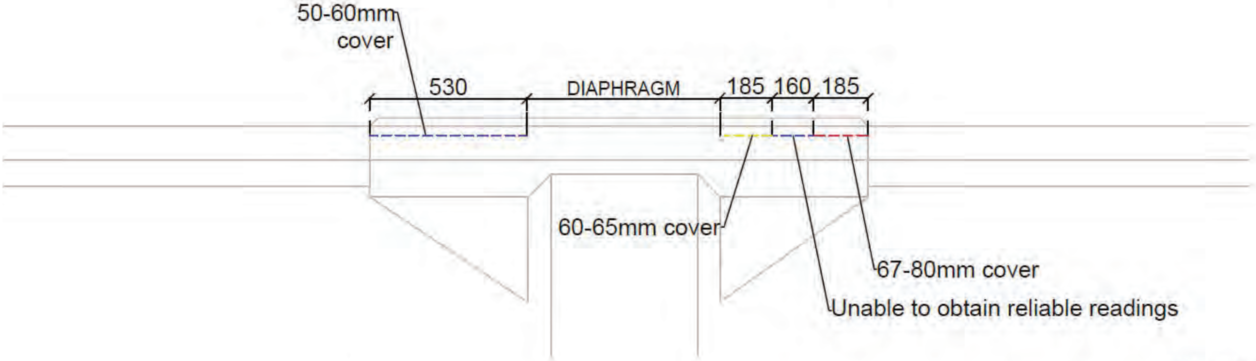
ADAC/CML	CAIRNGORM FUNICULAR
File No: L/1728/18	Date: 25/10/2018

FIGURE 27

CAIRNGORM FUNICULAR RAILWAY



PIER 69 LEFT BEAM OUTSIDE ELEVATION- TOP COVER



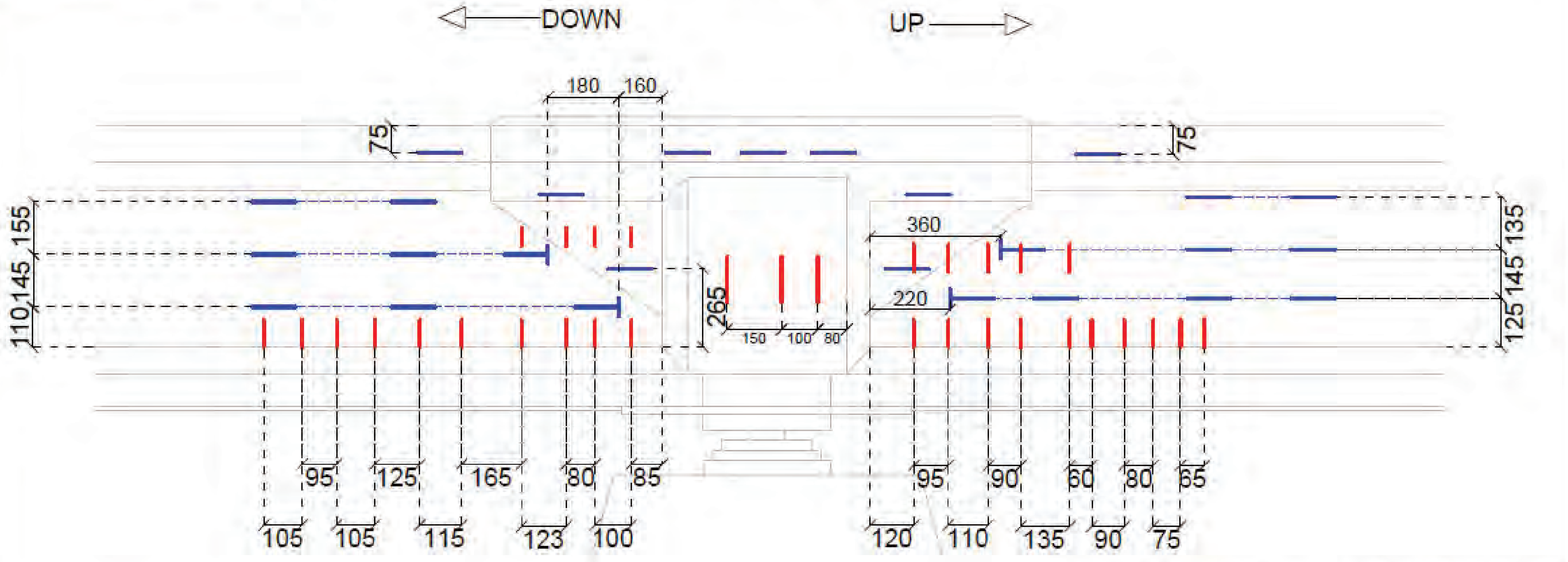
PIER 69 LEFT BEAM INSIDE ELEVATION- TOP COVER

Note: Not to Scale. Dimensions in (mm)

ADAC/CML	CAIRNGORM FUNICULAR
File No: L/1728/18	Date: 08/11/2018

FIGURE 28

CAIRNGORM FUNICULAR RAILWAY



Outside Elevation, Up, Link Bar Cover: 31, 31, 37, 31, 37, 28, 40, 29, 28, 36, 37, 36 and 37mm

Inside Elevation, Up, Link Bar Cover: 34, 44, 32, 37, 36, 37, 32, 39, 35, 40, 32 and 34mm

Outside Elevation, Down, Link Bar Cover: 36, 34, 34, 30, 33, 32, 33, 33, 32 and 31mm

Inside Elevation, Down, Link Bar Cover: 43, 33, 43, 31, 43, 30, 39, 34, 41 and 26mm

Note: Not to Scale. Dimensions in (mm)

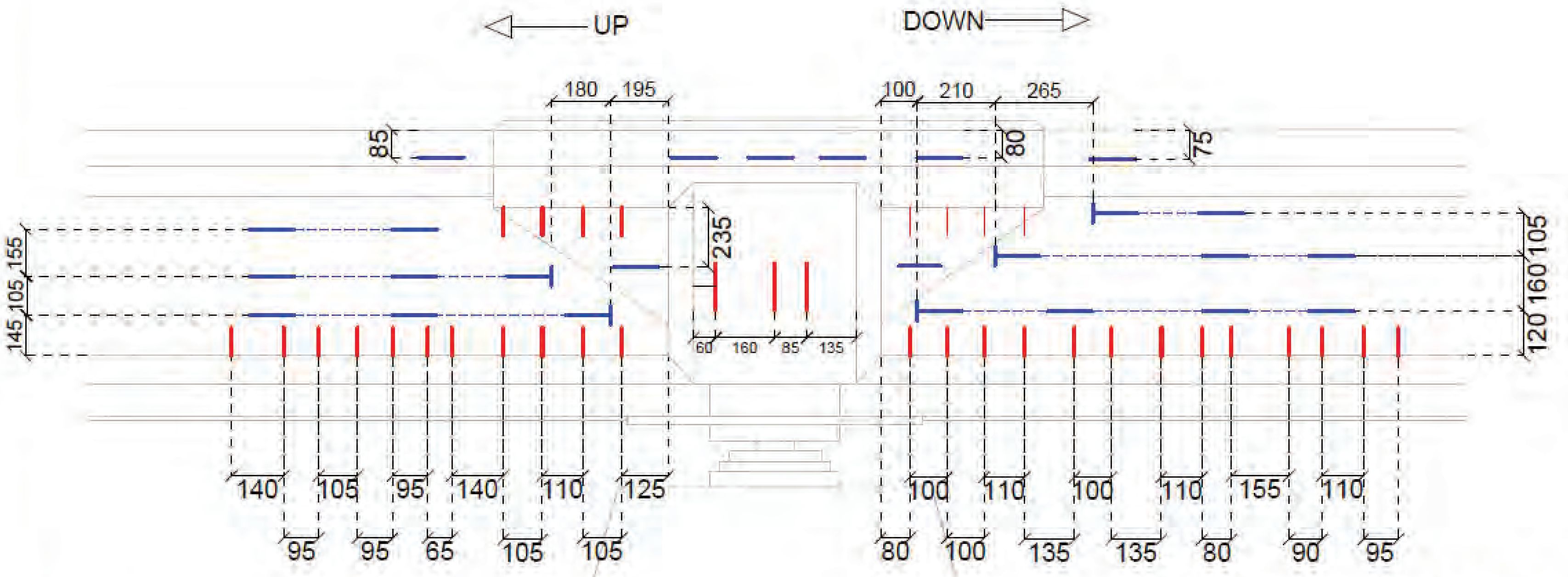
Note :-
Impulse radar equipment was unable to fit beneath the running rail at this location to scan the top flange.

PIER 80 RIGHT BEAM OUTSIDE ELEVATION

ADAC/CML	CAIRNGORM FUNICULAR
File No: L/1728/18	Date: 25/10/2018

FIGURE 30

CAIRNGORM FUNICULAR RAILWAY



Outside Elevation, Up, Link Bar Cover: 32, 34, 34, 35, 43, 31, 35, 50, 41, 41 and 42mm

Outside Elevation, Down, Link Bar Cover: 39, 36, 42, 35, 45, 31, 31, 35, 34, 32 and 32mm

Inside Elevation, Up, Link Bar Cover: 27, 42, 27, 28, 28, 30, 32, 43 and 32mm

Inside Elevation, Down, Link Bar Cover: 43, 44, 46, 38, 34, 47 and 39mm

Note: Not to Scale. Dimensions in (mm)

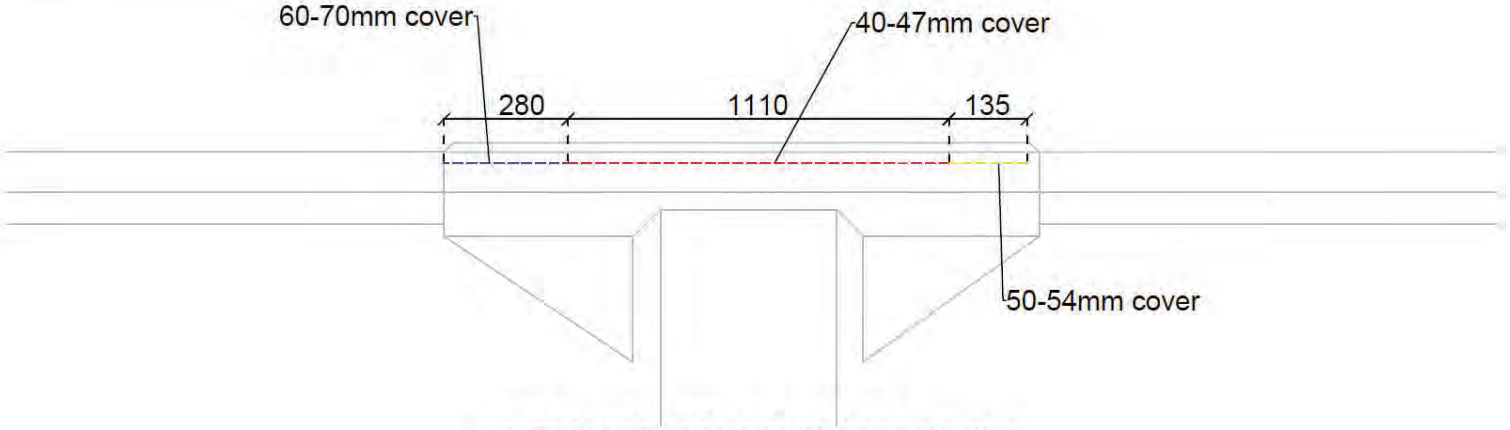
Note :-
Impulse radar equipment was unable to fit beneath the running rail at this location to scan the top flange.

PIER 80 LEFT BEAM OUTSIDE ELEVATION

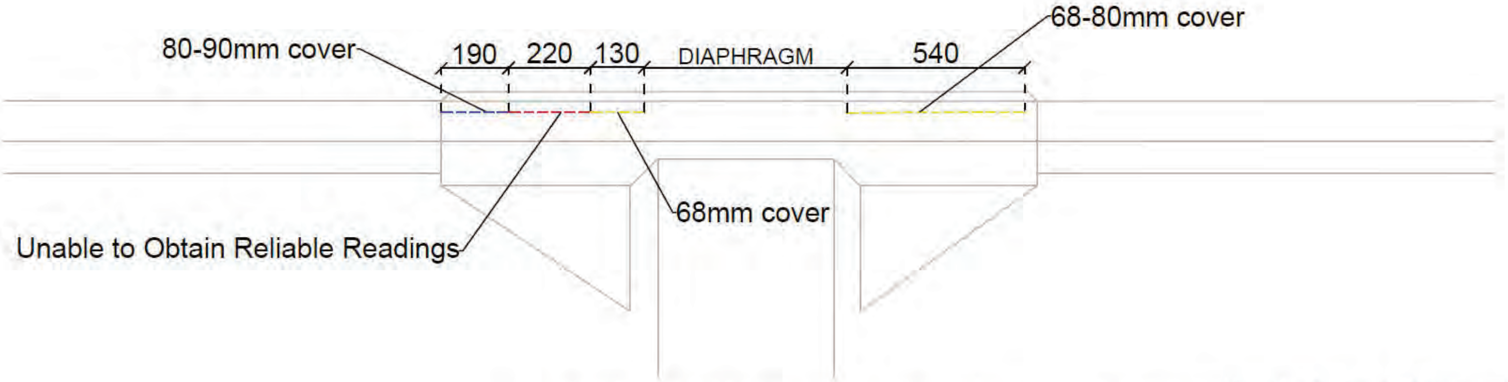
ADAC/CML	CAIRNGORM FUNICULAR
File No: L/1728/18	Date: 25/10/2018

FIGURE 31

CAIRNGORM FUNICULAR RAILWAY



PIER 80 LEFT BEAM OUTSIDE ELEVATION



PIER 80 LEFT BEAM INSIDE ELEVATION

Note: Not to Scale. Dimensions in (mm)

ADAC/CML	CAIRNGORM FUNICULAR
File No: L/1728/18	Date: 08/11/2018

APPENDIX B
PHOTOGRAPHS

Plate 1 – General view of funicular railway.

Plate 2 – General view of surface markings to Pier 9 Left Beam down.

Plate 3 – General view of surface markings to Pier 9 Left beam connection.

Plate 4 – General view of surface markings to Pier 9 Left Beam up.

Plate 5 – General view of surface markings to Pier 9 Right Beam up/down.

Plate 6 – General view of surface markings to Pier 9 Right Beam down

Plate 7 – General view of surface markings to Pier 9 Right Beam up

Plate 8 – General view of surface markings to Pier 9 Right Beam up/down.

Plate 9 – General view of surface markings to Pier 22 Right Beam down

Plate 10 – General view of surface markings to Pier 22 Right Beam up

Plate 11 – General view of surface markings to Pier 22 Left Beam up/down

Plate 12 – General view of surface markings to Pier 22 Left Beam up

Plate 13 – General view of surface markings to Pier 22 Left Beam down

Plate 14 – General view of surface markings to Pier 49 Left Beam up

Plate 15 – General view of surface markings to Pier 49 Left Beam down

Plate 16 – General view of surface markings to Pier 49 Right Beam up

Plate 17 – General view of surface markings to Pier 49 Right Beam down

Plate 18 – General view of surface markings to Pier 51 Left Beam up

Plate 19 – General view of surface markings to Pier 51 Left Beam down

Plate 20 – General view of surface markings to Pier 51 Right, Beam right support up

Plate 21 – General view of surface markings to Pier 51 Right Beam down

Plate 22– General view of surface markings to Pier 53 Left Beam, left support up

Plate 23– General view of surface markings to Pier 53 Left Beam, Left support down

Plate 24 - General view of surface markings to Pier 53 Right Beam, left support up.

Plate 25 - General view of surface markings to Pier 53 Right Beam, right support down.

Plate 26 - General view of surface markings to Pier 54 Right Beam down

Plate 27 - General view of surface markings to Pier 54 Right Beam, right support up

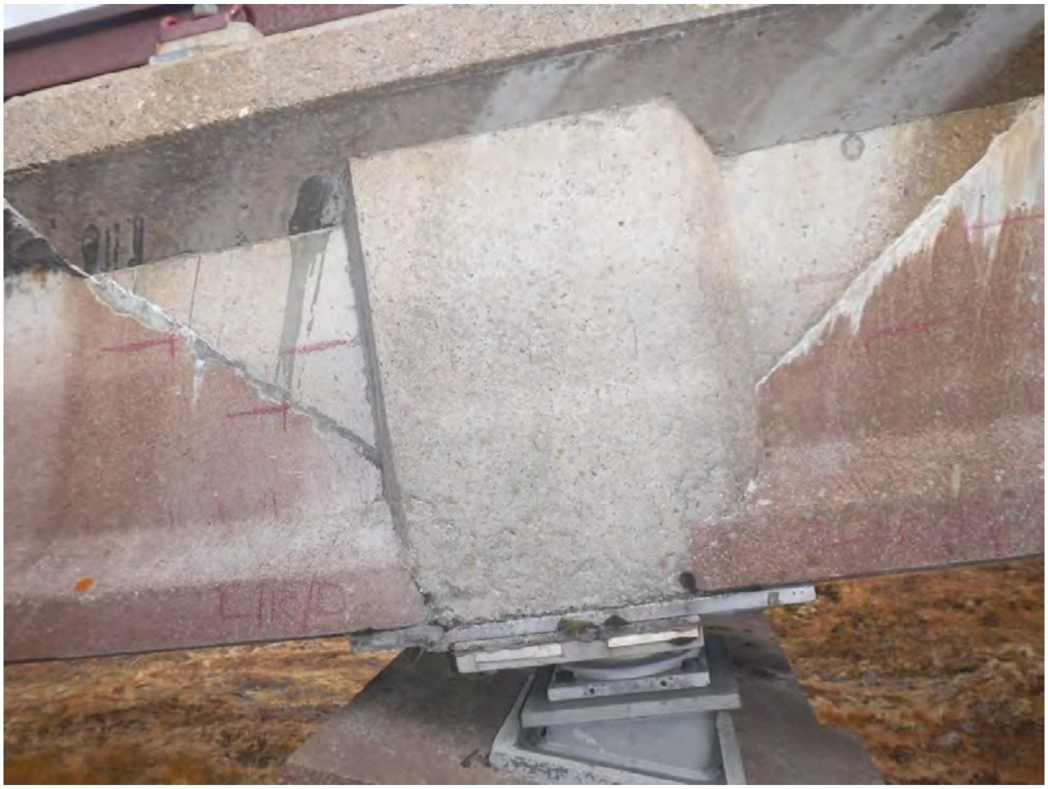


Plate 28 - General view of surface markings to Pier 54 Right Beam, left support up/down

Plate 29 - General view of surface markings to Pier 54 Right Beam, left support down

Plate 30 - General view of surface markings to Pier 56 Right Beam up

Plate 31 - General view of surface markings to Pier 56 Right Beam down

Plate 32 - General view of surface markings to Pier 56 Left Beam down

Plate 33 - General view of surface markings to Pier 56 Left Beam up

Plate 34 - General view of surface markings to Pier 69 Right Beam up

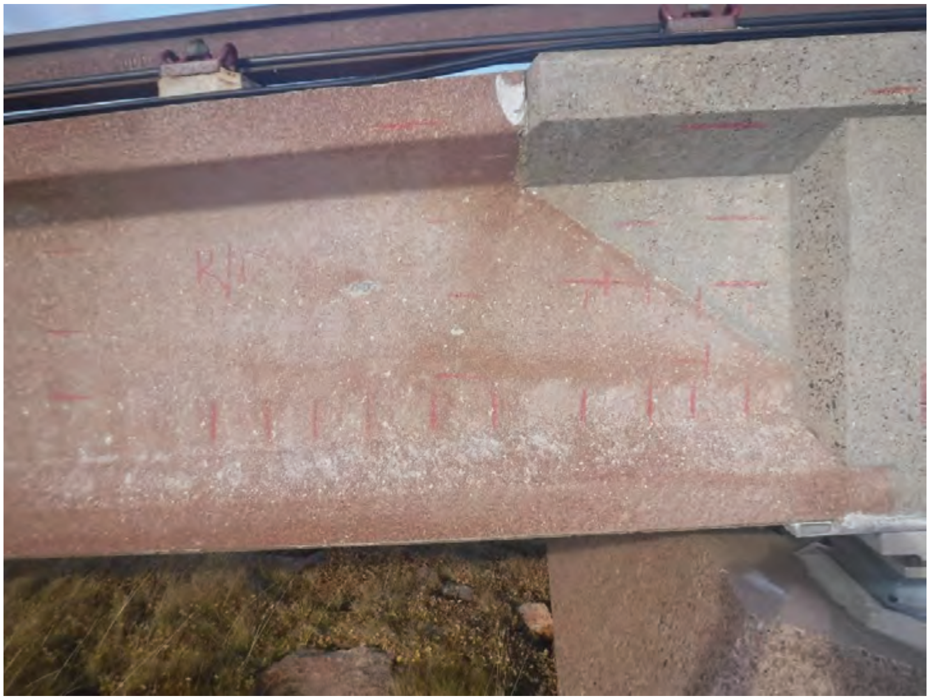


Plate 35 - General view of surface markings to Pier 69 Right Beam down

Plate 36 - General view of surface markings to Pier 69 Right Beam down

Plate 37 - General view of surface markings to Pier 69 Left Beam Up

Plate 38 - General view of surface markings to Pier 69 Left Beam Up

Plate 39 - General view of surface markings to Pier 69 Left Beam down

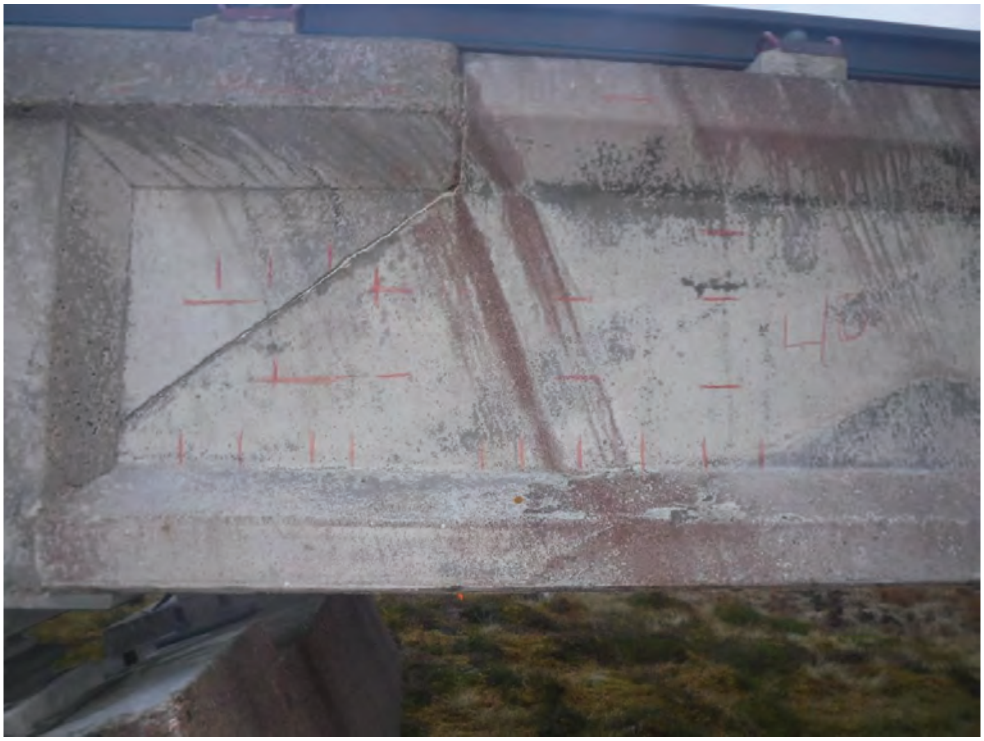


Plate 40 - General view of surface markings to Pier 69 Left Beam down

Plate 41 - General view of surface markings to Pier 80 Left Beam down

Plate 42 - General view of surface markings to Pier 80 Left Beam up

Plate 43 - General view of surface markings to Pier 80 Right Beam down

Plate 44 - General view of surface markings to Pier 80 Right Beam down

Plate 45 - General view of surface markings to Pier 80 Right Beam down

Plate 46 - General view of surface markings to Pier 80 Right Beam up

Plate 47 - General view of surface markings to Pier 80 Right Beam up

Plate 48 - General view of surface markings to Pier 80 Right Beam up/down

No page 63 received.

Appendix E Intrusive Investigation Report

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HIGHLANDS AND ISLANDS ENTERPRISE

CAIRNGORM FUNICULAR RAILWAY - REINFORCEMENT CONTINUITY INVESTIGATION

TECHNICAL NOTE: DESCRIPTION AND ANALYSIS OF INTRUSIVE INVESTIGATIONS

CONTENTS

1 Introduction

Past inspections of the Cairngorm funicular railway by ADAC Structures Ltd. identified that cracks between the in-situ joints and precast rail support beams opened at some piers during passage of the rail carriage. In particular, piers 22 and 56 were identified as having significant crack widths (~0.5 to 1 mm), although it is noted that the inspections were not comprehensive, and similar crack widths may also occur at other locations.

These crack observations led to concerns that there may be a lack of continuity in the top flange reinforcement connections within the in-situ joints. Where the structure is curved in plan, the top flange reinforcement is connected via a combination of grouted and threaded couplers. Where the structure is straight in plan, the top flange reinforcement is connected via a lap splice. Prior to intrusive investigations it was assumed that coupled connections are used at both piers 22 and 56 and therefore were of particular concern.

PROJECT NO.	DOCUMENT NO.
A116993	TN-03-004

VERSION	DATE OF ISSUE	DESCRIPTION	PREPARED	CHECKED	APPROVED
00	December 2018	First Issue	[redacted]	[redacted]	[redacted]

This report presents the results from investigations and tests (specified in COWI report A116993-SP03) that were undertaken to assess the continuity of the top flange reinforcement. Background information on the test program is available in COWI report A116993-SP03.

2 Test Description

The tests used dial gauges to measure deformations in the in-situ joint region during controlled passage of a rail carriage with a known weight. The dial gauges had a resolution of 0.01 mm. Two types of deformations were measured:

1. Crack widths at the in-situ to precast interface (see Figure 1). The dial gauge connection points were two steel angles fixed to the concrete on either side of the interface. These measurements were conducted at 13 No. locations. At three of the in-situ joints, measurements were taken on both the downhill and uphill interfaces of the joint, i.e. 2 No. locations at each joint. These locations allowed the total cracking deformation across the two interfaces to be assessed.

2. Reinforcement deformation across a gauge length of approximately half the in-situ joint length (see Figure 2). The dial gauge connection points were clamped directly onto reinforcement exposed by hydro demolition of the cover concrete. These tests were conducted at piers 22 and 56. At pier 22, the gauge length spanned across a grouted coupler connection. At pier 56, the gauge length spanned across a threaded coupler connection but not a grouted coupler.

Figure 1 Type 1: General setup for crack width measurement tests

Figure 2 Type 2: General setup for reinforcement deformation tests

The tests were conducted in two phases. In the first phase, carried out 22-23rd November 2018, only non-invasive crack width tests were conducted (test type 1). A carriage with a 4 tonne kentledge was used, giving a total carriage weight of 18,900 kg (as per the Doppelmayr Operations and Maintenance Manual). The full list of the first phase test locations is given in Table 1.

Table 1 List of tests conducted in phase 1 (all type 1)

Test No.	Pier	Beam	End	Adjacent Beam Type
1	9	Left	Downhill	1
2	9	Right	Downhill	1
3	16	Left	Uphill	2
4	20	Left	Uphill	2
5	21	Left	Downhill	2
6	21	Left	Uphill	2
7	22	Right	Downhill	2
8	22	Right	Uphill	2
9	23	Left	Downhill	2
10	23	Left	Uphill	2
11	24	Left	Downhill	2

In the second phase, carried out 29th November – 1st December 2018, reinforcement deformation was directly measured (test type 2). Interface crack width tests (test type 1) were also conducted at the same locations to allow for comparison. An empty carriage was used, with a total weight of 14,900 kg (as per the Doppelmayr Operations and Maintenance Manual). The second phase test locations are listed in Table 2.

Table 2 List of tests conducted in phase 2

Test No.	Pier	Beam	End	Adjacent Beam Type	Test type	Gauge length (mm)
12	22	Left	Uphill	2	2 (steel)	680
13	22	Left	Uphill	2	1 (conc.)	200
14	56	Right	Uphill	3	2 (steel)	1050
15	56	Right	Uphill	3	1 (conc.)	200

Videos of the dial gauge readings were taken during passage of the rail carriage. Audial signals were given to note the approximate position of the carriage at various times in the videos.

3 Test Data

The videos of the dial gauge readings were processed to determine the displacements, as well as the approximate position of the rail carriage at the time the maximum displacements occurred. Figure 3 shows the relationship between displacement and carriage location for Test No. 1 (see Table 1). The results are generally indicative of what would be expected to occur in a continuous beam, with compressive displacements occurring when the carriage is one span away and tensile displacements occurring when the carriage is on adjacent spans.

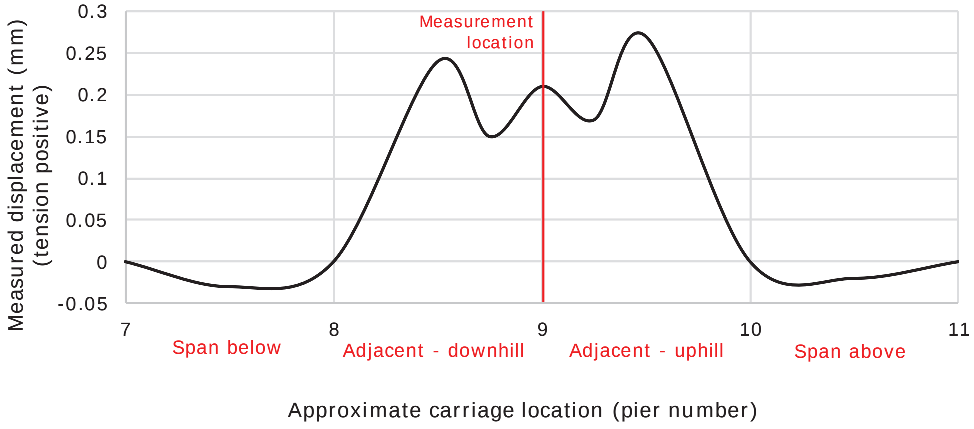


Figure 3 Displacement versus carriage position for Test No. 1 (typical of all tests)

Only maximum compressive and tensile displacements are here reported for the remaining tests. This information is given in Table 3. Several observations can be made from the data:

- For all tests, the carriage location at the time of maximum displacements is consistent with the response of a continuous beam.
- Compressive displacements are smaller than tensile displacements in all cases, consistent with the relatively smaller sagging moment than hogging moment that develops at supports in continuous beams.
- Maximum tensile interface displacements were similar (between 0.14 and 35 mm) in most cases but with two exceptions: Test No. 8 (0.05 mm) and Test No. 15 (0.6 mm).
- Tests on the reinforcement (Test Nos. 12 & 14) gave similar results to equivalent tests across the interfaces (Test Nos. 13 & 15), despite having significantly longer gauge lengths.

Table 3 Maximum measured displacements for all tests

Test No.	Maximum compressive displacement (mm)	Carriage span when max compressive displacement measured	Maximum tensile displacement (mm)	Carriage span when max tensile displacement measured
1	0.03	7 (below)	0.27	9 (adjacent - uphill)
2	0.01	10 (above)	0.15	9 (adjacent - downhill)
3	0.06	17 (above)	0.32	15 (adjacent - downhill)
4	0.03	21 (above)	0.14	19 (adjacent – downhill)
5	0.04	19 (below)	0.21	21 (adjacent – uphill)
6	0.04	19 (below)	0.14	20 (adjacent – downhill)
7	0.02	23 (above)	0.35	22 (adjacent – uphill)
8	0.03	23 (above)	0.05	21 (adjacent – downhill)
9	0.04	21 (below)	0.29	23 (adjacent – uphill)
10	0.06	24 (above)	0.24	22 (adjacent – downhill)
11	0.04	25 (above)	0.25	24 (adjacent – uphill)
12*	0.08	23 (above)	0.17	21 (adjacent – downhill)
13	0.06	23 (above)	0.2	21 (adjacent – downhill)
14*	0.02	57 (above)	0.55	56 (adjacent – uphill)
15	0.04	54 (below)	0.6	56 (adjacent – uphill)

** Denotes measurement as type 2*

Pairs of tests conducted on both the uphill and downhill interface of the same in-situ joint (Test Nos. 5 & 6, 7 & 8, and 9 & 10) allow for the total displacement due to interface cracking on both ends of the joint to be assessed. The maximum tensile displacements for the relevant in-situ joints are listed in Table 4. It is noted that the downhill and uphill interface tests were not ran concurrently, so the "total" displacements are taken as the sum of the maximum displacements measured in two separate tests. However, this is expected to cause little error as the response is elastic and similar maximum tensile displacements are obtained regardless which adjacent span the carriage is on (see Figure 3).

Table 4 Maximum tensile displacement across both in-situ-to-precast interfaces

In-situ joint	Downhill interface max. tensile displacement (mm)	Uphill interface max. tensile displacement (mm)	Total tensile displacement across both interface cracks (mm)
21 Left	0.21	0.14	0.35
22 Right	0.35	0.05	0.4
23 Left	0.29	0.24	0.53

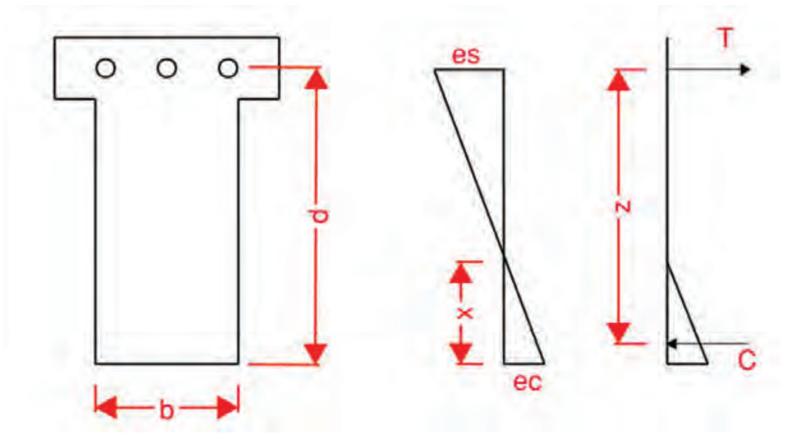
The following observations can be made based on the data in Table 4:

- Similar total tensile displacements were obtained at the three in-situ joints. The fact that the lowest displacements were observed at pier 21 is likely due to the observation of an additional crack within the in-situ joint, which was not picked up by the dial gauges.
- Pier 22 exhibited considerably different crack widths at the downhill and uphill interfaces, with almost all of the deformation occurring at the downhill interface.

4 Theoretical Deformations

Calculations were undertaken to assess the expected deformations due to passage of the rail carriage only, i.e. ignoring deformations due to permanent actions. Moments at pier centres due to the rail carriage loads were determined using analysis models previously developed as part of the viaduct structural appraisal (COWI report A116993-RP01). The analysis models used a gross sectional stiffness $E_c I_g$, where E_c was calculated as per BD 44/15.

Theoretical tensile strains in the top flange reinforcement (3 No. T32 bars in all cases) due to the hogging moments were assessed using a sectional analysis of the in-situ joint cross-section (Figure 4). A triangular concrete stress block and a constant section width of 340 mm was assumed. In all cases, maximum concrete stresses were approximately 5 MPa or below, indicating that the triangular stress block assumption was appropriate.



$$\varepsilon_s = \varepsilon_c(d-x)/x \qquad T = E_s\varepsilon_s * A_s = \frac{xbE_c\varepsilon_c}{2} = C \qquad z = d - \frac{x}{3}$$

Figure 4 Basis of reinforcement strain calculations

Theoretical reinforcement deformations were calculated directly by multiplying the theoretical tensile strains by the test gauge length. Theoretical deformations at the in-situ joint interfaces were calculated by multiplying the theoretical tensile strains in the top flange reinforcement by half of the in-situ joint length (i.e. 1550 mm / 2 = 775 mm). The calculation therefore assumes that all deformation within the in-situ concrete is concentrated at the interfaces, which is deemed appropriate due to a general lack of observations of cracks within the joint.

Theoretical crack widths assuming simply supported beams with no continuity were also calculated. These calculations used standard elastic beam deflection formulae to determine the theoretical rotations at the end of a simply supported beam due to a rail carriage centred on the span. A gross sectional stiffness E_cI_g was used, which is expected to give a conservative (low) estimate of rotation relative to the actual cracked condition of the structure. Rotations were converted to crack widths by assuming a centre of rotation at the beam centroid.

Table 5 compares the measured maximum tensile displacements against the theoretical maximum tensile displacements for both a continuous and simply supported structure. It is evident that the theoretical values corresponding to a continuous structure are significantly closer to the measured values than those corresponding to a simply supported structure. While the calculation procedures employed here have limitations, e.g. no consideration of concrete cover depths, it is believed that they are sufficient to indicate continuity of the top flange reinforcement.

Table 5 Measured versus theoretical displacements – continuous and simply supported

Test No.	Measured max. tensile displacement (mm)	Theoretical max. tensile displacement – continuous (mm)	Theoretical max. tensile displacement – simply supported (mm)
1	0.27	0.28	1.8
2	0.15	0.28	1.8
3	0.32	0.28	1.7
4	0.14	0.28	1.7
5	0.21	0.28	1.7
6	0.14	0.28	1.7
21 Left total (5+6)	0.35	0.56	3.4
7	0.35	0.29	1.7
8	0.05	0.29	1.7
22 Right total (7+8)	0.40	0.57	3.4
9	0.29	0.29	1.7
10	0.24	0.29	1.7
23 Left total (9+10)	0.53	0.59	3.4
11	0.25	0.28	1.7
12*	0.17	0.20	n/a
13	0.2	0.22	1.3
14*	0.55	0.42	n/a
15	0.6	0.31	1.2

** Denotes measurement as type 2*

5 Condition Assessment of In-Situ Joint Reinforcement and Couplers

After completion of the invasive tests at piers 22 and 56, additional hydro demolition of the in-situ joint cover concrete was carried out to enable visual inspection of the grouted couplers. Figure 5 shows a fully exposed grouted coupler at pier 22. The coupler was 500 mm long and had an internal diameter of 75 mm. The end of the coupler on the exterior side of the joint had a threaded connection, consistent with the design drawings. The grout inside the coupler was visually observed to be free from voids and appeared in good condition. Grout visible at the vent holes located on the upper surface of the coupler imply that the coupler was fully grouted.

It was observed that a section of the coupler had been cut out to allow for installation of the HALFEN channel that forms part of the rail plinth connection detail. A close-up view of the cut out is shown in Figure 6. This confirmed some evidence of site modifications to couplers seen in historical video footage.

Figure 5 Exposed grouted coupler at pier 22

Figure 6 Close-up view of grouted coupler cut-out around HALFEN channel at pier 22

The concrete breakout at pier 56 was contained between two rail plinths and therefore did not expose the full length of the grouted couplers. However, the breakout did expose nearly the full width of the top flange, as shown in Figure 7. This breakout allowed the staggered coupler arrangement shown in the design drawings to be confirmed albeit in a reversed format to that indicated on the drawings. At this location two grouted couplers and one threaded coupler at the lower end of the in-situ joint and one grouted coupler and two threaded couplers at the upper end were observed. The breakout also confirmed the additional 2 No. T25 top flange longitudinal reinforcement used in "type 3" rail support beams are terminated prior to the in-situ joints; they are not continuous (Figure 8).

Figure 7 Breakout at pier 56 showing staggered coupler arrangement

Additional observations were also made possible from the invasive investigation. Shear links within the in-situ joint were different to design drawings, being anchored around only the middle bar of top flange reinforcement where two grouted couplers were present (e.g. as shown in Figure 7). Where only a single grouted coupler is present, the shear links were as shown in the design drawings, but manually bent to accommodate the coupler diameter.

Shear links were observed as pairs in the precast concrete. No shear links were observed in the arrangement indicated on the drawings for the in-situ concrete stitch. No links were present connecting the outer bar and coupler to the other T32 longitudinal bars within the footprint of the coupler. Only one link was observed in the in-situ concrete over the area of the entire breakout at pier 22.

"DENSO" type tape was found to be plugged into the end of the grouted coupler (Figure 9), presumably to bung the end for grouting purposes. This tape had been left in-situ and thus acted as a de-bonder to the localised open end of a coupler and the in-situ concrete. US patent number 5.261.198 was observed on the side of a coupler. The coupler dimensions were similar to the "DB40" coupler as referenced on original design drawings. The annulus within the coupler of internal dimeter 75mm for a 32mm diameter bar is quite large for a grouted application.

Figure 8 *Breakout at pier 56 showing termination of T25 bar before the in-situ concrete interface*

Figure 9 *DENSO tape at grouted coupler end*

6 Conclusions and Recommendations

The following conclusions can be drawn based on the results of the continuity study:

- The top flange reinforcement is believed to generally have good continuity through the in-situ joints, due to the following:
    - The locations of the carriage at times of maximum tensile and compressive measured displacements were consistent with what would occur in a continuous beam.
    - Theoretical deformation calculations for a continuous structure had significantly better agreement with measured results than those for a simply supported structure.
- While the observed reinforcement detailing within the in-situ joint was in most cases consistent with the design drawings, some differences and evidence of poor workmanship were observed. Namely, shear link shape and location, HALFEN channel cut-outs, and T25 bar curtailment.

It is recommended that analysis on the viaduct appraisal report (COWI report A116993-RP01 v1) is updated to assume the top flange reinforcement as continuous over the piers.