Llywodraeth Cymru / Welsh Government

A487 New Dyfi Bridge

Environmental Statement – Volume 3: Appendix 10.3

Preliminary Geotechnical Design Report

900237-ARP-ZZ-ZZ-RP-CG-00004

Contents

		Page
1	**Introduction**	**1**
	1.1 Scope of report	1
	1.2 Scheme Proposals	1
2	**Earthworks**	**3**
	2.1 Cutting stability	3
	2.2 Embankments	6
	2.3 Reuse of materials	13
3	**Highway structures**	**14**
	3.1 Abutments	15
	3.2 Viaduct – ‘standard pier’	23
	3.3 Viaduct – fixed pier	29
	3.4 River bridge – main span and back span	31
	3.5 Total and differential settlements	34
	3.6 pH and Sulphate	35
4	**Strengthened earthworks**	**37**
5	**Drainage**	**38**
6	**Pavement design, subgrade & capping**	**39**
	6.1 Approach embankments	39
	6.2 Cuttings at the northern end of the scheme	39
	6.3 Areas at-grade	39
7	**Assessment of potential contamination**	**40**
	7.1 Summary of the extent of the contamination testing that has been undertaken	40
	7.2 Summary of the findings and conclusions of the risk assessments including the site remediation requirements that have been agreed with the regulatory authorities	42
	7.3 Details of contaminated material to be removed from site	45
8	**Ground treatment including treatment of any underground voids etc.**	**46**
9	**Specification appendices**	**47**
10	**Instrumentation and monitoring**	**48**
	10.1 Full details of purpose, installation requirements, restrictions and frequency of readings	48

10.2 Use of Observational Methods/Controls. Predicted and critical readings and restrictions on work 48
10.3 Pile testing requirements 48

11 References 49

Figures

Figure 1 - Location Plan

Figure 2 – General Arrangement

Figure 3 – Ground Investigation Location Plan

Figure 4 – Long Section

Appendices

Appendix A

Northern cutting - geological map overlay

Appendix B

Northern cutting - cross sections

Appendix C

Cattle and NMU underpass - cross sections

1 Introduction

1.1 Scope of report

This Preliminary Geotechnical Design Report has been prepared for the A487 New Dyfi Bridge Scheme to cover the preliminary design completed at Key Stage 3. This report documents the preliminary design completed to date along with the geotechnical considerations given to the scheme based on current proposals.

A desk-based study has been undertaken which is documented in the Preliminary Sources Study Report (Reference [1]). A preliminary site investigation has also been carried out (see Figure 3) which along with historical ground records, has been interpreted, and the findings presented in the Preliminary Ground Investigation Report (Reference [2]).

Additional ground investigation is proposed at Key Stage 6 to inform detailed design. During Key Stage 6, an update or addendum to the Ground Investigation Report will be prepared along with a supplementary Geotechnical Design Report on completion of detailed design. The supplementary Geotechnical Design Report will document the detailed design once complete, and may be submitted in parts to suit the design and construction programme.

1.2 Scheme Proposals

The scheme is located in the Afon Dyfi flood plain, a short distance to the north of Machynlleth; see Figure 1. Current proposals for the A487 New Dyfi Bridge scheme (Figure 2) comprise a 1050m long new section of single carriageway road to the south east of the existing trunk road, which will bypass the existing Pont-ar-Ddyfi Bridge.

The proposed new section of road includes a 726m long viaduct and river bridge, with associated approach embankments, across the Afon Dyfi and associated floodplain. The existing A487 will be de-trunked between the two ends of the scheme, and the existing Pont-ar-Ddyfi will be restricted to Non-Motorised Users only.

To the south of the Afon Dyfi, the proposed route crosses agricultural land in the bottom of a glacial valley. The approach embankment rises from existing ground level off the existing A487 (at approximately 7.5mAOD) to around 5m above ground level over a length of approximately 240m. The route then passes onto a viaduct comprising 18 spans, each typically around 34m in length.

The viaduct decks for the curved section of viaduct, at the southern end of the scheme, are proposed to be crane-lifted into place. The straight section of the viaduct, north of this, is proposed to be thrust-launched from a temporary structure to be positioned at the southern end of the scheme.

At the northern end of the viaduct, at the Afon Dyfi, the river bridge will include a 50m back-span and 70m main river span. The ground level on the northern side of the river rises sharply to around 7m above the valley floor (at approximately 15.6mAOD) and forms a plateau at the proposed position of the north abutment. An embankment approximately 2m in height will be required over the plateau to support the route behind the abutment. A cattle underpass, also to be used as a Non-Motorised Users (NMU) route, will be provided in front of the abutment.

The proposed route rejoins the alignment of the existing A487 in an area of rock cutting. In this area, scheme proposals include a ghost-island tie-in to the existing A487 to the west, and an access track to a private farm, which will be formed in rock cuttings.

Flood defences comprising earth bunds of no more than 2m in height are proposed in the south of the scheme area, along the northern boundary of the existing Eco Park.

The scheme also includes pumped drainage measures in the area of the railway bridge to deal with localised flooding in this area.

2 Earthworks

2.1 Cutting stability

2.1.1 Northern junction and farm access track cutting

Geometry

At the north eastern end of the scheme two cuttings, predominantly through rock, are required in order to connect the existing road to the new alignment and to form access to a private farm to the north west of the scheme (see Figure 2). The junction cutting is approximately 185m in length with typical height from toe to crest of 3m to 4m and a maximum height of 7.4m. The access track cutting is approximately 100m in length with typical height from toe to crest of 7m and a maximum height of 7.7m.

Summary of ground conditions

An interpretation of the geology in this area has been undertaken based on a review of the information collated during the desk-based study and the findings of the preliminary site investigation. This interpretation is contained within the Preliminary Ground Investigation Report (Reference [2]).

An overlay of the proposed cutting and the 1:10,000 geological maps for the area is included in Appendix A.

The preliminary stratigraphy and parameters used for design are summarised below.

Design Stratigraphy	Design Parameters
Granular Head Deposits 1.2m thickness (topography varies)	Moist bulk unit weight (γ_m) = 18kN/m² $\varphi'_{cv:k}$ = 36° $\varphi'_{pk:k}$ = 36° cohesion c' = 0kPa
Weathered Mudstone 1.0m thickness (topography varies)	The weathered bedrock was recovered as slightly silty sand and gravel, so it is considered that it will behave in a similar manner as the head deposits and therefore the same parameters have been adopted.
Intact Mudstone to depth (topography varies)	Moist bulk unit weight (γ_m) = 24kN/m² Uniaxial compressive strength (UCS) = 25MPa RQD = 75 – 100% Good to Excellent Sub-vertical discontinuities Modulus of Deformation E_m = 6x10^6 kN/m² φ' and c' shall be determined as part of the detailed ground investigation
Groundwater	No groundwater encountered during the ground investigation. The full depth of the cutting is anticipated to be dry throughout much of the year. Further investigation is

Design Stratigraphy	Design Parameters
	recommended at detailed design stage to confirm a groundwater level for design. Cut-off drains may be used if drainage is deemed to be required.
Relevant exploratory holes (see Figure 3)	TP24, TP25

Table 1 – Preliminary design stratigraphy and parameters for northern cuttings

Design considerations

It is currently proposed to form the cuttings in this area with slopes at 1 in 2. This is considered reasonable for the composite cuttings comprising head deposits overlying bedrock. However, given the strength of the rock encountered, it is considered that steeper slopes may be achieved in the rock strata. Indicative cross sections showing slopes at 1 in 2 and at 1 in 1 have been produced; see Appendix B.

Steeper slopes may be preferable if land take is to be reduced however steeper slopes would require more detailed investigation and analysis to ensure stability of the soil mass. The cross sections show that in some areas, slopes at 1 in 2 are nearly parallel with the natural slopes resulting in extended zones of cutting. Steepening the slope angle in these areas would be beneficial as it would reduce excavation works.

A stability assessment has not been carried out at this stage. The stability of the superficial deposits should be investigated using effective stress methods and relevant strength and groundwater data to provide the factor of safety for the long term or permanent works conditions. This could be undertaken with the support of slope stability software such as Oasys Slope. The rock assessment should consider discontinuities, potential groundwater flows, as well as the degree of weathering of the rock to determine the locations of potential failure surfaces. This should be undertaken using a well-established rock classification rating methodology such as Bieniawski (1979, 1989) and Romana (1985).

For this assessment the design actions and partial factors for the ultimate limit state case (ULS) and serviceability limit state (SLS) shall be in accordance with BS EN 1997-1:2004 and the associated UK national annex (References [8] and [9]).

This stability assessment will require additional ground investigation information, which is planned to be obtained at detailed design stage such as during Key Stage 6.

2.1.2 Cattle and NMU underpass cutting

Geometry

Current proposals are to provide a cattle underpass, also to be used as a Non-Motorised Users route, 3.5m wide in front of the northern abutment to allow the farm at the north-eastern end of the scheme to access the south-western side of the new road (see Figure 2). The underpass requires a cutting approximately 100m in length with typical depth from toe to crest of 2.5m, but with a maximum depth of 3.3m (see Appendix C).

Summary of ground conditions

An interpretation of the geology in this area has been undertaken based on a review of the information collated during the desk-based study and the findings of the preliminary site investigation. This interpretation is contained within the Preliminary Ground Investigation Report (Reference [2]).

The preliminary stratigraphy and parameters used for design are summarised below.

Design Stratigraphy	Design Parameters
Made Ground: 0m – 5m Existing ground level (~16mAOD) to 11.0mAOD	Moist bulk unit weight (γ_m) = 17 to 21 kN/m² $\varphi'_{cv:k} = 30^o$ $\varphi'_{pk:k} = 35^o$ cohesion c' = 0kPa
Glacial Sand and Gravel: 5m – 9m 11.0mAOD to 7mAOD	Moist bulk unit weight (γ_m) = 18.5kN/m² Drained Young's Modulus E'_d = 22,500 + 2750h where h is depth below the top of the strata $\varphi'_{cv:k} = 36^o$ $\varphi'_{pk:k} = 38.5^o$ cohesion c' = 0kPa
Bedrock: 9m to depth Below 7mAOD	Moist bulk unit weight (γ_m) = 24kN/m² Uniaxial compressive strength (UCS) = 25MPa RQD = 75 – 100% Good to Excellent Sub-vertical discontinuities Modulus of Deformation $E_m = 6x10^6$ kN/m² φ' and c' shall be determined as part of the detailed ground investigation
Groundwater	None considered
Relevant exploratory holes (see Figure 3)	TP20, TP21, BH5

Table 2 – Preliminary design stratigraphy and parameters for cattle and NMU underpass cutting

Design considerations

Excavation works are anticipated to be limited to within the Made Ground but could extend down to the sand and gravel depending on local variations of the stratigraphy. Due to the variable nature of the Made Ground, slopes at 1 in 2 for the cutting are considered reasonable at this stage. Preliminary slope stability design has been completed using Oasys Slope. The design actions and partial factors for the ultimate limit state case (ULS) were in accordance with BS EN 1997-1:2004 and the associated UK national annex (References [8] and [9]).

2.2 Embankments

2.2.1 Southern embankment

Geometry

The southern approach to the viaduct will be constructed on embankment. The embankment will rise from the existing A487 ground level (at approximately 7.5mAOD) up to around 5m above existing ground level (12.5mAOD) at the interface with the viaduct.

Summary of ground conditions

An interpretation of the geology in this area has been undertaken based on a review of the information collated during the desk-based study and the findings of the preliminary site investigation. This interpretation is contained within the Preliminary Ground Investigation Report (Reference [2]).

The preliminary stratigraphy and parameters used for design are summarised below.

Design Stratigraphy	**Design Parameters**
Alluvial Deposits: 0m – 3m Existing ground level (~7.75mAOD) to 4.75mAOD	Moist bulk unit weight (γ_m) = 17kN/m² Undrained shear strength (c_u) = 20kPa Drained Young's Modulus (E'd) = 2000kN/m² Undrained Young's Modulus (Eu) = 6000kN/m $\varphi'_{cv:k} = 26^o$ $\varphi'_{pk:k} = 26^o$ cohesion c' = 0kPa m_v = 0.917m²/MN c_v = 0.5 to 2 [see note at end of this table] C_α = 0.001 [see note at end of this table] Poissons ratio = (μ) = 0.3
Glacial Sand and Gravel: 3m to depth Below 4.75mAOD	Moist bulk unit weight (γ_m) = 18.5kN/m² $E'_d = 22{,}500 + 2750h$ where h is depth below the top of the strata $\varphi'_{cv:k} = 36^o$ $\varphi'_{pk:k} = 38.5^o$ cohesion c' = 0kPa
Groundwater	Assumed to be at ground level with periodical flooding.

Design Stratigraphy	Design Parameters
7.75mAOD	
Relevant exploratory holes (see Figure 3)	TP01 and TP02 (CCG 2016) TP2 and TP3 (CJ Associates 2001)
Note: c_v and C_α parameters were not derived in the GIR. Refer to design considerations section below for a discussion about their origin.	

Table 3 – Preliminary design stratigraphy and parameters for the southern embankment

Design considerations

Foundation options

There are currently four different options considered to construct the southern embankment as follows:

1. The design could allow the embankment to settle in a controlled manner that is acceptable to the temporary works and finished road construction. This could comprise programming construction of the embankment early on so that residual settlements are acceptably small by the time of the construction of highway drainage and road surfacing. There is the risk that a considerable time may be required after construction for completion of sufficient settlement to make this option viable. Stability of the embankment against potential bearing capacity failure in both the short and long term would need to be considered, and it is possible that the embankment may need to be constructed in stages. This option may or may not include surcharging, and will need more detailed analysis once additional ground investigation information is available at Key Stage 6. It should be noted that this is currently the preferred option.

2. The soft deposits could be dug out and replaced with general fill. The depth of the soft material underlying the embankment is expected to be less than 3m in places. One issue that would require careful consideration is whether potential groundwater ingress would be practical to manage during construction for this option. Significant water ingress when undertaking the trial pits during the recent ground investigation indicated that the groundwater level is high, and the surrounding and underlying soil is relatively permeable. Therefore, pumping and dewatering may be required, and compaction of fill may prove difficult. However, the investigation was undertaken during the winter in between rainfall events so conditions during the summer may be better.

3. Ground improvement could be carried out for the upper layer of softer soil. Options include mass improvement using a binder material intermixed with the existing soil, the use of vibro-stone columns / vibro concrete columns or an alternative form of ground improvement down to the more competent granular deposits. For these options, the embankment load could be spread across the improved formation

through the use of geogrid reinforcement, to limit any residual differential settlements. Residual settlements would be reduced, hence reducing any risks associated with differential settlement of the embankment and at the interface with the viaduct structure.

4. A piled raft could be constructed from the existing ground surface, to reduce or eradicate total and differential settlements of the embankment. As with the other options, consideration would have to be given to avoid any 'hard spot' effect where foundations transit back to less stiff non-piled option. It should be noted that this option is currently not favoured.

It is to be noted that the embankment is currently proposed to be constructed from arisings from the cutting in the north-eastern end of the scheme so works in this part of the scheme would have to be undertaken beforehand or in parallel.

Underlying material settlement assessment

Immediate settlement: based on option 1 discussed above and the cohesive nature of the layer of alluvium material underlying the embankment, settlement is expected to predominantly occur after construction.

Consolidation settlement: the layer of alluvium is relatively thin compared to the dimensions of the embankment, therefore it can be assumed at this stage that the consolidation will predominantly be one-dimensional and the following equation can be used to estimate the settlement due to consolidation:

$$s_c = m_v \,.\, \Delta\sigma' \,.\, H$$

Using a unit weight of $19kN/m^3$ and a height of 5m for the embankment fill to calculate the change in effective stress ($\Delta\sigma'$) over 3m of alluvium (H), a settlement of approximately 250mm is calculated for the centre of the embankment. Additional analysis were carried out using Oasys Pdisp, which adopts the Boussinesq approach to allow for the three dimensional, load spread beneath the embankment. The settlements obtained using this more refined approach were less than 150mm.

Secondary compression: the following relationship was used to calculate the secondary compression:

$$s_c' = C_\alpha \,.\, H \,.\, \log(t_2 / t_1)$$

Due to difficulty in obtaining undisturbed samples for the proposed testing for consolidation and secondary compression characteristics, a generalised $C\alpha$ value of 0.001 has been considered for the alluvium at this stage. Assuming the secondary compression starts once 90% of the consolidation settlement has occurred, values of 3.5mm to 5.5mm and 4mm to 6mm have calculated for 60 years and 120 years respectively.

Settlement duration: the time required for the consolidation settlement to occur was calculated with the following relationship:

$$t = T_v \,.\, d^2 / c_v$$

For assessment of the time required for 90% of total consolidation settlement to occur, a value of 0.848 was used for T_v (Craig's Soil Mechanics 7th Edition, Spon Press, 2004). An open layer has been assumed for the drainage path length d. Due to difficulty in obtaining undisturbed samples for the proposed testing for consolidation and secondary compression characteristics, a range of generalised c_v values have been considered for the alluvium at this stage. Based on the 'clayey silt' description in the logs and the Atterberg limits plotting on or below the A-line for the alluvium, a medium rate of consolidation is assumed, with corresponding c_v values ranging between 1 and 10 m^2/year as suggested by Lambe and Whitman (1979). The corresponding time for 90% completion of consolidation has been calculated from approximately 3 months to a maximum of approximately 2 years.

If following further ground investigation the more favourable consolidation characteristics of the alluvium are confirmed, it is likely that no particular ground treatment measures will be required for the embankment, which is planned to be constructed early in the construction programme. If however the more onerous consolidation characteristics are confirmed, measures will be required such as those highlighted earlier in this section. Whatever the outcome, it is anticipated that a specific construction detail may be required at the interface of the approach embankment and south abutment, to control differential settlements to acceptable levels over this key interface. It should be noted that difficulties were encountered in obtaining undisturbed U100 samples during the preliminary ground investigation, and therefore, consideration should be given to the potential need for use of alternative sampling techniques to achieve good quality undisturbed samples for appropriate consolidation testing, as part of the detailed ground investigation.

Potential for minor settlements is likely to remain in the long term, which should be manageable through long term maintenance such as routine resurfacing of the road. The long term settlements will be designed to lie within the acceptable limits of the structure.

Internal settlement assessment

It is currently proposed to re-use the arisings from the rock cutting in the north-eastern end of the scheme to construct the southern embankment. At this stage limited ground investigation information is available from the proposed area of cutting. On the basis of the exploratory hole log descriptions, the arisings are considered likely to comprise Class 1A granular general fill, although further assessment will be required following detailed ground investigation in the area.

Internal settlement of embankments formed of granular fill will generally occur as construction proceeds, and the resultant long term internal settlement following construction will typically be small or negligible.

Stability assessment

Slope stability analysis was undertaken using Oasys Slope using Bishop's circular arc analysis to assess shallow and deep seated slope instability in accordance with BS EN 1997-1 : 2004 (Reference [8]).

A surcharge of 20kPa was added across the top width of the embankment to account for the vehicle surcharge, with material parameters and applied loads factored in accordance with BS EN 1997-1 : 2004 (Reference [8]).

The analysis comprised a short-term undrained analysis and a long-term drained analysis. In both analyses deep seated failures passing through the weaker alluvium compared to the underlying sands and gravels were the critical mode of failure.

Considering the preferred option whereby the embankment is allowed to settle, the preliminary calculations have shown that slope gradients no steeper than 1 in 3 should be used to ensure stability where the embankment is proposed at greater heights. Deep seated failures have been found to be the governing factor for the stability of the embankment. If steeper slope gradients are required, some form of ground improvement or more favourably the incorporation of geogrid reinforcement at the base of the embankment could be used. Ground improvement would also bring benefits in relation to settlement, see the foundation options section above.

Gas main

A 150mm 24.1 bar high pressure gas main exists in the southern end of the scheme. Its alignment has been surveyed (see on Figure 2) but its depth is not currently known. The northern toe of the southern embankment is currently ending immediately on top of the gas main.

The differential vertical and horizontal settlements of the gas main due to the loading from the proposed embankment are potentially an issue. A remedial option would be to provide a protection slab. This will be considered further at detailed design stage.

2.2.2 Northern embankment

Geometry

The northern approach to the river crossing will be constructed on embankment. The embankment will commence at the interface with the northern abutment (at approximately 17.3mAOD) with a maximum height of approximately 5m and will reduce in height until it joins the existing A487.

Summary of ground conditions

An interpretation of the geology in this area has been undertaken based on a review of the information collated during the desk-based study and the findings of the preliminary site investigation. This interpretation is contained within the Preliminary Ground Investigation Report (Reference [2]).

The preliminary stratigraphy and parameters proposed for design are summarised below.

Design Stratigraphy	Characteristic Parameters
Made Ground: 0m – 5m (Existing ground level (~16mAOD) to 11.0mAOD)	Saturated bulk unit weight (γ_m) = 17 to 21 kN/m² Drained Young's Modulus (E'_d) = 6000kN/m² $\varphi'_{cv:k}$ = 30º $\varphi'_{pk:k}$ = 35º cohesion c' = 0kPa
Glacial Sand and Gravel: 5m – 9m (11.0mAOD – 7.0mAOD)	Saturated bulk unit weight (γ_m) = 18.5kN/m² Drained Young's Modulus E'_d = 22,500 + 2750h where h is depth below the top of the strata $\varphi'_{cv:k}$ = 36º $\varphi'_{pk:k}$ = 38.5º cohesion c' = 0kPa
Bedrock: 9m to depth (Below 7mAOD)	Saturated bulk unit weight (γ_m) = 24kN/m² Uniaxial compressive strength (UCS) = 25MPa RQD = 75 – 100% Good to Excellent Sub-vertical discontinuities Modulus of Deformation E_m = 6x10^6 kN/m² φ' and c' parameters will be considered at detailed design stage.
Groundwater: 7.5m (8.5mAOD)	Ground water level at 8.5mAOD with periodical flooding.
Other Issues	Negative skin friction will need to be considered for the foundations of the northern abutment.
Relevant exploratory holes (see Figure 3)	TP20, TP21 and BH05

Table 4 – Preliminary design stratigraphy and parameters for northern embankment

Stability assessment

Slope stability analysis was undertaken using Oasys Slope using Bishop's circular arc analysis to assess shallow and deep seated slope instability in accordance with BS EN 1997-1 : 2004 (Reference [8]).

Analysis has been carried out to assess the long term stability of the proposed geometry of the northern abutment. This has taken account of:

- The additional fill that will form the northern embankment.
- The cutting that will be formed in front of the abutment to accommodate the cattle and NMU route.
- The piled abutment, which will transfer local embankment loading through the piles down into the underlying rock.

- The existing slope that will remain in front of the abutment down to the river channel.
- A surcharge of 20kPa across the top width of the embankment with the associated bridge loading supported by the abutment piles.

The analysis comprised a short-term undrained analysis and a long-term drained analysis.

Consideration has been given to the temporary condition whereby an excavation is made down to pile commencement level, and a large piling rig, of up to approximately 50 tonnes, is positioned near the crest of the slope.

Design considerations

Similarly to the southern embankment, an assessment shall be carried out with regards to the settlement of the underlying material, self-settlement and slope stability. This will be undertaken at detailed design.

2.2.3 Flood bund

Flood bunds of up to approximately 2m in height are proposed around the Eco Park in the southern end of the scheme; see Figure 2. It is proposed that the detailed design follows the Environmental Agency guidance (Reference [4]) for embankments used for flood defences, the key points of which are summarised below:

- Upstream and downstream slopes of 1 in 2.5 to 1 in 3 are anticipated to be adequate for a flood bund constructed in a fine grained soil. This could potentially be sourced from the superficial deposits that are to be excavated in the north of the site, but consideration should be made to the potential need to import suitable material. The bund geometry should consider the maintenance regime to be adopted, and access ramps shall also be provided for maintenance.
- The permeability of the underlying strata shall be assessed for potential ground seepage.
- The effects of scour and ponding of floodwaters with receding levels should be assessed. As well as considering the wet side, the impacts on the dry side in case of overtopping shall be accommodated by the design. Typical measures would include seeding exposed slopes, with ‘harder’ measures such as the potential need for rip-rap protection only considered if locations are identified where water flows are likely to be very high.

Specific consideration shall be given to the following at detailed design stage:

- Ability of alluvium layer to prevent groundwater flows beneath the flood bunds.
- The effectiveness of the flood bund and formation under steady flow situations (not anticipated for 48 hours flood period).

- The potential for minor seepages of flood water to be collected by the proposed pumping measures at the existing railway underpass; refer to Section 3.11.1 of the Preliminary Ground Investigation Report (Reference 2) for more details.
- Further investigation will be required ahead of detailed design, some of which might be outside scheme boundaries (e.g. walk over, trial pits).
- Availability of suitable site-won material for construction.

The stability of the bund shall be assessed similarly to the southern embankment discussed in Section 2.2. Design parameters shall be determined at detailed design when bund requirements will have been assessed.

2.3 Reuse of materials

It is currently proposed to re-use the arisings from the rock cutting in the northern end of the scheme to construct the southern and northern approach embankments.

As the preliminary ground investigation was commenced early on during development of the scheme proposals, limited ground investigation has been carried out on the sources of material proposed for re-use. However, from the available ground investigation, the following is anticipated:

- An upper mantle of head deposits, typically comprising silt to a depth of around 1m. This material would typically classify as Class 2C in accordance with the Specification for Highway Works – Series 600. Due to the high silt content, this material is likely to be moisture sensitive, and therefore moisture control may prove critical in ensuring that the fill is able to be suitably placed and compacted to form a stable fill. Due to the relatively high cohesive content, it may be favourable to target reuse of this upper zone of material for construction of the proposed flood bunds.
- Below the head deposits, a layer of weathered rock is present. This material has been described as having been recovered as slightly silty, and was easily excavated with a standard tracked excavator. This material is likely to classify as Class 2C or 1A in grading. Where higher silt contents are present, this material may also prove moisture sensitive.
- Below approximately 2m, relatively unweathered mudstone bedrock is present. This material may prove difficult to excavate using standard earthworks plant, with ripping anticipated to be required. Following excavation, it is anticipated crushing and sorting will be required to obtain a Class 1A general fill for use in construction of the proposed embankments. Consideration may be given to processing this material to provide select materials for the works, although the feasibility of this would need to be further investigated at detailed design stage.

As this preliminary design stage, the anticipated earthworks quantities are as follows:

Type and location	Estimated earthworks quantities
Cut	
North of the river	16,500 m^3
South of the river	300 m^3
Total	16,800 m^3
Fill	
North of the river	2,300 m^3
South of the river (including flood bunds)	18,000 m^3
Total	20,300 m^3
Deficit	Approximately 3,500 m^3

The above quantities indicate a deficit. However, it should be noted that these quantities are a high level assessment which do not take into account potential bulking and unsuitable materials. A more detailed assessment will be undertaken by the Contractor at detailed design.

3 Highway structures

The viaduct comprises 18 spans and the river crossing structure comprises a mainspan and a backspan, each supported by piers formed by two independent columns. The deck of the viaduct and the river crossing typically sit on longitudinally guided bearings which limit lateral loading to the piers and piles (see Figure 2).

For the purpose of this report, the structure has been split up into the following components.

- The two abutments, situated at the southern end of the viaduct and on the northern bank of the river. The abutments will retain the south and north approach embankments, and will therefore be subject to associated lateral loading which the foundations will be designed to accommodate. Due to the relatively high lateral loading, pile groups that will act with a push-pull effect are proposed.

- The typical piers for the viaduct (Piers 1 to 10 and 12 to 17), which support bridge decks of 34m in span. As the bridge decks are supported on guided bearings, lateral loading to the piers and foundations will be relatively small, and a single pile is proposed for each pier column, ie, two ‘mono-piles’ per pier.

- The fixed connection pier that is positioned relatively centrally along the viaduct at pier 11. The fixed connection pier is proposed to take the lateral loading from the deck of the viaduct. Due to the relatively high lateral loading, single piles are not feasible for the pier columns, and pile groups that will act with a push-pull effect are proposed.

- The piers that support the southern end of the main river span, and the southern support to the back-span. As a 70m river span is proposed, and a 50m back-span to limit out of balance moments, significantly larger loads will be applied to these foundations relative to the other foundations. Single piles are therefore not feasible for these pier columns, and piles groups that will act with a push-pull effect are proposed.

It should be noted that the piers supporting the southern supports of the main river span and back-span are proposed to be skewed at a 30 degree angle to the normal of the road alignment to limit scouring effects during flood events. To limit the length of the main river span and hence its structural requirements, the northern embankment is proposed to be skewed at the same angle.

3.1 Abutments

3.1.1 Structure and foundation type

Geometry

The abutments are proposed to be L-shape reinforced concrete walls, each founded on a pile group arranged in two rows.

Southern abutment

The southern abutment will retain the approach embankment, that will be of approximately 5m in height. The fill will wrap around the abutment in an 'elephant ear' configuration, with the abutment wing walls orientated longitudinally to the highway.

Northern abutment

The main river crossing will span 73m and land on the northern abutment with a cattle and NMU underpass provided under the viaduct in front of the abutment (see Figure 2).

The wing walls to the northern abutment are proposed as reinforced concrete, and will return parallel to the mainline. The foundations are proposed to step up in level away from the abutment, as it transitions into an embankment slope.

3.1.2 Construction sequence

The construction sequence assumed for the preliminary design is as follows:

1. Excavate down to pile commencement level.
2. Construct piling mattress.
3. Bore piles to specified depth, using temporary casing, with an allowance for rock sockets where required.

4. Install reinforcement cage and place concrete in pile bore.
5. Remove temporary casing.
6. After the concrete has cured, cut down pile heads to cut off level, and trim pile reinforcement to required embedment length.
7. Construct reinforced concrete abutment and wing walls.
8. Backfill the abutment.
9. Install bearings and the main bridge deck.

Based on the relatively flat topography of the floodplain, no significant variations in piezometric pressure are anticipated, and therefore groundwater flows are anticipated to be relatively low. On this basis, concrete washout after removal of the temporary casing is not anticipated to be an issue. There is potential for higher groundwater flows to be present local to the river, at the elevated northern abutment, and where palaeochannels are present. The potential for concrete washout at such locations will be further considered at detailed design stage.

3.1.3 Design assumptions

Geometry

For Key Stage 3 stage, the bases of the abutment walls have been assumed to be 1m thick and the stems 0.4m thick. The bases have been assumed to be founded at existing ground level.

The spacing between the two rows of piles have been assumed as four times the piles diameter (centre to centre).

Consideration has been given to piles diameters of 0.6m, 0.75m and 0.9m, and pile groups made of 8 piles and 12 piles.

Southern abutment

The abutment is 5m in height and 17m in width based on the preliminary scheme layout.

Northern abutment

The abutment is 4m in height and 15m in width based on the preliminary scheme layout. To minimise the span of the main span of the river bridge, the abutment is skewed at a 30° angle to the transverse alignment of the highway.

3.1.4 Design ground conditions

The two abutments are at opposite end of the scheme and therefore have different ground conditions.

Southern abutment

The ground conditions over this length of the proposed route comprise topsoil overlying alluvial deposits, which in turn are anticipated to overlie glacial sands and gravels.

The alluvial deposits were recorded between 1.6m and in excess of 3.0m in thickness. It is proposed that the Key Stage 6 investigation should confirm the thickness and consolidation characteristics of alluvium in this area.

The base of the glacial sands and gravels was not proven during recent or historic investigations but has been confirmed to be in excess of 22m below existing ground level.

The preliminary stratigraphy and parameters used for design are summarised below.

Design Stratigraphy	**Characteristic Parameters**
Alluvial Deposits: 0m – 3m Existing ground level (~7.75mAOD) to 4.75mAOD	Moist bulk unit weight (γ_m) = 17kN/m² Undrained shear strength (c_u) = 20kPa Drained Young's Modulus (E'_d) = 4000kN/m² Undrained Young's Modulus (E_u) = 6000kN/m² $\varphi'_{cv:k} = 26^o$ $\varphi'_{pk:k} = 26^o$ cohesion c' = 0kPa m_v = 0.917m²/MN c_v = 1 to 10m²/year [see note at the end of this table] C_α = 0.001 [see note at the end of this table] Poissons ratio = (μ) = 0.3
Glacial Sand and Gravel: 3m to depth (below 4.75mAOD)	4.75mAOD to -7.0mAOD Saturated bulk unit weight (γm) = 18.5kN/m² E'_d = 22,500 + 2750h where h is depth below the top of the strata $\varphi'_{cv:k} = 36^o$ $\varphi'_{pk:k} = 38.5^o$ cohesion c' = 0kPa -7.0mAOD to -11.0mAOD Saturated bulk unit weight (γ_m) = 19.5kN/m² E'_d = 60,000 + 30,000h where h is depth below -7mOD $\varphi'_{cv:k} = 36^o$ $\varphi'_{pk:k} = 39^o$ cohesion c' = 0kPa -11.0mAOD to -17mAOD Saturated bulk unit weight (γm) = 21kN/m²

	E'_d = 60,000 + 30,000h where h is depth below -7mOD $\varphi'_{cv:k} = 36^o$ $\varphi'_{pk:k} = 43^o$ cohesion c' = 0kPa
Groundwater	Assumed to be at ground level with periodical flooding.
Relevant exploratory holes (see Figure 3)	TP01 and TP02, BH01 and BH02 (CCG 2016) TP2 and TP3 (CJ Associates 2001)
Note: c_v and C_α parameters were not derived in the GIR. Refer to design considerations of Section 2.2.1 for a discussion about their origin.	

Table 5 – Preliminary design stratigraphy and parameters for southern abutment

Northern abutment

The ground conditions over this length of the proposed route comprise topsoil overlying Made Ground, which in turn overlies glacial sand and gravels over bedrock.

The preliminary stratigraphy and parameters used for design are summarised below.

Design Stratigraphy	Characteristic Parameters
Made Ground: 0m – 5m (Existing ground level (~16mAOD) to 11.0mAOD)	Saturated bulk unit weight (γ_m) = 17 to 21 kN/m² Drained Young's Modulus (E'_d) = 6000kN/m² $\varphi'_{cv:k} = 30^o$ $\varphi'_{pk:k} = 35^o$ cohesion c' = 0kPa Poissons ratio = (μ) = 0.3
Glacial Sand and Gravel: 5m – 9m (11.0mAOD – 7.0mAOD)	Saturated bulk unit weight (γ_m) = 18.5kN/m² Drained Young's Modulus E'_d = 22,500 + 2750h where h is depth below the top of the strata $\varphi'_{cv:k} = 36^o$ $\varphi'_{pk:k} = 38.5^o$ cohesion c' = 0kPa
Bedrock: 9m to depth (Below 7mAOD)	Saturated bulk unit weight (γ_m) = 24kN/m² Uniaxial compressive strength (UCS) = 25MPa RQD = 75 – 100% Good to Excellent Sub-vertical discontinuities Modulus of Deformation $E_m = 6x10^6$ kN/m²

	φ' and c' shall be determined as part of the detailed ground investigation
Groundwater: 7.5m (8.5mAOD)	Ground water level at 8.5mAOD
Other Issues	Negative skin friction will need to be considered for the foundations of the northern abutment.
Relevant exploratory holes (see Figure 3)	TP20, TP21 and BH05

Table 6 – Preliminary design stratigraphy and parameters for northern abutment

3.1.5 Design actions

The design actions and partial factors for the ultimate limit state case (ULS) and serviceability limit state (SLS) are in accordance with BS EN 1997-1:2004 and the associated UK national annex, using Design Approach 1 Load Combination 1 and Load Combination 2.

The following were considered as actions for the ULS and SLS cases in accordance with BS EN 1997-1:2004:

- Superstructure dead load and bridge deck dead load.
- Highway vertical loading on the bridge deck.
- Highway vertical loading on the retained ground.
- The earth pressures that result from the mass of the retained soil.
- Vehicle braking behind the abutment both towards and away from the abutment.
- Vehicle skidding (transversal) on the bridge deck.
- Bearing friction due to thermal movements.
- Eccentricity due to construction tolerances on pile installation and bearing position.
- Eccentricity due to deflection at bearing level.

The characteristic highway loading applied to the bridge deck was in accordance with the BS EN 1991-2 and UK National Annex (References [5] and [6]).

The vehicle surcharge on the retained ground was applied in accordance with PD 6694-1 (Reference [11]) taking loadings for a SV/100 and SV/196.

The southern bearing was considered to allow movements in the longitudinal direction and to restrict movements in the transversal direction. The northern bearings were assumed to be free in both directions; see Figure 2.

3.1.6 Design approach

Design objectives

The objectives of the design are in accordance with BS EN 1997-1:2004 to:

- Ensure stability of the foundation.
- Control vertical and lateral deflections of the abutment, and differential movement between structural elements and adjacent earthworks to within acceptable limits.
- Ensure that the bending moments and shear forces that result from abutment deflections and reactions can be accommodated by the structure.

The differential settlements to be allowed for in the design have been determined based on consideration of the magnitudes of settlements that would result from different sizes and lengths of pile foundations, together with the interface between abutment and bridge deck that would accommodate these movements.

Design standards

The design has been carried out in accordance with BS EN 1997-1:2004 and the associated UK national annexes.

Serviceability Limit State (SLS)

The SLS case was modelled in accordance with clause 2.4.8. of BS EN 1997-1:2004.

Ultimate Limit State (ULS)

The ULS case was modelled in accordance with clause 2.4.7 of BS EN 1997-1:2004 to assess the following:

- Internal failure or excessive deformation of the structure (STR)
- Failure or excessive deformation of the ground (GEO)

Partial factors as presented in Annex A of BS EN 1997-1:2004 and their combinations as given for Design Approach 1 were adopted in the analyses.

Potential for flooding has been allowed for in the design.

3.1.7 Design methodology

Design calculations have been carried out in accordance with BS EN 1997-1:2004 to assess:

- Vertical and lateral geotechnical capacity of the piles to resist loading, taking account of the push-pull effect of the pile groups.
- Vertical deflection of the pile groups.

- Lateral deflections of the pile groups.
- Structural capacity of the piles to resist the resulting axial loading, shear forces and bending moments.

For each analysis, consideration was given to the worst case loading both in terms of construction sequence and in terms of loading direction.

Pile group effects were modelled using the design software PIGLET which enabled the push-pull effect of the pile groups to be analysed. Initial calculations were carried out using the appropriately factored loads and soil parameters to determine the required pile lengths to resist the applied axial loading under ULS conditions.

Once minimum pile lengths had been determined, PIGLET was used to analyse the lateral and vertical deflections of the pile group, talking account of the soil structure interaction effects.

The soil was modelled as a linear elastic material, with stiffness varying linearly with depth. For the southern abutment, as the thickness of alluvium is small relative to the anticipated pile embedment length within the glacial sands and gravels deposits, the piles were assumed to be entirely founded within the stiffer deposits. An additional length was added to the minimum pile length calculated above to account for the embedment within the alluvium. For the northern abutment, consideration was given to rock sockets due to relatively shallow rockhead level at this location.

Following completion of the PIGLET analysis, individual piles were analysed separately using the Oasys ALP soil structure interaction software. This enabled analysis to be carried out using more refined soil profiles and pile properties in order to provide a more accurate determination of loadings and deflections along the lengths of the individual piles and deflections of the pile groups. Detailed assessment of the structural capacity of the piles and pile caps to accommodate the resulting shear forces and bending moments has not been carried out at this stage, and should be carried out at detailed design stage.

It should be noted that if static pile tests are carried out, lower partial factors may be adopted for pile design in accordance with Clause A3.3.2 of NA A1 : 2014 (Reference [9]) of BS EN 1997-1 : 2004. The potential for carrying out static load tests and the benefits in terms of potential reductions in pile lengths should be considered at detailed design stage.

An adjusted elasticity method using Christian and Carrier influence factors (Reference [12]) was used to calculate the settlement of the pile groups in accordance with Annex F of BS EN 1997-1 : 2004. The average settlement of the pile group was estimated based on the following equation:

$$\rho = \mu_1 \mu_0 q_n B / E_u$$

The influence factors μ_1 and μ_0 were obtained graphically and were related to the depth and the length/breadth (B) ratio of the equivalent block foundation. The deformation modulus E_u was taken as the average drained young's modulus of the sands and gravels at mid-depth along the pile length.

3.1.8 Design outcomes

General

Pile lengths were found to be governed by axial loads, with the resulting configurations providing sufficient bending moment and shear force capacity for the applied lateral loading.

Southern abutment

Based on the preliminary calculations the following have been estimated:

No. of piles in group	Pile diameter (m)	Estimated pile length (m)	Founding level (mAOD)	Horizontal deflection at bearing level (mm)	Vertical deflection (mm)
16	0.6	25	-17.25	<50	<25
16	0.75	17.5	-9.75	<50	<25
16	0.9	15	-7.25	<25	<50
20	0.6	22.5	-14.75	<50	<25
20	0.75	15	-7.25	<50	<25
20	0.9	12.5	-4.75	<25	<50

Table 7 – Preliminary design outcomes for southern abutment

The options comprising 0.6m and 0.75m diameter piles have been found to develop significant tensional forces which are not considered satisfactory for design. These forces could be reduced by increasing the spacing between the two rows of piles however this would also increase the thickness of the pile cap. Alternatively, a third row of pile could be introduced to provide additional capacity.

Northern abutment

Based on preliminary calculations rock sockets of 3m will be required, resulting in the following pile lengths:

- 8-pile group
  - 0.6m diameter: 13m (3mAOD founding level)
  - 0.75m diameter: 13m (3mAOD founding level)
  - 0.9m diameter: 13m (3mAOD founding level)
- 12-pile group
  - 0.6m diameter: 13m (3mAOD founding level)
  - 0.75m diameter: 13m (3mAOD founding level)
  - 0.9m diameter: 13m (3mAOD founding level)

It is assumed that due to the rock sockets the vertical settlements are expected to be minimal at the northern abutment. At this stage of preliminary design development, lateral deflections have not been assessed for the northern abutment, although values are likely to be similar to those estimated for the southern abutment.

3.2 Viaduct – 'standard pier'

3.2.1 Structure and foundation type

The viaduct structure is to be supported on reinforced concrete piles generally installed through the upper alluvium layer. Piles are generally expected to be founded in the underlying granular glacial deposits, with loads taken in skin friction and end bearing.

Where bedrock is potentially present at relatively shallow depth, such as in the northern end of the structure, piles will be installed to shallower depth, by drawing upon additional resistance provided by construction of rock sockets. This presents a risk for differential settlements which will be considered.

As previously discussed in Section 3.2.1, a gas main is present in the south of the site. A survey of the gas main has been undertaken (see Figure 2), and the layout has been designed to avoid it.

3.2.2 Construction sequence

The construction sequence assumed for the preliminary design is as follows:

1. Construct piling mattress.
2. Bore piles to specified depth, using temporary casing.
3. Install reinforcement cage and place concrete in pile bore.
4. Remove temporary casing.
5. After the concrete has cured, cut down pile heads to cut off level, and trim pile reinforcement to required embedment length.

Based on the relatively flat topography of the floodplain, no significant variations in piezometric pressure are anticipated, and therefore groundwater flows are anticipated to be relatively low. On this basis, concrete washout after removal of the temporary casing is not anticipated to be an issue. There is potential for higher groundwater flows to be present local to the river and where palaeochannels are present. The potential for concrete washout at such locations will be further considered at detailed design stage.

3.2.3 Design assumptions

Geometry

Preliminary designs for single pile diameters of 1.2m and 1.5m have been developed, based on initial assessment of the feasibility of a wider range of diameters to function as 'mono-piles'.

3.2.4 Design ground conditions

The ground conditions over this length of the proposed route comprise topsoil overlying alluvial deposits, which in turn overlie glacial sands and gravels.

The alluvial deposits are around 1.6m thick at the southern end of the viaduct, increasing in thickness to in excess of 3.0m in TP07, which is close to the field boundary at Ch. 0+445m. The deposits then thin out again to around 2m in the north, with the exception of a slightly deeper area in the location of the former pond close to TP16. The base of the glacial sands and gravels was not proven in the southern and central areas of the viaduct, away from the river crossing. These deposits have been confirmed as extending to depths in excess of 27m below existing ground level.

The intention is to use the sands and gravels as the founding stratum for the majority of the viaduct piles with the exception of pier 17 which is anticipated to be founded in bedrock.

The preliminary stratigraphy and parameters used for design are summarised below.

Design Stratigraphy	**Characteristic Parameters**
Alluvial Deposits: 0m – 3m (8.5mAOD – 5.5mAOD)	Saturated bulk unit weight (γ_m) = 17kN/m² Undrained shear strength (Cu) = 20kPa Drained Young's Modulus (E'd) = 4000kN/m² Undrained Young's Modulus (Eu) = 6000kN/m² $\varphi'_{cv:k} = 26^o$ $\varphi'_{pk:k} = 26^o$ cohesion c' = 0kPa
Glacial Sand and Gravel: 3m – to depth (below 5.5mAOD)	<u>5.50mAOD to -7.0mAOD</u> Saturated bulk unit weight (γ_m) = 18.5kN/m² E'_d = 22,500 + 2750h where h is depth below the top of the strata $\varphi'_{cv:k} = 36^o$ $\varphi'_{pk:k} = 38.5^o$ cohesion c' = 0kPa

Design Stratigraphy	**Characteristic Parameters**
	-7.0mAOD to -11.0mAOD Saturated bulk unit weight (γ_m) = 19.5kN/m² E'_d = 60,000 + 30,000h where h is depth below -7mOD $\varphi'_{cv:k}$ = 36° $\varphi'_{pk:k}$ = 39° cohesion c' = 0kPa Below -11.0mAOD Moist bulk unit weight (γ_m) = 21kN/m² E'_d = 60,000 + 30,000h where h is depth below -7mOD $\varphi'_{cv:k}$ = 36° $\varphi'_{pk:k}$ = 43° cohesion c' = 0kPa
Bedrock (only anticipated for pier 17) 20.4m to depth (below 12mAOD)	Saturated bulk unit weight (γ_m) = 24kN/m² Uniaxial compressive strength (UCS) = 25MPa RQD = 75 – 100% Good to Excellent Sub-vertical discontinuities Modulus of Deformation E_m = 6x10^6 kN/m²
Groundwater: 0m (8.5mAOD)	Assumed to be at ground level with periodical flooding.
Other Issues	Negative skin friction will need to be considered for the foundations pier 1 due to the proximity of the southern embankment and associated tie-in.
Relevant exploratory holes (see Figure 3)	BH01 to BH04 TP01 to TP18

Table 8 – Preliminary design stratigraphy and parameters for 'standard piers'

3.2.5 Design actions

The design actions and partial factors for the ultimate limit state case (ULS) and serviceability limit state (SLS) are in accordance with BS EN 1997-1:2004 and the associated UK national annex, using Design Approach 1 Load Combination 1 and Load Combination 2.

The following were considered as actions for the ULS and SLS cases in accordance with BS EN 1997-1:2004:

- Superstructure dead load and bridge deck dead load.

- Column dead load
- Highway vertical loading on the bridge deck.
- Vehicle skidding (transversal) on the bridge deck.
- Bearing friction due to thermal movements.
- Eccentricity due to construction tolerances on pile installation and bearing position.
- Eccentricity due to deflection at bearing level.

For the preliminary design calculations, the action of wind load has not been considered as it is anticipated to lead to lower bearing friction forces than the action of thermal movements. The action of wind load will be considered at detailed design stage.

The characteristic highway loading applied to the bridge deck was in accordance with the BS EN 1991-2 and UK National Annex (References [5] and [6]).

The southern bearings were considered to allow movements in the longitudinal direction and to restrict movements in the transversal direction. The northern bearings were assumed to be free in both directions; see Figure 2.

3.2.6 Design approach

Design objectives

The objectives of the design are in accordance with BS EN 1997-1:2004 to:

- Ensure stability of the foundation.
- Control vertical and lateral deflections of the viaduct piers, and differential movement between structural elements.
- Ensure that the bending moments and shear forces that result from viaduct pier deflections and reactions can be accommodated by the structure.

The differential settlements to be allowed for in the design shall be determined based on consideration of the magnitudes of settlements that would result from different sizes and lengths of pile foundations, together with the different forms of bridge deck that would be required to resist the resulting displacements for these different foundations options.

Design standards

As per Section 3.1.6.

Serviceability Limit State (SLS)

As per Section 3.1.6.

Ultimate Limit State (ULS)

As per Section 3.1.6.

3.2.7 Design methodology

Consideration was given to the worst case loading both in terms of construction sequence and in terms of loading direction.

The worst set of loading was found to occur at the tallest piers; Pier 14 and 15 for the straight length of viaduct and Pier 3 for the curved length of viaduct.

The minimum pile lengths required to resist the maximum axial loading applied at the pier were calculated in accordance with the ULS case.

As the thickness of alluvium is small relative to the anticipated pile embedment length within the glacial sands and gravels, the piles have been assumed to be entirely founded within the stiffer deposits. An additional length was added to the minimum pile length calculated above to take into account the embedment within the alluvium.

As the majority of foundations for the scheme are for the ‘standard piers’, additional consideration was given to potential savings in pile length for this particular pier type, if static pile tests are carried out. Reduced pile lengths were calculated in accordance with Clause A.3.3.2 of NA A1:2014 to BS EN 1997-1:2004. The pile testing requirements allow lower partial factors and reduction in design pile lengths as follows:

- Working and/or preliminary pile testing to 1.5xSLS load to confirm serviceability and allow lower R4 factors; and
- Preliminary pile testing to confirm ultimate resistance and allow lower Model Factor.

The pier deflections at deck level resulting from the horizontal loading and moment applied at the top of the pile were calculated using Oasys ALP. The deflection value accounted for the deflection at the top of the pile and the contribution due to rotation at top of the pier.

The vertical settlements of the piles was calculated by estimating the load carried in shaft friction and the load transferred to the base. The pile head settlement was considered to be the sum of the elastic shortening of the shaft and the compression of the soil beneath the pile toe and was calculated using the following equation (Tomlinson 2008):

$$\rho = \frac{(W_s + 2W_b)L}{2A_sE_p} + \frac{\pi}{4} \cdot \frac{W_b}{A_b} \cdot \frac{B(1-v^2)I_p}{E_b}$$

Where:

W_s and W_b = loads on the pile shaft and base respectively

L = shaft length

A_s and A_b = cross-sectional area of the shaft and base respectively

E_p = elastic modulus of the pile material

B = pile width

ν = Poisson's ratio of the soil

I_p = influence factor related to the ration L/R

E_b = deformation modulus of the soil beneath the pile base

The influence factor I_p was taken as 0.5 and the deformation modulus E_b taken as E'_d for the corresponding depth. It should be noted that the selection of the latter parameter is reliant on the original in-situ density of the soil beneath the base which should be maintained through appropriate construction techniques.

3.2.8 Design outcomes

Preliminary calculations were undertaken for the worst case piers in the straight length of the viaduct (Pier 14 and Pier 15) and the curved length of the viaduct (Pier 3).

Based on the preliminary calculations, the following pile lengths have been estimated for Piers 14 and 15:

- 1.2m diameter: 20m
- 1.5m diameter: 15m
- If pile testing was undertaken, the above pile lengths could be reduced by approximately 3m.

Based on preliminary calculations, the following pile lengths have been estimated for Pier 3:

- 1.2m diameter: 22.5m
- 1.5m diameter: 17.5m
- If pile testing was undertaken, the above pile lengths could be reduced by approximately 5m.

Alp analyses indicate horizontal deflections at bearing level to be approximately 100mm and 50mm for 1.2m and 1.5m diameter respectively for these piers. The vertical settlements under the greatest vertical loads, at Pier 3, have been estimated as less than 50mm for the 1.5m and less than 25mm for the 1.2 diameter single piles. The choice of pile diameter may be governed by the ability of the structure, and in particular the bearings, to accommodate such displacements.

It should be noted that piles constructed with a rock socket, currently anticipated at pier 17, will experience significantly less vertical settlements.

3.3 Viaduct – fixed pier

3.3.1 Structure and foundation type

The typical viaduct piers are supporting the bridge deck on bearings free to move longitudinally. A fixed pier (Pier 11) will carry the longitudinal highway breaking load applied on the bridge deck. Due to the higher lateral forces at this location, a pile-group is proposed, as opposed to the mono-pile option for each pier column that will support standard piers.

3.3.2 Construction sequence

The assumed construction sequence for preliminary design is as follows:

1. Construct piling mattress.
2. Bore piles to specified depth using temporary casing
3. Install reinforcement cage and place concrete in pile bore.
4. Remove temporary casing.
5. After the concrete has cured, cut down pile heads to cut off level, and trim pile reinforcement to required embedment length.
6. Construct reinforced concrete pile cap.

Based on the relatively flat topography of the floodplain, no significant variations in piezometric pressure are anticipated, and therefore groundwater flows are anticipated to be relatively low. On this basis, concrete washout after removal of the temporary casing is not anticipated to be an issue. There is potential for higher groundwater flows to be present local to the river and where palaeochannels are present. The potential for concrete washout at such locations will be further considered at detailed design stage.

3.3.3 Design assumptions

Geometry

For preliminary design purposes at Key Stage 3, the pile cap has been assumed to be 1m thick, and formed entirely below existing ground level. There remains the option to form the pile cap above ground surface level which may prove favourable from a constructability point of view, due to the potential for high rates of ground water ingress into excavations within the valley floor. It is noted that with this option the pile cap would act as an obstruction during flooding.

The pile-group is assumed as two row of piles with spacing equal to four times the piles diameter (centre to centre).

Consideration has been given to pile diameters of 0.6m, 0.75m and 0.9m, and pile groups made of 8 piles and 12 piles.

3.3.4 Design ground conditions

The design ground and groundwater conditions, and associated design parameters are the same as the standard viaduct piers (see Section 3.2.4).

3.3.5 Design actions

The following were considered as actions for the ULS and SLS cases in accordance with BS EN 1997-1:2004:

- Superstructure dead load and bridge deck dead load.
- Column dead load.
- Highway vertical loading on the bridge deck.
- Highway skidding (transversal) on the bridge deck.
- Highway braking (longitudinal) on the bridge deck.
- Bearing friction due to thermal movements.
- Eccentricity due to construction tolerances on pile installation and bearing position.
- Eccentricity due to deflection at bearing level.

For the preliminary design calculations, the action of wind load has not been considered as it is anticipated to lead to lower bearing friction forces than the action of thermal movements. The action of wind load will be considered at detailed design stage.

The characteristic highway loading applied to the bridge deck was in accordance with the BS EN 1991-2 and UK National Annex (References [5] and [6]).

The bearings of both bridge decks have been assumed as being fixed in the longitudinal direction to provide restraint against lateral deflection of the bridge decks. The bearings of the south deck are assumed to be fixed in the transverse direction, at the connection to the pier, whilst the bearings for the north deck are assumed to be guided in the transverse direction at the connection with the pier; see Figure 2.

3.3.6 Design approach

Design objectives

As per Section 3.2.6.

Design standards

As per Section 3.1.6.

Serviceability Limit State (SLS)

As per Section 3.1.6.

Ultimate Limit State (ULS)

As per Section 3.1.6.

3.3.7 Design methodology

The design methodology was as detailed in Section 3.1.5. As the thickness of alluvium is small relative to the anticipated pile embedment length within the glacial till sands and gravels, the piles have been assumed to be entirely founded within the stiffer deposits. An additional length was added to the minimum pile length calculated above to account for the embedment within the alluvium.

It should be noted that if static pile tests are carried out, lower partial factors may be adopted for pile design in accordance with Clause A3.3.2 of NA A1 : 2014 (Reference [9]) of BS EN 1997-1 : 2004. The potential for carrying out static load tests and the benefits in terms of potential reductions in pile lengths should be considered at detailed design stage.

3.3.8 Design outcomes

Based on preliminary calculations the following outcomes have been estimated for the different options considered:

No. of pile in group	Pile diameter (m)	Estimated pile length (m)	Founding level (mAOD)	Horizontal deflection at bearing level (mm)	Vertical deflection (mm)
8	0.6	27.5	-19	<25	<25
8	0.75	20	-11.5	<25	<25
8	0.9	15	-6.5	<25	<50
12	0.6	22.5	-14	<25	<25
12	0.75	15	-6.5	<25	<50
12	0.9	12.5	-4	<25	<50

Table 9 – Preliminary design outcomes for fixed pier

3.4 River bridge – main span and back span

3.4.1 Structure and foundation type

At the river crossing, the main span and the back span are to be supported on pile groups, each acting as push pull systems, to accommodate the out of balance forces and the larger vertical loads.

3.4.2 Construction sequence

The construction sequence and the concrete washout consideration are assumed to be the same as for the Viaduct Fixed Pier (see section 3.3).

The river bridge will be crane-lifted into place.

3.4.3 Design assumptions

Geometry

The same geometry as the fixed pier has been assumed (see Section 3.3.3).

3.4.4 Design ground conditions

For the purpose of preliminary design, the design ground and groundwater conditions, and associated parameters have been assumed to be the same as the typical viaduct piers (see Section 3.2.4).

It should be noted that rockhead lies at a higher level at the proposed position of the river crossing, compared to the main viaduct to the south. Therefore there remains the potential to refine the outcomes of the preliminary design presented below, to take account of piled designed with a rock socket. This should be considered further at detailed design stage.

3.4.5 Design actions

The design actions for the ultimate limit state case (ULS) are in accordance with BS EN 1997-1:2004 using Design Approach 1 Load Combination 1 and Load Combination 2. For the serviceability limit state case (SLS) a factor of unity has been applied to all actions.

For preliminary design at Key Stage 3, the main span and back span foundations have been designed using preliminary loads derived at tender stage.

For detailed information on the design criteria and actions refer to the Approval in Principle (AIP) for the scheme (Reference [3]).

The following were considered as actions for the ULS and SLS cases in accordance with BS EN 1997-1:2004:

- Superstructure dead load and bridge deck dead load.
- Column dead load.
- Highway vertical loading on the bridge deck.
- Highway skidding (transversal) on the bridge deck.
- Eccentricity due to construction tolerances on pile installation and bearing position.

For the preliminary design calculations, the action of wind load has not been considered as it is anticipated to lead to lower bearing friction forces than the action of thermal movements. The action of wind load will be considered at detailed design stage.

The characteristic highway loading applied to the bridge deck was in accordance with the BS EN 1991-2 and UK National Annex (References [5] and [6]).

The southern bearing was considered to allow movements in the longitudinal direction and to restrict movements in the transversal direction. The northern bearings were assumed to be free in both directions; see Figure 2.

3.4.6 Design approach

Design objectives

As per Section 3.2.6.

Design standards

As per Section 3.1.6.

Serviceability Limit State (SLS)

As per Section 3.1.6.

Ultimate Limit State (ULS)

As per Section 3.1.6.

3.4.7 Design methodology

The design methodology was as detailed in Section 3.1.7.

It should be noted that if static pile tests are carried out, lower partial factors may be adopted for pile design in accordance with Clause A3.3.2 of NA A1 : 2014 (Reference [9]) of BS EN 1997-1 : 2004. The potential for carrying out static load tests and the benefits in terms of potential reductions in pile lengths should be considered at detailed design stage.

3.4.8 Design outcomes

Main span

Based on preliminary calculations the following outcomes have been estimated for the different options considered:

No. of piles in group	Pile diameter (m)	Estimated pile length* (m)	Founding level (mAOD)	Horizontal deflection at bearing level (mm)	Vertical deflection (mm)
8	0.6	45	-36.5	<25	<25
8	0.75	35	-26.5	<25	<25
8	0.9	25	-16.5	<25	<25
12	0.6	37.5	-29	<25	<25
12	0.75	27.5	-19	<25	<25
12	0.9	20	-11.5	<25	<25

*preliminary design has been based on piles supported by shaft friction only. If rockhead is present over the design pile length, a reduced length of pile is feasible based on consideration of a design rock-socket

Table 10 – Preliminary design outcomes for main span

Back span

Based on preliminary calculations the following outcomes have been estimated for the different options considered:

No. of piles in group	Pile diameter (m)	Estimated pile length* (m)	Founding level (mAOD)	Horizontal deflection at bearing level (mm)	Vertical deflection (mm)
8	0.6	27.5	-19	<25	<25
8	0.75	20	-11.5	<25	<25
8	0.9	17.5	-9	<25	<50
12	0.6	22.5	-14	<25	<25
12	0.75	17.5	-9	<25	<25
12	0.9	12.5	-4	<25	<50

*preliminary design has been based on piles supported by shaft friction only. If rockhead is present over the design pile length, a reduced length of pile is feasible based on consideration of a design rock-socket

Table 11 – Preliminary design outcomes for back span

It should be noted that the design outcomes above were estimated based on the sands and gravels as the founding stratum. Pile lengths may be reduced if bedrock is present over the preliminary design length indicated above, with the piles designed to a specified rock socket. For piles installed into rock, vertical deflections are likely to be significantly lower than indicated in Table 11.

3.5 Total and differential settlements

Total settlements of up to 50mm have been estimated for the piled foundations across the scheme. These total settlements will vary depending on whether single piles or pile groups are used, and whether piles are founded within the glacial sands and gravels, or socketed into rock.

As the foundation types and founding strata vary along the alignment of the viaduct and river crossing, there will be locations where adjacent piers/abutments will experience different magnitudes of total settlement. An upper bound value of 50mm has been estimated for the differential settlement between adjacent piers/abutments. This corresponds to the maximum total settlement estimated from the range of foundation options for the river crossing (50mm), compared to negligible settlement assumed for piles if they were to be founded into rock for the adjacent foundation to the north.

If the estimated total or differential settlements cannot be accommodate by the structure, pile lengths may be increased until the settlements are within reasonable limits. The settlement of the rock socketed pile is expected to be limited to the elastic shortening of the pile which will be negligible.

3.6 pH and Sulphate

Samples of soil and water were taken during the site investigation and tested for pH and water soluble sulphate in accordance with BRE Special Digest 1 (Reference [7]).

In accordance with the guidance provided in BRE Special Digest 1 (Reference [7]), the site has been identified as not lying in an area where pyrite may be present. Therefore, the design sulphate class has been assessed on the basis of pH and sulphate values only. Design sulphate classes and associated ACEC classes have been derived for each of the geological formations and are presented in Table 12 and 13 below. Full results are presented in Appendix C of Volume 2 of the GIR (Reference [2]).

Formation	**SO_4 (mg/l)**		**pH**		**Number of samples**	**Design Sulphate Class**	**ACEC Class**	**DC Class**
	Range	Characteristic value	Range	Characteristic value				
Made Ground (brownfield)	<20 - 90	90	5.7 - 11.2	5.7	4	DS-1	AC-2z	DC-2z
Alluvial deposits	<20 - 20	20	5.4 - 6.0	5.4	6	DS-1	AC-2z	DC-2z
Glacial sands and gravels	<20 - 20	20	7.8 - 8.1	7.8	7	DS-1	AC-1	DC-1

Formation	**SO_4 (mg/l)**		**pH**		**Number of samples**	**Design Sulphate Class**	**ACEC Class**	**DC Class**
	Range	Characteristic value	Range	Characteristic value				
Bedrock	<20 - 120	120	8.5 - 8.7	8.5	3	DS-1	AC-1	DC-1

Table 12 – Summary of design sulphate classes and ACEC classes for soil

Formation	**SO_4 (mg/l)**		**pH**		**Number of samples**	**Design Sulphate Class**	**ACEC Class**	**DC Class**
	Range	Characteristic value	Range	Characteristic value				
Alluvial deposits	2.7 – 4.0	4.0	5.2 - 5.6	5.2	2	DS-1	AC-2z	DC-2z
Glacial sands and gravels	5.3 – 7.2	7.2	5.3 - 5.5	5.3	4	DS-1	AC-2z	DC-2z

Table 13 – summary of design sulphate classes and ACEC classes for groundwater

4 Strengthened earthworks

Not used.

5 Drainage

Surface water runoff from the new highway is to be discharged into watercourses at three locations, with attenuation measures likely to be provided upstream of the discharge points. An outfall is proposed into the Afon Dyfi in the north of the scheme, and two discharges are proposed into existing drainage ditches at the southern end of the scheme, adjacent to the proposed approach embankment and the area where the scheme ties into the existing A487.

Where the new length of side road and the farm access track are proposed in the north of the scheme, it is recommended that allowance is made for cut-off drainage ditches at the crests of the cuttings. Toe drains, comprising a filter drain within the verge, should also be provided.

Cut-off drainage ditches are recommended at the toes of the embankments, for drainage of surface water that may flow towards the embankment, and down the embankment slopes.

6 Pavement design, subgrade & capping

6.1 Approach embankments

The fill to be used for construction of the approach embankments at the northern and southern ends of the scheme will be site-won. The source of the fill will be from the rock cuttings that are proposed for the side road and farm access track to the north, which typically comprise a shallow depth of superficial deposits (head deposits) over mudstone bedrock. The CBR value associated with this fill will be dependent on the proposed method of excavation, crushing/sorting, and placement. However, at this stage, it is reasonable to anticipate that values in excess of 5% should be achievable if the fill placed at sub-formation level is derived from the rock section of cuttings. Should fill derived from the superficial deposits be placed in the upper sections of the proposed embankments, lower CBR values may be applicable.

6.2 Cuttings at the northern end of the scheme

Where the side road and farm access track are proposed in the north of the scheme, these are to be founded on competent rock. It is reasonable to anticipate that CBR values in excess of 5%, and possibly much higher, will be achieved at formation level. Further testing shall be undertaken as there is a risk of the mudstone breaking down to an argillaceous material in the long term.

6.3 Areas at-grade

At the southern end of the scheme, where the shallow embankment will tie into the existing highway, the highway will be formed over low strength cohesive alluvium. Whilst there is a desiccated crust present over the ground surface, low CBR values, possibly lower than 2% may be applicable. These should be further investigated as part of a detailed ground investigation. It should be noted that there is a risk that the desiccated crust of the alluvium may soften if exposed to wet weather, which should be carefully considered during planning of construction.

At the northern end of the scheme, at the tie into the existing highway, the highway will be formed over variable shallow superficial deposits which include areas of Made Ground, silts and sands. CBR values are likely to be variable, possibly as low as or below 2% in places. This should be further investigated as part of a detailed ground investigation.

7 Assessment of potential contamination

7.1 Summary of the extent of the contamination testing that has been undertaken

The potential for contamination was assessed based on the logs and laboratory chemical tests undertaken on soil and water samples from the recent ground investigation.

7.1.1 Soil

Ground investigations undertaken along the proposed route encountered Made Ground in the south, possibly associated with a field track, and in the north associated with the Millennium Bridge and the former Corris Railway bridge.

Made Ground encountered in the southern end of the proposed route typically comprised clayey, silty gravel as well as brick fragments at one location. Thicknesses were between 0.3m and 0.5m beneath the topsoil.

Made Ground typically comprising clayey, silty sands and gravel and fragments of man-made materials, including plastic, glass, ceramic, brick, mortar, glass, tarmac, concrete, slate, roofing tile, coal and cinders were encountered on the northern bank of the Afon Dyfi from the ground surface to maximum proven depth of 8.6m, although a 5m depth was used for design due to its density.

Inclusions of tarmac, coal and cinders present potential sources of metals or hydrocarbons contamination, primarily of Polycyclic Aromatic Hydrocarbons (PAHs).

Inclusions of concrete and bricks may indicate the presence of demolition materials, a potential source of asbestos.

No other obvious evidence of contamination (including hydrocarbon or asbestos) was encountered during the fieldwork.

The results of the laboratory chemical testing indicate the presence of some contaminants including metals, PAHs and TPH within the soils at maximum recorded concentrations as follows:

- Arsenic at 21.3 mg/kg in TP3 at 0.4m
- Beryllium at 1.2 mg/kg in TP2 at 0.4m
- Cadmium at 0.9 mg/kg in TP14 at 0.6m
- Chromium at 43.1 mg/kg in TP19 at 0.4m
- Copper at 128 mg/kg in TP5 at 0.25m
- Lead at 718 mg/kg in TP14 at 0.6m
- Nickel at 52 mg/kg in TP19 at 0.4m

- Vanadium at 42.6 mg/kg in TP20 at 0.3m
- Zinc at 258 mg/kg in TP6 at 0.3m
- Total Petroleum Hydrocarbons at 343 mg/kg in TP20 at 0.3m
- Total PAHs at 6.08 mg/kg in TP01 at 0.4m
- No asbestos has been identified within the analysed samples.

7.1.2 Groundwater

A total of two rounds of groundwater sampling were undertaken. The samples were obtained from boreholes BH2, BH3 and BH4, all located to the south of the river; see Figure 3. No groundwater sampling was undertaken on the northern river bank however this will be undertaken during the detailed ground investigation.

The testing results for metals and inorganic compounds obtained from the first round of testing met the quality assurance requirements and therefore are considered representative. However, the results of chemical analyses of organic compounds such as petroleum hydrocarbons or Polycyclic Aromatic Hydrocarbons (PAHs) can only be used as indicative. This is due to the holding time being exceeded resulting in the sample deterioration and the measured concentrations potentially being lower than those present within the original sample.

All results of the second round of testing meet the quality assurance requirements.

No evidence of contamination was observed during monitoring.

In summary, the following determinands were detected:

- Calcium at between 4.31 and 14.6 mg/l (Round 1 and 2)
- Magnesium at between 2.34 and 7.84 mg/l (Round 1 and 2)
- Zinc at between 15 and 29 ug/l (Round 1 and 2)
- Total Petroleum Hydrocarbons at between <5 and 39.5 ug/l of aromatic compounds with carbon banding C21 to 35 (Round 1), and 108 and 260 ug/l of aliphatic compounds C8 to C10 and C16 to 21, and aromatic compounds C16 to 35 (Round 2). The increase in concentrations is a result of the presence of a greater range of petroleum hydrocarbon compounds. The source of these hydrocarbons is unclear but the range of the detected hydrocarbon fractions indicate a possible ongoing off-site source of hydrocarbon contamination. This is not considered significant in a context of the proposed scheme.
- Total Polycyclic Aromatic Hydrocarbons (PAHs) at between 0.4 and 1.59 ug/l (Round 1) and 0.37 and 2.11 ug/l (Round 2)
- pH was measures at between 5.3 and 5.4, which is considered mildly acidic.

The laboratory results are provided in the Volume 2 of the Ground Investigation Factual Report (Reference [2]).

7.2 Summary of the findings and conclusions of the risk assessments including the site remediation requirements that have been agreed with the regulatory authorities

The risk assessments for the scheme have been undertaken as part of the Environmental Statement considerations in relation to the effects that the scheme may have on land contamination issues (Reference [10]), in line with current industry best practice, including CLR11 (Reference [13]). In summary, the process comprises a tiered approach which starts with a simple and conservative Tier 1 assessment of potential risks from possible Pollutant Linkages (Source-Pathway-Receptor). Any potential risks identified at Tier 1 have then been studied in more detail through a Tier 2: Generic Quantitative Risk Assessment (GQRA) and, if necessary, a Tier 3: Detailed Quantitative Risk Assessment (DQRA).

The risk assessment process is underpinned throughout by the development of the Conceptual Site Model (CSM) which provides a schematic representation of the identified contaminated linkages.

The following assessment criteria have been applied where a potential pollution linkages have been identified in relation to:

- **Human health from direct exposure to soils:** The available soil chemical test results have been screened against published generic assessment criteria for a suitable land use scenario, such as DEFRA Category 4 Screening Levels (C4SLs) (Reference [14]), and where these are not available, the LQM/CIEH Suitable 4 Use Levels (S4ULs) (Reference [15]).

  Risk assessment for construction and maintenance workers has been undertaken applying a residential end use scenario due to the direct exposure to soils during any intrusive works. This is a conservative approach however it is considered suitable for identification of potential risks requiring consideration as part of the health and safety assessment of the works. The risk to site end users and scheme neighbours (workers of the Eco Park) has been undertaken for a commercial end use scenario.

- **Human health from direct exposure to groundwater:** the GQRA involves screening of the groundwater chemical testing results against Drinking Water Standards.

- **Controlled waters:** This is achieved by screening available water chemical testing results against the Environmental Quality Standards (Reference [16]) for annual average inland surface water (freshwater) values.

For hardness dependent determinands e.g. cadmium, the EQS value has been set at Class 2 hardness range. This is based on Water Hardness Map (Reference [17]).

Where the EQS is dependent on bioavailability, which is the case for copper, manganese and zinc, for the purpose of the assessments, it has been conservatively assumed that the measured concentrations reflect the bioavailable dissolved metals.

The Environmental Statement is currently under review with key environmental Stakeholders prior to submission as part of the planning process.

The potential risks posed by the scheme were assessed following a methodology set out in Reference [10], which is in line with current regulatory guidance. For the purpose of these assessments, the scheme was split into three sections characterised by different construction elements:

- approach embankment Ch. 0+000m to 0+220m and flood bunds,
- viaduct construction and river crossing Ch. 0+220m to Ch. 0+940m, and
- earthworks on the north river bank Ch. 0+940m to Ch. 1+100m.

A conceptual site model was derived for both construction and operations phases, which identified a number of potential pollution linkages together with the conclusions of the assessments have been summarised in Table 14 below.

Scheme element	**Identified receptors and pathways**	**Outcome of the risk assessments**
Construction		
General	Impact on human health (construction workers and scheme neighbours) and controlled waters due to localised unexpected contamination	An action plan should be developed and implemented for the construction works. This would be presented in the Construction Environmental Management Plan developed for the Scheme.
Approach Embankment and flood bunds	Impact on human health (construction workers and the Dyfi Eco Park workers) due to exposure to soils and groundwater impacted by contamination	No exceedances of applied assessment criteria were identified. The soils and groundwater are unlikely to pose a significant risk to identified receptors.
Viaduct Construction	Impact on human health (construction workers) due to exposure to groundwater impacted by contamination	Construction workers may be exposed to hydrocarbon impacted groundwater, also exhibiting a low pH. Although risk of a significant impact on human health is considered to be low, appropriate health and safety risk assessments required to further mitigate the risks.
Earthworks on the North River Bank	Impact on human health (construction workers and the A487 maintenance workers) due to exposure to soils impacted by contamination	No exceedances of applied assessment criteria were identified. The soils and groundwater are unlikely to pose a significant risk to identified receptors.

Scheme element	Identified receptors and pathways	Outcome of the risk assessments
	Impact on the river quality due to groundwater discharge as part of dewatering measures.	No data on groundwater quality is available therefore a potential impact on the river quality is unknown. Further investigations and assessments are required at the detailed design stage.
Operation		
Approach Embankment and flood bunds	Impact on human health (maintenance workers) due to exposure to soils and groundwater impacted by contamination	No exceedances of applied assessment criteria were identified. The soils and groundwater are unlikely to pose a significant risk to identified receptors.
	Leaching of contaminants from soils reused in construction of the embankment to groundwater	No data on soil leachate quality is available therefore a potential impact on the groundwater quality from site won soils reused within the embankment is unknown. The Specification for Highway Works (SHW) sets out requirements for all earthworks materials for reuse to be sampled, tested for potential contamination, and screened against threshold values for human health and the water environment. This would provide verification safeguards against potential adverse effects. This is to be developed at the detailed design stage.
Viaduct Construction	Impact on groundwater due to potential contamination migration along the piles	Groundwater contained within superficial deposits was found to be impacted by hydrocarbons and therefore there is a risk of vertical contamination into the groundwater contained within bedrock. A foundations works risk assessment is required at the detailed design stage to confirm the risks.
Earthworks on the North River Bank	Surface run-off and leaching of contaminants from soils exposed at surface as a result of construction works to surface water	No data on soil leachate quality is available therefore a potential impact on the surface water quality from soils exposed as a result of the scheme construction is unknown. Further investigations including soil leachate testing and assessments are required at the detailed design stage.

Table 14 – Summary of identified pollution linkages and conclusions of risk assessments presented in Reference [10].

In summary, the construction and operation works are unlikely to pose a significant risk to the identified human and environmental receptors. No site remediation requirements have been agreed with the regulatory authorities at this stage.

Although no remediation measures are considered necessary, the risk to the identified receptors could be further limited by the development of an action plan in relation to encountering unexpected contamination. Investigations into the groundwater quality in the northern part of the scheme would be required to inform the design of temporary works primarily associated with dewatering activities. This is in addition to implementation of appropriate controls for the reuse of materials and undertaking foundations works risk assessment.

7.3 Details of contaminated material to be removed from site

No specific areas of contaminated material have been identified at this stage. As presented above, further investigation as part of the detailed ground investigation at Key Stage 6, will be required particularly with respect to the confirmation of their suitability for reuse within the scheme and groundwater quality in the northern part of the scheme.

Following this detailed ground investigation, further assessments will be carried out as part of detailed design of the scheme. Acceptability criteria and a suitable material sampling and testing regime will be established for the construction phase, with the requirements to be incorporated into the earthworks specification for the scheme. Any material encountered that proves unacceptable for reuse during the works on account of it being in exceedance of the agreed acceptability criteria, will be required to be disposed of at a suitably licenced facility.

8 Ground treatment including treatment of any underground voids etc.

Ground treatment is only potentially required for the treatment of the soft ground underlying the southern embankment (see Section 2.2.1).

9 Specification appendices

The specification appendices shall be prepared as part of Keys Stage 6, detailed design.

10 Instrumentation and monitoring

10.1 Full details of purpose, installation requirements, restrictions and frequency of readings

To be confirmed at detailed design stage based on outcome of design.

10.2 Use of Observational Methods/Controls. Predicted and critical readings and restrictions on work

To be confirmed at detailed design stage based on outcome of design.

10.3 Pile testing requirements

To be confirmed at detailed design stage based on outcome of design.

11 References

[1] Report No. 900237-ARP-ZZ-ZZ-RP-CG-00002, A487 New Dyfi Bridge, Preliminary Sources Study Report, ISSUE 1 (Arup, March 2016).

[2] Report No. 900237-ARP-ZZ-ZZ-RP-CG-00003, A487 New Dyfi Bridge, Preliminary Ground Investigation Report Volumes 1 and 2, ISSUE 1 (Arup, July 2016).

[3] Report No. 900237-ARP-XX-XX-CB-RP-00003, A487 New Dyfi Bridge, Bridge Approval in Principle, DRAFT 2. (Arup, July 2016).

[4] Document No. 110_07, Technical design details – embankments, Version 1 (Environment Agency, January 2007).

[5] BS EN 1991-2 : 2003 – Eurocode 1: Actions on structures. Traffic loads on bridges (BSI, 2003)

[6] National Annex to BS EN 1991-2 (BSI, 2008)

[7] Special Digest 1 : 2005 – Concrete in aggressive ground (BRE, 2005).

[8] BS EN 1997-1 : 2004, Eurocode 7 – Geotechnical Design (BSI, 2004).

[9] National Annex to BS EN 1997-1 (BSI, 2007).

[10] Llywodraeth Cymru/ Welsh Government, A487 New Dyfi Bridge, Environmental Statement – Chapter 10, Geology and Soils (Arup 2016).

[11] PD 6694-1 : 2001 – Recommendations for the design of structures subject to traffic loading to BS EN 1997-1:2004 (BSI, 2004)

[12] Janbu, Bjerrum and Kjaernsli's chart interpreted. Christian, J. T. and Carrier, W. D. (Canadian Geotechnical Journal, Volume 15, pp. 123-8, 1978)

[13] Model Procedures for the Management of Land Contamination (CLR11) (Environment Agency and Defra 2004)

[14] SP1010 Development of Category 4 Screening Levels for Assessment of Land Affected by Contamination – Policy Companion Document, (Department for Environment Food and Rural Affairs (Defra) 2014)

[15] The LQM/CIEH S4ULs for Human Health Risk Assessment (Land Quality Management (LQM) / Chartered Institute of Environmental Health (CIEH) 2015)

[16] The Water Framework Directive (Standards & Classification) Directions (England and Wales) (Department for Environment Food and Rural Affairs (Defra) 2015)

[17] Water Hardness Map (Drinking Water Inspectorate 2001)

ANNEX 1

Completed Specification Table 1/5 and series 500, 600 and 1600 appendices including tables 6/1, 6/2 and 6/3.

The specification appendices shall be prepared as part of Key Stage 6, detailed design.

ANNEX 2

Completed SEAFs

Not used.

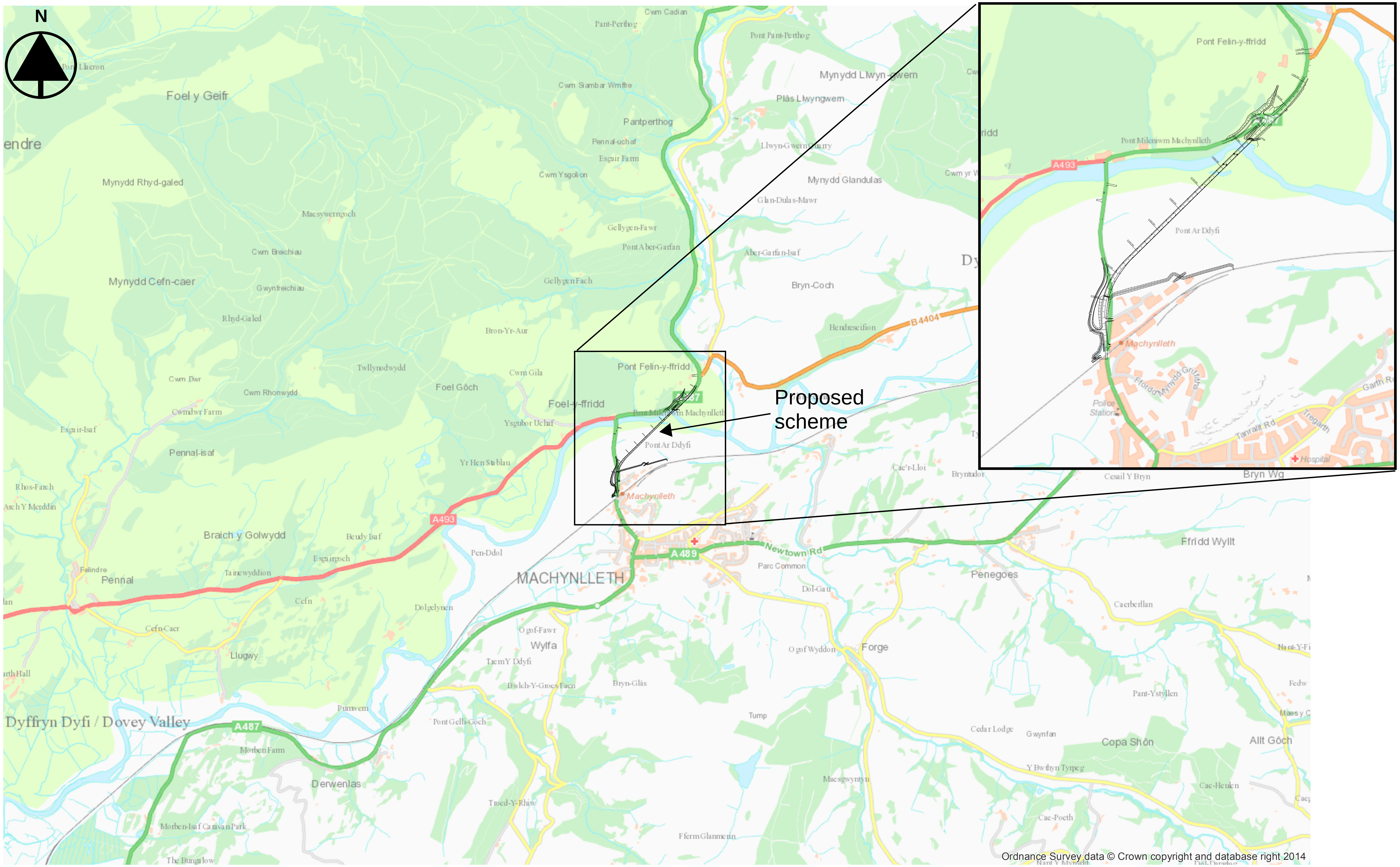
Figures

Figure 1 – Location Plan

Figure 2 – General Arrangement

Figure 3 – Ground Investigation Location Plan

Figure 4 – Long Section

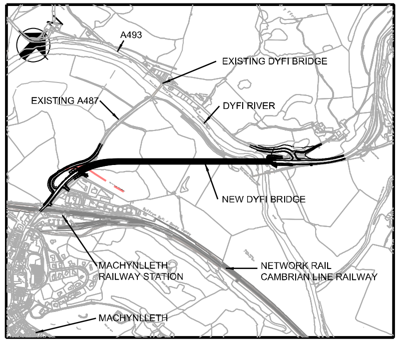


The scheme is centred approximately at National Grid Reference 274600, 301700

Site Location

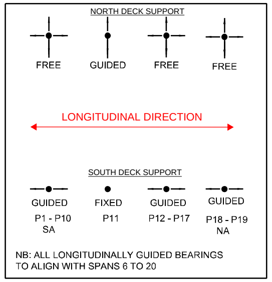
244562

FIGURE **01**



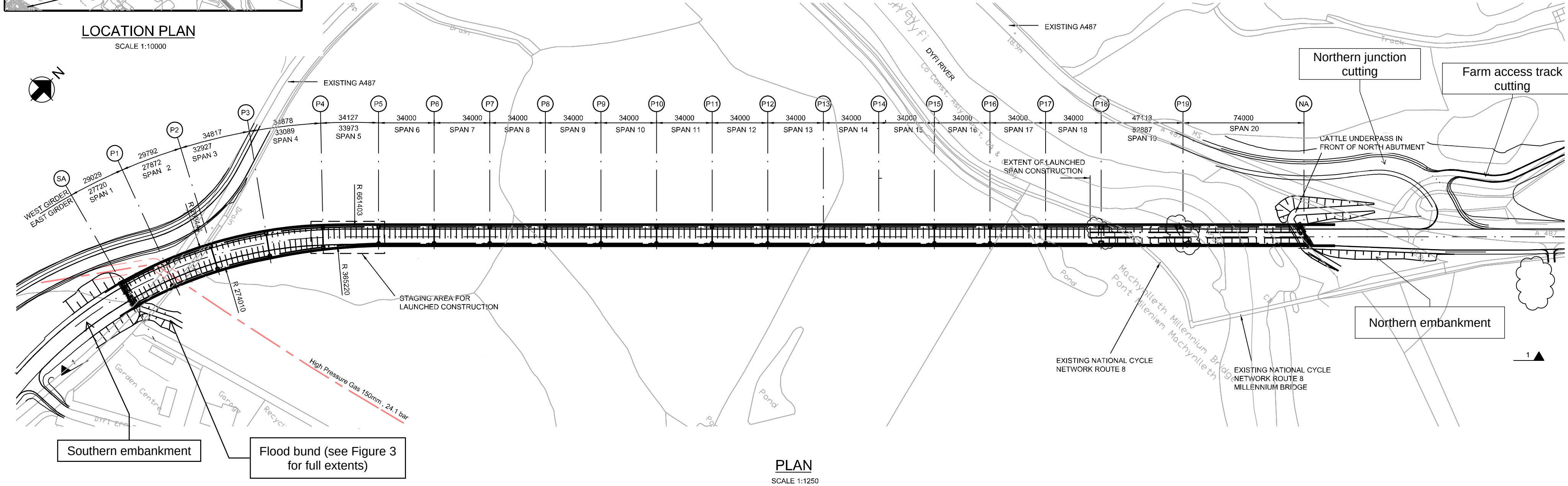
LOCATION PLAN

SCALE 1:10000



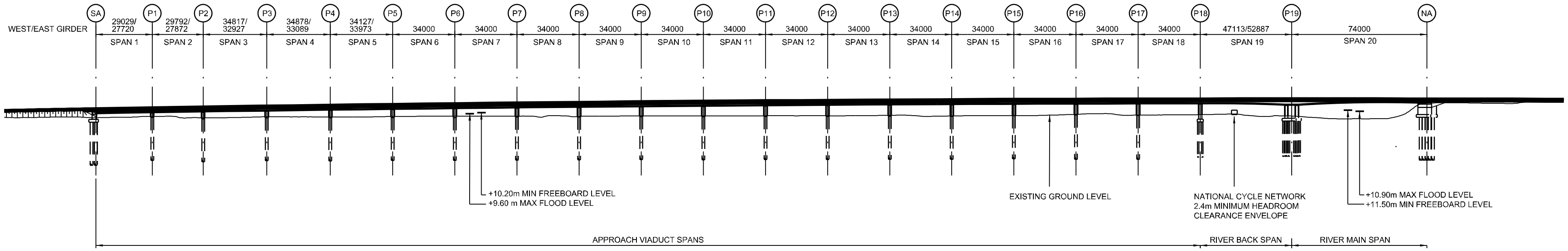
ARTICULATION PLAN

SCALE NTS



PLAN

SCALE 1:1250



ELEVATION 1-1

SCALE 1:1250

NOTES

1. THIS DRAWING REPRESENTS A PRELIMINARY DESIGN ONLY FOR PURPOSE OF ILLUSTRATING THE STRUCTURAL FORM AND DESIGN INTENT. CONFIRMATION OF DESIGN INCLUDING THE KEY MEMBER SIZES IS SUBJECT TO DETAILED DESIGN AND THE RESULTS OF THE GEOTECHNICAL INVESTIGATION;
2. THIS DRAWING IS TO BE READ IN CONJUNCTION WITH THE BRIDGE APPROVAL IN PRINCIPLE, DOCUMENT NO. 900237-ARP-XX-XX-CB-RP-00003;
3. THIS DRAWING IS TO BE READ IN CONJUNCTION WITH THE PROJECT CDM RISK REGISTER;
4. THE POSITION OF BOREHOLES SHOWN ARE APPROXIMATE;
5. FLOOD LEVELS VARY ALONG THE BRIDGE LENGTH. 1 IN 100 YEAR PLUS CLIMATE CHANGE FLOOD AND FREEBOARD LEVELS AS GIVEN ARE TO BE CONFIRMED DURING DETAILED DESIGN;
6. ALL DIMENSIONS ARE IN MILLIMETRES UNLESS STATED OTHERWISE;
7. ALL LEVELS ARE IN METERS ABOVE ORDNANCE DATUM (AOD) UNLESS STATED OTHERWISE.

Issue	Date	By	Chkd	Appd
P02	14/03/16	IE	DJ	SH
RIVER AND BACK SPANS AMENDED 5 SPANS ADDED AT SOUTH END				

ARUP
GRIFFITHS civil engineering and construction
WHS
Blythe Gate Blythe Valley Park
Solihull B90 8AE
Tel +44 121 213 3000 Fax +44 121 213 3001
www.arup.com

Client

Llywodraeth Cymru
Welsh Government

Project Title

A487 New Dyfi Bridge

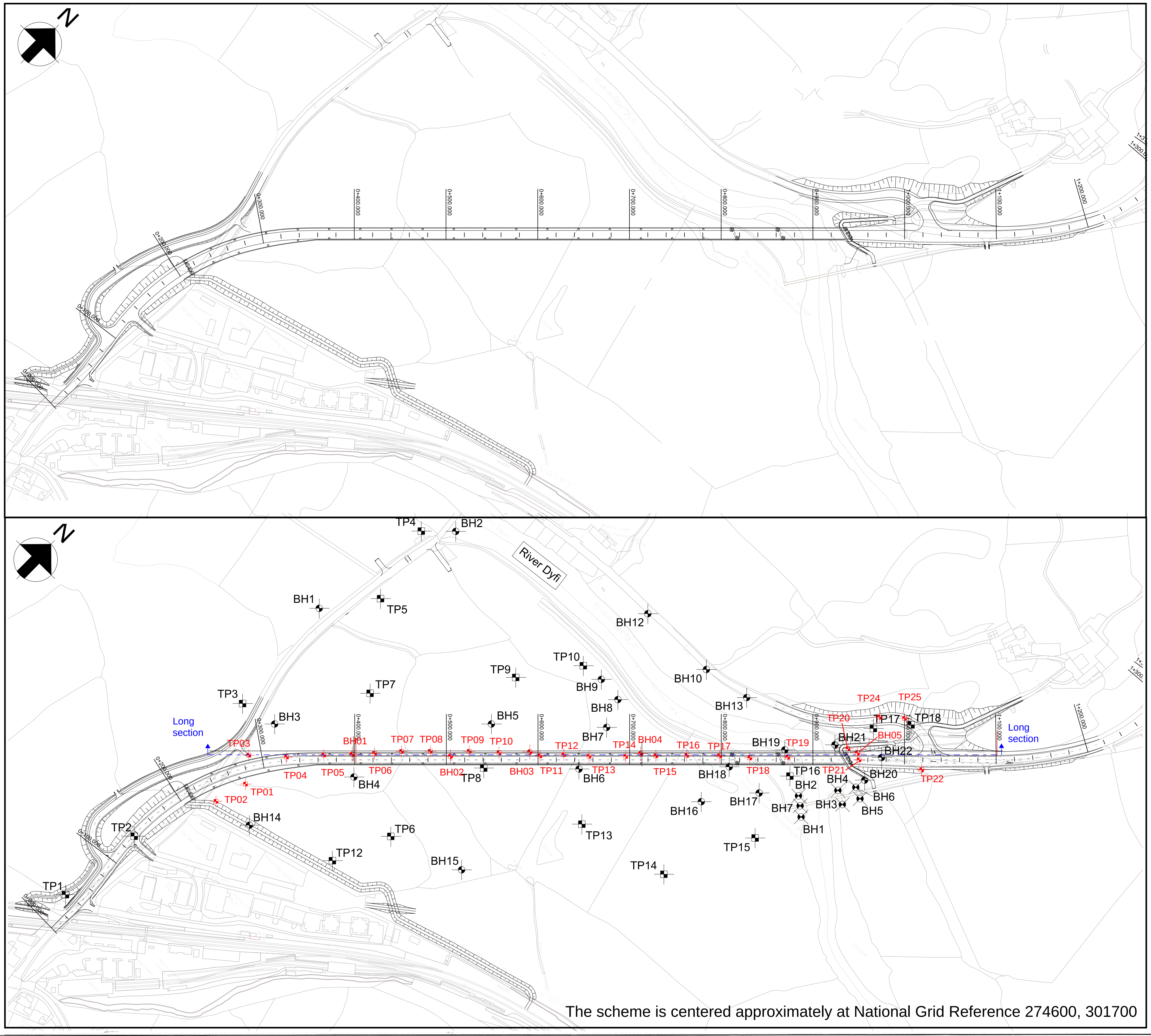
Drawing Title

New Dyfi Bridge
General Arrangement
Sheet 1 of 2

General Arrangement

FIGURE 02

Do not scale



Legend

- TP1 - Trial Pits
  CJ Associates (2001)
- BH1 - Boreholes
  CJ Associates (2001)
- BH1 - Boreholes
  CJ Associates (1999)
- TP1 - Trial Pits
  CC Geotechnical (2015-2016) - Key Stage 3
- BH1 - Boreholes
  CC Geotechnical (2015-2016) - Key Stage 3

Figure Title

EXPLORATORY HOLE LOCATIONS

FIGURE 03

SCALE V = 1:500, H = 1:1000 @ A3

Figure Title

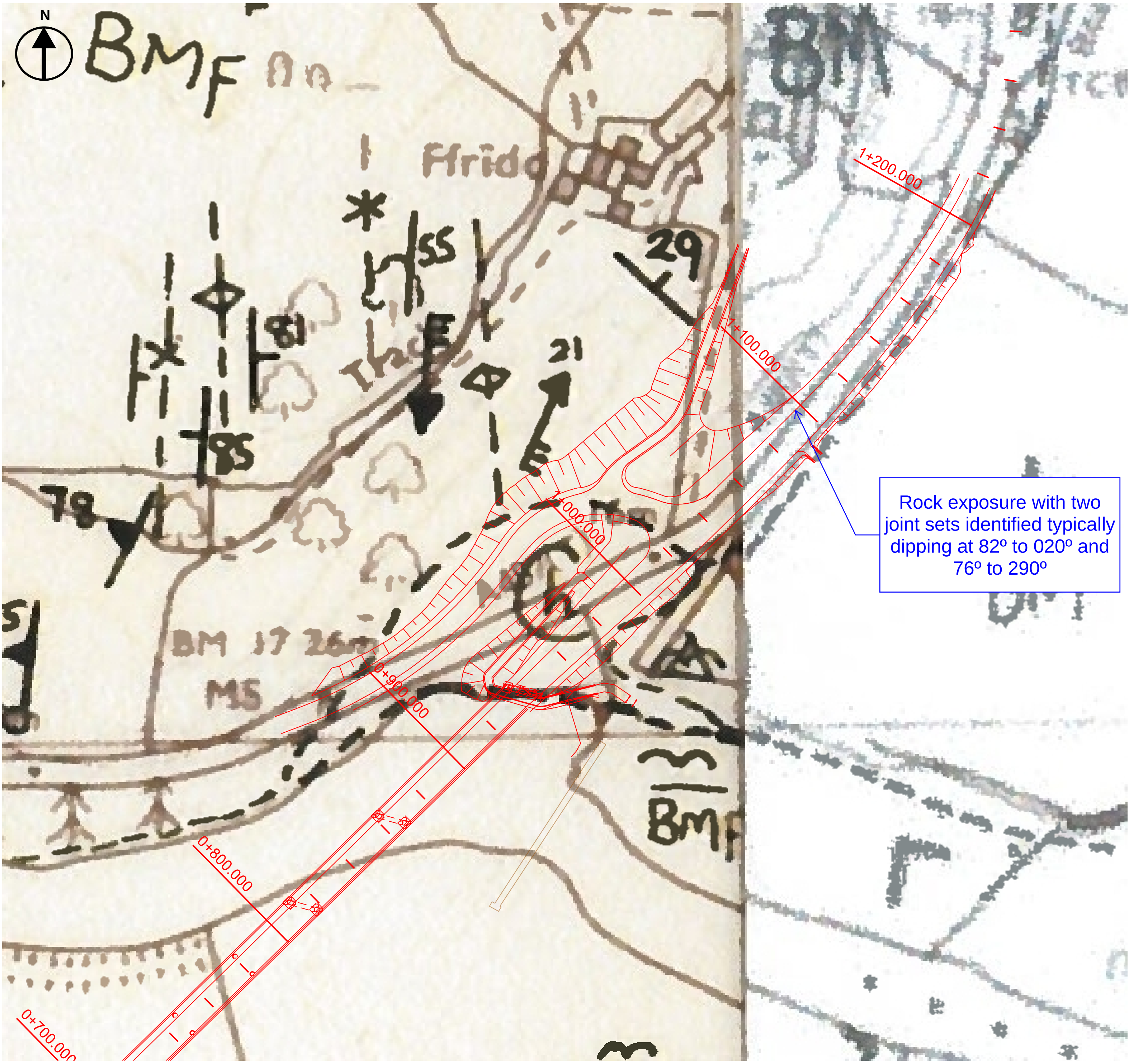
GEOLOGICAL LONG SECTION

FIGURE 04

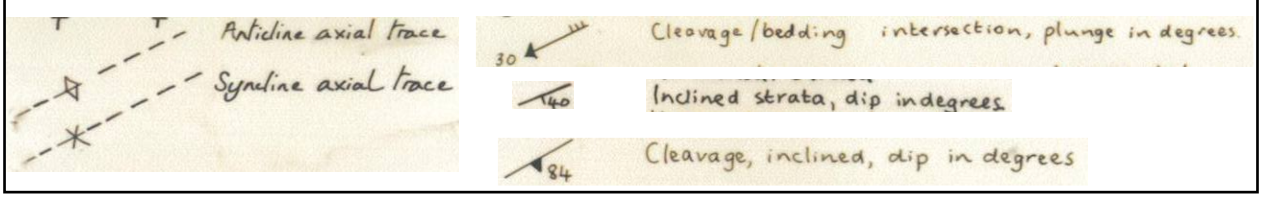
Appendix A

Northern cutting - geological map overlay

British Geological Survey 1 in 10,000 geological maps extracts from sheet SH 70 SW and SH 70 SE



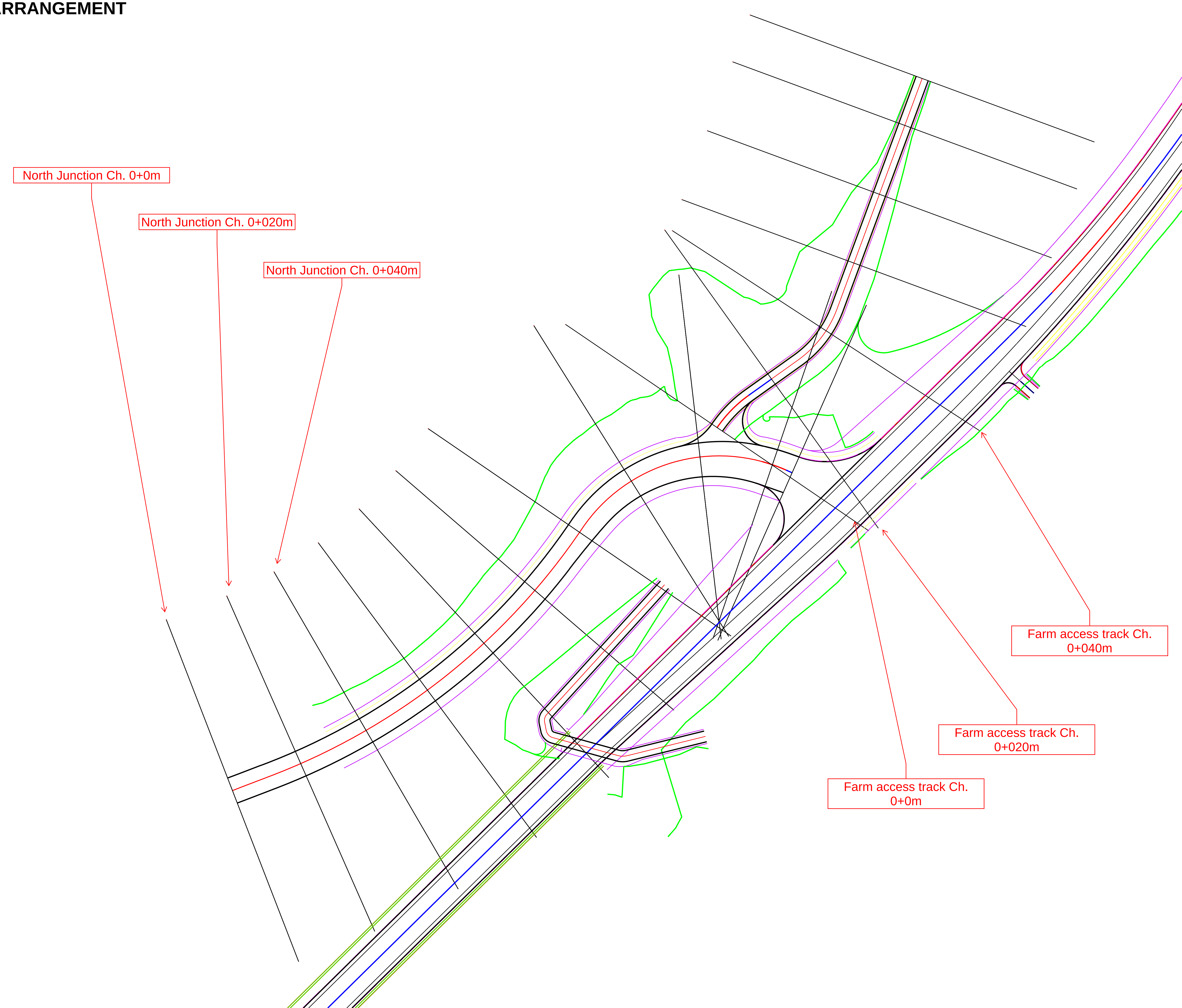
Key



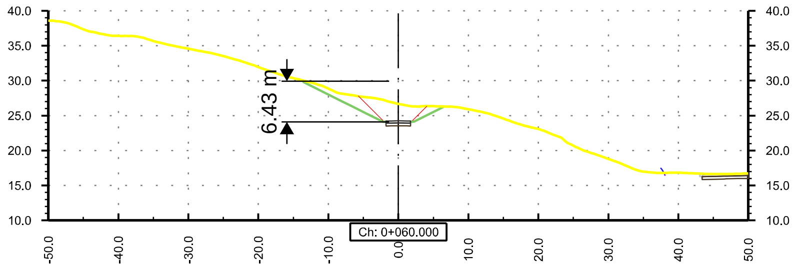
Appendix B

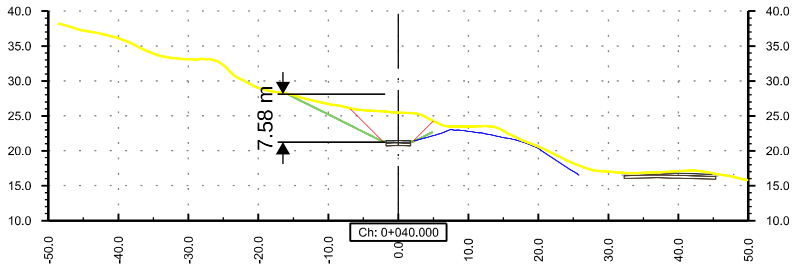
Northern cutting - cross sections

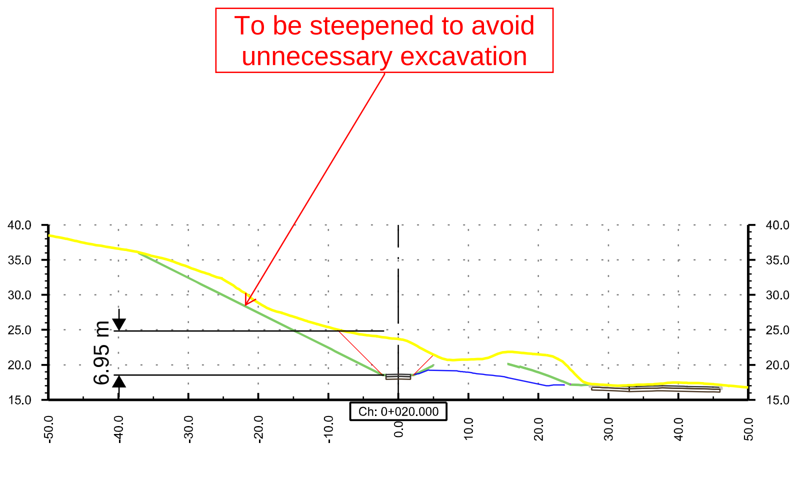
**NORTHERN CUTTINGS
GENERAL ARRANGEMENT**



FARM ACCESS
CROSS SECTIONS







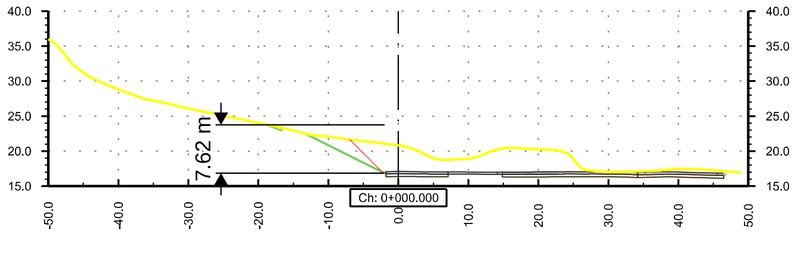
Legend

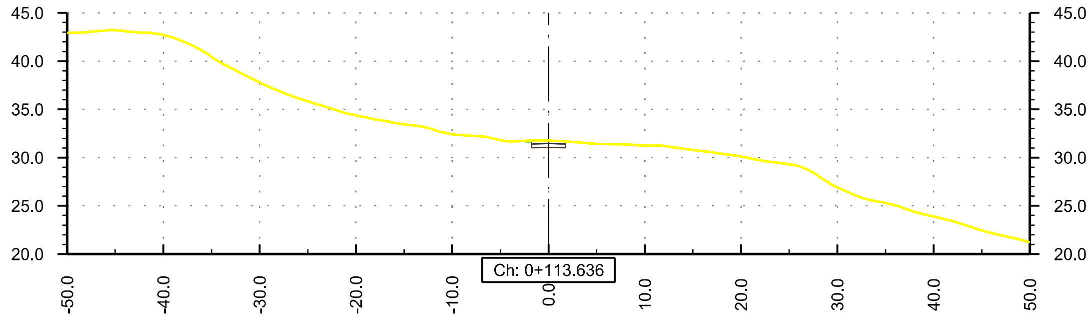
Cutting at 1 in 2 slope

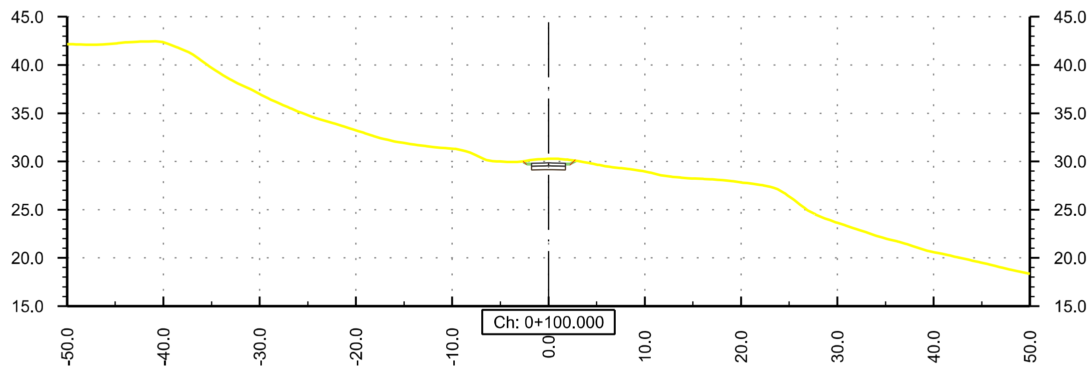
Cutting at 1 in 1 slope

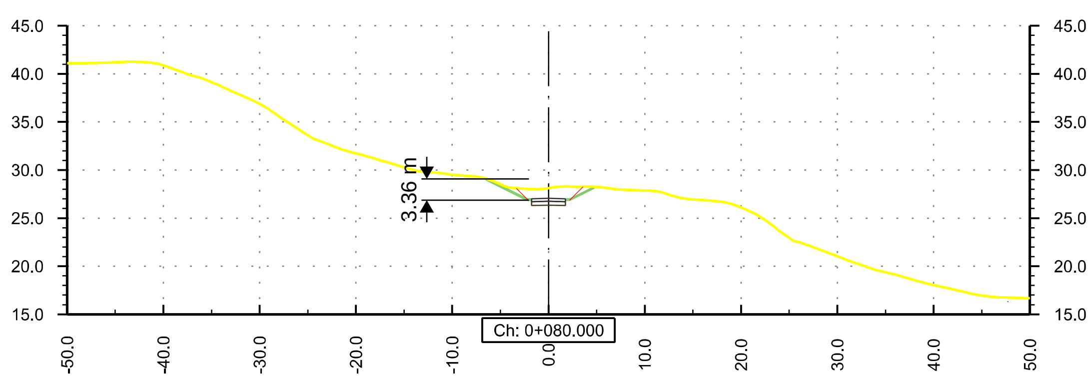
Smoothening between areas of cutting

Existing topography









Legen
d

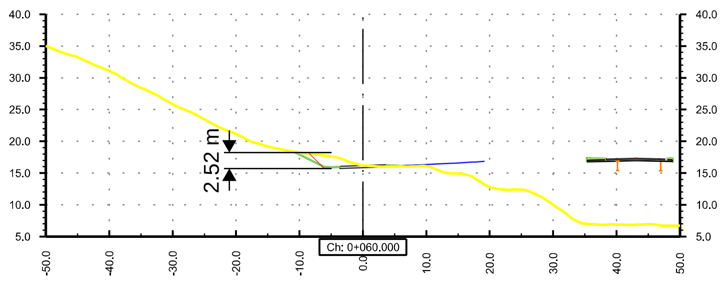
Cutting at 1 in 2 slope

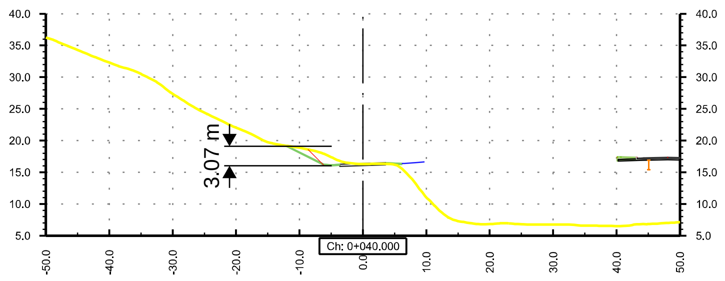
Cutting at 1 in 1 slope

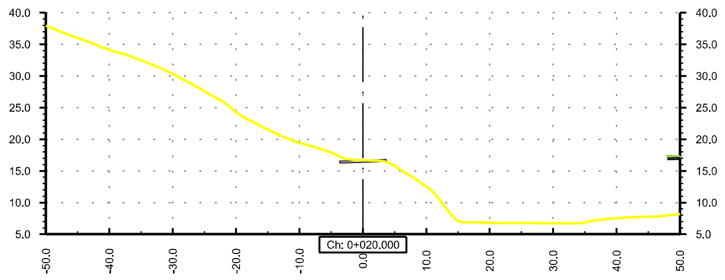
Smoothening between areas of cutting

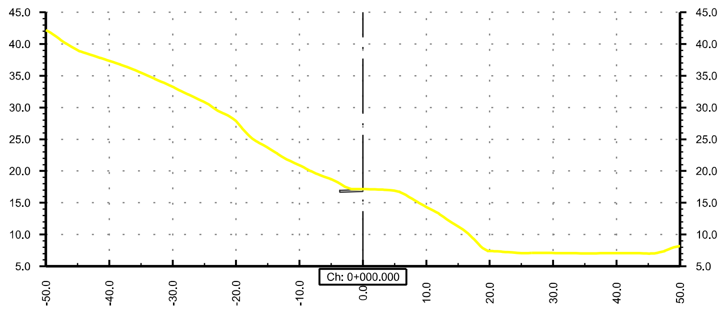
Existing topography

NORTH JUNCTION
CROSS SECTIONS



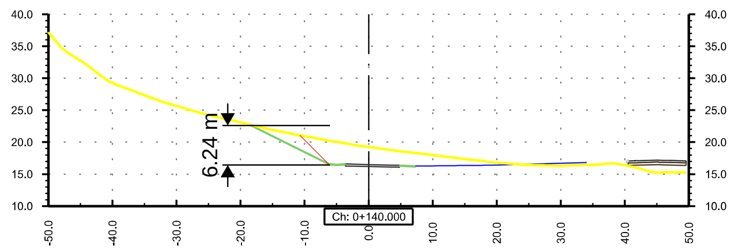


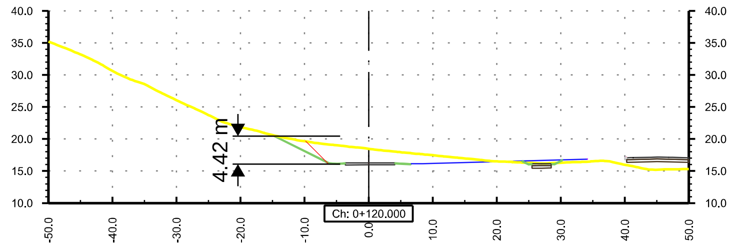


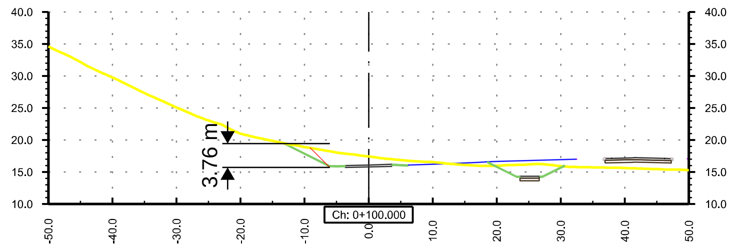


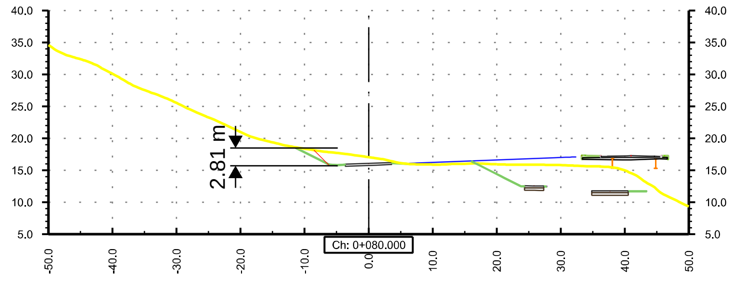
Legend

- Cutting at 1 in 2 slope
- Cutting at 1 in 1 slope
- Smoothening between areas of cutting
- Existing topography









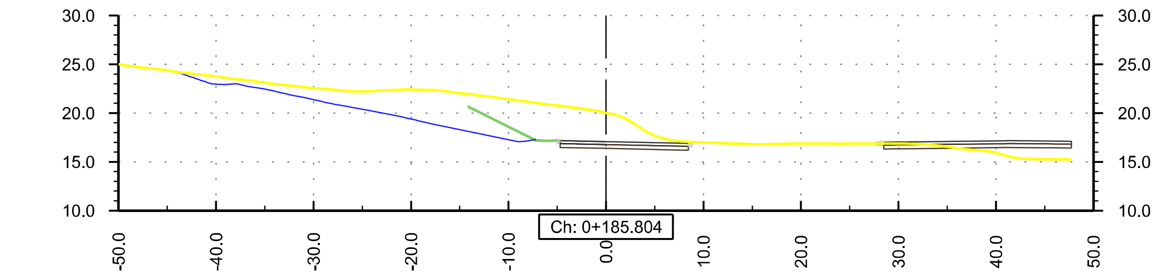
Legend

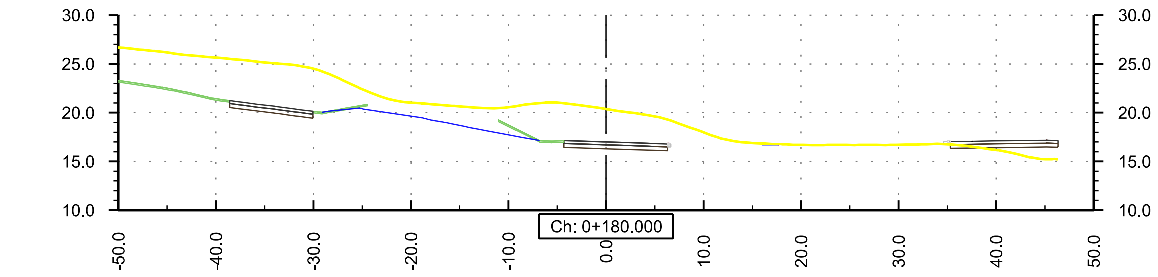
Cutting at 1 in 2 slope

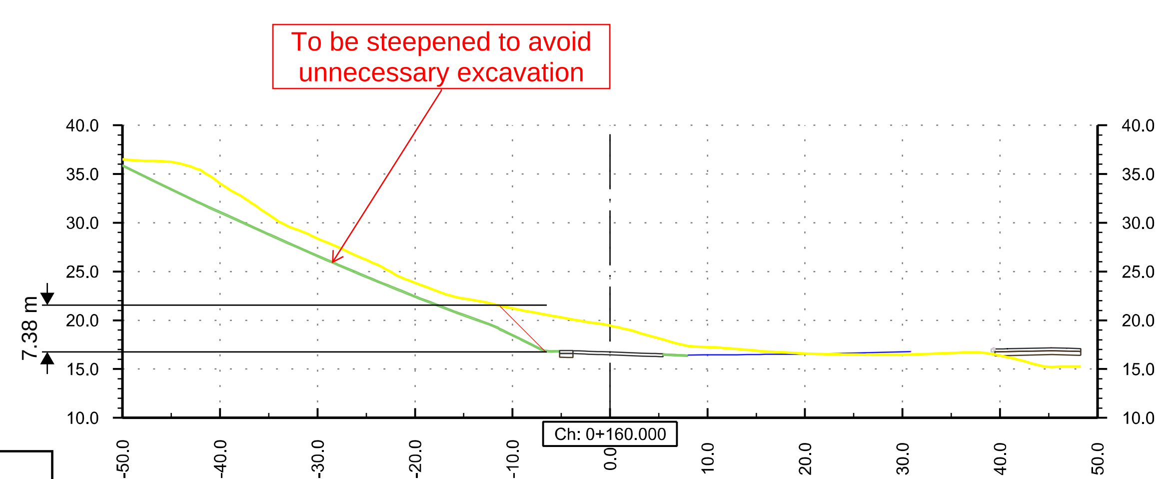
Cutting at 1 in 1 slope

Smoothening between areas of cutting

Existing topography







Legend

Cutting at 1 in 2 slope

Cutting at 1 in 1 slope

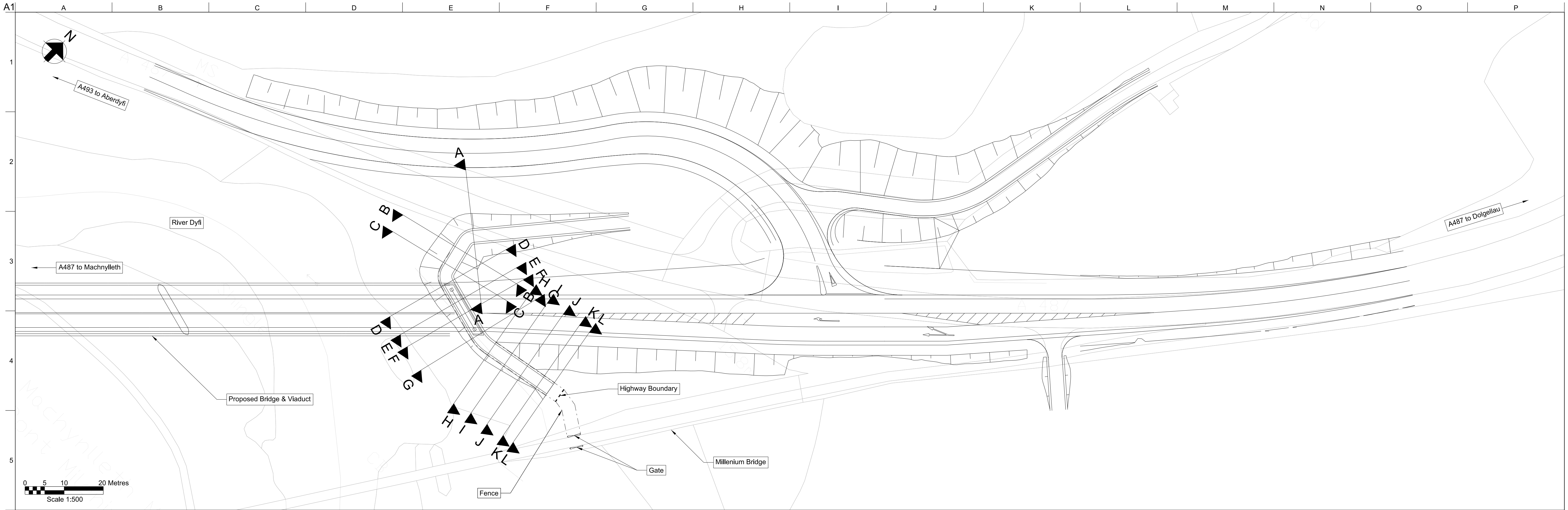
Smoothening between areas of cutting

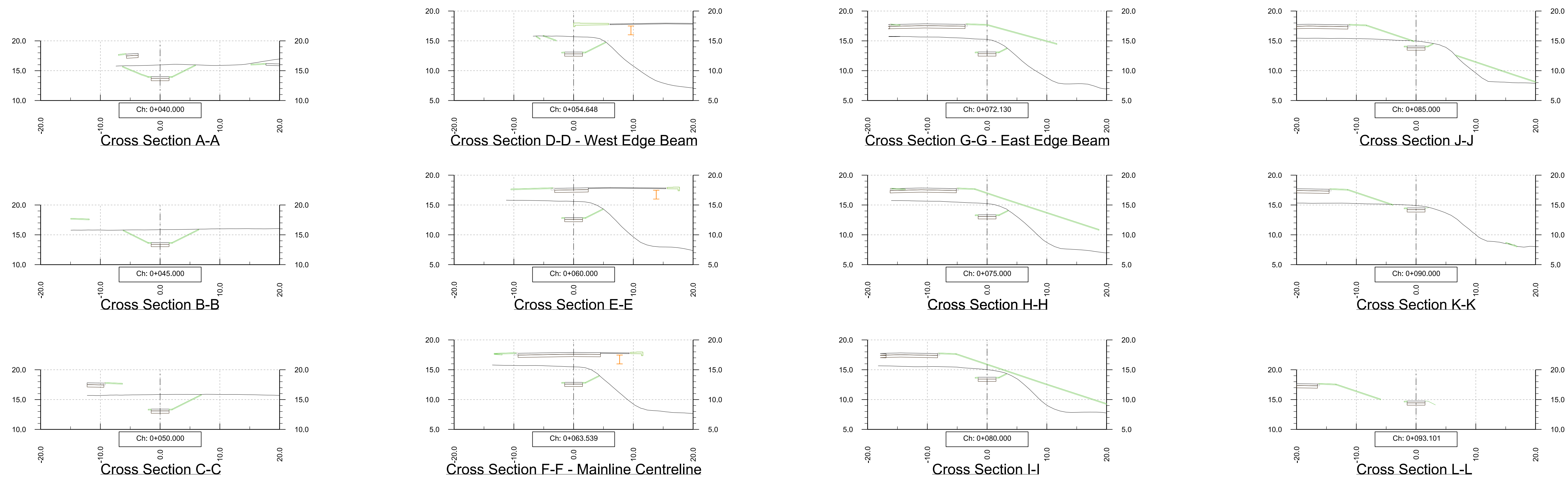
Existing topography

Appendix C

Cattle and NMU underpass - cross sections

A1





Notes:

Issue	Date	By	Chkd	Appd
P01.1	—	—	—	—

ARUP GRIFFITHS civil engineering and construction WHS

Blythe Gate Blythe Valley Park
Solihull B90 8AE
Tel +44 121 213 3000 Fax +44 121 213 3001
www.arup.com

Client
Llywodraeth Cymru
Welsh Government

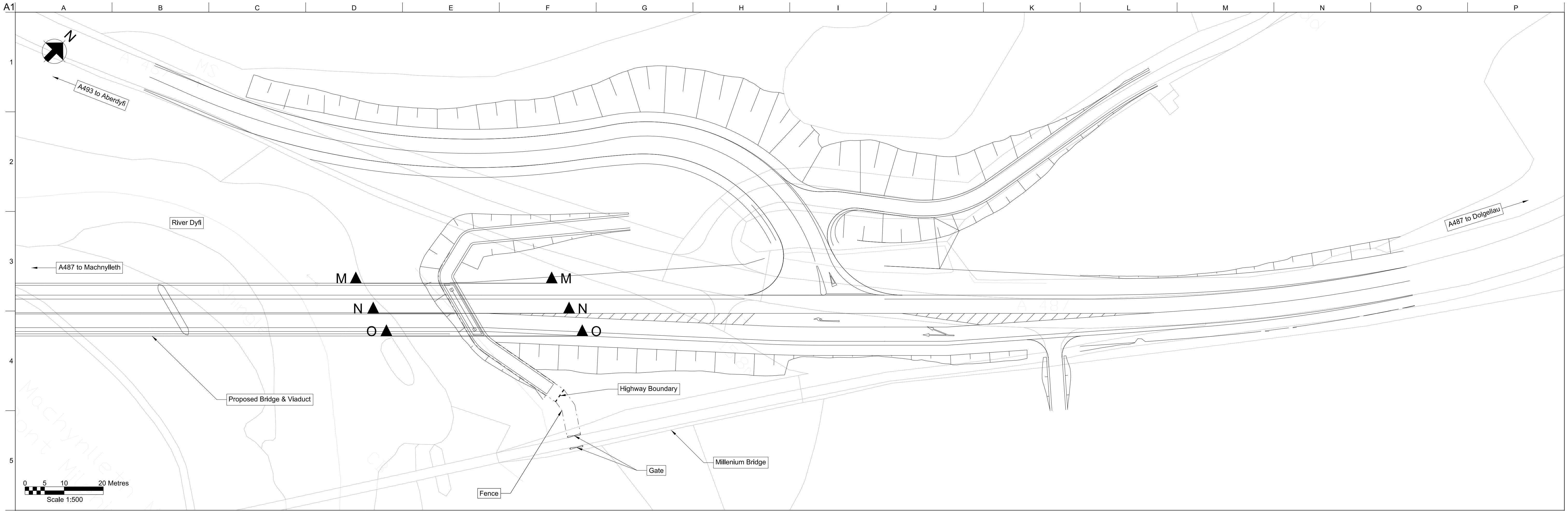
Project Title
A487 New Dyfi Bridge

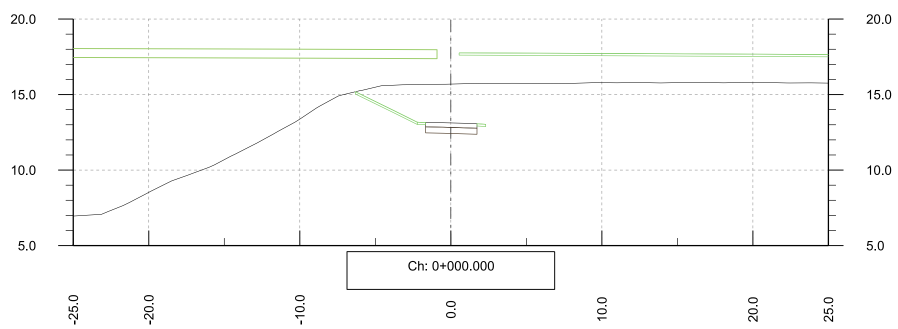
Drawing Title
Abutment Underpass
Cross Sections
Sheet 1 of 1

Scale at A1: 1:1000
Role: Civil - Highways
Suitability: S0 - Initial non-contractual
Arup Job No: 244562-00
Rev: P01.1
Name: 900237-ARP-AL-03-SK-CH-00002

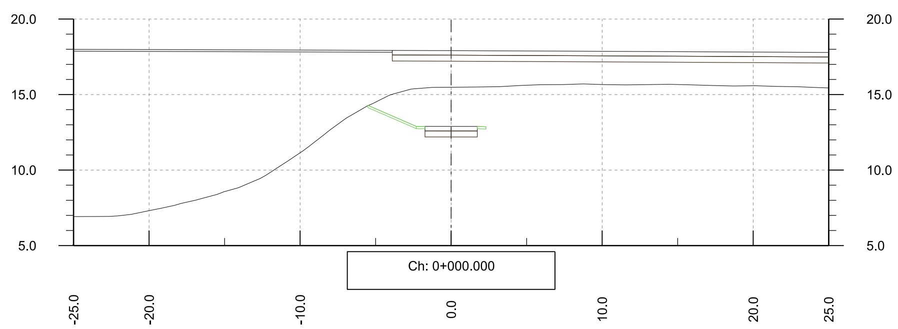
Do not scale

pw:\\MCCNTS04.global.arup.com:PW_ARUP_UKIDocuments\A487 New Dyfi Bridge\01 Arup\CH - Civil Engineer - Highways\SK - Sketch\900237-ARP-AL-03-SK-CH-00002

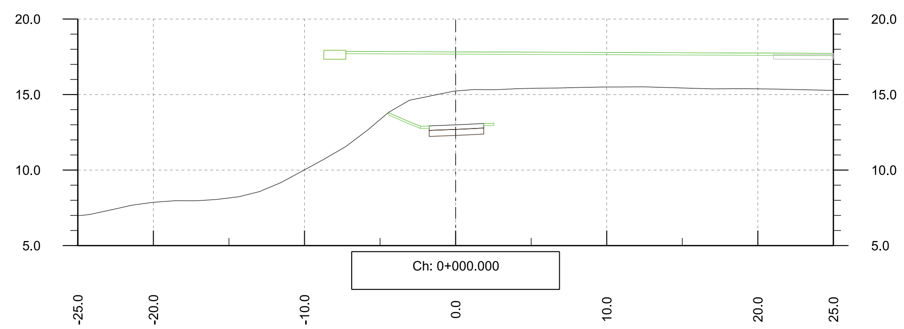




Cross Section M-M - West Edge Beam



Cross Section N-N - Mainline Centreline



Cross Section O-O - East Edge Beam

Notes:

Issue	Date	By	Chkd	Appd
P01.1	---	---	---	---

ARUP GRIFFITHS civil engineering and construction WHS

Blythe Gate Blythe Valley Park
Solihull B90 8AE
Tel +44 121 213 3000 Fax +44 121 213 3001
www.arup.com

Client
Llywodraeth Cymru
Welsh Government

Project Title
A487 New Dyfi Bridge

Drawing Title
Abutment Underpass
Cross Sections
Sheet 2 of 2

Scale at A1 1:1000
Role Civil - Highways
Suitability S0 - Initial non-contractual
Arup Job No 244562-00
Rev P01.1
Name 900237-ARP-AL-03-SK-CH-00003

Do not scale