WESTNEWTON BRIDGE MODELLING AND DESIGN

FINAL REPORT

Prepared for

Northumberland County Council

Prepared by

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flood risk | river restoration | diffuse pollution control | fisheries management | hydropower support

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GLOSSARY OF ACRONYMS

Acronym	Meaning
2D	2 Dimensional
AOD	Above Ordnance Datum
CAD	Computer Aided Design
cms	Cubic Metres per Second
Defra	Department of Environment, Food and Rural Affairs
DEM	Digital Elevation Model
EA	Environment Agency
FEH	Flood Estimation Handbook
GIS	Geographic Information System
hrs	Hours
LiDAR	Light Detection And Ranging
NCC	Northumberland County Council
NE	Natural England
OS	Ordnance Survey
Pa	Pascals
ReFH	Revitalised Flood Hydrograph
RTK-GPS	Real-time Kinematic Global Positioning System
SAC	Special Area of Conservation
SRH 2D	Sedimentation and River Hydraulics Two-Dimensional
SSSI	Site of Special Scientific Interest
TIN	Triangulated Irregular Network

1. INTRODUCTION

The College Burn at Westnewton Bridge, near Kirknewton, is a dynamic gravel bed system. Recent large flood events have resulted in channel migration and deposition of alluvial material in the vicinity of the bridge, significantly reducing conveyance capacity and potentially compromising the structural integrity of piers. Furthermore, general sediment deposition and accumulation of large wood material in the reach upstream has resulted in an increase in the elevation of the channel corridor, presenting a potential increased flood risk to Kirknewton. This report describes an assessment of the hydrodynamic and sediment transport processes on the College Burn and the subsequent development of measures to protect the bridge and reduce flood risk to Kirknewton.

1.1 BACKGROUND

The College Burn is a tributary of the River Glen, in turn a tributary of the River Till within the Tweed system (Figure 1.1). In the area of Westnewton Bridge, the burn is a dynamic gravel bed system with a high storage of sediment as alluvial bar features within the active river corridor. High sediment supply from upstream sources has been deposited in the lower gradient River Glen valley (the confluence with which is ~250 m downstream from Westnewton Bridge), creating a characteristic alluvial fan feature over the last 10,000 to 15,000 years.

Relatively recent human intervention has confined the burn into a narrowed active corridor through flood embankments constructed on either side of the channel. This has restricted sediment storage to a confined area which, over time, has resulted in this river corridor being raised relative to the adjacent floodplain. This situation now presents an increased flood risk to Kirknewton, situated under a kilometre to the east of the burn. Furthermore, dynamic channel process (mainly in relation to recent large flood events) had caused significant migration of the main channel in the location of the bridge, resulting in the westerly minor arch taking all of the flow under normal conditions, undermining of the bridge piers and some erosion of the easterly flood embankment.

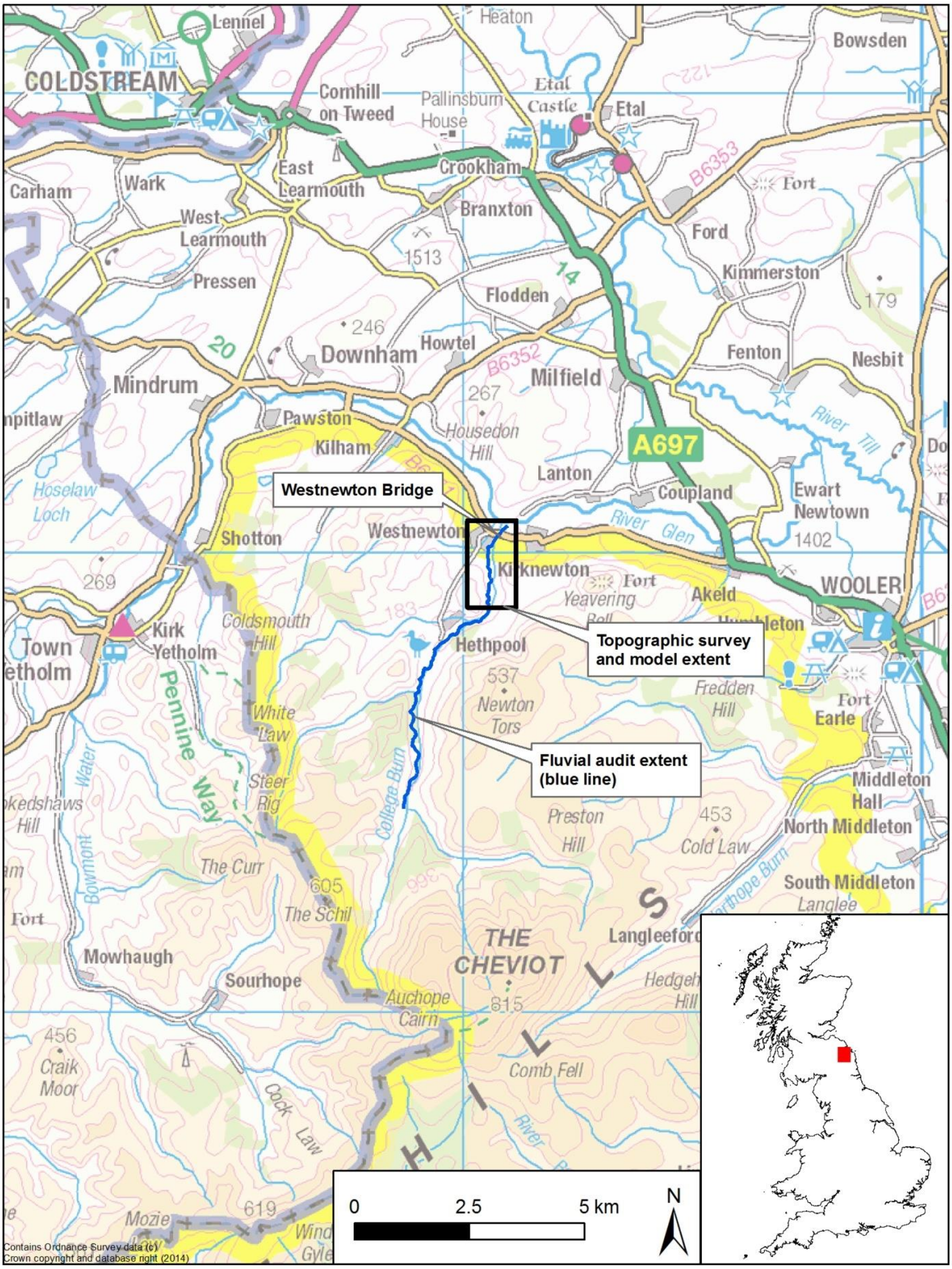
As a consequence of this, NCC undertook some emergency river engineering works to realign (straighten) the channel back through the centre major bridge arch and repair the damage to the bridge pier foundations. However, there are concerns that this alignment is unstable given the highly dynamic character of the river in this area. Indeed, already some evidence of lateral migration has been observed within the straightened section of the channel. Furthermore, given the protected status of the burn (both specific as an SSSI and as part of the broader River Tweed SAC), Natural England and the Environment Agency required that works to provide a longer-term and sustainable solution to the sedimentation issue were as unobtrusive as possible. The requirements were for the implementation of ‘soft’ as opposed to ‘hard’ engineering approaches and explicitly considering the issues of fish passage beyond the bridge. NCC asked cbec to model the hydrodynamics of the site to guide a design process that provided a more sustainable solution to the protection of the bridge and the management of flood risk to Kirknewton.

1.2 PROJECT APPROACH

Our approach to developing a sustainable management option was based on the philosophy of ‘process-based restoration’, as far as practically possible. The underlying concept is that tackling the impacts to the processes of water and sediment supply, transport and storage at the largest possible spatial scale will permit the river to recover naturally and in a stable, self-sustaining manner. In this way the river itself will subsequently do the work of maintaining a ‘natural’ and self-regulating environment with the minimal requirement for subsequent intrusive interventions. The approach therefore 'treats the cause rather than the symptom', providing a much more sustainable solution than traditional engineering approaches would. In the case of Westnewton Bridge, the practical application of this was to understand the drivers of dynamic geomorphic activity in the reach upstream of the bridge and to determine the processes that would optimally stabilise such a channel under natural conditions.

A number of methodologies were applied during the design process. These were used to determine the most appropriate general management approach and then provide the detail of the design. The initial stage was undertaken through a combination of historical assessment and field-based geomorphic assessment ('fluvial audit'), reported in Sections 2.1 and 2.2. The historical map-based assessment provided an understanding of 'reference conditions' for the site and how the legacy of human impacts has influenced the physical condition of the site. The 8 km fluvial audit (Figure 1.1) determined current geomorphic form/ process. The broad-scale understanding of the geomorphic characteristics of the College Burn provided context for the site-specific assessment.

Quantitative data collection and associated analyses within the area of the bridge (Figure 1.1) were used to provide further detail on existing conditions, to inform the design. These included sediment sampling of the project reach, hydrological assessment and topographic survey, with 2D hydrodynamic modelling of existing conditions (Sections 2.3 to 2.6). Initial designs were subsequently developed and refined through an iterative modelling – design adjustment process (Section 3).



Notes:		*Westnewton Bridge modelling & design* **Site Location**		
		Project No. U13-1003	Created By: CM	**Figure 1.1**

2. ASSESSMENT OF EXISTING CONDITIONS

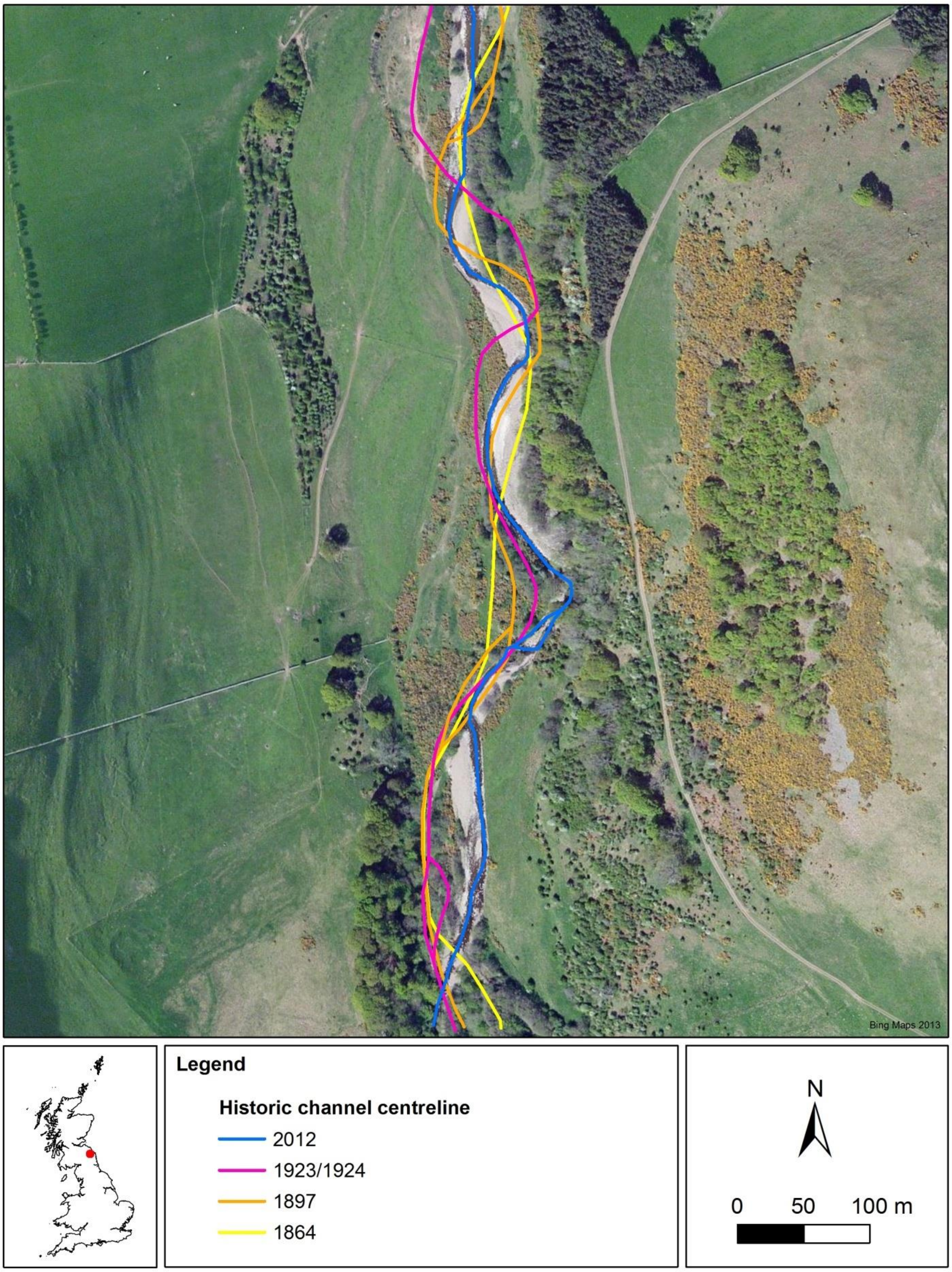
2.1 HISTORIC ASSESSMENT

Under the process restoration approach adopted here, an understanding of channel reference condition (i.e. the pre- or low-impact state of the river) provides an important basis for the subsequent development of designs that are physically appropriate to the system and its imposed conditions. Assessment of historic channel planform was used to determine the degree of dynamic behaviour of the channel in the reach upstream and downstream of Westnewton Bridge. Three OS maps for the area (published 1864, 1897 and 1923-1924) were geo-referenced and the river line for each date was digitised using GIS (Figure 2.1 and Figure 2.2).

Despite confinement by embankments, the channel centrelines indicate that the College Burn has experienced a significant degree of lateral migration over the previous 150 years. The channel migration has not been in a consistent direction over time at most locations; rather the maps suggest ongoing shifting of the channel centreline to various positions within a wider channel corridor. Such behaviour is consistent with wandering-type channels with high sediment supply and highlights the dynamic nature of the reach over a relatively long timescale. However, the confinement of the burn between embankments (i.e. prior to the earliest OS map) has meant that there have been no major changes in channel course of the channel through avulsion processes[1], as would be typical in a natural alluvial fan environment. Recent large-scale flood events occurred in 2008 and 2009, both of which had return periods of over 100 years. These caused significant channel change in many parts of the Till catchment, including the College Burn, contributing to some of the channel change observed in the historic assessment and responsible for the need for the recent emergency channel realignment works upstream of the bridge.

The maps indicate that a bridge has been present at Westnewton since prior to 1864 and that this has been a consistent constraint on the burn. The lowest degree of change in channel centreline location is in the 150 m of channel centred upon the bridge, suggesting a degree on ongoing channel management in this region. Downstream of the bridge, the historic channel lines indicate that the College Burn has moved progressively eastwards since 1864. Its confluence with the River Glen has moved downstream by approximately 100 m over this time period, at least some of this change having occurred during the 2008 and 2009 floods.

[1] Such an avulsion event breached the west embankment in an extreme event in the 1950s, although engineering works returned the active channel to the current river corridor shortly afterwards.



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Westnewton Bridge modelling & design

Historic Channel Centerline - Upstream

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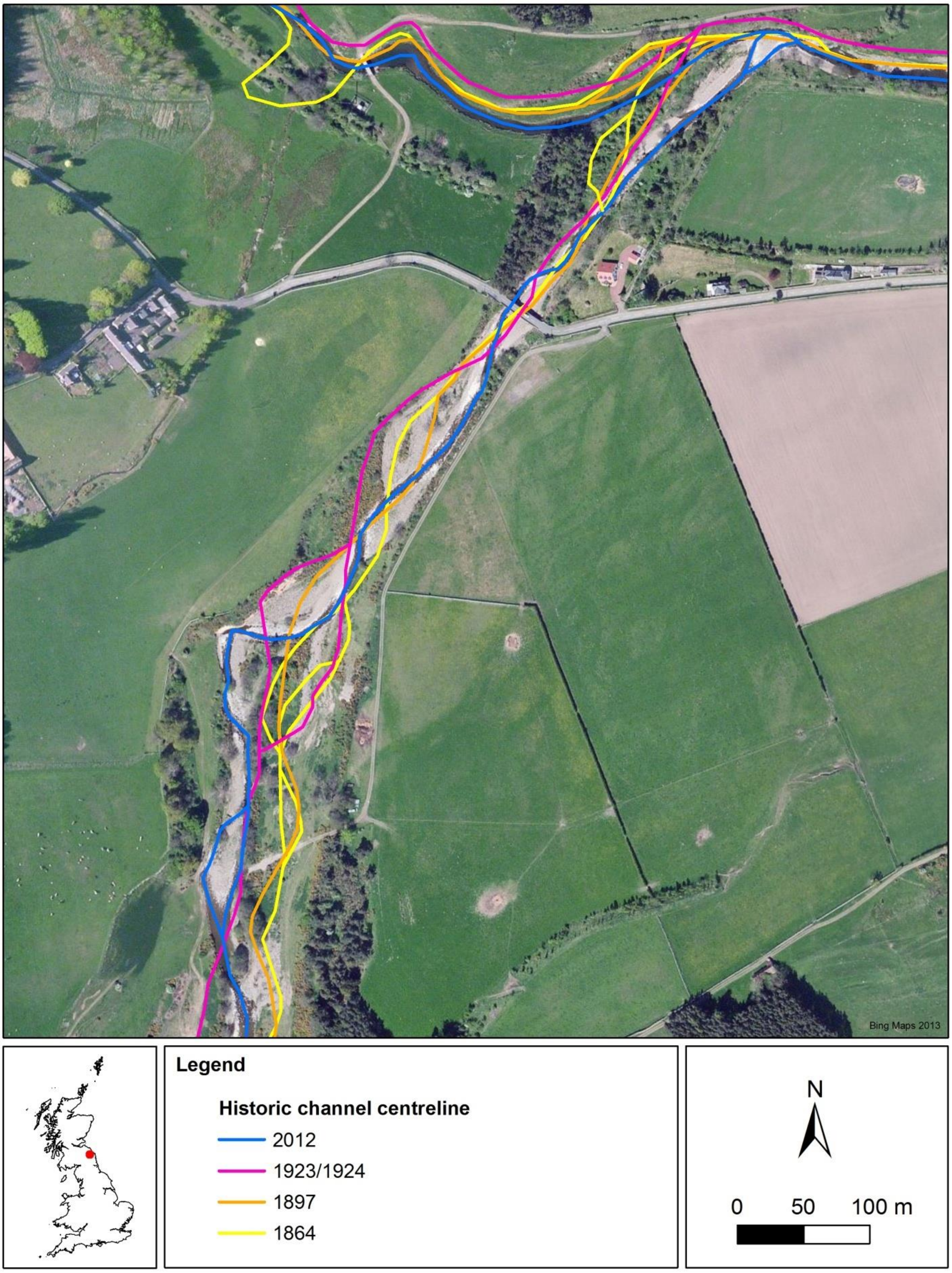

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Westnewton Bridge modelling & design

Historical Channel Centerline - Downstream

Project No. U13-1003 | Created By: CM | **Figure 2.2**

2.2 FLUVIAL AUDIT

A fluvial audit was undertaken to assess the current geomorphic character of the burn. The information generated was subsequently used to inform the selection of management options and to ensure that the general approach to design was appropriate to the contemporary physical process regime of the channel. The audit was carried out between Southernknowe (OS NGR NT 887 244) and the confluence with the River Glen (NT 909 306), a distance of approximately 8 km. The fluvial audit was undertaken in March 2013.

2.2.1. Approach

The survey method used was based on cbec's fluvial audit methodology, developed by Dr Hamish Moir. The general principle of the methodology is to characterise the geomorphic and sedimentary regimes of the river by classifying channel character in terms of observed morphology, the physical expression and integration of fluvial processes. The detailed methodology is provided in Appendix A.

Any feature that provided an indication of, or influence to, fluvial form/ process was recorded using a hand held GPS to mark its location and extent. The type of feature and any associated attributes were recorded. These were classified into a number of main categories:

- Reach type (i.e. channel morphology)
- Morphological units (e.g. pool, riffle, run, glide)
- Bed substrate material (classified in terms of dominant and sub-dominant sizes using the Wentworth terminology)
- Bank erosion (severity, bank height and material)
- Depositional features (i.e. alluvial barforms exposed at 'normal' flows, including type of feature, material, stability)
- Tributaries (significance, sediment supply)
- Instream and riparian engineered structures (type and severity/impact)
- Woody debris

The data were subsequently entered into a GIS (using ArcMap software) to allow visualisation and spatial analysis.

2.2.2. Findings

Maps showing the features recorded in the fluvial audit are provided in Appendix B and are discussed below.

The majority of the reach had a pool-riffle/ plane bed (i.e. pool-riffle dominant, plane bed sub dominant) reach type, with shorter sections of plane bed/ pool-riffle and pool-riffle (Figures B1 and B2). This is typical of wandering-type channels, where high rates of sediment supply, transport and storage give rise to the development of bedforms (e.g. riffles, alluvial bars). In the downstream-most 500 m of the survey reach, where recent dredging and straightening had taken place, the reach type became plane bed. There was a short cascade section where the channel was bedrock-controlled, at Hethpool Linn waterfall. Elsewhere, the channel substrate was dominated by gravel and cobble-sized material.

Sediment deposition was extensive throughout the surveyed reach (Figures B3 and B4), mostly in the form of alluvial cobble/ gravel bars (Figure 2.3). The majority of these were un-vegetated and appeared active. There were also a number of large-scale flood deposits, which occurred throughout the reach. It is likely that these are associated with the large floods that occurred in 2008 and 2009. Most of these deposits were semi-vegetated, suggesting that some stabilisation had occurred. Many of the larger alluvial deposits had associated secondary or high flow channels around the distal side of the feature. Towards the downstream extent of the reach, there were significant overbank gravel deposits resulting from recent flood events (Figure 2.4).

Bank erosion was similarly widespread within the surveyed reach (Figures B5 and B6). Most erosion was into alluvial floodplain deposits of cobble and gravel, often within a fine sediment (silt/sand) matrix, and eroding banks were typically between one and two metres in height (Figure 2.5). Erosion often occurred opposite depositional features, in response to their effect of forcing flow towards the outer bank. A number of locations of major erosion throughout the reach represented significant sediment sources and also indicated ongoing lateral channel adjustment. In several locations, erosion directly into higher terraces or hillslopes was taking place, exposing faces of up to 10 m in height and providing a further sediment source of gravel, cobble and fines (Figure 2.6).

Engineering directly impacting the active channel was limited throughout most of the reach (Figures B7 and B8). There were bridges at the upstream extent and in the centre of the reach, as well as Westnewton Bridge itself, towards the downstream extent. The two bridges further upstream both had associated ford structures, which effectively acted as weirs. There was some evidence of minor dredging activity and bank re-profiling in a limited area in the upper part of the surveyed reach. The majority of channel engineering was found in the downstream-most 500 m of the surveyed extent (Figure B8). There was an embankment on the right bank throughout this section. This had been recently rebuilt at its upstream end, where it was set-back from the channel. Further downstream, the embankment was immediately adjacent to the active channel corridor (Figure 2.7). The embankment was approximately 2 m high. The embankment also continued further upstream, but was old and indistinct here.

Bank protection was observed on both banks upstream of Westnewton Bridge. On the left bank this consisted of intermittent rip-rap behind an alluvial bar. On the right bank the protection was vertical wooden posts, which were protecting the embankment. Downstream of the bridge, rip-rap bank protection was found on the right bank for approximately 90 m (Figure 2.8). Further downstream, wooden bank protection extended along most of the right bank as far as the confluence with the River Glen.

Dredging and reworking of the channel had taken place around the bridge, leaving a straight channel within re-profiled gravel/cobble deposits (Figure 2.7). The extended for approximately 170 m upstream of the bridge and 60 m downstream. The re-aligned channel passed through the central arch of Westnewton Bridge, which is a three-arch stone bridge. There was evidence of past dredging activity, suggesting that such channel management has been required to maintain the conveyance of flows past the bridge.

The fluvial audit results highlight the dynamic nature of the system. The extensive depositional bar features and associated bank erosion indicate high rates of sediment transport and channel change, especially in response to large flood events. Development of a management strategy for Westnewton Bridge must therefore take into account the high rate of sediment supply to the reach, the high energy of the system and the potential for large-scale channel change.

Figure 2.3. Alluvial bar deposition with associated bank erosion.

Figure 2.4. Overbank gravel deposition towards the downstream extent of the study reach.

Figure 2.5. Bank erosion within the upper half of the survey reach.

Figure 2.6. Terrace erosion in the central part of the reach.

Figure 2.7. Straightened/ dredged channel upstream of Westnewton Bridge, showing embankment and bank protection on right.

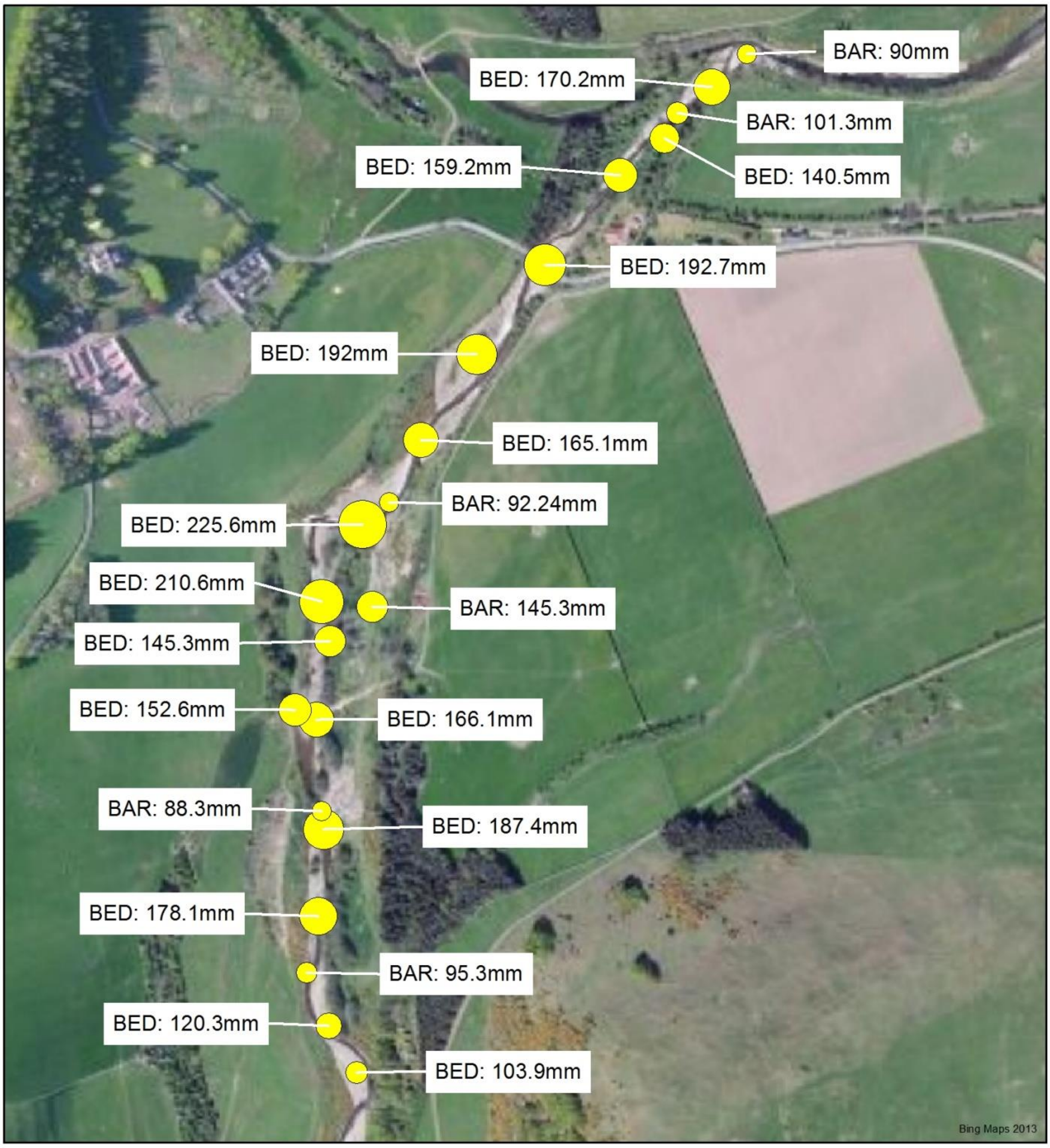
Figure 2.8. Looking upstream towards Westnewton Bridge showing straightened channel and bank protection on left of picture.

2.3 SEDIMENT CHARACTERISTICS

Sediment sampling of the project reach was conducted employing the 'Wolman-walk' pebble count methodology. A total of 21 samples were taken over a 1.1 km section of the channel extending from NT 906 296 to the confluence with the River Glen. Quantification of particle size was important to allow accurate representation of bed roughness within the subsequent hydrodynamic modelling.

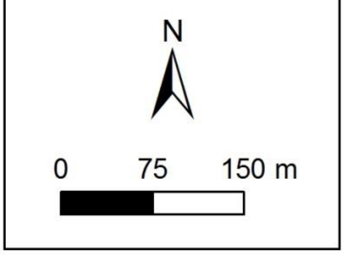
The locations of sediment samples are indicated on Figure 2.9. Samples were spaced throughout the reach and taken on a representative range of morphological units (i.e. bar surfaces and channel bed), indicated on Figure 2.9. For each sample the D_{84} (i.e. 84th percentile particle diameter) was calculated. This is indicated on Figure 2.9.

The D_{84} from all samples is within the cobble size bracket, indicating a relatively coarse bed and further highlighting the high-energy nature of the river here. There was no apparent systematic variation in particle size with distance downstream through the reach. However, in general, samples taken from the channel bed had a larger D_{84} than those taken on bars.



Legend

D84 (mm, 200 times exaggeration)



Notes:		*Westnewton Bridge modelling & design* **D_{84} from Sediment Samples**		
		Project No. U13-1003	Created By: CM	**Figure 2.9**

2.4 TOPOGRAPHIC SURVEY

cbec conducted a topographic survey of the 1.5 km study reach, extending upstream to OS NGR NT 905 294 and downstream to the confluence with the River Glen. The survey included the channel and associated features was conducted using a combination of RTK-GPS (Leica GS08 rover and base station) and Total Station (Trimble S6) instrumentation. All total station measurements taken from control points set with RTK-GPS. The survey points were typically captured by cross section including floodplain elevations of both banks where possible. The locations of cross sections were defined by the nature of the channel morphology and were surveyed to capture variation in topography of the vertical and horizontal planes. The morphological cues for cross section capture included hydraulic controls, grade control structures, breaks in channel slope, expansions or contractions in channel width, and flow divergence/ convergence points.

Where the channel exhibited uniform cross sectional characteristics cross sections were surveyed to capture channel sinuosity. Additional features were surveyed to capture channel heterogeneity including; terraces, braids, significant bar features, high flow channels through floodplain areas, and bridge piers. To facilitate interpretation of the survey data, and appropriate break line creation in CAD, all surveyed points were accompanied by an appropriate point code indicating the specific feature type.

A DEM was developed from the raw topographic data and additional LiDAR data (purchased from the Environment Agency) in Autodesk Civil 3D 2011 (CAD) to represent existing channel/ floodplain conditions for the site. This was used as the boundary surface for a 2D hydrodynamic model (using the SRH2D platform) of the site extents. To integrate the topographic survey with LiDAR data, points constituting the outer margins of the topographic survey were isolated and the relative elevations of the LiDAR were sampled in ArcGIS. The difference between the surveyed and LiDAR elevations were calculated and a colour-coded shapefile produced to provide a visual cue of where the topographic survey points and LiDAR data were in agreement and where significant difference was present.

Where little or no difference was observed, the points were used to form the boundary interface between the two datasets. Where the two datasets were not in agreement the boundary was offset using recent aerial images as cues to set the inner limits of the LiDAR data. The LiDAR was then clipped to the resulting inner boundary polygon to be combined with the existing ground surface created in AutoCAD Civil 3D from the topographic survey data.

2.5 HYDROLOGY

Flood discharges were calculated for the College Burn at 390850E, 630500N.

Table 2.1 Q_{2yr} and Q_{200yr} Estimated Discharges

Q_{2yr} Estimated peak flow (cms)	Q_{200yr} Estimated peak flow (cms)
26.3	81.7

The discharges were computed using catchment descriptors obtained from the FEH CD-ROM v3 software and generated using the Revitalized Rainfall Runoff method (ReFH). The recommended time step and duration were used as well as the recommended model parameters obtained from the catchment descriptors. The Q_{200yr} hydrograph, used as the test hydrograph for modelling, is shown in Figure 2.10**Error! Reference source not found.**.

2.6 MODELLING (EXISTING CONDITIONS)

Given that the recent 2008 and 2009 flood events at Westnewton were greater than the 100 year return event, all modelling work for design was undertaken with an estimated 200 year return period flood (Q_{200yr}).

The rising limb of a hydrograph can cause significantly greater stresses than are experienced using a steady state/ constant flow condition, so unsteady modelling of the hydraulics was also employed throughout. A state of the art unsteady 2D hydraulic solver SRH-2D (Lai, 2009; Pasternack, 2011) was used to compute the Q_{200yr} flow in the channel and onto the floodplains. This code was chosen for the College Burn because of its ability to accurately and efficiently model complex stream-wise and cross stream flow in the channel, flow over terrain modelled by spatially varying bed friction, complex patterns of shear stress, regions of subcritical, transcritical and supercritical flow, and flow over embankments onto floodplains. SRH-2D meets all of the requirements of the recent EA/ Defra review of 2D hydraulic modelling packages for use in the UK for flood risk studies (Neelz and Pender, 2009; Lai, 2009). SRH-2D is extensively validated (Lai, 2008) and has been used by cbec UK on many UK river restoration and design studies.

The modelling process consists of discretising the channel and floodplain with a computational mesh, where quadrilateral elements of small size are used in the main channel and largely aligned with the stream-wise direction, and larger, typically triangular or trapezoidal elements are used to discretise the floodplains. SRH-2D allows the use of elements of varying size within the channel- more detail (i.e. finer mesh resolution) was used in the main regions of interest at the run-up to the bridge and over the apron for example. The surface elevations of the mesh are interpolated from the CAD DEM and surface frictions are chosen to represent the finer scales and materials present on the bed. Bed and bar D84 particle sizes and photographs were used to determine the surface frictions and a map of friction polygons for the mesh. The surface friction distribution is shown in Figure 2.11 and tabulated in Table 2.2. The surface fiction at the exit to the domain was increased to ensure realistic subcritical flow at the exit of the model.

For Westnewton Bridge, the mesh process was an iterative one; first a course, medium and fine mesh was iterated to provide a model of existing conditions, then iterative design work was undertaken, then a common mesh structure for existing and design conditions was employed to compare flows under existing and design conditions. The differences between the existing conditions and design mesh were the surface elevations and bed frictions only. An overview of the final modelling mesh is shown in Figure 2.12. Mesh element sizes reflect the expected flow conditions: a small size of 1.5 m x 1 m is typical in the channel (around 15-20 elements from top of bank to top of bank for detail), 2 m x 2 m on the near bank floodplains, rising to 10 m x 10 m at the edges of the floodplain far from the channel. Extra detail was provided in the proposed design area at the run up to the bridge (0.5 m x 0.5 m) and on the bridge apron itself (1 m x 0.5 m).

On the apron, mesh elements were aligned parallel to the bridge piers. The bridge soffit or springing levels are not modelled with a 2D model. After initial modelling for existing conditions was undertaken, results were reviewed by the client, and it was decided to modify the mesh to represent naturally occurring log jams that were observed to significantly affect the flow during recent floods.

These jams were created in the model by raising the mesh bed level to create an obstruction and had the effect of recreating observed flooding patterns more accurately than the bare mesh (Brewis, pers. comm.). These log jams were used for all future model runs as they represent the worst case for inundation. Their positions are indicated in Figure 2.13.

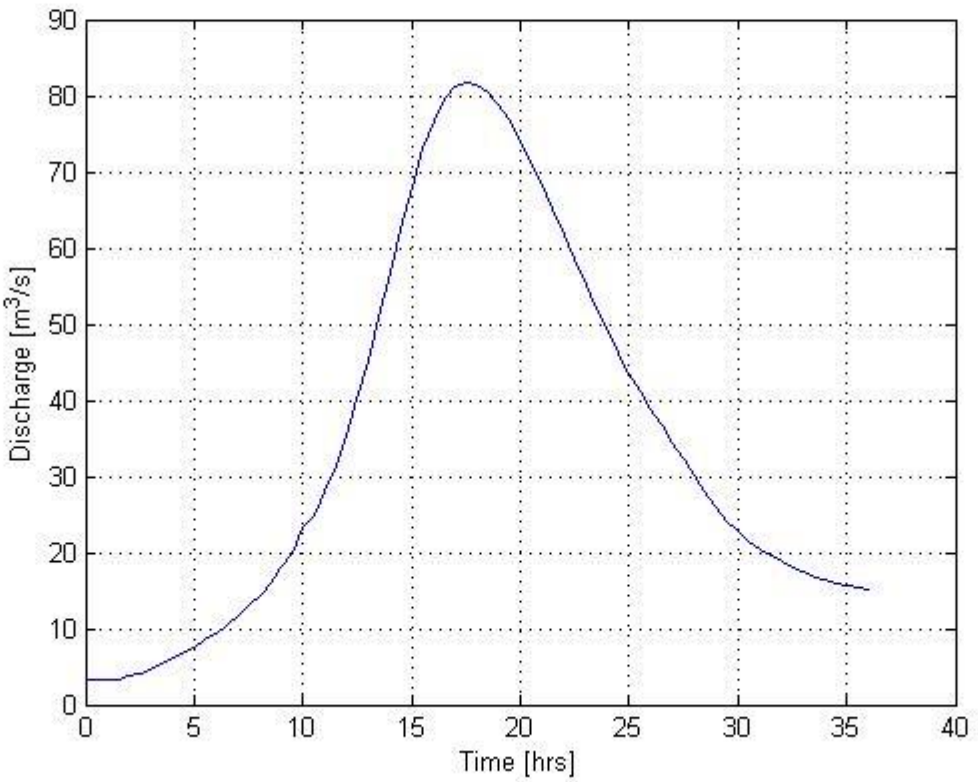


Figure 2.10. Inlet hydrograph established using the ReFH methodology for the College Burn at Westnewton Bridge. Peak is 81.7cms; Q2yr median discharge is approximately 23cms.

Table 2.2. Friction Values

Material	Number	n
Bed	1	0.039
Bed	2-5	0.035
Bed	6	0.039
Floodplain	7	0.06
Floodplain (rough)	8	0.07
Gorse	9	0.039
Gorse (thick)	10	0.044
Apron (existing bed)	11	0.035
Apron (concrete bed)	11	0.015
Apron (roughened bed)	11	0.035
Tree/log	12	0.045
Tree/log	13	0.045

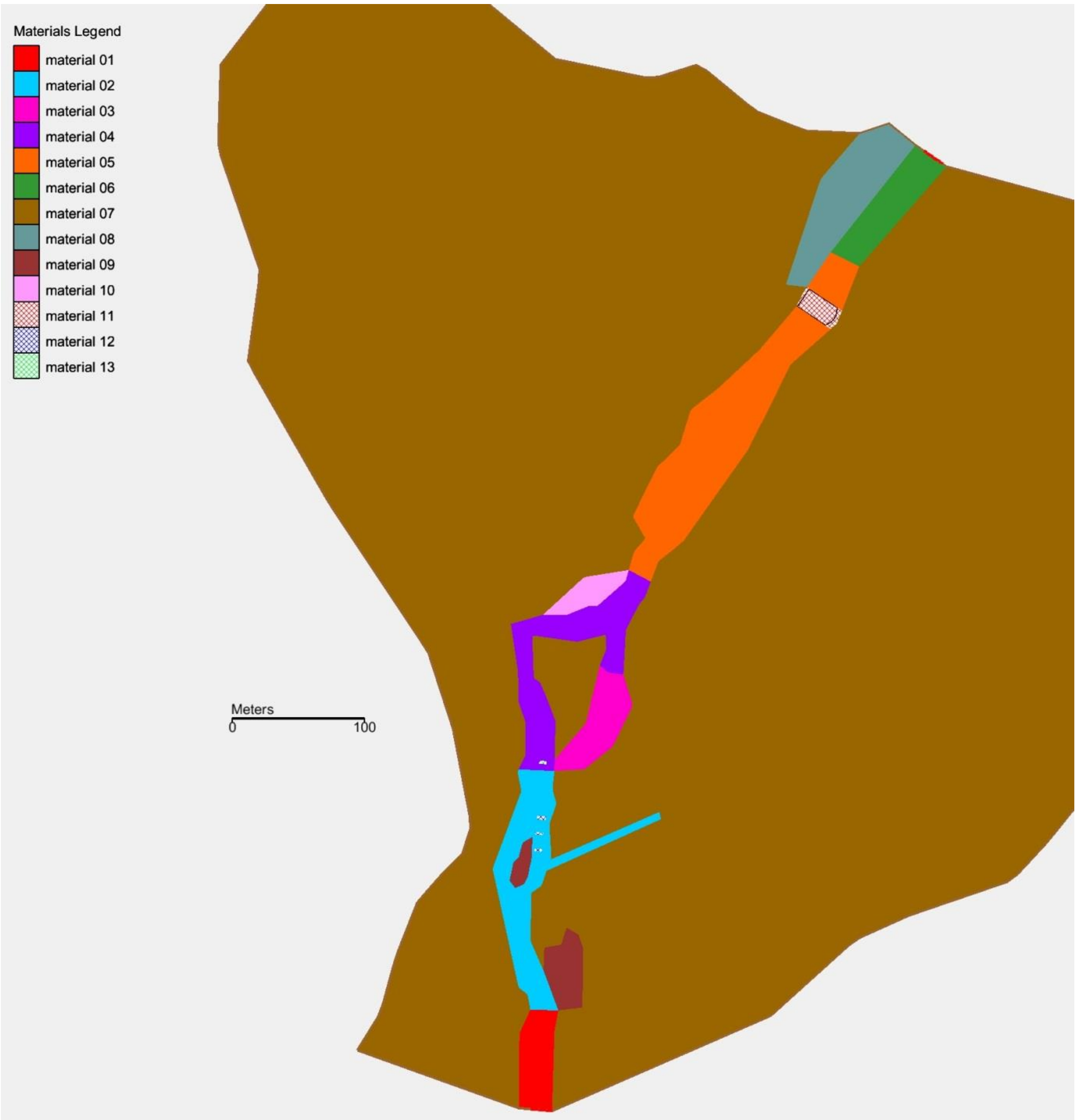


Figure 2.11. Surface material types for model (Manning n).

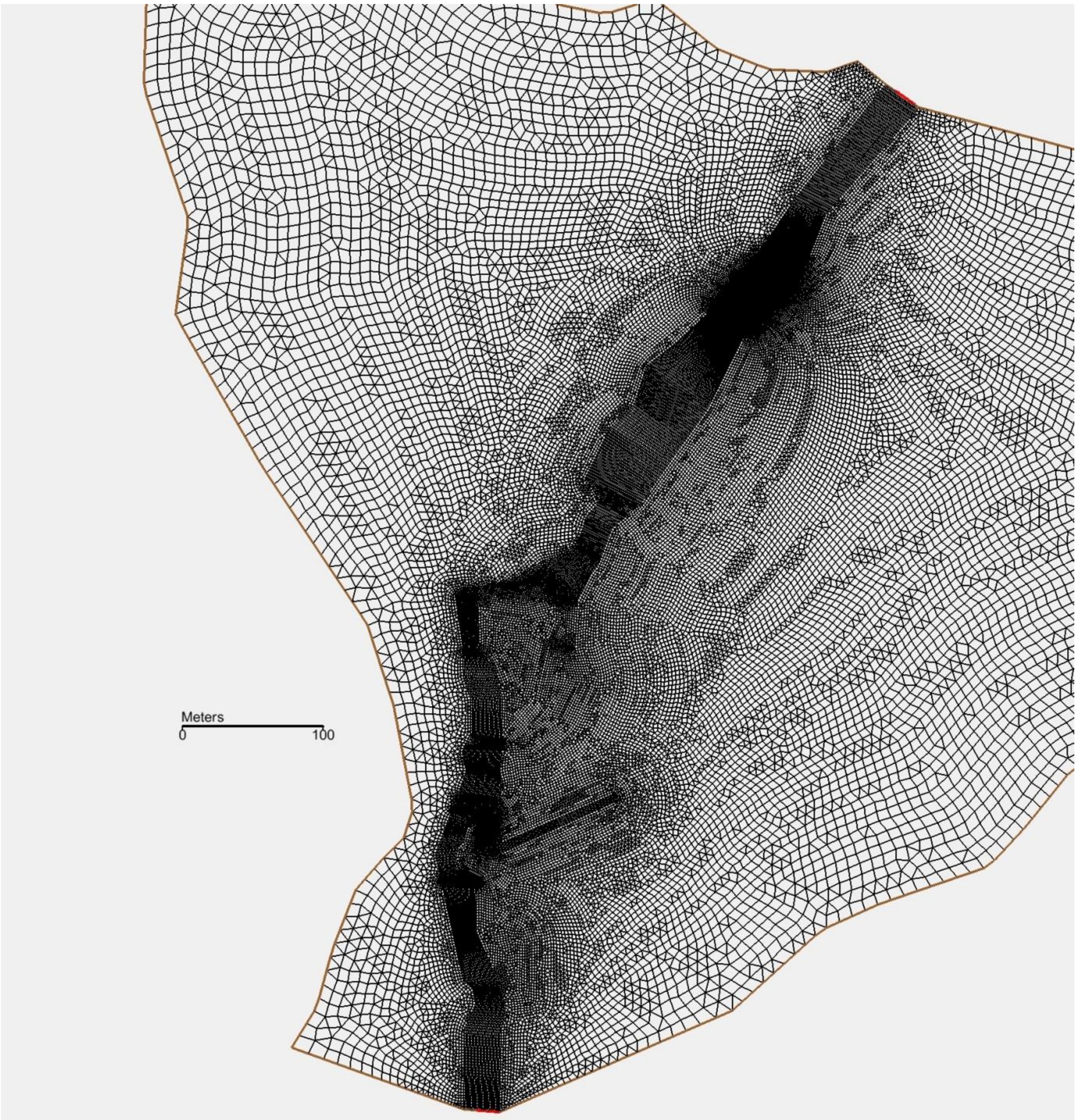


Figure 2.12. 2D hybrid mesh for model showing finer quadrilateral elements in channel and triangular and quadrilateral elements on floodplain.

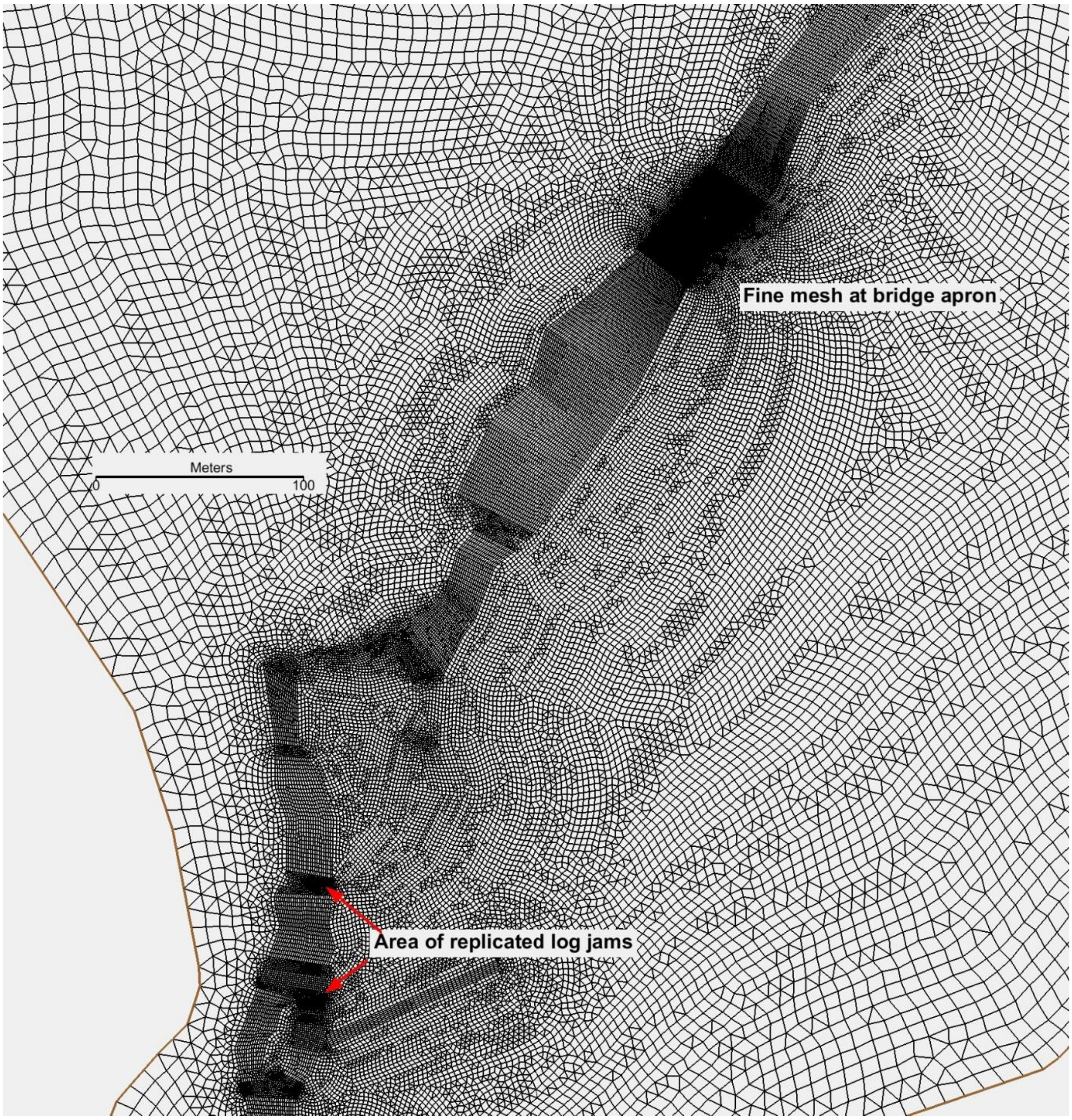


Figure 2.13. Detail of mesh showing position of typical natural log jams.

At the downstream end of the survey, the College Burn confluences with the River Glen. No data on water surface elevation at this point was available for the study and so the College Burn model was truncated to end upstream of the confluence at a point of approximately average slope and a normal depth assumption used to calculate water surface elevation. The slope at the exit was approximately 0.011. To obtain subcritical flow, the friction used at the exit was set as n=0.039. The normal depth assumption was significantly far from the design area not to affect levels at the bridge, and this was confirmed by varying the exit water surface level.

2.6.1. Existing Conditions Model Results

During the Q_{200yr} hydrograph, the Q_{2yr} peak value of 26 cms is reached at approximately 10.5 hrs. At this point, the discharge in the channel is close to the median discharge, and this level is often taken to be close to 'bank full' for UK rivers. Figure 2.14 shows the water depth predicted for this point of the hydrograph. There are three main areas of out of bank or incipient out of bank flow. First, water is seen to run up the track from the right bank approximately a quarter of the way from the bottom of the figure and onto the gravel area further downstream (as observed anecdotally). Secondly, there is flow on the right bank up to the embankment on the straightened section upstream of the bridge. Thirdly, there is flow onto the banks just upstream of the railway abutment in the upper part of the figure, and this abutment is seen to cause a significant restriction to the flow.

Figure 2.14. Water depth for existing conditions computed at a discharge of 26cms (~Q2yr peak) during the Q200 year hydrograph

Figure 2.15 shows the bed shear stress computed at the Q_{2yr} discharge, 10.5 hrs into the Q_{200yr} test hydrograph. Bed shear peak is 147 Pa at the sharp right bend central to the figure, but there are high shears of 110 Pa immediately upstream of the bridge and 120 Pa immediately downstream. There is a further peak of 100 Pa at the downstream railway abutment.

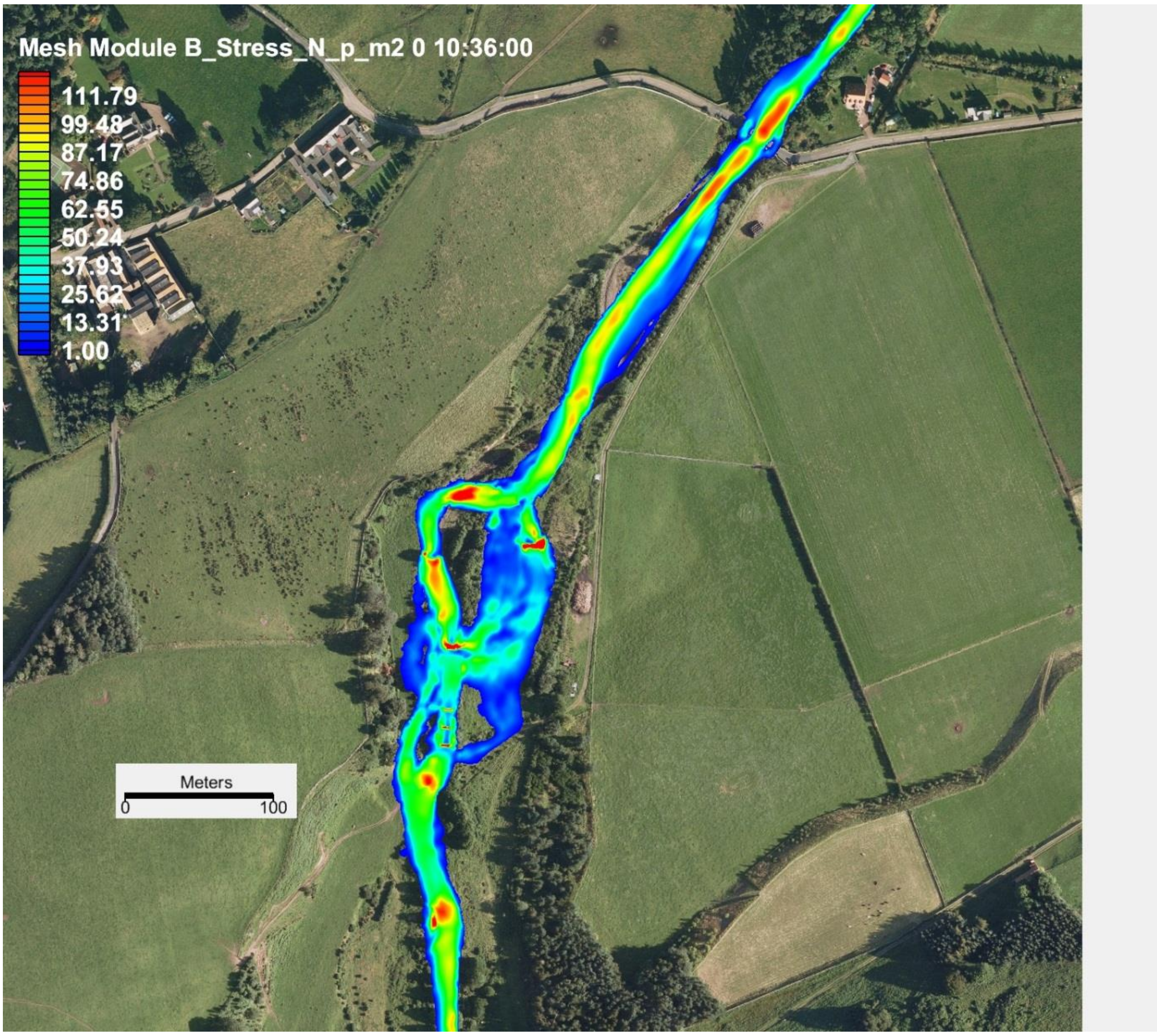


Figure 2.15. Bed shear stress for existing conditions computed at a discharge of 26 cms (~Q_{2yr} peak) during the Q_{200yr} hydrograph.

Maximum inundation predicted for existing conditions during the Q_{200yr} event is shown in Figure 2.16. At the upstream end of the model (bottom of the figure) there is a flood route towards Kirknewton onto the right hand floodplain. At the downstream end (top end of the figure) there is significant out of bank flow running from the old railway abutment on the left bank along the embankment to the left. Moreover, there is significant potential for the straightened section upstream of the bridge to re-meander and bypass the bridge. These three areas are the foci for design. There are some significant areas of high shear stress predicted at the Q_{200yr} peak flow. These are shown in Figure 2.17; (1) at the upstream end of the straightened section of channel there is a local peak of 196 Pa, (2) immediately upstream of the bridge there is a local peak of 177 Pa, (3) immediately downstream there is a local peak of 181 Pa, (4) immediately downstream of the railway abutment there is a peak of 200 Pa.

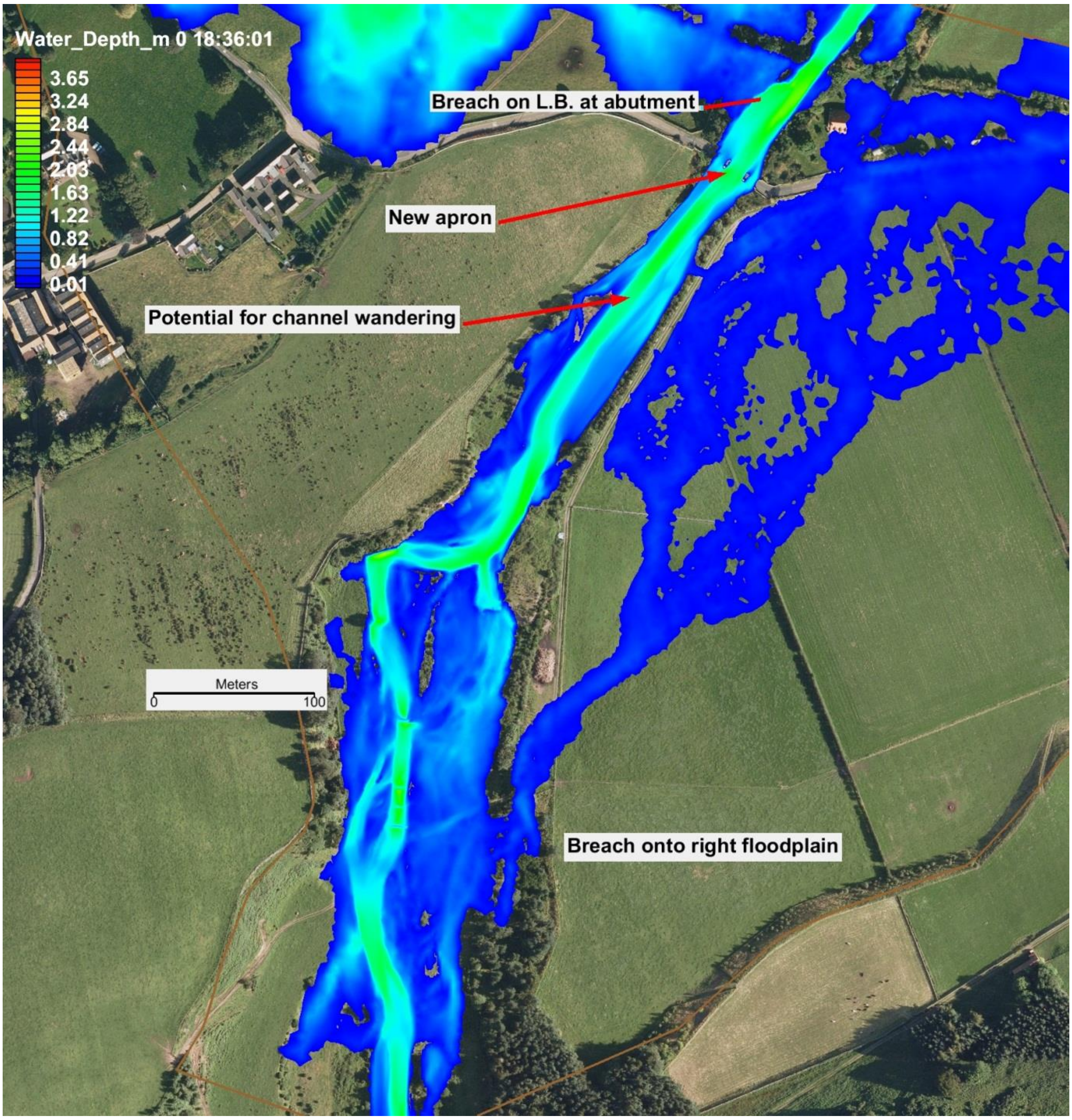


Figure 2.16. Maximum inundation during a Q_{200yr} flood hydrograph, showing potential redesign areas.

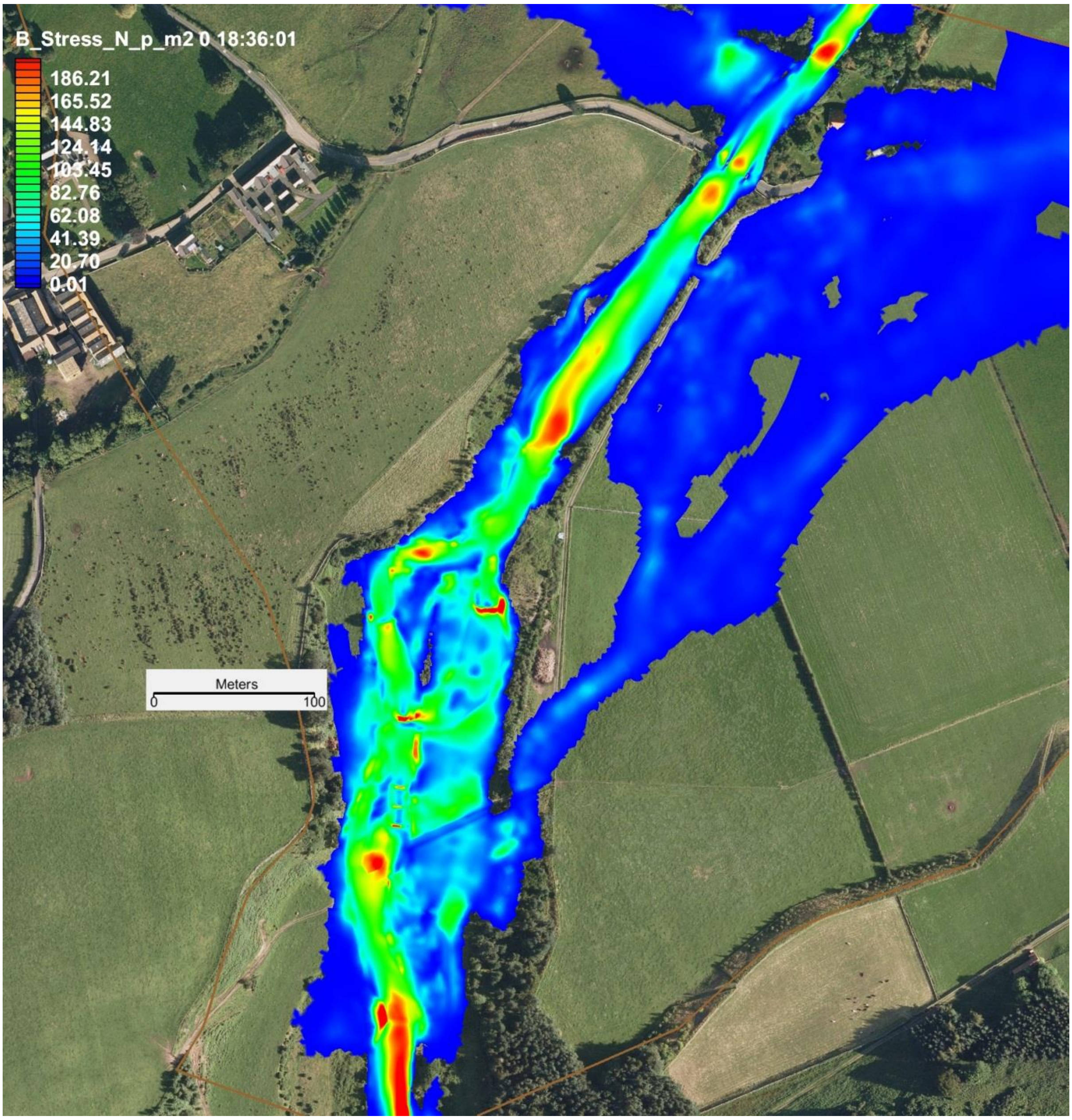


Figure 2.17. Bed shear stress at Q_{200yr} peak for existing conditions.

3. DESIGN PROCESS

The requirements of the design were to:

1. 'Train' the channel upstream of the bridge so that it approached the central arch normal to the orientation of the structure.
2. As much as was practicable, utilise a 'soft' engineering approach that considered natural fluvial processes (relating to the protected status of the burn).
3. Ensure unhindered fish passage beyond the bridge under normal flow conditions.
4. Introduce a new embankment to prevent flow onto the right hand floodplain.
5. Modify the abutments of the dismantled railway bridge ~100 m north (downstream) of Westnewton Bridge.

The primary component of the design is the training of the channel upstream of Westnewton Bridge. Based on the geomorphic character of the site and the constraints related to the protected status of the site/ environmental legislation, it was decided that the use of large wood structures was the most appropriate design strategy. In natural settings, large wood structures can provide a significant stabilising effect on local channel course, training flow towards the channel centre and protecting banks composed of otherwise highly erodible material. For the Westnewton design, it was proposed that a series of large logs (with root balls intact) were to be introduced to the channel margins. These were to be arranged in left-right bank pairs with the root ball end at the channel margin and the trunks buried into the banks pointing away from the channel at ~45 degrees to the direction of flow and ~3% slope angle from root ball to bank intersect.

The specific configuration (longitudinal and lateral spacing between logs, trunk orientation) of the structures was optimised through an iterative modelling process. Initially a single log pair (i.e. individual logs situated opposite each other on left and right banks of the channel) was introduced to the design and the design Q_{200yr} hydrograph modelled to determine the hydraulic (and inferred sediment transport) influence of the structure. Based on analysis of this output, a series of appropriately spaced and orientated log pairs were introduced to the design and the same flow magnitude was modelled. The effect on hydraulics of the interactions between the series of log pairs was then assessed and their configuration further refined to provide training at both medium to high flows.

A spreadsheet was provided that computed drag and buoyancy loads on the partially buried logs at the peak Q_{200yr} flow. These loads depend implicitly on the sourced wood for the log structures (lengths, widths and protruding height), but exemplar values are given in this report (Appendix C).

In terms of unhindered fish passage (predominantly considering sea trout and Atlantic salmon), the design of the new bridge apron required to provide appropriate/ suitable hydraulic characteristics under the typical range of flow (discharge) conditions under which upstream migration would occur. This involved the roughening of the concrete apron structure with cobble and boulder sized material to provide variation of flow depths and velocities that could be exploited by fish. Specifically, the design involves a series of paired boulders that alternate on either side of the apron centre-line and induce a sinuous flow pattern. This reduces mean flow velocity through the lower elevation centre of the apron with hydraulic resting areas in the lee of boulders. The entire extents of the apron (i.e. including the central section with the paired boulders) are to have 100 – 200 mm river cobbles inset into the concrete at approximately 50% areal coverage in order to reproduce the bed roughness characteristics of the channel immediately upstream of the apron. This aspect of the design was to prevent a step change in shear stress at the edges (upstream and downstream) of the apron, and prevent accelerated flow that would occur over smooth concrete. This limits the potential for differential sediment transport patterns (i.e. erosion or deposition) in the vicinity of the apron that could reduce its functionality (both in terms of conveyance of flow and passage by fish) over time.

Given the very dynamic nature of the burn (as identified during the historical and geomorphic assessments stages of the project), it is unlikely that any design will remain stable in the long term and there will be the requirement for periodic repair/ adjustment of the design, particularly in response of large flood events. An outline strategy for a post project monitoring plan and considerations for the 'adaptive management' of the design are provided in Section 4. However, the design approach proposed aims to provide the most sustainable and stable option given the highly dynamic geomorphic condition of the site and the restrictions imposed by the protected status of the site.

In addition to the training of flow through the bridge piers, additional design was done to raise an embankment upstream where existing conditions modelling indicated a significant flood route. The required level of this embankment (72.2 m AOD) was determined by computing water levels in the existing conditions model and raising the local level at the breach to the flood level (71.8 m AOD) plus 0.5 m freeboard.

The final design change consisted of remodelling the railway abutment on the left bank downstream of the bridge. Here, an approximately 2-m-wide shelf[2] was created by levelling a portion of the abutment. This increases conveyance and removes a large part of the restriction to flow caused by the abutment.

The results of the design model, incorporating the training log pairs, a roughened apron, extra embankment and modified abutment are shown in the next section.

3.1 PRODUCTION OF DESIGN TIN

The Triangulated Irregular Network (TIN) surfaces of the design components were created using Autodesk Civil 3D 2011 (Civil 3D) for the existing ground conditions and four separate design areas. The design areas are:

1. The training logs
2. The bridge apron
3. The embankment track access
4. The rail bridge abutment setback

A TIN surface was created from the field surveyed topographic survey points and connecting break lines added ensure correct interpretation by Civil 3D's triangulation algorithms. The resulting surface was then combined with LiDAR data to produce surface that accurately represents the existing ground conditions. For the Training Logs a feature line CAD entity was created in Civil 3D representing a typical log with a 3% gradient rising from the root ball end. Sub-surfaces were then developed of mirrored pairs of logs embedded into the existing ground at design locations, elevations, and rotated to the design angles of inflection relative to the flow. These sub surfaces were exported for entering in to the mesh for 2D modelling. Further iterative design steps followed until the final arrangement was determined. For the Rail Bridge Abutment Setback a design 'top of bank' was defined by creating a

[2] The setback shelf was not consistently 2 m but is 'field-fitted' to link the upstream 'normal' bank elevation to an appropriate equivalent downstream bank elevation.

line connecting the exiting 'top of bank' upstream of the Rail Bridge to a representative location downstream of the structure. The 'bank toe' of these two locations where also connected to form the design bank face. The line of the existing 'bank toe' was extracted and joined to the design 'bank toe' to form a polygon prescribing the additional bed area resulting from the design. The design 'top of bank' was offset by 2 m to form a bankfull bench and then the 'grade to surface' tool used to grade this bench a 3:1 to the existing ground. The resulting DEM was exported to the modelling mesh and 2D modelling showed no further adjustments where required. For the Bridge Apron design only the materials and central fish passage assisting rock alignments differed from original drawings supplied by Northumberland County Council. Consequently this required only scaled graphical representation of these changes. The required design elevation at the Embankment Track Access was determined independently within the 2D modelling process. An additional 'freeboard' of 0.5 m was added (resulting in an elevation of 82.3 m AOD) and a representative embankment at that elevation was designed in Civil 3D.

3.2 DESIGN MODEL RESULTS

The design (as shown by the accompanying drawings, Appendix D) consists of a raised embankment on the right bank, upstream of the track at 390622E, 629978N; a series of log pairs to maintain stable 'trained' flow leading to the bridge apron; a bench area to increase conveyance at the railway abutment at 390804E, 630441N. In addition to the detail contained in the model, the bridge apron is to be roughened by 50% coverage 100mm-200mm diameter inset cobble and larger alternate boulder pairs near the centre. In the design model, the concrete apron was given a roughness similar to the adjacent channel (n=0.035).

The design modelling demonstrates that the design is successful in reducing inundation (especially for the right bank flood route to Kirknewton), training the flow to be stable through the bridge during the Q_{200yr} flood, and removing the restriction to flow at the railway abutment. The design shear stresses are similar (on average) to existing conditions but the design produces some localised peaks at the final log pair upstream of the apron[3] (from the root balls of the final log pair to immediately upstream of the apron lip, ~15 m longitudinally) and a higher shear stress at the downstream apron lip[4] (mainly located on the apron itself but also ~3 m downstream of the lip). Armouring the bed with a grade of 200-400 mm boulders for 15 m from the root balls of the final log pair to the upstream lip of the apron and of a thickness equivalent to one maximum particle size diameter (i.e. 0.4 m) is advised. Also, 3 m of the channel bed from the downstream apron lip should be armoured with a grade of 200 – 400 mm diameter particles, of one maximum particle size diameter in thickness (i.e. 0.4 m). Each aspect of the design model results is discussed in detail below.

3.2.1 Inundation extents at Q_{200yr} peak

The inundation onto the right hand floodplain towards Kirknewton is prevented by the upstream embankment, as shown in Figure 3.1. The downstream inundated area, running along the old railway embankment, is also reduced by increasing conveyance with the design shelf at the railway abutment. The figure also shows that the training logs are fully inundated at the Q_{200yr} peak. Figure 3.2 shows a comparison of total modelled inundation extent for the existing conditions and design model. Note

[3] This peak in shear stress indicates incipient mobility for 200-240 mm diameter particles.
[4] This peak in shear stress indicates mobility of particles under 200 mm diameter.

however, that the side edges of the model, except at the exit channel, are modelled as walls; in reality, flow would continue out of the downstream edges. Inundation is clearly seen to be reduced for design.

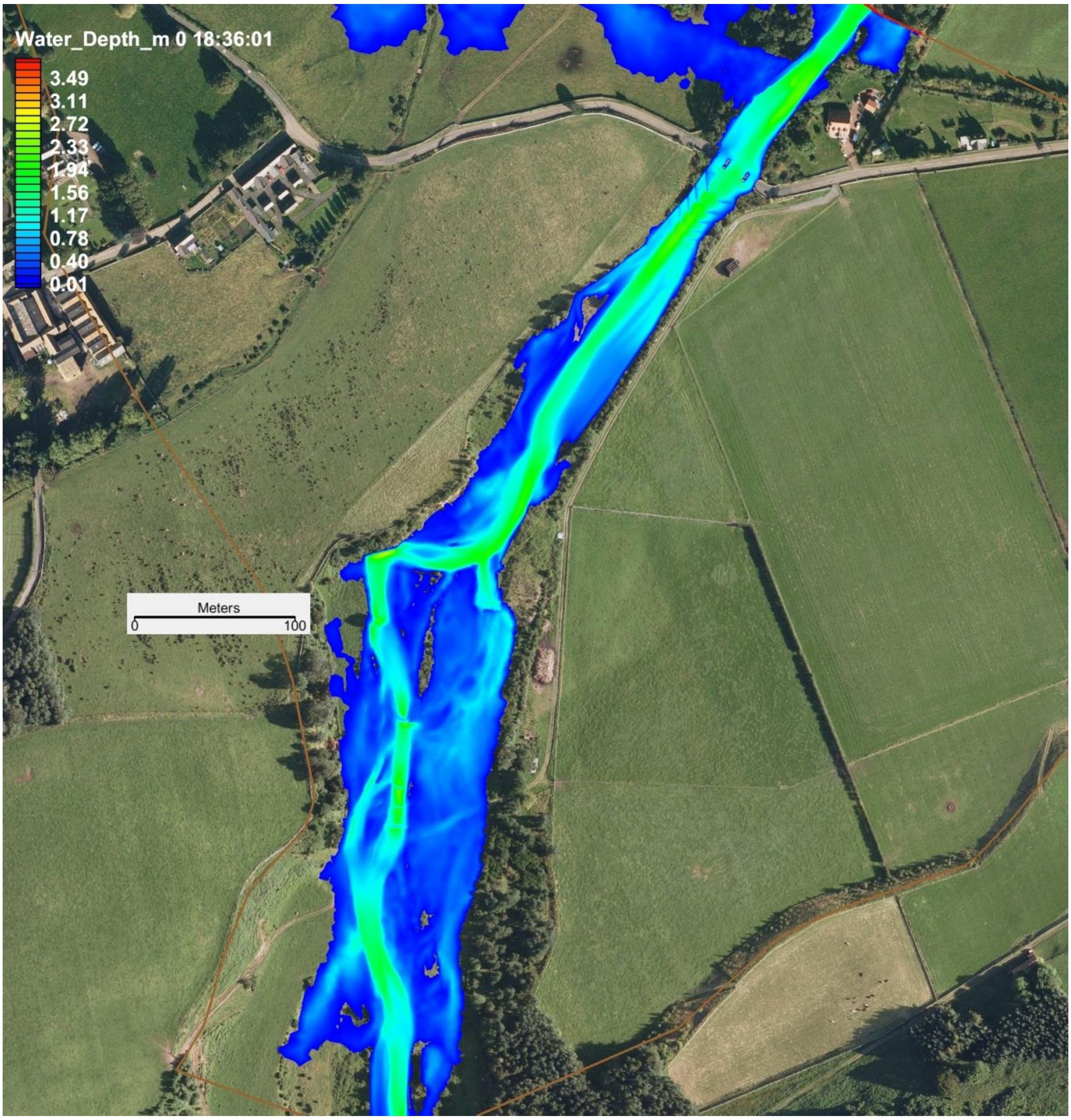


Figure 3.1. Maximum inundation predicted for combined design changes at Westnewton Bridge (Q_{200yr}).

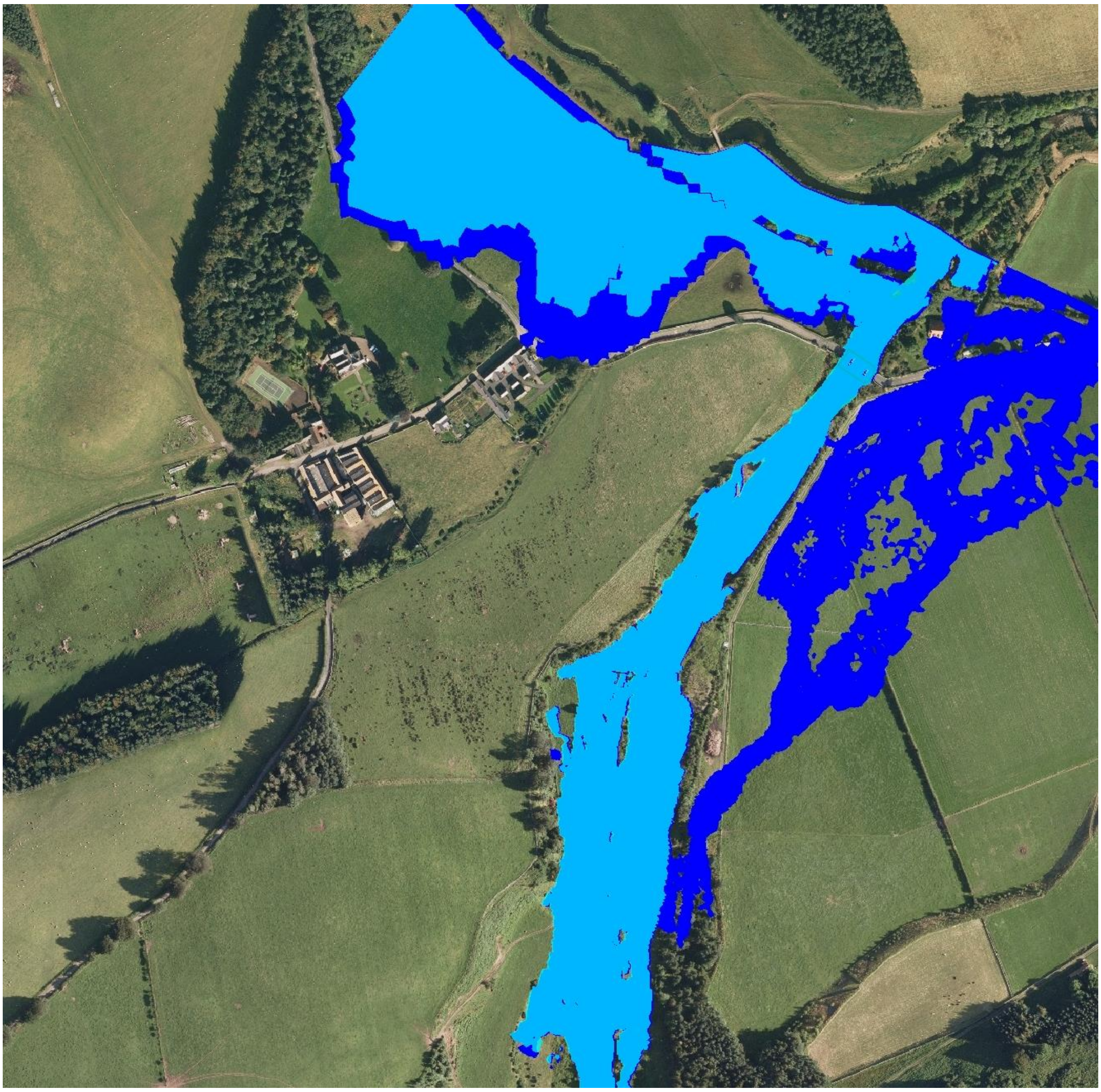


Figure 3.2. Comparison of inundation extents between existing and design at Q_{200yr} peak. Dark blue- existing conditions; light blue- design.

3.2.2 Training effect of the log pairs.

The major design modification to the College Burn is a set of 15 logs; the downstream 12 of these are arranged in pairs (i.e. six pairs), angled at approximately 45 degrees to the channel direction while the three furthest upstream (one left bank, two right bank) are angled closer to the channel direction and largely buried. The logs are partially submerged and buried at their bank end, and further anchoring is also recommended to prevent the logs moving or being undermined. The near log velocities calculated by the hydraulic model were used in a spreadsheet to estimate drag and buoyancy loads on each log (refer to log load calculations, Appendix C). Logs that are significantly exposed to the flow require ballast or anchoring to assure stability and longevity of the design.

The logs, if maintained in place, result in an afflux and redirection of flow towards the channel centre. This afflux is not large enough to cause the existing river embankments to breach at Q_{200yr} peak. Figure 3.3 shows water depth and velocity vectors for design, centred on the log array, at the Q_{2yr} discharge level. A clear angling of the flow towards the centre of the channel is observed, and all of the log pairs are effective in redirecting flow. Figure 3.4 shows this training effect at the Q_{200yr} peak. At this level, most of the logs are submerged in deep water, and so their training effect is diminished- however, the final log pair is still effective at directing flow towards the channel centre, and hence stabilising the flow direction. Figure 3.5 is instructive in demonstrating the added stability the logs give to the flow direction during the design Q_{200yr} event; the figure shows the mean flow velocity angle (weighted by depth) on the approach to the bridge on the apron. For existing conditions there is a larger range of flow angles for the Q_{200yr} event (existing mean of 47 degrees, range of 5.8 degrees and standard deviation of 1.9 degrees) than for design condition with log stabilisation of the flow (mean of 48 degrees, range of 3.9 degrees and standard deviation of 1.2 degrees). The flow direction is therefore stabilised through the bridge by the design log pairs.

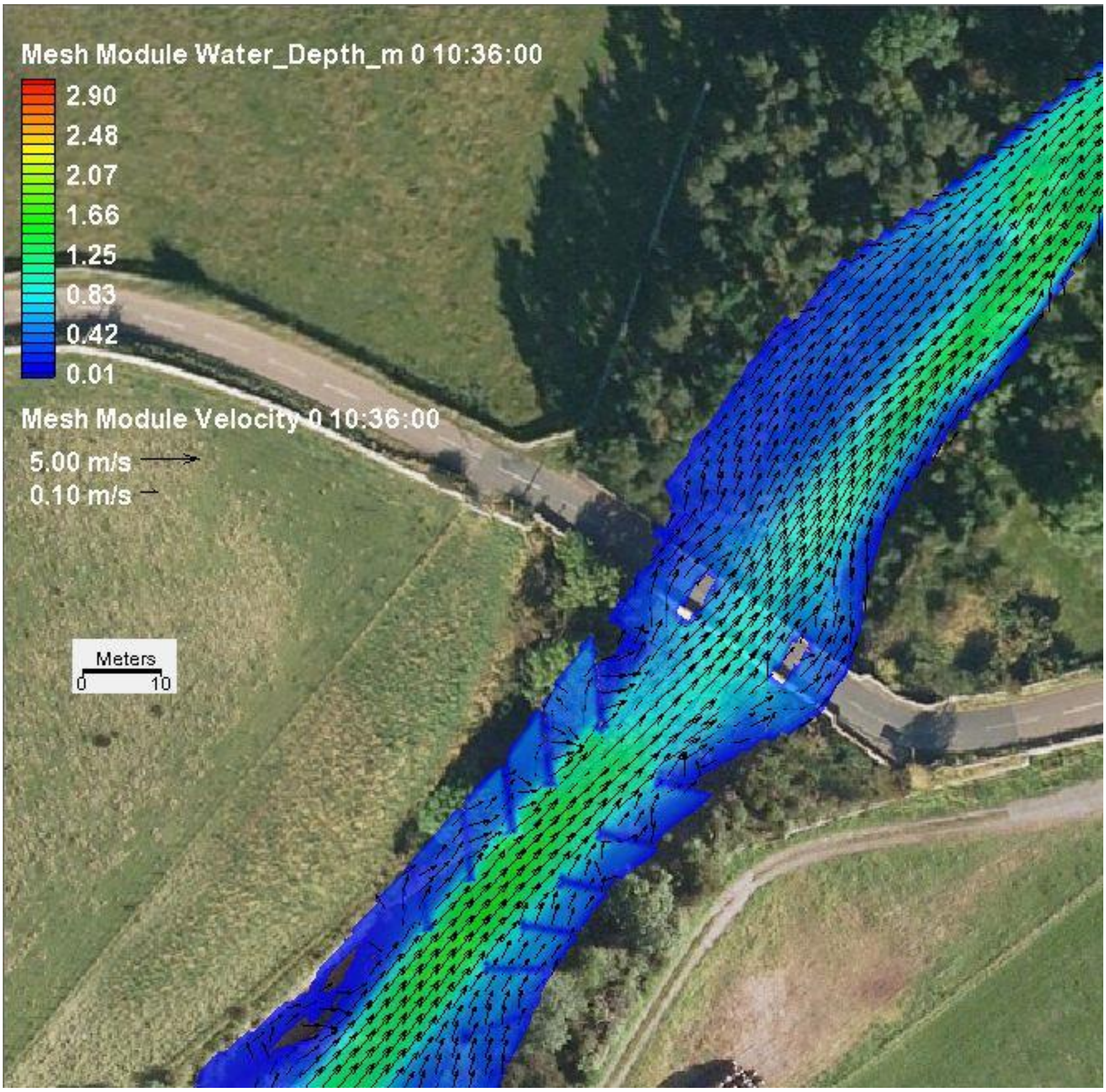


Figure 3.3. Training effect of log pairs at Q_{2yr} peak flow. All pairs act to direct flow into centre channel.

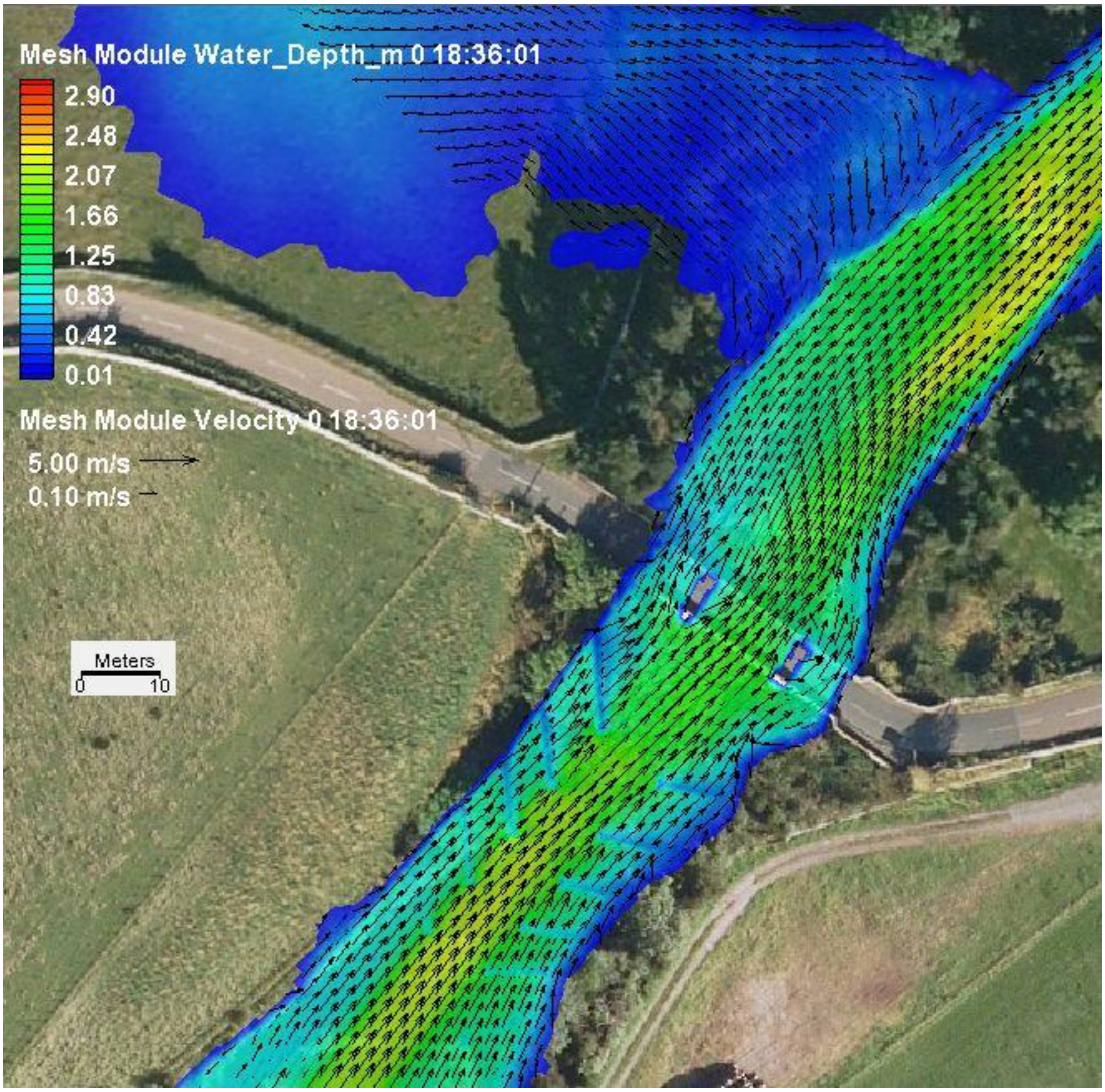


Figure 3.4. Training effect of log pairs at Q_{200yr} peak flow. Final log pair still acts to direct flow into centre channel.

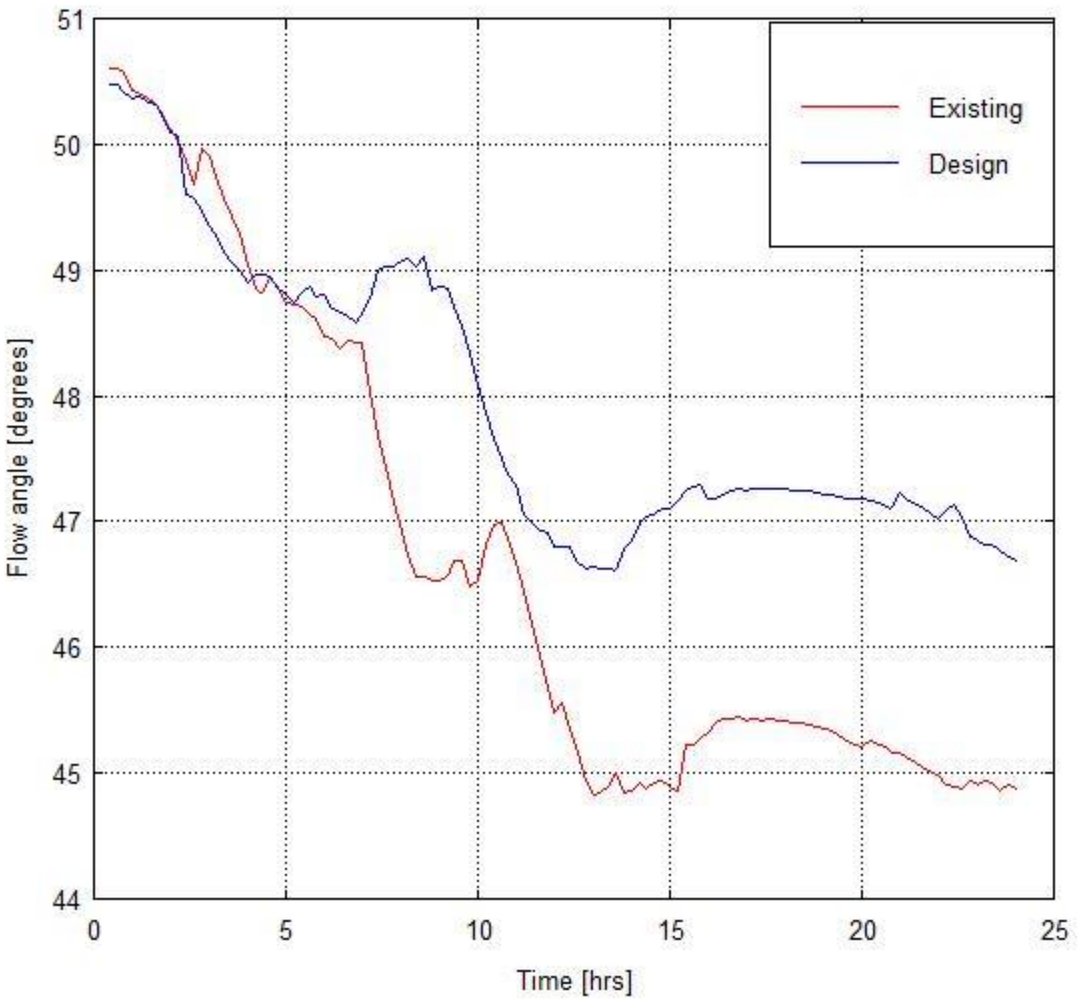


Figure 3.5. Mean water flow angle on approach to the bridge on apron during Q_{200yr} hydrograph. Existing condition mean 46.7 degrees with a range of 5.8 degrees; Design condition mean 48 degrees, with a range of 3.9 degrees. Design condition is stable over the Q_{200yr} peak and closer to the mean thalweg direction than existing conditions.

3.2.3 Bed shear stresses

The peak mean shear stress measured along the thalweg during the Q_{200yr} hydrograph is 119 Pa for existing conditions and 117 Pa for design conditions, i.e. similar for existing and design.

Figure 3.6. shows the bed shear stress at the peak of the Q_{200yr} hydrograph for design conditions showing that the pattern of shear stress is slightly modified by the design at the log locations, the apron, immediately downstream of the apron and at the old railway abutment. Figure 3.7 shows the bed shear stress and velocity vectors over the log pairs and bridge apron at the peak of the Q_{200yr} hydrograph for design conditions.

Notably, the shear in the channel is reduced on the upstream approach to the bridge adjacent to the logs (caused by the afflux of the combined log pairs) but experiences a peak in the channel at the last log pair (likely resulting in some local scour here unless increased armouring is implemented). The design hydraulic model treats the apron as a region of constant roughness/friction, and this model predicts a secondary peak in shear stress centred on the concrete apron, and this increase over existing conditions carries on to the channel downstream of the apron lip. In reality, the shear stress distribution on the apron, and immediately downstream, will be complex and affected by the un-modelled local effects of the boulder pairs[5] recommended to assist fish passage. It might be expected that the larger boulder pairs will further dissipate energy over, meaning that model may represent the 'worst case scenario' in terms of shear stresses over the apron.

Figure 3.8. and Figure 3.9 show the shear stress values for existing and design conditions along the thalweg (from the ~90 degree channel bend mid-reach along a 400m stretch of straightened channel through the log pairs, bridge and down to the railway abutment) for both the Q_{2yr} level the Q_{200yr} peak. The position of the upstream and downstream apron lip and the railway abutment are shown. The maximum mobilised particle diameter, based on a Shields parameter of 0.04 for incipient motion, is shown in Figure 3.10 and Figure 3.11. These figures indicate that the higher shear stress values at the apron lip may warrant some armouring of the channel to prevent local scour from undermining the apron, and that this situation should be regularly monitored after high flood events. The extent of the required armouring may be estimated from Figure 3.10 and Figure 3.11 and is discussed in Section 3.2. However, the average sediment transport in the channel is not predicted to change greatly as a result of the design.

Notably, the design changes to the abutment increase conveyance and reduce the water velocity in the channel downstream of the abutment position. This design change can be clearly seen to reduce a peak in channel shear stress and will reduce erosion in this area.

[5] Each boulder will create a wake of much slower velocity and shear stress with the interactions between the wakes of the boulder pairs being complex; these interactions have not been modelled explicitly.

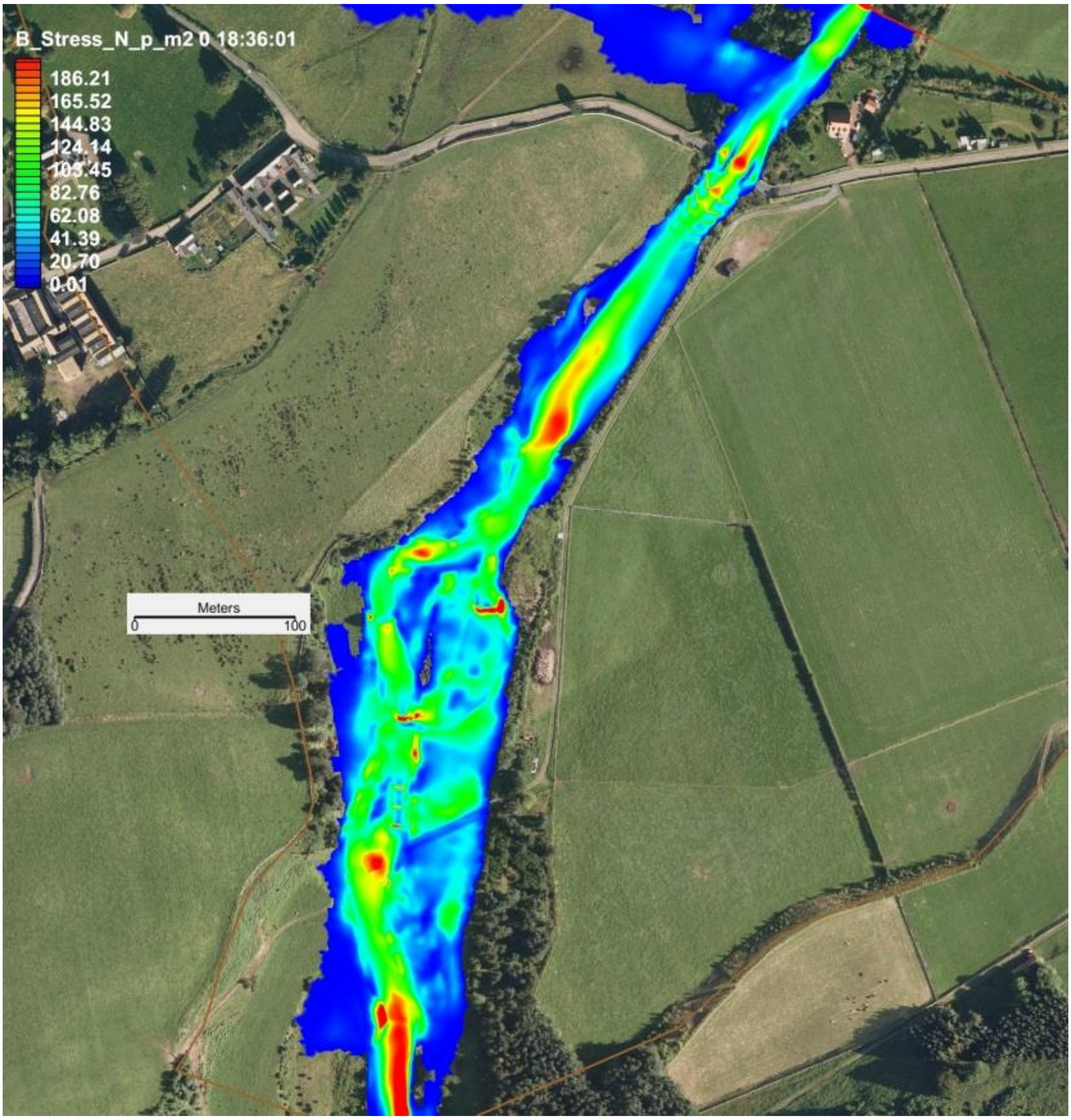


Figure 3.6. Bed shear stress for design condition at Q_{200yr} peak.

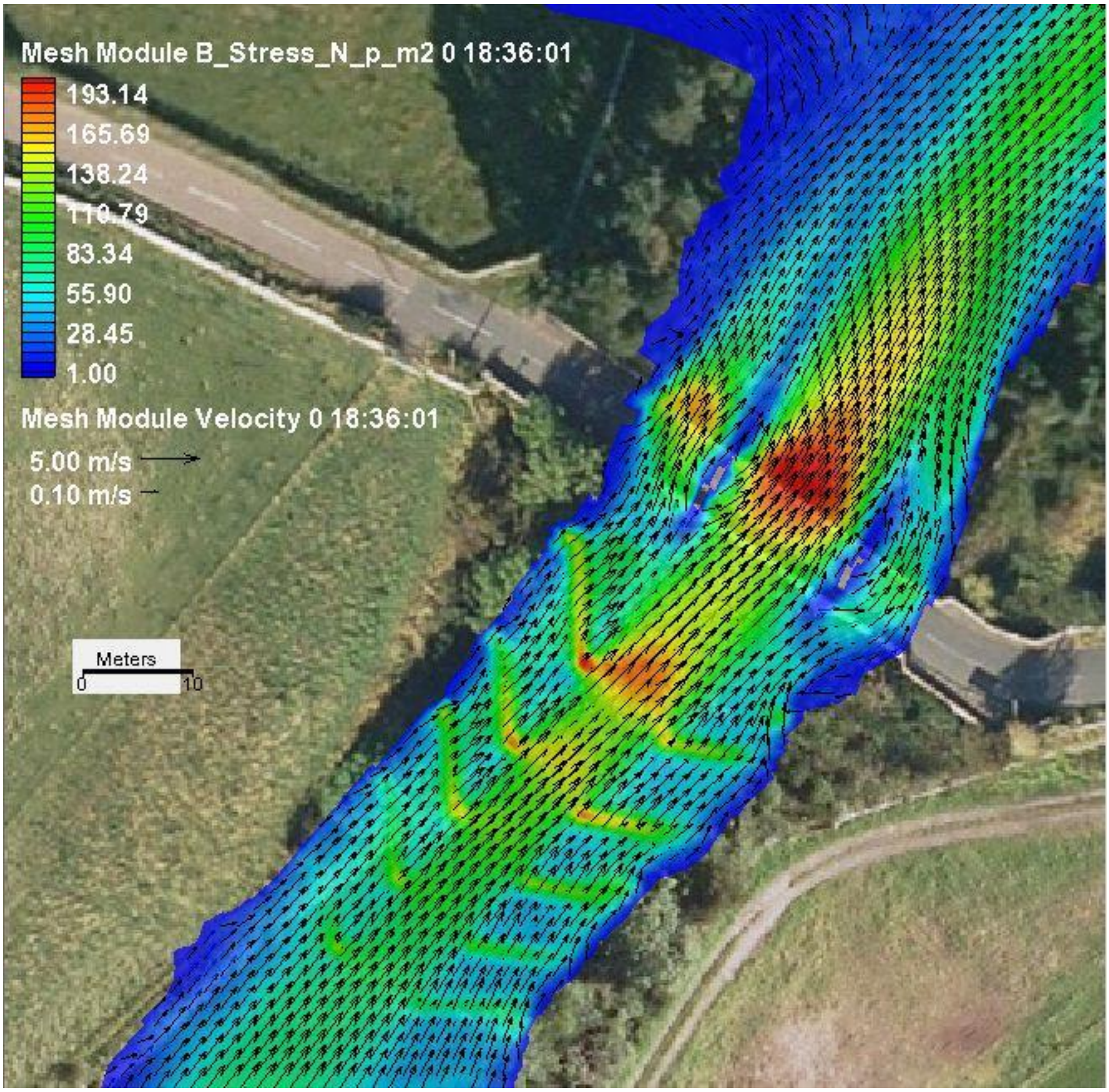


Figure 3.7. Bed shear stress and velocity vectors over the log pairs and on the bridge apron at Q_{200yr} peak.

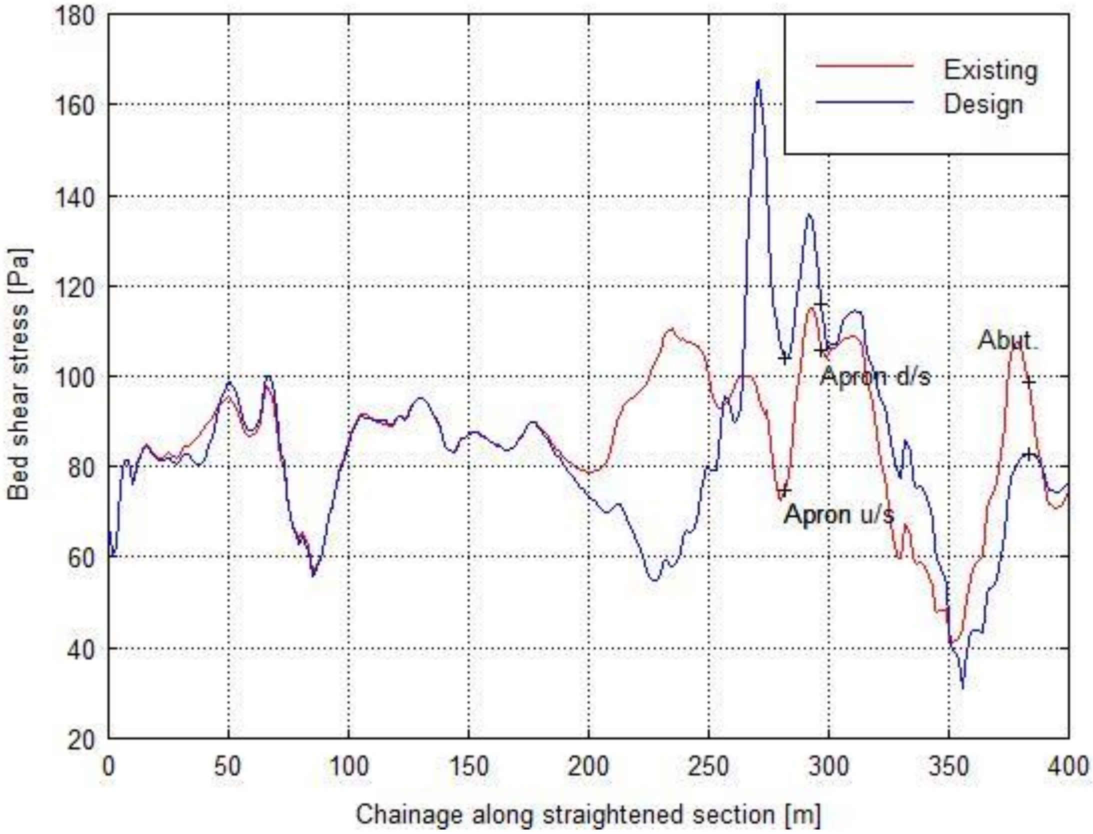


Figure 3.8. Bed shear stress along thalweg for existing (red line) and design (blue line) at Q_{2yr} level. Positions of apron and abutment indicated. Only the thalweg from the mid-reach 90 degree bend along the straightened section to the model exit is shown.

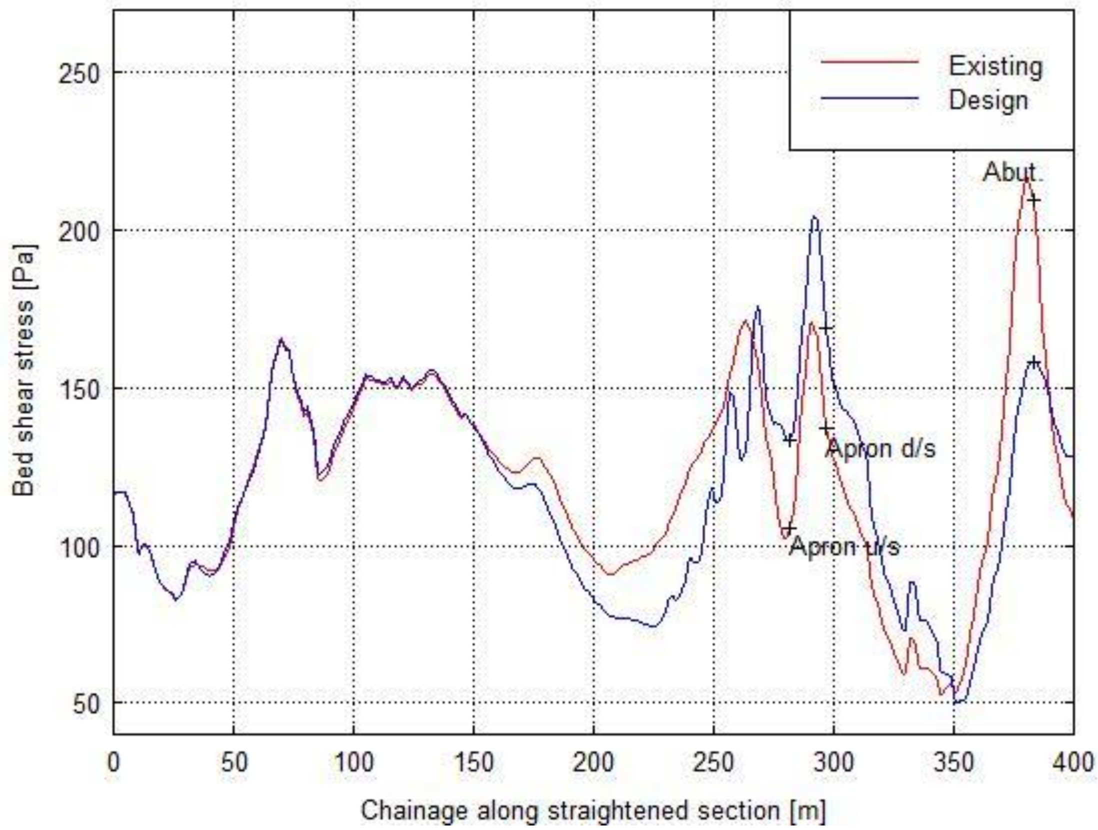


Figure 3.9. Bed shear stress along thalweg for existing (red line) and design (blue line) at Q_{200yr} peak. Positions of, apron and abutment indicated. Only the thalweg from the mid-reach 90 degree bend along the straightened section to the model exit is shown.

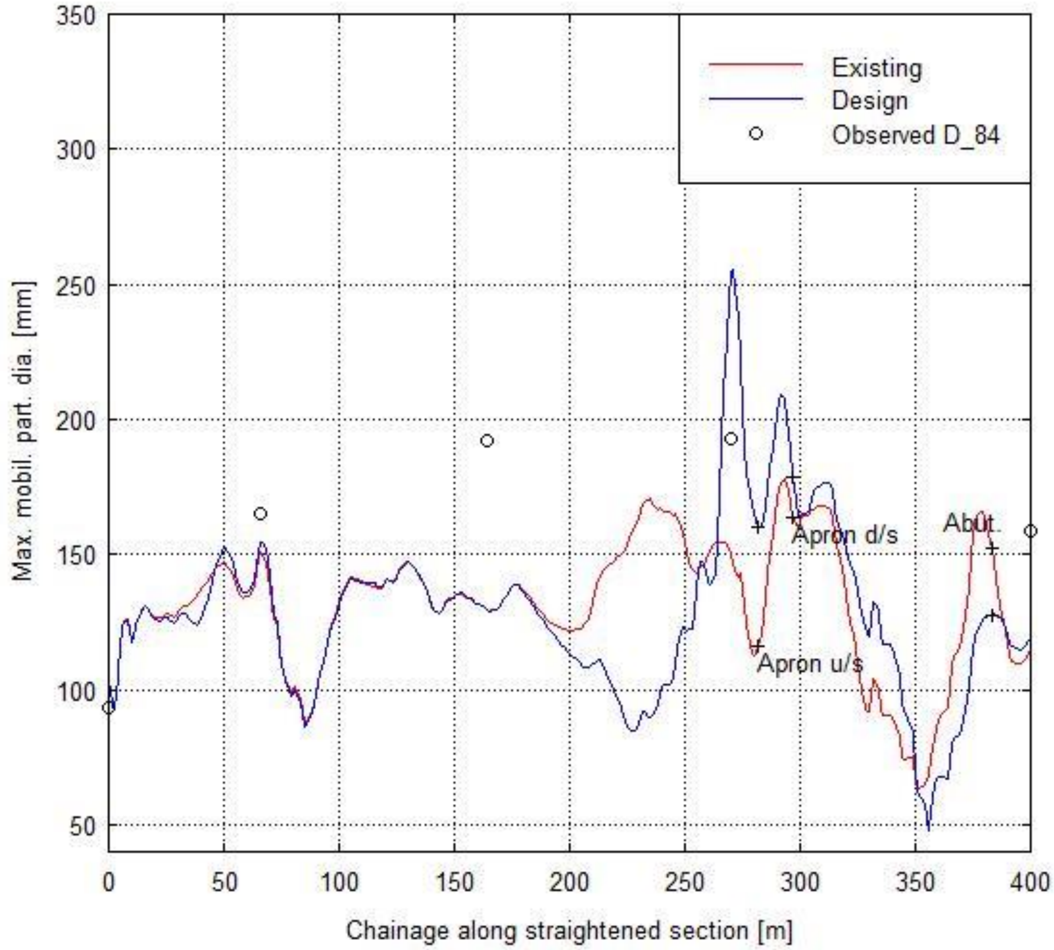


Figure 3.10. Maximum mobilised particle diameter for a Shields stress = 0.04 at Q_{2yr} peak. Only the thalweg from the mid-reach 90 degree bend along the straightened section to the model exit is shown. Observed D_{84} particle size shown for comparison.

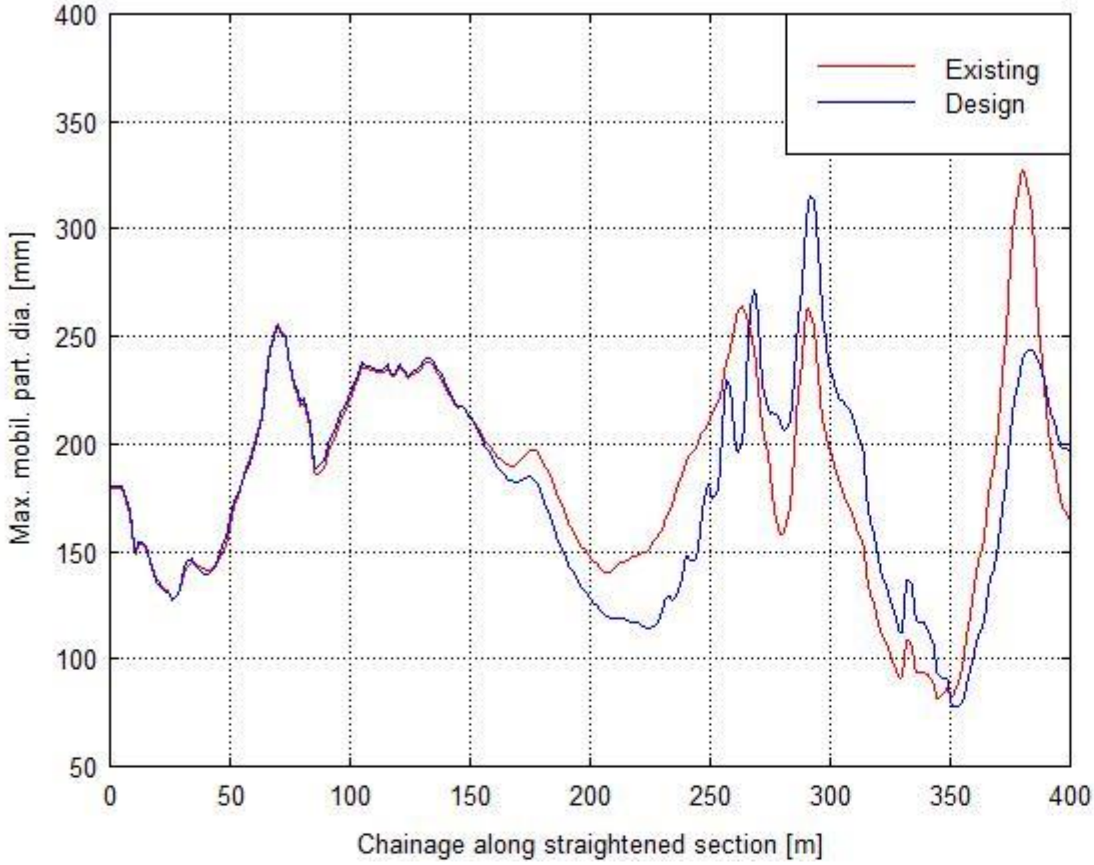


Figure 3.11. Maximum mobilised particle diameter for a Shields stress = 0.04 at Q_{200yr} peak. Only the thalweg from the mid-reach 90 degree bend along the straightened section to the model exit is shown.

3.3 LOG LOAD CALCULATIONS

The log load calculations are presented in Appendix C and provided in the associated Excel spreadsheet. The spreadsheet was created during the project to estimate the drag and buoyancy loads on the training logs during the Q_{200yr} peak. The methodology adopted was that of Brooks et al., 2006 and Abbe and Brooks, 2011.

Calculations on engineered log structures in rivers are subject to uncertainty, and the risks of a log structure coming free are significant- especially at Westnewton Bridge where the logs are large and could potentially block or damage the bridge itself. There are uncertainties in the magnitude of the discharge though the channel, actual (rather than depth averaged) velocity on approach to the logs, and estimation of the drag coefficient of wood structures. As a result, the calculations of the loads generated on the logs should be treated with caution, and a factor of safety has been used. For buoyant forces, a factor of safety of 2 has been used (and a dry wood density of 900 kg/m^3 assumed); for drag forces, a factor of safety of 1.6 is assumed. The log load calculations depend implicitly on the actual log dimensions used in construction- for the spreadsheet given a sample log size of 15 m length and 0.5 m diameter has been used. Depth averaged velocities vary over the logs, but the approach velocity of the water (i.e. as near as can be estimated to the free-stream) is 1.8 - 2.3 m/s. These approximate values have been used to determine drag forces based on projected area in the flow (i.e. not including the area of the log buried into the bank) and assuming that the flow is normal to the log (the worst case for drag). The spreadsheet estimates a required ballast force or anchoring force required for each log. As an approximate guide the maximum load experienced is 1.6 tonnes, but this assumes that the log is maintained in place and is not undermined. Appendix C should be referred to for the detail in the log load calculations, and the values of length, width and exposed height of log adjusted depending on the 'as built' dimensions. Care should be taken during construction to protect the logs from undermining, scour, etc. If very rough logs are used in construction, the drag coefficient may be increased within the suggested range.

3.4 ENGINEERING DESIGN DRAWINGS

Engineering drawings were created in Civil 3D to give a precise and accurate representation of the designs (Appendix D). These drawings include scaled plan views of the designs showing the horizontal positioning of all design elements. Scaled cross sections and long profiles that were extracted at critical design locations to inform the vertical dimensions of the design, and their relationship to the existing ground, were provided where necessary. Detail sheets were also produced to show the design elements, not sufficiently represented in plan, section or profile views.

4. MONITORING/ ONGOING MANAGEMENT OF DESIGN

As discussed above, the highly dynamic geomorphic character of the site means that no design can be guaranteed to be stable in the long-term. Although the design produced is the most appropriate given physical processes and site constraints, post-construction monitoring and the potential for 'adaptive management' are strongly advised. A full topographic survey of a section of the channel extending ~200 m upstream and ~100 m downstream of the bridge should be conducted immediately after construction with further periodic resurveys undertaken after significant flood events (but at least annually even if no large flow events occur). Sediment sampling (Wolman-walk pebble counts) should also be conducted at the same time as topographic surveying, the information gathered providing further indication of sediment transport processes that may indicate implications for the design. Periodic walk-over surveys should also be conducted, inspecting specific elements of the design (e.g. condition of the log structures, sediment accumulating on the bridge apron, etc.) and recording condition with detailed photographs.

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APPENDIX A

FLUVIAL AUDIT METHODOLOGY

Fluvial Audit Methodology

INTRODUCTION

The procedure used to characterize the geomorphic and sedimentary regimes of the College Burn is an adaptation of the 'Fluvial Audit' methodology proper as described in Sear et al. (1995). It has been developed by Dr. H. Moir over the last 10 years for the application to Scottish river systems and used in peer-reviewed, published scientific literature (Moir et al. 2004, Moir and Pasternack 2008) and a number of grey literature project reports.

The approach is explicitly more process-based than 'Fluvial Audit', classifying channel character in terms of observed morphology, the physical expression and integration of fluvial processes. 'Fluvial Audit' is process-based in principal but is designed to incorporate data from the River Habitat Survey (RHS) methodology which is not process-based. The inventory approach to classifying stream character in RHS tends to confuse form and process and is therefore not the best approach to inform as to controls on the geomorphic character of a river system. Where there is not access to existing RHS data there is little justification in employing the 'Fluvial Audit' methodology proper.

APPROACH

The approach adopted here has two central theoretical concepts that concern the controls on the distribution of geomorphic process regimes in a river network. These are:

1. The hierarchical organisation of fluvial networks.

The physical characteristics of river systems are organized in a nested hierarchy, with physical processes operating at larger scales influencing those at successively finer resolutions (Frissell et al., 1986), ultimately controlling the micro-scale distribution of hydraulic and sediment transport processes (Figure A-1). The micro-, meso- and reach scales are therefore all equally critical elements within this hierarchy, with different geomorphic and ecological processes being relevant at each resolution. For instance, micro-scale factors will dictate the specific location that an animal selects habitat while the spatial distribution of meso-scale features will control the locations within a particular reach type where such conditions exist. The classification approach adopted here concentrates on the reach and morphological unit (meso) scales. The reconnaissance nature of the methodology precludes characterisation at the micro-scale that would require some degree of quantitative measurement and significantly increase survey time. Reach type is typically characterized at a relative scale of 10-100 channel widths in length with morphological units in the range 1-5 channel widths.

2. Basic physical controls on channel morphology.

The morphological character of the channel at a given location (i.e. reach type) is defined in terms of the relative balance between sediment supply and transport capacity (Figure A-2). In the case of Scottish upland gravel-bed streams, reach types typically progress from 'wandering' (Ferguson and Werritty, 1991) to pool-riffle to plane bed to step pool to cascade as the ratio of sediment supply to transport capacity decreases. This sequence represents a decreasing storage of alluvial sediment (mainly gravel and cobble sizes) within the active channel. The continuum also tends to be associated with increasing channel slope and mean substrate size and a decrease in the frequency of dynamic channel behaviour. Some stretches of a stream may exhibit features of more than one reach type and are therefore classified as having transitional morphologies (e.g. pool-riffle/plane bed). An additional reach type of 'slow glide' is required in lower energy systems. This is a morphology indicative of a channel condition with low sediment supply and transport capacity (Figure A-2). At the next spatial scale down, characteristic morphological units are associated with each reach type over a longitudinal scale of many (>10) channel widths (Montgomery and Buffington, 1997); indeed, the assemblage of morphological units to some extent defines reach type. Despite being explicitly linked through the concept of hierarchical organization, reach type and morphological unit scale data provide different information. Reach type indexes the general spatial distribution of 'geomorphic regime' of a river system (i.e. the approximate ratio of sediment supply to transport capacity) while morphological unit data provides higher resolution qualitative insight as to meso-scale hydraulic, sedimentary and instream habitat conditions.

Set within these central concepts, the spatial distribution of channel morphology classifications (at the reach and morphological unit scales) and factors that influence the sediment supply and transport capacity regimes are recorded. All spatial information is obtained from a hand-held Global Positioning System (GPS) with typical accuracy ±5m.

Reach type classification

This is a qualitative, expert judgment classification approach and developed from established procedures (Montgomery and Buffington, 1997; Brierley and Fryirs, 2000). As discussed above, it is based on the physical character of the channel, particularly the presence and type of bedforms. Classification is not carried out based on a single point observation. Rather, channel condition is observed over at least 10 channel widths so that the classification is commensurate with the spatial resolution defined for reach type (Figure A-1).

Classification of controls on process regime

Additional to the reach-scale morphological classification, factors that influence the process regime of the channel (i.e. those potentially influencing the delivery and movement of sediment to the channel) are also recorded. These data can subsequently be linked to the morphological data to provide some insight as to the dominant controls on spatial patterns of physical channel condition. These factors are recorded as linear (e.g. bank erosion, tree cover, bank protection) or point data (e.g. tributary input, large woody debris, weir). The upstream and downstream limits of linear features are recorded. Where relevant, the river bank the data is associated with is also recorded. Data is collected in the following three categories:

i) *Sediment input/ storage*:

a. Bank erosion (including poaching by livestock). Linear (often point for poaching). This is categorized in terms of severity depending on the condition of the bank, height of bank and other indicators as to sediment input rate (e.g. previously bank-side fences within channel. Collapsing bank-side trees, condition of adjacent channel bed).
b. Tributaries. Point. These are characterized as low, moderate or large relative sediment input depending on the character of main-stem channel at the confluence

(e.g. presence of confluence bar) and the characteristics of tributary sub-basin (e.g. drainage area relative to the main-stem channel, relief, rainfall).

c. Depositional sedimentary features. Linear. The longitudinal extent, type (e.g. point, lateral, transverse, medial) and 'dynamic condition' of bar features is recorded. 'Dynamic condition' is a subjective definition depending on the appearance of the bar (e.g. vegetated or not, sorting, abrasion marks on clasts etc).

ii) *Vegetation*:

a. Bank-side tree cover. Linear. Recorded when tree cover is suffiently close to the active channel to influence fluvial process (e.g. local hydraulics, bank stability).
b. Large Woody Debris (LWD). Linear or point depending on extent of feature. The degree to which the feature spans the active channel (and, therefore, impacts fluvial processes) can be recorded.
c. Macrophytes. Linear. Sections of the channel bed exhibiting extensive macrophyte cover are recorded. Macrophythes can be a very important control on fluvial process in low energy river systems.

iii) *River engineering*:

a. Bank protection. Linear. The extent, type (e.g. gabions, boulder, wall etc) and state of repair of bank protection is recorded and then categorized in terms of likely impact to fluvial processes as low, moderate or high.
b. Bridges. Point. The number of bridge piers impacting fluvial process (i.e. piers within the active channel) and the clearance from the channel bed to the bridge span (indicating the likelihood of impedance of flood flows) are recorded.
c. Weirs. Point. The height and state of repair of weir structures are recorded.
d. Croys/ groynes. Point. The height, state of repair and extent into the active channel of croys/ groynes are recorded.
e. Fence crossings. Point. Fences that transversely cross the channel and potentially impact the movement of water, sediment and debris are recorded. Evidence of trapped debris or associated sedimentary deposits is recorded.
f. Ford crossings. Point. Vehicle ford crossings are recorded with the observed impact to the channel bed noted.

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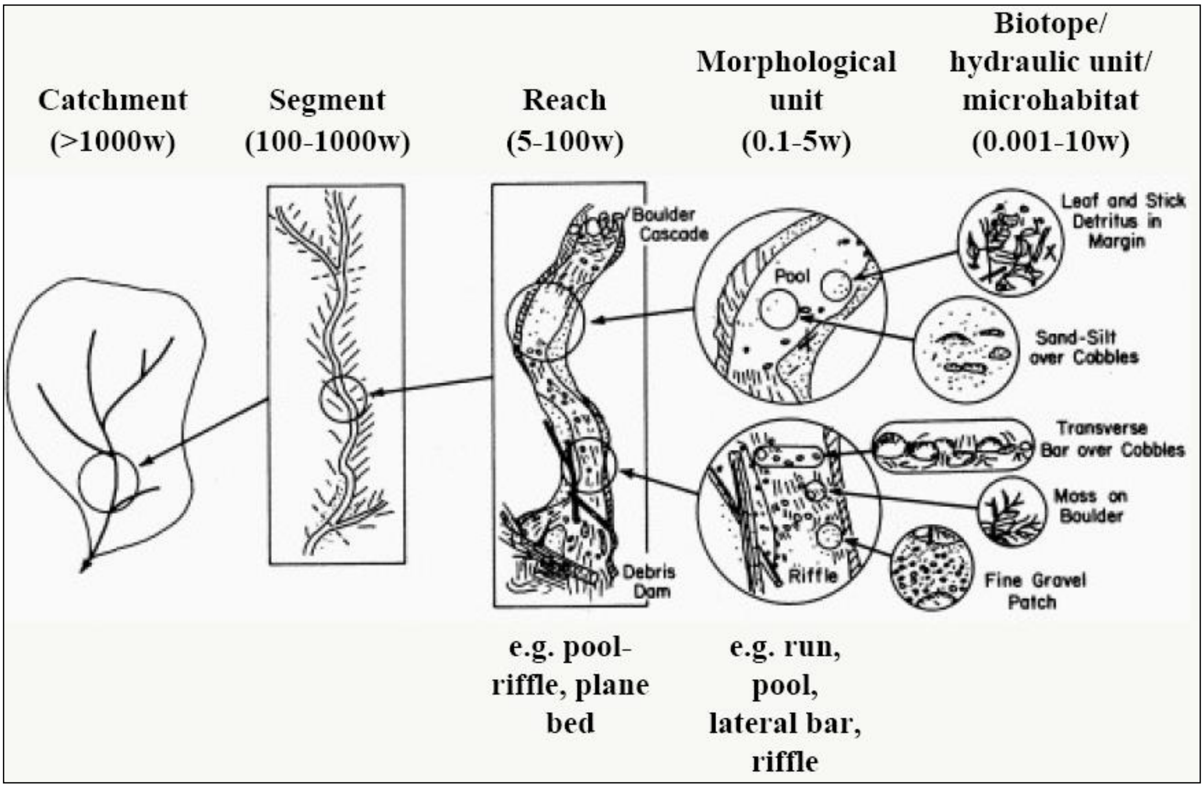


Figure A-1. Conceptual diagram of the spatial hierarchical organization of river networks (modified from Frissell et al. 1986).

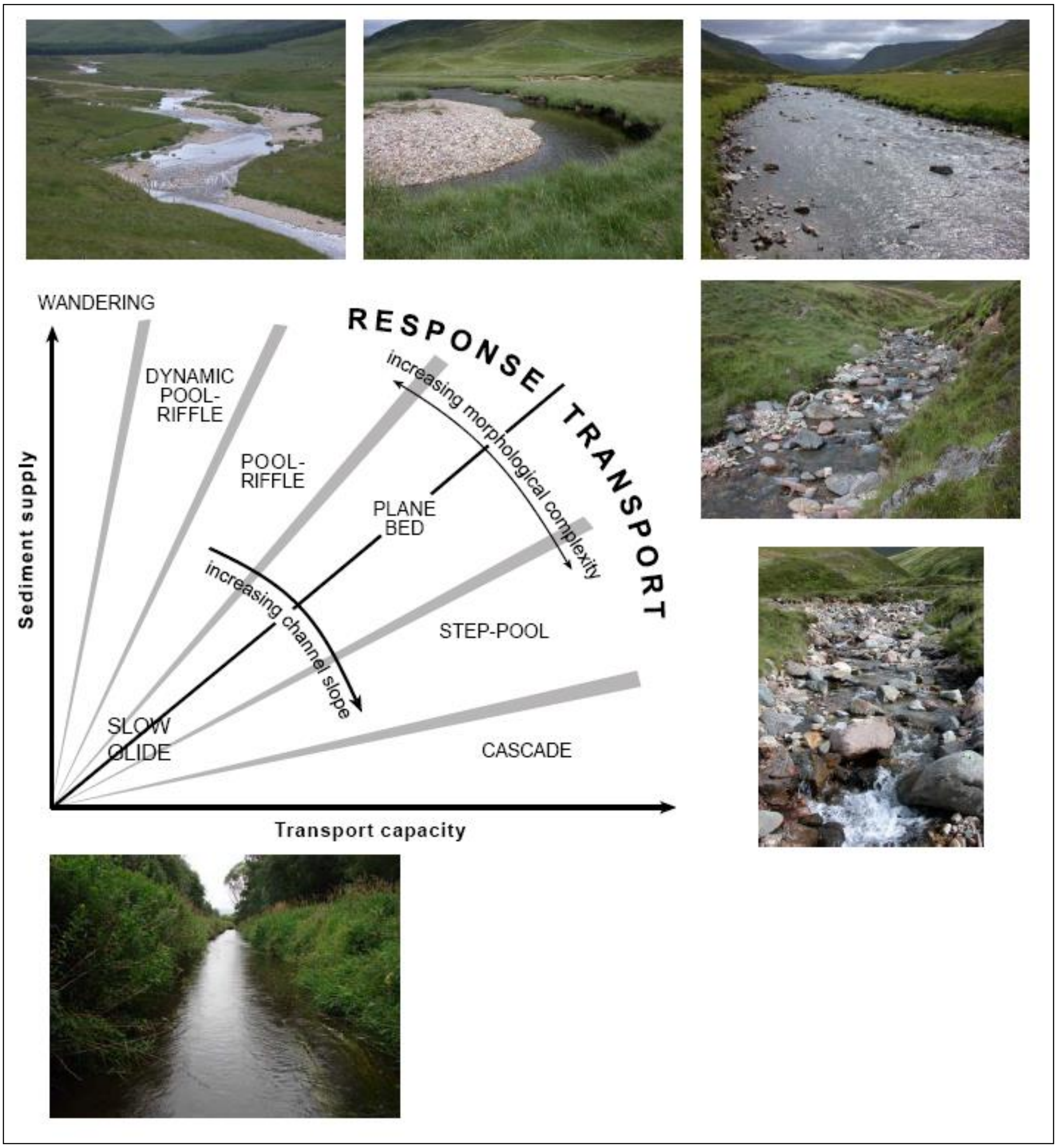
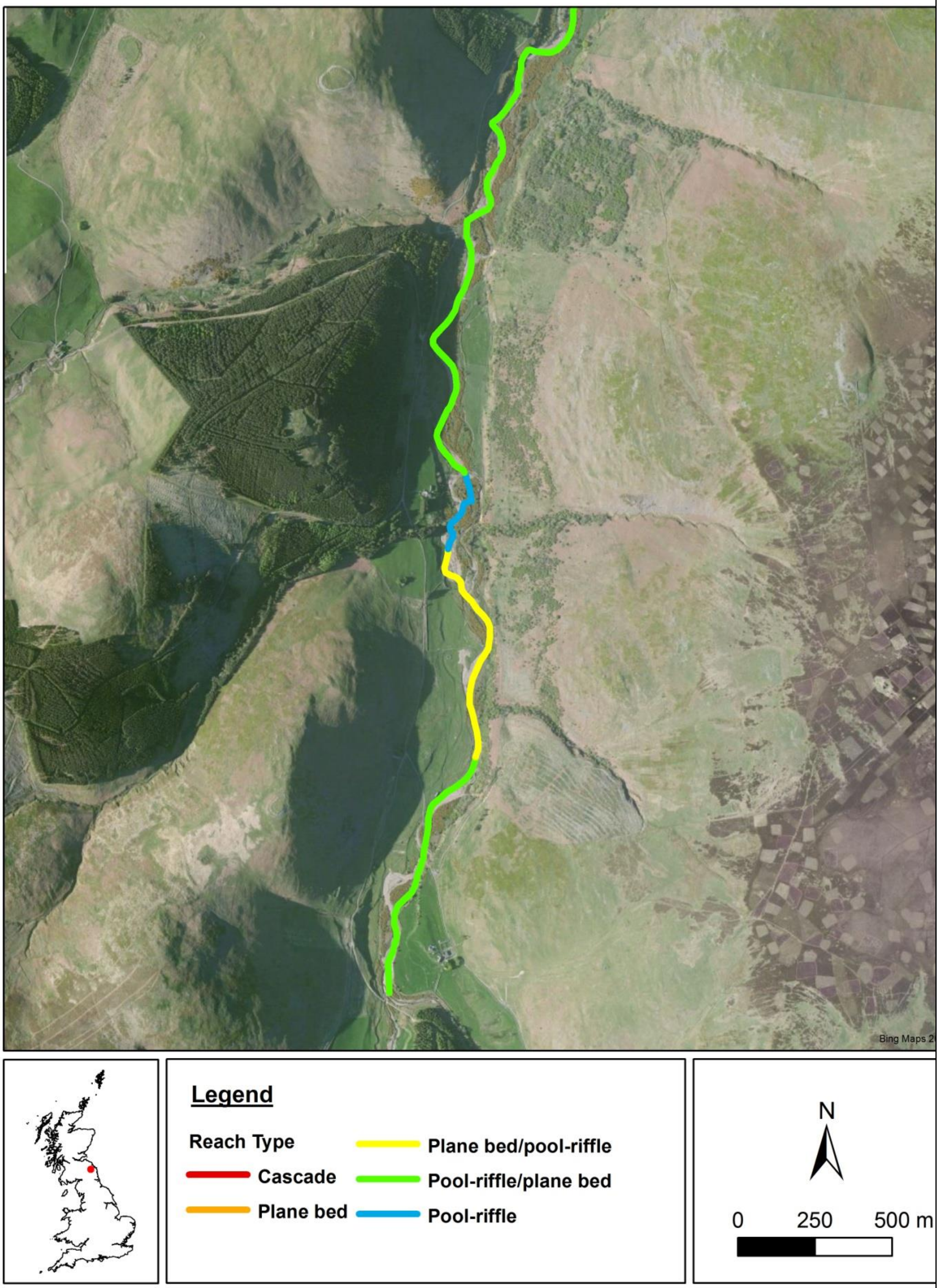


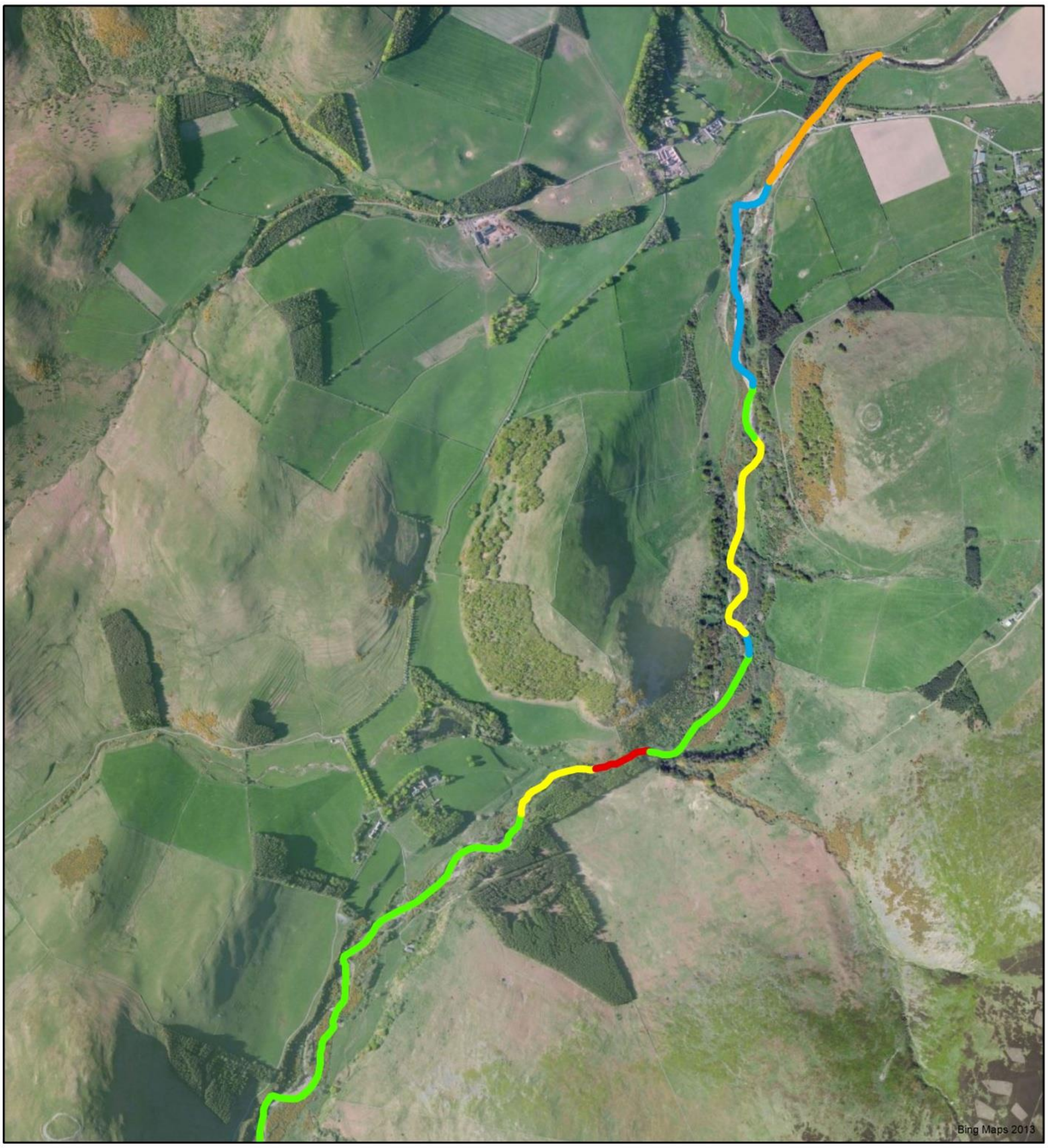
Figure A-2. Conceptual diagram of the physical controls on reach-scale channel morphology (modified from Moir et al., 2004).

APPENDIX B

FLUVIAL AUDIT MAPS



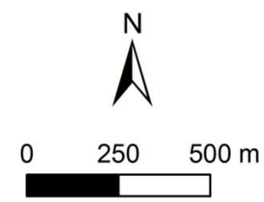
Notes:		*Westnewton Bridge modeling & design* **Reach type - upstream**		
		Project No. U13-1003	Created By: GK	**Figure B1**



Legend

Reach Type

- **Cascade**
- **Plane bed**
- **Plane bed/pool-riffle**
- **Pool-riffle/plane bed**
- **Pool-riffle**



Notes:	cbec eco engineering	*Westnewton Bridge modeling & design* **Reach type - downstream**		
		Project No. U13-1003	Created By: GK	**Figure B2**

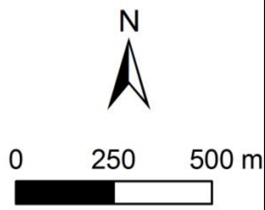
Legend

Sediment Deposition — **Surveyed River**

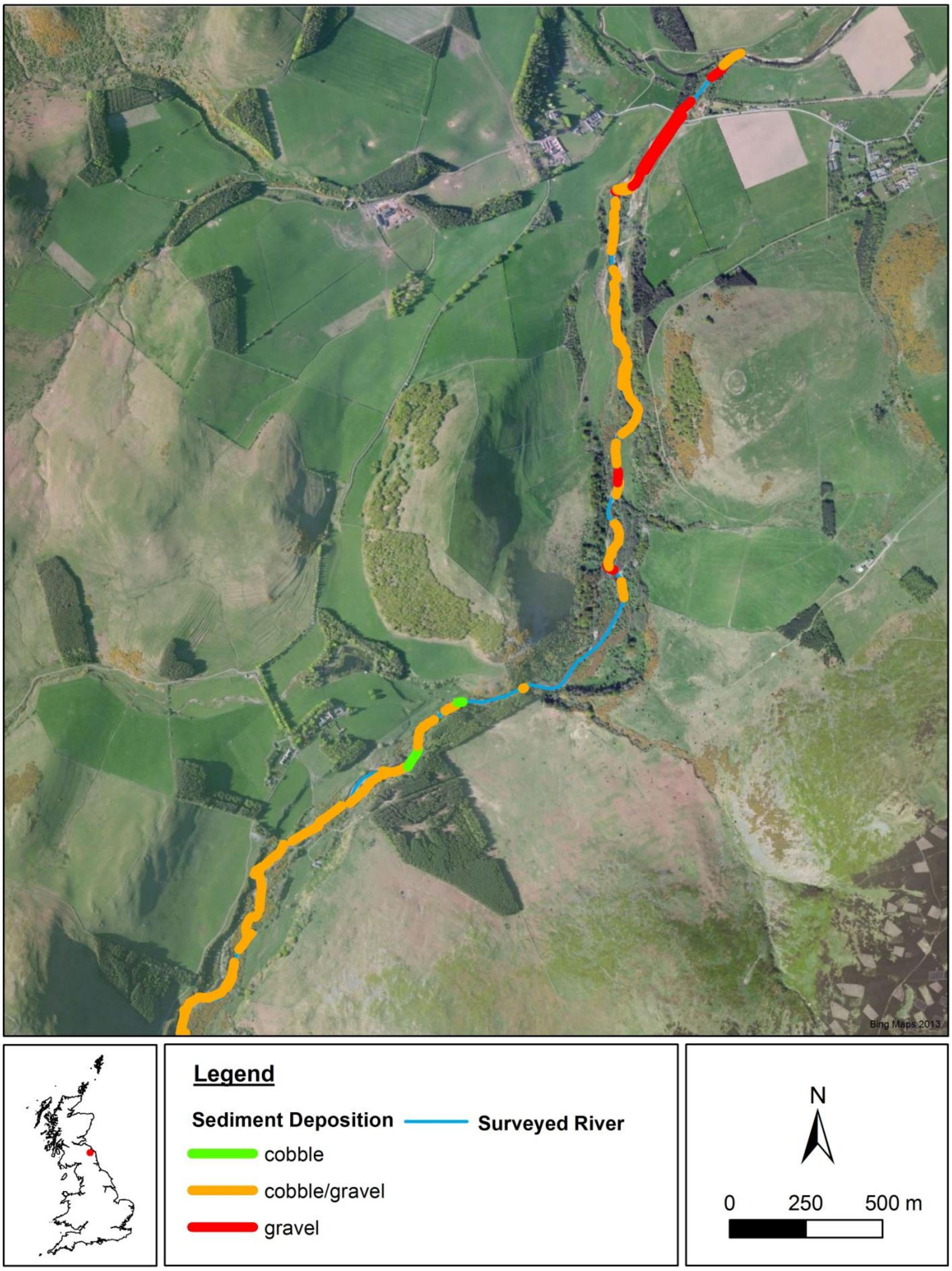
cobble

cobble/gravel

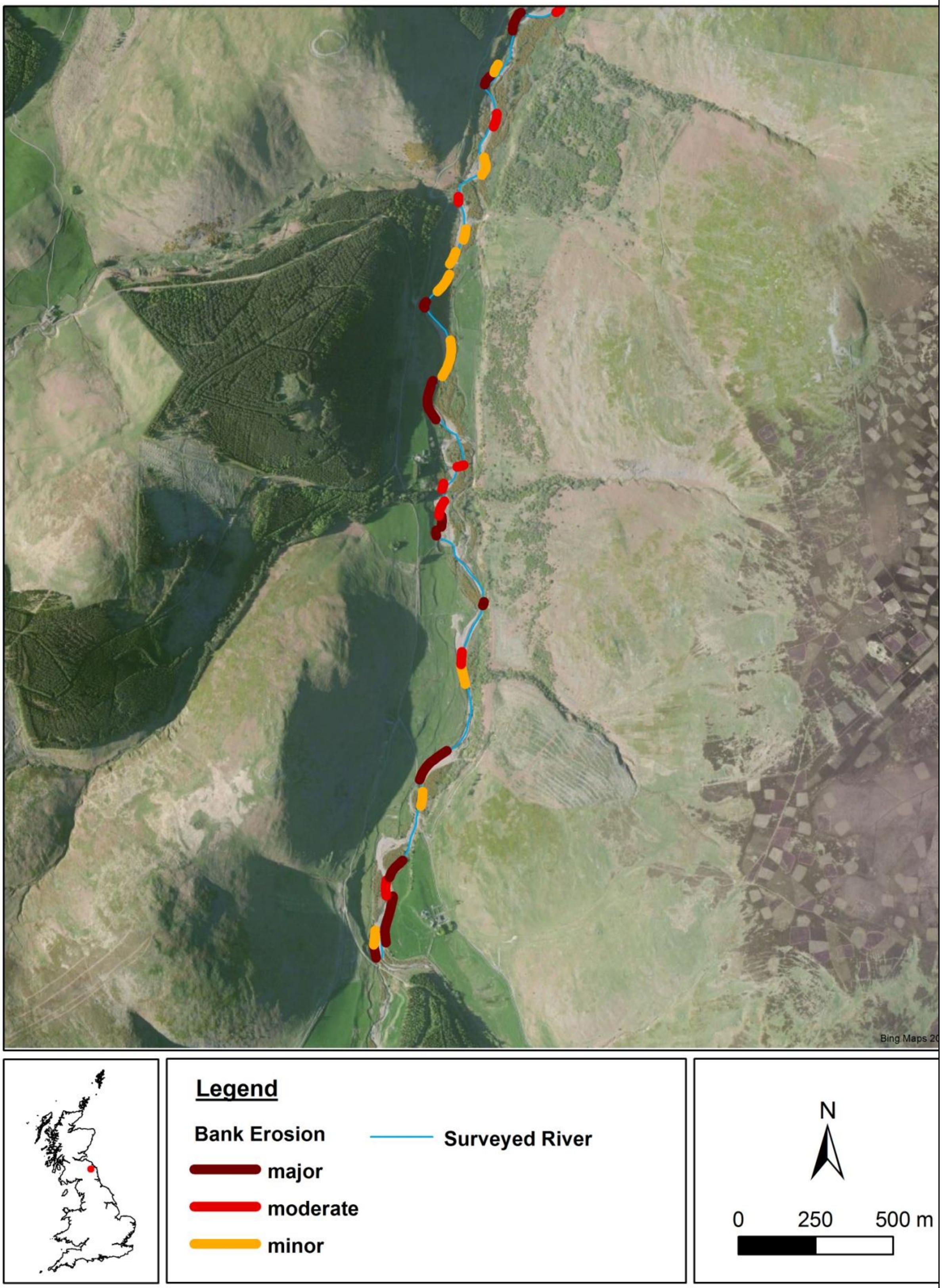
gravel



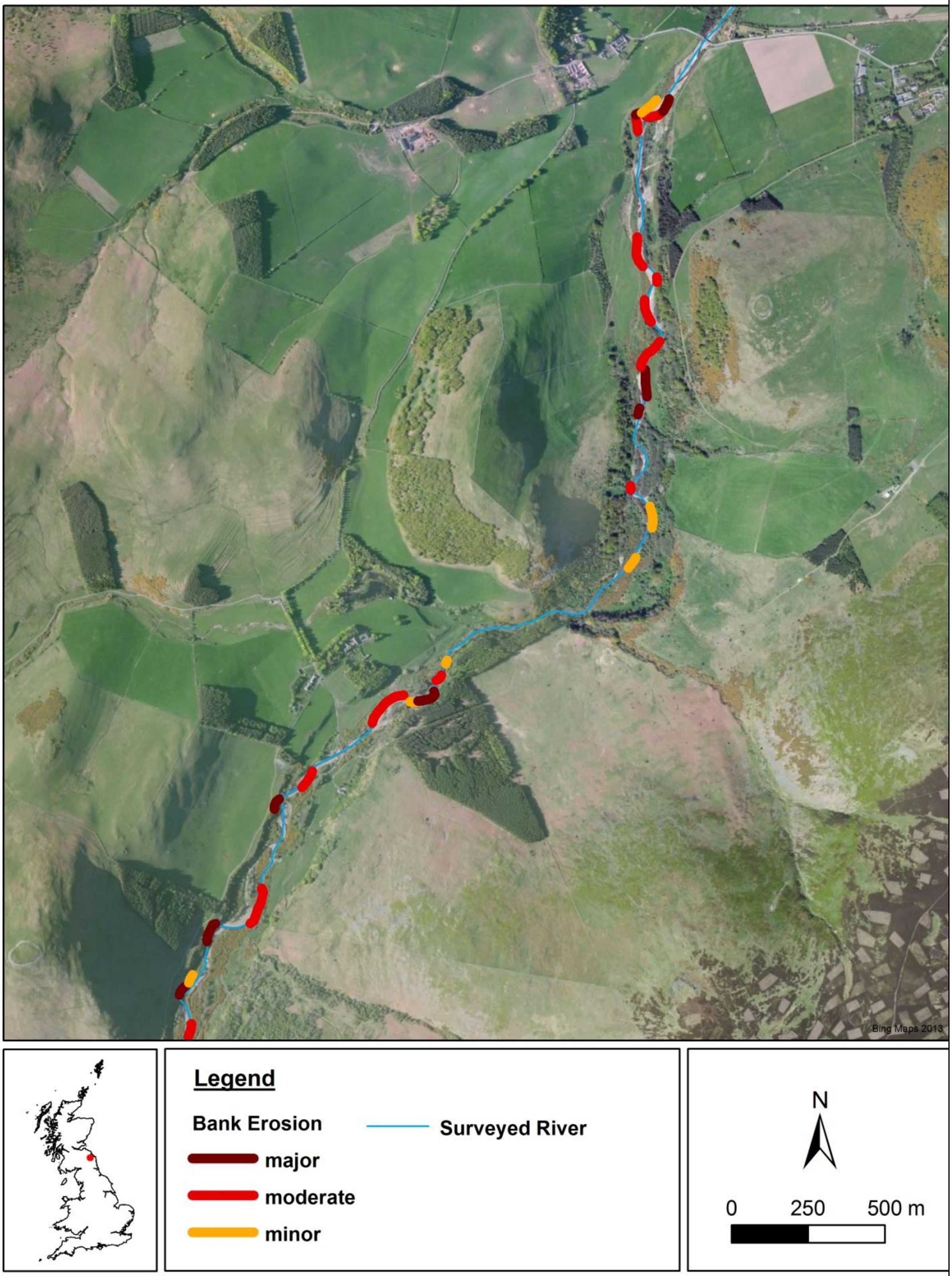
Notes:	cbec eco engineering	*Westnewton Bridge modeling & design* **Alluvial deposition - upstream**		
		Project No. U13-1003	Created By: GK	**Figure B3**



Notes:	cbec eco engineering	*Westnewton Bridge modeling & design* **Alluvial deposition - downstream**		
		Project No. U13-1003	Created By: GK	**Figure B4**



Notes:	cbec eco engineering	*Westnewton Bridge modeling & design* **Bank erosion - upstream**		
		Project No. U13-1003	Created By: GK	**Figure B5**



Notes:	cbec eco engineering	*Westnewton Bridge modeling & design* **Bank erosion - downstream**		
		Project No. U13-1003	Created By: GK	**Figure B6**

Legend

Engineering

- Bank Protection
- Bridge (no piers)
- Bridge (with piers)
- Dredged/straightened
- Other
- Weir
- Embankments
- **Surveyed River**

N

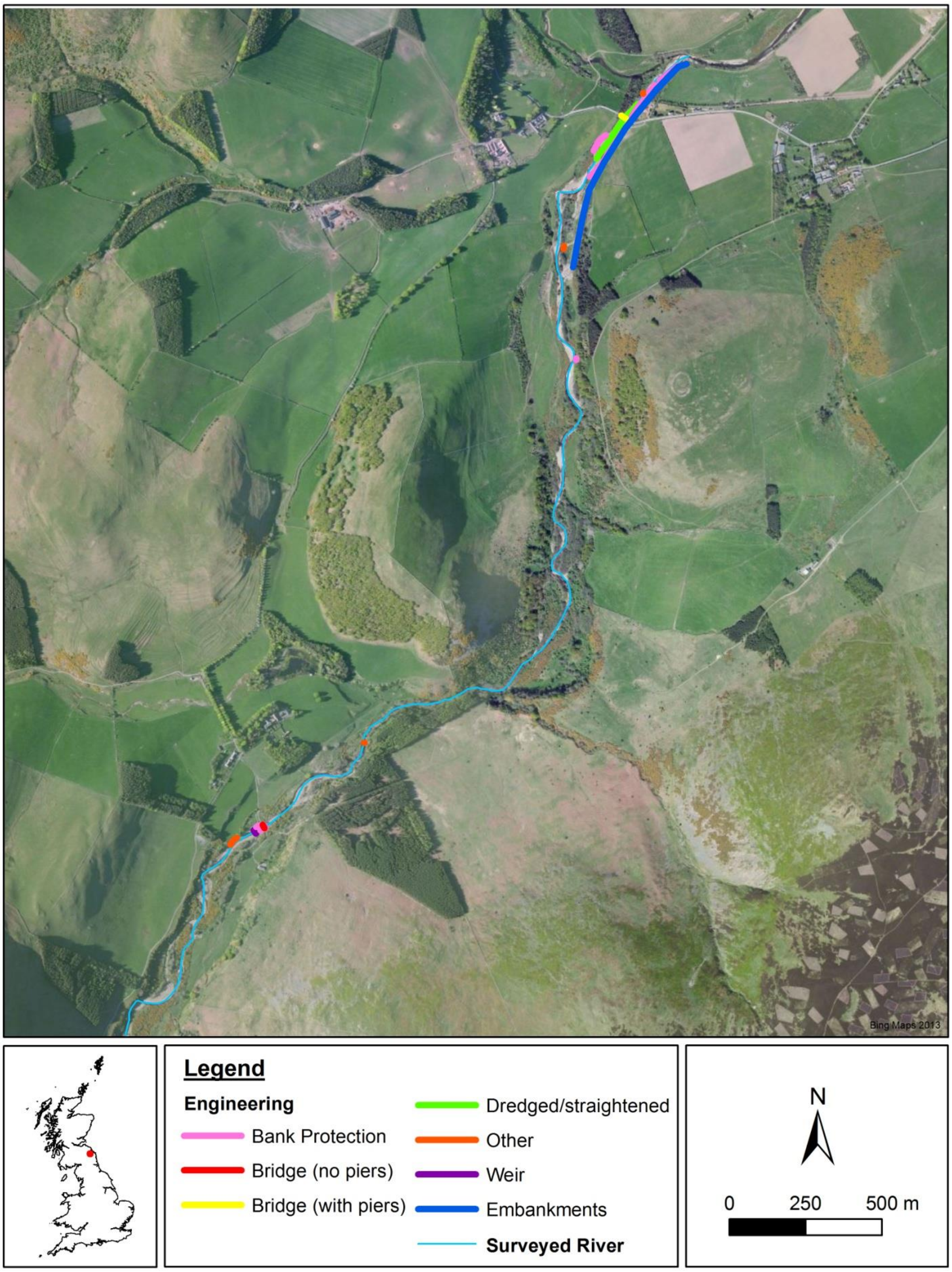
0 250 500 m

Notes:

Westnewton Bridge modeling & design

Engineering - upstream

Project No. U13-1003	Created By: GK	**Figure B7**



Notes:	cbec eco engineering	*Westnewton Bridge modeling & design* **Engineering - downstream**		
		Project No. U13-1003	Created By: GK	**Figure B8**

APPENDIX C

LOG LOAD CALCULATIONS

A spreadsheet was created during the project to estimate the drag and buoyancy loads on the training logs during the Q200year peak. The methodology adopted was that of Brooks et al., 2006, and Abbe and Brooks, 2011.

Calculations on engineered log structures in rivers is subject to uncertainty, and the risks of a log structure coming free are significant – especially at Westnewton Bridge where the logs are large and could potentially block or damage the bridge itself. There are uncertainties in the magnitude of the discharge though the channel, actual (rather than depth averaged) velocity on approach to the logs, and drag coefficient of wood structures. As a result, the calculations of the loads generated on the logs should be treated with caution, and a factor of safety has been used. For buoyant forces, a factor of safety of 2 has been used (and a dry wood density of 900kg/m^3 assumed); for drag forces, a factor of safety of 1.6 is assumed. The log load calculations depend implicitly on the actual log dimensions used in construction- for the spreadsheet given a sample log size of 15m length and 0.5m diameter has been used. These values (factor of safety, basic log dimensions and height of log protruding above surface) are in green shading at the top of the spreadsheet and may be altered depending on construction. See Figure C12.

GREEN SHADING INDICATES VARIABLES WHICH MAY BE ALTERED BY DESIGNER

NOMENCLATURE & DEFINITIONS

GENERAL:		
FSD	1.6	FACTOR OF SAFETY FOR DRAG
FSB	2	FACTOR OF SAFETY FOR BOUYANCY
D	0.5	LOG DIAMETER (m) (DEPENDS ON LOGS AS USED IN CONSTRUCTION)
L	15	LOG LENGTH (m) (DEPENDS ON LOGS AS USED IN CONSTRUCTION)
H	0.4	HEIGHT OF LOG ABOVE SURFACE (m) (CONSERVATIVE DESIGN BUT BEFORE EROSION)

Figure C12 Basic input variables for spreadsheet

A plan view of the design logs placed in the hydraulic model DEM is also given in the spreadsheet and shown here in Figure C13.

REFERENCES

Abbe T, Brooks A, 2011. Geomorphic, Engineering, and Ecological Considerations when using wood in River Restoration, in Stream Restoration in Dynamic Fluvial Systems, A Simon, S Bennet and J Castro editors, AGU Washington

Brooks, A. et al., 2006. Design guideline for the reintroduction of wood into Australian stream, Land & Water Australia, Canberra.

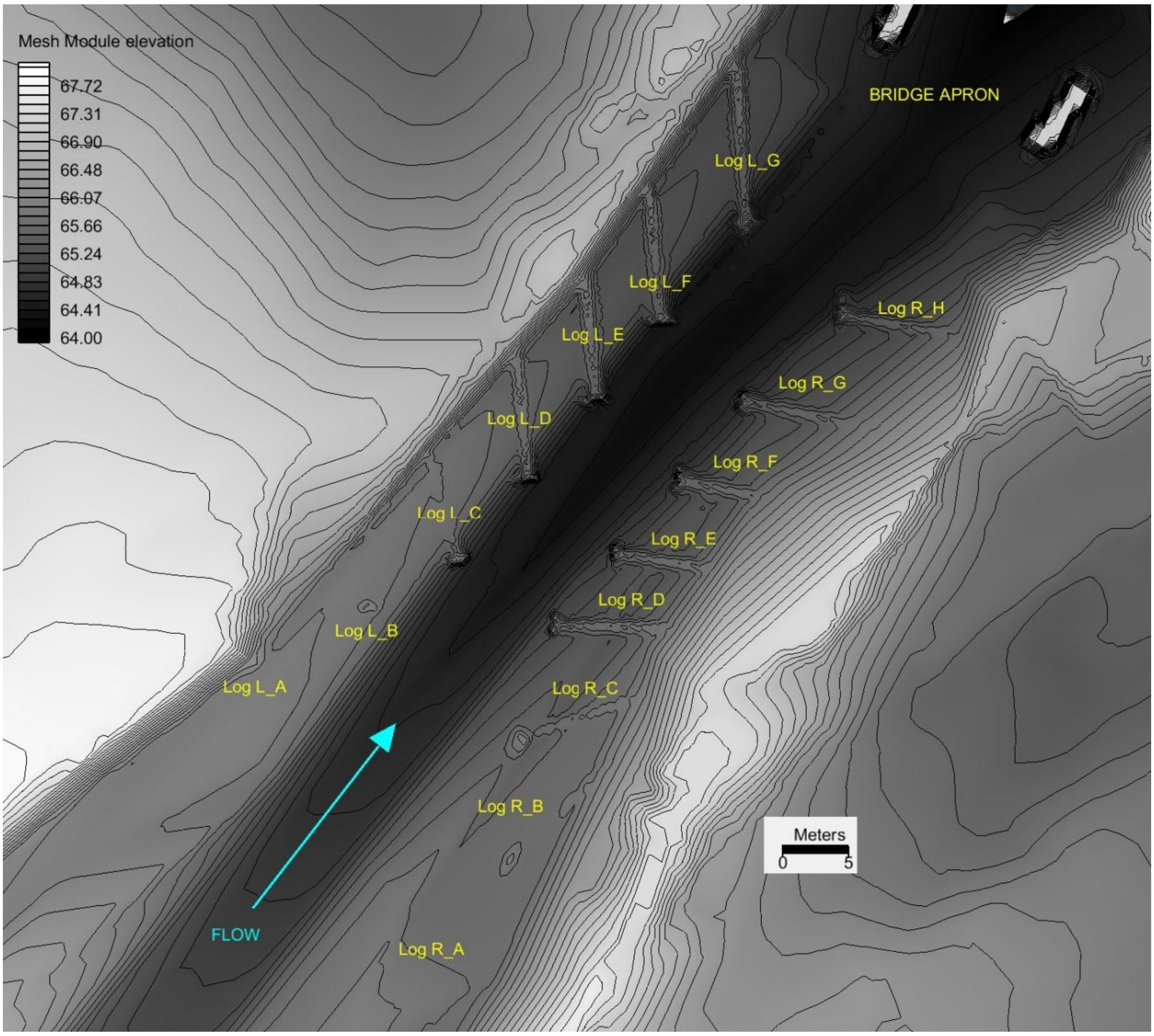


Figure C13 Plan view of design log pairs.

The right bank logs are labelled R_A to H and the left bank logs L_ A to G. Only logs C and downstream are modelled in the spreadsheet as logs A to B are more fully submerged in the bed material. Locations of the log root ball and buried bank end are taken from the appropriate CAD drawings and are as modelled in the 2D hydraulic model.

To calculate drag forces on the semi-submerged logs an area consisting of the exposed height times exposed length is used, and the drag coefficient is estimated as that of a cylinder (0.8-1.2) a range of values published in Gippel et al., 1996.

For the root wads, no extra anchoring resistance is assumed.

For the ballasting weight of bed material a density of quartz gravel has been assumed (2650kg/m^3), and a friction angle of 40 degrees assumed (Brooks et al., 2006).

The formulae used to determine the buoyant forces and drag forces on each log are summarised in Figure C14. The computations of drag force require an estimate of the 'freestream' depth average velocity in front of each log and this has been estimated from the 2D hydraulic model output at the Q200 year peak (shown on the spreadsheet). Note though that the drag force varies with the square of velocity. Bed shear stress on each log was estimated by the 2D hydraulic model at the Q200 year peak. Pressure drag (from the drag coefficient) dominates the drag forces.

Description	Symbol or formula	Unit
Factor of safety	F_s	-
Length of log	L	m
Diameter of log	D	m
Length of log exposed to flow	L_E	m
Height of log exposed to flow	H	m
Projected area of log exposed to flow	$S_E = HL_E$	m^2
Volume of log submerged in flow	$V_S = \pi(D/2)^2 L_E$	m^3
Density of water	ρ	3
Density of wood	ρ_{wood}	3
Depth average flow velocity	U	m/s
Drag coefficient	C_D	-
Angle of repose for gravel	ϕ	radians
Buoyancy force	$F_B = V_S \rho g \left(1 - \frac{\rho_{\text{wood}}}{\rho}\right)$	N
Ballast weight req. for buoyancy	$W_B = F_B F_s$	N
Pressure drag	$F_{Dp} = \frac{1}{2}\rho U^2 S_E C_D$	N
Shear force	$F_{Ds} = L_E D \tau$	N
Total drag force	$F_D = F_{Dp} + F_{Ds}$	N
Friction force	$F_f = (W_B - F_B)\tan\phi = F_D F_s$	N
Ballast weight req. for drag	$W_B = F_B + \frac{F_D F_s}{\tan\phi} - \pi(L - L_E)(D/2)^2 \rho_{\text{wood}} g$	N

Figure C14 Formulae used to determine drag and buoyant forces on logs.

The spreadsheet determines the amount of extra ballast/ anchoring load required for each of the logs. It is recommended that the maximum value for any of the logs is used on each log, given that the velocities over the logs are dependent on all of the logs being in place. For the sample dimensions and assumptions described above, the maximum required restraining force is 1.6tonnes (15.6kN).

The log structures should be maintained to ensure adequate integrity of ballast and or anchors, structural integrity of the logs, trapping of debris, deposition, scour and undercutting of the logs by flow.

COLLEGE BURN AT WESTNEWTON BRIDGE
VERTICAL LOADS ANALYSIS ON DESIGN TRAINING LOGS (NO MOMENTS/TORQUES CONSIDERED).
PREPARED BY CBEC UK LTD, APRIL 2014
LARGE FACTORS OF SAFETY SHOULD BE USED, AND BED EROSION/MAINTENANCE CONSIDERED.

ORANGE SHADING IS OUTPUT OF MAX BALLAST WEIGHT REQUIRED FOR EACH LOG.
GREEN SHADING INDICATES VARIABLES WHICH MAY BE ALTERED BY DESIGNER

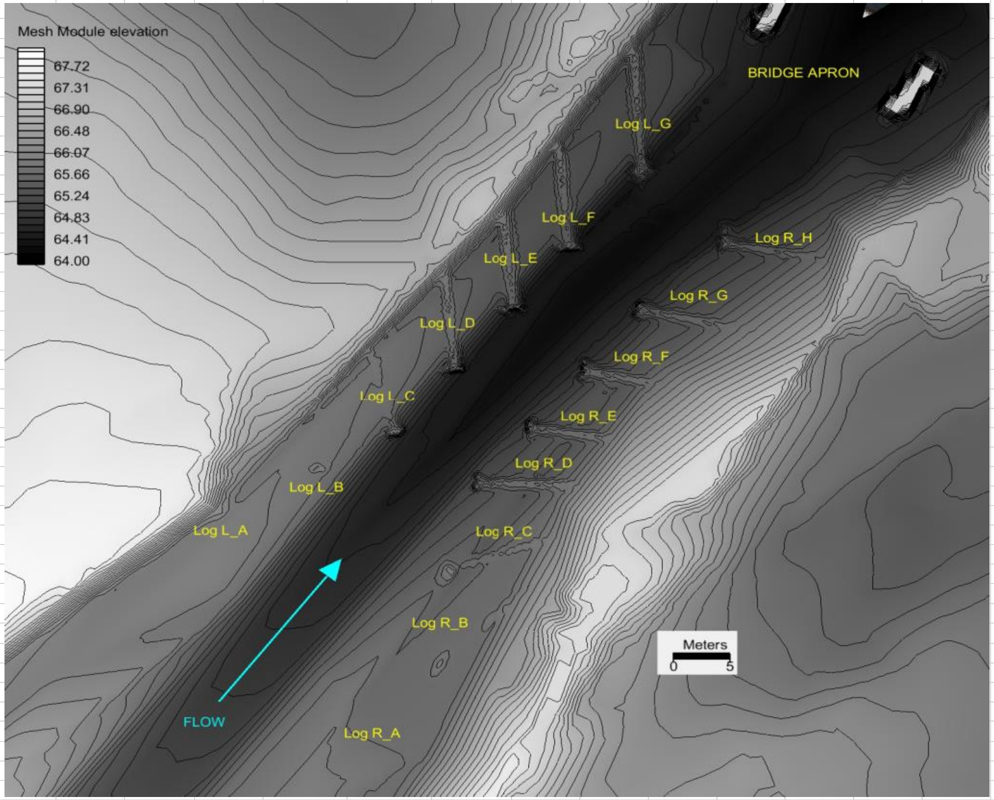
NOMENCLATURE & DEFINITIONS

GENERAL:		
FSD	1.6	FACTOR OF SAFETY FOR DRAG
FSB	2	FACTOR OF SAFETY FOR BOUYANCY
D	0.5	LOG DIAMETER (m) (DEPENDS ON LOGS AS USED IN CONSTRUCTION)
L	15	LOG LENGTH (m) (DEPENDS ON LOGS AS USED IN CONSTRUCTION)
H	0.4	HEIGHT OF LOG ABOVE SURFACE (m) (CONSERVATIVE DESIGN BUT BEFORE EROSION)
X(ROOT)		APPROX EASTING OF ROOT BALL (m)
X(BURIED)		APPROX EASTING OF POINT WHERE LOG BECOMES BURIED (m)
Y(ROOT)		APPROX NORTHING OF ROOT BALL (m)
Y(BURIED)		APPROX NORTHING OF POINT WHERE LOG BECOMES BURIED (m)
RHO_WOOD	900	DRY DENSITY OF LOGS (kg/m^3) [ASSUMED]
L_E		EXPOSED LENGTH (m)
S_E		AREA EXPOSED TO FLOW (PROJECTED) (m^2)
V_S		SUBMERGED VOLUME (m^3)
press		POST SCRIPT- PRESSURE RELATED FORCES
shear		POST SCRIPT- SHEAR RELATED FORCES
_D		POST SCRIPT- DRAG RELATED
_B		POST SCRIPT- BOUYANCY RELATED
FLUID MECHANICS:		
RHO	1000	DENSITY OF WATER (kg/m^3)
RE	2.20E+06	REYNOLDS NUMBER
Cd	0.8	CYLINDER DRAG COEFFICIENT [RANGE 1.2-0.8]
F_D		DRAG (N)
TAU		BED SHEAR (Pa)
F_B		BOUYANCY FORCE (N)
U		FLOW VELOCITY (m/s)
BED MATERIAL:		
WBL		BALLAST WEIGHT (kg)
PHI	40	FRICTION ANGLE COARSE GRAVEL (DEGREES) [D'AOUST AND MILLAR, 2000]

OTHER ASSUMPTIONS

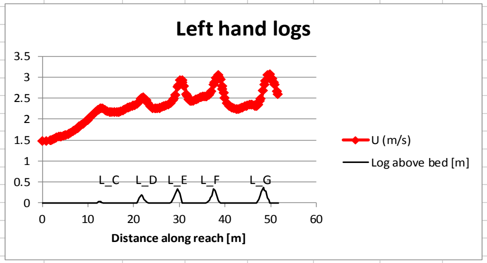
DESIGN FLOW IS Q200yr ESTIMATE	
H=0.4M	EXPOSED HEIGHT IS 0.4M MAX FOR DESIGN SCENARIO- THIS MAY INCREASE WITH EROSION
U	MAX DEPTH AVERAGED VELOCITY MAGNITUDE ON APPROACH TO EACH LOG (SEE FIGS)
TAU	MAX BED SHEAR OVER LOG SECTIONS (AT MIDPOINT) FROM 2D HYDRAULIC MODEL
_A,_B LOGS ARE SO WELL BURIED THAT NO LOAD COMPUTATIONS NECESSARY	

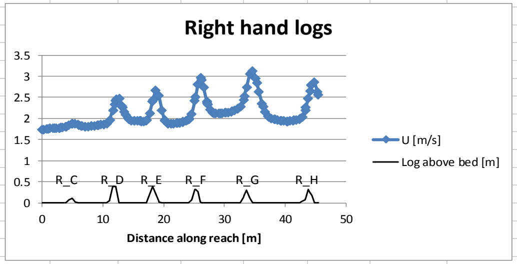
ORIENTATION PLOT FROM 2D HYDRAULIC MODEL



COMPUTATIONS					Exposed	Projected	Subm.			Total	*Pressure*	*Friction*	Total	Extra	Extra	**Ballast required**	
	LOG GEOMETRY FROM CAD MODEL/DEM				length	area	volume	**HYDRAULICS**		bouyancy (	*drag (N)*	*drag (N)*	drag (N)	ballast (N)	ballast (N)	**TONNES**	**(kg/m)**
LOG NAME	**X (ROOT)**	**Y(ROOT)**	**X(BURIED)**	**Y(BURIED)**	**L_E**	**S_E**	**V_S**	**U**	**TAU**	**F_B**	**F_D press**	**F_D shear**	**F_D**	**WBL_D**	**WBL_B**	**WBL**	**WBL/L**
L_C	390742.1	630345.1	390739.9	630352.9	8.08	3.23	1.58	1.90	64	1555	4665	258	4923	-1553	3110	**0.32**	**46**
L_D	390747.4	630351.4	390746.6	630360.5	9.16	3.66	1.80	2.20	78	1764	7094	357	7451	5167	3527	**0.53**	**90**
L_E	390752.6	630357.4	390751.8	630366.3	8.97	3.59	1.76	2.30	100	1727	7592	448	8040	5749	3454	**0.59**	**97**
L_F	390757.6	630363.4	390756.6	630373.2	9.91	3.96	1.94	2.50	110	1907	9908	545	10453	11972	3815	**1.22**	**240**
L_G	390764.0	630371.0	390763.3	630382.8	11.77	4.71	2.31	2.30	109	2267	9965	642	10607	15674	4533	**1.60**	**495**
R_C	390747.4	630331.3	390756.4	630334.4	9.47	3.79	1.86	1.80	42	1823	4908	199	5107	1593	3646	**0.37**	**67**
R_D	390749.8	630339.8	390758.7	630339.5	8.86	3.54	1.74	1.90	67	1705	5116	297	5413	812	3411	**0.35**	**57**
R_E	390754.5	630345.2	390761.7	630344.0	7.28	2.91	1.43	2.00	77	1402	4660	280	4940	-3095	2803	**0.29**	**37**
R_F	390759.3	630350.8	390765.9	630349.4	6.73	2.69	1.32	2.00	90	1296	4307	303	4610	-4830	2591	**0.26**	**32**
R_G	390764.3	630356.8	390773.1	630354.9	9.00	3.60	1.77	2.10	106	1732	6349	477	6826	3436	3465	**0.35**	**59**
R_H	390771.8	630363.4	390780.5	630361.8	8.87	3.55	1.74	1.90	93	1708	5125	413	5538	860	3417	**0.35**	**57**

FIGURES: HEIGHT ABOVE BED OF PROTRUDING LOG (BLACK LINES); VELOCITY ALONG A LINE CONNECTING LOG MID-POINTS) RED (LEFT) AND BLUE (RIGHT) LINES.



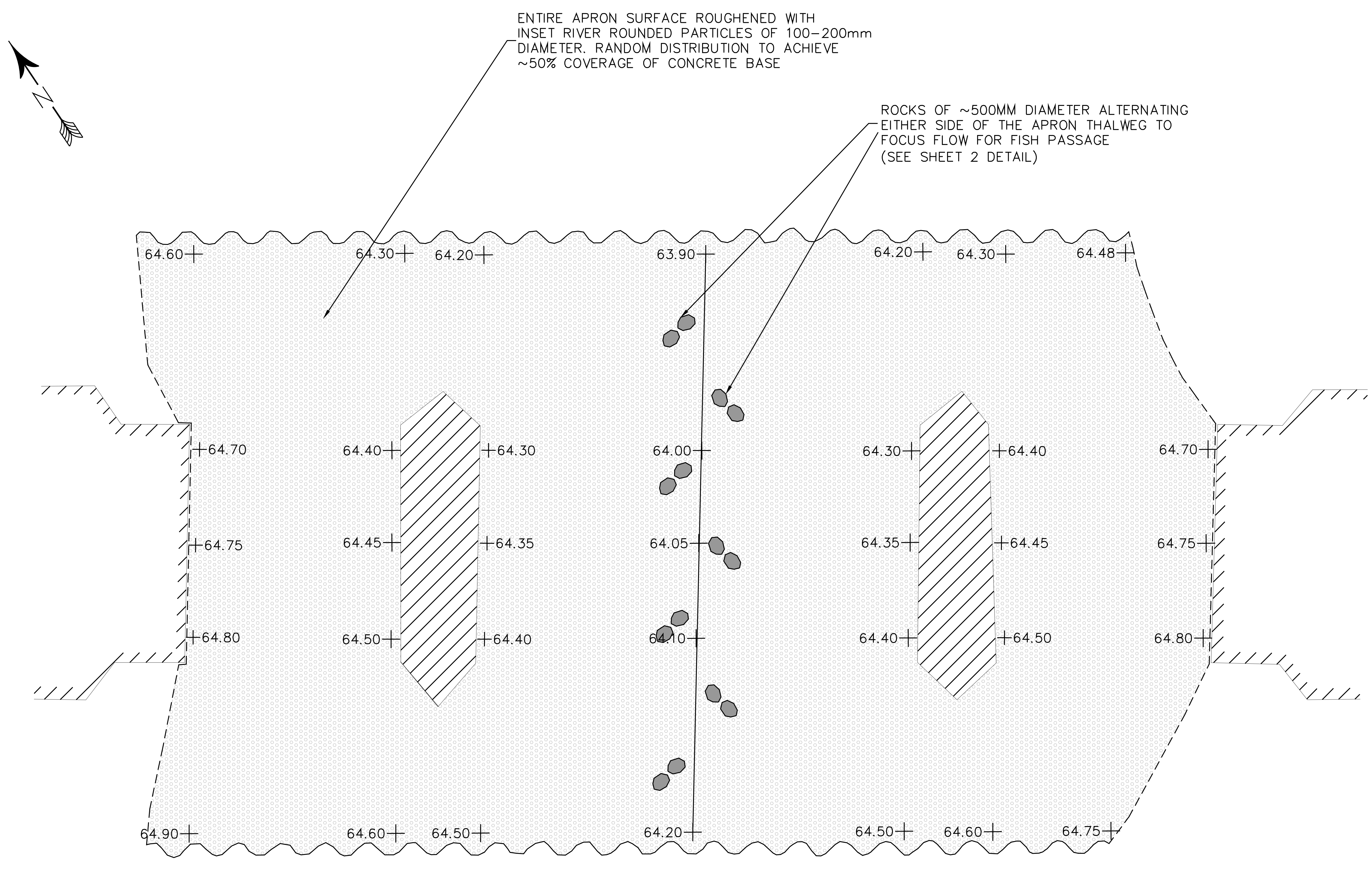


FORMULAE USED (SEE SIMON ET AL, STREAM RESTORATION IN DYNAMIC FLUVIAL SYSTEMS, AGU, 2011)

Description	Symbol or formula	Unit
Factor of safety	F_s	-
Length of log	L	m
Diameter of log	D	m
Length of log exposed to flow	L_E	m
Height of log exposed to flow	H	m
Projected area of log exposed to flow	$S_E = HL_E$	m^2
Volume of log submerged in flow	$V_S = \pi(D/2)^2 L_E$	m^3
Density of water	ρ	3
Density of wood	ρ_{wood}	3
Depth average flow velocity	U	m/s
Drag coefficient	C_D	-
Angle of repose for gravel	ϕ	radians
Buoyancy force	$F_B = V_S \rho g \left(1 - \frac{\rho_{\mathrm{wood}}}{\rho}\right)$	N
Ballast weight req. for buoyancy	$W_B = F_B F_s$	N
Pressure drag	$F_{D_p} = \frac{1}{2}\rho U^2 S_E C_D$	N
Shear force	$F_{D_s} = L_E D \tau$	N
Total drag force	$F_D = F_{D_p} + F_{D_s}$	N
Friction force	$F_f = (W_B - F_B)\tan\phi = F_D F_s$	N
Ballast weight req. for drag	$W_B = F_B + \frac{F_D F_s}{\tan\phi} - \pi(L - L_E)(D/2)^2 \rho_{\mathrm{wood}} g$	N

APPENDIX D

DESIGN DRAWINGS



NOTE: ALL ELEVATIONS ARE ABOVE ORDNANCE DATUM

NOTE: ONLY THE ELEVATIONS AND SURFACE MATERIALS OF THE APRON DIFFER FROM NORTHUMBERLAND COUNTY COUNCIL DRAWING HB127276/B/B6351/0622 WEST NEWTON BRIDGE GA OPTION 1 (DATED 26/06/13). FOR ALL OTHER ELEMENTS REFER TO THE NCC DRAWING

0 1m 2m 4m

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NORTHUMBERLAND
COUNTY COUNCIL

WEST NEWTON
BRIDGE

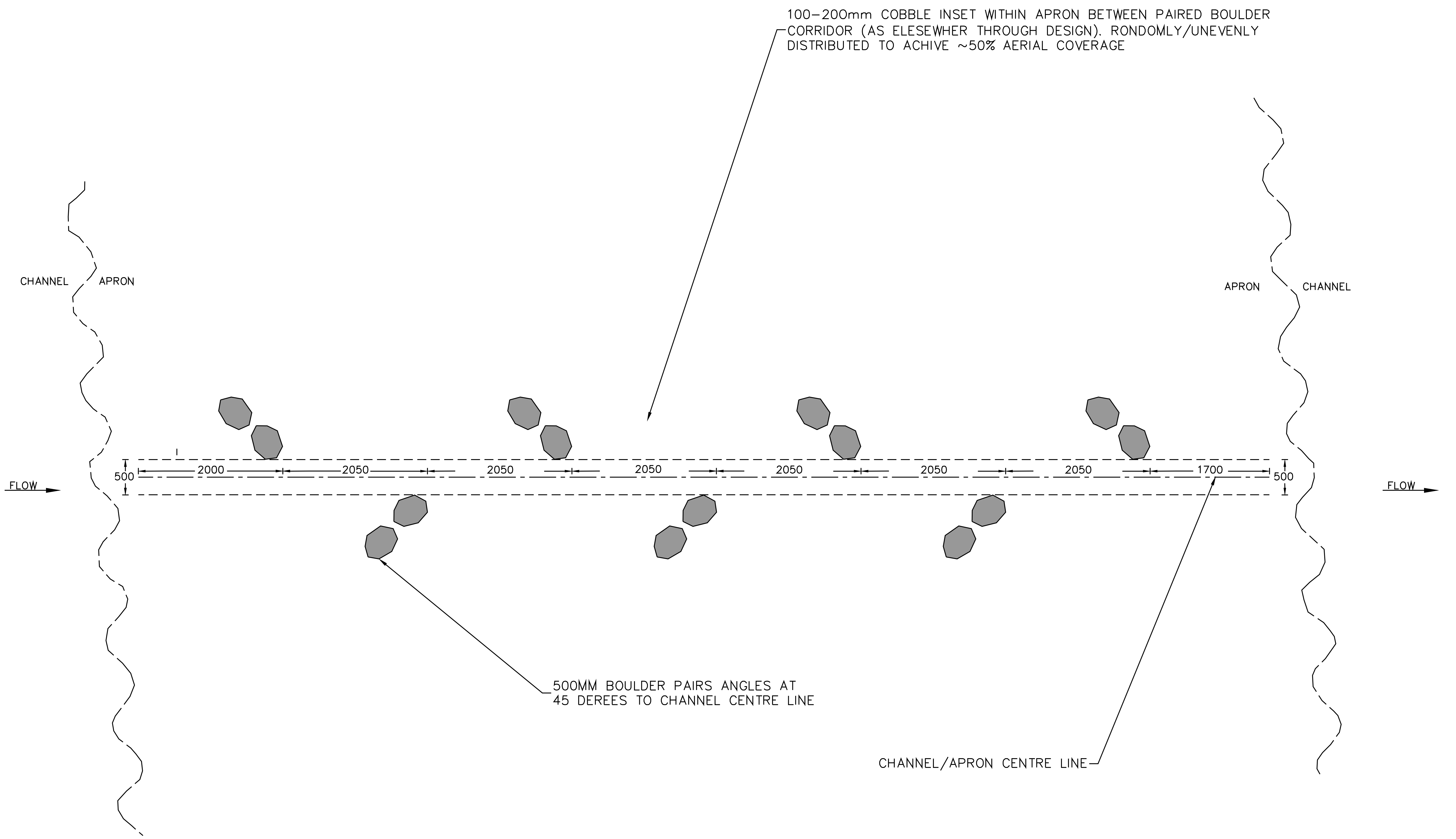
REVISED APRON PLANVIEW

FILE NAME
WNB_APRON

JOB NUMBER
U13-1003

DATE
MAY 2014

SHEET NO.
1 OF 2



NOTE: ALL DIMENSIONS ARE IN MILLIMETERS

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WEST NEWTON BRIDGE

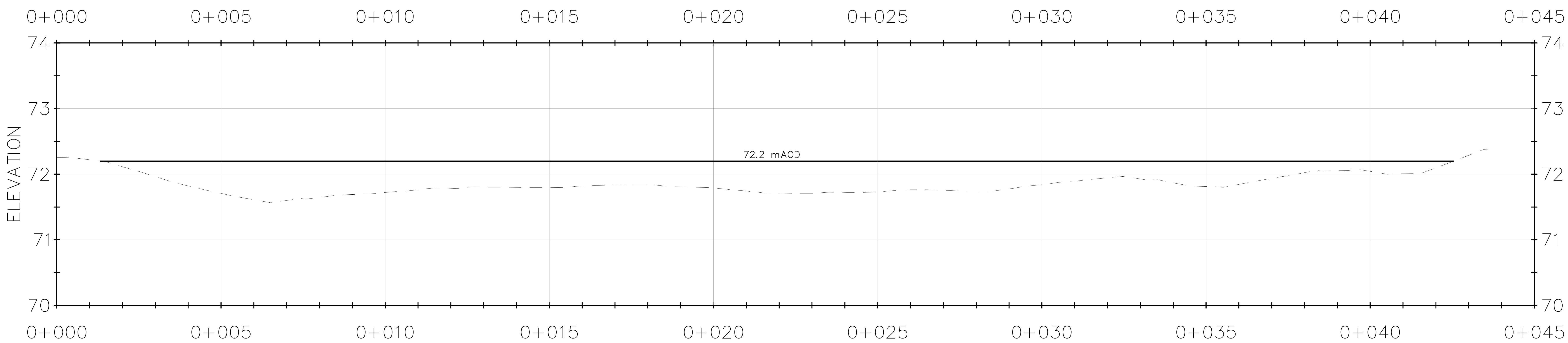
APRON DETAIL

FILE NAME WNB_APRON
JOB NUMBER U13–1003
DATE MAY 2014
SHEET NO. 2 OF 2

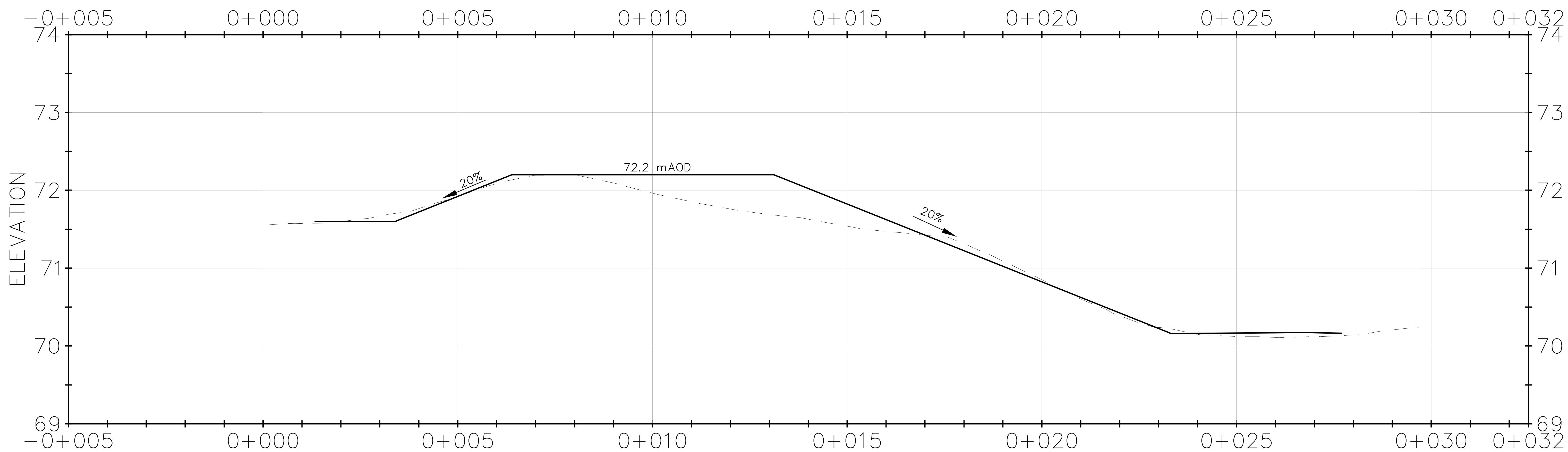
A B C D E

1 2 3 4

EMBANKMENT PROFILE A–A



EMBANKMENT TRACK PROFILE B–B



NOTE: ALL SLOPING FACES OTHER THAN VEHICLE ACCESS TRACKS TO BE GRADED TO 33.33%

NOTE: EMBANKMENT DESIGN IS INDIACTIVE AND CAN BE ALTERED DEPENDING ON AVAILABLE MATERIALS. THE CRITICAL FACTORS ARE THAT VEHICLE ACCESS IS POSSIBLE AND THAT THE THE UPPER SURFACE OF THE EMBANKMENT IS AT 72.2 m AOD

VERTICAL SCALE 0 0.5m 1m 2m

HORIZONTAL SCALE 0 1m 2m 4m

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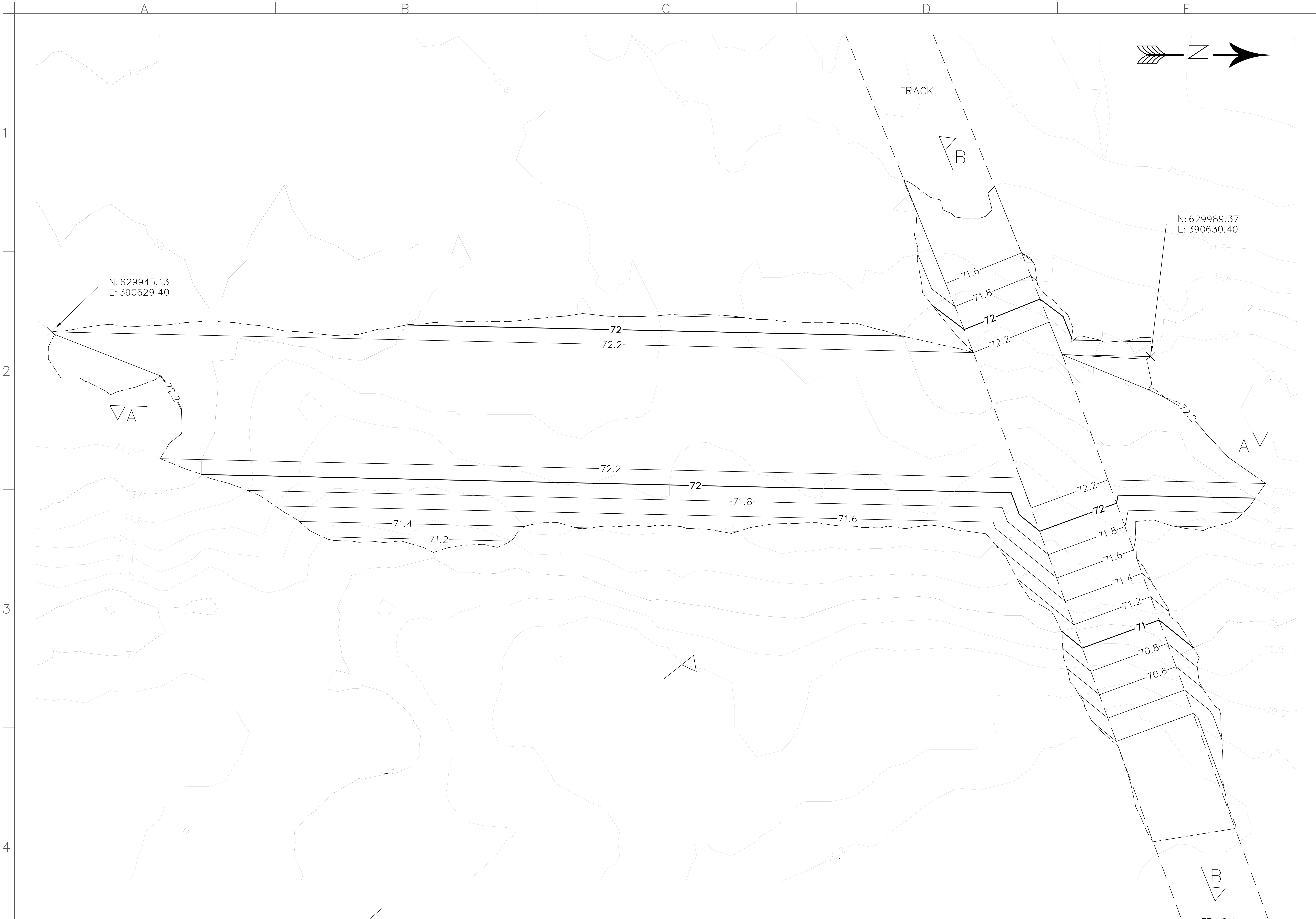
WEST NEWTON BRIDGE
EMBANKMENT SECTIONS

FILE NAME	WNB_embank
JOB NUMBER	U13-1003
DATE	APRIL 2014
SHEET NO.	2 OF 2



NOTE: EMBANKMENT DESIGN IS INDIACTIVE AND CAN BE ALTERED DEPENDING ON AVAILABLE MATERIALS.
THE CRITICAL FACTORS ARE THAT VEHICLE ACCESS IS POSSIBLE AND THAT THE THE UPPER SURFACE
OF THE EMBANKMENT IS AT 72.2 m AOD

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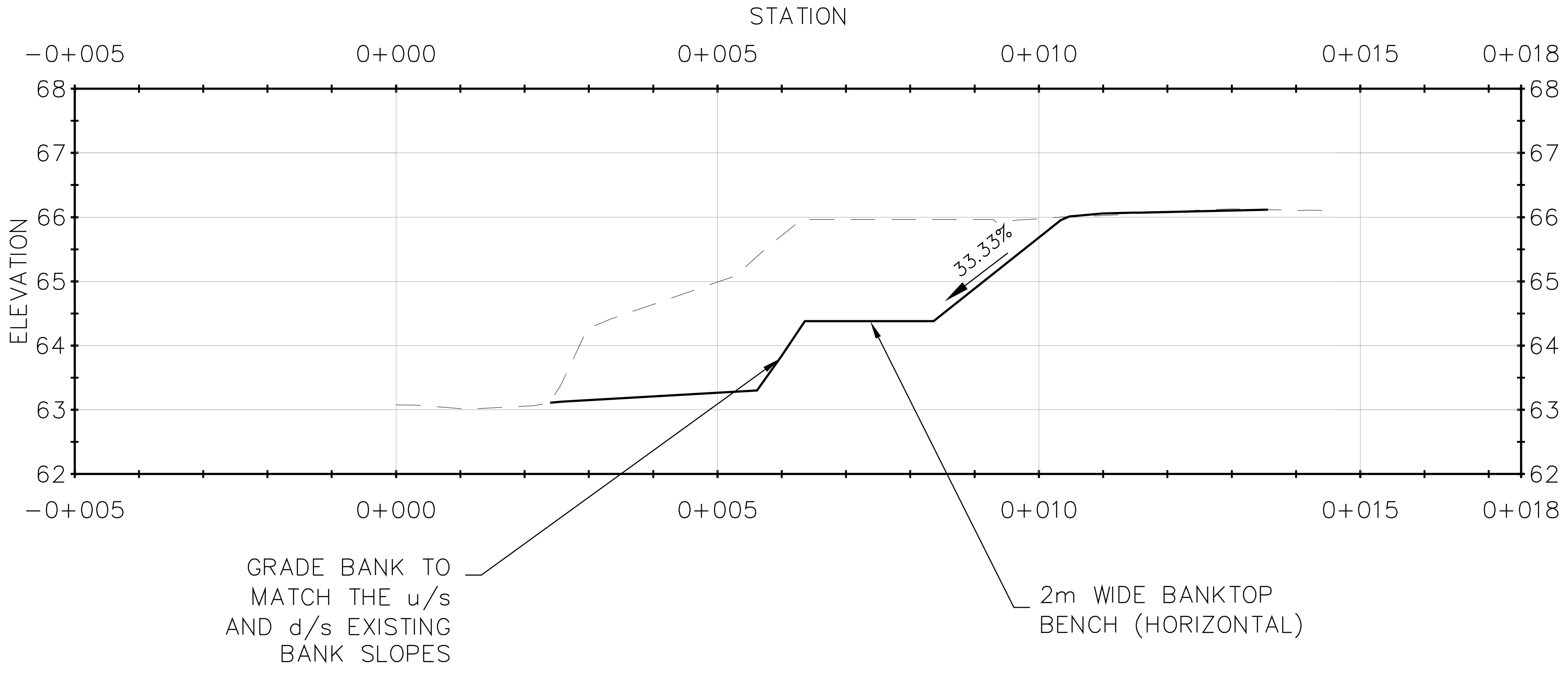
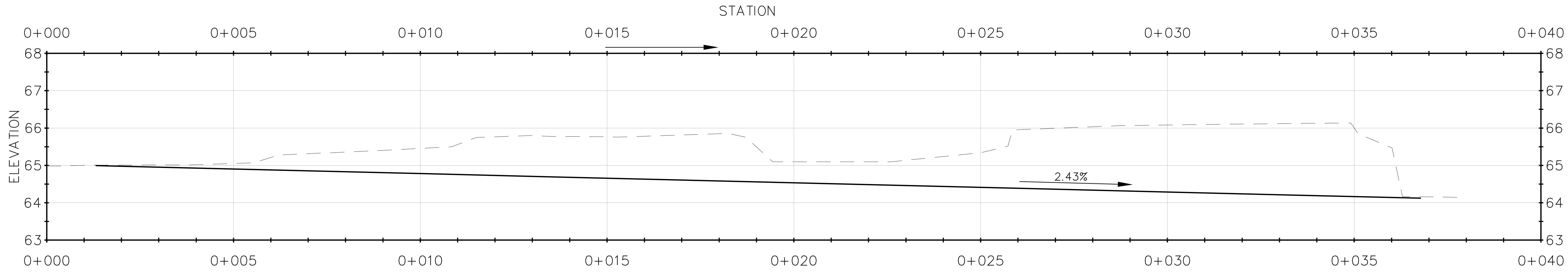
WEST NEWTON BRIDGE
EMBANKMENT PLANVIEW

FILE NAME	WNB_embank
JOB NUMBER	U13-1003
DATE	APRIL 2014
SHEET NO.	1 OF 2

ABUTMENT SETBACK SECTION A–A



PROFILE B –B ON BANKFULL BENCH



HORIZONTAL and VERTICAL SCALE

0 1m 2m 4m

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WEST NEWTON BRIDGE

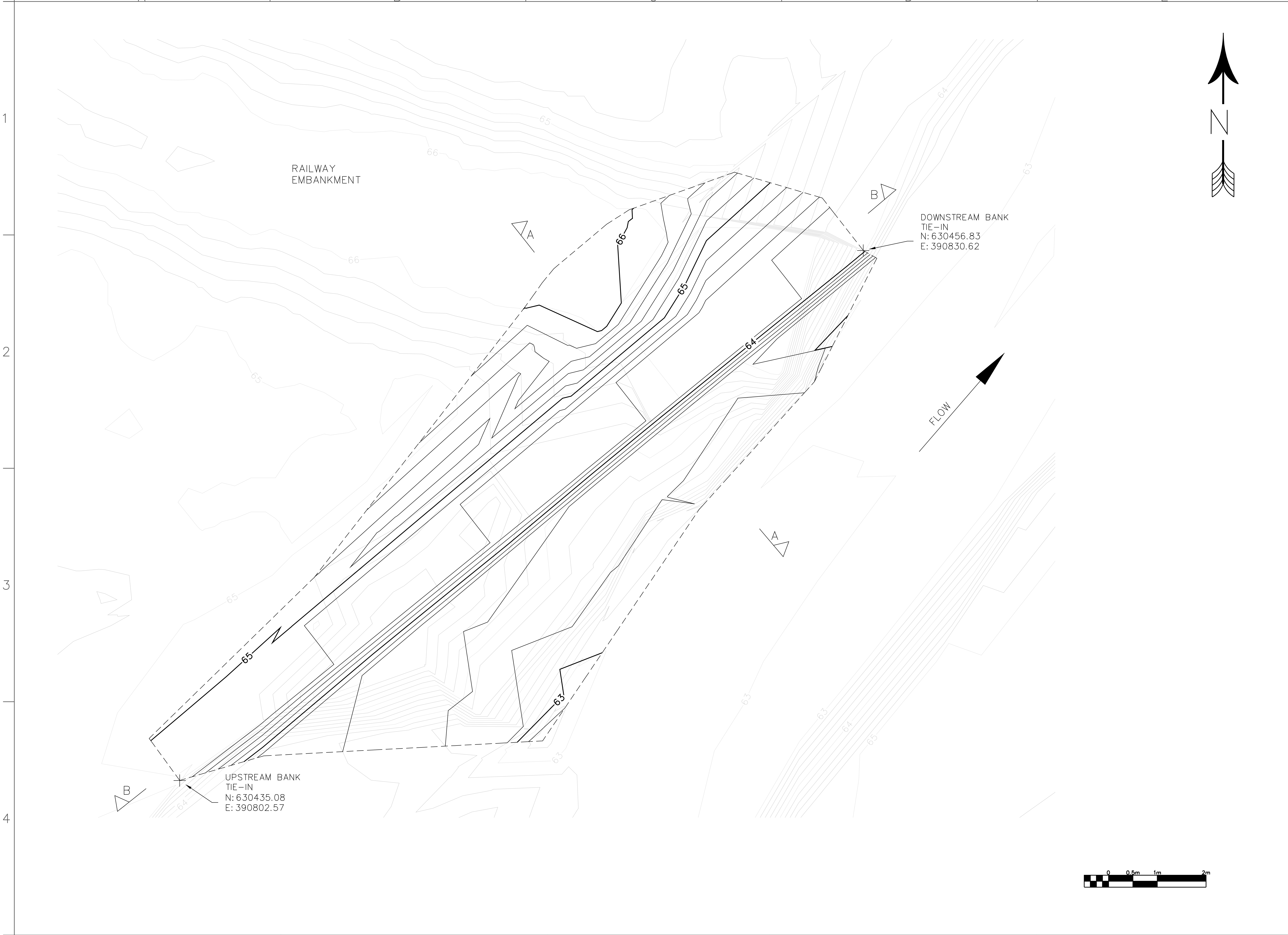
ABUTMENT SECTIONS

FILE NAME WNB_abutment

JOB NUMBER U13–1003

DATE APRIL 2014

SHEET NO. 2 OF 2



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ABUTMENT PLANVIEW

FILE NAME
WNB_abutment

JOB NUMBER
U13-1003

DATE
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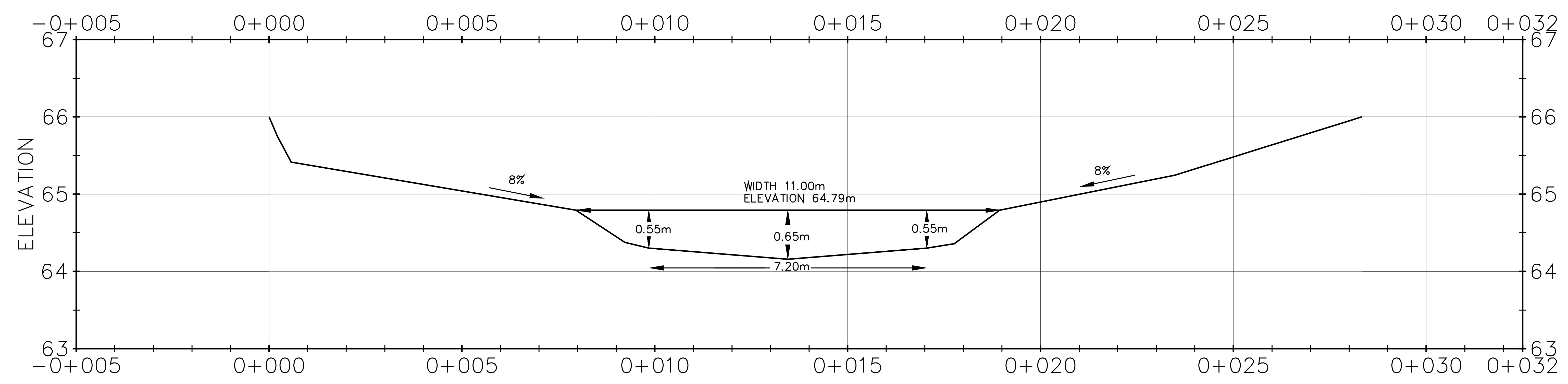
CHANNEL REINSTATEMENT SECTION G – G



NOTE: ORIENTATION OF SECTIONS G-G ONLINE WITH THE STEM BASE POINTS OF LOGS L7 & R8

NOTE: INSET CHANNEL BANK TOP ELEVATION EQUAL TO THE RESPECTIVE LOG BASE STEM ELEVATION MINUS 500mm (ASSUMING STEM BASE DIAMETER OF 1000mm)

VERTICAL SCALE 0 0.5m 1m 2m

HORIZONTAL SCALE 0 1m 2m 4m

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SECTION G

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JOB NUMBER U13-1003
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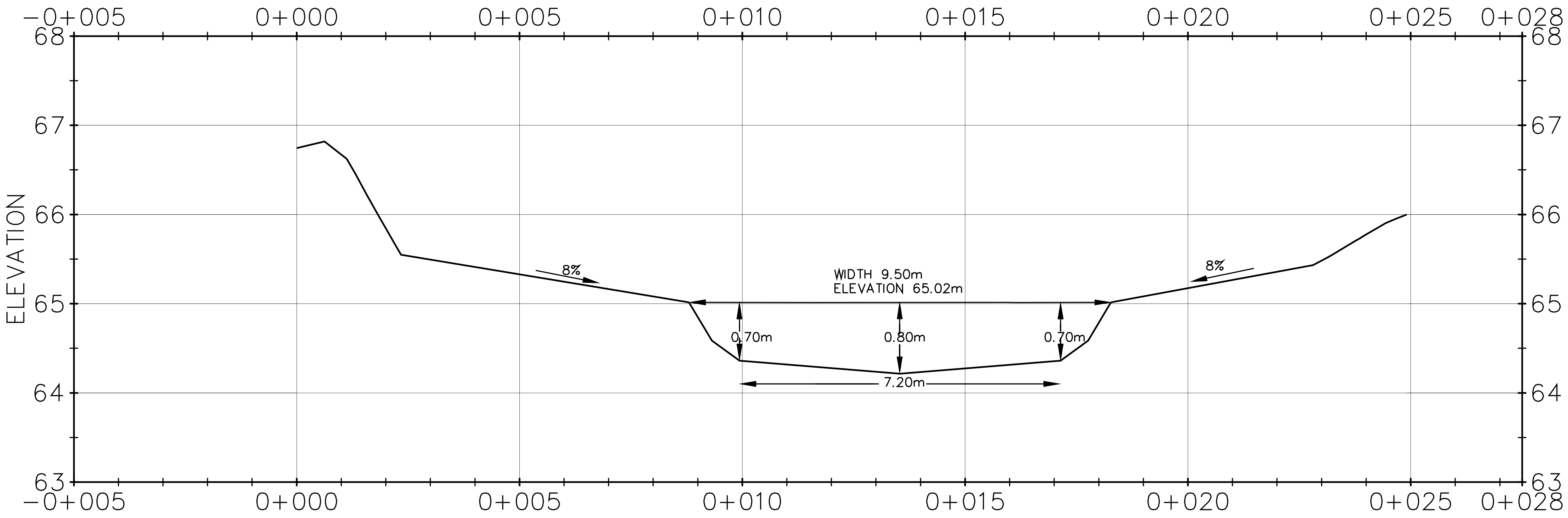
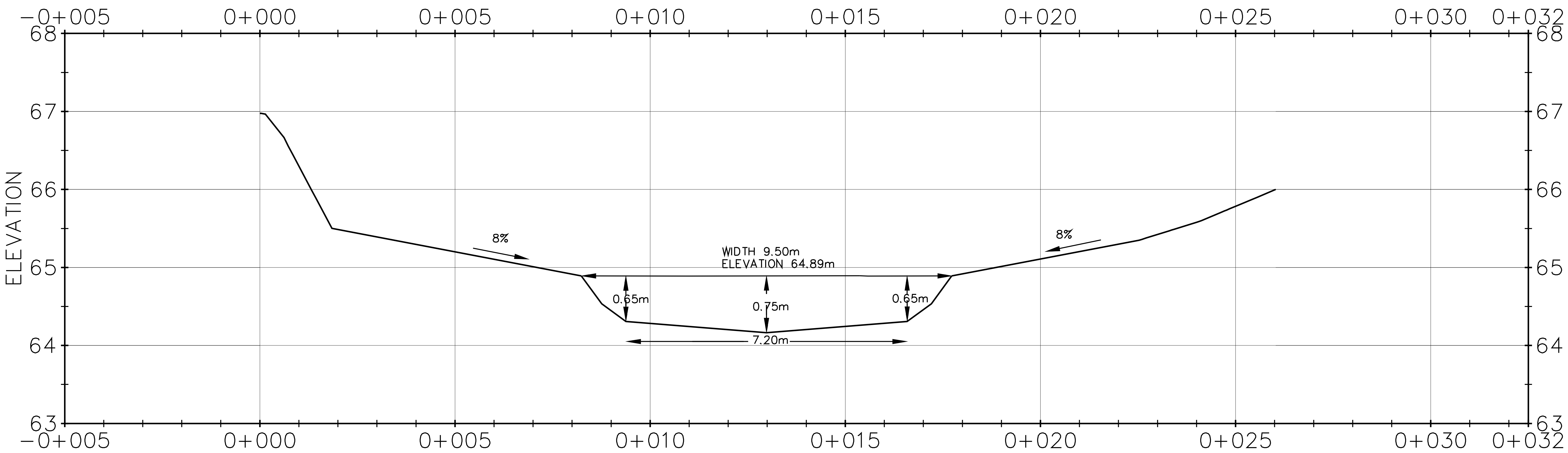
CHANNEL REINSTATEMENT SECTION E – E



CHANNEL REINSTATEMENT SECTION F – F



NOTE: ORIENTATION OF SECTIONS A–A AND B–B ONLINE WITH THE STEM BASE POINTS OF LOGS L1 & R2, AND L2 & R3, RESPECTIVELY

NOTE: INSET CHANNEL BANK TOP ELEVATION EQUAL TO THE RESPECTIVE LOG BASE STEM ELEVATION MINUS 500mm (ASSUMING STEM BASE DIAMETER OF 1000mm)

VERTICAL SCALE 0 0.5m 1m 2m

HORIZONTAL SCALE 0 1m 2m 4m

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SECTIONS E & F

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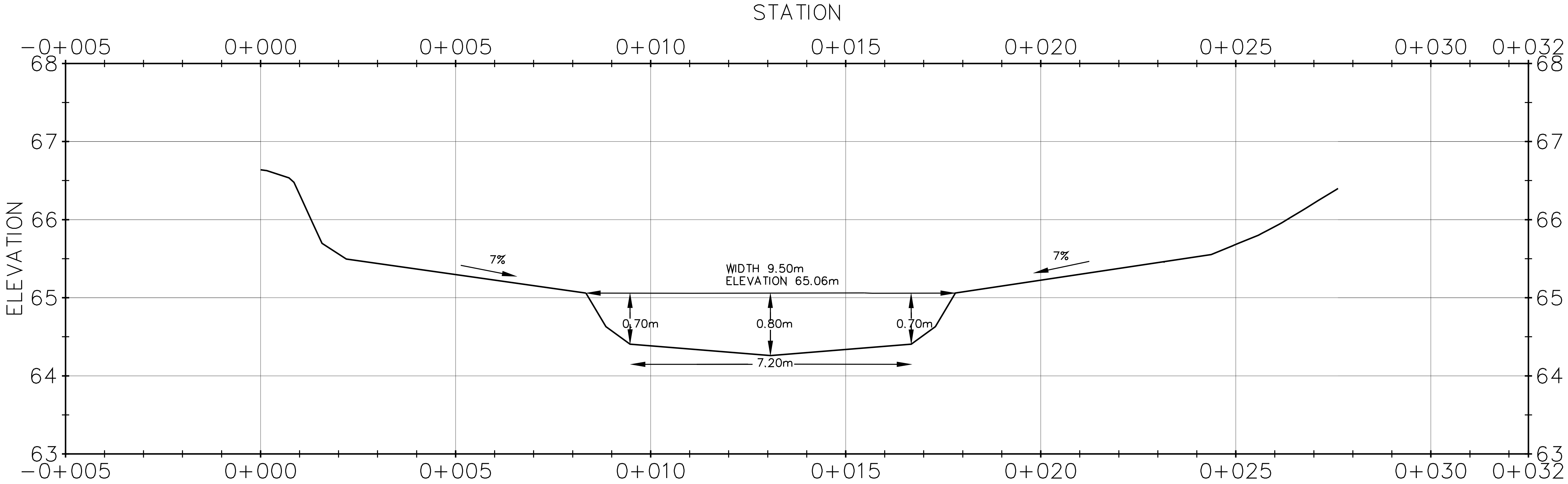
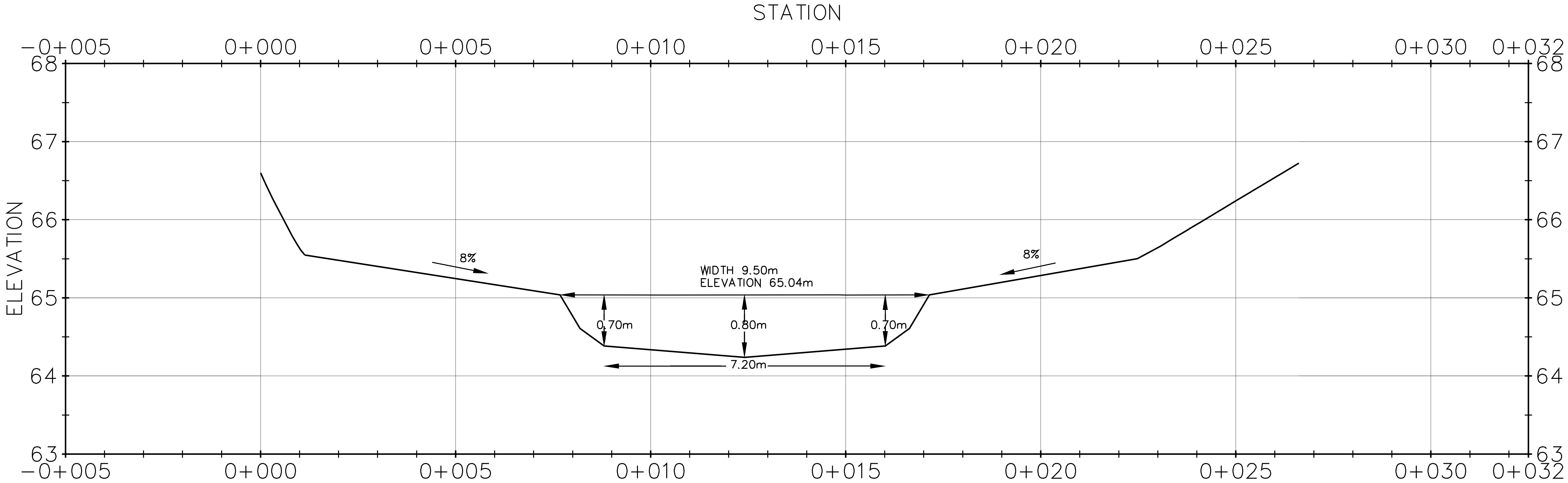
CHANNEL REINSTATEMENT SECTION C – C



CHANNEL REINSTATEMENT SECTION D – D



NOTE: ORIENTATION OF SECTIONS A–A AND B–B ONLINE WITH THE STEM BASE POINTS OF LOGS L3 & R4, AND L4 & R5, RESPECTIVELY

NOTE: INSET CHANNEL BANK TOP ELEVATION EQUAL TO THE RESPECTIVE LOG BASE STEM ELEVATION MINUS 500mm (ASSUMING STEM BASE DIAMETER OF 1000mm)

VERTICAL SCALE 0 0.5m 1m 2m

HORIZONTAL SCALE 0 1m 2m 4m

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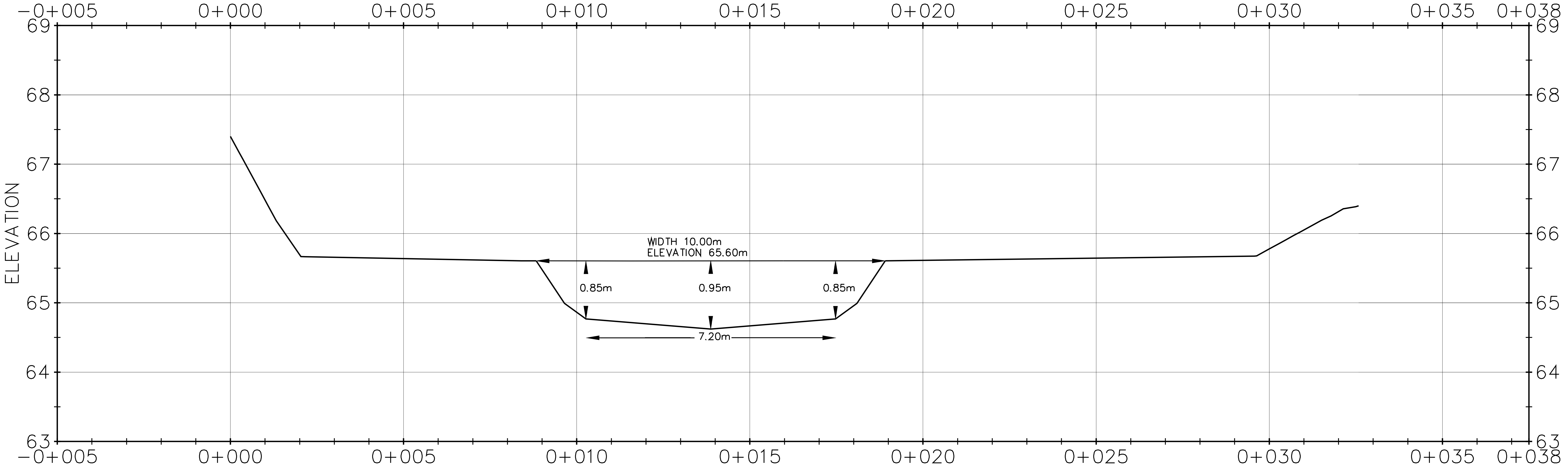
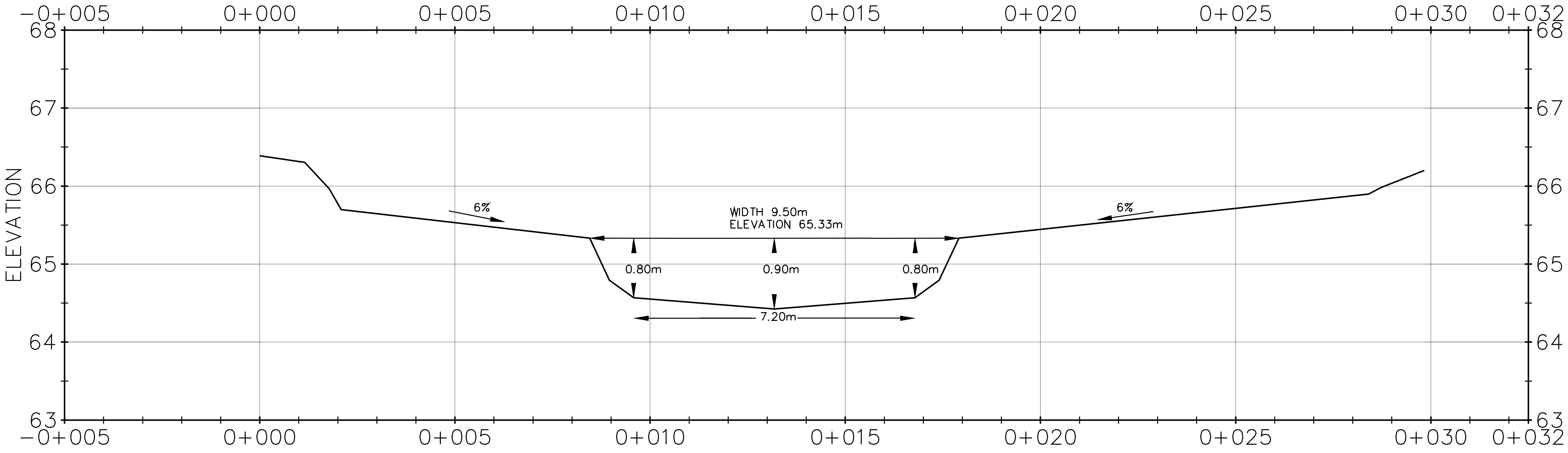
CHANNEL REINSTATEMENT SECTION A – A



CHANNEL REINSTATEMENT SECTION B – B



NOTE: GRADE SECTION A–A INSET FLOODPLAIN SLOPE TO MATCH UPSTREAM CONDITIONS

NOTE: ORIENTATION OF SECTIONS A–A AND B–B ONLINE WITH THE STEM BASE POINTS OF LOGS L1 & R2, AND L2 & R3, RESPECTIVELY

NOTE: INSET CHANNEL BANK TOP ELEVATION EQUAL TO THE RESPECTIVE LOG BASE STEM ELEVATION MINUS 500mm (ASSUMING STEM BASE DIAMETER OF 1000mm)

VERTICAL SCALE 0 0.5m 1m 2m

HORIZONTAL SCALE 0 1m 2m 4m

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SECTIONS A & B

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JOB NUMBER: U13–1003

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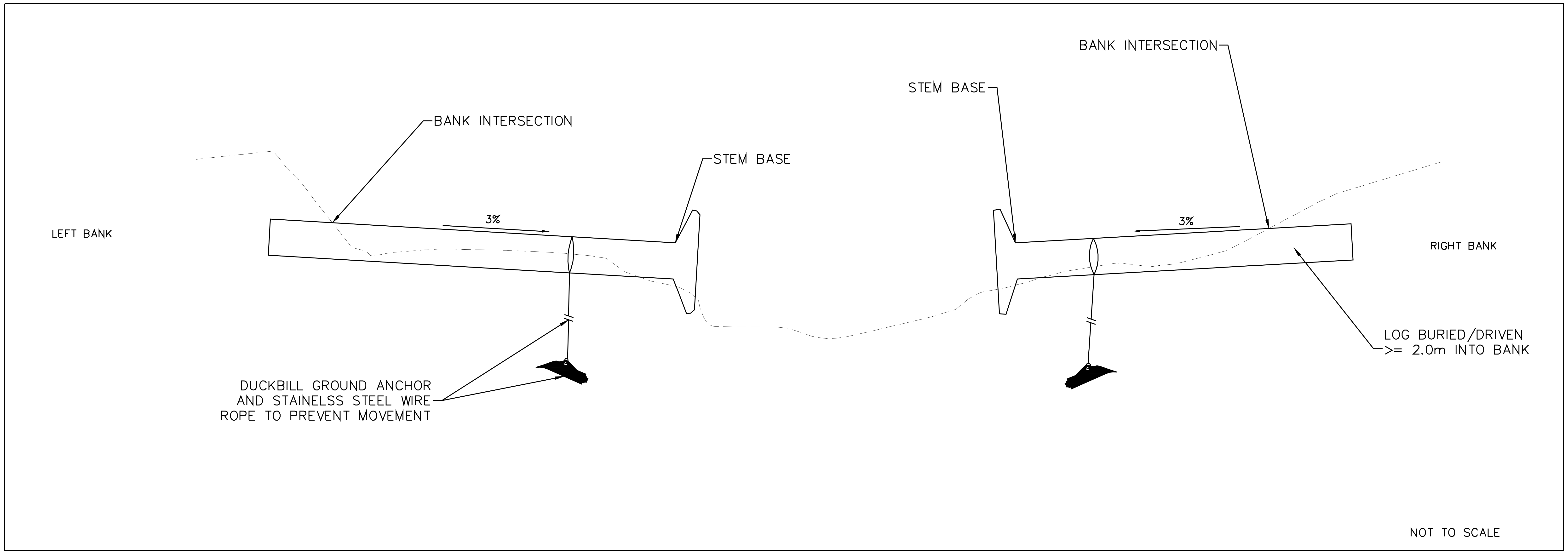
EXAMPLE OF TYPICAL INSTALLATION OF TRAINING LOGS



STEM BASE AND BANK INTERSECTION CO-ORDINATES AND ELEVATIONS

Log Id	Point Location	Easting	Northing	Elevation
LOG L1	STEM BASE	390726.74	630337.28	65.61
	BANK	AS ABOVE	AS ABOVE	AS ABOVE
LOG L2	STEM BASE	390735.25	630341.68	65.58
	BANK	390735.17	630347.15	65.75
LOG L3	STEM BASE	390742.05	630345.11	65.56
	BANK	390739.85	630352.88	65.80
LOG L4	STEM BASE	390747.43	630351.36	65.54
	BANK	390746.57	630360.48	65.81
LOG L5	STEM BASE	390752.64	630357.36	65.52
	BANK	390751.78	630366.29	65.78
LOG L6	STEM BASE	390757.57	630363.37	65.39
	BANK	390756.63	630373.24	65.69
LOG L7	STEM BASE	390764.05	630371.03	65.29
	BANK	390763.34	630382.78	65.64

Log Id	Point Locatio	Easting	Northing	Elevation
LOG R1	STEM BASE	390744.52	630312.55	65.63
	BANK	390746.18	630314.40	65.71
LOG R2	STEM BASE	390746.20	630322.67	65.61
	BANK	390753.82	630329.15	65.90
LOG R3	STEM BASE	390747.44	630331.33	65.58
	BANK	390756.39	630334.42	65.86
LOG R4	STEM BASE	390749.80	630339.80	65.56
	BANK	390758.65	630339.52	65.82
LOG R5	STEM BASE	390754.48	630345.16	65.54
	BANK	390761.67	630344.06	65.75
LOG R6	STEM BASE	390759.33	630350.77	65.52
	BANK	390765.91	630349.37	65.72
LOG R7	STEM BASE	390764.26	630356.79	65.39
	BANK	390773.07	630354.93	65.66
LOG R8	STEM BASE	390771.81	630363.39	65.29
	BANK	390780.54	630361.80	65.56

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LOG PLACEMENT DETAIL

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JOB NUMBER U13-1003
DATE APRIL 2014
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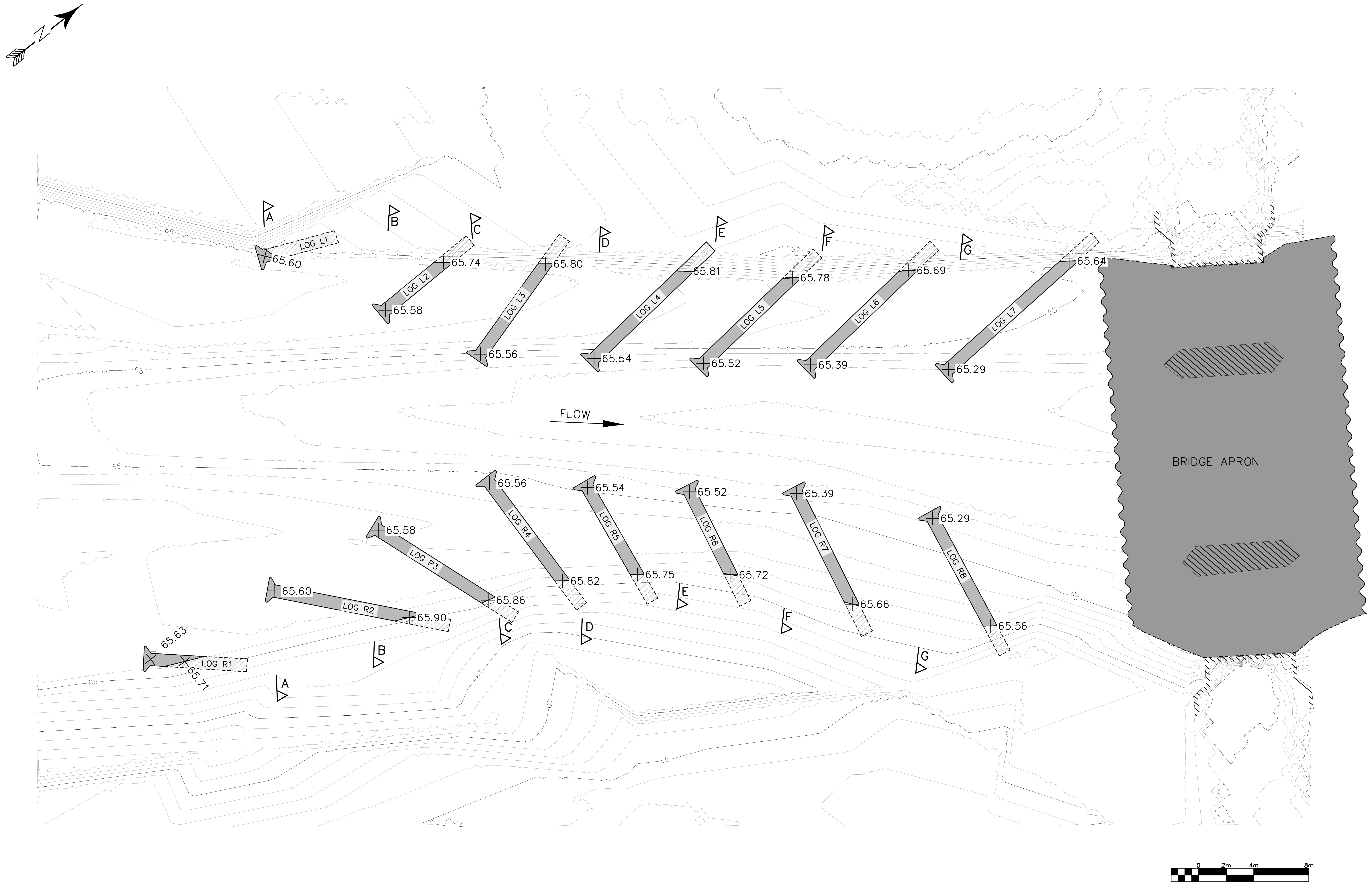

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Company Number: SC385000 | VAT Number: GB111101405