Flood Risk Assessment
Stobart Group Land Holdings
Ditton, Widnes, Cheshire

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Stobart Group Ltd

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1 Introduction

1.1 Background

ENVIRON UK Limited was commissioned by the Stobart Group to carry out a Flood Risk Assessment (FRA) of its current landholdings in Ditton Widnes. This effectively represents one site but comprises a number of land parcels for which there are redevelopment proposals, but these may be the subject of separate planning applications. On the assumption that each of these development areas will require or have already required an FRA, the decision was taken to produce one master FRA for all of the land-holdings. This ensures that flood risks to the whole integrated site are considered and also avoids substantial repetition and reworking of the FRA data to produce a dedicated FRA for each planning application on the same site. This approach has been agreed with the Environment Agency.

The site centre is located Ditton, near Widnes in Cheshire (NGR: SJ 503 844). The site is located approximately 1 km south west of Widnes town centre and comprises all land between Desoto Road to the east and Hale Road to the west and the former Tessenderlo chemical works site to the east of Desoto Road.

The area is located on the north bank of the Mersey Estuary close to Widnes industrial areas and approximately 1 km downstream of the Widnes - Runcorn road bridge.

The location of the site and key hydrological features are presented in Figure 1.1. Notably, three Brooks currently flow through the proposed development site: Ditton Brook, Marsh brook and Steward's Brook.

The site as a whole comprises seven principal areas as set out in Figure 1.2:

- Area A - the Foundry Lane Offices adjacent to the west of Ditton Brook (0.7 ha);
- Area B - the Foundry Lane Estate adjacent to the east bank of Ditton Brook (9.7 ha);
- Area C - the Reclamation Site (or "The Mound") adjacent to the east bank of Ditton Brook and immediately south of the Foundry Lane Estate (11.8 ha);
- Area D – the area to the north of the Reclamation Site (4.3 ha);
- Area E - the central area of the West Bank Dock Estate (also known as the Mathieson Road Site) in the central parts of the site, between the existing locations of Steward's and Marsh Brooks (21.8 ha);
- Area F – the small area to the north of the railway (2.9 ha); and
- Area G - the area which lies to the east of Marsh Brook (23.6 ha).

1.2 The Proposed Development

The intention of the proposed development of the site as a whole is to provide a high quality inter-modal freight park which involves the demolition of a number of old, redundant and possibly unsafe buildings around the West Bank Dock Estate area and the construction of new, purpose built warehouse facilities and associated ancillary uses. In order to facilitate this, there will be some considerable earthworks on the site, which will involve the excavation and treatment / stabilisation of all the Galligu (a by-product of chemical works in the local area) above ground level in the reclamation mound, with this area then being developed over (i.e. hard-surfaced).

The proposed new warehouses will be served by road and indirectly by rail (from the adjacent O'Connors operation, also part of Stobart Group). The new logistical enhancement of the site and the proposed facilities will include better circulation of traffic and optimised site usage. The individual new units that are to be built will comprise:

A large regional distribution centre on the main Foundry Land/West Bank Dock Estate (including part of the reclamation mound area);

A new warehouse facility to the south of the west bank dock part of the site (adjacent to PDM) which will be for the relocation of Rehau (an existing site tenant); and

A refrigerated goods storage centre on the former Tessenderlo area.

Each of these developments, whilst on the same site, will be initiated in phases and be the subject of separate planning applications. This FRA, however, covers all of the geographical areas concerned and should be regarded as the FRA report for each application.

In addition to the new proposed buildings, to accommodate the largest planned unit (the regional distribution centre which already has planning consent), the scheme will involve the diversion of Steward's Brook which currently bisects the site. The realignment will lead to the infilling of approximately 415 m of Steward's Brook and the creation of 640 m of new channel that will pass to the north of the new Rehau building and eventually join Marsh Brook. Marsh Brook and its outfall will be substantially upgraded as part of this scheme.

In order to facilitate the overall works and enable demolition of older buildings, temporary offices and associated parking areas are required on Area A (adjacent to Foundry Lane). This too is the subject of a planning application and again the FRA applies to this area also.

1.3 Proposed Diversion of Steward's Brook

The proposed redevelopment of the site involves the infilling of approximately 415 m of Steward's brook and the construction of a new channel which will flow into Marsh brook before discharging into the Mersey Estuary (Figure 1.3).

This new channel will run along the eastern boundary of the West Bank Dock area in a north to south direction for approximately 240 m. The watercourse will then flow in a west to east direction, for a further 400 m, before flowing into Marsh Brook. Therefore, the in-filled section of Steward's Brook will be replaced by approximately 640 m of channel which equates to a net gain in channel length of 225 m. This work will also involve improvements to the Marsh Brook channel in proximity to the site.

A hydraulic modeling exercise has been undertaken for this proposed diversion channel. Further details of this hydraulic modeling exercise are provided in Annex C.

1.4 Requirements for a Flood Risk Assessment

The requirements for a Flood Risk Assessment (FRA) are provided in "Planning Policy Statement 25: Development and Flood Risk" (PPS25), which came into effect in December 2006. This document confirms that flood risk is a material consideration that must be taken into account when considering applications for planning permission.

Paragraph E9 of PPS25 requires that an FRA should be submitted with planning applications for all proposals for new development in Flood Zones 2 and 3. As such, an FRA is an essential element in the overall assessment of the economic viability of the development as well as the acceptability in planning terms.

Guidance on the content of FRAs is contained in Annex E of PPS25 (which is reproduced in this report as Appendix A) and within PPS25 Development and Flood Risk – A Practice Guide (June 2008). Both these documents have been used to inform the scope and content of this FRA.

1.5 Consultation

In preparing this FRA, the EA (Gwen Scott – Environment Manager and Graham Todd – Development Control Officer) has been consulted in terms of their requirements for an FRA and the extent of available information on flood risk affecting the proposed development site and the adjacent watercourse.

Their views were also sought on the likely mechanisms of flooding and what measures they consider would be acceptable, with particular regard to the diversion of Steward's Brook.

1.6 Structure of this Report

This document constitutes a Master Flood Risk Assessment which covers the baseline conditions and the external flood risks to the whole proposed development area (all land owned by the Stobart's Group in the area, i.e. Areas A to G).

A feasibility study of options for the use of Sustainable Drainage Systems (SUDs) has been undertaken as part of this report. Further details on drainage will be prepared to accompany each subsequent planning application. These reports will offer a detailed assessment of the feasibility of adopting SUDs within each proposed development area.

The requirements of PPS25 for each planning application will be met by the Master FRA and the accompanying drainage assessment.

The Master FRA is structured as follows:

- Section 2: Baseline Environmental Conditions – a description of the site location, its topography, hydrology and geology;
- Section 3: External Flood Risk – an assessment of the vulnerability of the proposed development area to flooding from rivers, the sea, groundwater and sewers;
- Section 4: Drainage Principles – a brief assessment of the surface water management requirements over the whole proposed development area including an assessment of sustainable drainage opportunities; and
- Section 5: Summary and Conclusions – a review of the suitability of the development proposals in the context of site vulnerability to external flood risks and the requirements of PPS25.

2 Baseline Environmental Conditions

2.1 Location and Topography

The proposed development area is located to the west of Widnes on the north bank of the Mersey Estuary and approximately 1 km downstream of the Widnes-Runcorn road bridge.

The site is approximately 74 ha in total area and comprises seven principal areas as listed in Section 1.1.

Area A was previously greenfield but has been developed with temporary offices and associated parking. It is estimated that these offices will be in use for approximately 3 years.

Area C consists of a mound of Galligu (a by-product of chemical works in the local area) which has been vegetated and planted with trees.

The land within **Areas B, D, E, F** and **G** is currently occupied by industrial units and associated parking and access facilities.

The River Mersey runs to the south of the site. Ditton, Steward's and Marsh Brooks flow in a generally north-south direction through the site as detailed in Section 2.2.

2.1.1 Site Levels

A topographical plan of the site is shown in Figure 2.1. The levels of each area within the site are discussed below.

Area A

This site lies adjacent to the west bank of Ditton Brook (Figure 1.2). The site is split into two areas (north and south) by a central bank which connects with elevated land which surrounds the site to the north, west and south. These banks range in height from 8.71 to 9.52 metres above Ordnance Datum (mAOD).

Levels within a small proportion of the site at the northern corner drop to 7.29 mAOD. The majority of the northern section of the site ranges in height from approximately 7.5 to 7.83 mAOD. The development does not include any building within this section; the proposed temporary offices are located within the section to the south of the central bank, where levels range from 7.8 to 7.98 mAOD.

Area B

There is a slope along the northern boundary of Area B from east to west. Levels are approximately 8.5 mAOD in the north eastern corner of this area. In the north western corner, levels drop to approximately 6.8 mAOD.

Levels along the western boundary, adjacent to Ditton Brook, range from 6.0 to 7.0 mAOD.

Within the centre of the site, there are extensive areas which are raised due to buildings or tarmac areas. Levels here range from approximately 6.0 to 8.0 mAOD

Area C

The Reclamation Site is an engineered mound which rises above the surrounding land from 7.5 mAOD to a maximum of 28.5 mAOD. The mound will be flattened as part of the site's redevelopment. The re-modelled site level will be at a minimum of 8.1 mAOD.

Area D

Levels within Area D are all above 8.06 mAOD, which is the recorded elevation at the eastern boundary of the area adjacent to the current Steward's Brook channel. The site slopes to the south west with levels reaching 9.66 mAOD along the western boundary and 11.27 mAOD at the most southerly extent of the area.

Area E

Area E slopes from a maximum elevation of 15.7 mAOD in the south west to 8.5 mAOD in the north eastern corner at the junctions of Desoto and Mathieson Roads. The proposed Steward's Brook diversion channel will flow through this area before converging with Marsh Brook and flowing south into the Mersey (Figure 1.3).

Area F

Topographical information within Area F is sparse. Levels are known to range between 7.19 and 7.9 mAOD within the east of this area. Levels are not known to the west of this area.

Area G

Levels along the eastern boundary of the site, adjacent to Macdermott Road, range from 8.6 to 10.18 mAOD. The land slopes to the west with levels ranging from 7.0 to 7.5 mAOD within the western half of this area.

2.2 Hydrology and Flooding

The location of the site and key hydrological features are presented in Figure 1.1.

The Environment Agency's Indicative Floodplain Map (Figure 2.2) suggests that some areas of the site are within Flood Zones 2 and 3.

Three Brooks currently flow through the proposed development site: Ditton Brook, Marsh Brook and Steward's Brook. These channels receive *ad-hoc* discharges from the on-site drainage system and all three watercourses discharge into the Mersey Estuary. The River Mersey is situated immediately south of Area G, approximately 250 m south of Area E, 150 m south of Area C and 500 m south of Area B.

Ditton Brook flows along the western boundary of Areas B and C in a southerly direction and along the eastern boundary of Area A.

Steward's Brook currently enters the proposed development area at the boundary of Areas D and E. The Brook then flows in a south westerly direction along the eastern boundary of Area C before converging with Ditton Brook and flowing into the River Mersey.

The Marsh Brook channel enters the proposed development site from the north and then runs south along the boundaries of Areas E and G into the River Mersey. There is currently negligible flow through the Marsh Brook channel as there are several collapsed inflow and outflow pipes.

Ditton, Steward's and Marsh Brooks are all tidal. The normal tidal limit for Ditton Brook is upstream of the bridge, near the northern limit of Area B. The current normal tidal limit for Steward's Brook is approximately half way between the access road connecting Area C with Area E and the confluence of Ditton and Steward's Brooks. The tidal limit for Marsh Brook is not known as the outflow into the River Mersey has collapsed.

2.3 Flooding History

Consultation with the EA has revealed that the Foundry Lane Estate area (within Area B) was subject to tidal flooding in February 1990. This was a result of high tides backed by strong winds. It was determined that the event had a return period of 1 in 70 years. A flood level of 7.03 metres Above Ordnance Datum (mAOD) was measured at the Golden Triangle Complex.

The National Rivers Authority (now part of the EA) carried out a flood defence scheme in the mid 1990s and constructed a sheet pile wall along Ditton Brook. This wall has a level of 7.7 mAOD.

2.4 Geology and Hydrogeology

According to the British Geological Survey Solid and Drift Map for Runcorn (Sheet 97, Scale 1:50,000), the site is directly underlain by recent Marine and Estuarine Alluvium (the River Mersey is tidal within this area). This is further underlain by the Upper Mottled Sandstone of the Triassic Sherwood Sandstone Group at depths ranging from 12 metres below ground level (mbgl) to over 35 mbgl. The Sherwood Sandstone Group is underlain by Permian sandstones and further underlain by the Carboniferous Coal Measures to depth.

Estuarine alluvial deposits commonly comprise soft dark grey clays, silts and occasionally sands, which may be organic and contain shelly material. The Sherwood sandstone group comprises sandstones which are typically red in colour, occasionally mottled with yellow and white patches. The fine-grained sandstone bedrock is variably weathered and is encountered in varying forms of compaction from loose sand to hard rock.

The current course of the River Mersey flows to the south of the site and upstream [east] through an obvious geological gap between Runcorn and Widnes based upon approximate rock head contours. Discussions with the Contaminated Land Officer at Halton Borough Council (HBC) conjectured that a previous channel (palaeo-channel) lies to the north of the Runcorn promontory and was subsequently naturally infilled with fluviatile [possibly fluvio-glacial] deposits. This stratum may either be the result of deposition from glacial meltwaters during the Devensian [last Ice Age], a recent infilled meander of the River Mersey or a combination of both. A review of the Drift edition BGS Sheet 97 - Runcorn indicates that the depth to rock head in this area is in the region of -20 m to -40 m relative to OD.

2.5 Drainage

Area A was previously greenfield but now has some hardstanding areas as a result of the construction of the temporary offices. Runoff from this area drains directly into Ditton Brook.

Areas B and D are generally covered with hardstanding or limestone chippings. The hardstanding drains to Ditton Brook and the drains are in poor condition with no interceptor. The northern part of the site is drained by a 100 - 300 mm piped network, which has partially collapsed in places. The middle part of the site includes a 100 – 225 mm network and the southern site a 150 – 750 mm network. All three systems drain to the Ditton Brook and are thought to be in poor condition and will be replaced as part of the site wide redevelopment works.

The Reclamation Site (Area C) was used for the large scale disposal of wastes up to the late 1990s including Galligu and other materials. The County Council undertook remediation of the site which included: installing marginal sheet piling along Steward's Brook and Ditton Brook; the installation of a 1 m thick clay cap, overlain by 1.5 m of sub soil at the centre of the site and 0.4m to 1.0m in other areas.

Surface water drains have been installed in the clay cap of this area and discharge directly to Ditton Brook via the culverted channel between the site and Foundry Lane.

Areas E, F and G include several buildings (as surveyed by Atkins in 2000) and other structures including gas, oil and fuel storage tanks. The majority of the site is covered with hardstanding although this is in poor condition in the north eastern part of the site. The site drainage systems include two outfalls to Steward's Brook and one to Marsh Brook, which could not be located during the site walkover.

3 Assessment of External Flood Risk

3.1 Flooding Mechanisms

There is potential at the proposed redevelopment site for flooding from Ditton, Steward's and Marsh Brook as a result of extreme flood events.

The EA has advised that although fluvial flooding from the three Brooks could occur in high water levels, the highest flood levels are due to tidal conditions in the Mersey Estuary. The conditions which are most likely to cause high flood levels at the site are a combination of high astronomical tides, storm surges and westerly onshore winds. Flooding in these conditions could occur directly as a result of overtopping of flood defences along the Ditton shoreline to the south. Alternatively, flooding could occur as a result of backing up of water within the Brooks as a result of extreme tides / tide locking.

3.2 Tidal Flood Levels

As detailed in Section 2.2, the site lies partly within the EA's designated 200 year tidal floodplain (Figure 2.2). However, this floodplain does not take into account the presence of existing flood defences. A more precise definition of extreme tide levels are required to confirm whether alleviation options or restrictions on development should be considered.

An objective of this FRA is to define the 200 year tidal flood level at the site, to determine which parts of the site are above this limit and hence whether the development will have an impact on floodplain storage or will be affected by flooding.

A study into tidal flood levels at the proposed development site was undertaken as part of a previous Flood Risk Assessment at the site (ENVIRON 2007), which has been submitted to HBC and the EA associated with the planning application (Project Goldfinger) for the regional high bay distribution centre.

The conclusions of this study showed that the 200 year tide level was 7.30 m AOD. Further details of this study are provided in Annex B.

3.2.1 Climate Change Impact on Flood Levels

PPS25 requires that the definition of defence standards for tidal areas should include an allowance for climate change for the "design life of the development" which is often taken as 60 years. Current guidance suggests that in the North West of England an allowance of 4 mm per year should be added to predicted sea levels. Based on a 60 year design life, an allowance for 61 years of climate change has to be added to the 200 year extreme tide level derived in 2007. This equates to a 244 mm addition to the level of 7.3 mAOD. **The 200 year tide level used in this FRA is therefore 7.54 mAOD**.

3.3 Comparison of Site Levels with Extreme Tide Levels

Direct Flooding from The Mersey

The existing defence levels along the Mersey Estuary are believed to be at 9.0 mAOD and well above the predicted 200 year climate change corrected tidal flood level of 7.54 m AOD. Tidal flooding direct from the Mersey Estuary over the shoreline flood defences is therefore unlikely to occur as these defences are higher than the predicted extreme sea levels even allowing for wave heights of 1.0 m.

Flooding from Ditton Brook

This is most pertinent to Area A. The temporary offices are located above the climate change corrected 200 year tide level and the risk of flooding of these offices is, therefore, thought to be negligible. However, part of the Foundry Lane access road (which is outside of Stobart Group's ownership and control) could, under extreme conditions be prone to shallow flooding. The northern (unoccupied) corner of the site would also be at risk of flooding during the climate change corrected 200 year tidal event along with the section of Foundry Lane approaching the entrance to the site from the north. The predicted worst case flooding of these areas of the site would occur to a depth of approximately 250 mm, which is passable by vehicle, but this assumes a climate change corrected 200 year tide event (which is projected out over 60 years as sea levels gradually rise at an estimated 4mm per annum). The offices, are in fact temporary offices of relatively short term duration and will have ceased use long before there have been significant global sea level rises. For example, if such an event occurred in year 1, the flood depth may only be 10mm. Consequently, this flood risk is a long-term flood risk issue many decades off and is not material in the light of the current short term use of the site which would be the subject of any current planning application. In conclusion therefore, the temporary offices are not at flood risk, but there is a small long-term possibility that parts of the access road could flood to shallow depths, but not to an extent that makes it impassible, as such flooding of this area is not a cause for concern.

Area B is protected from the Ditton Brook by a 1.2 m high concrete wall between the site and Ditton Brook which the EA indicates has a minimum height of 7.7 mAOD. The southern and northern end of the wall runs into higher ground above 7.7 mAOD. The flood risk from Ditton Brook to Area B will, therefore, be negligible as the site is protected to at least the level of the climate change corrected 200 year tide. The flood risk to the Area C will also be negligible as ground levels will be at least 500 mm above the 200 year flood level of 7.54 mAOD.

Flooding from the New Steward's Brook and Marsh Brook Channels

The proposed diversion of Steward's Brook will converge with Marsh Brook at the border of Areas E and G, approximately 150 m south of the junction of Desoto and Macdermott Roads.

The improved Marsh Brook channel will then run south along the border of Areas E and G, immediately adjacent to the east of Desoto Road, towards the River Mersey.

If this proposed channel were to be overtopped, there would be significant risk of flooding, but the planned remediation and development of the former Tessenderlo site will raise site levels by up to 2m.

Hydraulic modelling of this new channel (Annex C) has shown that the residual risk of flooding from tidal levels within Steward's Brook or Marsh Brook is negligible.

The performance of the channel was tested in a scenario of Q_{med} flows within Marsh Brook and Stewards Brook occurring simultaneously with the 200 year tide event.

The water surface elevation, based on model parameters, can be considered to be a conservative estimate as a result of the choices used in setting up the hydraulic model as set out in the model report (Annex C). The model has been submitted to the EA and comments are yet to be received.

The channel was proved to be of sufficient cross-sectional area to prevent overtopping of the proposed diversion channel during the range of fluvial and tidal conditions modelled in the exercise.

The flood risk within areas D, E, F and G is, therefore, also negligible.

4 Drainage Principles

4.1 Summary of PPS 25 (ANNEX F): Management of Surface Water

PPS25 states that those proposing development are responsible for drainage designs which reduce flood risk to the development and elsewhere, potentially through the use of Sustainable Drainage Systems (SUDS).

In order to satisfactorily manage flood risk in new development, appropriate drainage arrangements are required to manage surface water and the impact of the natural water cycle on people and property.

PPS25 states that: "*Surface water arising from a developed site should, as far as is practicable, be managed in a sustainable manner to mimic the surface water flows arising from the site prior to the proposed development, while reducing the flood risk to the site itself and elsewhere, taking climate change into account. This should be demonstrated as part of the flood risk assessment*."

4.2 Requirements of Surface Water Management

Flooding results from sources external to the development site and rain falling onto and around the site. The sustainable management of this rainfall, described as surface water, is an essential element in reducing future flood risk to both the site and its surroundings.

Undeveloped sites generally rely on natural drainage to convey or absorb rainfall, the water either soaking into the ground or flowing across the surface into watercourses, providing a natural flow of environmental and ecological benefit. Sites currently or previously used for agricultural purposes may additionally have systems of underground drainage pipes as well as open ditches and watercourses.

To satisfactorily manage flood risk in new developments, appropriate surface water drainage arrangements are required, to manage surface water and the impact of the natural water cycle on people and property.

Sustainable Urban Drainage Systems (SUDS) aim to reduce the volume of runoff arising from a site at source and/or attenuate those flows that do arise such that the runoff rates being released into the wider environment are reduced. SUDS can rely on infiltration of runoff into the ground (using soakaways, swales, infiltration trenches and permeable pavements) or attenuation of flow at the surface (using filter strips or swales and attenuation basins / ponds). Infiltration-based SUDS require favourable ground conditions (i.e. uncontaminated and highly permeable ground) and surface SUDS require sufficient land to be available for the siting of SUDS structures.

SUDS cover the whole range of sustainable approaches to surface water drainage management including:

- source control measures including rainwater recycling and drainage;

- infiltration devices allow water to soak into the ground, that can include individual soakaways and communal facilities;
- filter strips and swales, which are vegetated features that hold and drain water downhill mimicking natural drainage patterns;
- filter drains and porous pavements to allow rainwater and runoff to infiltrate into permeable material below ground and provide storage if needed; and
- basins and ponds to hold excess water after rain and allow controlled discharge that avoids flooding.

The use of SUDS, where they are feasible, provides a significant contribution towards more sustainable development since they:

- manage environmental impacts at source, rather than downstream;
- manage surface water runoff rates, reducing the impact of urbanisation on flooding;
- protect or enhance water quality;
- are sympathetic to the environmental setting and the needs of the local community;
- provide opportunities to create habitats for wildlife in urban watercourses; and
- can encourage natural groundwater recharge.

4.3 SUDS Feasibility Assessment

On the basis of SUDS matrix (Table 4.1), there are a limited number of techniques that would be appropriate for use at this site given the shallow groundwater and the contamination status of the underlying shallow geology. Groundwater contamination precludes the use of infiltration-based SUDS and insufficient space and economic viability preclude the use of swales and / or balancing ponds. Therefore, the most appropriate SUDS option in this instance is to provide attenuation using an underground gravel sub-base beneath the car parking areas. However, such parking areas are relatively limited in extent and unlikely to make a significant contribution to surface water management.

TABLE 4.1: SUDS FEASIBILITY MATRIX

Technique	***Physical Constraints***	***Feasibility***
Permeable pavement / car park	Ideally requires a level site.	Feasible – requires an engineered, lined sub-base due to underlying ground contamination.
Green roofs	Sometimes classed as brown roofs and garden roofs; different levels of attenuation intensity etc; flat roofs are ideal; weight can be a structural constraint	Not feasible – due to weight bearing effects on roof of buildings
Bio-retention – shallow landscaped infiltration areas	Primarily used to remove pollutants from runoff and due to their shallow nature are not as effective at runoff attenuation as other SUDS techniques	Unfeasible – insufficient space available.
Soakaways and infiltration trenches	Require infiltration rates of 1 x 10^{-6} m/s or greater. Shallow soakaways or infiltration trenches would be required where groundwater is shallow (i.e. less than 2.0 mbgl).	Unfeasible – due to contaminated ground conditions.
Grassed filter strips – wide gently sloping areas of grass or other vegetation	Normally used to treat polluted runoff from car parks or roads. Not as effective at runoff attenuation as other SUDS techniques	Unfeasible – insufficient space available.
Infiltration basins / swales	Are widely applicable for attenuation and treatment of surface runoff by infiltration into the ground. Require slope of no more than 4-10% and can act as a substitute for soakaways where groundwater is shallow – need to consider the impact these techniques have on local groundwater levels	Unfeasible – due to contaminated ground conditions.
Filter drains	These are normally used adjacent to areas of car parking or roads and convey runoff via flow through an engineered substrate (normally gravel).	Feasible
Balancing ponds	These are permanent ponds that provide storage above the resting water level in the pond. Are appropriate for most sites but require suitable space. Require impermeable soils, or can be lined	Unfeasible – insufficient space available.

In the case of this site, there is also a fairly unique situation in that Steward's Brook is already heavily contaminated by the time it enters the site and there is a strong positive benefit to introducing as much clean dilution water as possible. Similarly, Marsh Brook has been a chemical drain for many decades and although it is to be relined and improved, there is still the potential for contaminated inflow from upstream of the area under Stobart Group's control. This too would benefit from clean water inputs. As such, the preference and planned approach for this site is to direct all surface waters to these brooks. There will, however, be some rainwater harvesting for the refrigerated storage unit on the former Tessenderlo site, which can use such water for the refrigeration plant.

Further details of the specific schemes will be provided in individual planning applications for each development area, but the assumption in the hydraulic modeling and flood risk assessment is that the surface waters incident on the site as a whole will be directed to the local surface water courses. The details will be provided for each discharge point as the scheme develops.

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5 Conclusions

The various land parcels that collectively constitute the site require an FRA on the basis of parts of the site being partly located within Flood Zones 2 and 3 and being over 1 hectare in surface area.

The key conclusions of this FRA are as follows:

- the 200 year (climate change corrected) tidal flood level of 7.54 mAOD has been used as the critical flood level for the site. This has been compared to proposed site levels and it has been demonstrated that none of the proposed buildings would be affected by flooding during this event (as summarised below);

- Area A: - the temporary office buildings are located above this level. The access to the temporary offices could be flooded during an extreme event but the depth of flooding is estimated to be shallow (i.e. less than 250 mm) and passable and this is a long term risk assuming gradual sea level rises over the coming decades. In the context of the use of the site for short term temporary offices and modular buildings, the flood risk to the road is negligible and the FRA predicts there is no flood risk to the offices themselves;

- Areas B and C: both these areas are above the 200 year climate change corrected flood level and protected from it by the defences along Ditton Brook;

- Areas D, E, F and G: hydraulic modelling of the proposed diversion of Stewards Brook for combined fluvial and tidal scenarios (the 20 year tide level and Qmed fluvial flow: Annex C), concludes that these areas would not flood from either Steward's Brook or Marsh Brook as the flood flows would be retained within bank;

- this Master FRA has included a SUDS feasibility assessment for the full site area, but there are limited opportunities for attenuation given the contaminated nature of the site (although there will be rainwater harvesting on the former Tessenderlo area). The general principles for controlling surface water will be similar for each area,,i.e. discharge of surface waters to the Brooks to provide clean "dilution" waters and help improve the quality of these water courses and their discharges.

In summary, the FRA predicts that the flood risks are not considered to be significant for this site or any of the individual development parcels within it and development need not be hindered or dictated by flooding issues.

Figures

Figure 2.2: Topographic Survey Plans

Annex A: The Assessment of Flood Risk

(Annex E of PPS25)

THE ASSESSMENT OF FLOOD RISK, PPS25 (ANNEX E)

GENERAL PRINCIPLES

E1. Properly prepared assessments of flood risk will inform the decision making process at all stages of development planning. There should be iteration between the different levels of flood risk assessment.

E2. Any organisation or person proposing a development must consider whether that development will not add to and should where practicable reduce flood risk. The future users of the development must not be placed in any danger from flood hazards and should remain safe throughout the lifetime of the plan or proposed development or land use.

E3. At all stages of the planning process, the minimum requirements for flood risk assessments are that they should:

- be proportionate to the risk and appropriate to the scale, nature and location of the development;
- consider the risk of flooding arising from the development in addition to the risk of flooding to the development;
- take the impacts of climate change into account;
- be taken by competent people, as early as possible in the particular planning process, to avoid misplaced effort and raising landowner expectations where land is unsuitable for development;
- consider both the potential adverse and beneficial effects of flood risk management infrastructure including raised defences, flow channels, flood storage areas and other artificial features together with the consequences of their failure;
- consider the vulnerability of those that could occupy and use the development, taking account of the sequential and exception tests and the vulnerability classification including arrangements for safe access;
- consider and quantify the different types of flooding (whether from natural and human sources and including joint and cumulative) and identify flood risk reduction measures, so that assessment are fit for the purpose of the decisions being made;
- consider the effects of a range of flooding events including extreme events on people, property, the natural and historic environment and river and coastal processes;
- include the assessment of the remaining (known as ‘residual’) risk after risk reduction measures have been taken into account and demonstrate that this is acceptable for the particular development or land use;

- consider how the ability of water to soak into the ground may change with development, along with how the proposed layout of development may affect drainage systems; and

- be supported by appropriate data and information, including historical information on previous events.

SITE SPECIFIC FLOOD RISK ASSESSMENTS (FRAS)

E8. At the planning application stage, an appropriate FRA will be required to demonstrate how flood risk from all sources of flooding to the development itself and flood risk to others will be managed now and taking climate change into account. Policies in LDD's should require FRA's to be submitted with planning applications in areas of flood risk identified in the plan.

E9. Planning applications for development proposals of 1 hectare or greater in Flood Zone 1 and all proposals for new development located in Flood Zone 2 and 3 should be accompanied by a FRA. This should identify and assess the risks of forms of flooding to and from the development and demonstrate how these flood risks will be managed, taking climate change into account. For major new developments in Flood Zone 1, the FRA should identify opportunities to reduce the probability and consequences of flooding. A FRA will also be required where the proposed development or change of use to a more vulnerable class may be subject to other sources of flooding, or where the Environment Agency, Internal Drainage Board and/or other bodies have indicated that there may be drainage problems.

E10. The FRA should be prepared by the developer in consultation with the LPA. The FRA should form part of an environmental statement when one is required by the Town and Country Planning (Environmental Impact Assessment) (England and Wales) Regulations 1999 as amended.

Annex B: Tide Level Calculations

Taken from a previous FRA for the proposed redevelopment site (ENVIRON 2007)

Tidal Flooding

In terms of the source of potential flooding, the overtopping of the flood defences or over banks on the adjacent watercourses could result in inundation of the development site. Therefore, an assessment of tidal flood levels and associated probabilities is required; these levels can then be compared to the levels of the defences.

Extreme Tide Levels

For many applications in the UK, such as Shoreline Management Plans and coastal flood defence schemes, estimates of extreme sea levels are taken from literature sources such as Graf[1] and Coles & Tawn[2] (Table 3.1). Their estimates were based on the fitting of extreme value distributions to annual maximum tide level data, but were based on slightly different fitting methods and different periods of record.

Table B.1: Extreme Sea Levels (m ODN) from Graff (1981) and Coles & Tawn (1990)

Location	Source	Return Period						Period of Record	No. of Years
		1	10	50	100	250	1000		
Hilbre Island	Graf	4.9	5.3	5.6	5.7	5.8	-	1854-1977	76
Hilbre Island	C&T	-	5.5	-	5.8	-	5.96	1854-1981	80
Gladstone Dock	Graf	5.3	5.7	5.9	6	6.1	-	1956-1977	20
Gladstone Dock	C&T	-	6.1	-	6.2	-	6.3	1956-1977	20
Princes Pier	Graf	5.4	5.8	6	6.1	6.2	-	1941-1977	37
Princes Pier	C&T	-	6.1	-	6.2	-	6.31	1941-1977	37
Eastham Lock	Graf	5.8	6.1	6.3	6.3	6.4	-	1956-1977	19
Eastham Lock	C&T	-	6.4	-	6.5	-	6.49	1956-1977	19

The results from these two sources are similar, principally because they are based on the same data sets at Eastham Lock, Gladstone Dock and Princes Pier. However, some differences are apparent and these differences reflect the use of different methods of fitting extreme value distributions to the annual maximum data.

The results suggest a 100 year tide level at Eastham Lock of 6.3 m (Graf) or 6.5 mAOD (Coles and Tawn) with a slight gradient up the lower part of the Mersey Estuary. Although these provide useful historical estimates of extreme sea levels, they are based on the

1 Graf (1981) An Investigation of the Frequency Distributions of Annual Sea level maxima at Ports around Great Britain. Estuarine, Coastal and Shelf Science, 12, 389-449.
2 Coles & Tawn (1990) Statistics of Coastal Flood Protection. Phil. Trans. R Soc London A, 332, 457-476.

available annual maximum data at that time and pre-date the major flood events detailed in Table 2.3. Table B.1 provides an outdated estimate of extreme tide levels as there are at least 23 years more data at the sites, which can make a significant difference to estimates provided by extreme value distributions.

The JBA 1998 report provides extreme sea level estimates (Table B.2) based on updated annual maximum tide level data for the Mersey Estuary and the results suggest a 100 year tide level of 6.1 - 6.4 m, with a slight gradient up the lower part of the Mersey Estuary.

Table B.2: Extreme Sea Levels using GEV-MLE Method, JBA (1998)									
	Return Period (years)								
Site	**1**	**5**	**10**	**25**	**50**	**100**	**200**	**500**	**1000**
Hilbre	4.27	5.22	5.38	5.57	5.71	5.85	5.98	6.14	6.26
Gladstone	4.7	5.63	5.76	5.92	6.02	6.11	6.2	6.3	6.37
Princes Pier	4.86	5.73	5.89	6.08	6.22	6.35	6.48	6.65	6.78
Eastham	5.18	6.07	6.17	6.29	6.36	6.42	6.47	6.53	6.57

As part of the EA North West Region Coastal Flood Risk Mapping study, Posfords[3] produced a report in July 2001 based on the JBA 200 year estimates at Gladstone and Princes Pier (Table B.2). For tide levels further up the Mersey Estuary, they used the slope of maximum recorded water levels for the 1990 and 1997 event which suggests water levels during extreme events increase by 1.147 m to 1.233 m between Eastham Lock and Howley or Westy over a distance of 36 km (Table B.3).

Table B.3: Record Levels for the 1990 and 1997 Events			
Location	**Chainage**	**26-Feb-90**	**10-Feb-97**
Hilbre Island	0	5.97	5.65
Gladstone Dock	1.5	6.04	5.89
Princes Pier	6.5	6.22	6.29
Eastham Lock	12	6.39	6.07
Howley/Westy	48.5	7.537	7.303

On this basis, Posfords extrapolated the JBA 100 and 200 year tide levels at Gladstone and Princes Dock upstream to Westy (Table B.4). This suggests a 100 year tide level of 7.15 m and a 200 year level of 7.28 mAOD at the confluence of Ditton Brook and the River Mersey.

Table B.4: Extreme Sea Levels, Posfords (2001)			
Chainage	**Location**	**100 Year (m ODN)**	**200 Year (m ODN)**
0	Confluence with Coast	6.11	6.2
1.5	Gladstone Dock	6.11	6.2
6.5	Princes Pier	6.35	6.48
30	Ditton Brook	7.15	7.28
48.5	Westy Flow Gauge	7.67	7.8

3 Posford Duviver (July 2001) Coastal Flood Risk Mapping, Environment Agency North West Region

The use of extreme value distributions for historic tide level data can be considered inappropriate. Therefore, alternative methods of estimating extreme sea levels are also considered. Measured tides are a combination of the "harmonic" tide due to lunar and solar planetary movements and a meteorological or "surge" element due to changing atmospheric pressure. The measured data, particularly extreme AMAX series, may include the harmonic tide and an unknown surge element, the peak of which may or may not have coincided with the harmonic high tide. Therefore, in calculating a design sea level for a future date it is essential to consider the combined probability of extreme harmonic tides and tidal surges. Estimates, based on the extreme value analyses of raw AMAX data, consider the measured harmonic and surge elements combined but there is no way of knowing how much surge is included in the measured data.

A second major problem with extreme sea level estimates based on recorded data sets is that extrapolation of 19 years of data (as at Eastham Lock) to obtain the 100, 500 or 1000 year events is fraught with uncertainties. At the higher return periods, which are well beyond the length of the data set, the confidence limits in such estimates are often very large. In flood frequency analyses the Flood Studies Report[4] (FSR) recommended the extension of frequency curves to no more than 2N, where N is the number of years of records and the more recent Flood Estimation Handbook (FEH) suggests 0.5*N as the maximum extension. The error bands at extreme return periods are often very large and such extrapolation is not recommended.

Another area of concern is that Westy and Howley are far upstream of the tide gauges and Posford's linear interpolation of the water surface slope is based on only one event. A large amount of AMAX data exists for gauges along the Mersey; therefore, this data has been used in this FRA to estimate extreme tide levels at Ditton Brook.

Spatial Revised Joint Probability Method

A detailed analysis of the combined probability of surge and astronomical tide levels has been provided by the Proudman Oceanographic Laboratory (POL) and is referred to as the Spatial Revised Joint Probability Method (SRJPM). The reasons for adopting the SRJPM method, as opposed to literature or frequency analyses of AMAX values are as follows:

- the SRJPM uses hourly values from a data series hence more data is used than annual maxima;
- the design levels are not based on extrapolation of the data;
- long records of AMAX data sometimes show the 18.6 years of lunar periodicity. This means that the record is not random (or stationary), and the use of extreme value analysis is theoretically incorrect; and
- the method considers and then combines still water levels and surge separately and therefore gives a better representation of the T year level from combined probability analyses.

4 Natural Environmental Research Council (1975) Flood Studies Report

The basis of SRJPM method is detailed in Dixon & Tawn's POL Report 1165 but the combined probability analyses can be simplified to:

Design Level = 1 year level + T year growth level + MSL rise (Trend) + ODN correction

Each of these factors is considered to provide the 2000 and 2060 extreme sea levels estimates in the Mersey Estuary.

The 1 year tide level (i.e. the tide level with a return period of one year) can be extracted from the POL report at the nearest tidal node in the POL report (Hilbre Island) which provides a one year level of 5.16 mAOD. This 1 year level may be updated if tide level data exists at a nearby gauge. Although the JBA report provided 1 year tide levels at Hilbre, Gladstone Dock, Princes Pier and Eastham Lock these were based on available data at that time and the one year level at each gauge has been recalculated based on AMAX data and the recommended Generalised Extreme Value distribution. Therefore, the GEV values in Table B.5 are adopted.

Table B.5: The One Year Sea Level at the nearest Tide Gauges				
SRJPM Node	**Site**	**SRJPM (mAOD)**	**JBA (mAOD)**	**Gringorten Plotting Posn.**
64	Hilbre	5.381	4.27	4.51
	Gladstone	-	4.70	4.96
	Princes	-	4.86	5.07
	Eastham	-	5.18	5.44

The increase in sea level (or growth curve) for various return periods is provided in the POL report. The data-adjusted 1 year level at these gauges is already to OD and for 2000 and thus further adjustments using these parameters are not required.

The design levels for the year 2000 are given in Table 3.6 based on data estimates of the one year level and SRJPM growth curves.

Table 3.6: SRJPM Extreme Seal Levels (m AOD) for Year 2000								
Location	**Return Period (Years)**							
	1	**10**	**25**	**50**	**100**	**200**	**500**	**1000**
Hilbre	4.51	5.06	5.28	5.41	5.63	5.78	5.99	6.17
Gladstone	4.96	5.51	5.73	5.86	6.08	6.23	6.44	6.62
Princes	5.07	5.62	5.84	5.97	6.19	6.34	6.55	6.73
Eastham	5.44	5.99	6.21	6.34	6.56	6.71	6.92	7.10

For the year 2000, Table 3.6 indicates the 200 year level of 6.23 m AOD at Gladstone, 6.34 m AOD at Princes Pier and 6.71 m AOD at Eastham Lock.

Water Surface Slope

5 Dixon and Tawn (1997), Estimates of Extreme Sea Level Conditions - Spatial Analyses for the UK Coast. POL Report 118.

As detailed above, the Posford's[6] report provided extreme sea levels for the Mersey Estuary based on the calculated T year level at Eastham Lock tide gauge and the increase in water levels between Eastham and Westy.

The surface water slope for the 1990 and 1997 events of 1.147 m and 1.233 m were adopted and assumed to apply linearly over a distance of 36 km (Table B.4). The slope of the maximum recorded water level for other events (Table B.7) suggests an average difference of 0.929 m between Eastham Lock and Howley Weir and that levels at Ditton would be 0.458 m higher than those at Eastham Lock based on linear interpolation. Whilst there are a range of differences in water levels, these do not appear to relate to high levels at Eastham, but to high levels at Westy. This suggests a fluvial influence may result in different and higher surface water slopes in 1990 and 1997 and that Posfords may have overestimated the upstream water levels. However, the maximum water level difference of 1.2 m has been adopted, which will provide a worst case or more extreme estimate of the T year tide levels.

Table B.7: Difference in Water Levels between Eastham Lock and Howley Weir

Year	Eastham Dock Level (mAOD)	Howley Weir Level (mAOD)	Difference (m)
1956	5.66	6.64	0.98
1957	5.87	6.77	0.90
1958	5.99	6.49	0.50
1959	5.72	6.70	0.98
1960	5.66	6.49	0.83
1961	5.78	6.70	0.92
1962	5.90	6.64	0.74
1963	5.93	6.72	0.79
1964	5.75	7.39	1.64
1965	5.90	6.95	1.05
1966	6.02	6.92	0.90
1967	5.90	6.80	0.90
1968	5.99	7.00	1.01
1970	5.72	6.80	1.08
1974	5.81	6.64	0.83
1975	5.76	6.49	0.73
1976	6.26	6.90	0.64
1977	6.16	6.95	0.79
1990	6.39	7.54	1.15
1997	6.07	7.30	1.23

The revised extreme sea levels for the Mersey Estuary are provided in Table B.8.

Table B.8: Revised Extreme Sea Levels (m AOD) for Year 2000, based on a slope of 1.2 m

Chainage	Location	Posfords 200 Year	200 Year	500 Year	1000 Year

6 Posford Duviver (July 2001) Coastal Flood Risk Mapping, Environment Agency North West Region

Table B.8: Revised Extreme Sea Levels (m AOD) for Year 2000, based on a slope of 1.2 m					
Chainage	**Location**	**Posfords 200 Year**	**200 Year**	**500 Year**	**1000 Year**
0	Confluence with Coast	6.20	5.78	5.99	6.17
1.5	Gladstone Dock	6.20	6.23	6.44	6.62
6.5	Princes Pier	6.48	6.34	6.55	6.73
12.0	Eastham		6.71	6.92	7.10
30.0	**Ditton Brook**	**7.28**	**7.30**	**7.51**	**7.69**
48.5	Westy Flow	7.80	7.91	8.12	8.30

This provides a 200 year extreme level at Ditton of 7.30 mAOD, a 500 year level of 7.51 m AOD and a 1000 year level of 7.69 m AOD. Whilst these are similar to the extreme tide level estimates in Table B.4, they are based on the latest methods and we can have greater confidence that these provide reasonable estimates.

Annex C: Hydraulic Model Report

Details of the Hydraulic Model for the Proposed Diversion of Steward's Brook

Hydraulic Model Report

The proposed scheme will involve the diversion of Steward's Brook which currently bisects the site. The realignment will lead to the infilling of approximately 415 m of Steward's Brook. This channel will be replaced by a new channel (of total length 640 m) which will flow into Marsh Brook.

The proposed channel will include the removal of a reverse weir from the bed of Steward's brook as it enters the site. Concerns were raised through consultation with the Environment Agency (EA) that this reverse weir would lead to increased siltation.

Objectives

The objective of this hydraulic modelling exercise is to demonstrate the hydraulic performance of the proposed Steward's Brook diversion channel during a range of different fluvial and tidal conditions. The scenarios modelled, in consultation with the Environment Agency (EA), are as listed in Table C.1.

Table C.1 – Hydraulic Modelling Scenarios

Scenario	Fluvial Flow	Tidal Condition
A	Q20 Flow	Mean High Water Springs
B	Q100 Flow	Mean High Water Springs
C	Qmed Flow	200 year Tide level

The removal of the reverse weir will reduce the average slope of the proposed channel. The average slope of the proposed channel prior to the removal of the weir would be 1 in 510. This slope will be reduced to 1 in 600 by the removal of the weir.

The aim of the model is to determine that the reduced slope of the diversion channel is sufficient to maintain flow in all fluvial and tidal conditions requested and to ensure that flow will not overtop the banks of the proposed diversion channel.

The bed invert levels of the proposed diversion channel lie at elevations between 2.52 and 4.13 m above Ordnance Datum (AOD).

Flood Flow and Tide Level Estimation

For flood probability studies, the derivation of a flood frequency curve should be based on the methodology described in the Flood Estimation Handbook (FEH). The calculation of flood flows and a description of the hydraulic model are provided below.

A range of flood flows have been calculated (Qmed, 20 and 100 year return periods) using the FEH statistical method and the revitalised rainfall-runoff (ReFH) method.

The Mean High Spring Tide Level and 200 Year Tide Level have been determined using methods set out in a previous Flood Risk Assessment (FRA) undertaken for the proposed development in 2 October 2007 by ENVIRON UK Ltd.

Rainfall Run-off Method

Flood flows can be estimated indirectly using the rainfall-runoff method. This is often the preferred method where rainfall records are more plentiful than river flow records, e.g. in small ungauged catchments. Rainfall records are generally longer than river flow records.

The rainfall-runoff method involves creating a design storm and running it through a simple catchment model. It can thus be used to provide peak flows and hydrographs and compared to results derived using the Statistical method.

Several problems have been identified in the FEH rainfall-runoff method and the ReFH method has recently been developed in order to make improvements to the methodology. The ReFH method has been used to derive a rainfall hydrograph and a flow hydrograph for the same storm event.

Flow rates for the required return periods have been calculated for both Steward's Brook and Marsh Brook separately for input into the hydraulic model.

The full results of the ReFH method are shown in Appendix A of the original FRA (ENVIRON, 2007) and the resulting peak flows are presented in Table C.2 below:

Table C.2 – Estimated Flows for Marsh brook and Steward's Brook (m^3/s)

	Marsh Brook	Steward's Brook
Qmed	0.5	2.5
Q20	1.0	4.7
Q100	1.5	6.7

Statistical Method

Flood flows can be estimated using the FEH Statistical method. For most applications, this approach is considered more robust than the FEH rainfall-runoff method.

The Statistical Method has been applied to Steward's Brook in this exercise in order to offer a comparison to and to validate the ReFH method.

As there are no gauging stations in the immediate vicinity of Steward's Brook, it is appropriate to use the FEH Statistical pooling group method at this site.

The pooling group method is based on a two stage approach:

- calculation of the index flood (the median annual flood, QMED) derived from catchment descriptors (QMED-CD), this is then adjusted using the QMED-CD to QMED-flow data ratio derived at nearby (donor) or hydrologically similar (analogue) EA gauging stations; and
- once the index flood (QMED) has been derived, various extreme value distributions are fitted to a pooled group of annual maximum flow data to estimate the T year flows and produce a flood frequency curve.

This procedure is described in the following sections.

The Index Flood

The Index Flood for Steward's Brook at the upstream site boundary has been calculated using FEH catchment descriptors (Table C.3) extracted from the FEH-CD ROM at the required location. This suggests a QMED of 2.472 m^3/s. A full definition of the parameters in Table C.3 is given in the FEH.

Table C.3 Catchment Descriptors

Place	**Steward's Brook**
Grid Ref	SJ,49600,84150
AREA	4.8
FARL	1
PROPWET	0.37
ALTBAR	22
BFIHOST	0.376
DPLBAR	2.87
DPSBAR	19.2
LDP	6.03
SAAR	798
SPRHOST	39.8

Table C.3 Catchment Descriptors	
Place	**Steward's Brook**
URBEXT 1990	0.267
URBEXT adjustment[7]	0.283
QMED-rural	2.429
QMED-urban	2.472

Flood Frequency Curve

Calculation of a flood frequency curve and the 20 and 100 year floods for Steward's Brook requires the construction of a pooling group and the fitting of an extreme value distribution to the pooled group data. The FEH derived pooling group was constructed with a target return period of 110 years to allow for removal of stations and yet maintain the 5T requirement of FEH.

Sites within the pooling group were removed based on their correlation with Steward's Brook in terms of values for BFIHOST, FARL, SAAR and Area. The resulting pooling group had 20 sites with 593 station years (Table C.4).

Table C.4 Final Pooling Group for Leamonsley Brook						
Site	**Yrs**	**L-CV**	**L-Skw**	**L-Kurt**	**Discord**	**Dist**
Crimple at Burn Bridge Near Pannal	31	0.195	0.063	1.125	0.234	0.557
Hodge Beck at Bransdale	42	0.225	0.297	0.238	0.142	0.941
Bevern Stream at Clappers Bridge	34	0.217	0.268	0.257	0.228	1.096
Leven at Easby	26	0.376	0.421	0.336	1.788	1.167
Usway Burn at Shillmoor	22	0.316	0.334	0.140	1.845	1.197
Browney at Lanchester	15	0.222	0.212	0.054	1.432	1.211
Dove at Hollinsclough	25	0.272	0.388	0.292	0.499	1.413
Crumlin at Cidercourt Bridge	24	0.174	0.253	0.182	0.638	1.425

[7] Using National Average Model of Urban Growth

Table C.4 Final Pooling Group for Leamonsley Brook						
Site	**Yrs**	**L-CV**	**L-Skw**	**L-Kurt**	**Discord**	**Dist**
Hamps at Waterhouses	18	0.287	0.397	0.397	1.186	1.436
Lod at Halfway Bridge	30	0.274	0.207	0.118	0.448	1.444
Boyd at Bitton	30	0.260	0.125	0.116	0.571	1.461
Conder at Galgate	37	0.187	0.059	0.079	1.056	1.475
Bedburn Beck at Bedburn	43	0.225	0.225	0.127	0.671	1.479
Rookhope Burn at Eastgate	20	0.152	.0117	0.117	0.991	1.509
Chad Brook at Long Melford	36	0.317	0.167	0.236	1.763	1.534
Ravarnet at Ravarnet	31	0.192	0.193	0.213	0.471	1.540
Laver at Ripon	26	0.213	0.368	0.281	0.681	1.542
Whittle at Quidenham	35	0.351	0.169	0.117	1.747	1.578
Uck at Isfield	40	0.278	0.343	0.371	1.069	1.579
Coal Burn at Coalburn	28	0.191	0.415	0.367	1.540	1.580
Total	**593**					
Weighted means		**0.245**	**0.242**	**0.197**		

Two extreme value distributions are often used on the pooled group data: (i) the Generalised Logistic (GL) and (ii) the General Extreme Value (GEV) distribution both fitted to the annual maximum data by the method of L-Moments. The GL is a 3 parameter distribution and normally produces a similar fit to the GEV. The findings of the FEH indicate that the GL distribution can often provide the best fit to extreme value flood series although WINFAP can be used to indicate the best fit distribution for each site.

The L-moments fitting method is recommended by the FEH as it was found to give marginally better results than more traditional methods such as probability weighted moments. For this reason, it was adopted as the preferred method of fitting. The theory behind L-moments and a more general coverage of frequency distributions is given in the FEH.

The results of the frequency analyses, extended to the 100 year return period and based on the adjusted QMED of 2.472, are summarised in Table C.5.

On the basis that the GL distribution is the recommended FEH distribution by WINFAP, the 100 year flow was determined to be 6.076 m^3/s. This compares to a 100 year flow using the ReFH method of 6.7 m^3/s.

Table C.5 Results of Generalised Logistic Frequency Analysis for Steward's Brook					
Return Period (Years)					
2	**5**	**10**	**25**	**50**	**100**
2.472	3.325	3.901	4.494	5.351	6.076

Discussion

Selection of Flow Estimation Method

The FEH Statistical method is generally preferred as it is more robust and involves fewer assumptions than the ReFH method.

However, the flows derived by the ReFH method have been selected for use in the hydraulic model. The ReFH method is often preferred for small ungauged catchments and it provides a higher peak flow for the 100 year return period than using the statistical method and is therefore more conservative.

Climate Change

Due to uncertainties in flood estimation and expected climate change impacts, hydrological analyses of flood flows and definition of defence standards should include an allowance for increased flows due to climate change. PPS25 requires an allowance for a 20% increase in peak flows over the next 60 years for fluvial rivers.

The ReFH method estimated peak floods in Table C.2 are therefore adjusted to include an allowance for climate change by increasing peak flows by 20%. The results of this adjustment are shown in Table C.6.

Table C.6 – Estimated Flows for Marsh Brook and Steward's Brook (m^3/s) including an allowance for Climate Change		
	Marsh Brook	**Stewards's Brook**
Qmed	0.6	3.0
Q20	1.2	5.64
Q100	1.8	8.04

Tide Levels

A flood level study was undertaken as part of a previous FRA undertaken by ENVIRON UK Ltd (October 2007) for the proposed redevelopment. The details of this study are presented in Annex B of this report and have been used to inform this modelling exercise in terms of deriving extreme tide levels.

The previous FRA suggested that the 200 year tidal level, prior to climate change adjustment, at Ditton is 7.30 mAOD.

PPS25 requires that the definition of defence standards for tidal areas should include an allowance for climate change for the "design life of the development" which is often taken as 60 years. Current guidance suggests that in the North West and north East of England an allowance of 4mm per year should be added to predicted sea levels. Based on a 60 year design life, an allowance for 61 years of climate change has to be added to the 200 year extreme tide level derived in 2007. This equates to a 244 mm addition to the level of 7.3 mAOD. The 200 year tide level used in this modeling exercise is therefore 7.54 mAOD.

The previous FRA suggested that levels at Ditton would be 1.07 m higher than those at the Proudman Oceanographic Laboratory (POL) tide gauge at Gladstone Dock, based on linear interpolation. Using this assumption, the Mean High Water Springs level at Ditton will be 5.46 mAOD based on a MHWS level of 4.39 mAOD at the POL gauge.

The estimated fluvial and tidal conditions for each model scenario are presented in Table C.7.

Table C.7 – Hydraulic Model Scenario Details

Scenario	Fluvial and Tidal Conditions	Steward's Brook Flow m^3/s	Marsh Brook Flow m^3/s	Tide Level mAOD
A	Q20 Flow and MHWS	5.64	1.2	5.46
B	Q100 Flow and MHWS	8.04	1.8	5.46
C	Qmed Flow and 200 year tide level	3	0.6	7.54

Hydraulic Model

A HECRAS hydraulic model of the Steward's Brook diversion channel and the re-constructed section of Marsh Brook has been developed to estimate the potential impacts of a range of fluvial and tidal conditions on local hydraulics. The model set up, calibration and results are detailed in this section.

The model is based on the design drawings for the proposed channel over a distance of 1034 m. This covers the extent of the proposed channel from where Steward's Brook enters the site from the north via a cutting under the railway.

The Brook bed area and wildlife berm area is modelled as the main channel. This is the lowest part of the channel cross section and is designed to maintain flow during normal and low flow conditions. The embankment areas within the channel cross section have been modelled as right and left out-of-bank areas. This area is designed to accommodate flow during extreme events and will be naturally landscaped.

The designations as main channel or out-of-bank areas have necessitated different values of Mannings 'n' to be applied in the hydraulic model. This represents the difference in hydraulic roughness characteristics between the main channel and the banks.

The model includes a proposed confluence with Marsh Brook and the outflow of the combined channel into the Mersey via a culverted section. The model also includes three other culverted sections where bridges cross the channel.

Two 'dummy' cross sections were added at the upstream boundary of Steward's Brook. These cross sections have been set as copies of the proposed channel cross sections. This allows the full Q100 flow of 8.04 m^3/s to enter the proposed diversion channel. However, in reality, the channel inflow will be significantly restricted by an existing pipe that conveys flow under the railway which has a known capacity of 1.85 m^3/s (calculated using Manning's equation). By allowing a larger flow to enter the diversion channel than would be possible from the existing pipe, the model approach is therefore extremely conservative but has been set up in this way to demonstrate the performance of the diversion channel in a hypothetical worst-case scenario.

A 'dummy' section has also been added at the downstream boundary of the model to represent the outfall of Marsh Brook into the River Mersey.

The section spacing and the Mannings 'n' values adopted for the left floodplain, right floodplain and river channel are presented in Table C.8.

A value of 0.035 for Mannings 'n' within the channel has been chosen based on the proposed nature of the channel bed. The right and left out of bank areas are to be landscaped and therefore a Manning's 'n' value of 0.05 has been selected.

Table C.8: River Section Details					
Section ID	**Section Description**	**Chainage (m) from downstream section**	**'n' left**	**'n' channel**	**'n' right**
0	Dummy Section representing the outflow of the combined Brooks into the Mersey	0	0.05	0.035	0.05
59	Section upstream of outflow culvert	59	0.05	0.035	0.05
409	Section after confluence of Marsh and Steward's Brooks	350	0.05	0.035	0.05
459	Section of Marsh Brook upstream of confluence with Steward's brook	50	0.05	0.035	0.05
469	Section at upstream model boundary on Marsh Brook	10	0.05	0.035	0.05
434	Section of Steward's Brook between confluence and culvert near roundabout	25	0.05	0.035	0.05
459	Section of Steward's Brook upstream of culvert near roundabout	25	0.05	0.035	0.05
584	Section downstream of middle culvert	125	0.05	0.035	0.05
634	Section upstream of middle culvert	50	0.05	0.035	0.05
809	Section on bend in Steward's Brook channel	175	0.05	0.035	0.05
929	Section Downstream of Culvert 1	175	0.05	0.035	0.05
1034	Dummy Section upstream of culvert 1	105	0.05	0.035	0.05
1059	Dummy section at upstream model limit of Steward's Brook	25	0.05	0.035	0.05

The coefficients used in the model (e.g. channel roughness, weir coefficients) are based on estimates provided in the HECRAS User and Reference manuals as supplied with the HECRAS model.

The main hydraulic structures included in the Steward's Brook model are detailed in Table C.9.

Table C.9: Hydraulic Structure Details

Section	Length (m)	Shape	Height (m)	Width (m)	US Invert (m OD)	DS Invert (m OD)	Bridge Deck (m OD)
58	58	Boxed Culvert	1.5	3.6	2.52	2.39	8.495
458	15	Boxed Culvert	1.5	3.6	3.19	3.16	9.165
624	25	Boxed Culvert	1.5	3.6	3.46	3.41	9.435
1029	100	Boxed Culvert	1.5	3.6	4.13	3.98	9.995

The HECRAS model includes the major flood flow routes and the key structures on Steward's Brook.

The model was used in steady state backwater mode with a variety of flow/tide scenarios as presented in Table C.7.

The upstream boundaries of the two Brooks are where the flows in Table B.7 are assumed to occur. The downstream boundary is sufficiently downstream of the area of interest to allow the impact of any errors in the boundary to be reduced. The calculated water levels are based on a known water depth downstream boundary condition using the tidal levels set out in Table C.7.

Model Calibration

Wherever possible, a hydraulic model should be calibrated against recorded flows and/or water levels from observed flood events, historic information and/or anecdotal accounts of flooding. Due to the lack of any flood level information, the model has not been calibrated. A range of parameters has therefore been tested and the sensitivity of changing the channel and floodplain roughness and the blockage of structures has also been considered (Table C.11).

Model Results

Performance of the Proposed Diversion Channel

With the initial model parameters, the average slope of the proposed diversion channel (1 in 600) is sufficient to maintain flow.

Flow is contained within the main channel in all modelled scenarios. There is therefore no significant risk that surface water elevations will be sufficient to overtop the proposed diversion channel.

Table C.10 – Bank Levels and Water Surface Elevations (mAOD) for Scenario C (Qmed flow with 200 year tide level)

River Section	Right Bank	Left Bank	Water Elevation
Marsh Brook 0	8.91	7.21	7.54
Marsh Brook 59	9.04	7.34	7.58
Marsh Brook 409	9.62	7.92	7.58
Marsh Brook 459	9.703	8.003	7.58
Marsh Brook 469	9.719	8.019	7.58
Steward’s Brook 439	9.66	7.96	7.58
Steward’s Brook 459	9.71	8.01	7.61
Steward’s Brook 584	9.918	8.218	7.61
Steward’s Brook 634	10.001	8.301	7.63
Steward’s Brook 809	10.292	8.592	7.64
Steward’s Brook 929	10.492	8.792	7.64
Steward’s Brook 1034	10.667	8.967	7.67
Steward’s Brook 1059	10.709	9.009	7.67

Sensitivity Analysis

In order to test the sensitivity of the model, several runs were undertaken with blockages in the culverts included in the model.

Table C.11 – Increases in Water Surface Elevation as a result of a 30% Blockage of Culverted Sections

River Station	Increase (mm)
Marsh Brook 0	0
Marsh Brook 59	50
Marsh Brook 409	50
Marsh Brook 459	50
Marsh Brook 469	50
Steward's Brook 439	50
Steward's Brook 459	70
Steward's Brook 584	70
Steward's Brook 634	110
Steward's Brook 809	100
Steward's Brook 929	100
Steward's Brook 1034	140
Steward's Brook 1059	140

Running the model with a 30% blockage of all culverts leads to a water surface elevation during a Q20 event and a 200 year tide level maintained within the proposed Steward's brook diversion channel.

A test of Mannings 'n' values has also been undertaken. Variations in the Mannings 'n' value of 10% within the cannel and the over bank areas have been modelled.

There are negligible changes in water surface elevation with changes of +/- 10% in the Mannings 'n' value in the channel and the overbank areas.

Conclusions

The water surface elevation, based on initial model parameters, can be considered to be a conservative estimate as a result of the choices used in setting up the hydraulic model as listed below:

- the model has been set up in the steady state backwater mode. This does not take into consideration any floodplain storage in the model;
- the ReFH method has been used to derive flood flows in Steward's Brook and Marsh Brook. This method is known to provide higher flow rates than other methods such as the statistical method; and
- the model is based on unrestricted Qmed, Q20 and Q100 flows into the upstream boundary of the Steward's Brook diversion. It is known, however, that the existing piped inflow restricts flow to a maximum rate of 1.85 m^3/s.

The average slope of the proposed channel after the removal of the reverse weir in the bed of Steward's Brook as it enters the site will be 1 in 600. This modelling exercise has demonstrated that this average slope of the proposed diversion channel is sufficient to maintain flow during all modelled scenarios (Table C.1).

The channel is of sufficient cross-sectional area to prevent overtopping of the proposed diversion channel during the range of fluvial and tidal conditions agreed with the Environment Agency.