Environment Agency

Newlay Weir Investigation

Independent review into the failure of Newlay Weir in January 2021

Final Issue

Job number 274242-26

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Glossary

Annual Exceedance Probability	The chance or probability of a flood occurring annually, expressed as a percentage.
Bathymetry	Elevation of the surface level below the water line.
Bulk bag	A large bulk storage bag often made of lightweight flexible polypropylene canvas and filled with granular material. Bulk bags are used to provide basic temporary flood protection.
Catchment	An area which drains water to a river.
Masonry stone setts	Stone that is individually carved by a mason
Cofferdam	Watertight enclosure from which water is pumped to expose the bed of a body of water in order to permit construction. Cofferdams are made by driving sheet piling, usually steel, into the bed to form a watertight fence.
Contractor	Suttle Projects Ltd was the appointed contractor commissioned to deliver the design and construction of the Newlay Weir Fish Pass.
Datum	A fixed starting point on a scale or gauge.
Downstream	Looking in the natural river flow direction.
Flood event	A real-life flood incident; or A theoretical flood event commonly described as a probability of occurrence.

Gauge	In this context, an instrument that measures and gives a display of river level, depth, velocity or flow.
Goit	A small artificial channel carrying water. Usually used with respect to channels built to feed mills. Also known as a Leat
Hydraulic model	A hydraulic model uses mathematical equations to approximate the flow of water. A representation of the study area is generated using topographical information (for example ground elevations, building locations, land classification such as fields, roads, gardens) together with hydrology data (such as river flows, both calculated and measured, rainfall data). The mathematical equations are typically built into industry standard software. The software then calculates flood extents/areas, water depth, water velocity and direction of flows.
Hydrology	The science of water and the movement of water on land.
Left bank	The bank on the left-hand side of a river when looking in a downstream direction.
LiDAR	Light Detection and Ranging, a method for measuring distance and a common method of surveying topography.
RAMS	Risk Assessment and Method Statement. A document that sets out the methodology for carrying out construction work in a safe manner.
Return period	The frequency of a given flood event.
Right bank	The bank on the right-hand side of a river when looking in a downstream direction.

Sand bag	A fabric bag filled with sand and used to build temporary walls, dams, etc.
Sheet pile	Sheet piles are sections of sheet materials with interlocking edges that are driven into the ground to provide retention and excavation support. Sheet piles are most commonly made of steel.
Temporary works	Part of construction works which are required to enable permanent works to be built.
Thalweg	A line drawn to join the lowest points along the length of a river bed (long profile), defining its deepest channel. The thalweg marks the natural direction (profile) of a water course and its dominant flow path.
Topography	The natural and artificial features of land/area.
Topographical survey	A survey which captures key topographical features (such as roads, footpaths, buildings) to create a detailed map of an area with height information.
Upstream	The opposite of the natural flow direction in a river.
Weir	A low structure built across a river to raise the level of water upstream or regulate its flow.
1D-2D	1D relates to a one-dimensional hydraulic model, which assumes that velocity and depth only change in one dimension (upstream or downstream from a cross section). 2D relates to a model that calculates velocity and depth in two dimensions (but assumes that velocity doesn’t vary with depth). 1D-2D relates a model that includes both these modelling methodologies.

Acronyms

AEP	Annual Exceedance Probability
AOD	Above Ordnance Datum
DTM	Digital Terrain Model
FAS	Flood Alleviation Scheme
LiDAR	Light Detection and Ranging

Executive summary

Newlay Weir is a historic structure on the Lower River Aire, constructed in 1690. In June 2020, work started to construct a fish pass adjacent to the weir. The design included temporary works, consisting of a sheet piled cofferdam installed through the weir to create a dry working area.

During the construction, following a storm in January 2021, the weir failed and a large portion of it was washed downstream. Arup was commissioned to investigate the factors that could have contributed to the collapse.

The findings are summarised as follows:

- The weir was in Fair condition.
- The storm in January 2021 was a significant event, but the resulting high river flows had been exceeded 8 times over the past 10 years.
- There were no substantive changes in the bed levels observed in the analysis of the topographic survey between 2015 and 2018. It is therefore unlikely that active erosion process, occurring during this time period and the subsequent years leading up to construction, were a contributory factor.
- Hydraulic modelling of the river showed that the works in the river channel would have increased depths and velocity slightly, but not to a significant extent such that it would be sufficient to cause failure on its own.
- The sheet piles were angled across the face of the weir and this was observed to direct flow across the face of the weir and create turbulent flow patterns. This could have caused localised erosion to the bed in front of the weir and potentially contributed to the failure.
- The sheet piles pushed the main flow and velocity towards the centre of the weir and potentially contributed to erosion in front of the weir.
- There were four periods of high flow following the installation of the sheet piled cofferdam, prior to Storm Christoph. These events would have caused similar flow conditions and are likely to have caused ongoing erosion.
- The cofferdam exceeded the levels and extents set out in the Environmental Permit. This would have increased the impact of the high flow events.
- The sheet piles were vibrated into position. The investigation has found that pile installation represents a low risk of damage to the weir.

The report concludes that the temporary works are likely to have contributed to the collapse, through changing the flow patterns over the weir during Storm Christoph and the preceding high flow events, which may have resulted in localised erosion to the riverbed in front of the weir. This could lead to the progressive collapse mechanism described in the report.

1 Introduction and Scope

1.1 Introduction

The River Aire is part of the Humber River Basin and is covered by the Humber River Basin Management Plan (RBMP)[1], under partnership, with a vision of "a healthy and wildlife rich water environment that is valued and enjoyed, bringing social and economic benefits to all". As part of this, the RBMP aims to restore salmonid populations using an 'estuary to source' route, outlining the importance of fish passages to achieve this.

In line with this aim and as part of the strategy for ecological improvements on the River Aire, the DN Aire Project proposed the construction of a Larinier fish pass along the right bank of Newlay Weir. The fish pass was designed and constructed by Suttle Projects Ltd (the Contractor) and was designed to allow fish passage over the weir. In order for the fish pass to be constructed, a sheet piled cofferdam was installed in August 2020 to allow work on the dry surface of the weir to begin.

On Thursday 21st January 2021, after a period of relatively high floodwaters, including Storm Christoph, which peaked on 21st January, a member of the public witnessed failure to the downstream face of the weir. The failure progressed over the next two weeks, resulting in a significant failure and loss of the central portion of the weir (Figure 1).

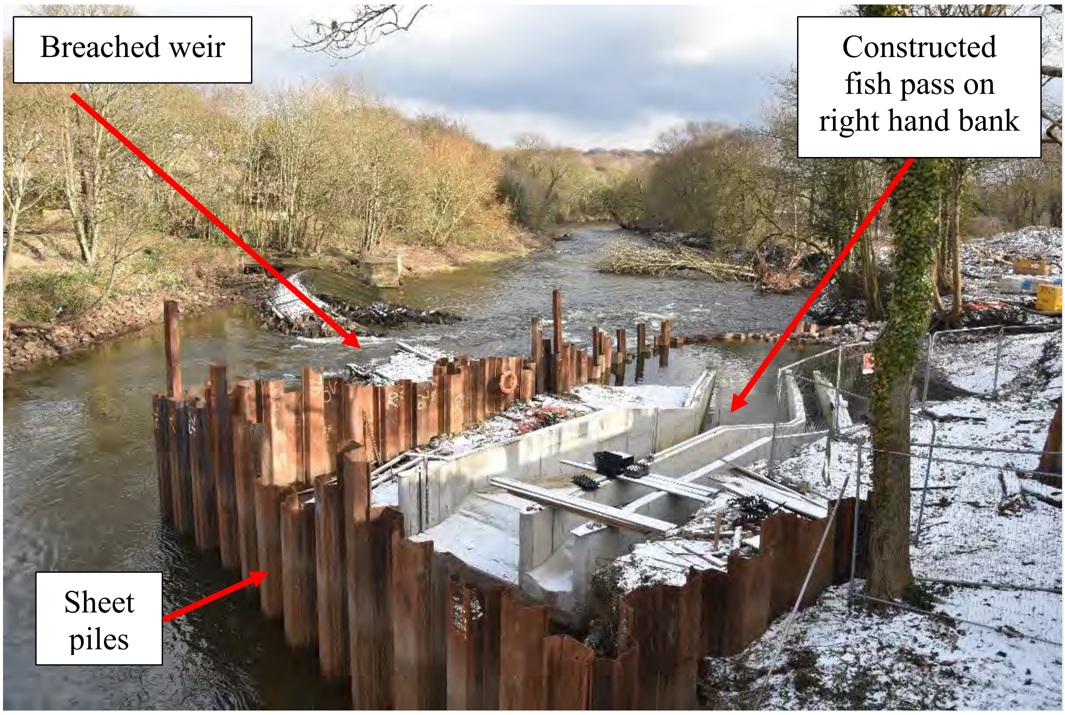


Figure 1 Newlay Weir post failure (12-02-2021)

Arup was commissioned in April 2021 to undertake an Independent Review, to determine the factors that we consider most likely contributed to the weir collapse.

The review was carried out by a team of Engineers and Scientists utilising our Engineering Judgement and Expert Opinion. In coming up with our findings, we reviewed survey information, design information, construction methods, river flow information, hydraulic modelling and information gleaned from photos and videos. The sources of information are provided in a reference section towards the end of this report.

To assist with understanding of the scheme, the team visited Newlay Weir and walked the length of the reach of the Aire to the former Newlay Lower Weir and also met with the Contractor’s Site Engineer to obtain answers to queries that arose during the investigation.

2 Newlay Weir

Newlay Weir (grid reference SE23927 36949) is situated on the River Aire, in the lower River Aire valley, upstream of Leeds. The River Aire catchment area is 691.5 km^2 at the site, draining from the eastern Pennines from its source in Malham Tarn[2].

Initial development in the Newlay area began with the construction of a weir and goit in the 12th century to power the corn mill at Kirkstall Abbey. The current Newlay Weir was constructed in 1690 to provide water to Kirkstall Forge Mill, specifically to supply a head of water to Lower Forge. The forge was fed via a long leat or goit leading to a dam. The later upstream railway crossing was constructed in 1819.

In 1938, repairs were made to the weir (Figure 2) but little else is known about the weir during the intervening years. The reasons for the repairs undertaken in 1938 are not known, but it is thought that the repair included the replacement of stone setts with concrete on the right side of the weir, looking downstream.

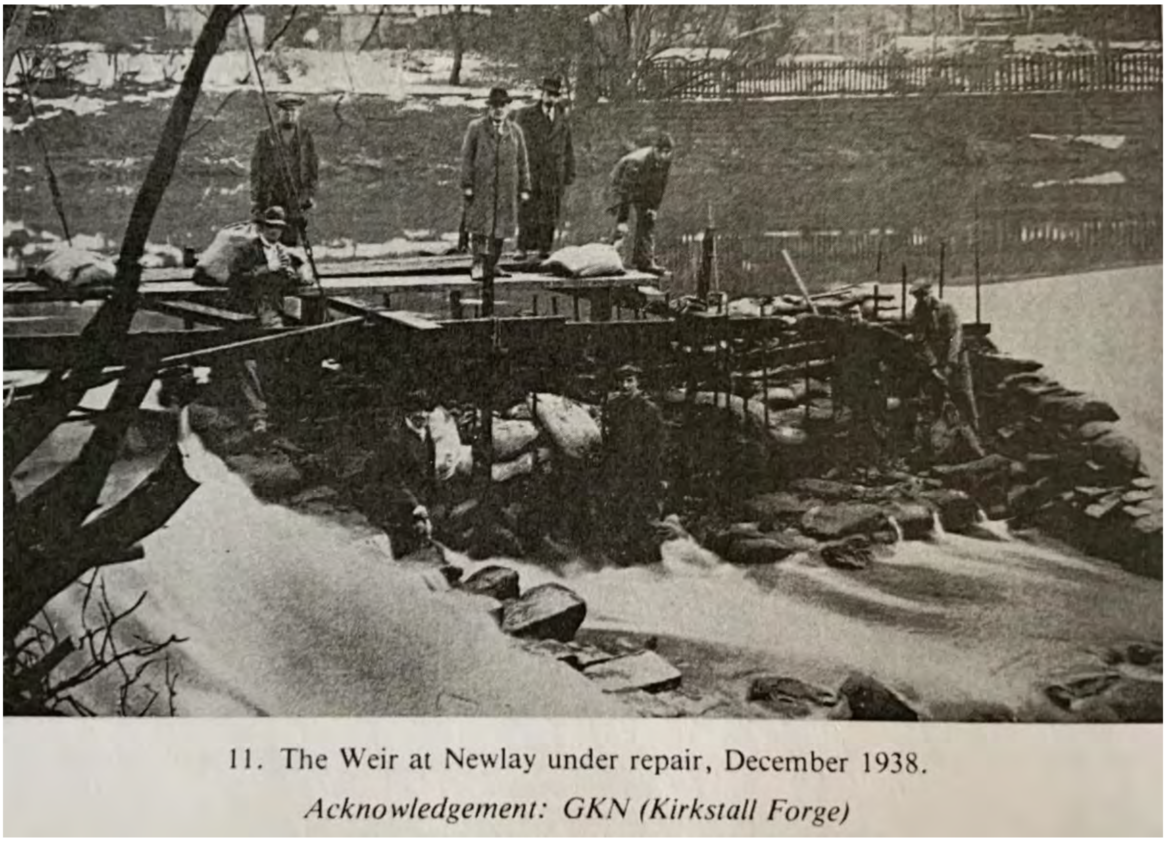


Figure 2 Photograph of repairs undertaken on Newlay Weir in 1938

The weir was listed (Grade II) in 1996 under the Planning (Listed Buildings and Conservation Areas) Act 1990 for its special architectural/historic interest, reference NHLfE 1256648; SE 23910 36950[3]. The weir is located downstream and contributes to the setting of the grade II Newlay Bridge (NHLfE 1375481)[4]. The weir has a 55.7m long curved weir crest. Top levels along the weir are generally uniform with a drop in elevation of 2m from upstream to downstream. The surface of the weir is comprised of Gritstone setts. Figure 3 shows the location of Newlay Weir and the fish pass.

Figure 3 Drone view of Newlay Weir (30-11-2020)

2.1 Construction of Newlay Weir

During site walkovers and through observations and measurements made at that time we have been able to build up a picture of how the weir was constructed and its key structural properties (these are discussed further in the sections below). Newlay Weir is a suppressed rectangular weir which was used to raise the water level of the River Aire. The weir extends across the entire channel, so the length of the weir equals the width of the channel.

A cross section sketch describing the different features of the weir and the nomenclature that will be used throughout the report when referring to Newlay Weir, can be seen in Figure 4. Left Bank and Right Bank of the river are defined as if looking down the river in a downstream direction.

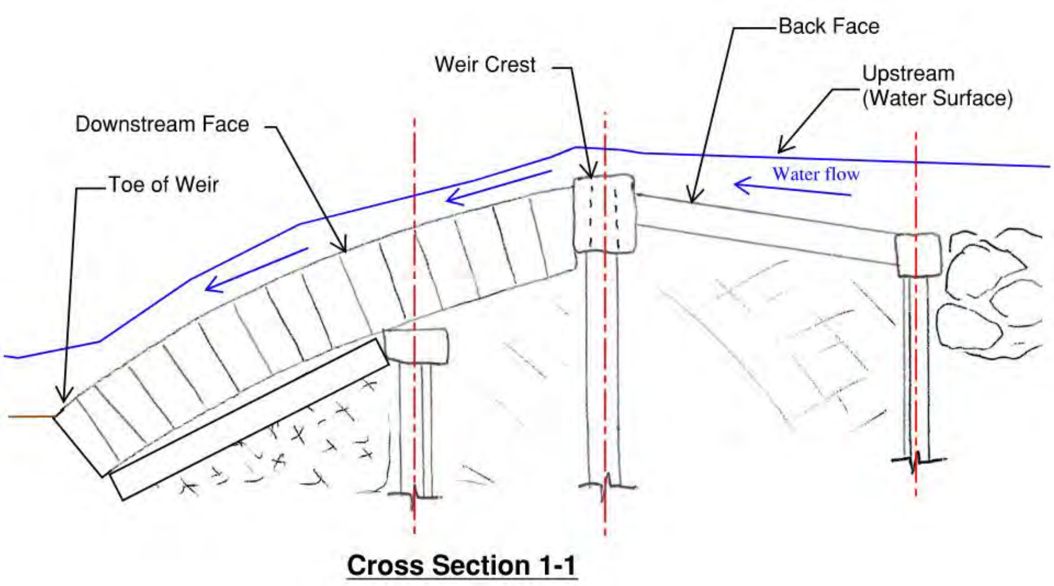


Figure 4 Newlay Weir features and nomenclature

A sketch of Newlay Weir construction drawn from observations of the exposed weir structure (as seen in Figure 5) is described in Figure 6.

Figure 5 Construction/build-up of Newlay Weir (11-05-2021)

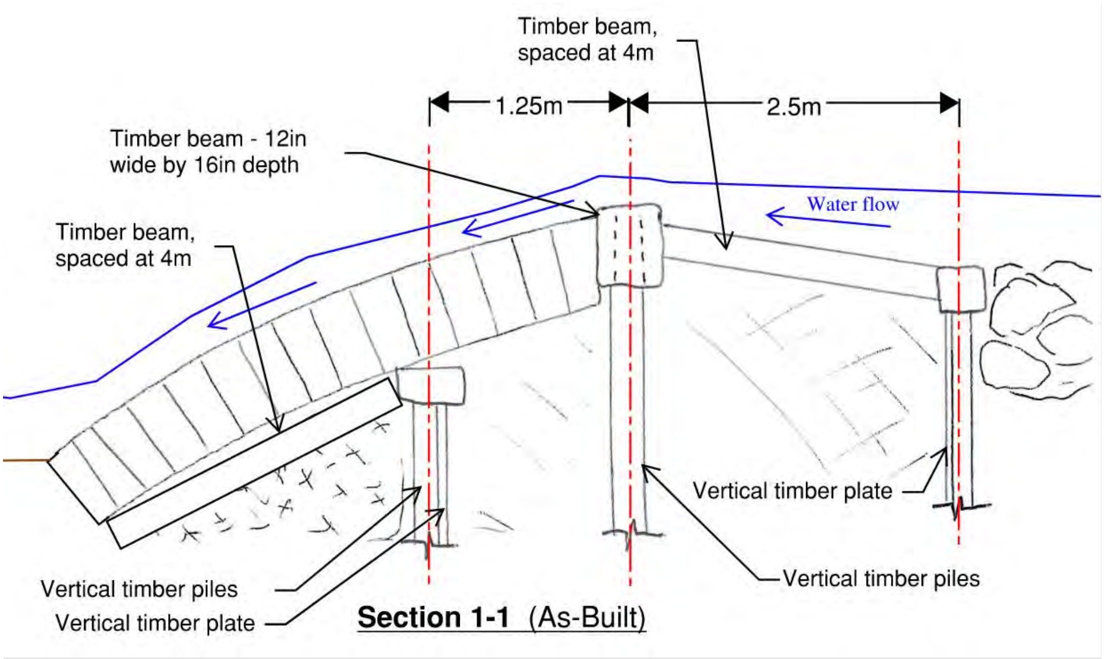


Figure 6 Typical cross section of Newlay Weir

The weir was built with a structural oak timber frame and then infilled with a granular material. The upstream and downstream face are masonry stone blocks (Figures 7 and 8) which are supported by the timber frame and infill granular material. The method of construction is similar to a beach groyne where vertical timber piles are installed and then linked with timber boards.

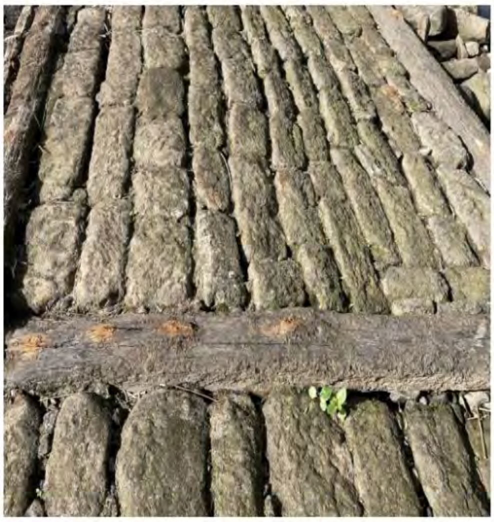


Figure 7 Build-up of the stone blocks on downstream and upstream face on Newlay Weir (11-05-2021)

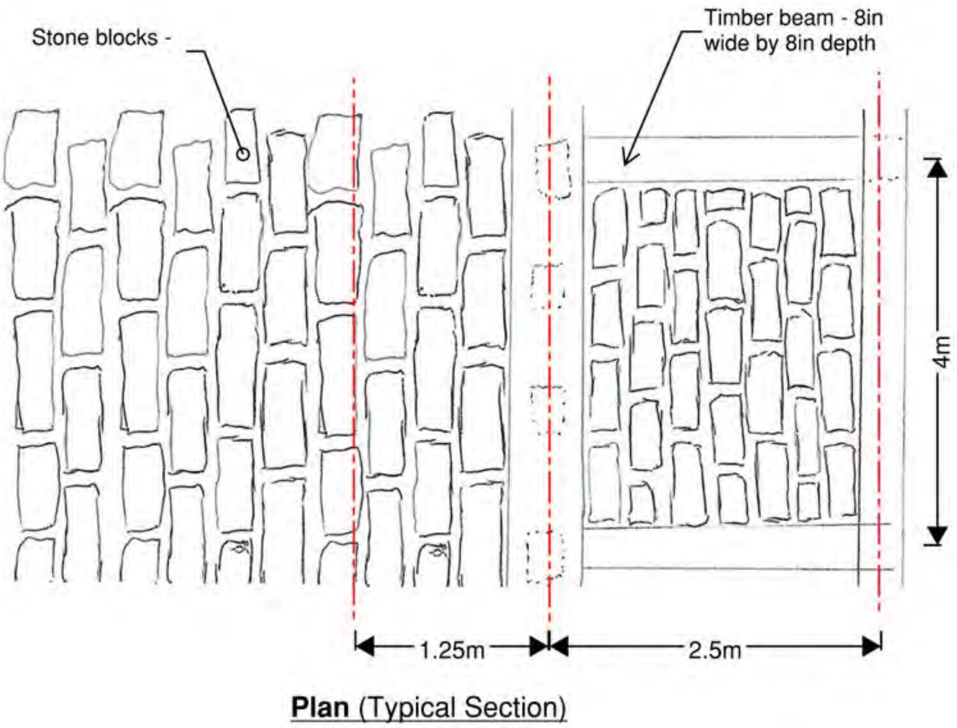


Figure 8 Typical Plan of Newlay Weir

2.2 Structural inspection and pre-condition survey

Arup attended site on 11th May 2021 to inspect the collapsed weir an undertake a visual structural assessment, this is supported by the topographic survey which was undertaken at the same time (Figures 9 and 10).

Previously, a visual structural inspection was undertaken on 19th June 2018 by AECOM ahead of the fish pass construction works when the weir was fully submerged by the river.

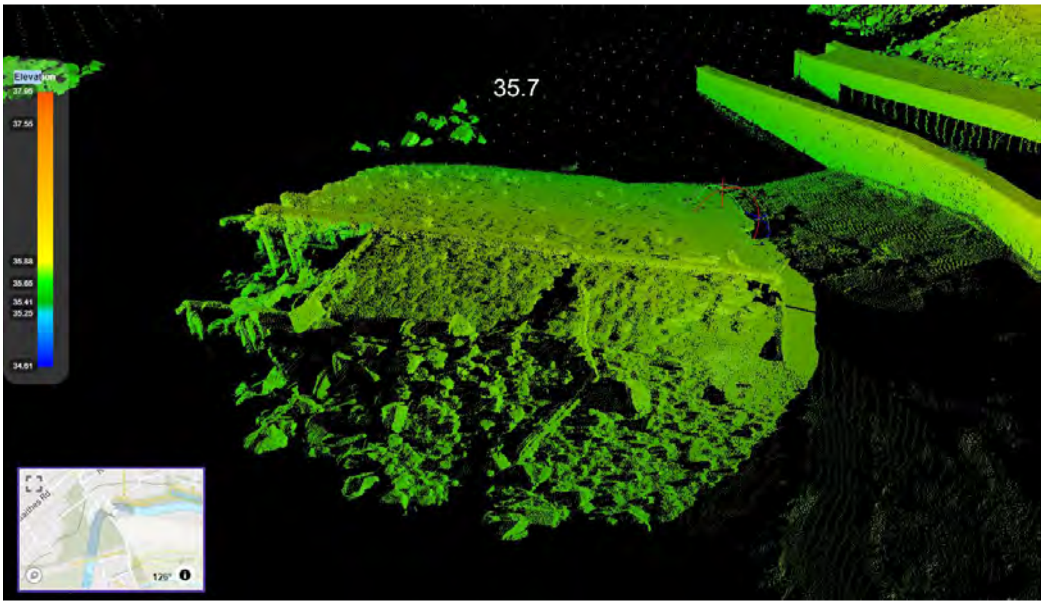


Figure 9 LiDAR scan of point cloud elevation data for the remaining weir on the right bank adjacent to the fish pass. Weir elevation data looking downstream (May 2021)

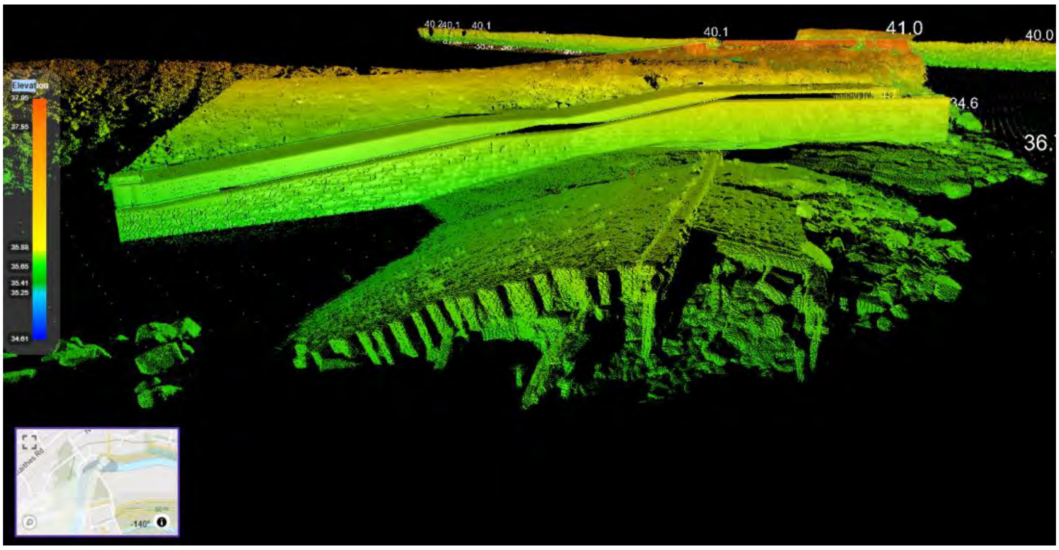


Figure 10 Weir elevation data looking towards the right bank (May 2021)

Figure 11 Images of the collapsed Newlay Weir - 11/05/2021

Observations made by Arup’s structural engineer on the site visit concluded that for the age of the weir, the timber frame was in fair condition, but there were signs of failed timber connections between the vertical and horizontal timbers where the breach had occurred, and the structure was exposed to the flow of the river. The main timber crest was still intact away from the breach (Figure 11). This shows that the main vertical timber piles still have enough capacity to withstand the forces acting on the weir. It is unknown if the timber crest at the breach location was still in fair condition prior to the breach. However, according to the AECOM report (2018), the crest of the weir appeared to be level.

The report highlighted that some of the blocks near the location of the proposed work have been compromised. These blocks appeared to have buckled / displaced from the curved shape of the weir in the location of the fish pass construction area (Figure 12). There were no missing blocks or apparent surface damage or wear on the crest of the weir. There was minimal debris to the top of the weir. A pre-condition photographic survey undertaken by Fishtek[5] on the right side of weir face shows turbulent flow which indicates irregularities in the surface of the weir, supporting AECOM's observations.

In the location of the proposed fish pass, the slope appeared to be consistent in gradient, and flow over the slope was uniform indicating that the surface was potentially free of surface damage and erosion. However, in the area adjacent and downstream of the buckled weir blocks, the slope surface has been eroded with displaced and missing stone setts. This can be seen in Figure 14 taken by the Environment Agency during the construction works, exposing the buckled weir blocks and missing stone setts mentioned in the AECOM report[6].

Figure 12 Drone view of right side of weir (21-04-2020)

Figure 13 Sand filled bags to allow access to the weir (taken by the Environment Agency)

Figure 14 Sand bags placed on weir to allow access (taken by the Environment Agency)

Figure 13 and Figure 14 show sand-filled bulk bags, placed at the crest of weir to reduce the river flow in the location of the proposed fish pass. The low flow at the time of these photographs reveals detail of the condition of the weir that is not evident from the 2018 AECOM structural inspection or the Fishtek 2020 pre-condition photographic survey.

Masonry stone setts extend to the right bank, but there are missing stone setts on the downstream face between approximately 3m to 8m from the right bank. Concrete infill repair is evident at approximately 4m from the right bank and is thought to relate to historical repairs in 1938. Deterioration of the weir is evident through missing stone setts on the downstream face, approximately 5m from the right bank between the stone setts and the concrete repair. This is in the area mentioned in the Fishtek pre-condition survey. Loss of stone setts at the top of the downstream face is also evident.

The AECOM inspection did not carry out any intrusive works, i.e., structural core samples. Internal defects cannot be ruled out, but there was no evidence of damage to the face or crest of the weir during the condition surveys.

The only observed variability in flow patterns over the face of the weir during the inspections was between the section on the right-hand side that was repaired in 1930's and the remaining original weir face.

Prior to any construction works carried out for the fish pass, the weir showed historical repair and missing stone setts to the right side of the weir in the location of the fish pass. However, this was removed following excavation within the cofferdam and would have made no contribution to the collapse. The pre-condition visual structural survey did not show any observable damage to the weir in the area of the collapse. There are no records of maintenance for the weir structure. However, although it is possible that the lack of maintenance could have played a role in the weir collapse, we conclude that the weir was in fair condition.

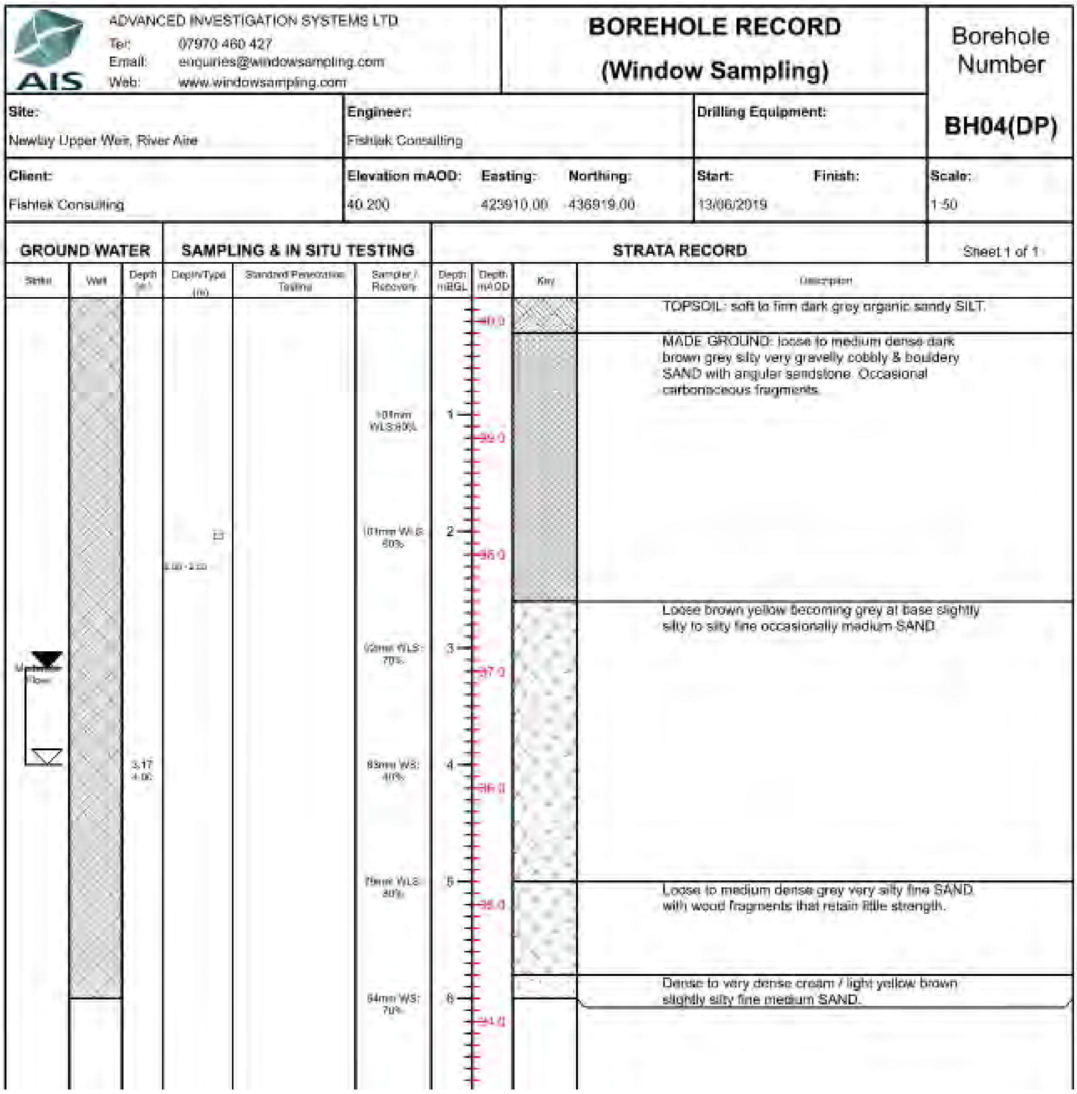
2.3 Ground conditions

British Geological Survey data shows that the weir and river are underlain by Alluvium (Clay, Silt, Sand & Gravel). At depth, interbedded Sandstone of the Carboniferous Rough Rock Sandstone Formation (coarse grained felspathic Sandstone) is indicated.

A Ground Investigation was undertaken by AIS Ltd June 2019[7] comprising four dynamic probes on the right bank of the river to depths of 0.87m (refusal), 4.48m (refusal), 7.74m (refusal) and 12.0m BGL. Bedrock comprising the Rough Rock Sandstone was not encountered by this investigation. The Borehole record is shown in Figure 15.

Alluvial deposits comprised loose becoming medium dense, silty, fine occasionally medium sand and fine medium & coarse gravel. The river substrate was found to consist predominantly of cobbles and boulders.

The factual report noted that construction of the fish pass could potentially affect and potentially mobilise impounded sediment behind and above the weir structure.



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BOREHOLE RECORD
(Window Sampling)

Borehole Number
BH04(DP)

Site: Newlay Upper Weir, River Aire	Engineer: Fishtek Consulting		Drilling Equipment:	
Client: Fishtek Consulting	Elevation mAOD: 40.200	Easting: 423910.00	Northing: 436919.00	Start: 13/06/2019 Finish:	Scale: 1:50

GROUND WATER | SAMPLING & IN SITU TESTING | STRATA RECORD | Sheet 1 of 1

Depth mBGL (approx.)	Description
0.0 – 0.3	TOPSOIL: soft to firm dark grey organic sandy SILT.
0.3 – 2.6	MADE GROUND: loose to medium dense dark brown grey silty very gravelly cobbly & bouldery SAND with angular sandstone. Occasional carbonaceous fragments.
2.6 – 5.0	Loose brown yellow becoming grey at base slightly silty to silty fine occasionally medium SAND.
5.0 – 5.8	Loose to medium dense grey very silty fine SAND with wood fragments that retain little strength.
5.8 – 6.0	Dense to very dense cream / light yellow brown slightly silty fine medium SAND.

Figure 15 Borehole record on the right bank for pre works ground investigation

3 Fish pass construction methodology

Figure 16 Drone view of Newlay Weir (30-11-2020)

3.1 Temporary works

The temporary works[8][9] design for the fish pass (Figure 16) adopted by the Contractor was for the following:

- An interlocking steel sheet pile cofferdam to accommodate up to 2.70m of excavation at the top of the weir face. The design philosophy considered two options for the spine walls and headwalls dependent on ground conditions encountered and depth of water upstream of the weir.
- The design was carried out using Reward 2.8 software and was designed to Eurocode 7 (EN 1997-1, design approach 1).
- Embedment depth to be 5.32m on the upstream headwall and spine wall and 3.93m on the downstream headwall.
- The ground level for piling is at 38.17mAOD. The top of cofferdam level is 38.65mAOD, excavation level is 35.77mAOD.
- Sheet pile type is L603 600mm width Larssen sheet pile at 7m in length.
- Due to the requirement for a prop to be installed, all sheet piles were of the same length.
- The downstream headwall to have a larger upstand to accommodate the prop. Cofferdam designed for 1 in 10 year event + 150mm freeboard.

- No surcharge loading was considered.
- The proposed temporary works design allowed for 1m clearance for the sheet piles from the excavation level.

3.2 Piling installation

The proposed methodology for piling installation as stated in the temporary works design provided by the Contractor was for the following:

- Sand filled bulk bags to be placed at the crest of the weir. A ramp was constructed from right bank adjacent to crest and the excavator boom reached across to place the filled bags.
- A piling mat comprising imported stone was constructed on the right bank to install the upstream sheet piles nearest the bank.
- An additional pile mat comprising imported stone was constructed on the weir face to provide access for upstream pile installation further out into the river and spine wall along the weir.
- Sheet pile installation for the cofferdam was undertaken between 6th and 19th August 2020.
- The sheet piles were installed outside of the damaged weir face as shown in Figure 14 and so the 1938 repair is considered to lie within the cofferdam. The Contractor informs that the top timber beam at the weir crest observed by the Arup team during a post-breach site walkover had already been cut, presumably during these historical repairs.
- A stone causeway was placed over the upstream side of the weir to allow the excavator to track over the weir to avoid point loading the weir.
- Removal of stone setts from the face of the weir.
- Sheet piles were handled, pitched and installed utilising a Movax high frequency vibratory piling hammer on a 30 tonne excavator.
- Installation was by high frequency continuous vibration and did not involve impact driving type equipment.
- For each pile the excavator gripped and raised the pile utilising the 4th jaw plate. The operator then laid the pile onto its tracks and repositioned the Movax so that the pile was held in by the side grip arms.
- The pile was then pitched and driven until it was self-supporting.
- The hammer was withdrawn, and the “pile hand” operative remotely checked the pile for plumb and alignment, utilising a spirit level and directing the rig operator to re-pitch or proceed with driving accordingly.
- The sheet piles that were higher than the others, were installed to refusal. Others were lowered to the level specified in the flood risk permit.
- On completion of the piling installation, excavation commenced inside the cofferdam by breaking out the existing weir blocks using a 5 tonne excavator fitted with breaker attachment; lifted into the cofferdam by a 21 tonne excavator.

The Risk and Method Statement (RAMS)[10] stated that all piling will be executed as per the Institution of Civil Engineers SPERW (Specification for Piling and Embedded Retaining Walls) and piling equipment to be chosen to ensure noise and vibration are within agreed environmental nuisance limits.

Figure 17 shows the piling installation and Figure 18 18 shows the installed piles. A boulder platform has been constructed upstream of the weir tying into the weir crest to allow the installation of the piles from the river using a Movax side grip attached to a 30 tonne tracked excavator. The steel piles used were between 6.0m and 7.5m in length long as depicted by the spray paint numbers Figure 18 does not appear to show removal of stone setts prior to installation of the piles.

Figure 17 Piling installation for cofferdam

Figure 18 Installed sheet piles

It was proposed that removal of the piles would be a vibration method using the Movax to pull on the top of the pile, followed by side grip to extract the pile until the pile becomes de-clutched. Following the breach in the weir, the piles were not all vibrated out, but were cut down to just above bed level on the upstream side and on the weir face. Sheet piles downstream of the weir were vibrated out.

As described in Section 4.3, the piles were not installed to the level specified by the Flood Permit[7] and as such, did not follow the proposed methodology. The permit stated that the piles should be installed 0.6m below the top of bank. In fact, some piles were installed to a level around 1m above the top of bank.

The permit also required that the cofferdam should not reduce the width of the channel by more than 10m. The channel width was actually reduced by 13.6 m.

3.3 Archaeological watching brief

Following construction of the cofferdam, an archaeological watching brief was undertaken by West Yorkshire Archaeological Services (WYAS) during the excavation of the weir for the fish pass. The excavation comprised a rectangular trench against the right bank measuring 22.80m long, by 10.80m wide as shown in Figure 19.

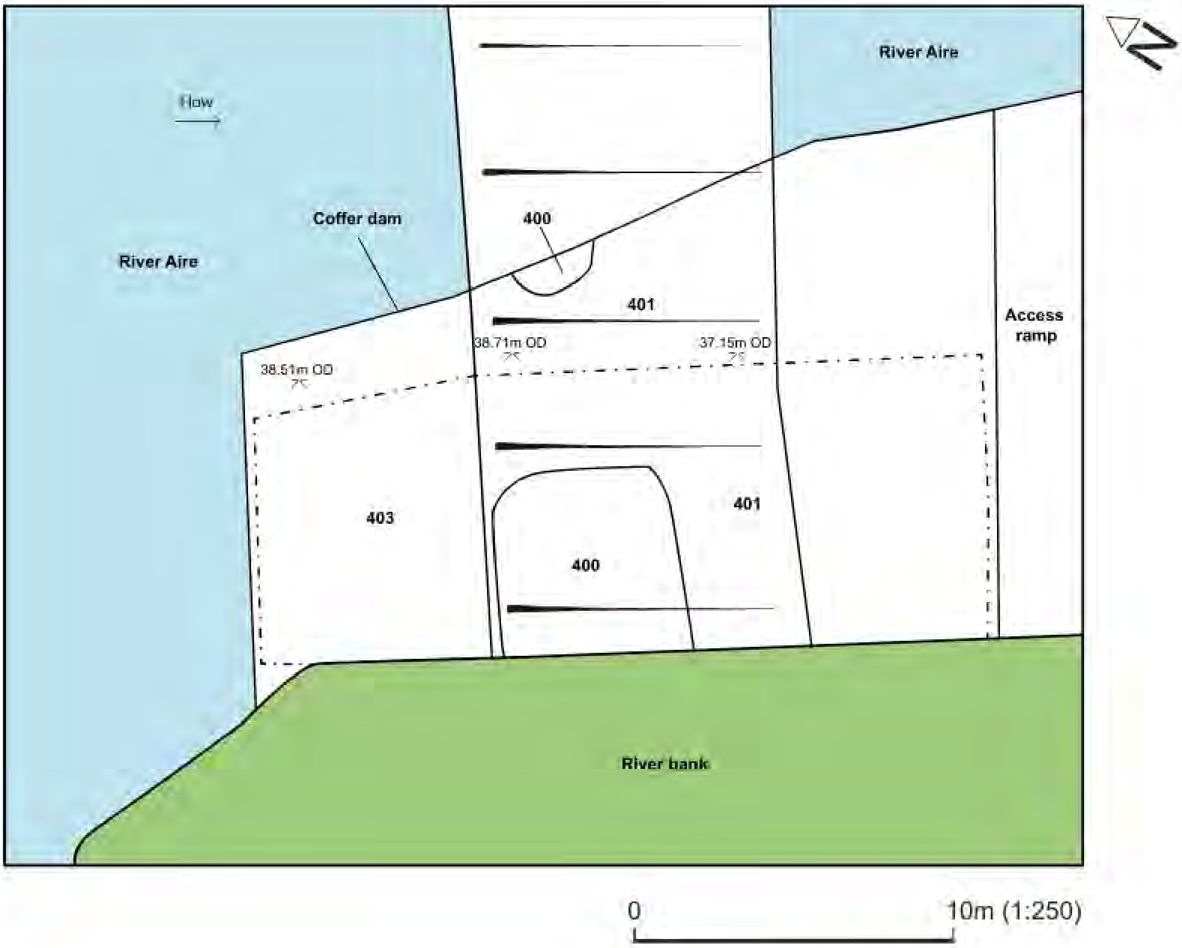

Figure 19 Trench layout with references

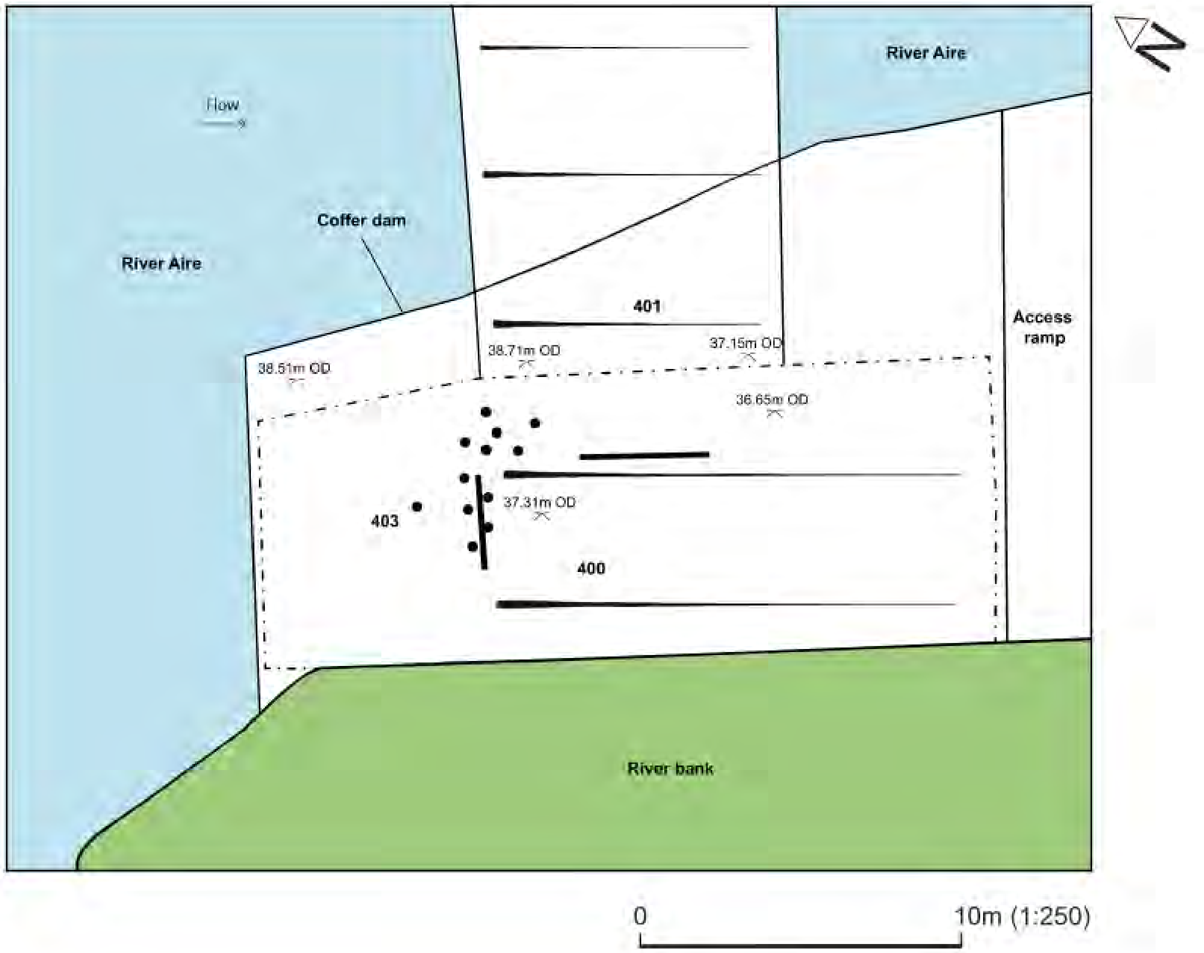

Figure 20 Trench layout with locations of exposed timber stakes and beams

An area of the original weir face was located close to the right bank and measured c. 5m by 5m labelled as 400 as shown in Figure 19 and Figure 21. The sloping weir face was constructed of uniform dressed gritstone blocks, lain in tight rows and had a depth of 0.28m to 0.32m. The remaining surface within the trench was concrete repair labelled as 401, presumed to be from 1938. It was 0.16m to 0.27m thick. Within the concrete, scrap metal pipes had been incorporated to reinforce the surface. No original weir masonry was underlying the concrete repair. A photo of the excavation is provided in Figure 21.

Figure 21 Exposed timber uprights

3.4 Review of vibrations from sheet pile installation

3.4.1 Introduction

This assessment is based on a retrospective analysis of available information. Arup was not involved with any vibration measurement or assessment before or during the works.

The May 2021 post-breach survey data shows:

- The fish pass, weir structure, extent of breach and alignment of the sheet piles used for the temporary works.
- The horizontal distance from the sheet pile to the initial breach is approximately 10m.

The closest pile at the toe of the downstream face to the breach at the time of the survey was 6.23m but photographic evidence from section 6 indicates that the initial breach was further into the channel; estimated to be 10m from the closest pile as shown in Figure 22.

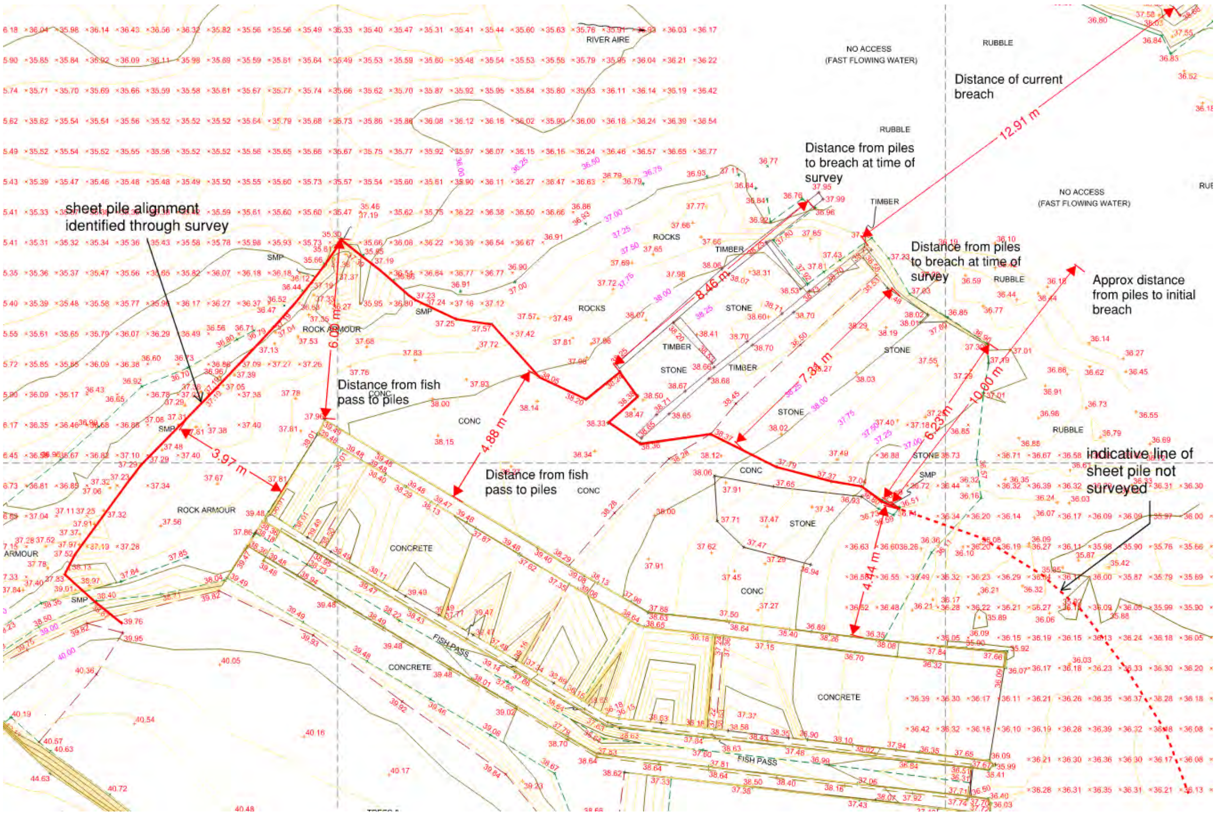

Figure 22 Sheet pile location in relation to breach

3.4.2 Vibration damage criteria

Sheet pile installation was undertaken between 6th and 19th August 2020. The breach did not occur until five months after installation. No vibration monitoring for peak particle velocity (PPV) was undertaken during the works but target markers were installed in the wall by the road to monitor movement, on the bridge and at 2 Rein Road (following complaints of noise and vibration). Seven days of readings on August 6th, 7th, 10th, 11th, 17th, 24th and 28th form base readings for horizontal, vertical and elevation shows up to 4mm displacement. The Contractor's design risk assessment[10] identified a tolerance of up to 10mm movement prior to works stopping but it is not clear what this tolerance is based on.

Potentially two primary mechanisms caused by vibration could have presented some risk of affecting the integrity of the weir:

- Dynamic strain or relative displacement within the structure; and
- Compaction of underlying ground leading to differential settlement or displacement across the weir.

The weir is a type of structure for which there is no information available regarding what magnitude of vibration could begin to present a risk to the structure. The following therefore provides some insight but cannot be considered conclusive.

BS7385-2:1993 provides guidance on levels of vibration that may cause damage, quantified as the PPV. It is noted in the standard that '*Important buildings which are difficult to repair may require special consideration on a case-by-case basis. A building of historical value should not (unless it is structurally unsound) be assumed to be more sensitive*.'

The standard provides guide values '*judged to give a minimal risk of vibration-induced damage*'. The PPVs specified are given for transient vibration that does not give rise to resonance in structures and suggests that a reduction of 50% is commonly applied for continuous vibration.

Section 7.4.1 of the standard states that '*the probability of damage tends towards zero at 12.5mm/s*'. Although this relates to residential buildings, it is often used as a basis for assessing risk of possible onset of damage to other buildings, with the level being 6mm/s for continuous vibration, i.e. the type of vibration that would have been caused during the piling works.

Susceptibility to consolidation or densification due to vibration is dependent on the type of ground. BS 7385-2[12] states that loose and especially water-saturated cohesionless soils are vulnerable to vibration and may start to become vulnerable to vibration at about 10mm/s PPV. The German standard DIN4150-3[13] also notes the sensitivity of such soils, particularly in relation to use of vibratory hammers for sheet piling and for piling below the groundwater table.

3.4.3 Estimate of vibration during piling

Research on construction induced vibration carried out at the Transport Research Laboratory[14] provided a suite of empirical prediction equations, including vibratory pile driving. The equations are included in British Standard BS5228-2[15] on control of construction vibration.

For vibrodriving, TRL429 shows that vibration can be higher during the transient periods of operation, as the driver starts up and runs down, than during continuous driving.

The PPV (v in mm/s) is calculated from:

$$v = \frac{k_v}{x^d}$$

Where k_v = 60 (50% probability of the predicted level being exceeded);
k_v = 126 (33% probability of the predicted level being exceeded);
k_v = 266 (5% probability of the predicted level being exceeded);
x = horizontal distance from the pile (m);
d = 1.3 for all operations;
d = 1.2 for start up and run down;
d = 1.4 during continuous / steady state driving.

The Movax vibratory pile driver used at Newlay Weir was an SG65. This has a variable eccentric moment mechanism that would allow the start up and run down transient periods to be avoided if the variable eccentric moment system was balanced on start up and run down. If not, then vibration could have been higher for some of those stages of the driving. Inspection requirements for the Movax piling rig is undertaken every six months. As there are no calibration certificates, calculations have been done for in-balance and out of balance modes of operation to illustrate the likely range of vibration as shown in Figure 23.

Figure 23 below presents the predicted PPVs during driving with start up and run down.

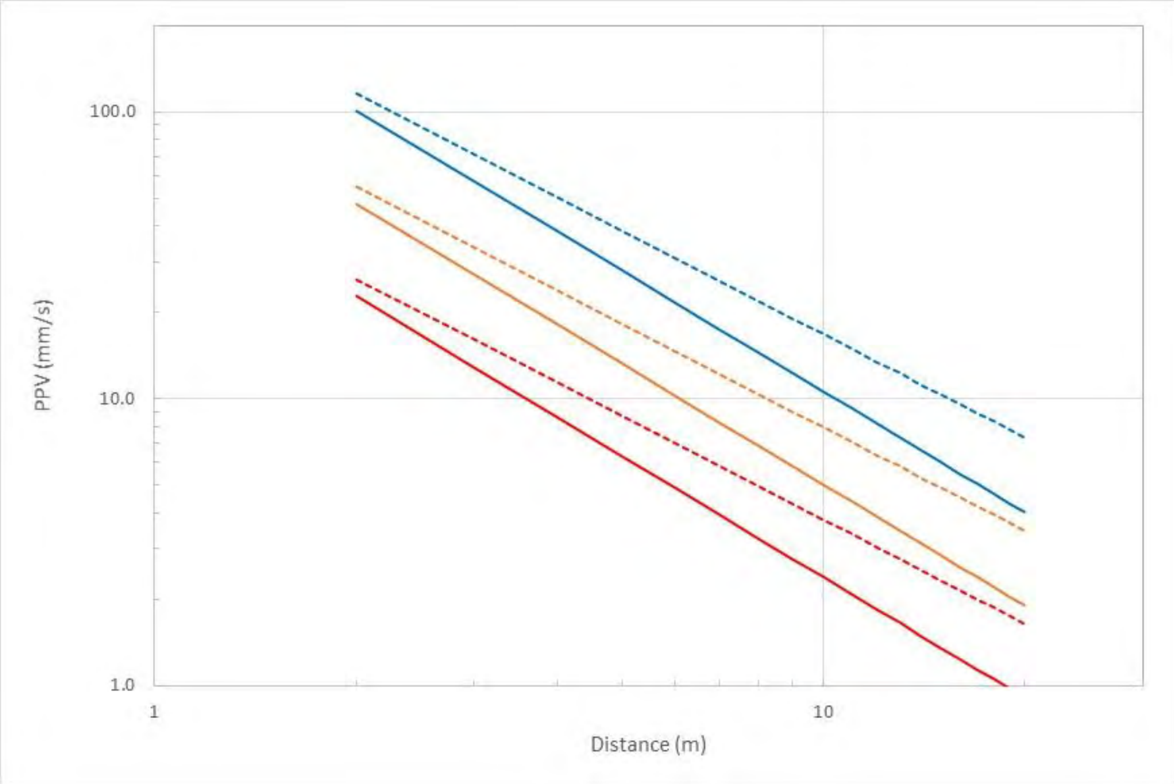

Figure 23 Predicted PPV during pile driving

Figure 23 assumes balanced eccentric moment (solid lines) and out of balance during start up / run down (broken lines) for three levels of probability that the level could be exceeded (50% red; 33% amber; 5% blue). Assuming the failure was initiated approximately 10m from the closest pile, Table 1 shows the PPVs that may have occurred.

Table 1 Predicted PPV at 10m from the closest pile

Operating mode	**PPV at 10m with probability that the level could have been exceeded**		
	50%	**33%**	**5%**
Balanced	2.4mm/s	5.0mm/s	10.6mm/s
Out of balance	3.8mm/s	8.0mm/s	16.8mm/s

Interpreting Figure 23 in a different way, Table 2 shows the distances from the source of the vibration (the pile installer) at which there is a probability that 10mm/s PPV could have been exceeded.

Table 2 Predicted distances beyond which 10mm/s could have been exceeded

Operating mode	**Distances beyond which there is a probability that 10mm/s PPV could have been exceeded**		
	50%	**33%**	**5%**
Balanced	3.6m	6.1m	10.4m
Out of balance	4.5m	8.2m	15.4m

If the assessment is based on 6mm/s, the PPV above which there is some risk that direct disturbance to the structure could have occurred, then clearly the probability of occurrence would have been higher at a given distance, or the distances from the pile at which the PPV occurred would have been less.

3.4.4 Interpretation and conclusions

It cannot be categorically stated whether vibration caused, or contributed to, the weir collapse as there is no way of accurately knowing the vulnerability of the structure, or the underlying ground, to vibration. Pre-construction photographs show degradation of the weir in the location of the fish pass before the works started although there is no evidence of degradation within the area of the breach.

The site walkover post-event did not show evidence of displaced beams or setts between the pile line and the breach which indicates that the mechanical movement of the weir structure associated with piling is not considered to be a contributory factor.

No vibration measurement data for PPV is available from the works other than deflection monitoring described in Section 3.4.2. However:

- the structural inspection and pre-condition site photographs indicated some evidence of damage within the proposed fish pass; and
- the ground investigation identified soils below the weir that can be susceptible to vibration.

Structures and loose granular soils as described in Section 2.2, are more susceptible to continuous vibration than during exposure to impulsive vibration. Although vibratory piling causes periods of continuous vibration, the empirical prediction from the PPV analysis show that high levels of vibration occur very locally.

Predictions made using the empirical method from BS5228-2 indicate a low probability that vibration could have exceeded the criteria at which there would be some risk for both potential damage mechanisms. Additionally, there was a period of five months between the pile installation and first evidence of collapse during which there were high flow events. It is considered that if vibration had been an important factor in the breach, it would have been apparent sooner.

It is therefore concluded that the piling works represent a low probability of contributing to the failure of the weir.

4 Hydraulic conditions

To understand the role of the hydraulic conditions on the collapse of the weir it is important to appreciate both the baseline conditions and the conditions at the time of the event. The rarity (or not) of the flood conditions has also been assessed. To assess the hydraulic condition within the river channel around the time of the collapse as a result of both the flow conditions at the time and the effect on those flow conditions due to the temporary works, hydraulic modelling has been undertaken using a numerical model of the river. The sections below describe these analyses.

4.1 Historical flow events

According to the Environment Agency's historic flood map[16], flood events have occurred at the site in 1978, 2000, 2002, 2015 and 2020. The extent of these events is presented in Figure 24. A high flow event also occurred in January 2021 during Storm Cristoph. However, no flooding was recorded in the locality.

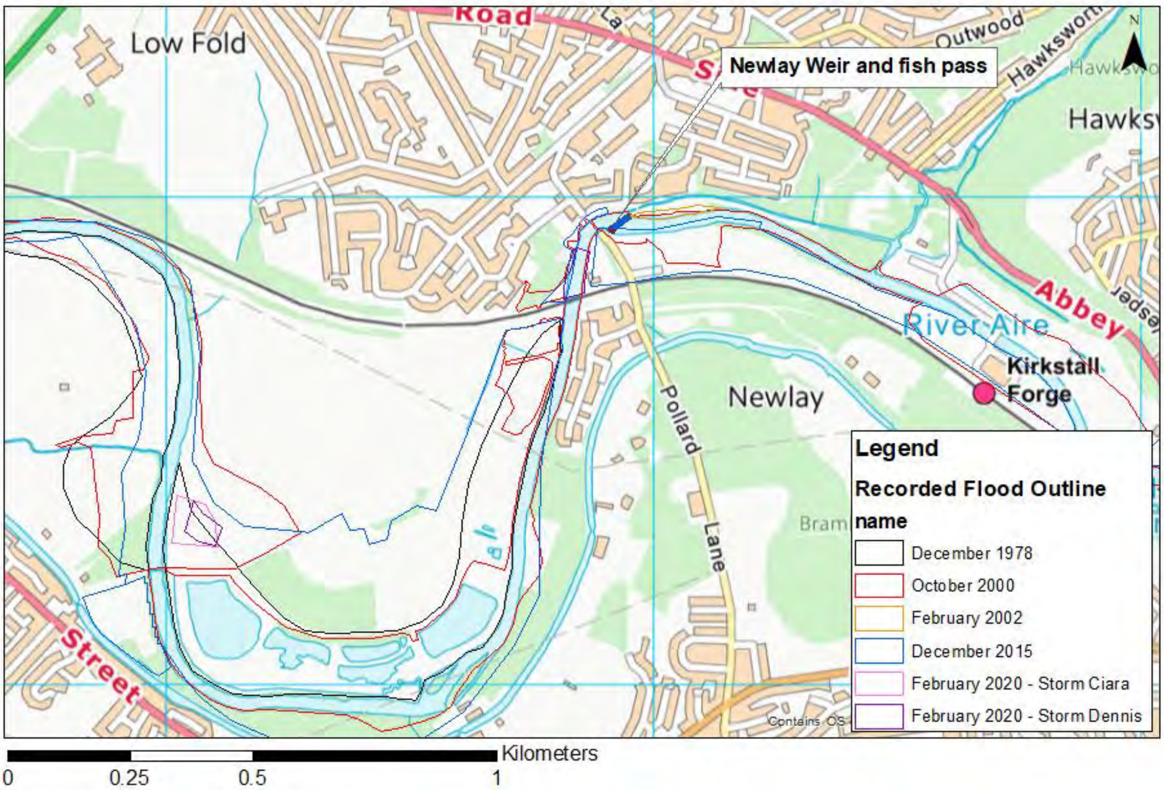

Figure 24 Map of EA Historic Flood Outlines

The Environment Agency has a flow gauge at Armley (6.2 km downstream of the site). There is also a level gauge at Kirkstall (2.5 km downstream of the site), but that gauge has no associated flow data, which is required for the hydraulic analysis. Data from the Armley gauge captured during the EA recorded flood events are presented in Table 3.

Events	Peak River Level (m)	Peak Flow (m^3s^{-1})
28/12/1978	3.323	170
Millennial Storm Event 31/10/2000 06/11/2000	 4.025 3.398	 230 177
11/02/2002	3.323	170
15/11/2015	3.71	202
12/12/2015	3.814	211
27/12/2015 – Storm Desmond	5.217	344
09/02/2020 - Storm Ciara	4.134	240
16/02/2020 - Storm Dennis	3.417	178
21/01/2021 – Storm Cristoph	3.279	168

Table 3 Recorded daily maximum river level and flow values captured at the Armley gauge for the EA recorded flood events

4.2 Hydraulic conditions during construction

On 21st January 2021, when Storm Cristoph peaked in Yorkshire, the peak water level at Armley reached 3.279 m. All events in Table 3 exceeded the January 2021 peak water level. The flow associated with this level is 168 m^3s^{-1}. There is little inflow between the Newlay Weir and the Armley gauge, and consequently knowing this flow allows reasonable representation of the flows applicable to Newlay Weir.

According to the hydrological annual maximum series (AMAX) for this gauge, the likelihood of this flow being exceeded in any given year (the annual exceedance probability, or AEP) is 18.0% (or an event with a return period of approximately 5 years). The return period is the likelihood of the event being equalled or exceeded in that period (over a very long term). It has, in fact, been exceeded eight times in the preceding ten years. Previous flow events resulted in greater peak flow values, with Storm Desmond, the highest on record, reaching 344 m^3s^{-1} on 27/12/2015.

It can be concluded that the flood event preceding the collapse (Storm Cristoph) was a significant, but not a particularly rare event.

In addition to AMAX, flow data has been evaluated between August 2020 and January 2021 to assess whether the number of high flow events during this period were unusual. There were 3 peaks over the Environment Agency Threshold (POT) for the Armley gauge, over the period. The mean between August and January for previous years is 2.3 events and the median is 2. This occurrence puts the flow record over the period into the 33rd percentile, meaning that 33% of years have more peaks over the threshold than were experienced during the construction period.

The sheet piled cofferdam was installed in the weir between the 6th and 19th August 2020. Between the installation date and January 21st 2021 when Storm Christoph occurred, high flows stopped construction work on two previous occasions (stopping for approximately a week each time, commencing 5th October and 2nd November). In addition, prior to the 21st January, work had been stopped for about a week due to high flows. The periods when construction work was stopped due to high flow is shown in Figure 25. Site work was also paused during the Christmas break during which a high flow event occurred on 27th December. This has not been included in Figure 25 because this pause was not a direct result of flow conditions although it is possible that work may have ceased at some point during this period due to a comparable high flow event at the end of December.

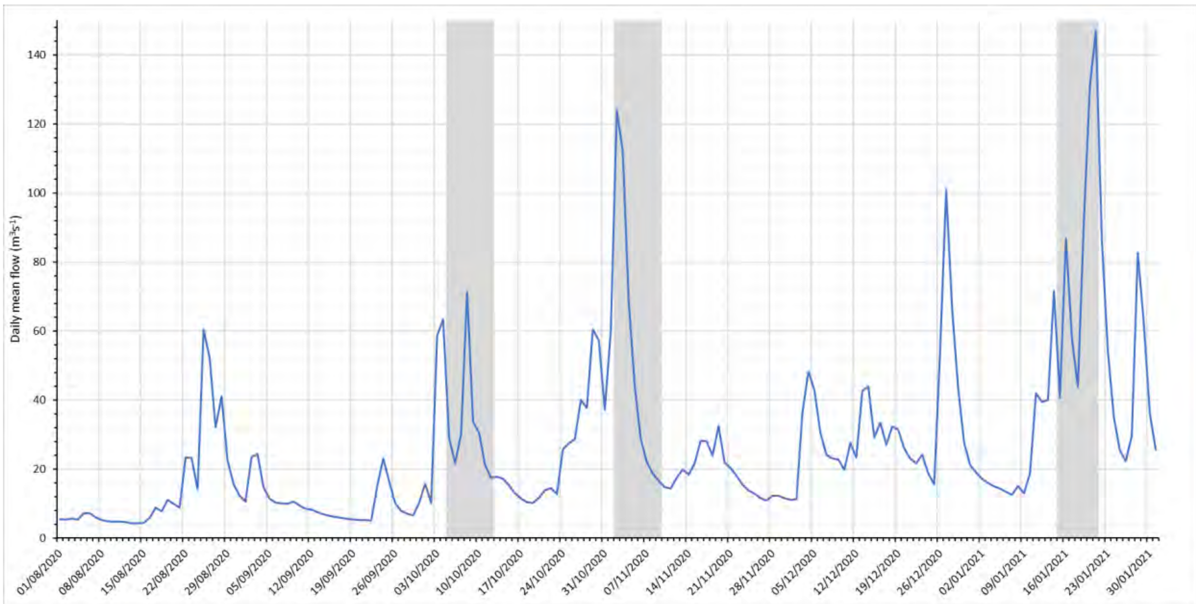

Figure 25 Daily mean flow at Armley between 01/08/2020 and 31/01/2021 with periods (in grey) when flow was too high to carry out work on the fish pass

4.2.1 Hydraulic modelling of the weir

A recent model (Leeds Flood Alleviation Scheme Phase II) of the River Aire and associated floodplains was made available to Arup to use for this assessment. The model was built using industry standard software and uses Flood Modeller Pro (1D for the in-channel model) and TUFLOW (for the floodplains), The model extends for several kilometres upstream of Newlay Weir, and downstream to beyond Leeds Rail Station.

To help understand the flow characteristics associated with the weir and the water velocities and depths associated with the river levels around the time of the collapse, hydraulic modelling has been carried out using this model. The work is described in a Technical Note included in Appendix A and summarised below.

4.2.2 Hydrology

The Leeds FAS Phase II modelling included many storm event profiles in excess of this peak value; the smallest is the 5 year return period design storm event, with an AEP of 18%. This was used in the analysis, as the modelled peak flow in this event (168 m^3/s at the Armley gauge), is the same flow experienced by the weir during Storm Christoph (Section 4.2)

A 200 year return period event (statistical peak flow at Armley: 365 m^3/s) was also simulated on the baseline morphology only. This is close in magnitude to the largest event on record

(2015 Boxing Day floods with a peak flow of 344 m^3/s), and consequently demonstrates the greatest flows, depths, and velocities known to have been imposed on the weir by the river.

4.2.3 Model build – Baseline

The model morphology was based on a 2016 topographical survey. However, more recent survey (September 2018) collected prior to the construction of the fish pass (see Section 5.2), were used as part of this assessment to update the model cross sections and representation of the weir.

The weir is non-standard and therefore representing it in a numerical model requires some subjective choices. The method of modelling the weir in the model has been reviewed and updated. In the original model it was represented with a general "spill" unit, with a coefficient of discharge of 1.2. The spill unit is a way of defining a weir structure by inputting the weir crest profile according to the levels obtained by topographical survey. The coefficient of discharge defines how efficient the structure is at passing flow over it: a lower number indicates a low efficiency, meaning it's "more difficult" for flow to pass over it. The Arup review concluded that the coefficient used in the original model was too low and didn't reflect the efficiency of the weir structure and as such, the coefficient was increased to 1.6.

The original model also had a Modular Limit for the weir of 0.9. The Modular Limit represents the ratio between the upstream and downstream weir level, above which the downstream level impacts on the upstream level and flow over the weir ceases to discharge freely. The Arup review also concluded that the Modular Limit was too high and as such, a limit of 0.75 was selected.

These changes would serve to reduce the head over the weir but increase the velocity on the weir face.

During the Arup site visit it was noted that the exposed shape of the weir was similar to a crump weir shape. The crump weir is a British Standard weir shape, with known coefficients of discharge that vary with depth, obtained from extensive testing on standardised structures. The crump weir is triangular in section, with a 1 in 3 upstream sloping face and a 1 in 5 downstream sloping face. Although the geometry of Newlay Weir does not conform to this shape exactly, it was deemed to be sufficiently similar to warrant modelling it using that unit, to determine the effect on the results. As such an additional model baseline scenario was undertaken with the same updated topography, where the 55.7m long weir crest was replaced with a formal crump weir unit in the Flood Modeller software, with the left and right bank bypass flow retained in the original spill unit (spill coefficient: 1.0, suitable for overland flow) within the model the crump weir unit has a variable Modular Limit; other geometry details were derived from the upstream and downstream cross-sections.

4.2.4 Construction model build

The information provided by the contractor regarding the position and height of the sheet piling was limited to a proposed alignment detailed in the temporary works drawing rather than the as-built locations.

The 2021 topographic survey was conducted after the sheet piles had been removed but, as they were cut down to water level rather than being removed totally, the survey was able to capture their location precisely. This information was used to determine the reduced width of the weir. It was found that the net effect of the sheet piles was to reduce the effective width of the weir crest from 55.7m to 42.1m, a reduction of 25%.

The height of the sheet piles during construction was more difficult to determine as no as-built survey information was provided, but photographs taken whilst the sheet piling was still in position (e.g. Figure 26 allowed these heights to be estimated, by comparing the level of the piles in relation to the height of the fish pass (known to be 1.5m), and the standard height of the Heras fencing on the bank (2m).

Figure 26 The collapsed weir during construction of the fish pass

The height of the sheet piles has been estimated based on photography as approximately 1m above ground level on the land side, reduced by 0.3m further in the channel. See Figure 27 for details of how this and the spill profiles are represented in the baseline and construction models.

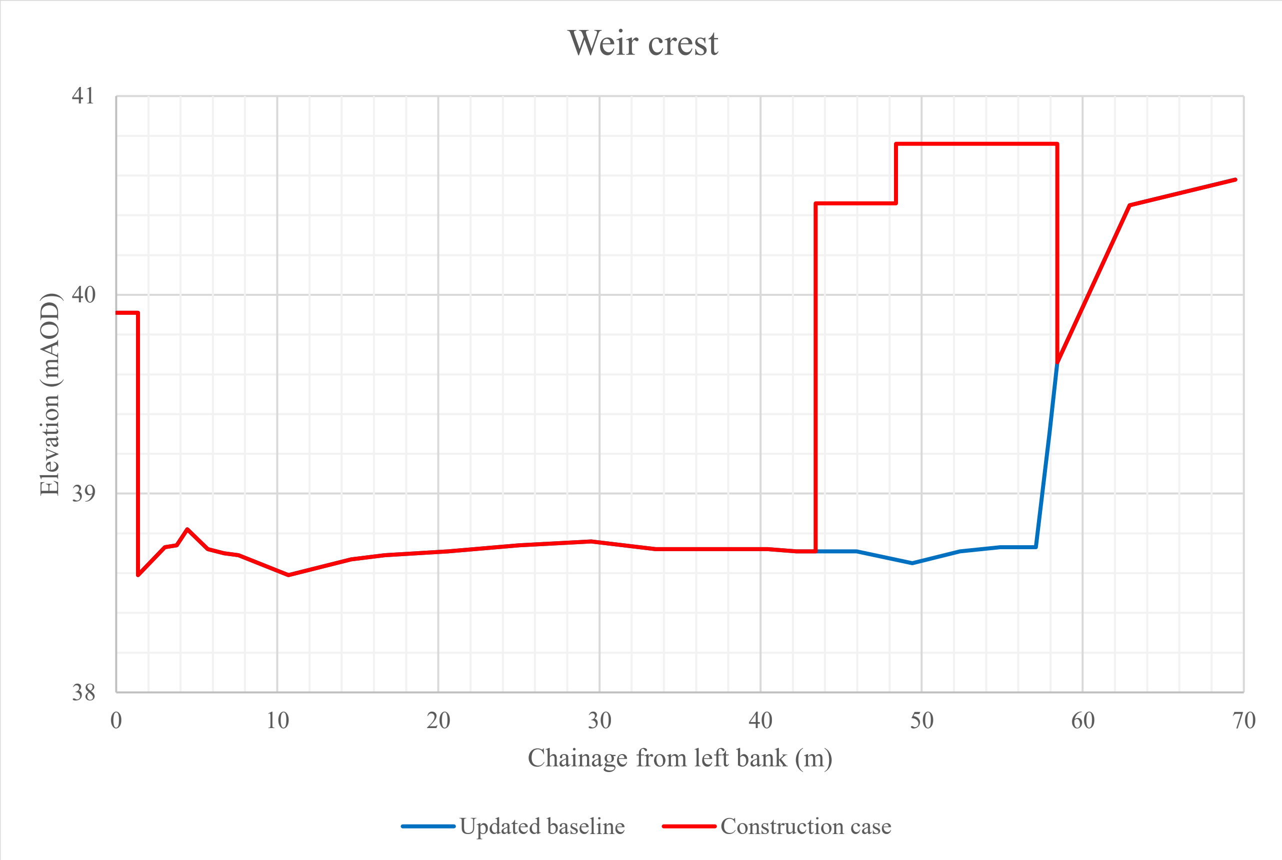

Figure 27 Comparison of updated baseline and construction case cross-section for the weir crest

The baseline model utilising a crump weir unit was also set up for the 42.1m weir crest, with bypass spill units for the left and right bank areas either side of the weir (spill coefficient: 1.0, which is appropriate for overland flow), and an overtopping spill unit for the sheet piles (spill coefficient: 1.7 as the sheet piles would be relatively efficient at passing flow).

4.2.5 Model results

The two weir formulations defined in section 4.2.3 represent two specific cases: a general purpose spill unit which requires a justified estimate of discharge coefficient and modular limit; and a very specific type of weir (a crump weir) with well-defined curves of coefficients and modular limits. The former is defined subjectively, based on the experience of the modeller, and the latter may not apply in this case, the weir geometry not fully matching the assumptions of a crump weir.

As such, it is the patterns of the outcomes that are important as opposed to the actual numbers. For both weir formulation cases, the construction case increased the modelled velocity and depth at the upstream toe and weir crest by a similar percentage although the actual numbers varied (albeit with similar magnitudes). These numbers are still lower than those seen in the model outputs for the largest magnitude event on record (1 in 200 year event comparable to Storm Desmond in 2015). The full set of results are given in Table 1 and 2 of the Technical Note in Appendix A and the high-level results and relative changes suggested by the model are outlined below in Table 4 and the bullet points below.

Table 4 Summary of model outputs for the two different weir formulations

		Original Spill weir formulation		Crump weir formulation	
Location on weir	**Flood event magnitude**	**Average velocity m/s**	**Water depth m**	**Average velocity m/s**	**Water depth m**
Upstream toe	Baseline 5 yr	0.89	3.36	1.00	3.06
	Construction 5 yr	0.95	3.67	1.08	3.31
	Baseline 200 yr	1.23	4.73	1.37	4.46
Weir crest	Baseline 5 yr	2.75	1.03	3.38	0.84
	Construction 5 yr	3.03	1.24	3.71	1.02
	Baseline 200 yr	3.35	2.89	3.85	2.61
Weir face	Baseline 5 yr	3.03	0.94	3.03	0.94
	Construction 5 yr	3.33	1.13	3.33	1.13
	Baseline 200 yr	3.55	2.49	3.31	2.51
Downstream toe velocity	Baseline 5 yr	0.99	3.57	0.99	3.57
	Construction 5 yr	1.01	3.56	1.02	3.56
	Baseline 200 yr	1.21	5.58	1.24	5.61

For a 1 in 5 year event with the "Spill" weir model formulation, the baseline (pre-construction) model results compared to the model results during the construction show that:–

- **Upstream toe:**
  - Average velocity is approximately 7% greater;
  - Greatest depth increases by 0.31m, approximately 9% greater.
- **Weir crest:**
  - Average velocity is approximately 10% greater 2.75 m/s increases to 3.03 m/s;
  - Weir depths increase by 0.19–0.21m, approximately 20% greater.
- **Downstream toe:**
  - Average velocity is approximately 2% greater;
  - Greatest depth decreases negligibly.

For a 1 in 5 year event with the Crump weir model formulation the baseline (pre construction) model results compared to the model results during the construction show that:–

- **Upstream toe:**
  - Average velocity is approximately 8% greater;
  - Greatest depth increases by 0.25m, approximately 8% greater.

- **Weir crest:**
  - Average velocity is approximately 10% greater (same values as Spill model, as Critical flow develops on face;
  - Weir depths increase by 0.18–0.19m, approximately 20–21% greater.

- **Downstream toe:**

- o Average velocity is approximately 3% greater;
- o Greatest depth decreases negligibly.

All parameters experienced by the weir during Storm Christoph are still lower than those experienced during the 2015 Boxing Day floods, with the exception of weir downstream face velocity with the crump weir formation when the construction case is 1% higher than the 200 year return period results. This discrepancy may be due to the formulation of the weir downstream face velocity, which is defined only for free flow and consequently ignores the influence of downstream water levels; in reality, a higher velocity during the transition from free to drowned flow may occur.

It should be noted that the 1D-2D model is not best suited to investigating the varying profile of velocities across a cross-section, as noted in the subsection on velocity profiles in Appendix A. However, it was the best available to the investigation, and is a reasonable approach for understanding relative changes in velocity.

In order to determine the true velocities throughout the cross-sections, it is necessary to use either a scale physical model, or a quasi-dynamic three-dimensional computational fluid dynamics (CFD) model of the Newlay Weir reach. These are relatively expensive options compared to the analysis above, which is likely to be sufficiently accurate for the purposes of comparing the baseline case to the construction case.

Given that the weir has experienced depths and velocities during other flood events in the past which were greater than those experienced during the construction phase, it is unlikely that the increased velocity and depths caused by the narrowing of the section, themselves contributed to the failure. However, the increased velocity being directed across the face of the weir, could have caused erosion to the river bed in front of the weir, as described in Section 5.4.

4.3 Flood Permit

The flood permit for the work[17], issued by the Environment Agency under the Environmental Permitting (England & Wales) Regulations 2016, stated the following condition:

"The top level of the dewatering structure shall be 0.6m below the level of the top of the bank. -No dewatering structure shall be constructed so that it projects more than 30% of the width of the channel, or 10 metres (whichever is the smallest) in a horizontal plane from the top edge of the riverbank to where it is constructed."

As described in Section 4.2.4, the net effect of the sheet piles was to reduce the effective width of the weir crest from 55.7m to 42.1m, a reduction of 25%. This is less than the 30% stipulated by the Permit, but it protruded 13.6m into the channel, compared to the permitted 10m.

As can be seen in Figure 26 and Figure 27, most sheet piles were installed to a height greater than the stipulated 600mm below the top of the adjacent bank. It is acknowledged, however, that defining what constitutes as being the "top of bank" is problematic.

The increased height and extent will have resulted in an increased impact during periods of high water levels on the weir.

5 Geomorphology

This section will summarise the baseline geomorphology in relation to the basal (river bed) topography pre-collapse and also consider any implications of significant erosion or deposition following alterations to the local infrastructure. The basal topography post-collapse will also be discussed, and a comparison made to the baseline geomorphology. The influence of the temporary structure used for construction of the fish pass on flow patterns will also be considered in relation to the geomorphological implications.

5.1 Baseline topographic data and cross-sections

Topographic data available for this assessment was captured in 2015 by Engineer Paddler Designs Limited and in 2018 by Met Geo-Environmental. The topographic data is described and compared below and has been used to extract long profiles along the channel. A post-breach topographic survey upstream and downstream of the weir was undertaken by Storm Geomatics in May 2021[18]. The three topographic surveys vary in levels of detail and coverage. In order for the surveys to be compared, the survey points were converted into Digital Elevation Models (DEMs). During DEM generation, bed elevations between the individual spot heights were interpolated to create a 2D model of the surface of the channel.

5.2 Channel morphology (pre breach)

Before the breach, Newlay Weir (Figure 28) impounded the flow upstream resulting in the water being ponded upstream. Downstream of the weir, the flow was dominated by run flow types with some riffles. It was noted in 2014 that the riffles present within the channel appeared to be fairly static and smothered in silt[19]. The channel bed downstream of the weir was dominated by silt and sand substrate, although coarser sediment was also present (fine gravel through to boulder). Immediately upstream of the weir, the channel was approximately 23 m wide with little variation between wetted and bank top width. At the weir itself, the channel width increases to 56 m as the weir was built skewed across the river to channel water through the goit (which is no longer connected to the river hydraulically in normal flow conditions). In the reach immediately downstream of the weir, the channel width reduces to approximately 30 m.

Figure 28 Newlay weir and Newlay bridge (June 2020)

The channel banks upstream of Newlay Weir, between the weir and the footbridge, are predominantly non-natural, with blockstone walls present along the majority of the banks, with residential properties within the immediate land use on the left bank and tree lined and vegetated on the right bank. Upstream of the footbridge, the banks are composed of more natural material and vegetated with trees and undergrowth. Similarly, downstream of the weir, the channel banks are more natural with grasses, herbaceous plants and trees present along both banks. The left bank is steep (near vertical) throughout this section whilst the right bank has a much shallower gradient.

The topographic survey undertaken in 2015[20] and shown in Figure 29) shows a deep scour pool that starts under Newlay Bridge (bed level approximately at 35.2 mAOD), bed levels then increase up to 37 mAOD at the upstream toe of the weir. Downstream of the weir there is scour pool at 35.1 mAOD.

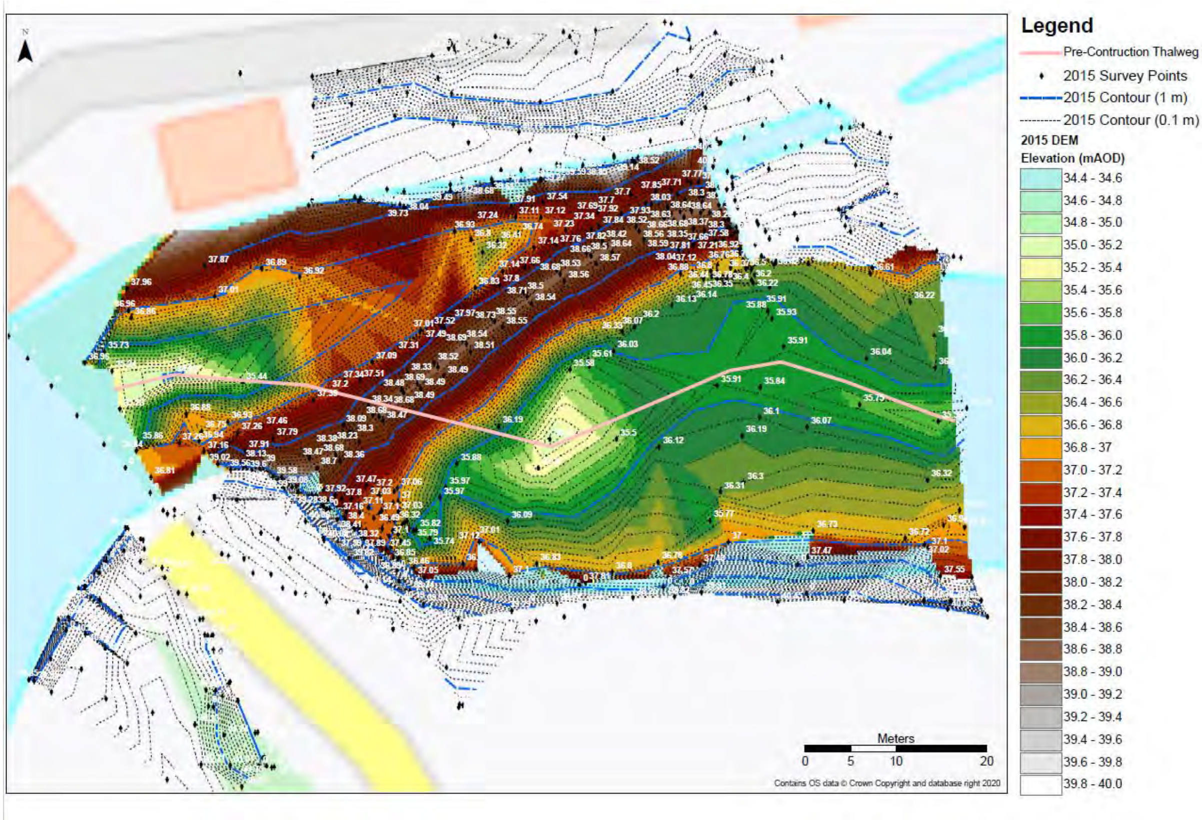

Figure 29 2015 Topographical map of the River Aire surrounding Newlay Weir

The thalweg (deepest part of the river) follows the line depicted in Figure 29; this is the route of the fastest flowing water. Although in 'typical' rivers, this thalweg is often on the outside of the meander bend – due to the tight meander bend, the over-widened channel at the weir and the deep pool under the Newlay Bridge, the faster flowing water at Newlay is closer to the inside of the meander. The channel morphology in 2018[21] (Figure 30) shows a similar pattern of bed levels with the large pool areas being in the same locations with similar depths.

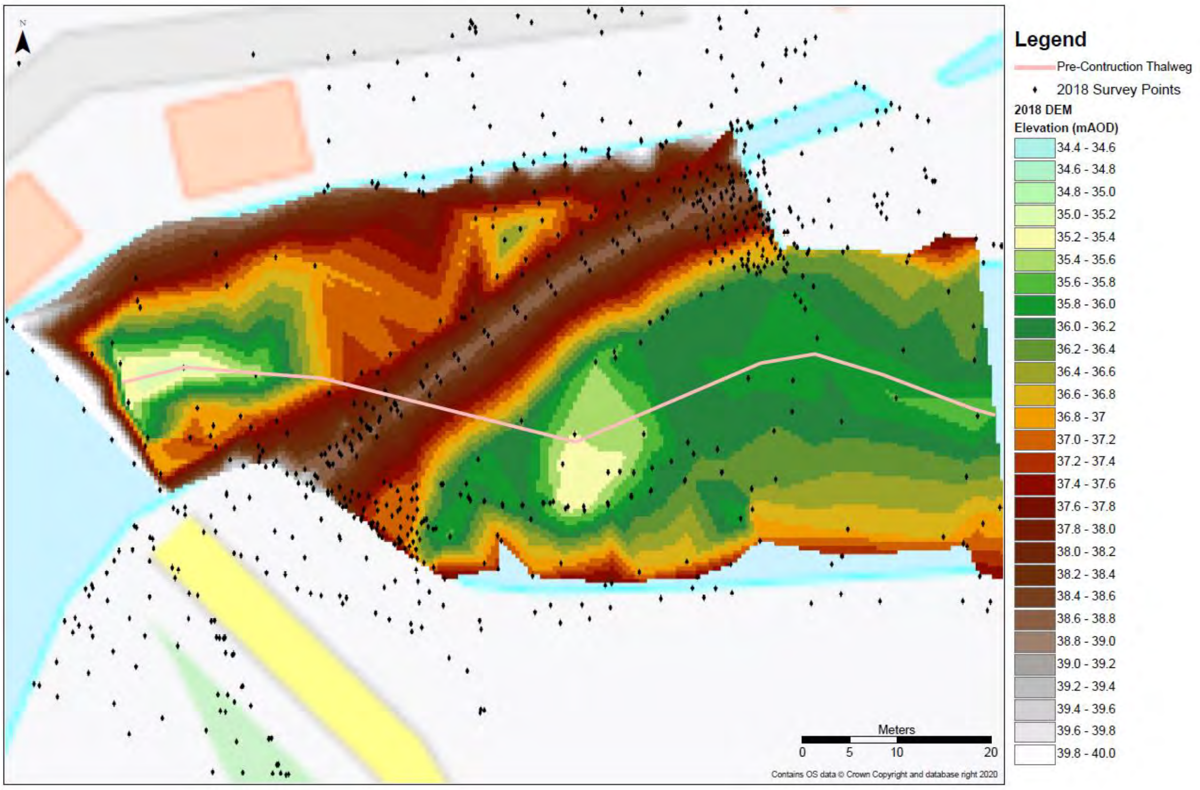

Figure 30 2018 Topographical map of the River Aire surrounding Newlay Weir

In order to compare the topographic data between 2015 and 2018, a DEM (Digital Elevation Model) of difference has been extracted by removing the 2015 raster from the 2018 raster to represent topographic elevation changes between the two dates. Values between –0.1 m and 0.1 m have been discounted.

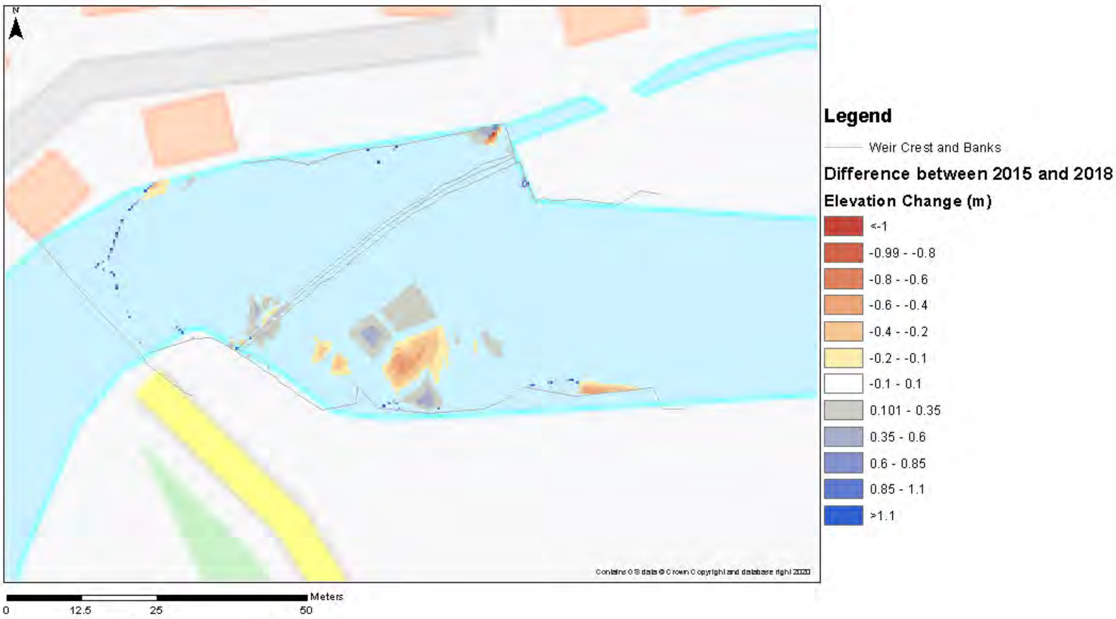

Figure 31 DEM difference plot between 2015 and 2018 surveys

Figure 31 shows that between 2015 and 2018 there was very little change observed in the bed levels both upstream and downstream of the weir. The pool located downstream of the weir deepened by up to 0.8 m at its deepest point with the majority of the pool area deepening by 0.1 m to 0.4 m. There is also some minor accumulation of sediment observed directly upstream of the pool with bed levels increasing by 0.2 - 0.5 m.

Figure 32 shows the long profile of the thalweg for both the 2015 and 2018 surveys highlighting those small changes in bed levels downstream of the weir between the two dates. It is interesting to note that Storm Desmond (1 in 200 year event) occurred after the 2015 survey and it is possible that these changes in bed morphology occurred during this extreme flood event.

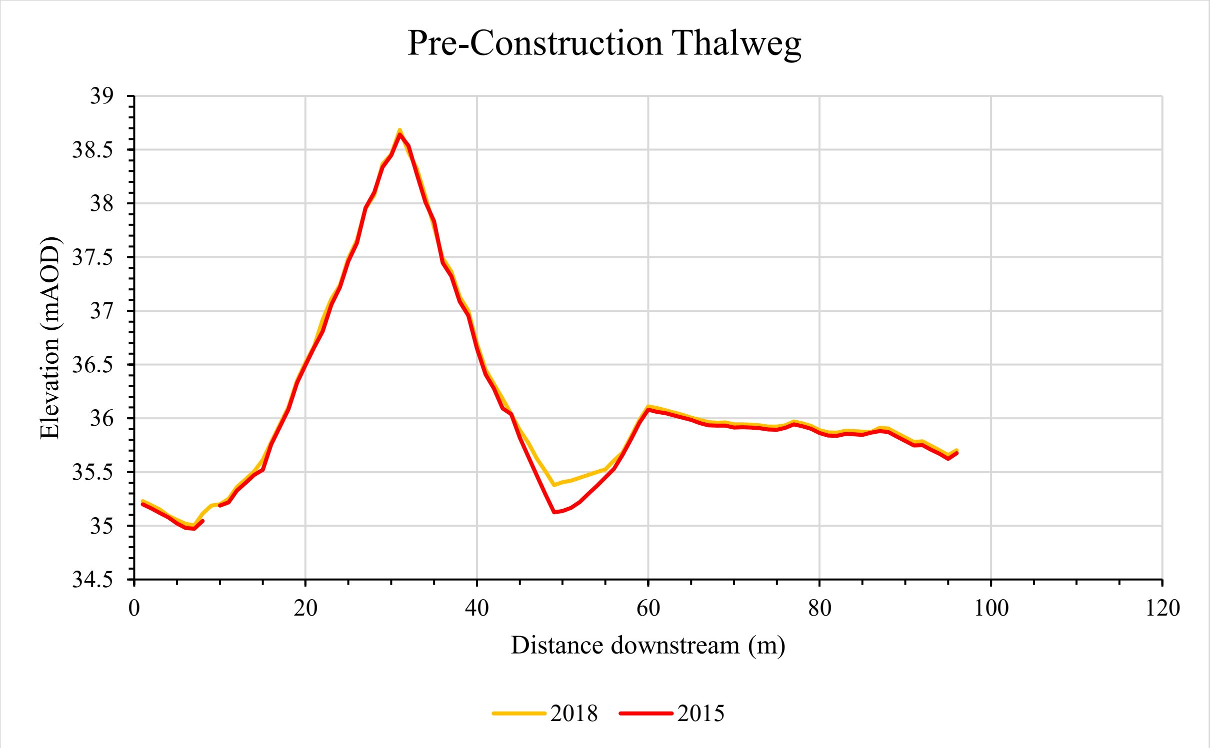

Figure 32 Long profile of thalweg for 2015 and 2018

5.3 Lower weir removal

The partially collapsed Newlay Weir Lower was removed in 2015. A contemporary report[19], written before the removal, acknowledged an unlikely but medium impact risk to infrastructure, a probable but minor impact risk relating to tree loss, bank collapse and knickpoint migration, and a likely but minor impact risk to a reduction in upstream water levels, velocity and erosion increase, and downstream sediment deposition. However, the report did note that there was no evidence of the banks adjusting as a result of the reduction in water levels caused by the partial collapse.

Figure 33 Newlay Weir Lower in 2014 showing the partial collapse of the structure

As shown in Figure 33, Newlay Weir Lower was already in poor condition before its removal, with a collapse on its sloping face and missing the top courses of stone, and the weir also exhibited a small head drop[19]. The removal of the weir is therefore not likely to have affected the hydraulics upstream. The 2015 and 2018 topographic surveys also illustrate that there was little change in bed levels over this period (see Figure 31) which indicates that the removal of Newlay Weir Lower is unlikely to have had an impact on Newlay Weir upstream.

5.4 Channel morphology (post breach)

A post-breach bathymetric and topographic survey were undertaken in May 2021 upstream and downstream of the weir to capture elevation data for the weir, extent of breach, riverbank, fish pass structure, bed levels and alignment of the sheet piles used for the temporary works. A DEM was created by interpolating between topographic and bathymetric survey points (Figure 34). Due to high flows within the breached section of the river, it was not possible to capture bathymetrical data in this area (area noted as "too deep to survey" within Figure 34). However, the bed elevation values of the channel at this location were extrapolated from the survey points along the boundary of the breach area to create a digital surface.

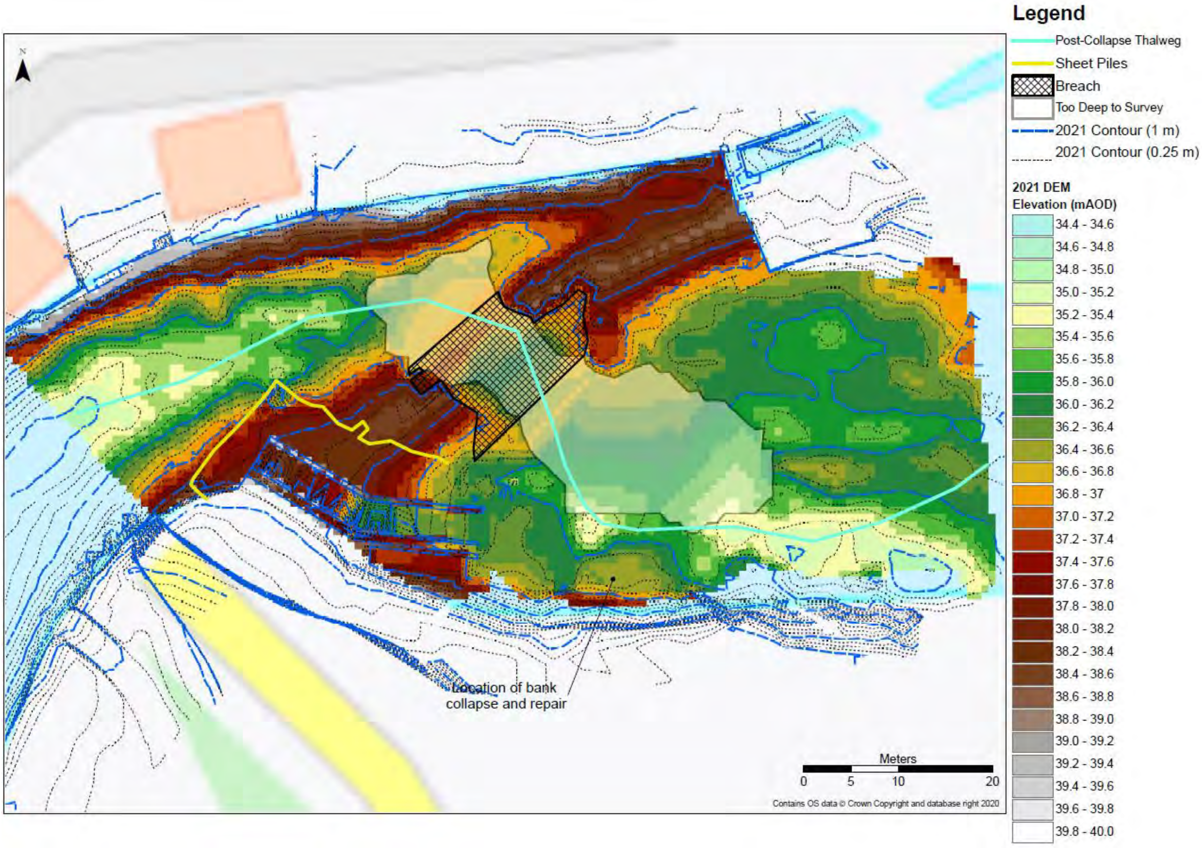

Figure 34 2021 Topographical map of the River Aire surrounding Newlay Weir

Figure 35 Breached Weir (June 2021)

As can be seen in Figure 35 and the topographic survey data, sediment has accumulated immediately downstream of the weir creating a riffle feature, filling in the downstream scour pool with material likely to comprise mainly of the collapsed weir. Further downstream, a bar has developed (Figure 36), likely to be formed from material entrained downstream during high flows that occurred after the initial breach and/or material from the bank collapse that occurred on the right bank during the high flows after the breach (see lighter colour stone that has been used to repair the failure).

Figure 36 Bar formed downstream of weir

Upstream of the Newlay Bridge (Figure 37) water levels have dropped significantly and are now fast flowing (in comparison to the ponded reach that was present when the weir was intact).

Figure 37 View of the channel upstream of the footbridge

There is still little geomorphological diversity in this reach, apart from the exposed foundations of the Monks Bridge crossing that can be seen in the upstream extent of the photo. The recent stonework on the left bank has been installed to protect the toe of the bank which had been exposed as water levels had dropped, where properties were considered to be at risk.

A comparison of the 2018 and 2021 topographic data was also carried out using the same process as the pre-breach assessments with the results shown in Figure 38.

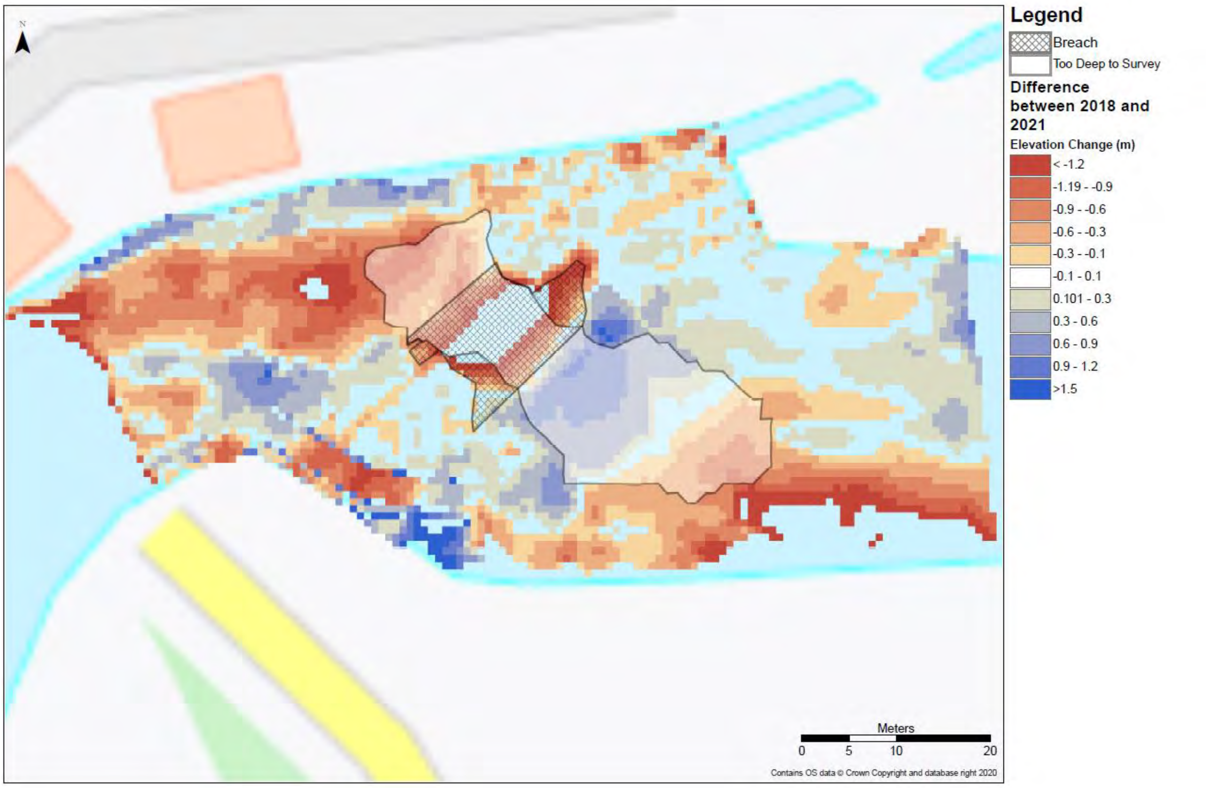

Figure 38 Net elevation loss and gain at Newlay Weir between 2018 and 2021

As observed in the site walkovers and photos, the DEM difference plot shows that following the breach there has been significant change to the channel topography. Firstly, upstream of the weir, erosion of the bed (approximately 1 m lowering) has occurred within the middle of the channel, where the large scour hole under the Newlay bridge, extending towards the weir, was observed in 2015 and 2018. This feature has extended further into the centre of the channel, creating a new thalweg through the breach in the weir. This large-scale scour is likely to have happened predominately during high flows after the initial breach as water levels dropped and the impounding nature of the weir was lost.

There is also a large area of erosion on the right bank downstream of the weir which again aligns to the main flow of water post-breach. It is likely that the bed immediately in the path of this flow (Figure 39) would have been deeper after the initial breach, but the bank collapsed at this location (reported 08/02/2021) and was repaired (18/02/2021) with stone (as noted in Figure 36) filling in the deeper scour area that would have formed, resulting in the bank collapse.

However, there is still a large scour hole further downstream of the initial flow path, showing the power of the water during the initial draw down of water post collapse and during further high flow events. There are a number of areas upstream of the weir where bed levels have changed suggesting that sediment has been deposited in these areas and in some cases with approximately 0.6 – 0.9 m increase in bed levels.

Deposition is also evident with the plots within the vicinity of the breach although, as the area where the main flow of water is currently flowing, water levels and velocities meant that the bathymetry of the bed was not directly surveyed in 2021 in the area depicted in Figure 38.

However, the DEM was able to extrapolate levels from surveyed points and provide an estimate of current bed levels, with the exception of the weir itself, allowing an estimate of bed level changes in this area to be made.

As such it is likely (as was observed at lower flows) that the pool identified downstream of the weir in both the 2015 and 2018 topographic surveys has been infilled with a bed level changes of approximately 0.3 m to 0.9 m.

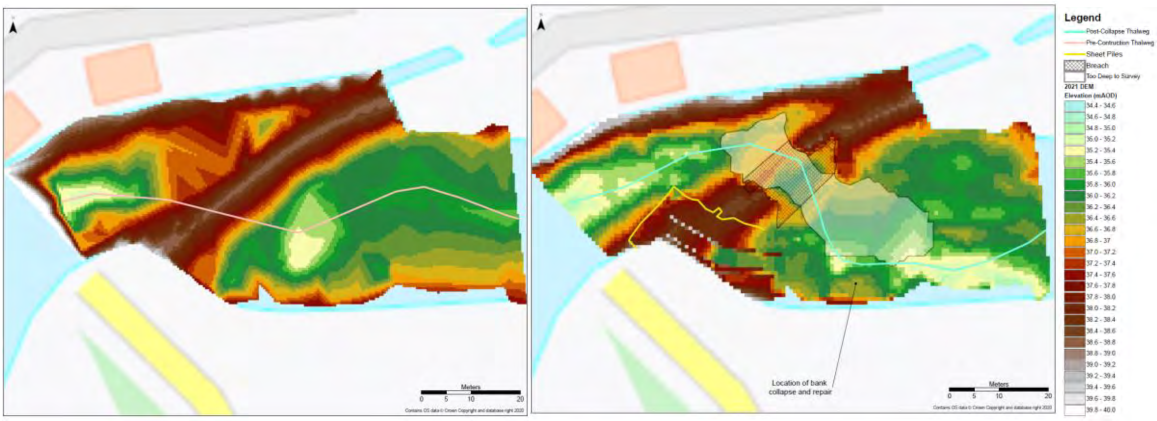

Figure 39 DEM plots from 2018 and 2021 with corresponding thalwegs

Analysis of the DEM outputs and topographic survey allows long profiles to be extracted that show the route of the thalweg both pre and post collapse (Figure 39). The long profile presented in Figure 40 compares the 2015, 2018 and 2021 elevation data along the post-collapse (2021) thalweg, clearly showing how the bed levels have changed since the collapse of the weir and supporting the discussions above.

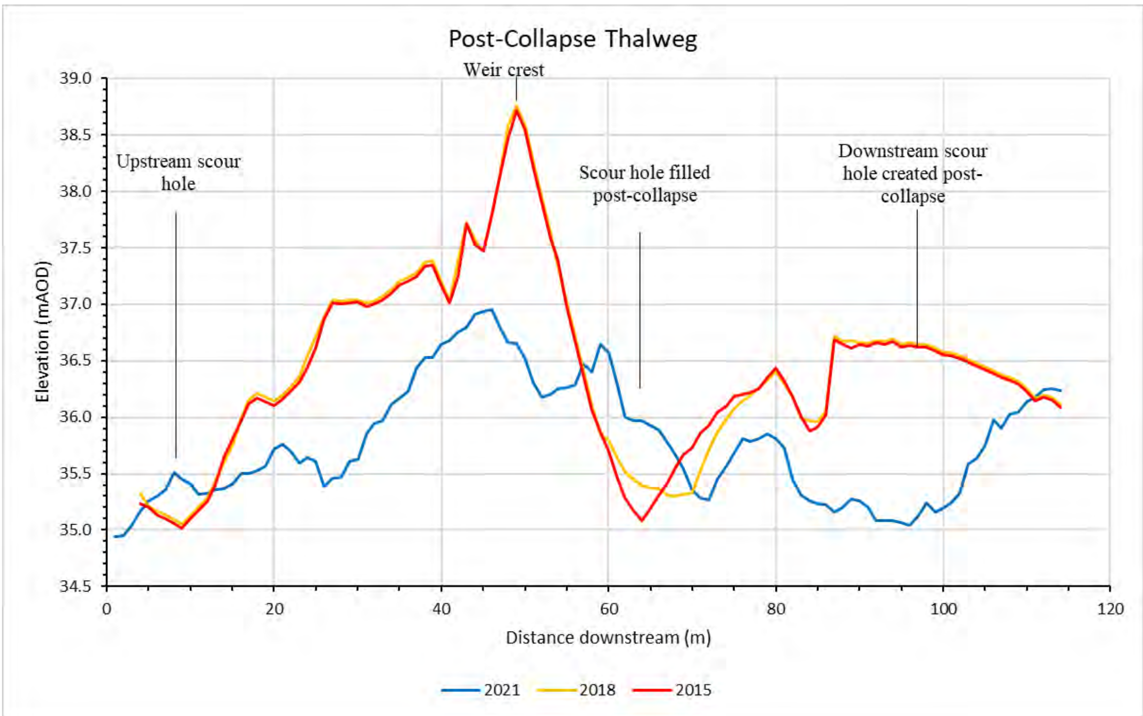

Figure 40 Long profile for post-collapse thalweg (2021) with 2015, 2018 and 2021 DEMs

5.5 Geomorphological and hydraulic response to the temporary works

In order to gain a better perspective on the geomorphological and hydrological processes that may have contributed to the collapse of the weir, it is important to look at the positioning of the temporary works and how they may have changed the flow paths and erosion process in the river, in particular during periods of high flows.

The DEM difference plot of the 2015 and 2018 survey dates shows that there was no significant change in channel morphology around the weir despite the occurrence of the 2015 Boxing Day Flood, which would have had the ability to move larger sediment than a typical annual winter flood event. There were minor changes between 2015 and 2018 evident within the channel, but overall, the channel morphology remained the same. As such, it is unlikely that the baseline channel morphology was a driving force in the collapse of the weir, especially as it had remained stable during a recent extreme flood event.

The 2018 survey long profile data (Figure 41) shows that the large scour hole under Newlay Bridge was over 16 m away from the toe of the weir (surveyed in at 37 mAOD) with a difference of 2 m from the bottom of the scour hole to the toe of the weir (1 in 8 slope).

Downstream of the weir there is a similar profile and approximately a 1 in 5.5 gradient from weir toe to bottom of scour hole. It is unlikely that the gradient of this profile alone would have resulted in a collapse of the weir without some other influencing factor. Therefore, we need to understand what might have changed and how this may have resulted in the collapse of the weir.

Figure 41 Long profile 2018

During the temporary works and the construction of the fishpass, sheet piling was installed to create a dry working area. The installation of this is discussed in Section 3, which details how the temporary works were undertaken and describes the installation process.

The sheet piles were in place for a period of 5 months prior to the collapse of the weir. Images of the construction site during 'normal' flow conditions show that most of the disruption to flow dynamics occurs on the interface between the sheet pile and the downstream face of weir.

However, during higher flow events, the flow dynamic appears to change with the upstream corner of the sheet piling influencing the flow more as shown in Figure 42, forcing the main flow of the water towards the centre of the weir.

Figure 42 Photos taken on 14th January (public survey photo)

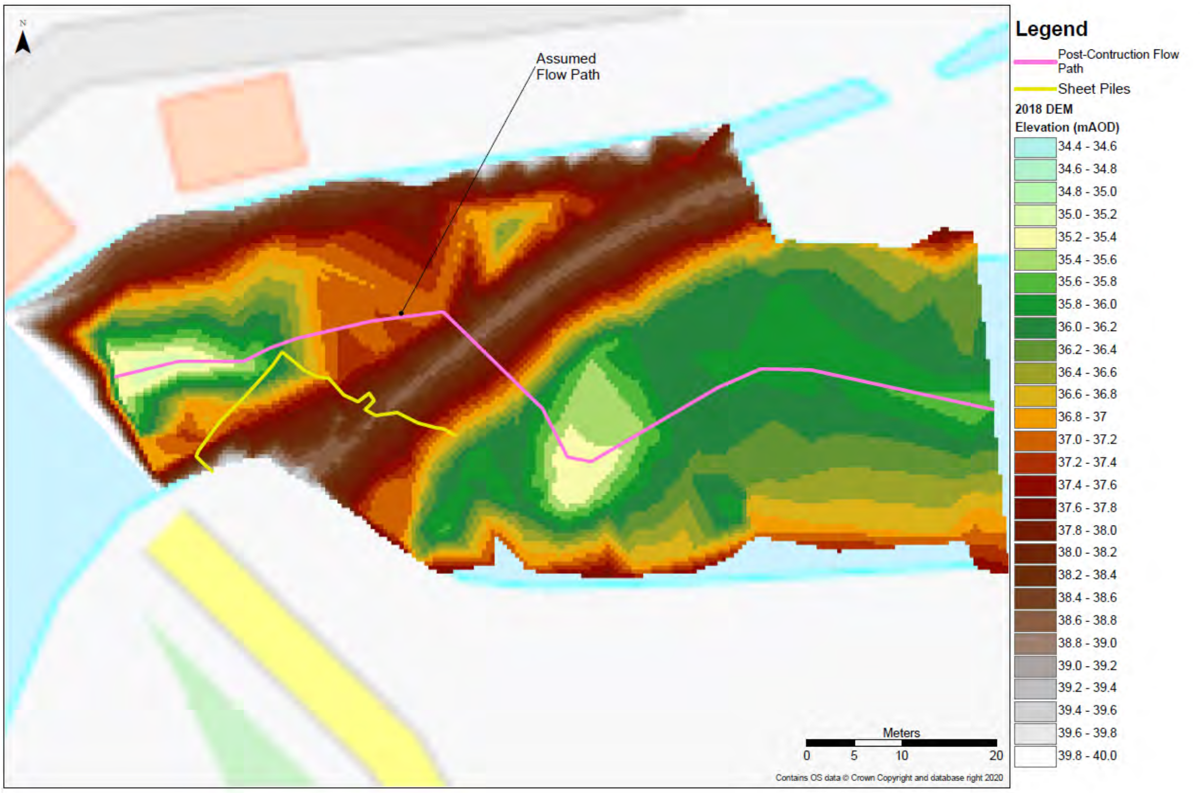

Figure 43 2018 DEM with sheet piles and assumed flow path during construction

With the raking angle of the sheet piles directing the main flow across the face of the weir, as shown in Figure 43, and the reduction in channel functional width by 25%, resulting in an increase in velocity over the front face of the weir of 10% (as detailed in Section 4.2.5) during a 1 in 5 year event; it is possible that these changes have caused erosion at the toe of the front face of the weir resulting in the collapse of weir. The angle of deflection can be observed during low flows in relation to the area of collapse, in Figure 62. At low flows, the redirected flow almost reaches the area that the collapse initiated in. At higher flows (and therefore velocities), the zone of influence of the redirected flows would be likely to reach the red line in Figure 62.

At high flows, the video and photographic evidence (see Figure 44) shows that flow was deflected across the front of the weir, was very turbulent and rolling waves were present. The rolling waves would not have been present without the raking piles. This circulating action, in conjunction with the force of the redirected jet of water, at a velocity 10% higher than it would have been without the piles, would have caused changes in flow patterns and focused flow in areas that it would not usually be focused. This could therefore have resulted in erosive forces, causing toe scour that may have contributed to the collapse of the weir.

Figure 44 Turbulence and flow deflection across face of the weir – 21st Jan 2021

As discussed in Section 4.1, there were at least four high flow events between the date that the sheet piles were installed and Storm Christoph. Each of these events could have caused progressive erosion and movement of material at the downstream toe of the weir as a result of altered flow patterns.

As outlined in Section 4.3, the sheet piles were installed to a higher level and further into the channel than permitted. The extension into the channel will have further increased the reduction of the functional channel in comparison to the temporary works design.

Furthermore, the increase in height of the sheet piles will have resulted in more of the flood flows being retained in the channel for longer; the higher sheet pile meaning that it wouldn't completely overtop as outlined in the temporary works design. This will have increased the velocity and had the effect of making the turbulent conditions last longer than would have been the case if the sheet piles had been installed to the permitted level, which will have overtopped earlier in the flood event, removing the constriction on the channel. As shown in Figure 45, even when the majority of the fish pass is submerged, the higher sheet piles are still restricting flows and increasing turbulence, redirecting flows towards the centre of the channel.

Figure 45 Photo taken on 7th February (public survey photo)

The morphological response to this change in flow dynamics and increase in velocities can also be interpreted using the images in Figure 46 and the long profile in Figure 47 shows the DEM generated for the 2018 data with two different flow routes for pre-construction and during-construction.

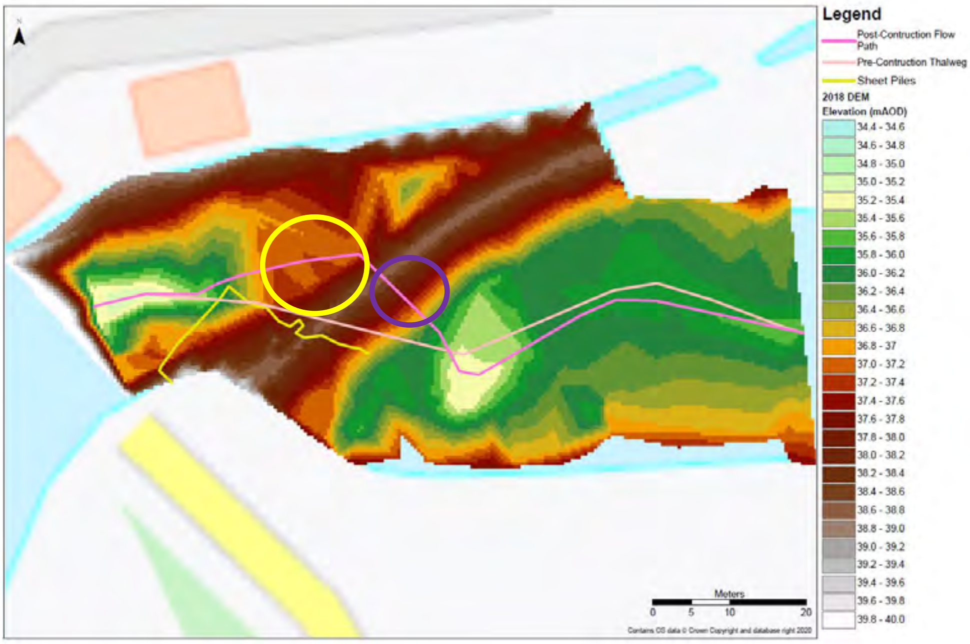

Figure 46 DEM plots 2018 with pre-construction and during-construction flow paths

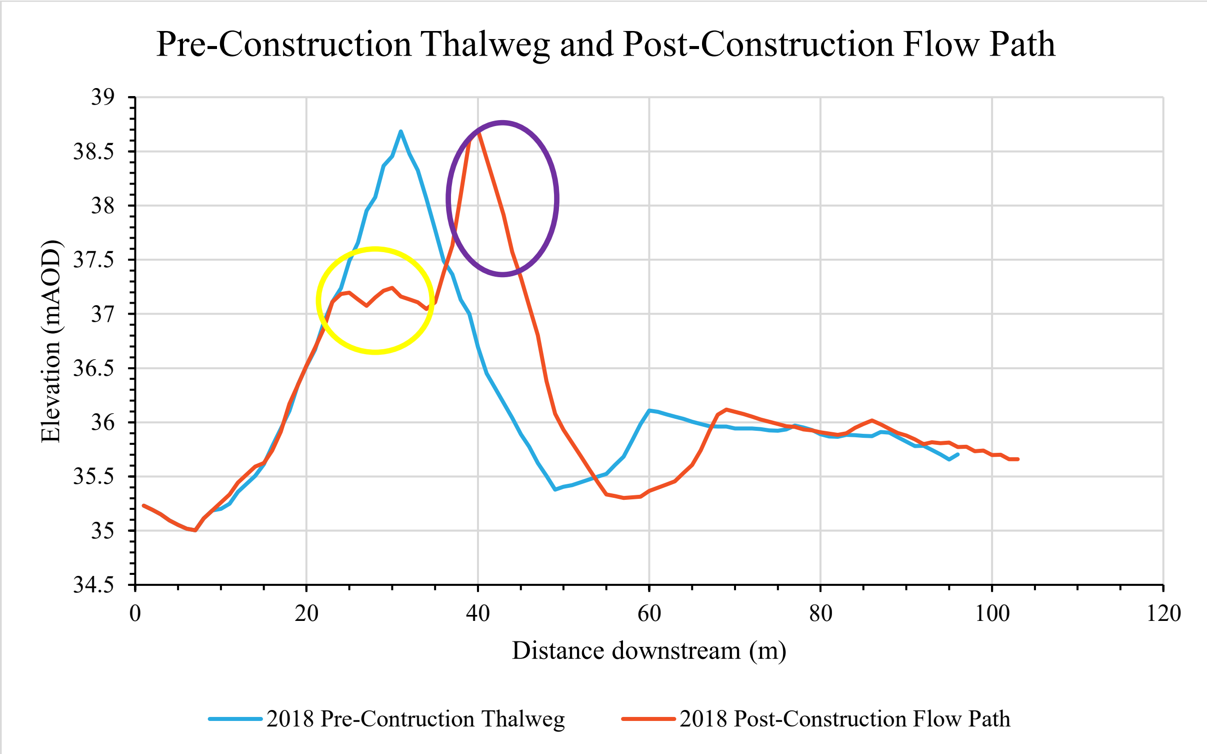

Figure 47 2018 DEM with long profile pre and during construction

The first flow route (blue line) as expected, follows the deepest part of the channel and flows over the crest of the weir in the general vicinity of the fish pass construction area. Due to the presence of the sheet piling, the second flow route during construction is altered and flow is directed towards the centre of the weir crest. It is difficult, without modelling, to determine the exact route of this flow over the weir crest – but it is likely that it would be drawn towards the deep pool area downstream of the weir and return to a similar flow path as the pre-construction route.

The river bed upstream of the weir, highlighted by the yellow circle on both Figure 46 and Figure 47, and the downstream face of the weir (purple circle) will be the areas with the main focus of the flow and most affected by the changes associated with the restriction in channel width discussed in Section 4.2.5. The locations of these areas of focus are also shown in Figure 48 for both the 2018 DEM and 2021 DEM. These areas are where the initial breach took place, confirming that the restriction in the channel is likely to have had some influence over the failure of the weir.

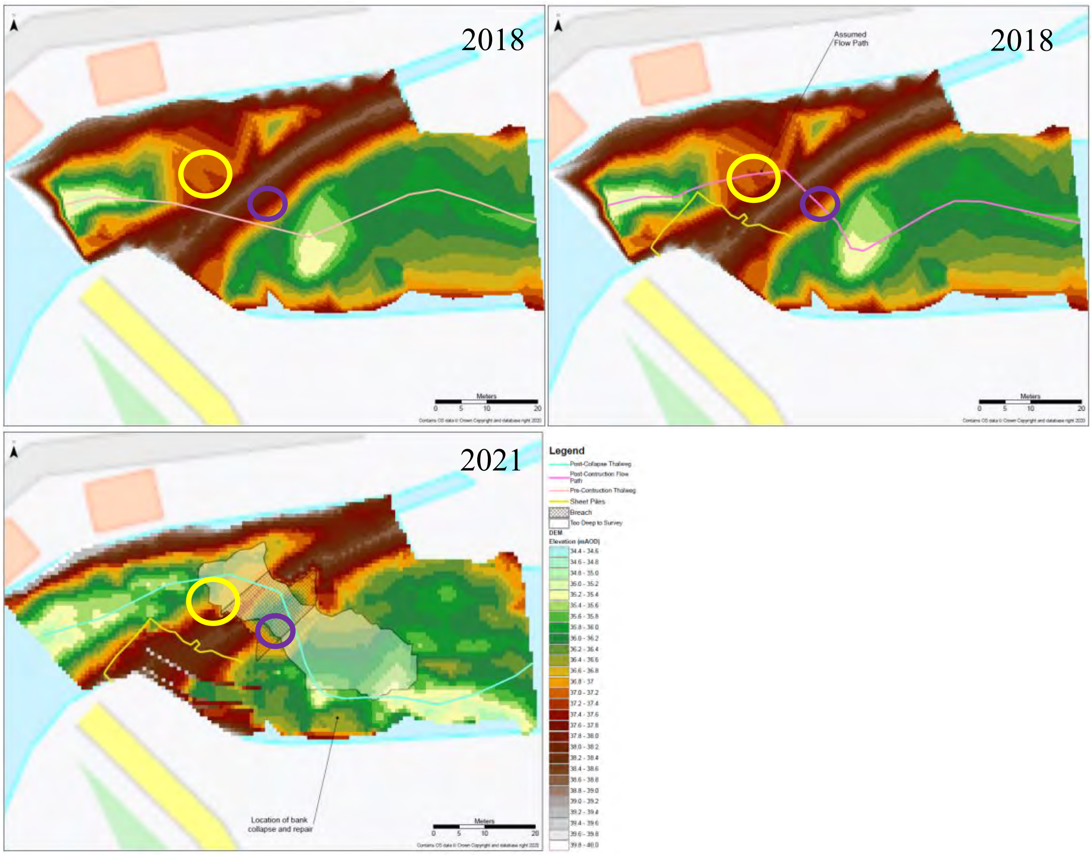

Figure 48 DEM plots from 2018, 2021 with varying thalwegs/flow paths (areas of focus in purple and yellow)

6 Weir collapse

6.1 The timeline

A timeline has been created for events that took place before and after the initial collapse of Newlay Weir. In general, the timeline details key event and date information relevant to the investigation into the cause of the weir collapse. A detailed version of both timelines can be found in Appendix B. The events captured in both timelines include storm events, reports published, topographic survey dates, photographs of the weir from social media, members of the public[22] and from Arup's site investigation. Events referred to in the pre-collapse timeline have been discussed throughout previous report sections.

From the timeline of events post-initial failure, it is evident that the initial collapse began on 21st January 2021 where a concave depression is visible in video footage taken by a member of the public at 8:45am.

Figure 49 Stills from a video captured by a member of the public showing the initial collapse taken at 8:45am on 21st January

The collapse then progressed on 25th January as the river level subsided after Storm Cristoph. On this date, the concave depression on the weir is more pronounced. The 25th January is also the initial date when it is understood that a member of the public contacted the Environment Agency to report damage to Newlay Weir.


Figure 50 Photos taken on 25th January provided for the purpose of the investigation

Photographic evidence provided by the public for the 3rd and 4th February shows the concave failure retreating upstream, breaching the crest of the weir.


Figure 51 Photos taken on 5th February provided for the purpose of the investigation

On the 5th February, the failure has retreated upstream, breaching the upstream face of the weir, creating an opening in the weir and exposing the weir's timber frame. Between the 6th and 7th of February, the weir has fully breached with a clear path for the flow to move through.


Figure 52 Photos taken on 7th February (public survey photo)

On the 8th February, the Environment Agency were notified of the total weir breach. A site visit was then conducted the following day on 9th February by the Environment Agency.

6.2 Possible mechanisms of weir failure

The common causes for weir failure include:

1. **Failure of direct uplift** - If the weight of the weir is insufficient to resist water uplift pressure, this can result in a bursting of the weir floor. This will reduce the effective length of the impervious floor and can cause failure of the weir.

2. **Failure of piping and undermining** – This occurs when there is an act of seeping between the upstream and downstream face of the weir. This results in an exit gradient of water seeping under the base of the weir and creating a weak point in the soil underneath the weir. This progressive erosion at the upstream face results in formation of a 'pipe' underneath of the floor of the weir.

3. **Hydraulic jump** – A hydraulic jump is formed at the downstream (toe) of the weir. A hydraulic jump can develop on a weir due to change in flow, water level and flow direction. The phenomenon of hydraulic jump is water's way to gain stability in response to an unstable, high energy, condition. If this energy is not correctly managed, this can result in erosion to the toe.

4. **Scouring of the downstream toe** – A change in flow can result in scouring of the weir toe, this scouring can undermine the toe and result in a slip of the front face of the weir and result in failure.

6.3 Predicted collapse mechanism

Based on the investigations described in this report, the observations of an expert team and supporting information, it seems likely that scouring of the downstream toe contributed to the first stage of the collapse mechanism of Newlay Weir. However, all possible failure mechanisms highlighted including hydraulic jump, piping/undermining and failure of direct uplift played their role in the weir failure. This section describes a potential scenario of the sequence of collapse of Newlay Weir.

6.3.1 Stage 1 – 21st January 2021

Stage 1 occurred on the 21st January; the photograph taken by a local resident (Figure 54) shows the front face of the weir has dropped and slipped. This movement then allows water to penetrate the front face of the weir (Figure 53), this will further accelerate the erosion underneath the weir and change the position of the hydraulic jump, further accelerating erosion.

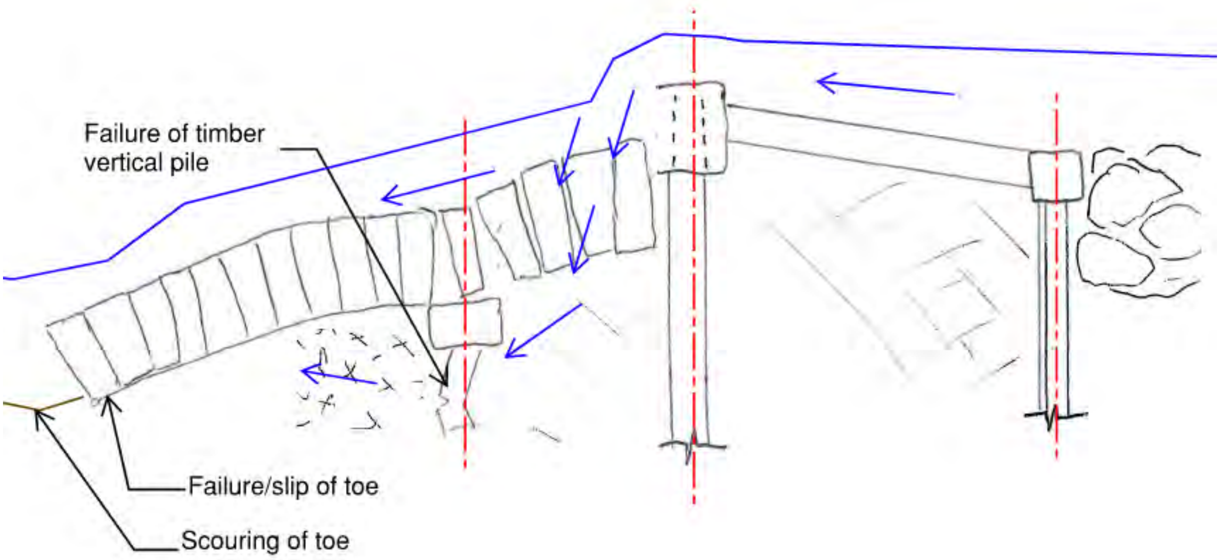

Figure 53 Stage 1 of the Predicted Collapse Mechanism

Figure 54 Initial weir failure (21-01-2021)

6.3.2 Stage 2 – 25th January 2021

Stage 2 was the process of further erosion to the front face (downstream) of the weir. Due to the gaps created by the toe failure the front face is no longer impermeable, and water can travel underneath the weir. As shown in Figure 55, the water is now weakening the structure by eroding the soil below and damaging the timber substructure underneath.

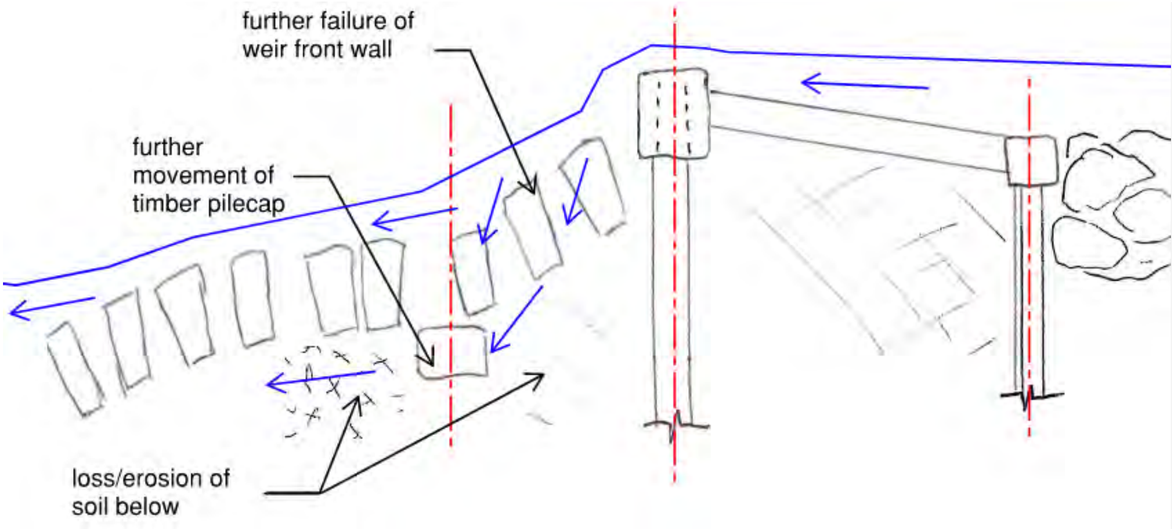

Figure 55 Stage 2 of Predicted Collapse Mechanism

Figure 56 Front face failure (25-01-2021)

6.3.3 Stage 3 – 4th February 2021

Stage 3 is when the front face of the weir section has been lost due to erosion, and the passive force which was keeping the weir together has been removed. Pre-failure, the passive force for the weir came from the solid material (soil, stonework and deposits) that resists the active water pressure and soil behind the weir crest. The photograph (Figure 58) taken on the 4th

February 2021 shows that the main timber crest and vertical timber piles are the only element which are resisting the active force. The timber substructure for Newlay weir was designed as a composite structure (fill and timber) and therefore does not have enough capacity to resist this force.

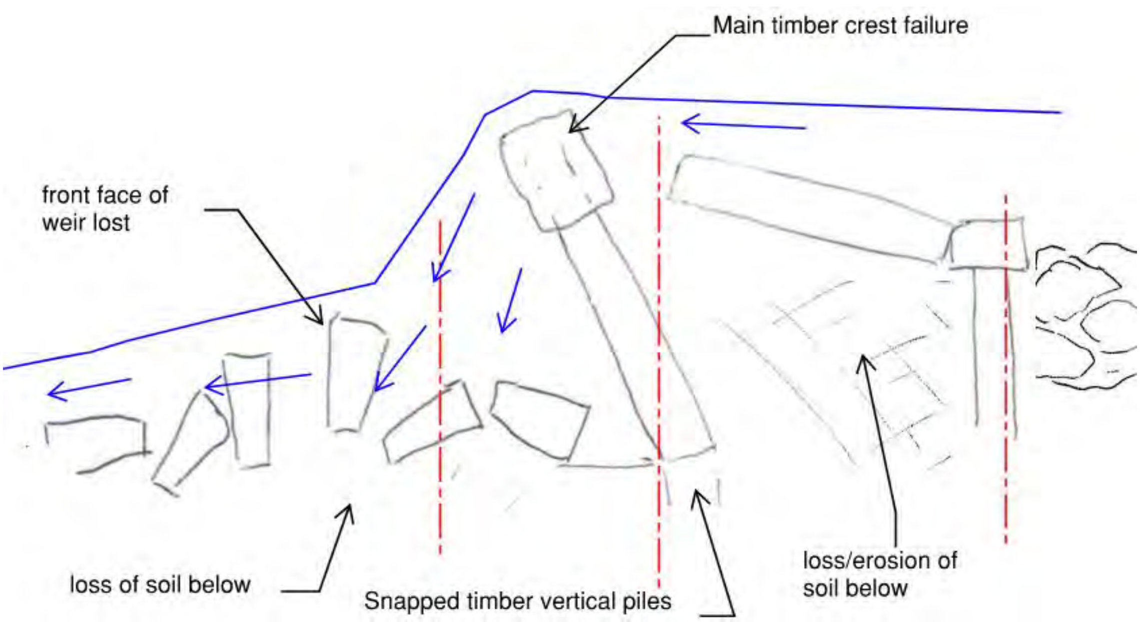

Figure 57 Stage 3 of Predicted Collapse Mechanism

Figure 58 Progression of front face failure (04-02-2021)

6.3.4 Stage 4 – 5th February 2021

Stage 4 is 24 hours after stage 3, the timber crest and timber vertical piles could not resist the active water pressure and failed. In Figure 59, the broken tops of the timber vertical piles are visible. The water has now breached the weir, and this then allowed the water to further erode around the failed section of the weir as shown in Figure 60.

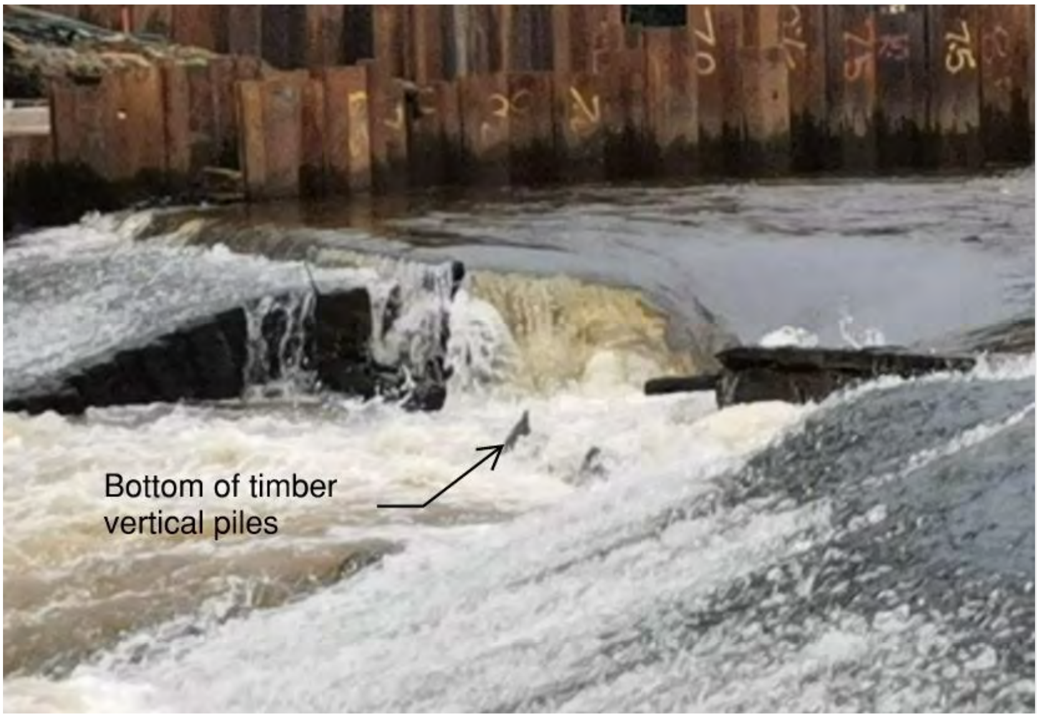

Figure 59 Progression of failure (05-02-2021)

Figure 60 Progression of failure (06-02-2021)

6.3.5 Stage 5 – Breached Weir and Total Collapse

Stage 5 is the when the weir is breached and the water is now flowing though this breached zone of the weir. This zone will continue to widen and the edges of weir will continue to widen until the river channel finds its natural width. Figure 61 shows the makeup of the weir and shows the zone of the breach on 11th May 2021. Figure 62 shows a plan view of the breach pre 06-02-2021 and post 06-02-2021.

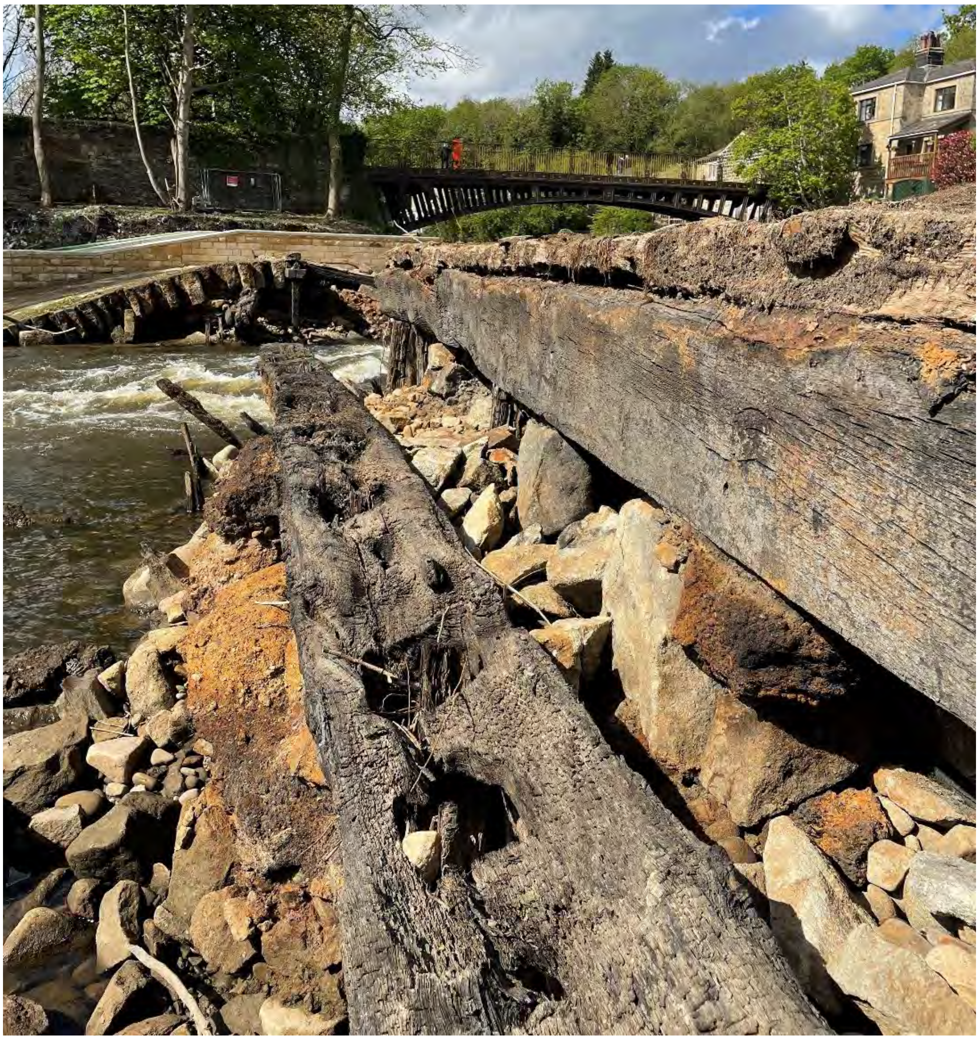

Figure 61 Site visit (11-05-2021)

Figure 62 Plan view of breach progression (red = pre 06-02-2021, yellow = post 06-02-2021)

7 Summary and conclusions

The investigation has assessed the factors that have potentially contributed to the collapse of Newlay Weir.

The condition of the weir prior to the Fish Pass construction showed degradation of the weir in the location of the fish pass before the works started although it was still intact and functioning. There was no evidence of degradation within the area of the breach.

The analysis of the survey data shows that there had been no significant changes in channel morphology around the weir. As such, it is unlikely that the baseline channel morphology influenced the collapse of the weir as it has remained stable over time and there are no significant scour holes immediately upstream or downstream of the weir face that could have resulted in its instability.

The narrowing of the weir section as a result of the piles, although causing a general increase in velocity and depth over the weir, is not thought to be sufficient to contribute to the failure. However, photographic and video evidence of the flow patterns produced during high flow events, as a result of the angled, raking alignment of the sheet piles across the face of the weir, could have caused erosion to the front of the weir.

The piles caused the main flow path (with the greatest velocities) to move towards the centre of the weir and the location of the collapse and could have contributed to erosion in front of the weir.

Following the installation of the sheet piles, there were four notable high flow events, two of which caused a contractual delay to the project. Also, the river levels preceding Storm Christoph were elevated for over a week (again resulting in a delay). All these events could have caused erosion as a result of the flow deflection.

If the events did cause erosion to the bed in front of the weir, then this could result in the undermining of the toe of the weir and subsequent progressive collapse under high flow loading. The potential collapse mechanism, supported by photographic information, has been described.

The review of the available information relating to vibration for the installation of the sheet piles and empirical prediction from the PPV analysis, suggests a low risk that the vibration contributed to the damage to the weir. Additionally, there was a period of five months between the pile installation and first evidence of collapse during which there were high flow events. It is considered that if vibration had been an important factor in the breach, it would have been apparent sooner.

The weir is constructed from a matrix of timbers, with longitudinal timbers joined together across the length of the weir. The installation of the sheet piles through the body of the weir could have displaced these longitudinal timbers and caused weakening of the structure.

Table 5 details the potential factors identified and summarises the likely relative significance of each of the factors.

Table 5 Relative Significance of Potential Factors likely to have contributed to the collapse.

Potential Contributory Factor	**Relative Significance of Contribution (None, Very Low, Low, Medium, High, Very High)**	**Comments**
Tree/Debris Impact	None	No evidence of a tree or debris strike
Lower Weir removal	None	No change to the bed levels in the time between the 2015 and 2018 surveys to indicate any impact of the lower weir removal on the Upper Newlay weir. The lower weir was already in poor repair and with a small head, so there is unlikely to have been much change to the hydraulics in the area.
Ongoing Scour	Very Low	No evidence of recent changes in the channel morphology which could have undermined the weir resulting in collapse.
Weather / Hydraulic conditions	Low	The flood event preceding the collapse (Storm Cristoph) was a significant, but not a particularly rare event. There were 3 flood peaks over the Environment Agency Threshold (POT) for the Armley gauge, over the period August 2020 and January 2021 when the piles were in place. This occurrence puts the flow record over the period into the 33rd percentile, meaning that 33% of years have more peaks over the threshold than were experienced during the construction period.
Flood Permit	Medium	The sheet piles were installed to a higher level and further into the channel than permitted. This will have increased the velocity and had the effect of making the turbulent conditions (described below) last longer than would have been the case if the sheet piles had been installed to the permitted level / extent.
Scour as a result of the presence of the sheet piling	High	The temporary works (sheet piling) resulted in a reduction in weir crest length by 25% and increased flow velocity by around 10% over the crest of the weir. Video evidence shows that during the peak of the flood the flow was very turbulent over the weir and flow was deflected across the front of the weir as a result of the raking angle of the sheet piles. In our opinion, this redirected jet of water would have caused changes in flow patterns and focused flow in areas that it would not usually be focused, and that could have resulted in toe scour, contributing to the collapse of the weir. In addition, the presence of the piles pushed the main flow and associated high velocities towards the centre of the weir where the collapse occurred. This could have contributed to erosion in front of the weir. There were four periods of high flow following the installation of the sheet piled cofferdam, prior to

		Storm Christoph. These events would have caused similar flow conditions and are likely to have caused ongoing erosion.
Vibration caused from installation of the sheet piles	Low	It cannot be categorically stated whether vibration caused, or contributed to, the weir collapse as there is no way of accurately knowing the vulnerability of the structure, or the underlying ground, to vibration. Predictions made using British Standards for estimating vibration show a low risk that vibration could have exceeded the criteria at which there would be displacement within the structure and differential settlement. There was a period of five months between the pile installation and first evidence of collapse during which there were high flow events. It is considered that if vibration had been an important factor in the breach, it would have been apparent sooner.
Condition/age of the weir	Low / Medium	The weir prior to any construction works carried out for the fish pass showed historical repair and missing stone setts and eroded concrete to the right side of the weir in the location of the fish pass. This was however removed following excavation within the cofferdam. The pre-condition visual structural survey did not show any observable damage to the weir in the area of the collapse. It is possible that hidden defects were present underneath the weir as the inspection did not carry out any intrusive works, such as structural core samples. However, any hidden defects underneath the weir such as scouring would generally impact the face of the weir. This would happen because the stones of the weir would drop to fill this void. This would then have been noted in the pre-condition visual survey. Therefore, it is considered that the weir was in Fair condition and would not be the sole reason for the weir failure.
Installation of the sheet piling and cutting through the weir core.	Low	The weir itself is not one singular element, but a chain of elements which create a structure to resist the water pressure upstream. The act of cutting through the weir could have damaged the substructure and increased erosion at this location, however as the failure did not occur at the sheet piles this shows the cutting through the weir was not a significance factor for the weir collapse. It is understood that the top timber beam was cut previously from historical repairs. The site walkover post event did not show evidence of displaced beams or setts between the pile line and the breach which indicates that the mechanical movement of the weir structure associated with piling is not considered to be a contributory factor.

In conclusion, our investigation has found that the condition of the weir prior to construction was not a likely main contributory factor and there were no previous erosive processes observed that could have reduced the weir's strength.

We consider that the fish pass temporary works, in the form of a "raking" sheet pile design across the front of the weir, is likely to have contributed to the failure of the weir by deflecting flows across the face of the weir. In addition, the presence of the piles pushed the main flow and associated high velocities towards the centre of the weir where the collapse occurred. This could have contributed to erosion in front of the weir.

The height and extent of the sheet piling exceeded that allowed by the Flood Permit. This would have the effect of prolonging the erosive hydraulic conditions generated during high river flow events as a result of the temporary works.

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[27] Photos provided by the general public both during the Investigation and as a result of the formal request for photos taken between 15th January and 15th February.

Appendix A

Hydraulic analysis

Technical Note

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Project title	Newlay Weir Investigation	Job number	274242-26
cc		File reference	Data and Documents Library / Reports
Prepared by		Date	14 July 2021
Subject	Hydraulic modelling of Newlay Weir		

1 Introduction

As part of the independent expert investigation into the collapse of Newlay Weir, hydraulic modelling has been undertaken to verify the hydraulic conditions at or just prior to collapse.

Newlay Weir is located on the River Aire in Horsforth, Leeds, West Yorkshire. Constructed c. 1690, it is a 50m long stone weir previously used to control water levels in the Kirkstall Forge goit. This minor watercourse has since been abandoned, but the weir remains and is a Grade II listed structure.

Shortly after the Storm Christoph event (19/01/2021 to 21/01/2021), a large central portion of the weir collapsed. Construction works were ongoing at the time, to construct a fish pass through the weir on its south side. The collapse may have been due to the hydraulic conditions imposed on the weir during the Storm Event, during which flow in the River Aire peaked at around 168 m^3/s (according to the gauge at Armley). As such, an existing 1D-2D river model has been modified to determine what these hydraulic conditions may have been during the event.

This document outlines the available data and models (Section 2), the work undertaken to update the model to the latest data (Section 2), changes to the model to simulate the channel geometry during construction (Section 4), the hydrological events to which these models were subjected (Section 5), the results of these investigations (Section 6), and any shortcomings or assumptions made which may influence the pertinent outcomes (Section BBB).

2 Available data and models

2.1 Hydraulic model

A recent model of the River Aire and associated floodplains was made available to Arup for assessment, which is the Leeds FAS Phase II model. This is a 1D-2D model in Flood Modeller Pro (in-channel model) and TUFLOW (floodplain model). The author has good familiarity with the model and the area in question, having worked on the Leeds FAS scheme for several years beforehand.

The in-channel part of this model extends for several kilometres upstream of Newlay Weir, and downstream to beyond Leeds Rail Station. As such, it covers enough of the River Aire to make this assessment hydraulically relevant, and not subject to limitation by assumed boundary conditions.

2.2 Survey data

Several surveys of the River Aire and its surroundings have been made over the years. The hydraulic model in this area is based on a 2016 survey undertaken as part of the Leeds FAS Phase II modelling work. This survey has not been re-examined as part of this modelling exercise.

More recently, surveys were undertaken prior to the construction of the fish pass (pre-condition survey), during the construction phase, and post-collapse (as-built survey, including the collapsed weir sections).

- Early project planning survey (March 2015)
- Pre-construction survey (September 2018) – *P18-01319*
- As-built topographical survey (May 2021) – *274242-15_10*

2.3 Additional material

Several photographs of the weir were provided, both before and after the collapse of the central section, and some of which included the location and data from which the height of the sheet piles above ground level could be inferred. Some of this data was transferred to a plan drawing of the area to indicate the extent of sheet piles, which was used to inform cross-section modification for the construction case.

3 Baseline updates

The pre-construction topographical survey (P18-01319) was incorporated into the model as follows:

Weir upstream toe

The cross-sectional data for the weir upstream toe were replaced with those abstracted from the topographical survey, at a point a few metres upstream of the crest. This updated section has a slightly deeper bed level than the previous section, but the width is comparable. See Figure 1 for details.

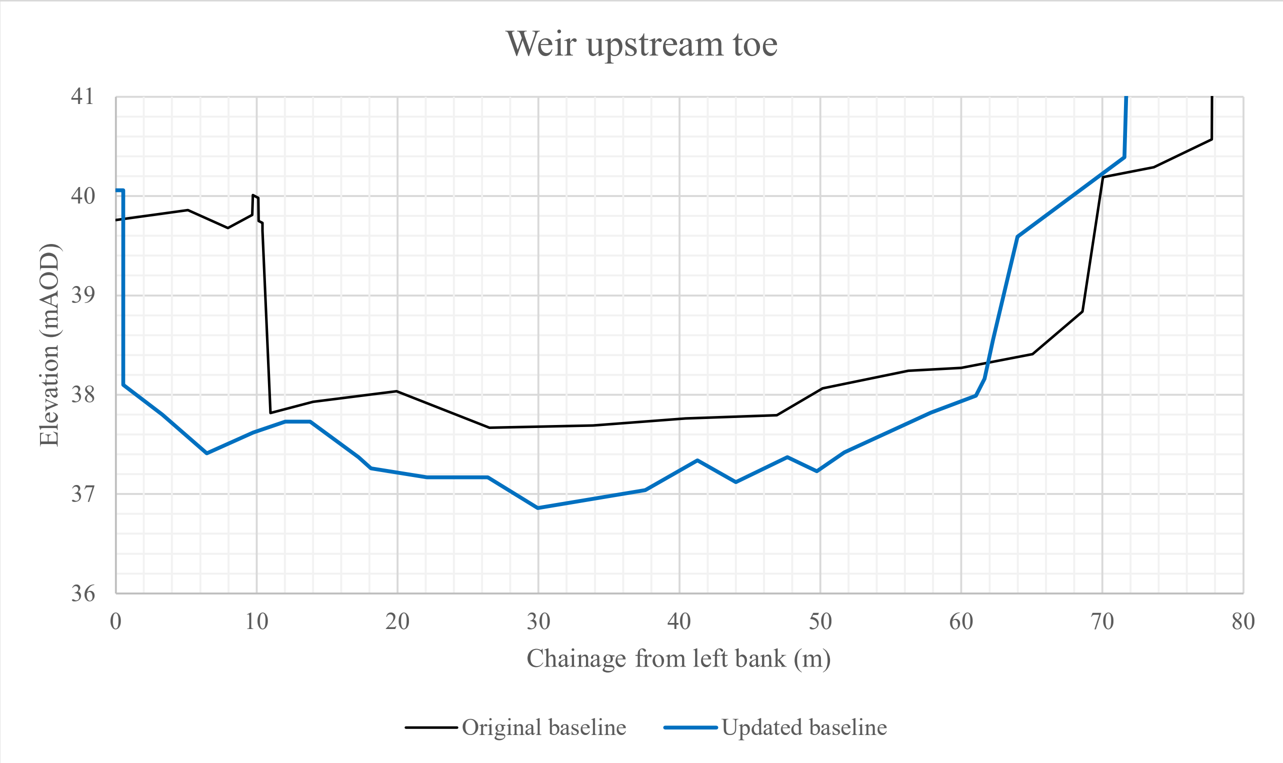


Figure 1 : Model cross-section for weir upstream toe, for original and updated baseline

Weir crest

The cross-sectional data points for the weir crest were replaced with those abstracted from the topographical survey, directly across the weir crest itself. This updated section is very similar to the previous model. See Figure 2 for details.

The previous model used a weir coefficient of 1.20, used to inform the relationship between head and flow. This value has been revised based on an assessment of the weir's shape (similar to a trapezoidal broad-crested weir, and not dissimilar to a crump weir); ordinarily, a value of 1.60 might be considered appropriate for such a weir, but noting the condition of the downstream face, this was reduced to 1.50 in the updated baseline model.

The modular limit (the ratio of upstream to downstream head at which the weir operates in a drowned state) has been changed from the default 0.9 to 0.75, similar to that of a crump weir.

Following the equations used to define flow across a weir, the expected result is to reduce the head over the weir by 14% for the same flow. This increase in efficiency would increase the velocity at the upstream face of the dam, for the same flowrate.

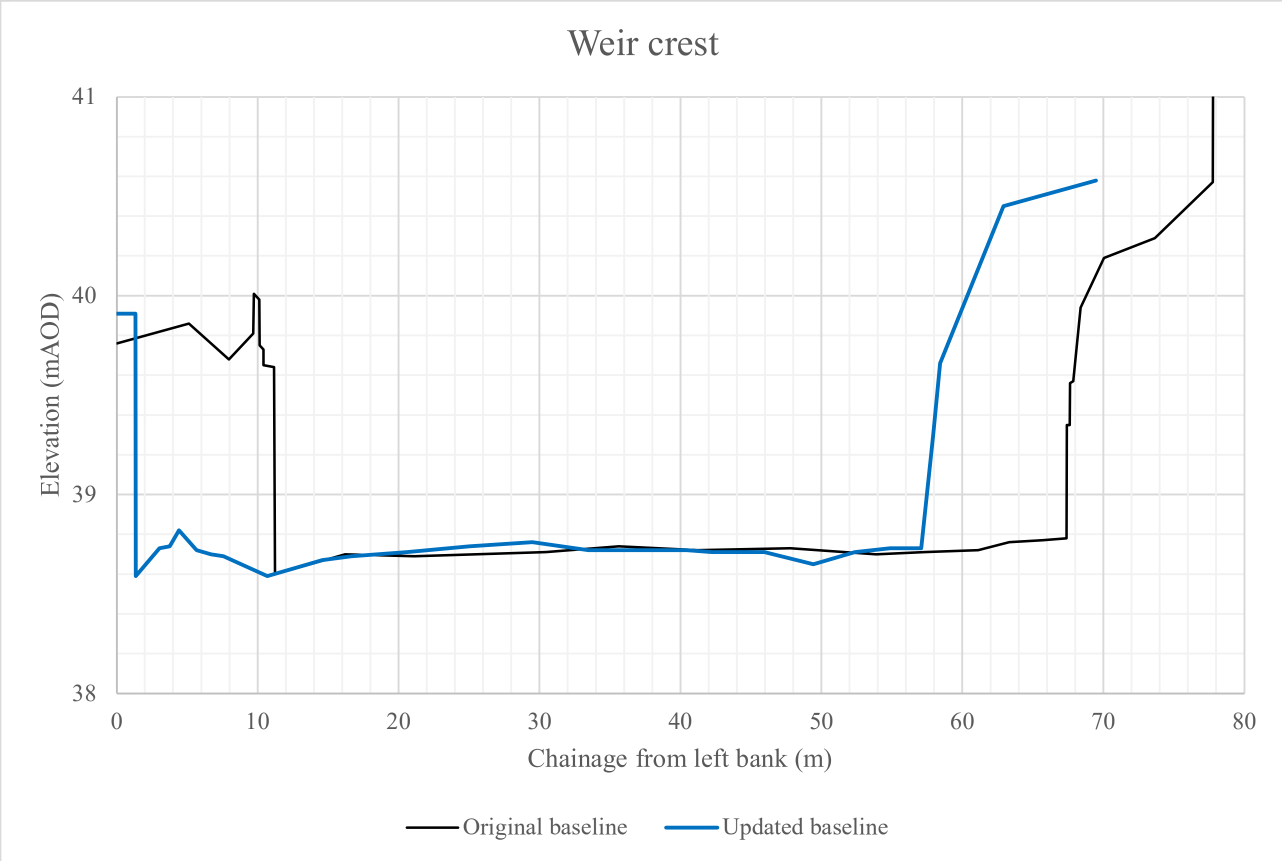


Figure 2 : Model cross-section for weir crest, for original and updated baseline

Crump weir formulation

An additional case was undertaken where the 55.7m long weir crest was replaced with a formal crump weir unit in Flood Modeller, with the left and right bank bypass flow retained in the original spill unit (spill coefficient: 1.0). This crump weir has a variable modular limit; other geometry details were derived from the upstream and downstream cross-sections.

Weir downstream toe

The cross-sectional data for the weir downstream toe were replaced with those abstracted from the topographical survey, at the first location downstream where consistent level data were available. The bed level is lower than in the original model, but has better spatial definition. Widths are comparable between the two. For details, see Figure 3.

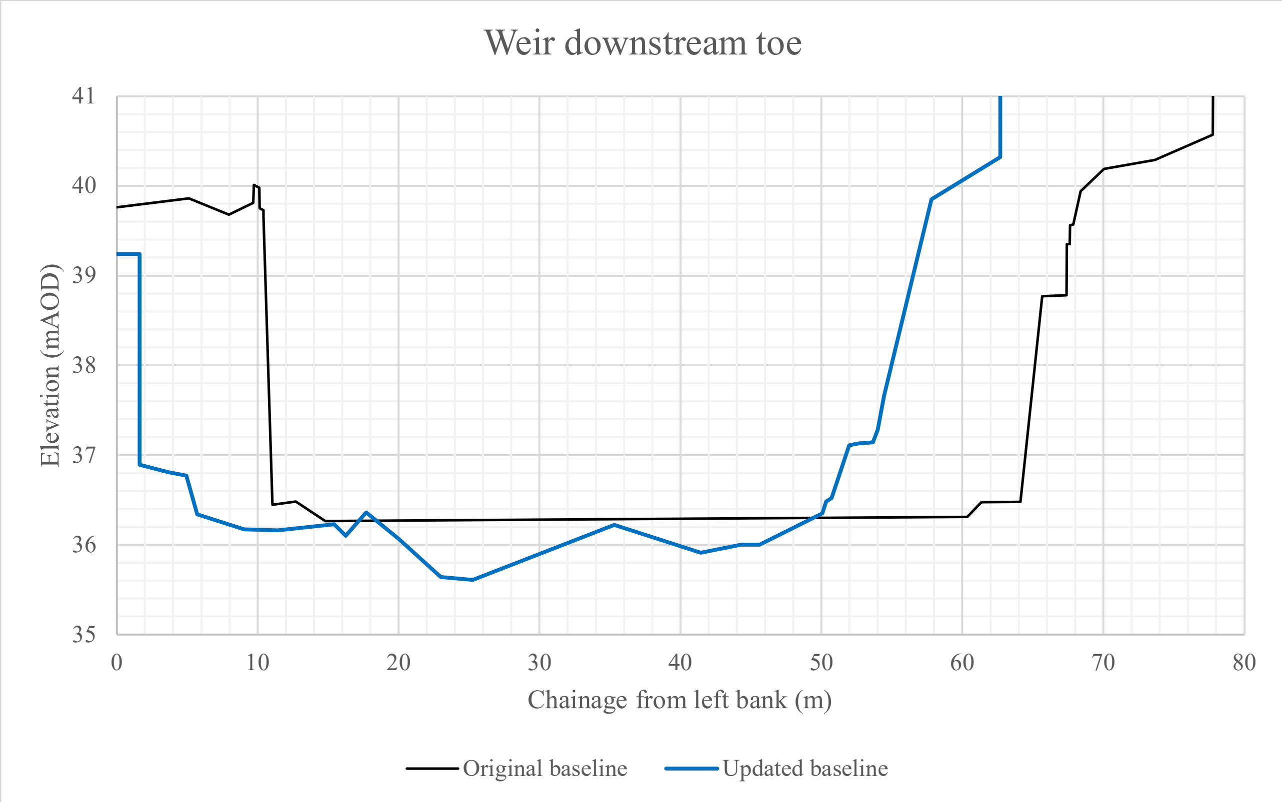


Figure 3 : Model cross-section for weir downstream toe, for original and updated baseline

4 Construction case

For the construction case, photographs and as-built topographical data were analysed to give a likely alignment and height of the sheet piles used as coffer dams during construction. In general, the presence of the piles reduces the available cross-section width by around 13.6m at the weir crest and 15m at the upstream toe.

Weir upstream toe

At the upstream toe, the sheet piles block off approximately 15m of the channel towards the right bank. Rather than tying into high ground, they stop adjacent to a tree, leaving a gap to one side through which high water can bypass the coffer dam and pass through the fish pass construction working area.

The height of the sheet piles has been estimated based on photography as approximately 1m above ground level on the land side, reduced by 0.3m further in the channel. See Figure 4 for details.

Figure 4 : Comparison of updated baseline and construction case cross-section for the weir upstream toe.

Weir crest

Details for the sheet piles were transferred from the weir upstream toe to this cross-section, and consequently these are similar. For details, see **Error! Reference source not found.**. Notably, the bypass mechanism is retained; it is assumed that flow through the construction working area passes directly to the downstream toe of the weir, which based on the topography and hydraulic connectivity of the construction working area is a valid assumption.**Error! Reference source not found.**

Figure 6 shows the weir post-collapse but *with* the sheet piles used in the construction of the fish pass. Note the height of the fish pass (known to be 1.5m), fencing (2m), which were used to infer the height of the sheet piles. Note also the exposed shape of the weir; this was used to suggest the modifications to the weir definition in the model.

The net effect of the sheet piles is to reduce the effective width of the weir crest from 55.7m to 42.1m, a reduction of 25%. According to the equations predicting flow over a weir, this indicates that for a similar flow, the head over the weir would be increased by approximately 21%.

Figure 5 : Photograph showing the collapsed weir during construction of the fish pass.

Crump weir formulation

Again, a crump weir case was also set up for the 42.1m weir crest, with bypass spills units for the left and right bank areas (spill coefficient: 1.0), and an overtopping spill unit for the sheet piles (spill coefficient: 1.7).

Weir downstream toe

Approximately 10m of river flow area at the weir downstream toe is taken up with the sheet piles. The cross-section is reduced in width from the original right bank location by 10m to reflect the area protected by the piles. See Figure 6 for details.

Several metres downstream of the downstream toe, the full flow width has been restored. Consequently, the cross-section has been replicated 10m downstream, but without the curtailment 10m from the right bank. For completeness, this replication of the weir downstream toe cross-section has also been incorporated into the baseline case.

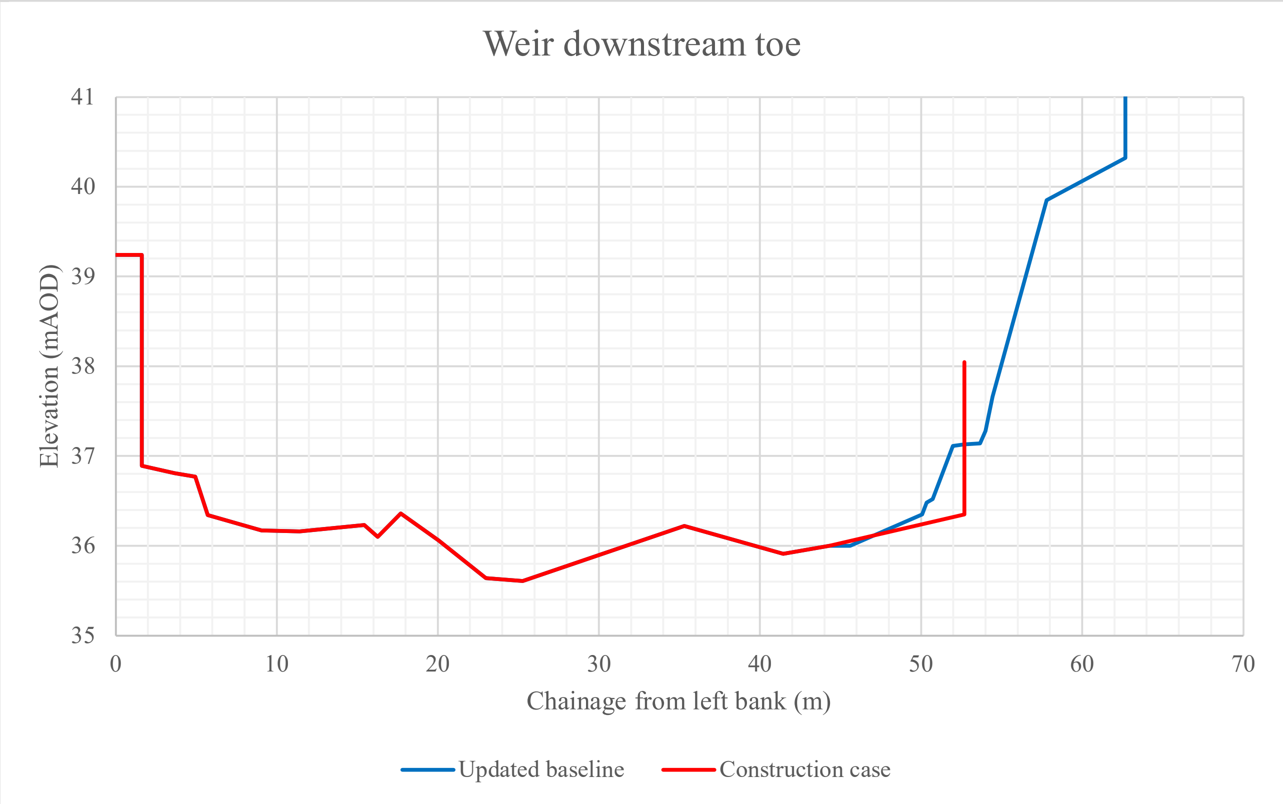


Figure 6 : Comparison of updated baseline and construction case cross-section for the weir downstream toe.

5 Hydrological events

The National River Flow Archive (NRFA) data for the river gauge at Armley indicates that the peak flow in the River Aire during Storm Christoph was 168 m^3/s (although checks against the rating suggests 167m^3/s may be more appropriate).

According to the hydrological annual maximum series (AMAX) for this gauge, the likelihood of this flow being exceeded in any given year (the Annual Exceedance Probability, or AEP) is 18.0%, and has occurred about eight times in the preceding ten years.

The Leeds FAS Phase II modelling included many storm event profiles in excess of this peak value; the smallest is the 1 in 5 year design storm event, with an AEP of 18.1%[*]. As modelled, this has a flow of 168m^3/s at the Armley gauge and as such is a close match to the Storm Christoph event. Note however, the flow at Newlay Weir is slightly lower than this (158m^3/s), due to inflows entering between the weir and the gauge.

In addition to the 1 in 5 year (Q5) event, a 1 in 200 year event (statistical peak flow at Armley: 365m^3/s) was simulated on the baseline case only. This is close in magnitude to the largest event on record (2015 Boxing Day floods), and consequently demonstrates the greatest known flows, depths, and velocities imposed on the weir by the river.

[*] While it is a common misnomer to assume that AEP = 1/(Return period), the correct formulation is AEP = 1 – EXP(-1/(Return period)), and the distinction between these two approaches to its calculation are more notable at high AEP's. Hence the AEP of a 5yr return period event is 18.1%, not 20% as might be expected.

6 Results

6.1 Original weir formulation

Results are presented in Table 1 for the updated baseline and construction case in the 1 in 5 year event, and the 1 in 200 year event for the baseline case only. These are results at the peak of the event.

- **Flow** is output directly from the Flood Modeller software and is consistent throughout the weir upstream, crest, and downstream sections;
- **Average velocity** is output directly from Flood Modeller for the upstream and downstream cross-sections only and is one dimensional (i.e. has no implied direction, other than perpendicular to the cross-section). It is calculated as flow divided by area, not as a function of depth or channel position;
- **Stage** is output directly from Flood Modeller, and is the water level relative to ordnance datum;
- **Greatest depth** is output directly from Flood Modeller for the upstream and downstream cross-sections only, and is calculated by subtracting the minimum bed level from the stage;
- **Weir depths** are calculated for the weir methodology as outlined in APPENDIX 1;
- **Weir velocities** are calculated as per the methodology outlined in APPENDIX 1;
- **Flow mode** states whether the weir operates in "free flow" mode, or "drowned flow" mode, as output from Flood Modeller, and indicates whether the tailwater influences flow over the weir (free flow and drowned, respectively).

The results show that, during the construction:–

- **Upstream toe:**
  - Average velocity is approximately 7% greater;
  - Greatest depth increases by 0.31m, approximately 9% greater.
- **Weir crest:**
  - Average velocity is approximately 10% greater;
  - Weir depths increase by 0.19–0.21m, approximately 20% greater.
- **Downstream toe:**
  - Average velocity is approximately 2% greater;
  - Greatest depth decreases negligibly.

Nonetheless, all parameters experienced by the weir in its construction condition during Storm Christoph are still lower than those experienced by the weir in its unaltered condition during the 2015 Boxing Day floods.

6.2 Crump weir formulation

Results are presented in Table 2 for the updated baseline and construction case in the 1 in 5 year event, and the 1 in 200 year event for the baseline case only. These are results at the peak of the event.

Results show that, during the construction:–

- **Upstream toe:**
  - Average velocity is approximately 8% greater;
  - Greatest depth increases by 0.25m, approximately 8% greater.
- **Weir crest:**
  - Average velocity is approximately 10% greater;
  - Weir depths increase by 0.18–0.19m, approximately 20–21% greater.
- **Downstream toe:**
  - Average velocity is approximately 3% greater;
  - Greatest depth decreases negligibly.

Furthermore, all parameters experienced by the weir during Storm Christoph are still lower than those experienced during the 2015 Boxing Day floods, with the exception of weir downstream face velocity when the construction case is 1% *higher* than the Q200 results. This discrepancy may be due to the formulation of the weir downstream face velocity, which is defined only for free flow and consequently ignores the highest flowrates; in reality, a higher velocity during the transition from free to drowned flow may occur.

6.3 Comparison of weir representations

The two weir formulations above represent two specific cases: a general purpose spill unit which requires a justified estimate of discharge coefficient and modular limit; and a very specific type of weir (a crump weir) with well-defined curves of coefficients and modular limits. The former is defined subjectively based on the experience of the modeller, and the latter may not apply in this case, the weir geometry not fully matching the assumptions of a crump weir.

As such, it is the patterns of the outcomes that are important as opposed to the actual numbers. In each case, the construction case *increases* the velocity and depth applied at the upstream toe and weir crest, but these numbers remain *below* those seen in the largest magnitude event on record.

Table 1 : Results for original weir formulation.

Location	**Parameter**	**Peak parameter during simulation**		
		Baseline Q5	*Construction Q5*	*Baseline Q200*
Throughout	Flow	158 m^3/s	158 m^3/s	341 m^3/s
Upstream toe	Average velocity	0.89 m/s	0.95 m/s	1.23 m/s
	Stage	40.22m AOD	40.53m AOD	41.59m AOD
	Greatest depth	3.36m	3.67m	4.73m
Weir crest	Weir velocities	Crest: 2.75 m/s Face: 3.03 m/s	Crest: 3.03 m/s Face: 3.33 m/s	Crest: 3.35 m/s[†] Face: 3.55 m/s[†]
	Flow mode	Free flow only	Free flow only	Drowned flow around peak
	Weir depths	Crest: 1.03m Face: 0.94m	Crest: 1.24m Face: 1.13m	Crest: 2.89m[‡] Face: 2.49m[§]
Downstream toe	Average velocity	0.99 m/s	1.01 m/s	1.21 m/s
	Stage	39.18m AOD	39.17m AOD	41.19m AOD
	Greatest depth	3.57m	3.56m	5.58m

Table 2 : Results for crump weir formulation

Location	**Parameter**	**Peak parameter during simulation**		
		Baseline Q5	*Construction Q5*	*Baseline Q200*
Throughout	Flow	158 m^3/s	158 m^3/s	348 m^3/s[**]
Upstream toe	Average velocity	1.00 m/s	1.08 m/s	1.37 m/s
	Stage	39.92m AOD	40.17m AOD	41.32m AOD
	Greatest depth	3.06m	3.31m	4.46m
Weir crest	Weir velocities	Crest: 3.38 m/s Face: 3.03 m/s	Crest: 3.71 m/s Face: 3.33 m/s	Crest: 3.85 m/s† Face: 3.31 m/s†
	Flow mode	Free flow only	Free flow only	Drowned flow around peak
	Weir depths	Crest: 0.84m Face: 0.94m	Crest: 1.02m Face: 1.13m	Crest: 2.61m‡ Face: 2.51m§
	Flow around sheet piles	0.949 m^3/s	0.610 m^3/s	15.1 m^3/s
Downstream toe	Average velocity	0.99 m/s	1.02 m/s	1.24 m/s
	Stage	39.18m AOD	39.17m AOD	41.22m AOD

[†] Upper bound, occurring before peak when weir is in free flow mode; velocities will be lower when weir is drowned.
[‡] Upper bound, based on head over weir using upstream peak stage.
[§] Upper bound, based on head “under” weir using downstream peak stage.
[**] Note, because some flow bypasses the weir in the Q200 scenario, representation of the weir as a slightly more efficient crump weir means that more flow passes over the weir in Table 2.

	Greatest depth	3.57m	3.56m	5.61m

7 Assumptions

A number of assumptions and inaccuracies have been made or noted during this analysis. This section briefly outlines these.

Hydrology

The events used to derive the outcomes were design storm events, which maximise peak water levels at the Armley gauge, and are calibrated against several storm events including the 2015 Boxing Day floods. It is known that the verified peak water level during Storm Christoph is lower than the Q5 event, for instance, but this is conservative. Similarly, the Q200 event is different to the actual event experienced during 2015, but the flowrate is likely to be similar or higher in the design event.

In reality, it may be preferable to use actual rainfall data or flow-time data as an inflow to the model, to better estimate the true flood response at Newlay Weir. This data was not available, but the figures presented here are likely to be on the conservative side.

Weir formulation

See Subsection 6.3 for a discussion of the shortcomings of the weir representation. In general, this weir does not have the full geometry of a crump weir, and the coefficients and modular limits used in the general spill unit formulation are subjective. This may have an effect on the absolute values of outcome parameters, but not the general trend.

Velocity

Velocities reported are average velocities only; they refer (generally) to the bulk property of total volumetric flow at a cross-section flow divided by cross-sectional area of the water. In reality, this does not apply: a profile of depth to velocity is typical, with the greatest velocity occurring near the surface.

Similarly, this applies widthways across a typical river section, with lower velocities in the shallows than may be experienced at the surface of the deeper parts of the river.

However, for highly turbulent flow through a control section such as at a weir, it is likely that the velocities reported are sufficiently accurate for the purposes of comparison.

River geometry

Geometry has been updated based on the surveys at the indicated cross-sections only, and not upstream or downstream of the weir area. This is unlikely to impact on the results.

Bypass flow

All flow which bypasses the weir is assumed to flow directly across the banks or through the fish pass construction working area and return to the river at or just beyond the toe of the weir. In reality there will initially be a delay, as this area must fill with flood water, but this routing effect is likely to be negligible and short-lived.

Choice of modelling type

The 1D-2D model is not best suited to investigating velocities through a cross-section, as noted in the subsection on velocity profiles. The 1D model lacks the spatial resolution to define velocity at each point in the cross-section, and the 2D model has only a coarse resolution in this reach.

Were the weir to be instead represented in 2D only, with a sufficiently fine spatial and temporal resolution, and with high quality topographical data, the velocity profile may improve by width, but not by depth.

In order to determine the true velocities throughout the cross-sections, it is necessary to use either a scale physical model, or a quasi-dynamic three-dimensional computational fluid dynamics (CFD) model of the Newlay Weir reach. These are relatively expensive options compared to the analysis above, which is likely to be sufficiently accurate for the purposes of comparing the baseline case to the construction case.

Appendix 1: Weir depths and velocities

In general, throughout this analysis we assume that flow is acting on a rectangular weir cross-section, such that the bypass width is ignored but the bypass flow is included. Given the low magnitude of the bypass flow, the effect is likely to be negligible.

Free flow mode

Weir crest

Through a control section such as a weir or flume, flow tends to pass through critical depth.

Head over the weir H is calculated as the difference between upstream total energy E_0 and weir crest level z_c, assuming no energy loss between the two. To convert this into a critical depth d_c, this value is multiplied by 2/3, following the standard relationship for a rectangular cross-section. The critical depth is converted into a flow area A by multiplying it by weir crest level L_c; finally, the velocity V is determined using flow Q as $V = \frac{Q}{A}$.

As such:

$$d_{crest} = \frac{2}{3}(E_0 - Z_c)$$

$$V_{crest} = \frac{Q}{d_{crest} L_c}$$

Weir downstream face

The flow down the weir downstream face is assumed to be critical in the free flow case, and flowing through a rectangular cross-section, although it is acknowledged that it is likely to achieve super-critical flow over a short distance. As such, the critical depth can be determined using a standard relationship for flow in a rectangular cross-section:

$$d_{face} = \sqrt[3]{\frac{Q^2}{L_c^2 g}}$$

for gravity g. Again, velocity V can be calculated by:

$$V_{face} = \frac{Q}{d_{face} L_c}$$

This approach is likely to underestimate velocities on the downstream face, but will give results that give a reasonable indication of the orders of change expected between the baseline and construction scenarios.

Drowned flow mode

Velocities in the drowned flow mode are not calculated; the largest velocities from the free flow mode of the event are determined instead using the above methodology. During drowned flow, velocities will be lower than this.

In the drowned flow case, depth is assumed always to be subcritical. The upper bounds of this are determined using the stage upstream of the weir Z_0, the stage downstream of the weir Z_1, and the weir crest level z_c. Then, depth is calculated as:

$$d_{crest} = Z_0 - z_c$$

$$d_{face} = Z_1 - z_c$$

This assumes a rounded transition from upstream head to downstream head, such that depth at any point on the weir downstream is not less than these two extremes.

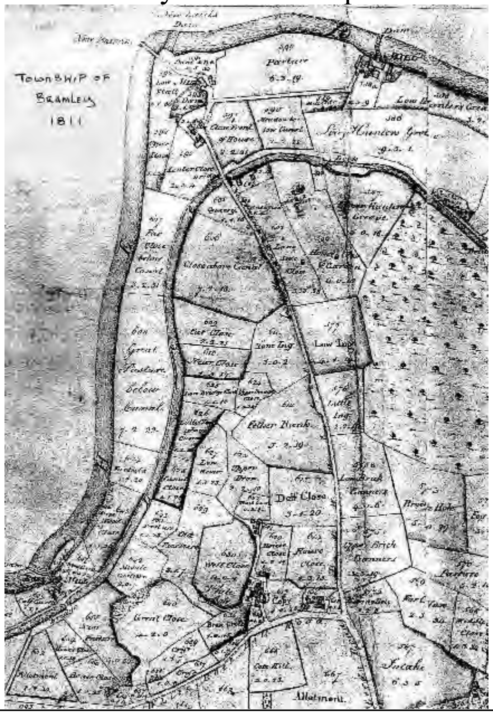
These can be converted to velocities, but it is noted that these will be smaller than the free flow velocities before and after the drowned case.

Name			
Signature			

Appendix B

Timeline

Newlay Weir – Timeline Pre-Initial Collapse

12th century	Weir and Mill Goit constructed to power corn mill at Kirkstall Abbey[1].
1690	Newlay weir was constructed. Built to provide a water supply for nearby mills and supplied a head of water to Lower Forge. Possibly also used by fisheries during earlier periods[2].
1783	A timber bridge upstream of the weir had been constructed by John Pollard[3].
1811	Jonathan Taylors Township of Bramley 1811 
1819	Completion of the iron bridge upstream of the weir was completed by John Pollard[3].
1851	1st Edition Ordnance Survey map of 1851[3].

[1] Newlay Conservation Society (2021) *River Aire & Newlay Bridge*. Accessed on 13/08/2021 via: https://newlayconservationsociety.wordpress.com/history/the-river/
[2] Historic England (2021) *Weir and Retaining Walls on River Aire, Pollard Lane*. Accessed on 13/08/2021 via: https://historicengland.org.uk/listing/the-list/list-entry/1375482
[3] Leeds City Council (2008) Newlay Conservation Area Appraisal and Management Plan. Accessed on 13/08/2021 via: https://www.leeds.gov.uk/docs/Newlay%20Conservation%20Area%20Appraisal%20and%20Management%20Plan.pdf

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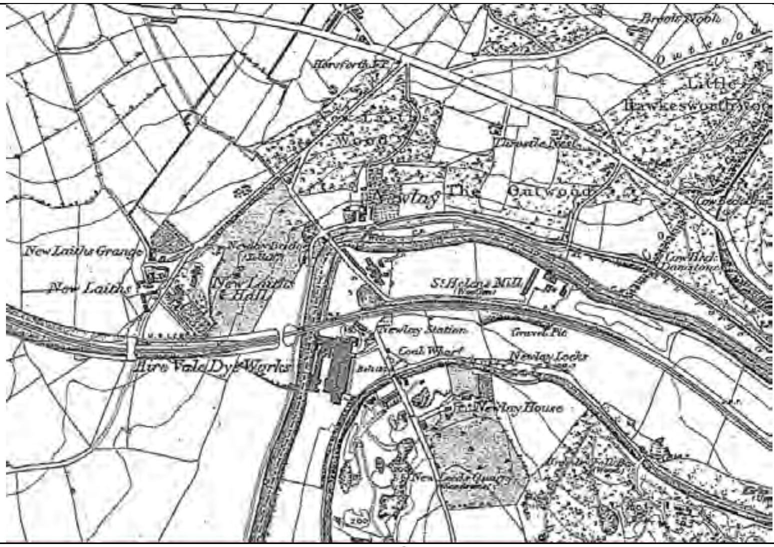
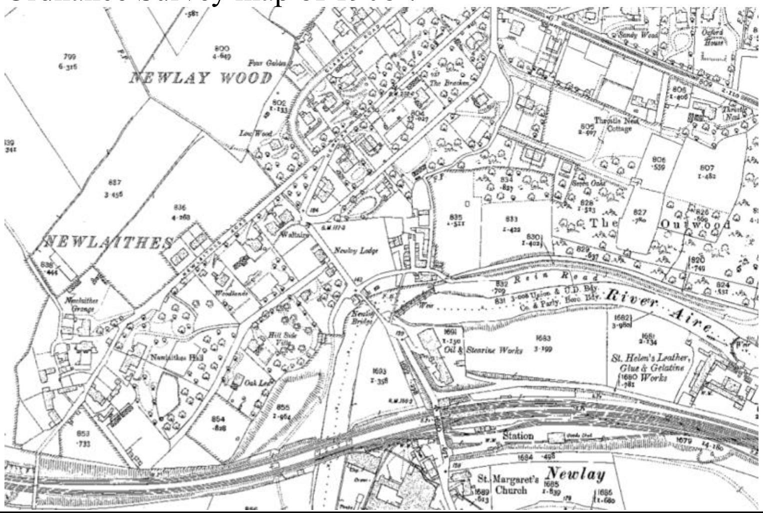
<tr><td></td><td>
</td></tr>
<tr><td>1908</td><td>Ordnance Survey map of 1908[3].

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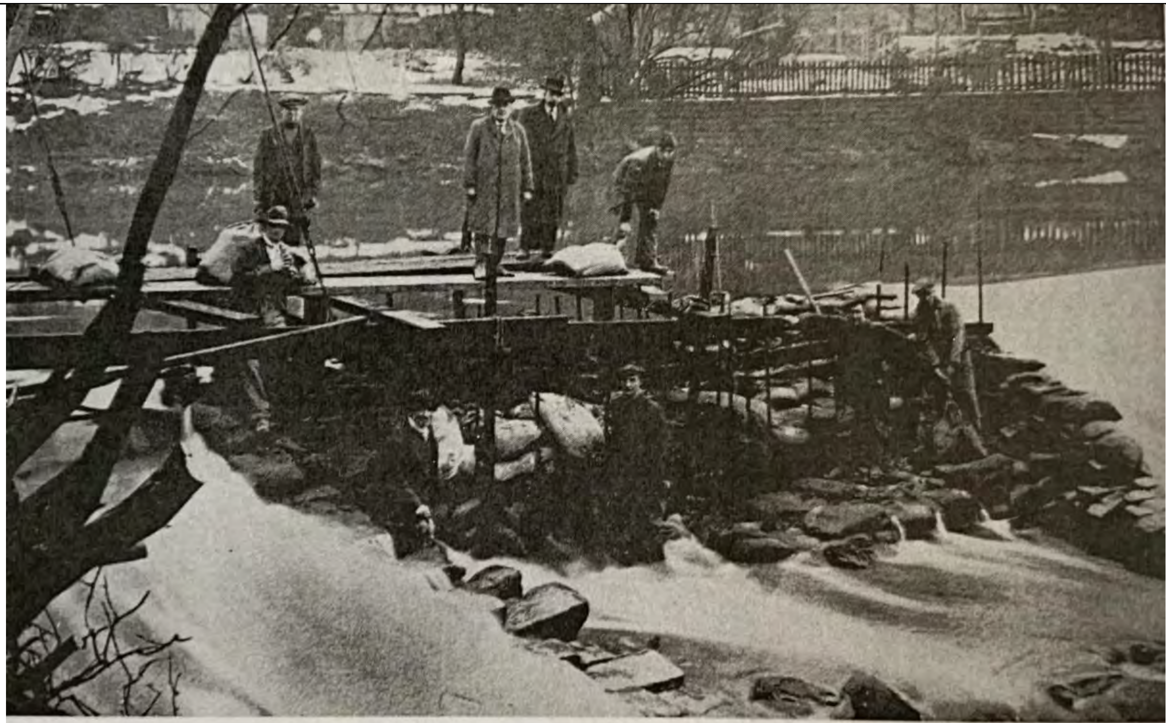
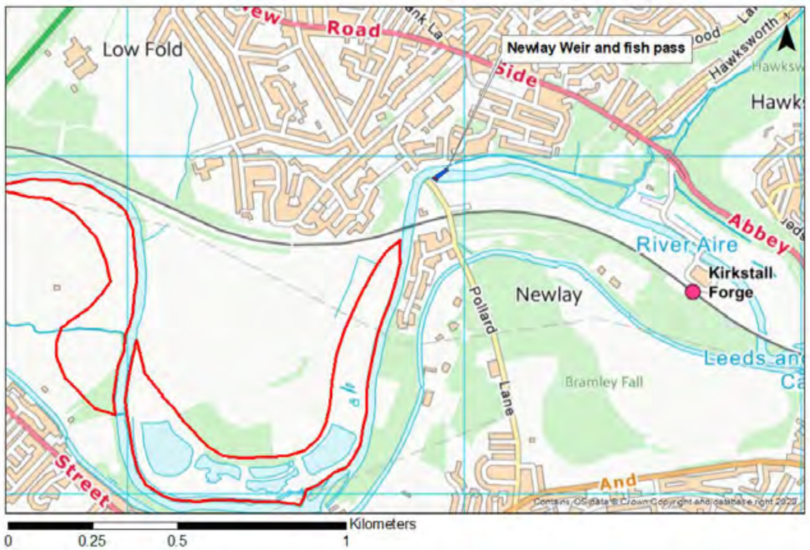
1938	 11. The Weir at Newlay under repair, December 1938. *Acknowledgement: GKN (Kirkstall Forge)* Repairs at Newlay Weir in 1938. • An archaeological report (conducted in 2020) also found evidence of 20th century repairs through concrete across the weir face.
1947	Historic Flood Event • 20/03/1947 " ......The floods in the West Country subsided after 20 March, but rivers continued to rise in eastern England. The Wharfe, Derwent, Aire and Ouse all burst their banks and flooded a huge area of southern Yorkshire. The town of Selby was almost completely under water. Only the ancient abbey and a few streets around the market place escaped inundation. 70 percent of all houses in the town were flooded..... [4]"
1984	Renovations to the bridge upstream[1].
28/12/1978	Flood event

[4] Met Office(2021) *Great weather events: the winter of 1946/47.* Accessed on 13/08/2021 via: https://www.metlink.org/wp-content/uploads/2020/12/19461947_winter.pdf



- In December 1978 there was a storm event within the catchment. The peak river flow and level values recorded at Armley were:

Events	Peak River Level (m)	Peak Flow (m^3s^{-1})
28/12/1978	3.323	170

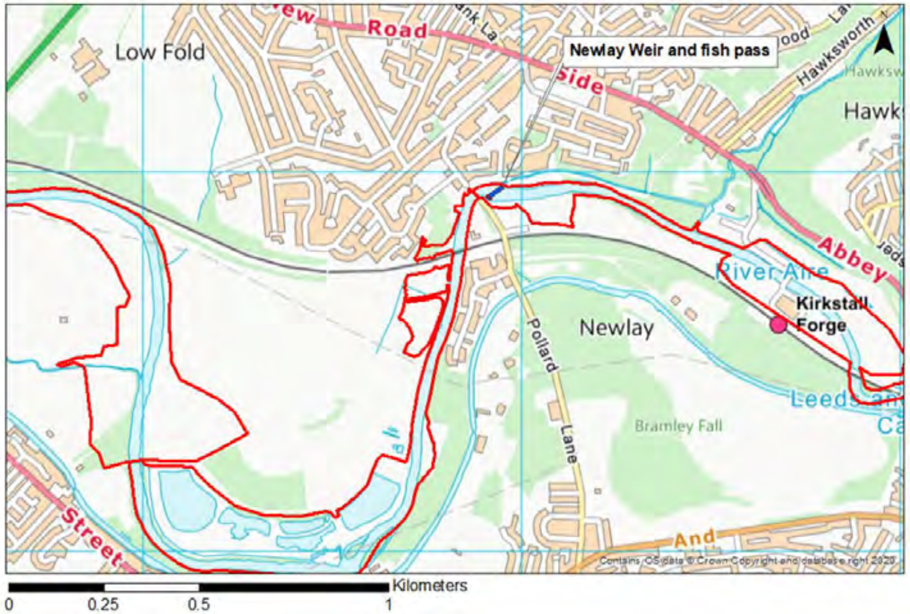
*Data obtained from EA historic flood record[5] and from the National River Flow Archive[6].

11/09/1996	Date first listed as Grade II (Historic England)[2].

[5] Environment Agency (2021) *Historic Flood Ma1p.* Accessed on 13/08/2021 via: https://data.gov.uk/dataset/76292bec-7d8b-43e8-9c98-02734fd89c81/historic-flood-map

[6] UK Centre for Ecology and Hydrology (2021) National River Flow Archive: 27028 - Aire at Armley. Accessed on 13/08/2021 via: https://nrfa.ceh.ac.uk/data/station/info/27028

31/10/2000	Millennial Storm Event



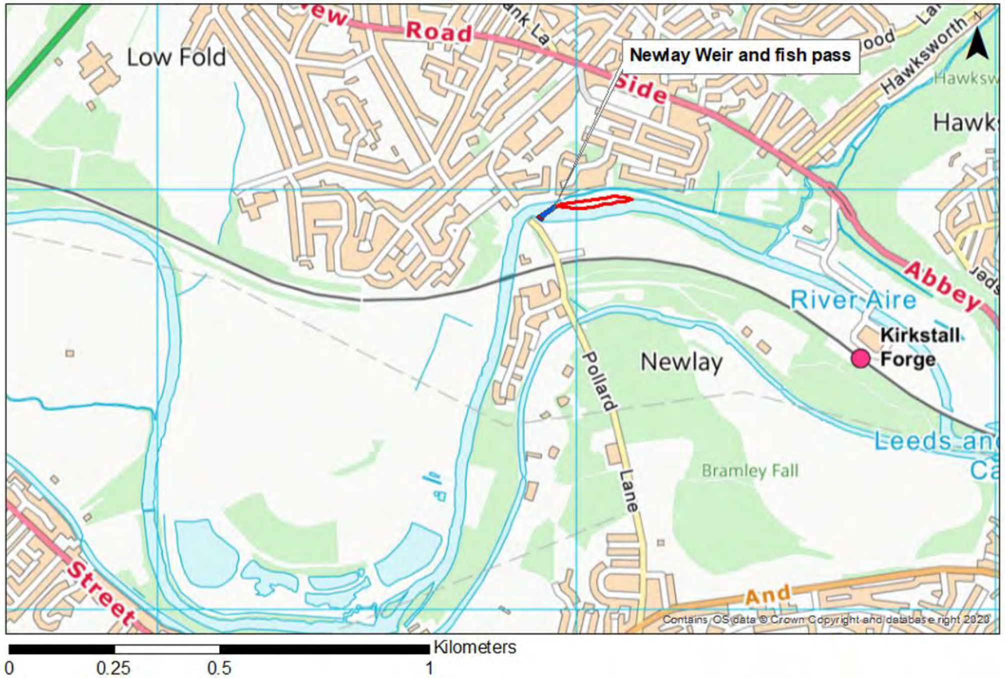
- In October/November 2000 there was a storm event within the catchment. The average daily river flow and level values recorded at Armley were:

Events	Peak River Level (m)	Peak Flow (m^3s^{-1})
Millennial Storm Event		
31/10/2000	4.025	230
06/11/2000	3.398	177

*Data obtained from EA historic flood record[5] and from the National River Flow Archive[6].

10/02/2002

Flood event



- In February 2002 there was a storm event within the catchment. The peak river flow and level values recorded at Armley were:

Events	Peak River Level (m)	Peak Flow (m^3s^{-1})
11/02/2002	3.323	170

*Data obtained from EA historic flood record[5] and from the National River Flow Archive[6].

11/09/2014

Geomorphological Site Investigation Report – Horsforth Weir (aka Newlay Lower) *(received as part of investigation):

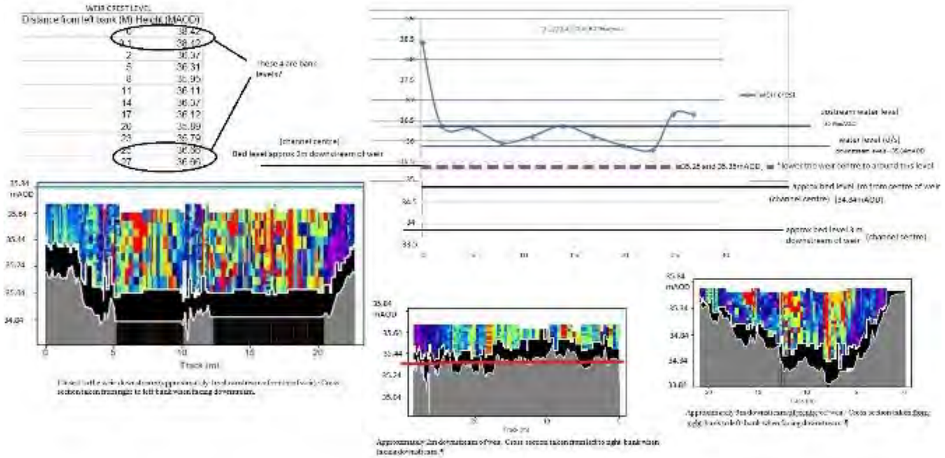
- The purpose of this report was for a weir lowering feasibility assessment

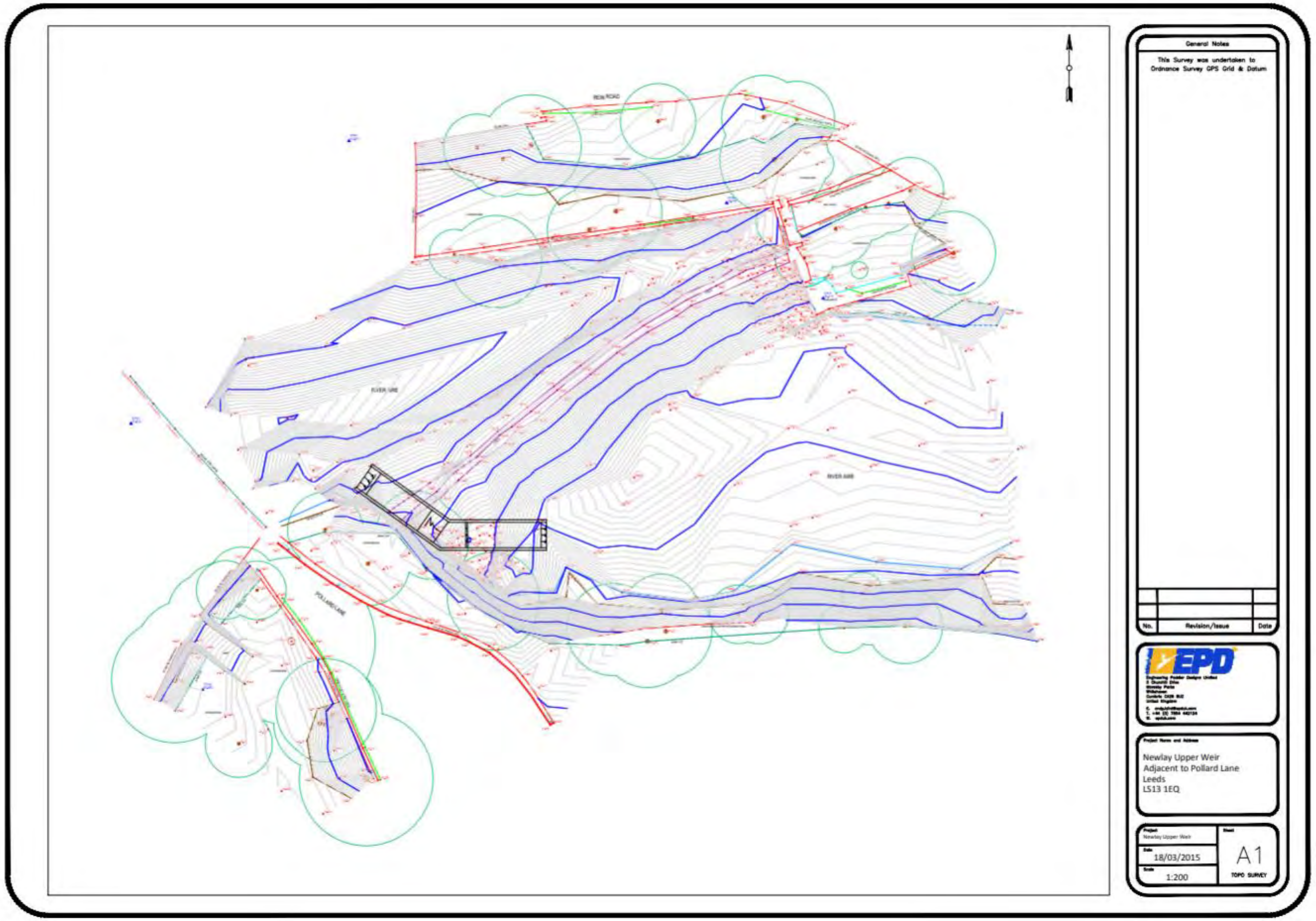
  <u>Summary</u>

  Preliminary risk assessment:

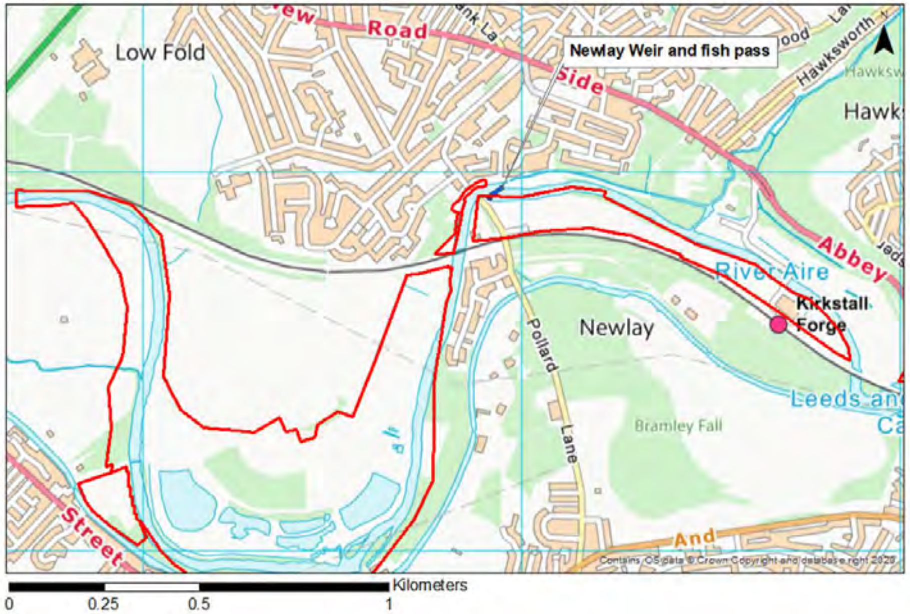
Impact	Minor	Medium	Major

Process	Unlikely	Probable	Likely
Headcutting and knick point migration		Minor	
Infrastructure at risk	Medium		
Water level drops upstream			Minor
Bank collapse		Minor	
Tree losses		Minor	
Velocities and erosion increase			Minor
Sediment redistribution and deposition downstream			Minor

09/09/2014	A Partnership Project Mandate report was put together for the removal of Lower Newlay Weir to help improve fish passage
15/11/2014	Lower Newlay Weir Removal Project Report, including cross sections and a Discharge Measurement Survey, were produced *(received as part of investigation): 
01/01/2015	Newlay Briefing document for weir removal at Lower Newlay Weir to the public *(received as part of investigation)

18/03/2015	Topographic survey of Upper Newlay Weir was undertaken by Engineer Paddle Designs Ltd *(received as part of investigation): 
March-June 2015	Newlay Lower Weir was removed between March and June 2015[7]

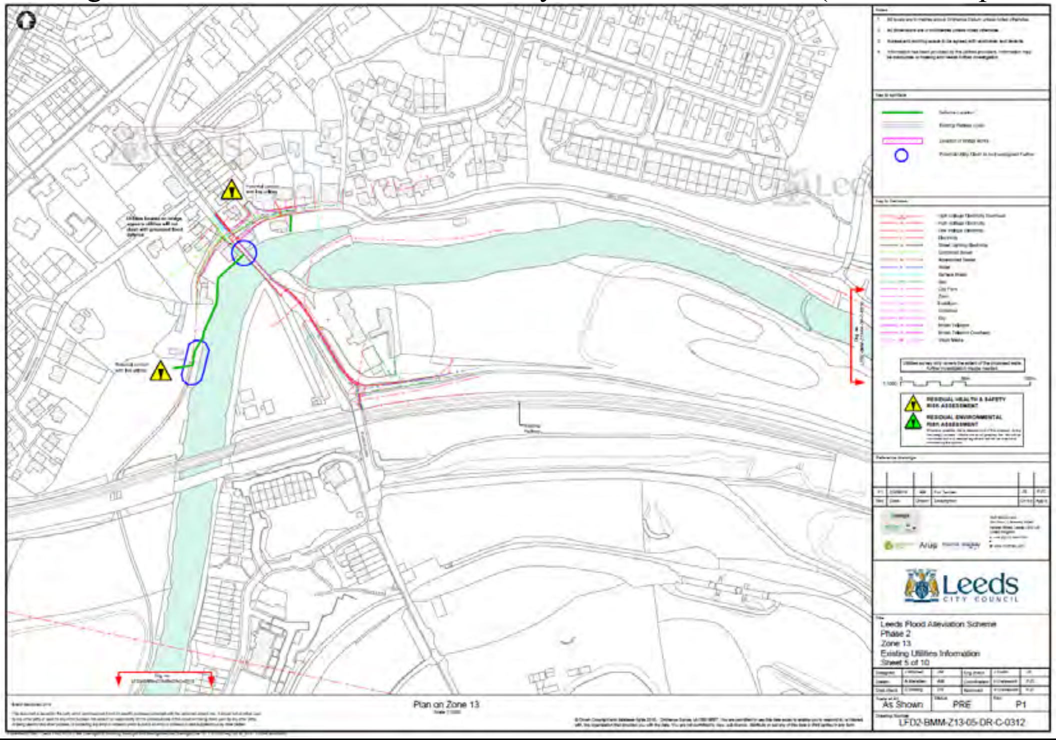
[7] Environment Agency (2015) *Putting fish on the 'highway' to success.* Accessed on 13/08/2021 via: https://environmentagency.blog.gov.uk/2015/06/22/putting-fish-on-the-highway-to-success/

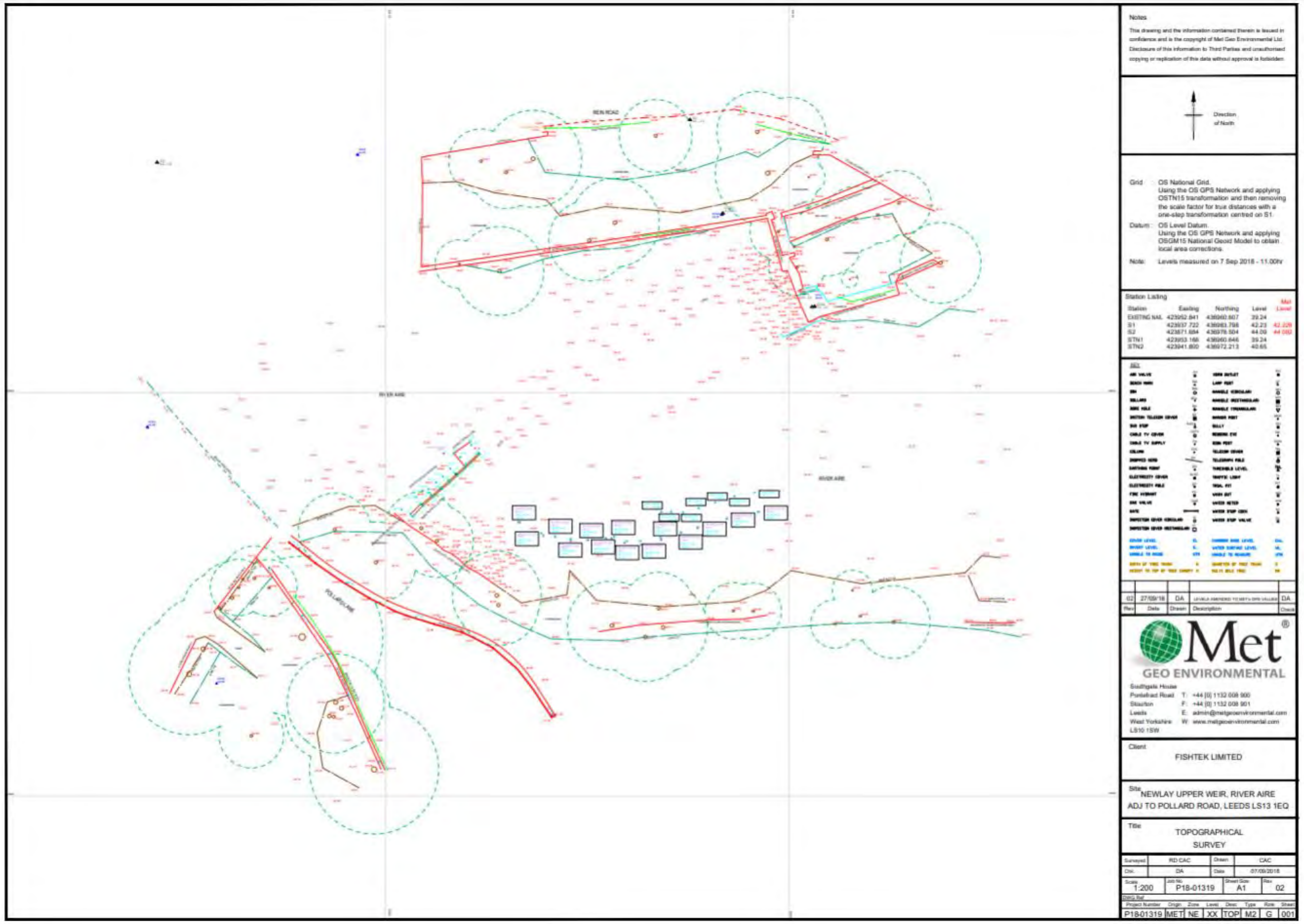
<table>
<tr><td>15/11/2015
–
27/12/2020</td><td>Flood event


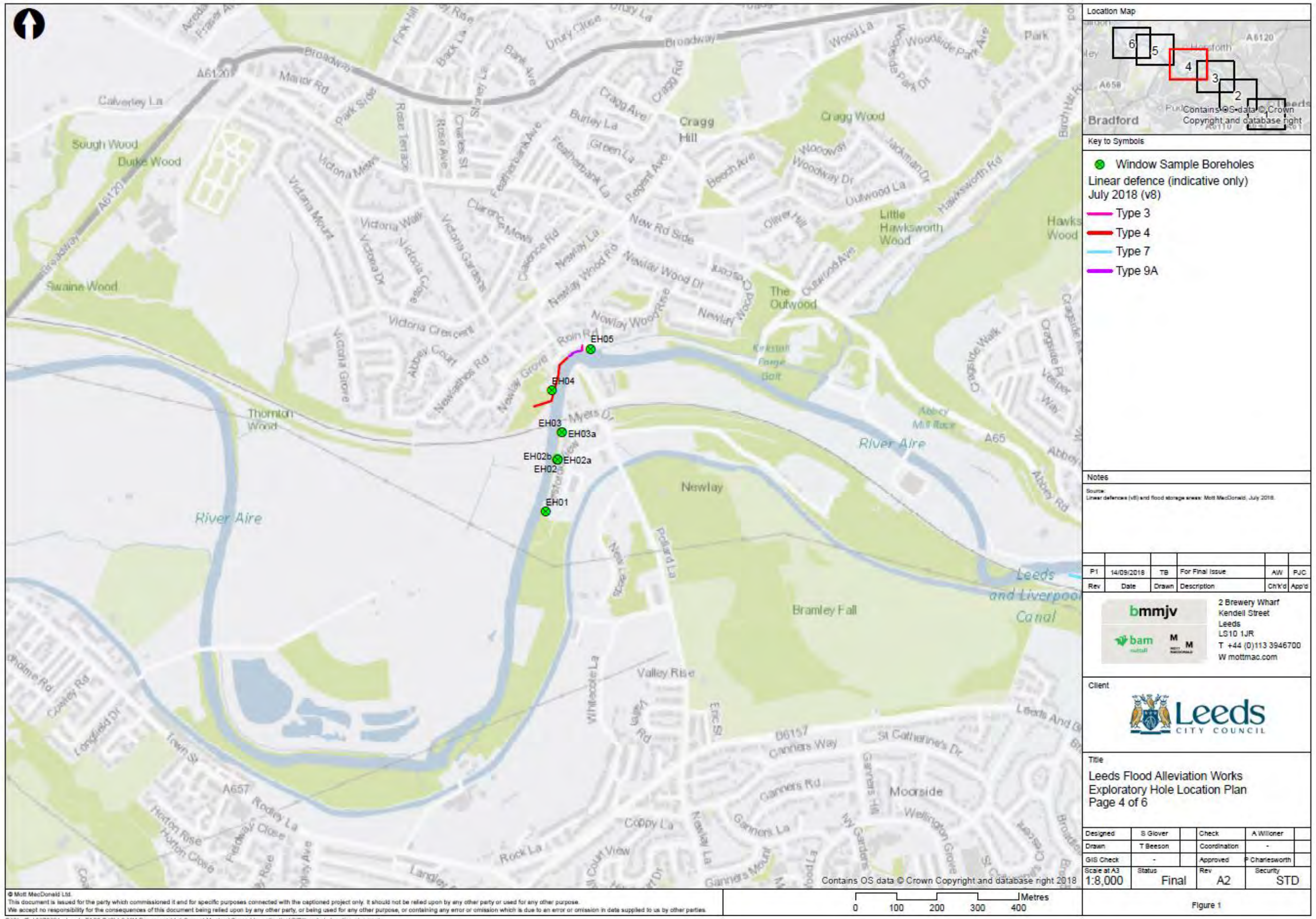
In December 2015 there was a storm event within the catchment. The peak river flow and level values recorded at Armley were:
<table>
<tr><td>Events</td><td>Peak River Level (m)</td><td>Peak Flow (m^3s^{-1})</td></tr>
<tr><td>15/11/2015</td><td>3.71</td><td>202</td></tr>
<tr><td>12/12/2015</td><td>3.814</td><td>211</td></tr>
<tr><td>27/12/2015 – Storm Desmond</td><td>5.217</td><td>344</td></tr>
</table>
*Data obtained from EA historic flood record[5] and from the National River Flow Archive[6].</td></tr>
<tr><td>18/06/2018</td><td>Preliminary Ecological appraisal report carried out by BL Ecology Ltd *(received as part of investigation):
This report mentions habitat and flora and fauna within the area and how the proposed works my impact upon these.</td></tr>
</table>

19/06/2018	River Aire Weirs Structural Inspection (prepared by AECOM at Newlay Weir - low flow) *(received as part of investigation): • The inspection was undertaken "as part of the due diligence procedures in advance of the proposed works". This was based on observed data and no intrusive of in-river inspections were conducted. • The precise composition of the weir couldn't be determined due to flows over the weir but it was assumed to be a stone masonry type of construction. Images from the inspection:  Photograph 4.1 : General view of Newlay Weir  Photograph 4.3 : View of right hand bank to receive proposed fish pass Comment on upstream weir block (pictured below): • "The top level of the weir appeared to be level, however, some of the blocks near the location of the work have been compromised. These blocks appear to have buckled / displaced from the curved shape of the weir. There were no missing blocks or apparent surface damage or wear. There was minimal debris to the top of the weir."  Photograph 4.5 : View of top weir block at proposed fish pass, buckled line of blocks Comment of chute slop (pictured below):

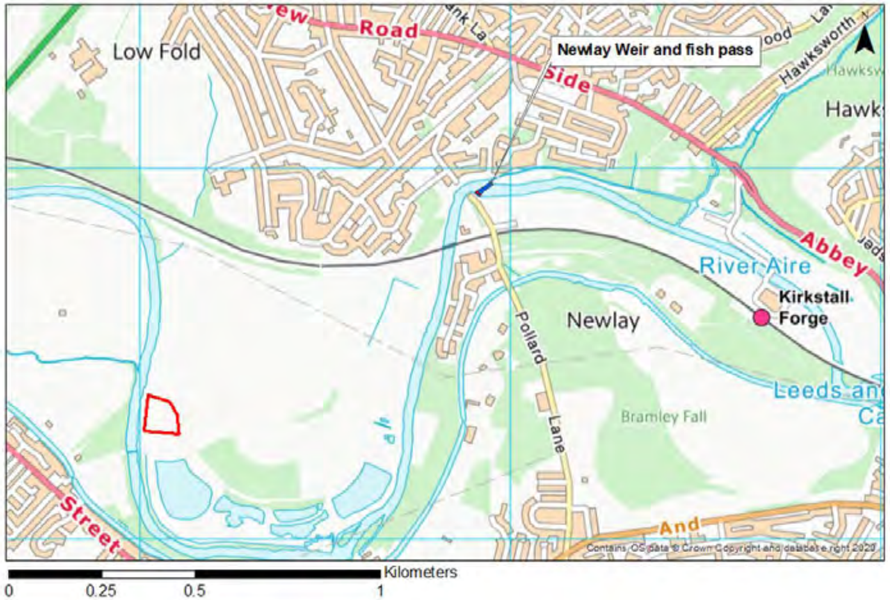
	• In the location of the proposed fish pass, the slope appeared to be consistent in gradient, and flow over the slope was uniform indicating that the surface was potentially free of surface damage and erosion. However, in the area adjacent and downstream of the buckled weir blocks (Photograph 4.5), the slope surface has been eroded with displaced and missing stone setts.  Photograph 4.6 : View of right hand bank chute slope, eroded surface d/s of buckled weir Comments on condition outside location of the proposed fish pass: “Generally, beyond the buckled top weir blocks, the top line of the weir appears to be level, with no further breaks or missing weir blocks.”

03/09/2018	Existing utilities information undertaken by Mott MacDonald *(received as part of investigation): 

07/09/2018	Topographic Survey (Met Geo Environmental) + CAD drawings by FishTek Ltd *(received as part of investigation): 

<table>
<tr><td>14/09/2018</td><td>Ground Investigation Data by GeoCon Site Investigations Ltd *(received as part of investigation):

</td></tr>
<tr><td>04/01/2019</td><td>FCRM Environmental Assessment Form*(received as part of investigation)</td></tr>
</table>

09/02/2020	Storm Ciara

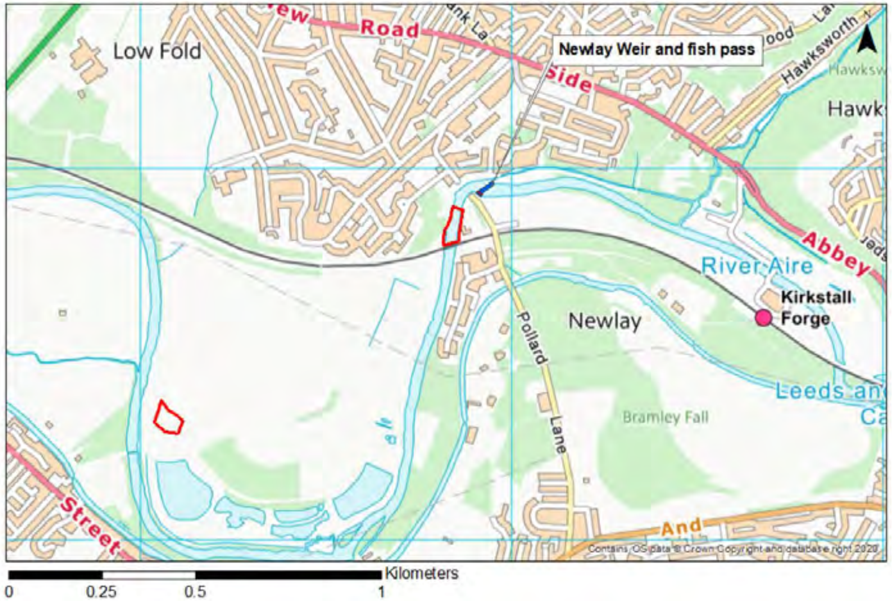


- In February 2020 there was a storm event within the catchment. The peak river flow and level values recorded at Armley were:

Events	Peak River Level (m)	Peak Flow (m^3s^{-1})
09/02/2020 - Storm Ciara	4.134	240

*Data obtained from EA historic flood record[5] and from the National River Flow Archive[6].

16/02/2020	Storm Dennis



- In February 2020 there was a storm event within the catchment. The peak river flow and level values recorded at Armley were:

Events	Peak River Level (m)	Peak Flow (m^3s^{-1})
16/02/2020 - Storm Dennis	3.417	178

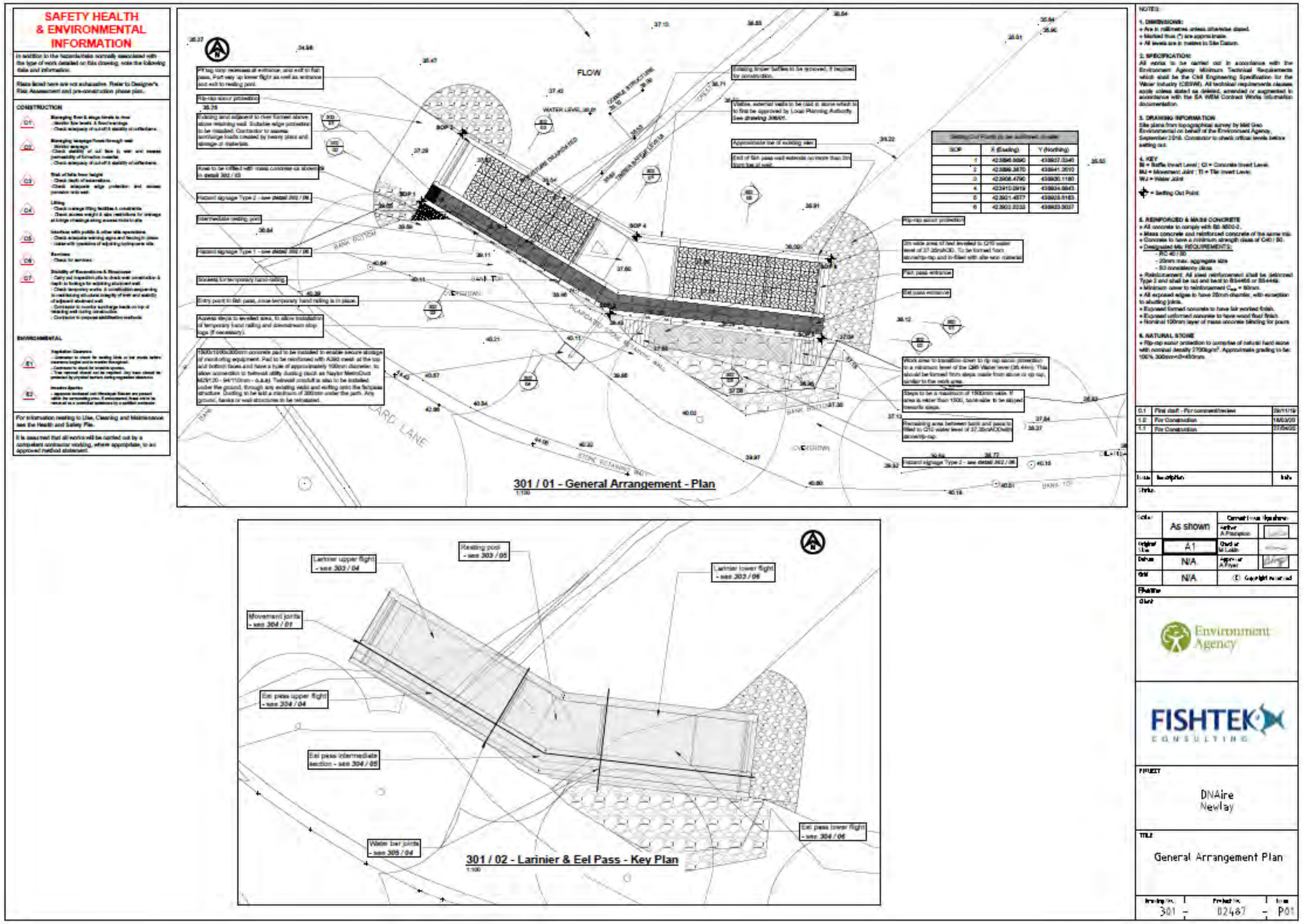
*Data obtained from EA historic flood record[5] and from the National River Flow Archive[6].

21/04/2020	Newlay Pre-condition Survey (Fishtek consulting) – photographic record *(received as part of investigation):

Comments on the fish pass site where drone cameras were used:

	• "Unfortunately, due to the high flow conditions on site at the time of the survey, it was not possible to assess the condition of the weir. However, using a still from drone camera footage taken a few days beforehand, it can be seen that the section of weir to be worked on and the section immediately adjacent seem to be in a moderate condition. From another still of this footage, the extent of the collapsed river wall can be seen."  *Figure 14: Condition of weir, aeration on weir face could indicate damage - Annotation 2020-04-21 100917.*  *Figure 15: View of river wall, irregular patterns locations of wall collapses - Annotation 2020-04-21 101010.*

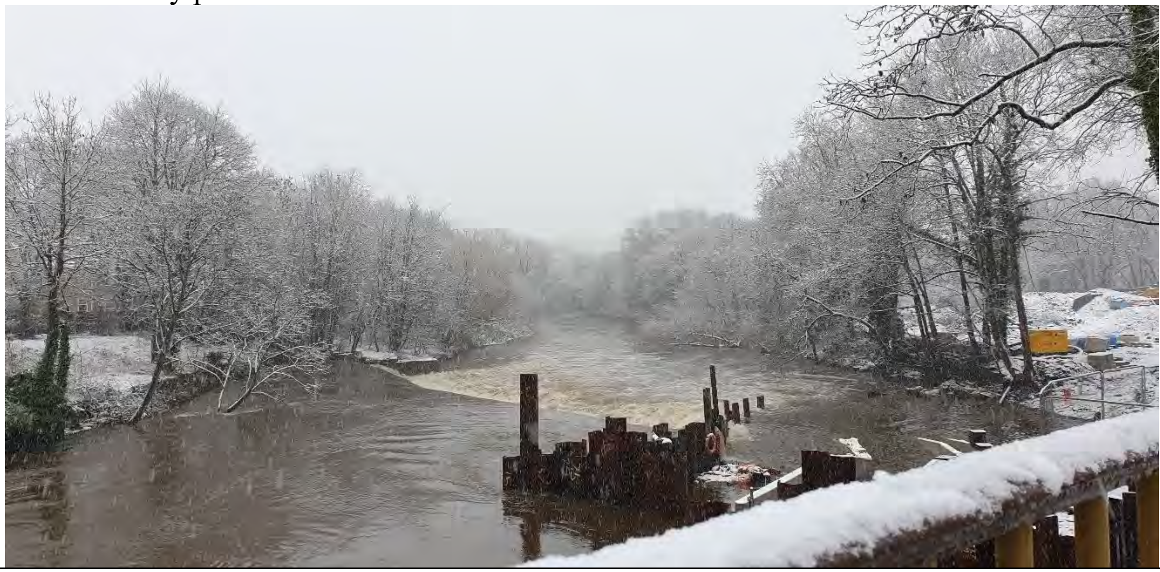
27/04/2020	Fish pass drawing from Fishtech *(received as part of investigation):



01/08/2020	Image from social meda (Twitter) of sand bags placed where the fish pass was constructed: 
06/08/2020	Sheet pile installation took place between 06/08/2020 and 19/08/2020 *(dates provided as part of the invetigation)

07/10/2020	Environment Agency Twitter – “We have installed a cofferdam, broken into the weir and constructed the first section of the fish pass.” Image taken during installation of sheet piles 

25/12/2020	Public Survey Image: 

05/01/2021	Public Survey Image: 
14/01/2021	Public Survey photo 11:24am: 

15/01/2021	Public survey image and video provided: 

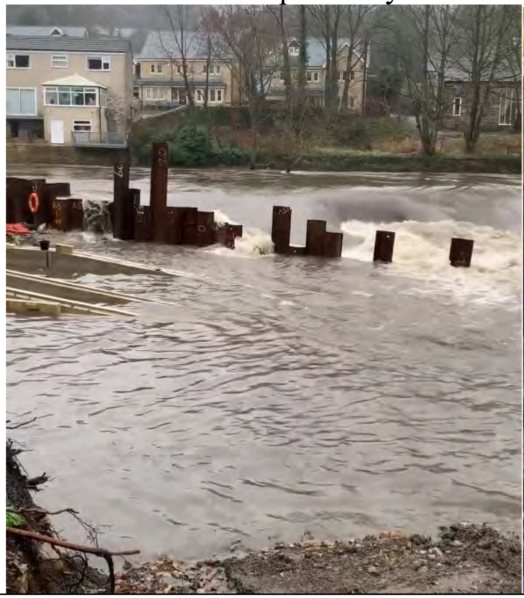
19/01/2021	YouTube video uploaded by Vintage Drone Services showing an aerial view of the weir before and after fish pass construction  
21/01/2021	Initial Collapse begins

Newlay Weir – Timeline Post-Initial Collapse

<table>
<tr><td>21/01/2021</td><td>Initial Collapse begins</td></tr>
<tr><td>21/01/2021</td><td>Storm Cristoph

- In January 2021 there was a storm event within the catchment. The peak river flow and level values recorded at Armley were:

<table>
<tr><td>Events</td><td>Peak River Level (m)</td><td>Peak Flow (m^3s^{-1})</td></tr>
<tr><td>21/01/2021 – Storm Cristoph</td><td>3.279</td><td>168</td></tr>
</table>

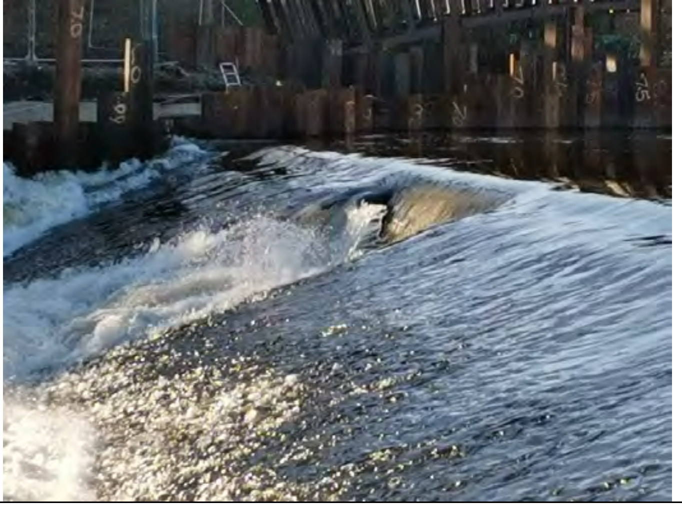
*Data obtained from EA historic flood record[5] and from the National River Flow Archive[6].</td></tr>
<tr><td>21/01/2021</td><td>Stills from a video captured by a member of the public showing the initial collapse taken at 8:45am:

</td></tr>
</table>

Public survey image taken at 8:44am:

	
22/01/2021	Public survey photo: 

25/01/2021	An individual called the EA emergency number to report signs of damage to the weir.
25/01/2021	Photos provided for the purpose of the investigation:   

29/01/2021	Public survey photo: 

30/01/2021	Public survey photo: 

Late 01/2021	Local residents contact the EA emergency helpline after noticing that “a small section of the weir had been dislodged on the downriver side, yet no action was taken by the EA” – Newlay Weir collapse letter from local residents
01/02/2021	Public survey photo: 

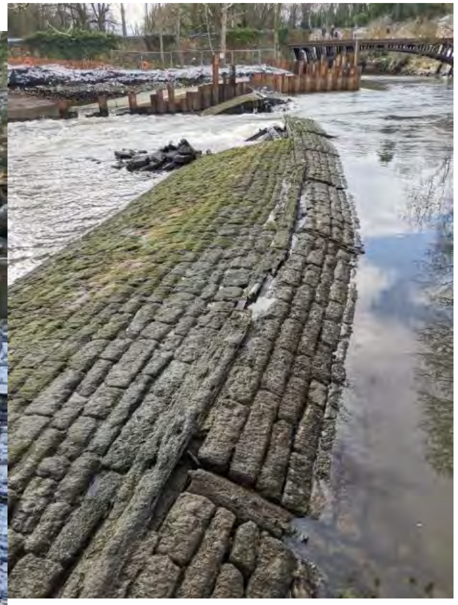
03/02/2021 - 04/02/2021	Photos provided for the purpose of the investigation:  
05/02/2021	Photos provided for the purpose of the investigation:  

Public survey photo:

	Public survey photo:   

	Public survey photo: 
06/02/2021 - 07/02/2021	**Full collapse**

07/02/2021	Photos provided for the purpose of the investigation:   Public survey photo:  

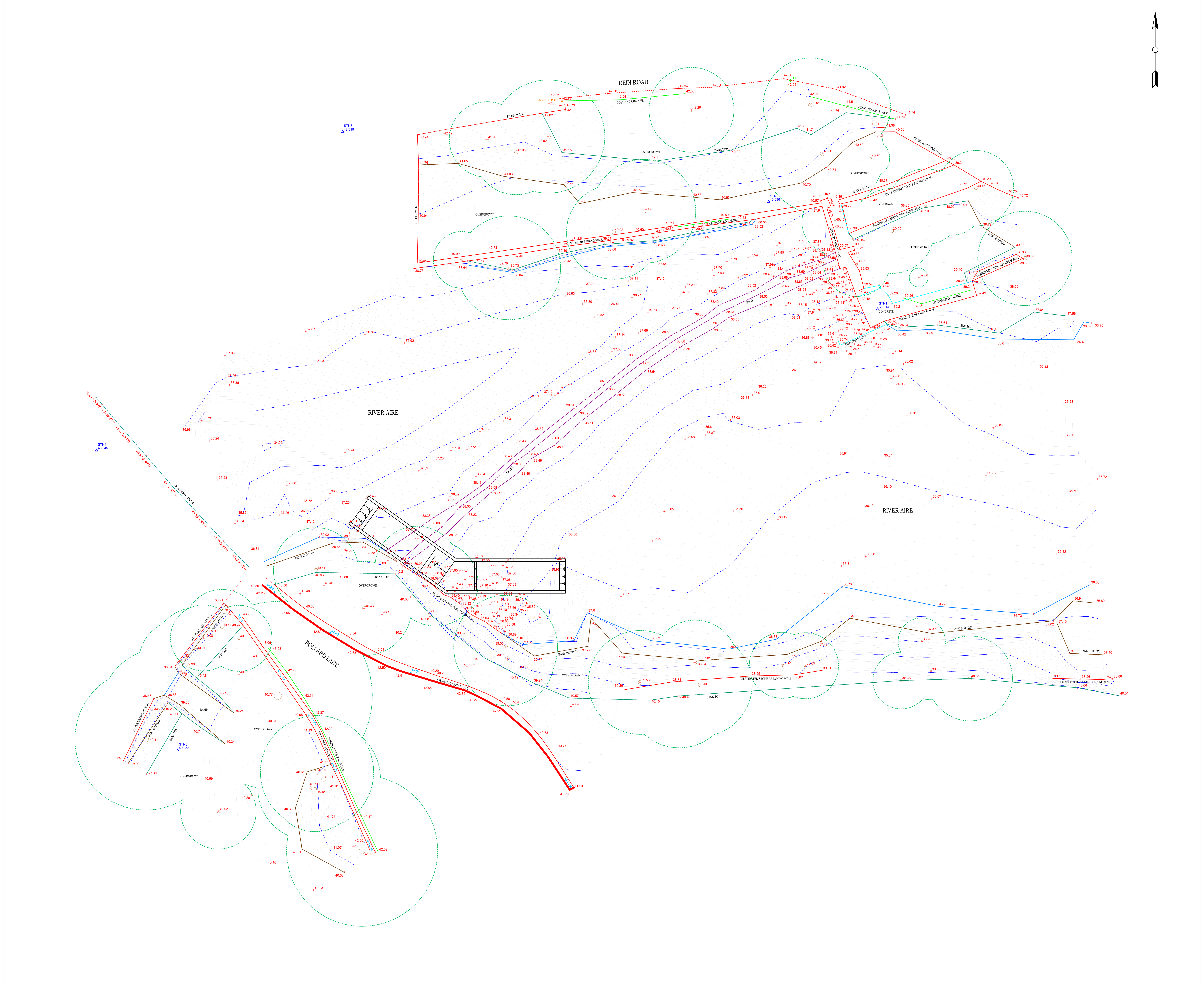
08/02/2021	Photos provided for the purpose of the investigation:   

10/02/2021	Photos provided for the purpose of the investigation:  
10/04/2021	Photos provided for the purpose of the investigation: 

11/05/2021	Arup site visit  

Appendix C

Topographic data



Newlay Upper Weir .dwg

23/03/2015

A1@1:200
TOPO SURVEY

General Notes

This Survey was undertaken to Ordnance Survey GPS Grid & Datum

No.	Revision/Issue	Date

EPD
Engineering Paddler Designs Limited
5 Churchill Drive
Moresby Parks
Whitehaven
Cumbria CA28 8UZ
United Kingdom
E. andy.laird@epduk.com
T. +44 (0) 7554 442124
W. epduk.com

Project Name and Address

Newlay Upper Weir
Adjacent to Pollard Lane
Leeds
LS13 1EQ

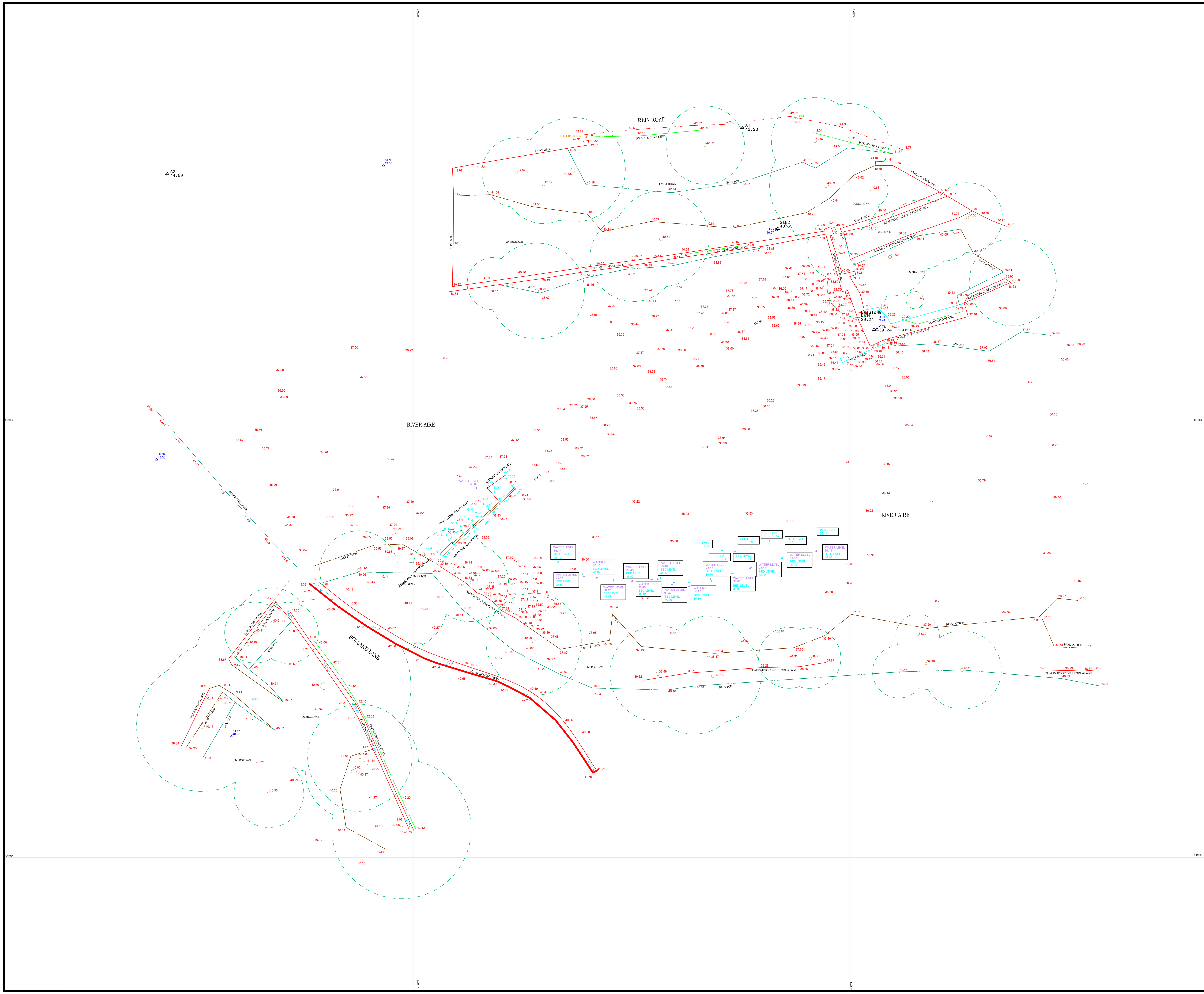
Project
Newlay Upper Weir

Sheet
A1

Date
18/03/2015

Scale
1:200

TOPO SURVEY



Notes

This drawing and the information contained therein is Issued in confidence and is the copyright of Met Geo Environmental Ltd. Disclosure of this information to Third Parties and unauthorised copying or replication of this data without approval is forbidden.

N

Direction of North

Grid : OS National Grid.
Using the OS GPS Network and applying OSTN15 transformation and then removing the scale factor for true distances with a one-step transformation centred on S1.

Datum : OS Level Datum.
Using the OS GPS Network and applying OSGM15 National Geoid Model to obtain local area corrections.

Note: Levels measured on 7 Sep 2018 - 11.00hr

Station Listing

Station	Easting	Northing	Level	Met Level
EXISTING NAIL	423952.841	436960.607	39.24	
S1	423937.722	436983.798	42.23	42.228
S2	423871.684	436978.504	44.09	44.092
STN1	423953.166	436960.646	39.24	
STN2	423941.800	436972.213	40.65	

KEY

AIR VALVE	AV	KERB OUTLET	KO
BENCH MARK	BM	LAMP POST	LP
BIN	BIN	MANHOLE (CIRCULAR)	MH
BOLLARD	BOL	MANHOLE (RECTANGULAR)	MH
BORE HOLE	BH	MANHOLE (TRIANGULAR)	MH
BRITISH TELECOM COVER	BT	MARKER POST	MKR
BUS STOP	BUS	GULLY	GU
CABLE TV COVER	CATV	RODDING EYE	RE
CABLE TV SUPPLY	CA	SIGN POST	SIGN
COLUMN	COL	TELECOM COVER	TEL
DROPPED KERB	DK	TELEGRAPH POLE	TP
EARTHING POINT	ER	THRESHOLD LEVEL	THL
ELECTRICITY COVER	ELEC	TRAFFIC LIGHT	TL
ELECTRICITY POLE	EP	TRIAL PIT	PIT
FIRE HYDRANT	FH	WASH OUT	WO
GAS VALVE	GAS	WATER METER	WM
GATE		WATER STOP COCK	SC
INSPECTION COVER (CIRCULAR)	IC	WATER STOP VALVE	SV
INSPECTION COVER (RECTANGULAR)	IC		
COVER LEVEL	CL	CHAMBER BASE LEVEL	ChL
INVERT LEVEL	IL	WATER SURFACE LEVEL	WL
UNABLE TO RAISE	UTR	UNABLE TO MEASURE	UTM
GIRTH OF TREE TRUNK	G	DIAMETER OF TREE TRUNK	D
HEIGHT TO TOP OF TREE CANOPY	H	MULTI BOLE TREE	MB

Rev	Date	Drawn	Description	Check
02	27/09/18	DA	LEVELS AMENDED TO MET's GPS VALUES	DA

Met® GEO ENVIRONMENTAL

Southgate House
Pontefract Road
Stourton
Leeds
West Yorkshire
LS10 1SW

T: +44 [0] 1132 008 900
F: +44 [0] 1132 008 901
E: admin@metgeoenvironmental.com
W: www.metgeoenvironmental.com

Client

FISHTEK LIMITED

Site

NEWLAY UPPER WEIR, RIVER AIRE
ADJ TO POLLARD ROAD, LEEDS LS13 1EQ

Title

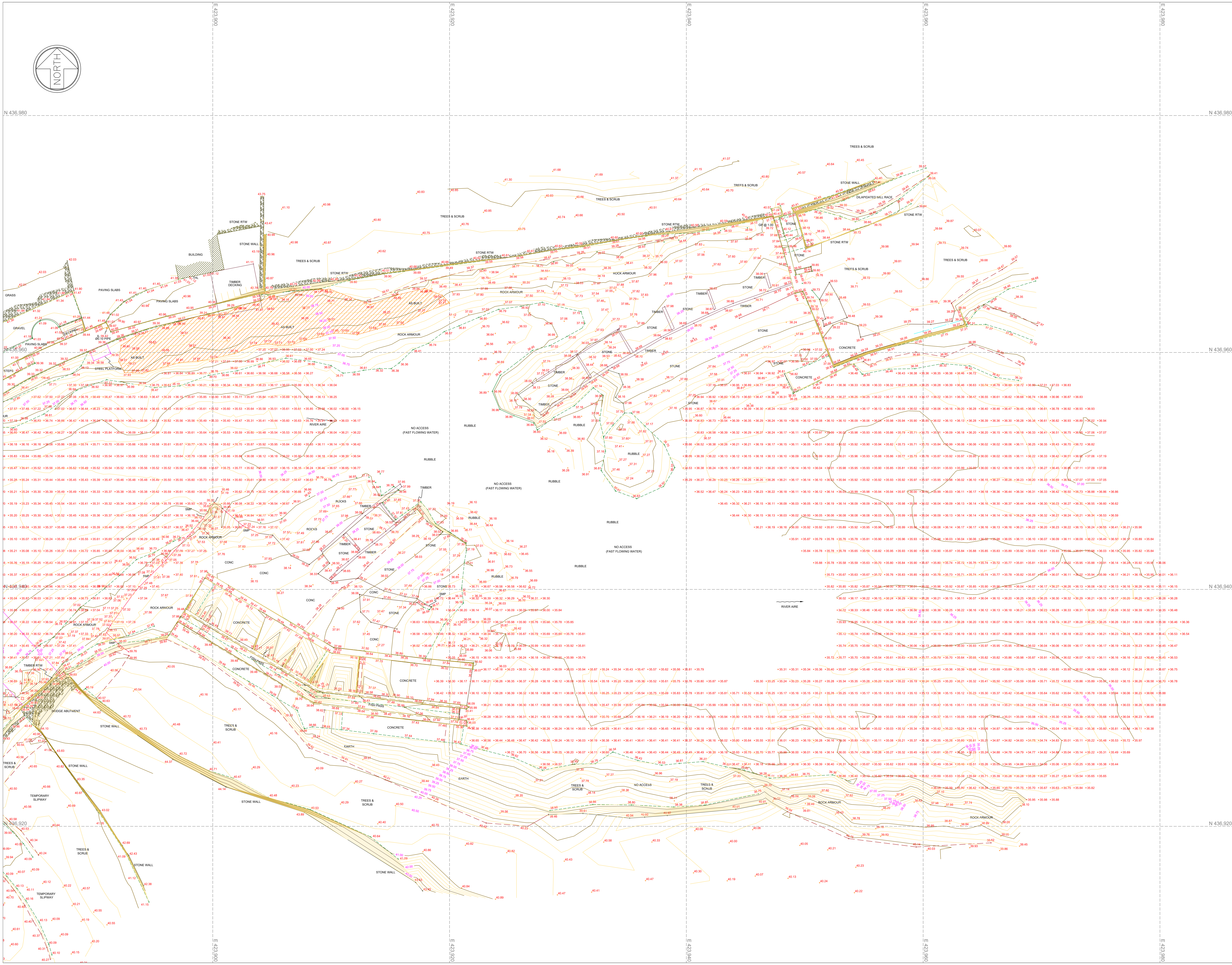
TOPOGRAPHICAL SURVEY

Surveyed	RD CAC	Drawn	CAC
Chk.	DA	Date	07/09/2018

Scale	Job No	Sheet Size	Rev
1:200	P18-01319	A1	02

DWG Ref

Project Number	Origin	Zone	Level	Desc	Type	Role	Sheet
P18-01319	MET	NE	XX	TOP	M2	G	001



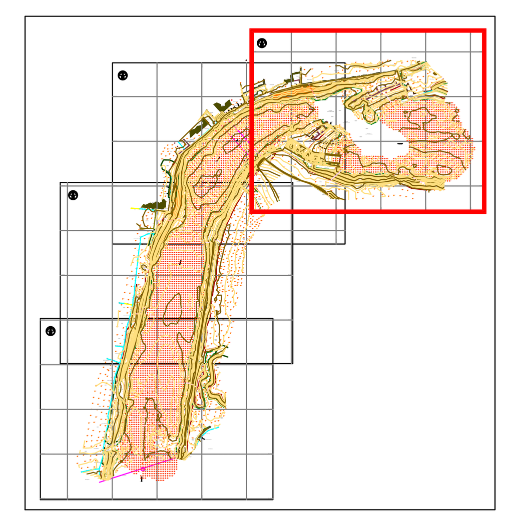
STORM GEOMATICS

Tel: 01608 664910
mail@storm-geomatics.com
www.storm-geomatics.com

SYMBOL KEY

- VERGE
- TOP OF BANK
- KERB
- FOLIAGE LINE
- FENCE
- CONCRETE
- BUILDING LINE
- BOTTOM OF BANK
- TREE - Species/Trunk Diameter/Height
- SLOPE SYMBOL
- TREE STUMP
- GATE
- SILT LEVEL
- HARD BED LEVEL
- WATER LEVEL
- INVERT LEVEL
- SOFFIT LEVEL
- GROUND LEVEL

SHEET LAYOUT (NOT TO SCALE)



NOTES:

ALL LEVELS ARE METRES ABOVE ORDNANCE DATUM UNLESS OTHERWISE STATED.
ALL DIMENSIONS & CHAINAGES ARE IN METRES UNLESS OTHERWISE STATED.
ADDITIONAL AS BUILT DATA PROVIDED BY BAM NUTTALL

SURVEY LEGEND

AB	AIR BRICK	FH	FIRE HYDRANT	PWF	POST & WIRE FENCE
AV	AIR VALVE	FHM	FIRE HYDRANT MARKER	RB	RIGHT BANK
B	BOLLARD	FL	FLOOR LEVEL	RE	RODDING EYE
BB	BELISHA BEACON	FP	FENCE POST	RS	ROAD SIGN
BDY	BOUNDARY	FWM	FOUL WATER MANHOLE	RTW	RETAINING WALL
BH	BOREHOLE	G	GULLY	RWP	RAINWATER PIPE
BL	BED LEVEL	GL	GROUND LEVEL	SC	STOP COCK
BRK	BRICKWORK	GP	GATE POST	SDP	STAND PIPE
BS	BUS STOP	GM	GAS MARKER	SK	SOAKAWAY
BM	BENCH MARK	GV	GAS VALVE	SL	SOFFIT LEVEL
BW	BRICK WALL	HW	HEAD WALL	SMH	SURFACE WATER MANHOLE
BWF	BARBED WIRE FENCE	IC	INSPECTION CHAMBER	SMP	SHEET METAL PILING
CBF	CLOSE BOARDED FENCE	IL	INVERT LEVEL	SP	SIGN POST
CIF	CORRUGATED IRON FENCE	IRF	IRON RAILING FENCE	SPF	STEEL PALLISADE FENCE
CL	COVER LEVEL	IWF	INTERWOVEN FENCE	STN	STATION
CLF	CHAIN LINK FENCE	JB	JUNCTION BOX	SV	SLUICE VALVE
CONC	CONCRETE	KIG	KERB INLET GULLY	SVP	SOIL VENT PIPE
CP	CONCRETE POST	LB	LEFT BANK	TBM	TEMPORARY BENCH MARK
CPF	CHESTNUT PALING FENCE	LFB	LIFEBUOY	TCB/TCP	TELEPHONE CALL BOX/POST
CR	CYCLE RACK	LP	LAMP POST	TC	TELECOM CABINET
CTV	CABLE T.V. MANHOLE	MB	MOORING BOLLARD	TMH	TELECOM MANHOLE
CUL	CULVERT	MF	MISCELLANEOUS FENCING	THL	THRESHOLD LEVEL
DK	DROP KERB	MH	MANHOLE	TL	TRAFFIC LIGHT
DL	DECK LEVEL	MKR	MARKER	TLB	TRAFFIC LIGHT BOX
DP	DOWNPIPE	MP	MOORING PILE	TP	TELEGRAPH POLE
DPC	DAMP PROOF COURSE	MRF	METAL RAILING FENCE	TRS	TIMBER RUBBING STRIP
DR	DRAIN	MS	MILE STONE	TS	TREE STUMP
DWB	DOG WASTE BIN	NB	NOTICE BOARD	TSR	TUBULAR STEEL RAILINGS
EA	ENVIRONMENT AGENCY	NRA	NATIONAL RIVERS AUTHORITY	VP	VENT PIPE
EB	ELECTRICITY BOX	OHC	OVERHEAD CABLE	WB	WASTE BIN
ECF	ELECTRIC CABLE FENCE	OS	OS Grid (15)	WL	WATER LEVEL/WATER LINE
ECP	ELECTRICITY CABLE PIT	OSR	OPEN STEEL RAILINGS	WM	WATER METER
EMH	ELECTRICITY MANHOLE	P	PILE	WMF	WIRE MESH FENCE
EP	ELECTRICITY POLE	PB	PILLAR BOX	WP	WOODEN POST
ER	EARTHING ROD	PM	PARKING METER	WPR	WOODEN POST & RAIL FENCE
ETL	ELECTRICITY TRANSMISSION LINE	PO	POST	WV	WATER VALVE
FB	FLOWER BED	PRF	POST & RAIL FENCE	YG	YARD GULLY
FBR	FOOTBRIDGE	PTM	PARKING TICKET MACHINE		

(Abbreviations apply to EA survey data only)

REV.	AMENDMENT	DRN	CHKD	DATE

CONTROL USED:

TYPE	EASTING	NORTHING	LEVEL
E5-1	423897.280	436935.729	39.492
E5-2	423953.225	436960.923	39.236

Environment Agency

Yorkshire Region (YOR)

Admiral House, Rose Wharf, 78 East Street, Leeds, LS9 8EE

PROJECT/WATERCOURSE

Newlay Remedial Works

SITE/LIMITS

Newlay Weir
Horsforth, Leeds
Topographic Survey
Sheet 1 of 4

SURVEYED BY: STORM GEOMATICS LIMITED Ref: 21AR21294

SURVEY DATE: MAY 2021

SCALE: 1:100 (A0 SHEET)	DRN: J.T	CHKD: L.W
DATUM: OS Newlyn (15)	DATE: MAY 21	DATE: MAY 21
GRID: OS Grid (15)	DRAWING NO. 274242–25_10	REV. –
CAD FILENAME: T_274242–25.dwg		