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Client: Folkestone and Hythe District Council

Technical Addendum – Discharge of Surface Water Runoff (Watercourse) for the Proposed Development at Princes Parade, Hythe, Kent

December 2018

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Contents Amendment Record

This report has been issued and amended as follows:

Issue	Revision	Description	Date
1	0	Draft report issued by email.	14 September 2018
2	1	Final report issued by email.	21 September 2018
3	2	Report revised to include additional details on volume storage. Revised Report issued in Draft.	30 November 2018
4	2	Report issued in final.	21 December 2018

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Document Verification

Issue	Revision	Date:	14 September 2018
1	0	Author(s):	Stephen Hayward
		Checked By:	Sebastian Bures
		Director Sign Off:	Simon Maiden-Brooks
Issue	**Revision**	**Date:**	**21 September 2018**
2	1	Amendments By:	Sebastian Bures
		Director Sign Off:	Simon Maiden-Brooks
Issue	**Revision**	**Date:**	**30 November 2018**
3	2	Amendments By:	Stephen Hayward
		Director Sign Off:	Simon Maiden-Brooks
Issue	**Revision**	**Date:**	**21 December 2018**
4	2	Amendments By:	Stephen Hayward
		Director Sign Off:	Simon Maiden-Brooks

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Contents Page

1	**Background Scope of Appraisal**	**1**
2	**Existing Surface Water Runoff Characteristics**	**2**
	2.1 Site Background and Makeup	*2*
	2.2 Royal Military Canal Tide Lock Analysis	*3*
	2.3 Topography and Cross Sections	*3*
	2.4 Surface Water Flow Route Analysis	*4*
	2.5 Existing Surface Water Runoff from Undeveloped Land	*4*
	2.6 Surface Water Runoff from Existing Developed Land (Catchment B)	*6*
3	**Proposed Development - Surface Water Runoff**	**8**
	3.1 Surface Water Management	*8*
	3.2 Indicative Drainage Layout Plan	*16*
	3.3 Proposed Surface Water Runoff Discharge Rates	*16*
	3.4 Proposed Surface Water Runoff Discharge Volume	*17*
	3.5 Drain Down Times and Long-Term Performance	*19*
	3.6 Additional and Alternative Opportunities for Reducing Drain Down Times	*20*
	3.7 Maintenance and Management	*21*
	3.8 Residual Risk	*21*
	3.9 Water Quality	*22*
	3.10 Environmental Permit for Flood Risk Activities (FRAP)	*23*
4	**Conclusions and Recommendations**	**24**
5	**Appendices**	

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1 Background Scope of Appraisal

This report has been prepared to supplement the Flood Risk Assessment (FRA) submitted within Technical Annex 4, dated August 2017, in relation to planning application Y17/1042/SH. The objective of this Technical Addendum is to outline the details of an alternative option with regard to the discharge of surface water runoff from the development site and to demonstrate that discharge into the adjacent watercourse (Royal Military Canal - RMC) presents a viable alternative solution. It should be recognised that this addendum does not supersede the main findings of the original drainage assessment (contained within the original FRA), nor is it intended to bypass the drainage hierarchy. Instead, this report has been prepared in response to the comments received from the Environment Agency (reference KT/2017/123369/03-L01) and provides additional information to enable the outstanding objection to be removed.

The opportunities for discharging surface water runoff from the proposed development have been assessed within the FRA, in accordance with the hierarchy stated within the (Non-Statutory) Technical Standards for Sustainable Drainage (NTSS). The original assessment identified that the preferred option is to discharge surface water directly to the sea, and the text iterates that this option should be considered above *all* alternative options within the drainage hierarchy. The FRA also confirms that infiltration into the ground may not be feasible at this location and therefore, it is recognised that one of the alternative options would be to discharge surface water into the RMC. Notwithstanding this, it is acknowledged from the discussions held with the EA that the option to discharge to the RMC will be considered if it is demonstrated that a direct discharge into the sea is not viable.

For the purpose of this assessment, it is assumed that the alternative (more preferable) options mentioned above are not viable. Therefore, this report explores the technical details associated with discharging surface water from the proposed development at Princes Parade into the adjacent RMC, to ensure that a sustainable solution will be available.

2 Existing Surface Water Runoff Characteristics

2.1 Site Background and Makeup

The existing site comprises an area where refuse was historically buried (a former landfill site) and since the abandonment of this practice, the majority of the site has remained undeveloped. The exception is the highway fronting the Princes Parade site, which runs along the southern boundary of the site, and a public car park and play area to the east of the site. These developed areas comprise hardstanding consisting of concrete and asphalt.

Geological maps for this area indicate storm beach / sand dune deposits across most of the site, and typically these types of deposits are freely draining. Flood Estimation Handbook point data (FEH 13) has been obtained for the site, and from this data the BFIHOST and the PROPWET values have been extracted. These values are 0.889 and 0.34 respectively. This suggests a relatively dry, permeable catchment with limited surface water runoff from the undeveloped parts of the site.

Site specific geotechnical investigations have been undertaken, by others, to better understand the true ground condition of the site (post landfill). The results of the ground investigation report confirms that made ground deposits are present beneath most of the site and consequently, these overlying deposits will exhibit different soil characteristics to the natural values described above.

A number of trial pits were dug across the site (to depths up to 0.4m Below Ground Level) and the results indicate sandy gravelly **clay,** mixed with general refuse material. The soil descriptions are summarised in Figure 2.1 below.

Location	Depth (m)	Soil Description
HP1	0.00-0.33	Brown slightly silty sandy gravelly CLAY. Gravel is fine to medium subangular to subrounded flint and bituminous surfacing. Two glass bottles, glass and metal fragments.
HP2	0.00-0.20	Brown slightly sandy gravelly CLAY with some rootlets. Gravel is subangular to subrounded flint.
	0.20-0.33	Pale brown gravelly CLAY. Gravel is fine to medium subangular to subrounded flint and concrete.
HP3	0.00-0.33	Brown sandy gravelly CLAY with some rootlets. Gravel is angular to surrounded flint, brick and bituminous surfacing.
HP4	0.00-0.30	Greenish brown to brown sandy slightly gravelly CLAY. Gravel is fine to medium subrounded flint and rare brick. Rare inclusions of glass and pottery fragments.
	0.30-0.40	Greenish brown to brown slightly sandy slightly gravelly CLAY. Gravel is fine to medium coarse subrounded flint.
HP5	0.00-0.34	Greenish brown to brown sandy slightly gravelly CLAY with some rootlets. Gravel is subangular to subrounded flint, bituminous surfacing, concrete and brick.
HP6	0.00-0.20	Brown sandy CLAY with rootlets.
	0.20-0.30	Yellowish brown to light brown very sandy gravelly CLAY. Gravel is medium to coarse angular to subrounded flint, brick and concrete. Single brick cobble.
HP7	0.00-0.33	Brown sandy slightly gravelly CLAY with rootlets. Gravel is fine to medium subangular to subrounded flint and brick. Occasional inclusions of plastic, glass and metal fragments. A single boot.
HP8	0.00-0.33	Brown sandy slightly gravelly CLAY with rootlets and roots. Gravel is fine to medium angular to surrounded brick, flint and rare bituminous surfacing. and a shoe.
HP9	0.00-0.33	Brown sandy slightly gravelly CLAY. Gravel is fine to medium subrounded to rounded flint.
HP10	0.00-0.40	Yellowish brown clayey SAND with rootlets. Occasional inclusions of plastic.

Figure 2.1 – Extract of trial pit descriptions from Idom Merebrook Ltd site investigation report.

The above figure suggests that the surface topsoil is therefore significantly less permeable than the underlying made ground and superficial geology (present at depth). Given the relatively high groundwater levels, and less permeable topsoil located over the permeable made ground and sand deposits, it is considered reasonable to assume that actual BFIHOST values are likely to be lower (i.e. will vary between 0.32 and 0.75).

Due to the makeup of the landfill and made ground it is not possible to quantify the actual HOST value, nonetheless, to quantify the greenfield discharge rates and volumes from the existing site a conservative approach can be adopted. This approach assumes a high BFI value for the site (0.889), which is based on the BGS data, rather than the BFI value attributed to the results of the site-specific ground investigations. Although this is not technically accurate, it does ensure that the greenfield runoff rates and volumes which have been calculated for the pre-developed site, using this method, are under-estimated (i.e. the rates would otherwise be higher if the site-specific data was applied).

2.2 Royal Military Canal Tide Lock Analysis

Basic analysis has been undertaken to determine the tidal Mean High Water Spring Tide (MHWS) within the vicinity of the site. The calculations show that the adjusted MHWS level at Folkestone is 3.43m Above Ordinance Datum Newlyn (AODN). A T2 surge (i.e. a surge with a 1 in 2 year return period) gives a value of 4.18m AODN.

As-built drawings for the outlet weir of the RMC show a plate level of 2.1m AODN. Assuming this is the level at which the RMC becomes tide locked, basic analysis indicates that during the MHWS tide the canal is tide locked for a period of 3 hours 45 minutes. During a T2 surge this period of tide locking is increased to 4 hours 45 minutes.

To assess the impact that discharging surface water runoff from the development into the RMC could have, both the existing (pre-developed) site and proposed (post-developed) site have been considered. To ensure a suitable comparison, the outfall from the canal has been assumed to be tide locked for 4 hours 45 minutes (i.e. 285 minute duration event) for both pre and post developed scenarios. This represents the T2 surge event, which probabilistically, has the potential to coincide with a design pluvial event at this location (i.e. 1 in 100 year rainfall scenario). A summary showing the Tide Lock Analysis Curve has been included within Appendix A.1.

2.3 Topography and Cross Sections

A topography survey has been undertaken on the site, which identifies that the majority of the site is relatively flat longitudinally from east to west. A series of cross sections have been drawn (from north to south) throughout the site and these cross sections can be found within Appendix A.3. The sections indicate a clear raised platform running through the centre of the site, which is likely to be attributed to the made ground over the landfill. The sections also indicate that the northern part of the site falls towards the RMC, whilst the southern part of the site falls towards Princes Parade (the road). There is a lowered section in the middle of the site, opposite the footbridge crossing the RMC, which would also direct surface water runoff towards the RMC.

The embankment to the north of the site slopes steeply at ~1:4 towards the RMC. The embankment to the south of the site slopes steeply towards the road.

2.4 Surface Water Flow Route Analysis

Based on the available topographical and Light Detection and Ranging (LiDAR) data for the site, a ground model has been constructed within the Causeway Professional Design Suite (PDS). The PDS software interpolates the ground data to build a Digital Ground Model (DGM) grid mesh layer. Utilising the DGM, a rolling ball flow analysis can be conducted which enables potential flow routes to be quantified. This process represents 'rain' falling on a specific point on the site and delineates the direction that the water will take if the ground is assumed to be fully saturated.

Figure 2.2 below shows the general flow trace lines on the northern part of the site, providing further clarification as to the general direction that surface water runoff would flow.

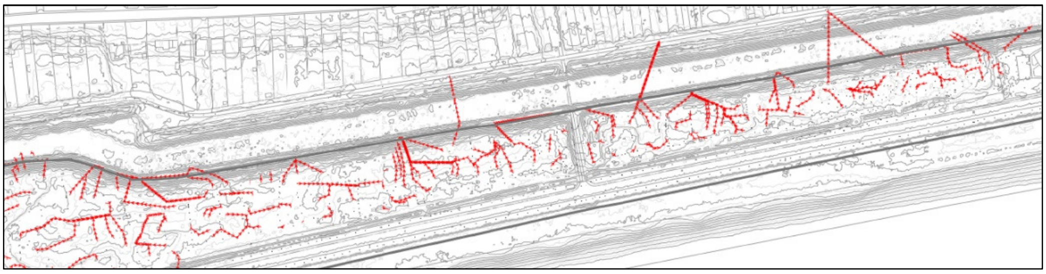

Figure 2.2 – Flow Trace lines across the site delineating flow direction.

With reference to Figure 2.2, it is evident that the flow direction is congruent with the general topography shown in the cross sections. This information suggests that when the ground is saturated, or where the ground is steeply sloped, surface water is likely to run off the surface of the site and will flow along the routes shown. In the northern part of the site, surface water currently flows northwards towards the RMC. In the southern parts of the site, water will flow towards the road.

2.5 Existing Surface Water Runoff from Undeveloped Land

From the information above, a catchment area has been derived to determine the amount of water that can currently enter the RMC. Figure 3 below shows the estimated catchment areas based on the flow route analysis and topography. This has been split into two discrete catchments (A & B respectively). Catchment A represents the undeveloped part of the site which has the potential to drain surface water into the RMC. Catchment B represents the developed area (i.e. car park and play area) which currently drains surface water into the RMC.

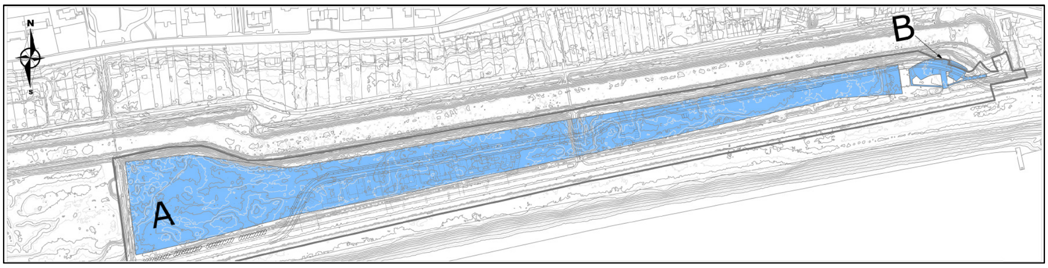

Figure 2.3 – Showing total catchment area falling towards the RMC. Undeveloped land forms part of Catchment A, developed land forms Catchment B.

Catchment Area A (comprising undeveloped land which would drain into the RMC if saturated) has been calculated as ~4.948 Hectares.

Catchment Area B (comprising developed land which drains directly into the RMC) has been calculated as 0.106 Hectares.

It is acknowledged that there are some areas within the existing site (e.g. along the canal banks), which are proposed to remain unchanged when the proposed site is developed (refer to Figure 2.4).

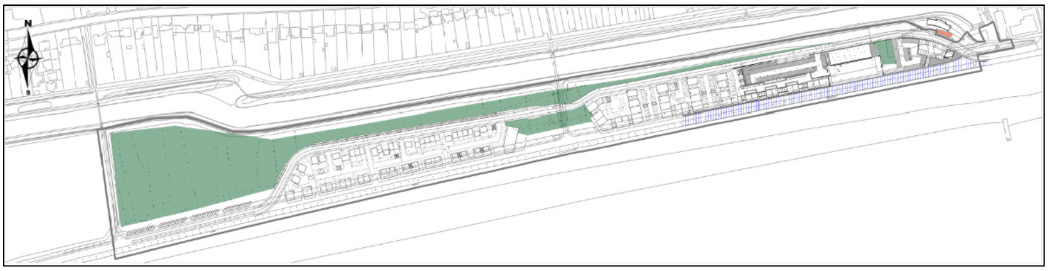

Figure 2.4 – Catchment area within the proposed development boundary which currently discharges into the RMC.

As the areas in Figure 2.4 will continue to discharge via the same mechanism post-development (i.e. directly into the RMC), these areas have been subtracted from Catchment Area A to determine the area which *currently* has the potential to discharge surface water runoff into the RMC, but is proposed to be developed. This catchment area measures ~2.636 Ha and is shown in Figure 2.5 below.

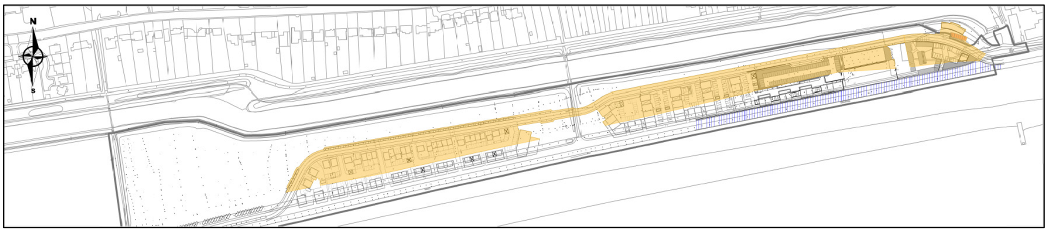

Figure 2.5 – Catchment area within the proposed development boundary which currently discharges into the RMC.

FEH13 point data has been applied to calculate both the greenfield runoff rates and volumes that are attributed to the catchment area (2.636 ha) delineated in Figure 2.5. This data includes the conservative BFIHOST value of 0.889, which does not consider the actual characteristics of the made ground (discussed in Section 2.1 of this report - which is likely to yield a lower BFIHOST value ranging between 0.32 and 0.75). To calculate the greenfield runoff rates and volumes, the SPR Host value (12.88) has been calculated using the correlation equation outlined within the IH126 report, applying both the conservative BFIHOST value (0.889) and the contributing catchment of 2.636ha. Greenfield runoff calculations have been included within Appendix A.4. of this report.

For comparison, the peak surface water runoff rates and total volumes for a pluvial event with a 285 minute duration and a 360 minute duration have been calculated for a range of return period events. These values and are shown in Table 2.1 below.

Return Period	Greenfield Surface Water Runoff Rate (BFIHOST 0.889)	Surface Water Runoff Volume (285minute)	Surface Water Runoff Volume (360minute)
1:2 year	~1.67 l/s	58.5 m^3	63 m^3
1:30 year	3.83 l/s	145 m^3	162 m^3
1:100 year	5.31 l/s	222 m^3	250 m^3

Table 2.1 – Greenfield runoff rates and volumes for the area currently draining to the RMC, that will form part of the proposed developed site.

The above figures represent the volume of surface water that can be discharged into the canal directly from the development, without having a detrimental impact. Similarly, the rate of discharge from this area has also been provided.

2.6 Surface Water Runoff from Existing Developed Land (Catchment B)

There is an existing public car park area located to the east of the site which consists of impermeable surfacing (concrete/asphalt). This existing car park currently drains informally towards the RMC, with surface water following the natural existing topography. The approximate impermeable area of the car park is 1,056m^2 and is shown in Figure 2.2 (labelled Catchment B). The runoff rates from this existing hardstanding area have been calculated using FEH13 data for a range of return period events (refer to Table 2.1). The CV value in the drainage model has been reduced to 0.6 to account for runoff draining across the neighbouring sloped embankment, thus providing a conservative estimate. The corresponding greenfield runoff volumes have also been provided in Table 2.2 for both the 240 minute and 360 minute duration events.

It should be recognised that a 240 minute duration has been applied instead of a 285 minute duration, as peak runoff rates from the 285 minute duration cannot be calculated within industry standard software. Nevertheless, this will result in a slight *under estimate* when the existing pre-developed runoff volume is calculated.

Return Period	Surface Water Runoff Rate (FEH-13)	Surface Water Runoff Volume (240minute)	Surface Water Runoff Volume (360minute)
1:2 year	11.6 l/s	14.5 m^3	16.5 m^3
1:30 year	31.5 l/s	31.4 m^3	35.2 m^3
1:100 year	40.4 l/s	40.4 m^3	45.7 m^3

Table 2.2 – Catchment Area B runoff rates and volumes.

3 Proposed Development - Surface Water Runoff

3.1 Surface Water Management

The drainage strategy which discusses each of the different elements of the proposed scheme is set out below. This does not represent a detailed surface water drainage design; it is simply an assessment to demonstrate that the objectives and requirements of the NPPF and NTSS can be met at the planning stage, for the alternative option of discharging surface water runoff to the RMC.

Based on the current masterplan and the topography of the site, the development site has been sub-divided into five separate drainage catchments. Figure 3.1 (below) shows the location of each drainage catchment across the site.

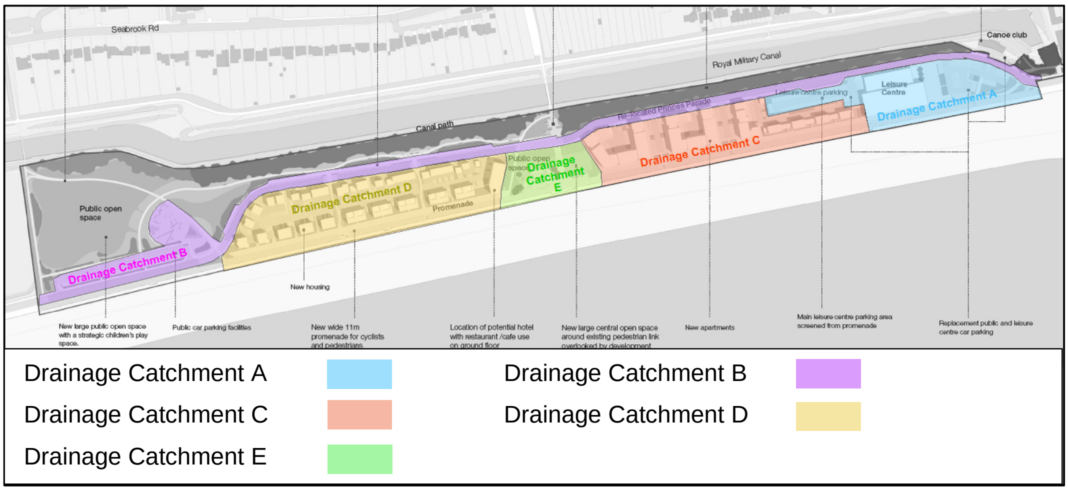

Figure 3.1 – Plan showing the drainage catchments across the site.

It is envisaged that runoff draining from the roads, buildings, and hardstanding within each of these drainage catchments will be managed within Sustainable Drainage Systems (SuDS) before being discharged at a restricted rate to the RMC.

Three separate outfalls into the canal are proposed;

- Drainage Catchments A and B will drain into the RMC via 2 separate outfall structures,
- Drainage Catchments C and D will drain into catchment E before being discharging via a single outfall into the canal.

A summary of the impermeable areas draining to SuDS within each drainage catchment is provided in Table 3.1 (below).

Drainage Catchment	Areas Draining to Drainage Catchment	Drains to	Total Impermeable Area Draining to Catchment (including a 10% allowance for urban creep)
Drainage Catchment A	Leisure centre and carpark	Royal Military Canal	0.75 ha
Drainage Catchment B	Public highway, play area, and public parking	Royal Military Canal	1.48 ha
Drainage Catchment C	Private development east	Drainage catchment E	0.92 ha
Drainage Catchment D	Private development west	Drainage catchment E	0.99 ha
Drainage Catchment E	Promenade, and public hardstanding + inflows from catchments C & D	Royal Military Canal	0.26 ha
Total			**4.4 ha**

Table 3.1 – Summary of drainage Catchments A-E.

A summary of the proposed SuDS to be used within each drainage catchment, along with calculations to confirm that the drainage system can manage the design rainfall event is provided below.

Drainage Catchment A

Runoff from the roof of the leisure centre will be discharged into permeable paving located across the carpark. The permeable paving will be laid on top of a 1m deep layer of open graded sub-base, which is proposed to be lined to ensure no interaction between any leachates and surface water. A summary of the Micro Drainage analysis for the permeable paving is shown in Table 3.2 below.

Drainage Catchment A (Leisure Centre and carpark)	**Value (1:100yr+20%cc event)**	
SuDS	Permeable Paving	
Storage Provided within	Permeable Paving	
Area draining to permeable paving (including 10% allowance for urban creep)	7,520 m^2	
Area of permeable paving	~ 4,490 m^2	
Sub-base depth	1000 mm	
Infiltration	Not Permitted Lined System (due to contamination)	
Flow control device	Orifice plate (30mm diameter)	
Discharges too	Royal Military Canal	
Maximum depth of water above base of the drainage system.	711 mm	
Overflow control device	Pipe set at 800mm above the base of the permeable paving, (overflows to beach)	
Critical storm duration	2880 minutes	
Return Period	**Half drain time**	**Peak discharge rate**
1 in 2yr+cc	3964 minutes	1.0 l/s
1 in 30yr+cc	4992 minutes	1.3 l/s
1 in 100yr+cc	6003 minutes	1.6 l/s

Table 3.2 – Summary of Micro Drainage analysis for Drainage Catchment A.

Drainage Catchment B

Runoff from the new trunk road, public parking areas, and play area will be drained into a swale. This swale will convey surface water runoff across the site to and discharge into a large pond. The pond will be designed to manage runoff from the design rainfall event before it is discharged via a vortex flow control device (Hydro-Brake or similar) into the RMC. To maximise the available amenity space, the pond and surrounding area will be terraced to keep low volumes of runoff from small storms within the permanently wet portion of the pond. It is envisaged that the final pond design will accommodate areas where water depths will be greater, however these areas would not be expected to be wet on a regular basis. A suitable pond profile and planting scheme will need to be specified such that "bogs" are not created (refer to Figure 3.2 below).

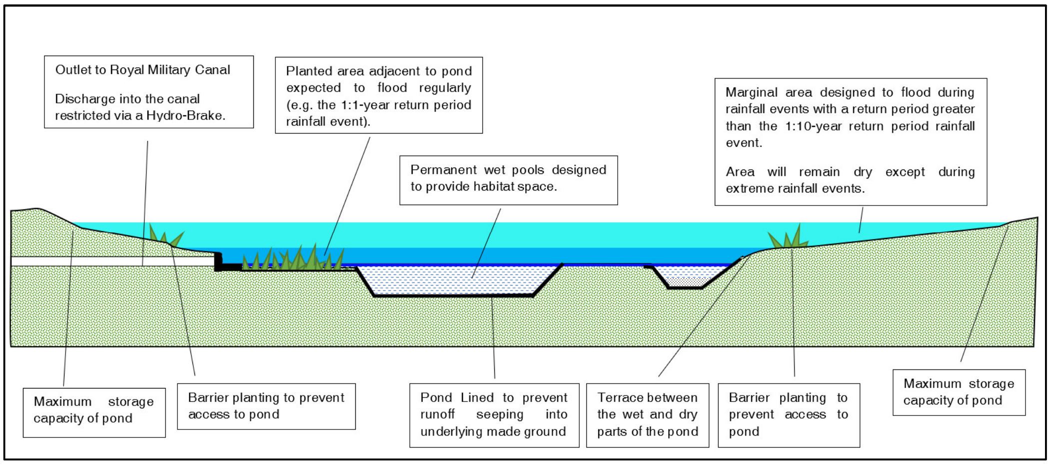

Figure 3.2 – Indicative sketch showing the profile of pond (outline design only).

A summary of the Micro Drainage analysis for Drainage Catchment B is shown in Table 3.3 below.

Drainage Catchment B (public highway, parking and play area)	**Value (1:100yr+20%cc event)**	
SuDS	Swale and Pond	
Storage Provided within	Pond	
Area draining to permeable paving (including 10% allowance for urban creep)	14,840 m^2	
Area of pond	~ 5,000 m^2	
Active storage depth within pond	~ 1 m	
Infiltration	Not Permitted, Lined System (due to contamination)	
Flow control device	Hydro-Brake	
Limiting Discharge Rate	2.0l/s	
Overflow control device	200mm wide weir with crest level at 960mm above normal water level within pond. Overflows to the Royal Military Canal	
Critical storm duration	5,760 minutes	
Return Period	**Depth of water**	**Peak discharge rate**
1 in 2yr+cc	486 mm	1.7 l/s
1 in 30yr+cc	696 mm	1.7 l/s
1 in 100yr+cc	862 mm	1.9 l/s

Table 3.3 – Summary of Micro Drainage analysis for Drainage Catchment B.

Drainage Catchment C

Runoff from the roofs of the dwellings can be drained into a lined permeable paving system, located across the driveways and parking areas, before discharging into a lined underground storage tank. The storage tank will discharge surface water runoff at a restricted rate into a second storage tank located in Drainage Catchment E (discussed below). It is envisaged that the primary storage tank can be located underneath the existing road, thus limiting the removal of any contaminated land and providing easy access for future maintenance.

A summary of the Micro Drainage analysis for Drainage Catchment C is shown in Table 3.4 below.

Drainage Catchment C (East part of the private housing development)	**Value (1:100yr+20%cc event)**	
SuDS	Permeable Paving and Underground Storage	
Storage Provided within	Underground Storage	
Area draining to permeable paving (including 10% allowance for urban creep)	9,160 m^2	
Assumed dimensions of underground storage	~ 1,000 m^2 x 1.5m (deep)	
Infiltration	Not Permitted, Lined System (due to contamination)	
Flow control device	Hydro-Brake	
Limiting Discharge Rate	2.0l/s	
Discharges into	Drainage Catchment E	
Overflow control	Pipe located ~1.3m above the base of the storage tank (discharges to beach)	
Critical storm duration	2,880 minutes	
Return Period	**Half Drain Time**	**Peak discharge rate**
1 in 2yr+cc	3736 Minutes	1.3 l/s
1 in 30yr+cc	4320 Minutes	1.5 l/s
1 in 100yr+cc	6397 Minutes	1.8 l/s

Table 3.4 – Summary of Micro Drainage analysis for Drainage Catchment C.

Drainage Catchment D

Runoff from the roofs of the dwellings can be drained into a lined permeable paving system located across the driveways and parking areas. This permeable paving will drain into a lined underground storage tank, designed to store surface water runoff before discharging at a restricted rate into a second storage tank located in Drainage Catchment E (discussed below). It is envisaged that the primary storage tank can be located underneath the existing road, thus limiting the removal of any contaminated land and providing easy access for future maintenance.

A summary of the Micro Drainage analysis for Drainage Catchment D is shown in Table 3.5 below.

Drainage Catchment D (West part of the private housing development)	**Value (1:100yr+20%cc event)**
SuDS	Permeable Paving and Underground Storage
Storage Provided within	Underground Storage
Area draining to permeable paving (including 10% allowance for urban creep)	9860 m^2
Assumed dimensions of underground storage	~ 1000 m^2 x 1.5m (deep)
Infiltration	Not Permitted, Lined System (due to contamination)
Flow control device	Hydro-Brake
Limiting Discharge Rate	2.0l/s
Discharges into	Drainage Catchment E
Overflow control	Pipe located ~1.4m above the base of the storage tank (discharges to beach)
Critical storm duration	2880 minutes

Return Period	**Half Drain Time**	**Peak discharge rate**
1 in 2yr+cc	4050 Minutes	1.3 l/s
1 in 30yr+cc	5467 Minutes	1.6 l/s
1 in 100yr+cc	6675 Minutes	1.9 l/s

Table 3.5 – Summary of Micro Drainage analysis for Drainage Catchment D.

Drainage Catchment E

Runoff from the communal hardstanding areas will be drained directly to a lined underground storage tank located beneath the central area of public open space. Runoff exiting the storage tanks within Drainage Catchment C and Drainage Catchment D will also be drained into this storage tank at an attenuated rate. To provide additional storage, the area above the storage tank will be landscaped to form an above ground detention basin, which can be designed to flood during *extreme* rainfall events. An overflow can be installed to allow excess water to be directed towards the beach if the detention basin reaches maximum capacity. A summary of the Micro Drainage analysis for Drainage Catchment E is shown in Table 3.6 below.

Drainage Catchment C (East part of the private housing development)	**Value (1:100yr+20%cc event)**
SuDS	Underground Storage and overlying detention basin.
Storage Provided within	Underground Storage
Area draining to permeable paving (including 10% allowance for urban creep)	2,590 m^2
Assumed dimensions of underground storage	~ 800 m^2 x 0.5m (deep)
Assumed dimensions of overlying detention basin	~ 1,000 m^2 x 0.5m (deep)
Infiltration	Not Permitted (due to contamination)
Flow control device (invert level 500mm below base of storage tank)	Orifice plate (41mm diameter)
Discharges into	Royal Military Canal
Overflow control device	Minimum 300mm diameter, pipe just below the top of the detention basin, (discharges to the beach).
Critical storm duration	4,320 minutes
Return Period	**Peak discharge rate**
1 in 2yr+cc	2.7 l/s
1 in 30yr+cc	3.2 l/s
1 in 100yr+cc	4.3 l/s

Table 3.6 – Summary of Micro Drainage analysis for Drainage Catchment D.

General Considerations

Runoff from the new promenade, which will replace the existing road, will be drained directly to the beach. This will prevent saltwater from entering the drainage system and thus reaching the RMC. The current road discharges unattenuated to the beach through a series of road drains (refer to Figure 3.3) and currently there are no pollution control measures, resulting in hydrocarbons being washed onto the beach. There will be no public vehicular access along the seafront and as such, any surface water discharged to the beach from the new area of promenade will be uncontaminated.

Figure 3.3 – Existing road drainage along Princes Parade.

With respect to the proposed surface water storage system (e.g. permeable paving, cellular storage etc.), it should be recognised that these systems provide attenuation for peak flow events and do not generally hold standing water. Consequently, these systems are not cleaned with chemicals and as such, present no risk to receiving watercourse (i.e. the RMC).

3.2 Indicative Drainage Layout Plan

An indicative drainage layout plan delineating how the proposed SuDS can be incorporated into the scheme proposals is located in Appendix A.6 of this report.

3.3 Proposed Surface Water Runoff Discharge Rates

It is proposed to provide 3 separate outfalls into the RMC. The first is at the proposed leisure centre, located to the east of the site, where water will be discharged directly from the permeable paving system. The second outfall will be located in the centre of the development, connecting the central storage tank to the RMC. The final outfall will be located to the west of the site, connecting the green space to the RMC. All of the proposed outfalls are likely to comprise a single pipe connected to a flow control device, ensuring the rate of discharge is both restricted and controlled.

The proposed discharge rates which are based on the above drainage strategy are summarised in Table 3.7 below:

Return Period	Leisure Centre	Residential	Highway	Combined Total
1:2 year + 20%	0.9 l/s	2.7 l/s	1.7 l/s	5.3 l/s
1:30 year +20%	1.3 l/s	3.2 l/s	1.7 l/s	6.2 l/s
1:100 year +20%	1.6 l/s	4.3 l/s	1.9 l/s	7.8 l/s

Table 3.7 – Runoff rates from the proposed development for each outfall.

The proposed surface water discharge rates have been compared to the existing surface water run off rates derived in Section 2 of this report (i.e. Table 2.0 and 2.1), for a range of return period events. The rates have been summarised in Table 3.8 below.

Return Period	Pre-development runoff rate	Proposed Development (with 20% cc) runoff rate	Betterment
1:2 year	13.0 l/s	5.3 l/s	60%
1:30 year	35.3 l/s	6.2 l/s	82%
1:100 year	45.7 l/s	7.8 l/s	83%

Table 3.8 – Comparison between pre and post development runoff rates.

The figures in Table 3.8 above demonstrate that with SuDS included within the proposed development, it is possible to restrict the discharge of surface water runoff into the RMC. In particular, under the design event (1 in 100 year return period event, including a 20% allowance for 100 years of climate change), it is evident that there is a significant reduction in the peak rate at which surface water runoff is discharged offsite when compared to the existing situation.

3.4 Proposed Surface Water Runoff Discharge Volume

An assessment has been made to determine the volume of water entering the RMC from the developed site, for two rainfall events with different durations (i.e. the 360 minute 'design event' and a 285 minute event). The latter event takes into consideration the entire period in which the RMC could be tide locked. A conservative estimate has been made when calculating the post development volume of runoff discharged to the RMC, by assuming that the flow restriction devices are discharging at the maximum rate for the entire duration of the storm event.

Table 3.9 below summarises the total volume of surface water discharged from the proposed development site during both scenarios.

Return Period	Post-development Total Volume (285min event)	Post-development Total Volume (360min event)
1:2 year + 20% cc	91m³	114 m³
1:30 year + 20% cc	106 m³	134 m³
1:100 year + 20%cc	133 m³	168 m³

Table 3.9 – Post development discharge volumes generated for rainfall events with a 285 minute and 360 minute duration, for a range of return period events. All calculations include a 20% increase in peak rainfall intensity to account for climate change.

A summary table comparing the total volume of runoff discharged into the RMC for both the pre and post developed scenarios, for a range of return period events, is provided in Table 3.10 (below).

Return Period	285min (total volume discharged to the canal)			360min (total volume discharged to the canal)		
	Existing (greenfield)	Proposed	Difference	Existing (greenfield)	Proposed	Difference
1:2 year + 20% cc	73 m³	91m³	18 m³ (increase)	80 m³	114 m³	<35 m³ (increase)
1:30 year + 20% cc	176 m³	106 m³	70 m³ (decrease)	197 m³	134 m³	63 m³ (decrease)
1:100 year + 20%cc	262 m³	133 m³	129 m³ (decrease)	296 m³	168 m³	128 m³ (decrease)

Table 3.10 – Showing approximate discharge volumes for a range of return periods (both pre and post development.

From Table 3.10 above it is evident that under the design flood event (i.e. the 1 in 100 year event including an allowance for 100 years of climate change), less water will be discharged into the RMC during the time over which the RMC is assumed to be tide locked (refer to Section 2.2). Furthermore, during the 360 minute design event, the volume of water discharged offsite has been reduced when compared to the pre-developed situation.

The exception to this is under the 1 in 2 year event, where the estimated volume discharged from the proposed development to the RMC is shown to increase by less than 35m³. However considering the number of conservative assumptions that have been made to derive this figure, it is recognised that this number is likely to be significantly lower, and in reality is unlikely to result in an increase.

Notwithstanding this, even if this small volume increase was assumed to be the 'worst case scenario', when compared to the overall size of the canal from West Hythe to Seabrook, it is evident that this volume of water would equate to an increase in the water level of less than 1cm during the design event, or tide lock scenario. Given the existing freeboard within the RMC is greater than 1cm, this small increase in water level is very unlikely to increase the risk of flooding offsite. Furthermore, given the proposed development would significantly reduce the volume of water discharged to the RMC during higher return period events, the benefits of the development significantly outweigh any marginal increase in water level (i.e. less than 1cm) under low return period events.

On balance, taking the above into consideration, it is evident that the drainage proposals will not increase the risk of flooding within the RMC during the design rainfall event, even when the RMC is tide locked during the design storm.

3.5 Drain Down Times and Long-Term Performance

Under the design rainfall event, the calculations show that the proposed drainage system would not have an adverse impact on the development site, or the RMC. Nonetheless, to achieve this, a low limiting discharge rate has been used for each of the flow control devices specified within the drainage network. These very low discharge rates result in slow drain down times for each of the SuDS, which as a result could make the drainage system susceptible to flooding during either; long duration (low intensity) rainfall events, or from multiple back to back storms. The performance of the proposed drainage system during both of these scenarios has therefore been assessed to ensure that there will be no increased risk of flooding to the RMC, or to the proposed development.

The critical storm duration defines the event where the drainage system is most susceptible to flooding. Therefore, by ensuring that the system is designed to function under the critical storm duration, it will by default ensure that the system will not flood during a storm event which has either a longer, or shorter duration.

Table 3.11 below shows each of the proposed SuDS, alongside the critical storm duration for the design event (i.e. an event with a 1 in 100 year return period).

Drainage Catchment	SuDS	Critical Storm Duration (minutes)
A	Permeable Paving	2880
B	Pond	5760
C	Underground Storage	2880
D	Underground Storage	2880
E	Underground Storage and Detention basin	4320

Table 3.11 – Critical storm durations for each of the proposed SuDS during the design rainfall event.

In addition to assessing the response of the drainage system during the critical storm event, a further sensitivity test has been undertaken to determine the impact of back to back rainfall events, e.g. the design rainfall event (1:100 year return period rainfall event, including climate change), followed by a 1:10 year return period rainfall event. The Susdrain and CIRIA factsheet *"Assessing attenuation storage volumes for SuDS",* questions the appropriateness of meeting 24 hour half drain times when long term storage for stormwater is provided. This factsheet suggests that a more appropriate solution could be to ensure that 24 hours after the design rainfall event, there is room within the drainage system to accommodate a subsequent 1:10 year event. This combination has therefore been selected to represent a realistic back to back rainfall scenario, albeit the probability of such extreme events coinciding is considered to be very low.

A summary of this analysis is provided in Table 3.12 (below).

SuDS	Max water level without overflow control (mm)	Max water level with overflow control (mm)	Volume overflowing drainage system (m^3)	Overflow discharge location
Permeable Paving (leisure centre)	984	960	155	Beach
Pond (highway & hardstanding)	954	954 (water level does not reach crest level of overflow weir)	0	RMC
Underground Storage (East)	1400	1394	44	Beach
Underground Storage (West)	1453	1452	12	Beach
Detention basin	N/A basin overflows	1414	1037	Beach

Table 3.12 – Summary of SuDS performance during the critical 1:100+cc event, combined with a 1:10 year rainfall event (60 minute duration).

From the above calculations it is evident that by oversizing the proposed SuDS, and by incorporating independent overflows within each of the proposed SuDS, there is a solution available which will ensures that no additional volume of water will be discharged to the RMC. Once the overflow is activated, any additional surface water can, in most cases, be directed onto the beach, or stored safely on site.

3.6 Additional and Alternative Opportunities for Reducing Drain Down Times

An alternative option to increasing the volume of storage provided onsite (to accommodate the additional runoff generated during back to back rainfall events), would be to increase the discharge rates to the RMC. However, it is recognised that the rate of discharge to the RMC should only be increased when the water levels within the RMC are low and flooding is unlikely to occur. This approach allows any additional water to be discharged to the sea without having any detrimental impact to the RMC and the surrounding area.

The increase discharge rates could be achieved by incorporating mechanical float valves, or actuated valves into the flow control system for the development. These devices would change the rate at which runoff can be discharged offsite based on the water level within the RMC, only allowing more water to be discharged offsite when this approach is unlikely to present a risk of flooding.

A similar, but alternative option to a mechanical system, is to use a telemetry based system. This type of system enables the flow rates into the RMC to be controlled remotely, based on weather forecasting (i.e. closing the outlets when a storm event is predicted, thus reducing the risk of flooding to the RMC).

If the development site is prevented from discharging surface water to the RMC, the SuDS would begin to store water onsite. A sensitivity test has been undertaken to determine the maximum length of time the site could be prevented from discharging to the RMC, before the SuDS reach maximum capacity. The calculations show that there is sufficient room available within the development to store water for 12 hours, before any overflow is activated. This water can be stored safely onsite, without increasing the risk of flooding to the new properties or surrounding area. (Refer to Section 3.8 for more details). Consequently, it is evident that there is more than sufficient time for the water level within the RMC to drop, before the site is required to discharge to the RMC.

3.7 Maintenance and Management

For any surface water drainage system to operate as originally designed, it is necessary to ensure that it is adequately maintained throughout its lifetime. This can be achieved by undertaking routine inspection and maintenance of the drainage system, including all outlet and overflow controls. Any manufacturer specific requirements should also be taken into consideration.

In this case, a regulated specialist management and maintenance company will be appointed to ensure maintenance is carried out regularly and to the appropriate standards, to ensure that there is no risk of flooding to the development site or to offsite properties.

3.8 Residual Risk

When considering residual risk it is necessary to consider the impact of a flood event that exceeds the design event, or the implications if the proposed drainage system becomes blocked/fails.

The underground storage systems, detention basin and permeable paving all include an overflow system, designed to discharge excess runoff through the seawall and onto the beach, thus minimising the risk of additional water entering the RMC.

For the pond system an overflow into the beach is not considered viable, due to the lower land levels to the north west of the site where the pond is located. Consequently, an overflow weir into the RMC has been specified to control the rate at which additional runoff is discharged. However, it should be recognised that significant additional storage has been included within the public open space to store a back to back rainfall event and as such, during an exceedance or blockage scenario this additional storage will become active. This will provide a visual warning in the event that the outlet has become blocked and will enable the problem to be rectified, without increasing the risk to the RMC.

Figure 3.4 (below) delineates the proposed overflow control systems and shows where additional runoff can be discharged offsite in the event of a blockage, or during a storm event which exceeds the design parameters for the proposed drainage system.

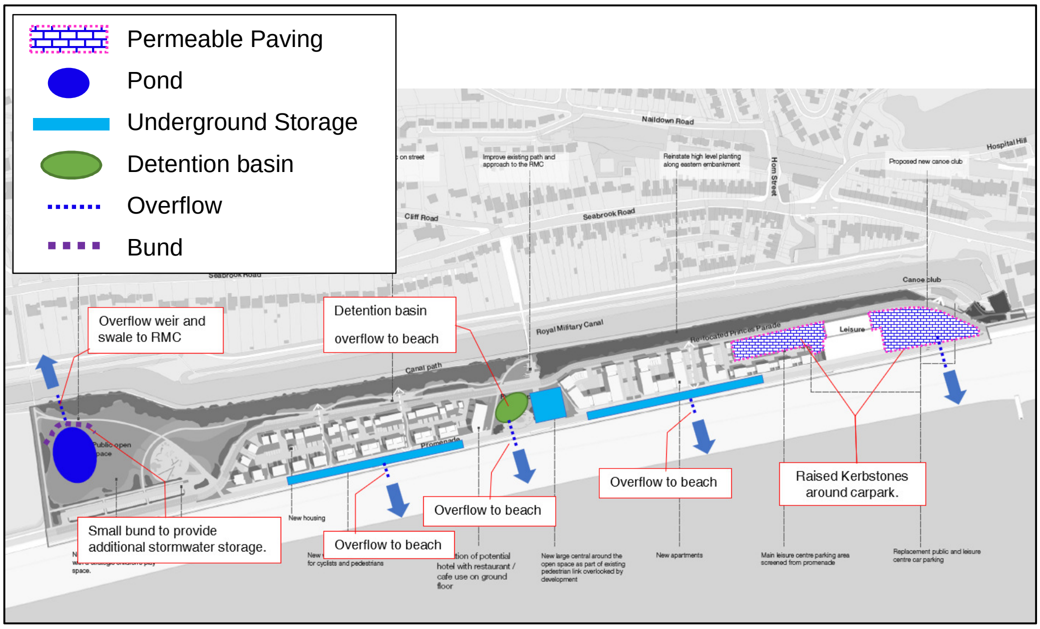

Figure 3.4 – Plan showing proposed SuDS and overflow control systems.

In addition to the overflow control systems, additional measures can also be used to further reduce the volume of water discharged offsite, either to the beach or to the RMC.

One option could be to use raised kerbstones and reprofile the land levels around the permeable car parking areas, allowing water to pond to a shallow depth above the surface of the carpark. Calculations suggest that 100mm of flooding across the entire carpark area will provide storage for an additional 450m^3 of water. Similarly, a small bund could also be located to the north of the pond, where land levels are lower. By incorporating this bund within the landscaping of the public open space it will be possible to store a large amount of additional stormwater, prevented it from entering the RMC.

3.9 Water Quality

Given the significant importance of the RMC with respect to ecology and biodiversity, it is evident that the risk of the development polluting the RMC needs to be considered. The pollution hazard indices for each part of the proposed development has been calculated using CIRIA C753 and the results of this analysis are summarised in Table 3.11 below. These values have been compared with the mitigation index for proposed SuDS at this site.

Parameter		Total suspended solids (TSS)	Metals	Hydro-Carbons
Leisure centre and carpark (permeable paving)	Pollution Hazard Index (4)	0.7	0.6	0.7
	Mitigation Index from permeable paving	0.7	0.6	0.7
Access road play area and public parking (Swale and Pond)	Pollution Hazard Index (5)	5	0.8	0.8
	Mitigation Index from swale and pond	0.85	0.95	0.85
Private dwellings parking and access (permeable paving and underground storage)	Pollution Hazard Index (3)	0.5	0.4	0.4
	Mitigation Index for permeable paving (underground storage provides no treatment)	0.7	0.6	0.7

Table 3.11 – CIRIA C753 simple index approach to water quality management.

With reference to Table 3.11 (above), the simple index approach to water quality management has been applied for each of the drainage catchments. Providing the SuDS outlined in the proposed strategy are adopted and designed in accordance with best practice (as outlined within CIRIA C753), it is evident that the pollution hazard index is considered acceptable.

The drainage system should be designed to capture the first 5mm rainfall event, which will ensure that any pollutants (such as surface hydrocarbons from the road, for example) are not discharged into the RMC. In addition, sediment traps and pollution control features (such as oil interceptors) can be specified as part of the detailed drainage design to ensure any unexpected pollution can be contained on site and prevented from reaching the RMC.

3.10 Environmental Permit for Flood Risk Activities (FRAP)

The RMC is designated as a Main River and as such, any discharge to this watercourse will require an environmental permit and agreement from the Environment Agency. Typically, a permit is required for the following reasons if work is to be carried out:

- in, under, over or near a main river (including where the river is in a culvert),
- on or near a flood defence on a main river,
- in the flood plain of a main river,
- on or near a sea defence.

It is recommended that the EA are consulted regarding the requirements for permitting at the detailed design stage.

4 Conclusions and Recommendations

The overarching objective of this report is to appraise the suitability of discharging the proposed development into the Royal Military Canal (RMC). This report provides additional details to supplement the Flood Risk Assessment and drainage strategy, submitted within Technical Annex 4 dated August 2017.

The original assessment acknowledges that the preferred method of discharging surface water runoff from the development is via a connection to the sea (in accordance with S1 of the NTSS). Although these conclusions are still valid, this assessment has been prepared on the assumption that a connection to the sea will not be viable and therefore, presents an alternative solution for draining the site in a sustainable way.

The runoff rates and the volume of surface water discharged from the existing site have been calculated, taking account of the sub-catchments within the development site. The results from a rolling ball analysis, site investigations, and hydrological data have all be used in this process to provide a baseline against which to compare the post development impacts.

The findings from the analysis show that by restricting the peak rate at which surface water is discharged from the development site, the risk of flooding will not increase. Whilst it is acknowledged that the total volume of runoff discharged from the development will be increased when compared to the exiting pre-developed conditions, under normal conditions there will be no detrimental impact to the RMC, or to the surrounding area. This is due to the flow restrictions placed on the three outfalls from the proposed development, which will limit the rate and safely control the volume of surface water runoff discharged from the development site.

The primary risk of flooding to the surrounding area is during the period when the RMC is at full capacity, during which period, discharging any additional volume of water to the RMC has the potential to exacerbate the risk of flooding. In response, several sensitivity tests have been undertaken to appraise the impact that the development could have when the RMC is tide locked. The results show that under a tide locked scenario, the development will discharge a lower volume of water into the RMC for the period at which the system is sealed and consequently, the development will have no detrimental impact under this scenario.

Notwithstanding this, it is recognised that the precautionary approach adopted to restrict the offsite discharge volumes has resulted in high half drain times and therefore, in the event that back to back storm events were to occur it is important to ensure that the risk of the drainage system flooding is not increased. Consequently, additional testing has been undertaken to confirm whether the redundant storage within the drainage system is sufficient to protect both the development and the RMC from flooding. The analysis shows that during a back to back storm event, parts of the site are susceptible to flooding, however, the depth of flooding can be managed appropriately.

The analysis also shows that during such an event, the drainage system can hold all of the surface water generated for up to 12 hours on site, without the requirement to discharge to the RMC. By including a series of overflows into the drainage system, additional flows can be directed onto the beach, further reducing the pressure on the RMC.

Alternative options including the use of telemetry, actuated values and float values have also been considered and may present a complementary solution to the final design.

In addition to the discharge rates and volumes, pollution control measures have been considered as part of the drainage strategy. It is evident that by incorporating a variety of SuDS within the development, the risk of pollutants entering the RMC can be minimised.

In conclusion, this report demonstrates that there is a drainage solution that will enable surface water runoff from the development at Princes Parade to be discharged into the RMC, as an alternative and sustainable solution for draining the site, and one which will not increase the risk of flooding.

5 Appendices

Appendix A.1 – Royal Military Canal – Tide Lock Analysis

Appendix A.2 – Catchment Area Drawing

Appendix A.3 – Topographical Sections

Appendix A.4 – Greenfield Runoff Calculations

Appendix A.5 – Surface Water Runoff Calculations & Drawings

Appendix A.6 – Indicative Drainage Layout