Proposed Residential Development

Park, Wexford

Applicant

Wm. Neville & Sons Ltd

Planning Application Engineering Report

Arthur Murphy & Co

Consulting Engineers

August 2020

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August 8th 2020.

Proposed Housing Development at Park – Wm Neville & Sons Ltd

Engineering Report

Location: Carcur Park, Wexford, Co.Wexford

Proposal: Proposed Development: 413 no. residential units (182 no. houses, 205 no. apartments), childcare facility and associated site works. Carcur Park, Wexford.

Introduction

This report deals with the engineering aspects of the proposed development including:

1. Following the tripartite meeting which took place on the 17th of June 2020, ABP issued Notice of Pre-Application Consultation Opinion which required responses to seven points. This document addresses Points 1, 2, 4, 5 and 6 of ABP's Notice. These points are dealt with under the relevant headings further down in this list.
    a. Point Nos. 1 and 2 are responded to in "Section 2 – The Storm Water System" below.
    b. Point No. 4 is responded to in "Section 4 - Road Design Confirmations" below.
    c. Point No 5 is responded to in "Section 5 - The importation of Fill to the Site" below.
    d. Point No 6 is responded to in "Section 6 - Carcur Landfill Gas Monitoring and Mitigation Measures" below.
    e. Engineering issues in Point No 7 are responded to in Section 7 - Response to the PA Opinion submitted 08 May 2020.
2. The Storm Water System.
3. Flood Risk Assessment
4. Road Design Confirmations
5. The Importation of Fill to the Site.
6. Carcur Landfill Gas Monitoring and Mitigation Measures.
7. Response to PA Opinion of 08 May 2020
    a. For Flood Risk Assessment see "Section 2 - The Storm Water System"
    b. Surface Water and SuDS issues raised see "Section 2 - The Storm Water System" and "Section 3 - Flood Risk Assessment"
8. Wastewater.
9. Water Supply.

This report contains the following Appendices:

A. Storm Water Report
B. Aquafact Ltd Aquaculture Impact Report
C. Construction Management Plan on The Importation of Fill and Related Ecological Protection Measures.
D. Report on Management, Future Monitoring and Mitigation of Gas Emissions from Carcur Landfill Site
E. Irish Water Approval and Documents Submitted for that Approval.

The following are the civil engineering drawings for the project:

PL 01 Site Overview
PL 02 Site Services Plans (1 of 2)
PL 03 Site Services Plans (2 of 2)
PL 04 Road Long Sections
PL 05 Foul Sewer Sections and Notes
PL 06 Storm Water Sections
PL 07 Water and Wastewater Details
PL 08 Sewage Pump Station
PL 09 Storm Water System Design
PL 10 Site Cut and Fill
PL 11 Shoreline Sections
PL 12 Construction Management Overview.

Section 2 Storm Water System.

The ABP Pre-Application Consultation Opinion states *(inter alia)* :

1. *A robust Water Environment Risk Assessment, Ground Water Management Plan, AA screening report and NIS which support and have regard to one another, and which inter alia, consider the possibility of contamination reaching the Estuary (An EU designated SPA and SAC with Qualifying Interests incl. shellfish / freshwater pearl mussel and consequent conservation objectives) from the proposed development site, through the medium of ground water.*
2. *A report on surface water drainage, surface water management strategy and flood risk which deals specifically with quality of surface water discharge to the Estuary and possible need for a Discharge Licence and or a Foreshore Licence.*

With respect to Point 1 above it is not intended to discharge to groundwater in the subject application and the documents in the Pre-Application phase showed discharge to the estuary as advised by the County Council in February 2020 and confirmed in their letter to ABP on 08 May 2020.

Storm water discharge direct to the estuary is required by Wexford County Council with attenuation of the 100 year storm. Their own development at Trinity Wharf at the east end of the town, which has recently received planning permission, employs direct discharge to the estuary with some under pavement attenuation. Their discharge points are close to active aquaculture areas of the estuary while the subject development is not.

With respect to Point 2 above a Storm System report outlining the storm water system layout and design is set out in Appendix A of this document. Since no discharge to groundwater is proposed of any kind no discharge licence is required. A foreshore licence is required and this is discussed in the report in Appendix A.

For reasons outlined in Appendix A, a robust report by Aquafact Ltd examining potential impacts of storm water on aquaculture in the estuary is included here as Appendix B. Their report concludes as follows:

To conclude, this report confirms that due to:

1. *The treatment and attenuation of the storm water,*
2. *The huge rates of dilution and*
3. *The fact that the freshwater will float on top of the heavier estuary saltwater*

the impact of the storm water discharge from the development on the estuary waters will be virtually unmeasurable and will not negatively affect aquaculture in the estuary.

Section 3 Flood Risk Assessment.

A Site Specific Flood Risk Assessment has been undertaken for the site by IE Consulting. This report accompanies this application.

The following are its principal conclusions:

- *Based on the Final CFRAM fluvial mapping in the vicinity of the site, the 1% AEP (1 in 100 Year – Flood Zone 'A') and 0.1% AEP (1 in 1000 year – Flood Zone 'B') extreme flood levels in the River Slaney in the vicinity of the proposed development site are predicted as 1.34 m OD (Malin) for both the 1% and 0.1% AEP events respectively.*
- *Based on the Irish Coastal Protection Strategy Study mapping in the vicinity of the site, the 0.5% AEP (1 in 200 Year – Flood Zone 'A') and 0.1% AEP (1 in 1000 year – Flood Zone 'B') extreme tidal flood levels in the River Slaney in the vicinity of the proposed development site are predicted as 1.76 m OD (Malin) and 1.95 m OD (Malin) for the Current Scenario and 2.76 m OD (Malin) and 2.95 m OD (Malin) for the High End Future Scenario respectively.*
- *It is proposed to raise the existing ground levels within the site area to a minimum level of 2.95m OD, which is equal to the predicted 1 in 1000 year (0.1% AEP) High End Future Scenario tidal flood level in the vicinity of the site. This level of 2.95m OD is 1m above the 1 in 1000 year tidal flood level for the Current Scenario.*
- *It is recommended that the finished floor levels are constructed a minimum of 0.3m above the predicted 1 in 1000 year tidal flood level (0.1% AEP) for the High End Future Scenario, i.e. 2.95 + 0.3m = 3.25m OD (Malin).*
- *It is recommended that any existing or proposed surface water pipes or culverts within the site boundary are fitted with appropriately designed tidal flap valves.*
- *In consideration of the Current Scenario, the volume of tidal flood waters that may be displaced by the proposed development site are negligible in consideration of the occurrence of an extreme 0.5% AEP or 0.1% AEP tidal flood event in the Slaney Estuary. Displacement of these negligible volumes of flood waters*

from the area of the proposed development site would simply be attenuated within the vast volume of flood waters within the Slaney Estuary and would have an imperceptible impact on the hydrological regime of the area.

- *In consideration of the predicted 0.1% AEP flow rate in the River Slaney in the vicinity of the site the volume of fluvial flood waters that may be displaced by the proposed development site are negligible in consideration of the occurrence of an extreme 1 % AEP or 0.1% AEP fluvial flood event in the River Slaney. Displacement of these negligible volumes of flood waters from the area of the proposed development site would simply be attenuated within the vast volume of flood waters within the River Slaney and would have an imperceptible impact on the hydrological regime of the area.*
- *The proposed surface water management system shall attenuate surface water runoff from the development to Greenfield Runoff rates in accordance with the GDSDS and shall not result in any displacement of flood waters in the area. As such there will be no increase in runoff from the site beyond the 'greenfield' runoff rate and therefore the development as proposed will not pose an increased flood risk to the area.*
- *As discussed in Section 9 above, development of the site is therefore not expected to have an adverse impact on the existing hydro-morphological regime of the Slaney Estuary.*
- *In consideration of the assessment and analysis undertaken as part of this Site Specific Flood Risk Assessment, overall development of the site is not expected to result in an adverse impact to the hydrological regime of the area and is not expected to adversely impact on adjacent lands or properties.*

Section 4 Road Design Confirmation

The ABP Pre-Application Consultation Opinion states *(inter alia)*:

2. *A report prepared by a suitably qualified and competent person demonstrating specific compliance with the requirements set out in the Design Manual for Urban Roads and Streets and the National Cycle Manual, as well as a map illustrating pedestrian, cycle and vehicular links through the site and connectivity with the wider area.*

The road system for the proposed development has been reviewed by NRB Consulting Engineers Ltd. They confirm that the roads comply with the "Design Manual for Urban Roads and Streets (2013)".

A full stage 3 level road safety audit has been prepared by Roadplan Consulting Ltd. demonstrates full compliance with the "Design Manual for Urban Roads and Streets (2013).   All construction traffic related matters are dealt with in Chapter 11 of the EIAR (prepared by NRB Consulting Engineers), which concludes the following:

It has been demonstrated that the construction and operation of the proposed development will have a negligible and un-noticeable impact upon the continued operation of the adjacent road network.

We conclude that the proposed development is not expected to have any adverse impact in terms of traffic capacity or safety on the surrounding road network. We therefore would encourage a grant of planning for the development from An Bord Pleanála.

Section 5 Importation of Fill.

The ABP Pre-Application Consultation Opinion states *(inter alia)*:

"the following specific information should be submitted with any application for permission:

5. *Construction and Demolition Waste Management Plan (CDWMP) that identifies and describes the extensive infill works and groundworks that are proposed.   Clarification of quantity and description of infill material to be imported in order to deal with the issue of flood risk.*

Appendix C is a report on the importation of fill detailing the quantities required and the type and quality of fill required. It includes details of the phasing of site works and otter zone protection as well as measure to collect, contain and settle site generated runoff to avoid discharge of silty water to the estuary during the construction of the development.

Section 6 Carcur Landfill Gas.

The ABP Pre-Application Consultation Opinion states *(inter alia)* :

"the following specific information should be submitted with any application for permission:

6. *A Report on management, future monitoring and mitigation of gas emissions from the disused landfill."*

Appendix D is a report on the management, future monitoring and mitigation of gas emissions from Carcur landfill site.

Section 7 Response to PA Opinion of 08 May 2020

The ABP Pre-Application Consultation Opinion states *(inter alia)* :

"the following specific information should be submitted with any application for permission:

7. *A response to matters raised within the PA Opinion submitted to ABP on the 08 May 2020*

The following engineering issues were raised:

Flood Risk
Although not located within a designated floodplain, the site is subject to coastal flooding and therefore a full Flood Impact Risk Assessment is required for the entire site.

See Section 3 - Flood Risk Assessment and associated Site Specific Flood Risk Assessment report which has been undertaken for the site by IE Consulting.

- *Applicant proposes to divert surface water to 6 no. connected gallery's which would attenuate storm water prior to discharge to ground water via an oil interceptor.*

This was not and is not proposed for the subject application. Storm water collected in a standard gravity storm water system, passed through a silt trap manholes and oil/petrol interceptors, attenuated for the 1 in 100 year storm event and discharged through 5 outfalls to the Slaney Estuary.

- *Applicant is advised to contact the EPA / Environment Section Wexford County Council to clarify the requirement of a discharge licence to groundwater.*

  No discharge to groundwater is proposed.

- *The Planning Authorities preferred option if possible would be to discharge surface water via attenuation ponds (otter pond) and oil interceptors directly into the estuary.*

  This is the current proposal and was submitted for the Pre-Application consultation.

- *Applicant is advised to consult with the Department of Agriculture, Food & Marine and other relevant bodies to discuss the feasibility of a Foreshore Licence which would permit the discharge of treated surface water into the estuary.*

  The Foreshore Section has been consulted in this matter; please see Appendix A for details and timing of the Foreshore licence application.

- *Surface Water Discharge licence if required to be submitted with the application.*
  No surface discharge licence is required, though a Foreshore Licence is required for the outfalls, see Appendix A for timing of the licence application.

- *Surface water attenuation will be required to be included in the initial phase of development.*

  No discharge to groundwater is proposed.

- *Surface water attenuation measures are required due to rising sea levels and flood risk from the River Slaney on the low lying portion of the site.*

  These are being provided as they are Wexford Co. Co. requirements.

- *Details for surface water attenuation, designed in accordance with SuDS guidelines, will be required to be submitted as part of any subsequent planning application.*

  Details are provided in engineering drawings PL 01, 02, 03, 06 and 07 and in the Storm Water Report in Appendix A.

Section 8 Waste Water

The waste water from the development is to be pumped to the Wexford town and environs sewage system. Twin force mains 80mm and 150mm diameter have already been installed, in 2010, with the agreement of Wexford Council for this purpose along the access road to the proposed railway bridge site. The 80mm pipe is to be used initially to avoid septic conditions arising in the force main. As the site is developed further the 150mm pipe will be used.

Irish water has agreed to the installation of 12 hours emergency storage at the pump station together with a facility for backup power generation. All elements are to be designed to recently issued Irish Water details and specifications.

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July 31st, 2020.

Appendix A - Storm Water Report .

This report contains the following:

1. An outline of the storm water system proposed.
2. Aquaculture concerns
3. Summary calculations for pipe network design
4. Sample and summary calculations for attenuation storage requirements together with a key plan.
5. Discharge and Foreshore Licences

Accompanying this report is a report by Aquafact Ltd. on the storm water discharge to the estuary confirming that the discharge of storm water to the estuary will not impact in any way on aquaculture in the estuary.

Outline of Storm System

It is proposed to install a standard gravity storm water collection system based on the Department of Environment “Recommendations for Site Development Works for Housing Areas”. The system includes the oil interceptors, silt traps and attenuation stores designed to attenuate the 100 year storm.

Surface water runoff generated within the site will be attenuated to Greenfield Runoff rates in accordance with the Greater Dublin Strategic Drainage Study to protect the hydrological regime of the area including the River Slaney and the Estuary.

There are five attenuation stores proposed within the development site, which have been designed to attenuate the 1 in 100 year rainfall event. The discharge from each of these attenuation systems is limited to Greenfield Runoff rates using a ‘Hydrobrake or other approved flow control device. The discharge pipes are to be fitted with tidal flaps and shall discharge to the estuary below the lowest low water level.

One of the attenuation stores, Store No. 4, discharges to the estuary through the otter pond at the reduced attenuated flow rate of 11.4 litres per second. Store No. 5 discharges to the estuary through the marsh at the eastern end of the site close to the railway line, at the reduced attenuated flow rate of 7.2 litres per second.

Stores 1, 2 and 3 discharge directly, after the treatments and attenuation, at rates of 16.9, 18.2 and 7.5 litres per second respectively.

It is not proposed to use the otter pond as an attenuation store as this would involve undesirably large fluctuations in water level in the pond. For that reason the flow is first attenuated in Store No. 4. Because the site is very flat it is not practicable to drain other stores through the otter pond as the pipe gradients would be too flat to guarantee self-cleansing of the pipes.

The discharge pipes discharge to the estuary and are buried under the shore with concrete protection to below the low tide mark. Each outfall is to be fitted with a non-return tidal flap.

Each attenuation store is preceded by an oil interceptor and a silt trap as indicated on the layout plans

These proposals are set out in Engineering Drawings PL 01, 02, 03, 06 and 09.

A key plan showing an outline of the system as well as sample and summary calculations are given at the end of this document.

Aquaculture Concerns

Wexford County Council have advised the client in February this year that they prefer that surface water be discharged to the estuary via oil interceptors and 100 year attenuation storage. Revised proposals for a standard storm water system including the oil interceptors, silt traps and 100 year attenuation stores were presented in the pre-application submission in March this year on this basis.

The Council confirmed their preference for this approach to discharge system in their letter to An Bord Pleanála on May 8th last (2020).

During the Section 5 Pre Application Consultation meeting on June 17th last, the Council again stated the above preference though unspecified concerns were expressed vis-a-vis aquaculture by Mr. Brendan Cooney Senior Scientist with Wexford. On June 23rd last a full set of the engineering drawings were sent to him by email as well as information on the massive dilution available for the treated and settled storm water by email and by letter, requesting his comments. To date no response has been received.

In view of this lack of information or guidance from the Council as to what the concerns are, the applicant was left with no alternative but to commission a report from Aquafact Ltd., nationally recognised experts in this field.

Their report which accompanies this report confirms that due to:

1. the treatment and attenuation of the storm water
2. the huge rates of dilution and
3. the fact that the freshwater will float on top of the heavier estuary saltwater

the impact of the storm water discharge from the development on the estuary waters will be virtually unmeasurable and will not negatively affect aquaculture in the estuary.

Aquafact Ltd. also states that attenuation of the storm water is not necessary. It is being provided now because it is the policy of the Council to require this.

Discharge and Foreshore Licences

No Water Pollution legislation licence is required for surface water discharge to the estuary. This fact is confirmed by Aquafact Ltd. in their report.

Foreshore licences are required for the surface water outfalls; however the Foreshore Section, Marine Planning, Policy and Development, Department of Housing and Local Government stated their preference that Planning Permission is first obtained before applying for a foreshore licence.

This is because any small changes to the proposals during the planning process will invalidate any licence issued and the licence process will need to start again from scratch.

A copy of an email from the Foreshore Section confirming their preference concerning timing is included at the end of this report.

Also accompanying this document as Appendix E are the following documents which were submitted to Irish Water during their approval process:

1. A Statement of Design Acceptance from Irish Water for the Water and Wastewater
2. Calculations of water demand for the project.
3. A mathematical modelling of the water pressure in the water supply network.
4. Pump Hydraulics at one third development completed
5. Pump Hydraulics at Full Development Completed.
6. Wastewater Pumping System Residence Time Calculations
7. Minimum Sump Volume Calculations Under Partial Development
8. Minimum Sump Volume Calculations at Full Development
9. Emergency Storage Volume Calculation
10. Ballast Calculations for Pump Station and Emergency Storage Tank
11. Pump and related equipment specifications from Xylem for Flygt pumps.

The on-site pumping station is located above the 1 in 1000 year flood level allowing for the OPW High End Scenario for sea level rise over the next 100 years. These levels are well above the levels required in Irish Water's letter to An Bórd Pleanála on May 12th, 2020.

This letter refers to the need for a flood risk assessment report. A Site Specific Flood Risk Assessment report has been undertaken for the site by IE Consulting and accompanies the application.

The letter also refers to upgrading works of Carcur Waste Water Pump Station. This pump station is an existing off-site Irish Water pump station, not to be confused with the on-site pump station which has been designed to their standards and approved by them.

Section 9 Water Supply

Water for the development is to be provided from the Wexford town public water supply and a supply main has already been installed with the agreement of Wexford County Council along the access road to the proposed railway bridge site. Water supply infrastructure will be constructed to Irish Water's specifications and details.

Digitally signed by Arthur Murphy
DN: cn=Arthur Murphy, o=Arthur Murphy and Co, ou, email=arthur@ameng.ie, c=IE
Date: 2020.08.11 11:07:46 +01'00'

Arthur Murphy B.E., M.Eng.Sc., MIEI, C.Eng.

Arthur Murphy

From:	foreshore <foreshore@housing.gov.ie>
Sent:	16 June 2020 10:32
To:	Arthur Murphy
Subject:	RE: Foreshore Licence.
Attachments:	Untitled attachment 00028.txt; Untitled attachment 00031.htm

Hi Arthur

If a development requires planning permission in addition to foreshore consent it is usually preferable to have planning permission first although they can be sought in tandem.

It is a matter for the developer to decide how best to obtain the necessary consents but if the project that gets planning is different to that applied for in the foreshore process, the foreshore process will have to be redone to match planning.

Regards

Danny O'Brien
Marine Planning, Policy and Development
Dept of Housing, Planning & Local Government
Newtown Road
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087 6656703

From: Arthur Murphy [mailto:arthur@ameng.ie]
Sent: 16 June 2020 10:27
To: foreshore <foreshore@housing.gov.ie>
Subject: Foreshore Licence.

Dear Sir / Madam

Can you let me know whether you prefer that an outfall on the foreshore has planning permission before a foreshore licence is applied for.

Best Regards

Arthur.

Arthur Murphy

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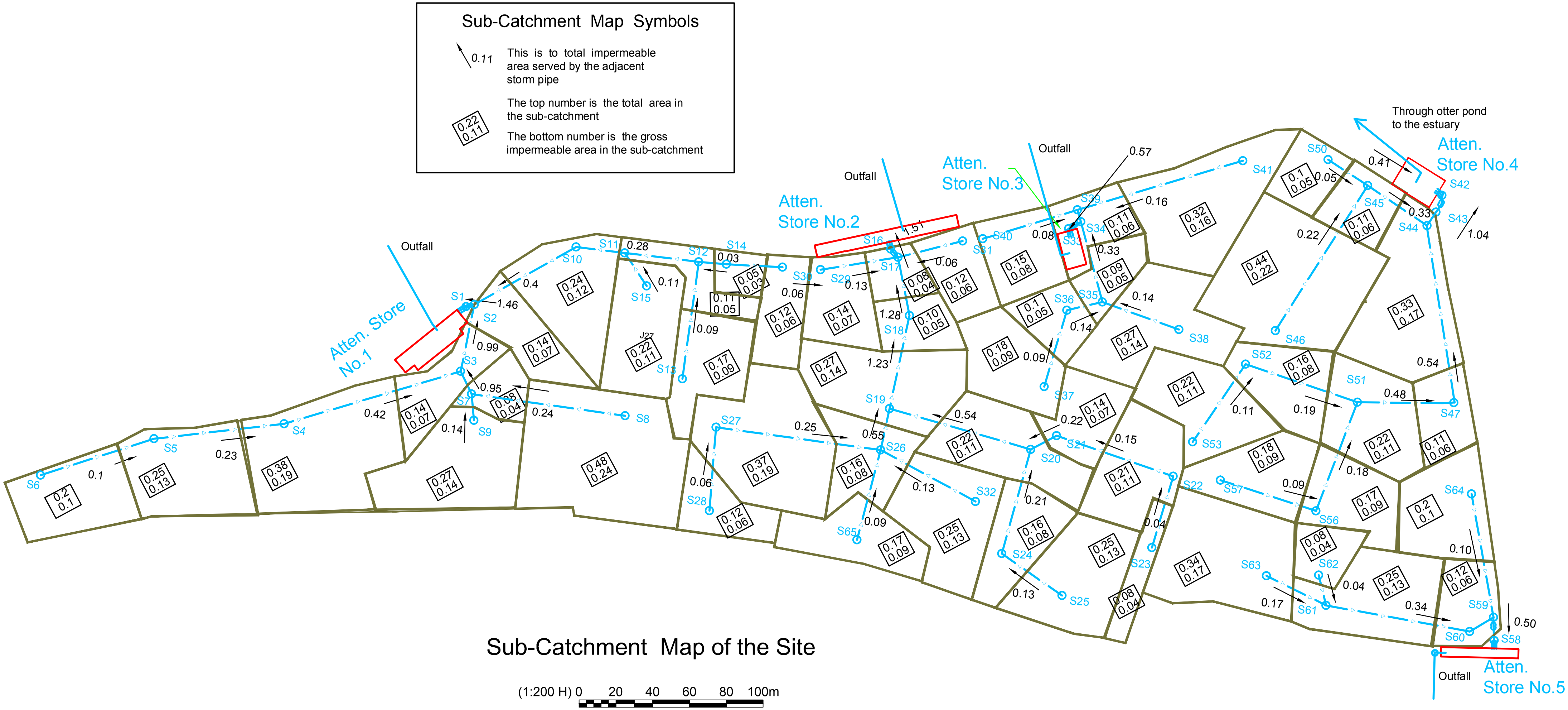

Sub-Catchment Map of the Site

STORM SEWER CALCULATIONS Park Wexford July 31st 2020

FROM	TO	t incr mins	Imperm A (ha)	L (m)	Tc min	I mm/hr	Pipe (mm)	s %	Manning n	V m/s	Qcap l/s	Qdes	
S8	S7		0.240	76	5.0	53	225	0.5	0.011	0.9	38	35	O.K.
S7	S3	1.34	0.420	12	6.3	47	300	0.25	0.011	0.8	57	54	O.K.
S9	S7		0.140	13	5.0	53	225	0.3	0.011	0.7	29	21	O.K.
S13	S12		0.090	58	5.0	53	225	0.35	0.011	0.8	31	13	O.K.
S12	S11	1.22	0.120	36	6.2	47	225	0.25	0.011	0.7	27	16	O.K.
S11	S10	0.90	0.280	24	7.1	43	300	0.25	0.011	0.8	57	34	O.K.
S10	S2	0.49	0.400	57	7.6	42	300	0.25	0.011	0.8	57	46	O.K.
S15	S11		0.110	20	5.0	53	225	0.3	0.011	0.7	29	16	O.K.
S14	S12		0.030	14	5.0	53	150	0.3	0.011	0.6	10	4	O.K.
S25	S24		0.130	37	5.0	53	225	1.14	0.011	1.4	57	19	O.K.
S24	S20	0.43	0.210	54	5.4	51	225	1.14	0.011	1.4	57	30	O.K.
S28	S27		0.060	42	5.0	53	150	2.34	0.011	1.6	28	9	O.K.
S27	S26	0.45	0.250	81	5.4	51	225	1.15	0.011	1.4	57	35	O.K.
S26	S19	0.94	0.550	22	6.4	46	300	0.4	0.011	1.0	72	71	O.K.
S32	S26		0.090	45	5.0	53	150	2.03	0.011	1.5	26	13	O.K.
S30	S29		0.130	37	5.0	53	225	0.3	0.011	0.7	29	19	O.K.
S29	S17	0.84	0.210	54	5.8	49	225	0.3	0.011	0.7	29	28	O.K.
S31	S17		0.060	32	5.0	53	300	0.5	0.011	1.1	81	9	O.K.
S38	S35		0.140	40	5.0	53	225	0.25	0.011	0.7	27	21	O.K.
S41	S39		0.160	48	5.0	53	300	0.25	0.011	0.8	57	24	O.K.
S40	S39		0.080	84	5.0	53	300	0.25	0.011	0.8	57	12	O.K.
S39	S34	1.73	0.240	6	6.7	45	300	0.3	0.011	0.9	63	30	O.K.
S50	S45		0.050	24	5.0	53	150	0.5	0.011	0.7	13	7	O.K.
S57	S56		0.090	49	5.0	53	150	1.57	0.011	1.3	23	13	O.K.
S56	S51	0.64	0.180	57	5.6	50	225	0.96	0.011	1.3	52	25	O.K.
S63	S61		0.170	33	5.0	53	225	1.56	0.011	1.7	66	25	O.K.
S65	S26		0.100	61	5.0	53	150	2.07	0.011	1.5	26	15	O.K.
S6	S5		0.10	58	6.0	48	150	1.02	0.011	1.0	18	13	O.K.
S5	S4	0.94	0.23	64	6.9	44	225	0.52	0.011	1.0	38	28	O.K.
S4	S3	1.11	0.42	90	8.0	40	300	0.99	0.011	1.6	114	47	O.K.
S3	S2	0.98	0.99	33	9.0	38	375	0.38	0.011	1.2	128	104	O.K.
S2	S1	0.47	1.46	7	9.5	37	450	0.38	0.011	1.3	208	149	O.K.
S23	S22		0.04	36	5.0	53	150	5.31	0.011	2.4	42	6	O.K.
S22	S21	0.26	0.15	60	5.3	52	225	1.14	0.011	1.4	57	22	O.K.
S21	S20	0.70	0.22	14	6.0	48	225	2.84	0.011	2.3	90	29	O.K.
S20	S19	0.10	0.54	72	6.1	48	300	0.91	0.011	1.5	109	72	O.K.
S19	S18	0.78	1.23	46	6.8	45	450	0.33	0.011	1.2	194	152	O.K.
S18	S17	0.63	1.28	29	7.5	42	450	0.33	0.011	1.2	194	150	O.K.
S17	S16	0.40	1.51	7	7.9	41	450	0.3	0.011	1.2	185	172	O.K.
S37	S36		0.09	39	5.0	53	150	2.16	0.011	1.5	27	13	O.K.
S36	S35	0.43	0.14	18	5.4	51	225	0.2	0.011	0.6	24	20	O.K.
S35	S34	0.50	0.33	41	5.9	48	300	0.2	0.011	0.7	51	44	O.K.
S34	S33	0.94	0.57	7	6.9	44	300	0.5	0.011	1.1	81	70	O.K.
S46	S45		0.22	17	5.0	53	225	0.48	0.011	0.9	37	33	O.K.
S45	S44	0.31	0.33	35	5.3	51	300	0.25	0.011	0.8	57	47	O.K.
S53	S52		0.11	46	5.0	53	150	1.6	0.011	1.3	23	16	O.K.
S52	S51	0.59	0.19	58	5.6	50	225	0.4	0.011	0.8	34	26	O.K.
S51	S47	1.14	0.48	42	6.7	45	375	0.15	0.011	0.7	80	60	O.K.
S47	S44	0.96	0.54	7	7.7	42	375	0.15	0.011	0.7	80	62	O.K.
S44	S42	0.16	1.04	7	7.9	41	450	0.15	0.011	0.8	131	119	O.K.
S62	S61		0.04	15	5.0	53	150	1.43	0.011	1.2	22	6	O.K.
S61	S60	0.20	0.34	71	5.2	52	300	0.5	0.011	1.1	81	49	O.K.
S60	S59	1.03	0.40	15	6.2	47	300	0.5	0.011	1.1	81	52	O.K.
S59	S58	0.22	0.50	7	6.5	46	300	0.99	0.011	1.6	114	64	O.K.

Sample STORM WATER ATTENUATION CALCULATION

The calculations are based on the Greater Dublin Strategic Drainage Study.

The attenuation storage is to be provided in an attenuation tank as detailed here and the discharge is to be limited, by means of a Hydrobrake or approved alternative, to the calculated allowable discharge set out below.

Extreme Rainfall Return Periods (Source - Met Eireann)

Location: WEXFORD

110 % of Maximum rainfall (mm) of indicated duration for the indicated return period.

	Return Period (years)				
Duration	5	10	20	50	100
60 min	19	23	27	34	40
2 hour	24	29	34	43	50
4 hour	31	37	43	53	62
6 hour	36	42	49	60	70
12 hour	46	54	62	75	86
24 hour	58	68	78	93	107

Site Details

CALCULATION OF GREEN FIELD RUNOFF

Given a site area of	1.22	hectares	Attenuation Tank No. 3.	
Area for calculation of AREA	50	ha (recommended minimum in Greater Dublin Study)		
AREA in km2	0.5	km^2 (SAAR)	1163	mm
SOIL	0.4	for Silty (Intermediate) soils		

Soil type at the site is aluvial Silt (Soil Map of Co. Wexford (National Soil Survey of Ireland))

QBAR rural for this AREA	0.31	m3/s	$= 0.00108 * A_{REA}^{0.89} * S_{AAR}^{1.17} * S_{OIL}^{2.17}$	
QBAR per hectare	6.16	l/s		
Permissable outflow will then be			7.5	l/s
Impermeable area			0.67	ha
% of impermeable area contributing to direct runoff to the drainage system			75	%
(Per Appendix E-2 Greater Dublin Strategic Drainage Study)				
Impermeable area contributing to the the drainage system			0.50	ha

RUNOFF VOLUME

The runoff volume, in cubic metres, from the catchment for all the storms listed in the rainfall table above is set out below:

	Return Period (years)				
Duration	5	10	20	50	100
60 min	95	115	137	172	203
2 hour	122	146	173	214	251
4 hour	156	185	218	266	310
6 hour	179	213	249	303	351
12 hour	229	270	313	378	434
24 hour	293	342	393	470	536

The allowable outflow and required storage for various durations are:

			allowable outflow		storage for 100 yr storm	
60 min	60	mins	27	m3	176	m3
2 hour	120	mins	54	m3	197	m3
4 hour	240	mins	108	m3	202	m3
6 hour	360	mins	162	m3	189	m3
12 hour	720	mins	325	m3	109	m3
24 hour	1440	mins	650	m3	0	m3

The storage required on site for a 100 year storm would be 202 m3

Aquacell Volume Calculations are as follows

Gross Storage allowing for voids	212	m3	Soffit level of tank	2.9	m OD
Proposed Aquacell invert level	1.7	m OD			
Internal operational level	1.2	m OD			
Floor area of Aquacells	177	m2			
Nett Storage provided	202	m3			

Summary of Attenuation Details					
Gallery Number	No. 1	No. 2	No. 3	No. 4	No. 5
Total Area (ha)	2.746	2.946	1.22	1.85	1.17
Percentage impermeable area (%)	55	55	55	55	55
Impermeable Area (ha)	1.5103	1.6203	0.671	1.0175	0.6435
% of Imp.area contributing directly to the drainage system (Dublin GSS)	75	75	75	75	75
Impermeable area directly contributing to the Gallery (ha)	1.13	1.22	0.50	0.76	0.48
Permitted Outflow (litres per second)	**16.9**	**18.2**	**7.5**	**11.4**	**7.2**
Attenuation Storage required	**454**	**487**	**202**	**306**	**193**
Depth of Aquacell array	1.2	1.2	1.2	1.2	1.2
Plan Area of Aquacell system	398	427	177	268	170
Aquacell array length	39.0	47.5	18.0	20.0	34.0
Average width	10.2	9.0	10.0	13.5	5.0
Attenuation Storage provided	454	487	205	308	193

Appendix B

Aquafact Ltd

Aquaculture Impact Report

Aquaculture Impact Report

Dispersion and dilution of storm water

from a proposed housing development near Wexford Town.

Produced by

AQUAFACT International Services Ltd

On behalf of

Arthur Murphy & Co Consulting Engineers

July 2020

AQUAFACT INTERNATIONAL SERVICES LTD.

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Introduction.

Mr. A. Murphy, Consulting Engineer, Wexford commissioned AQUAFACT on behalf of Neville and Sons Wexford to examine the dilution and dispersion of storm water out falls into the Slaney River from a proposed 400 housing development to the West of Wexford Bridge (see Figure 1 for site location) and to comment on the possibility of this water interacting with mussel beds in Wexford Harbour.

Figure 1. Location of development lands at Park, Wexford.

Storm water by its nature is generated during periods of high rainfall and the river is in a spate condition. Storm water is 100% fresh water and therefore has a salinity of 0 p.s.u. (practical salinity units) whereas the salinity of the Slaney Estuary waters is in the region of 30 p.s.u. *i.e.* ten times saltier (and therefore 10 times heavier) than the storm water. The result of this is that the lighter storm water will float on the sea water and given the spate conditions of the river, it will flow out to the open part of Wexford Harbour where it will eventually be absorbed by the sea water in a process called entrainment. In order for this to occur greater volumes of seawater will be required to bring about this absorption.

Storms (and associated rainfall) can occur in any month of the year though they are more frequently recorded in late Autumn and Winter months. Freshwater temperatures in these periods of the year are typically colder than marine waters because these water bodies are far smaller that marine waters and cool down faster than the sea. However, the difference is not sufficient to overcome the much greater difference in density and will not significantly alter the way the two water bodies interact.

Measurements of the flow rate of the Slaney River in spate conditions are as high as 1,738m^3 sec.

The permitted maximum out flow of all 5 storm water out falls from the proposed development is 62m^3 sec which is *ca* 0.003% of the spate flow. This alone will achieve dilutions of any solutes that remain in the storm water post sand and oil filter and attenuation pond treatment to levels that are so low as to only be able to be detected by very sophisticated analytical methods.

The proposed development is for a large strategic housing development. The proposed storm water runoff from the hard paved areas is to be collected, attenuated and discharged to the estuary via 5 storm water outfalls. The total impermeable area on the site is 4.097ha of which 75% is collected in the storm water system and discharged to the estuary via these five outfalls (refer to Engineering report and drawing PL 09 “Storm System Design” by Arthur Murphy and Co.). The total area of the development is *c*. 9.93ha. Four of the five storm water outfalls are located around the northern perimeter of the site and discharge to the Slaney Estuary below the low water tide level such, that at all stages of the tide the outfalls are fully submerged. The fifth discharges into a marsh area at the south east corner of the site.

No discharge licence is required for these outfalls; however, a Foreshore licence will be applied for.

Hydrology

The receiving waters for the storm water discharge from the proposed development is directly to the Slaney Estuary which flows east and southeast passing along the northern boundary of the site on its way to Wexford Harbour and then into the Irish Sea.

Tidal regime

The receiving waters of the Slaney are tidal with the tidal limit extending upstream to Enniscorthy. The tidal range for Wexford Harbour is 1.5m on mean Spring tides and 0.5m on mean Neap tides.

Mean Springs		Mean Neaps	
Highwater	Low water	Highwater	Low water
0.72m OD	-0.78m OD	0.22m OD	-.28m OD

Based on an in-house Telemac2d hydrodynamic model of Wexford Harbour and the Slaney Estuary developed by Hydro Environmental Ltd. (refer to Fig 4 for domain extents), the estimated tidal flushing volume in the estuary passing the proposed site is 6.78 million m^3 on a mean spring tide and 2.42million m^3 on a neap tides. This represents average ebbing and flooding tidal discharges of 301 m^3/sec on mean spring tides and 107.5m^3/s on mean neap tides.

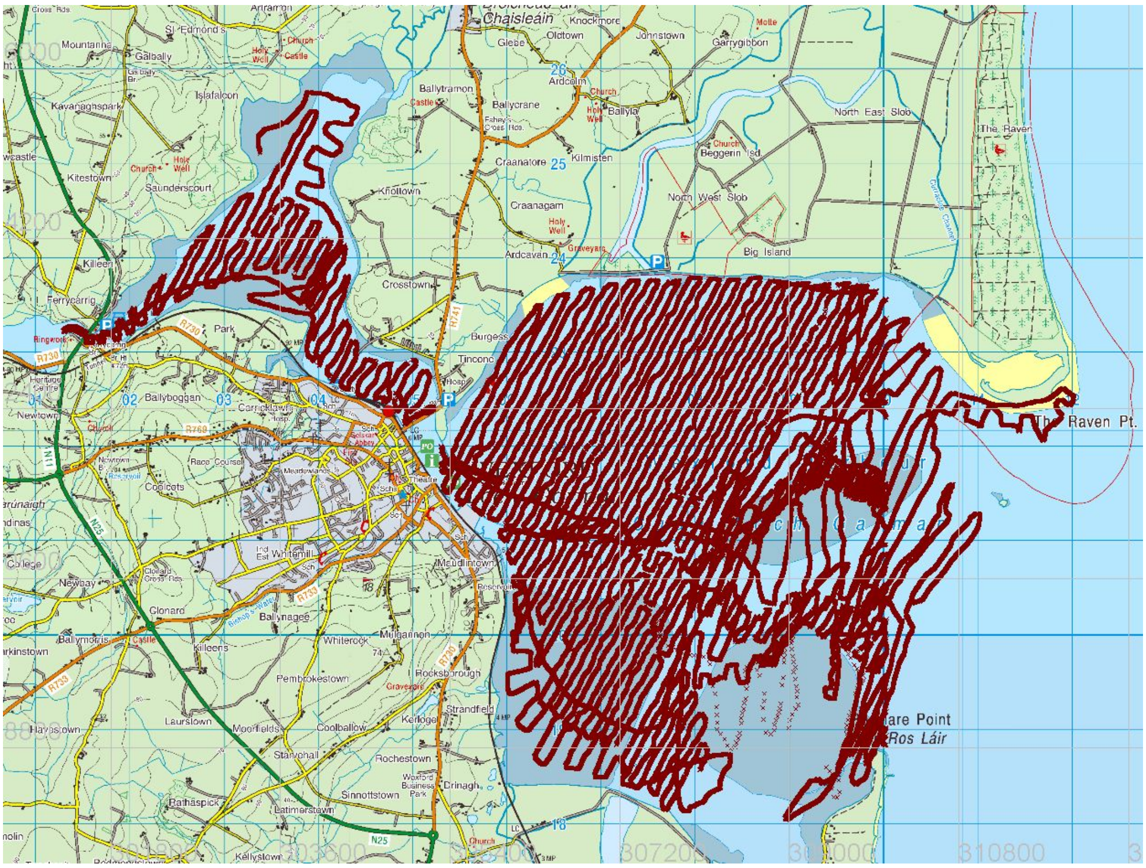

Figure 3. AQUAFACT survey tracks of Wexford Harbour used to define model bathymetry.

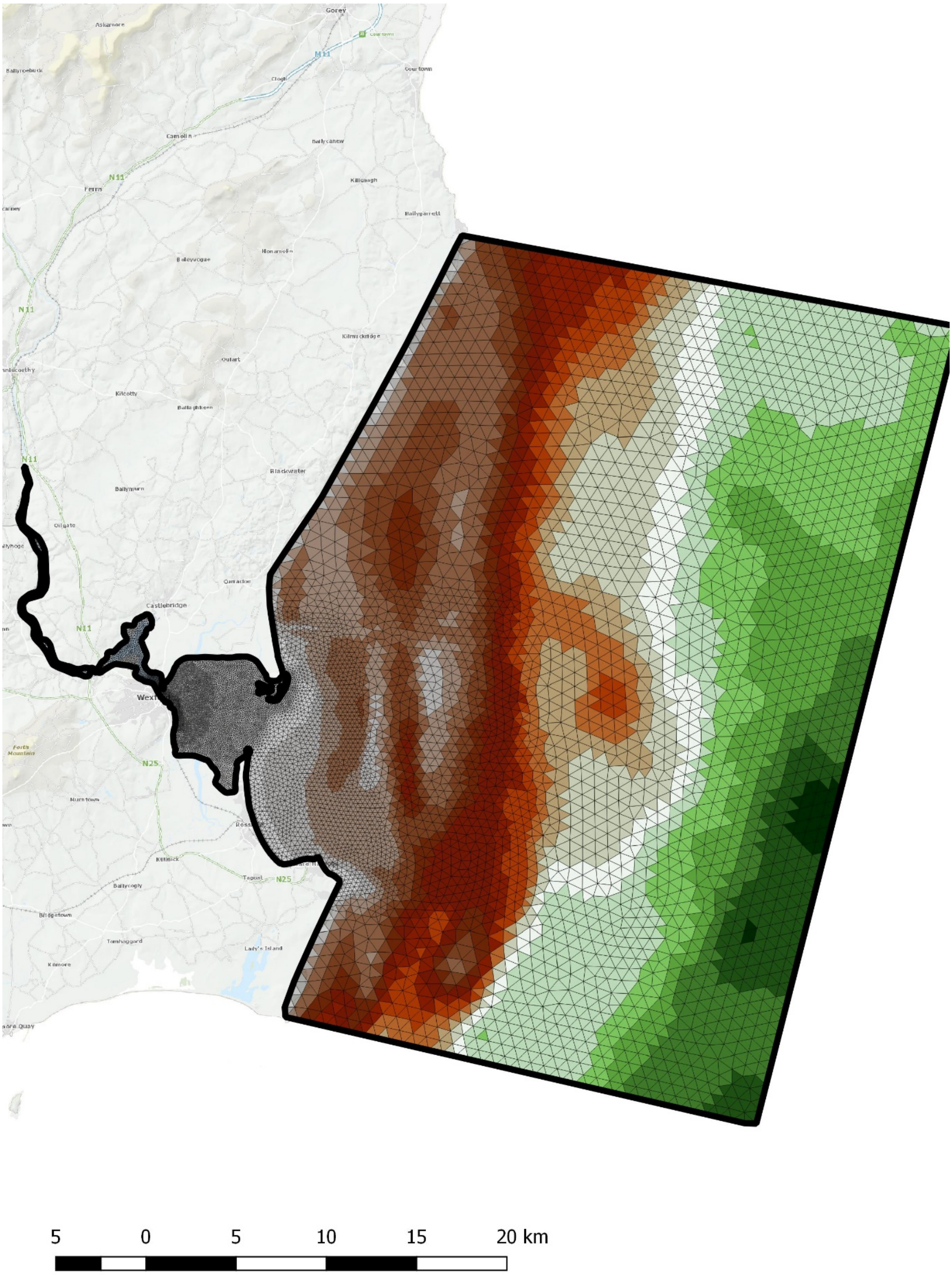

Figure 4. Hydrodynamic model domain of Wexford Harbour and Slaney Estuary.

Slaney River fluvial flows.

The Slaney River drains a total catchment area to Wexford Harbour of 1,850km^2. This represents a large contributing catchment in respect to Irish Rivers. The nearest hydrometric gauge for flow measurement on the Slaney is located at Scarrawalsh (12001) upstream of Enniscorthy, having a catchment area to the gauge of 1036km^2 (56% of the total catchment area to Wexford).

The median flow (exceeded 50% of the time in an average year) to Scarrawalsh is 13.9m^3/sec and the 95-percentile low flow is 2.99m^3/s, the extreme 99-percentile low flow is 0.793m^3/s. The mean annual rainfall to this station is 1170mm and the annual evapotranspiration is *c*. 570mm giving an effective rainfall of 600mm per annum representing an expected average runoff of 19.7 m^3/s. For the total catchment to Wexford Harbour of 1850km^2, the estimated median freshwater flow is 24.8m^3/s, the 95-percentile low flow is 5.3 m^3/s and the 99-percentile low flow is 1.42 m^3/s. The mean annual maximum flood in the Slaney River to Wexford Harbour is estimated using the OPW flood studies update methods (FSU) to be of the order of 226 m^3/s and the estimated 100 year flood at 393 m^3/s.

During periods of flooding, the time to peak of the flood hydrograph is 15.2 hours and the critical rain storm duration is 32 hours.

Storm water discharge and dilution

This storm water discharge represents rainwater from roofs, roads, pathways and driveways and consequently given the area's residential use, it will represent generally unpolluted discharge to the Slaney Estuary. The storm water is passed through storm water attenuation so as to control the rate of discharge and thereby prevent flooding. The attenuation storage is provided in the form of underground Wavin aquacells that have inlet and outlet pipes and provide some infiltration through the base. A flow control device is placed on the outlet pipe to restrict runoff to Greenfield flood runoff rates.

Upstream of all of the attenuation galleries and outfalls, a silt trap and petrol interceptor are proposed so as to protect the receiving marine waters from pollution.

The combined storage volume from the five tanks is 1,642m^3 with designed for a maximum permissible discharge rate of 61.2l/s. This rate is based on a flood study report soil index of 0.4 representing moderate infiltration and runoff. The calculations are presented in Arthur Murphy Drawing PL09.

Given that the proposed storm water discharge is directly to the Slaney Estuary, flood control through on site attenuation and flow control is not considered necessary given the large available capacity in the estuary which sees tidal flows incoming and outgoing of 301 and 107 m^3/s over a 6.2 hour period adjacent to the site and average river flow and annual flood rates of 20 and 226 m^3/s respectively over many hours and days. However, in keeping with Wexford County Council policy of attenuation for all storm water discharges from urban development up to the 100year design storm, these attenuation facilities have been proposed.

The critical storm duration for determining the attenuation storage based on the rainfall statistics and permissible runoff rate is 4 hours. The proposed attenuation will store excess over the permissible storm water and release it over a longer duration of *c*. 12 hours. At the 100 year 4 hour duration rainstorm event, 62mm of rainfall depth will occur representing a total storm volume of 2540m^3 which will be discharged to the estuary via the outfalls over a 12hour period. Such a rainstorm event could occur independent of a fluvial flood in the Slaney (local thunderstorm event) which could occur during a 95-percentile low flow in the Slaney of 5.3m^3 per second. In the absence of tidal flushing (which is not the case), the available dilution from the 95-percentile fluvial contribution provides a dilution of 87. The tidal saline contribution on Neap tide is 107.5m^3/sec which provides a dilution ratio of 1:1,750 and the tidal dilution during Spring tides is 1 in 5,000. The effect of the storm discharge on the overall salinity balance within the estuary at Wexford will be negligible.

Salinity change calculation

1 Low River Flow – Neap tides

Allowing a typical Irish Sea salinity of 33ppt in the inflowing seawater at tidal flushing rate of 107.5m^3/sec over 12.4hour neap tidal cycle.

Salinity of 0ppt freshwater from the Slaney River at 95-percentile low flow of 5.3m^3/s.

Proposed 100.year storm discharge from site at salinity of 0ppt at discharge rate of 0.0612m^3/s.

The average salinity will decrease from 31.449ppt to 31.432ppt (decrease in salinity of 0.017ppt).

2 Low River Flow – Spring tides

Allowing a typical Irish Sea salinity of 33ppt in the inflowing seawater at tidal flushing rate of 301m³/sec over 12.4hour spring tidal cycle.

Salinity of 0ppt freshwater from the Slaney River at 95-percentile low flow of 5.3m³/s.

Proposed 100 year storm discharge from site at salinity of 0ppt at discharge rate of 0.0612m³/s.

The average salinity will decrease from 32.429ppt to 31.422ppt (decrease in salinity of 0.007ppt).

3 Mean River Flow – Neap tides

Allowing a typical Irish Sea salinity of 33ppt in the inflowing seawater at tidal flushing rate of 107.5m³/sec over 12.4hour neap tidal cycle.

Salinity of 0ppt freshwater from Slaney at median river flow of 24.8m³/s.

Proposed 100 year storm discharge from site at salinity of 0ppt at discharge rate of 0.0612m³/s.

The average salinity will decrease from 26.814ppt to 26.802ppt (decrease in salinity of 0.012ppt).

4 Mean River Flow – Spring tides

Allowing a typical Irish Sea salinity of 33ppt in the inflowing seawater at tidal flushing rate of 301m³/sec over 12.4hour spring tidal cycle.

Salinity of 0ppt freshwater from Slaney at median river flow of 24.8m³/s.

Proposed 100 year storm discharge from site at salinity of 0ppt at discharge rate of 0.0612m³/s.

The average salinity will decrease from 30.488ppt to 30.482ppt (decrease in salinity of 0.006ppt).

5 Annual (2 year) River Flood – Neap tides

Allowing a typical Irish Sea salinity of 33ppt in the inflowing seawater at tidal flushing rate of 107.5m³/sec over 12.4hour neap tidal cycle.

Salinity of 0ppt freshwater from Slaney at median river flow of 226m³/s.

Proposed 100 year storm discharge from site at salinity of 0ppt at discharge rate of 0.0612m³/s.

The average salinity will decrease from 10.637ppt to 10.635ppt (decrease in salinity of 0.002ppt).

6 Mean River Flow – Spring tides

Allowing a typical Irish Sea salinity of 33ppt in the inflowing seawater at tidal flushing rate of $301m^3$/sec over 12.4hour spring tidal cycle.

Salinity of 0ppt freshwater from Slaney at median river flow of $226m^3$/s.

Proposed 100 year storm discharge from site at salinity of 0ppt at discharge rate of $0.0612m^3$/s.

The average salinity will decrease from 18.848ppt to 30.846ppt (decrease in salinity of 0.002ppt).

Conclusion

The large tidal flushing dilutions and large River Slaney freshwater inflows provide ample dilution for the proposed storm water discharge from the proposed development to ensure that the water quality status of the estuary will not be impacted. Added to this, the fact that the freshwater will float over the heavier saline water and be washed out to sea is a further reason why there being any measurable impact on seawater is extremely low. The impact on salinity within the estuary even at proposed 100 year design storm water discharge will be negligible.

To conclude, this report confirms that due to:

1. The treatment and attenuation of the storm water,
2. The huge rates of dilution and
3. The fact that the freshwater will float on top of the heavier estuary saltwater

the impact of the storm water discharge from the development on the estuary waters will be virtually unmeasurable and will not negatively affect aquaculture in the estuary.

Appendix C

Construction Management Plan

on

The Importation of Fill

and

Related Ecological Protection Measures.

July 31st, 2020.

Arthur Murphy & Co | Civil & Structural Engineering Consulting

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Appendix C

July 31st, 2020.

Construction Management Plan on The Importation of Fill and Related Ecological Protection Measures.

Introduction

This report contains the following:

1. The Need for Fill and the Required Fill Quantities.
2. Description of Fill Material Proposed.
3. Placement of Fill and Building Foundations.
4. Phasing of Ecological and Site Filling Works.
5. Measures for the Prevention of Flooding and Water Contamination.

   Included also are A3 copies of engineering drawings PL 10, PL11 and PL 12 which set out in some detail the nature and extent of the fill requirements, the treatment of the shoreline and otter protection measures and the phasing of the development.

Waste disposal arrangements and various ecology measures in addition to standard construction matters are covered in the separate document submitted with this application entitled “Construction Management Plan For development at Park, Carcur, Wexford Incorporating Site Specific Safety, Health & Welfare Statement” by Wm. Neville & Sons Construction Ltd.

The Need for Fill and the Required Fill Quantities

Significant importation of fill is required to raise ground levels as part of the development of the site. The extent of fill required can be seen in Engineering Drawings PL10.

The nett volume of fill has been established first of all assessing the gross fill including imported building stone for construction and including the volume of the attenuation tanks. The volume of building stone and the attenuation tank volumes were separately assessed and subtracted from the gross volume to give the nett volume of soil fill.

The volume of building stone used for road build-up, trench backfill and hardstanding and house subfloor stone was assessed as shown in Table 1 below. The gross quantity of fill required was assessed by taking sections across the site at 50 metre intervals. Three sample sections are shown on engineering drawing PL10 as well as a longitudinal section through the site. Table 2 below gives the cross-sectional area of cut and fill at each section and shows the calculation of the gross fill and nett fill requirement.

Tables 1 and 2 show that the gross fill including building stone and attenuation stores is 137,500 cubic metres and the volume of stone and the attenuation stores is 61,000 cubic metres. The volume of soil fill is then 76,500 cubic metres. It can be seen from Table 2 that by assuming a 10 year building period and 48 no. 5 day working weeks that the average number of trucks bringing soil for site build-up per working day is 3.5.

Table 2 Calculation of Volume of Site Build-up Fill Required				
Section at Station (m)	Cross Section Area (m^2)		Volumes (m^3)	
	Cut Area	Fill Area	Cut	Fill
0	0	32	0	1600
50	0	295	0	14750
100	0	625	0	31250
150	0	740	0	37000
200	0	627	0	31350
250	20	550	1000	27500
300	43	270	2150	13500
350	79	180	3950	9000
400	153	110	7650	5500
450	193	60	9650	3000
500	148	110	7400	5500
550	139	0	6950	0
600	33	0	1650	0
650	35	0	1750	0
700	6	0	300	0
Total Volumes of Cut			42450	
Total Volume of Fill				179950
Gross Volume of Imported Fill Required (Fill Volume less On-site Cut Volume)				137500
Volume of Imported Building Materials				61000
Nett Volume of Clay Fill Required				76500
Equivalent No. of Trucks at 9 m^3 per truck				8500
Trucks per year over 10 year construction period				850
Average trucks per day (48 no. 5 day weeks, 240 days)				3.5

Description of Fill Material Proposed

The imported fill will be clean inert soil from green field building projects in the vicinity of Wexford town. The fill will for the most part be clay with perhaps some gravel fill as may become available. Before a site is approved for use as a source of fill material for the development it will be assessed for suitability.

Only green field site excavation material is to be used. Incoming fill will be monitored visually on site to ensure maintenance of quality. In addition tests will carried out to inform appropriate compaction of the fill on site.

Table 3 below gives the results of a number of tests on 3 samples taken from a large residential site currently operated by Wm. Neville & Sons Ltd. at Clonard near Wexford town.

Table 3 – Analysis Results for 3 Samples from Wm Neville & Sons Ltd at Clonard Wexford

Chemtest
The right chemistry to deliver results

Results - Soil

Project: P20030 Wm Neville

Client: Priority Geotechnical Ltd				**Chemtest Job No.:**	20-07359	20-07359	20-07359
Quotation No.:				**Chemtest Sample ID.:**	982059	982060	982061
				Sample Location:	Sample 2	Sample 3	Sample 4
				Sample Type:	SOIL	SOIL	SOIL
				Date Sampled:	04-Mar-2020	04-Mar-2020	04-Mar-2020
Determinand	**Accred.**	**SOP**	**Units**	**LOD**			
Moisture	N	2030	%	0.020	11	10	9.9
Nitrogen (Total)	N	2115	%	0.010	0.10	0.10	0.10
Cation Exchange Capacity	N	2400	meq/100g	0.10	1.0	1.3	1.3
Calcium	N	2400	mg/l	20	700	900	950
Magnesium (Extractable)	N	2400	mg/l	2.0	180	200	190
Sodium	N	2400	mg/l	2.0	18	26	17
Organic Matter	U	2625	%	0.40	0.83	0.57	0.55

Placement of Fill and Building Foundations .

The fill will be placed in 150mm layers and compacted to give a CBR of 3 percent. Fill under roads will be placed in line with NRA standards for Roads.

Where necessary with deeper fill, ground stabilization with lime may be employed depending on site conditions and the compaction characteristic of the fill.

All buildings will be constructed on piled foundations except where the fill is less that 1m and the existing soil has adequate bearing properties.

In buildings constructed on piles sewer and similar services will be suspended from the ground floor slab.

Development Phasing and Related Ecology Protection Measures

The site will be developed in four phases starting with Phase 1 at the eastern end of the site and continuing westward with the three later phases. In each phase measures will be put in place to protect the otter zone by the edge of the estuary and to prevent silt laden water from entering the estuary. The measures are set out in Engineering Drawing PL 12 and consist of the following:

Phase 1 Overview of Ecology Related Measures

1. Construct new otter pond 6 months before commencing the main development.
2. After confirmation that otters are using the it fill in the existing small otter pond.
3. Clear the line of the proposed berm for the full length of the berm and construct a 1 metre high berm with a top width of 1m and 1 in 3 side slopes on the line shown for the full extent of the site to

prevent escape of silty water to the estuary and guide it to temporary siltation ponds as outlined below.

4. Construct a dog and intruder proof fence along access road and around the service compound to prevent site access and access to the beach.
5. Construct new otter pond and after its completion fill in the existing small otter pond.
6. Construct otter boundary fence for the Phase 1 area.
7. Install the five permanent storm water outfalls at Attenuation. This work to be done outside of the over-wintering period for water birds.
8. Construct siltation ponds at the future locations for the five Attenuation Stores all areas of the site grade to these ponds before discharge to the estuary after settlement via the installed outfalls.
9. Strip topsoil from Phase 1 and 2 areas and stockpile in Phase 3 area.
10. Import and consolidate fill in Phase 1 area.
11. Construct Phase 1 and as needed utilise topsoil from stockpile in Phase 3 area.
12. Replace the temporary siltation ponds in Phase 1 with the permanent attenuation stores and related silt traps and oil/petrol interceptors when most of the construction is completed and the danger of siltation of the stores has passed.
13. Remove berm in Phase 1, complete path and landscape.

Phase 2 Overview of Ecology Related Measures.

1. Construct a site security fence on the boundary between Phases 1 and 2
2. Reconfigure, and construct as necessary, the dog and intruder proof fence along access road and around the service compound to prevent site access and access to the beach.
3. Construct otter boundary fence for the Phase 2 area.
4. Strip topsoil from Phase 3 and stockpile in Phase 4 area.
5. Import and consolidate fill in Phase 2 area.
6. Construct Phase 2 and as needed utilise topsoil from stockpile in Phase 4 area and import additional topsoil as needed.
7. Replace the temporary siltation pond in Phase 2 with the permanent attenuation store, Attn. No. 3, and related silt trap and oil/petrol interceptor when most of the construction is completed and the danger of siltation of the store has passed.
8. Remove berm in Phase 2, complete path and landscape

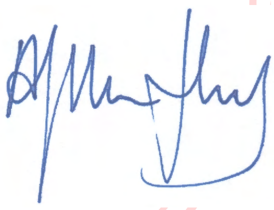
Phase 3 Overview of Ecology Related Measures.

1. Construct a site security fence on the boundary between Phases 2 and 3.
2. Clear scrub from the remainder of Phase 3 and from the services compound for Phase 3 and the stockpile area in Phase 4..
3. Reconfigure and construct as necessary the dog and intruder proof fence along access road and around the service compound to prevent sPHASE 3 Preparatory Work.
4. Construct a site security fence on the boundary between Phases 2 and 3
5. Clear scrub from the remainder of Phase 3 and from the services compound for Phase 3 and the stockpile area in Phase 4..
6. Reconfigure and construct as necessary the dog and intruder proof fence along access road and around the service compound to prevent site access and access to the beach.
7. Construct otter boundary fence for the Phase 3 area.
8. Strip topsoil from Phase 4 service compound and stockpile in Phase 4 area.
9. Import and consolidate fill in Phase 3 area.
10. Construct Phase 3 and as needed utilise topsoil from stockpile in Phase 4 area and import additional topsoil as needed.

11. Replace the temporary siltation pond in Phase 3 with the permanent attenuation store are related silt trap and oil/petrol interceptor when most of the construction is completed and the danger of siltation of the store has passed.
12. Remove berm in Phase 3, complete path and landscape

Phase 4 Overview of Ecology Related Measures.

1. Construct a site security fence on the boundary between Phases 3 and 4.
2. Clear scrub from the remainder of Phase 4.
3. Construct otter boundary fence for the Phase 4 area.
4. Construct the service compound at the location shown. Modification and relocation will be necessary in the later stages. Construction of the buildings closest to the access bridge will take place last and plant and service will be reduced and relocated as necessary in the latter stages.
5. Import and consolidate fill in the local low areas of Phase 4.
6. Construct Phase 4 and import topsoil as needed.
7. Replace the temporary siltation pond in Phase 4 with the permanent attenuation store are related silt trap and oil/petrol interceptor when most of the construction is completed and the danger of siltation of the store has passed.
8. Remove berm in Phase 4, complete path and landscape.



Digitally signed by Arthur Murphy
DN: cn=Arthur Murphy, o=Arthur Murphy and Co, ou, email=arthur@ameng.ie, c=IE
Date: 2020.08.11 11:08:52 +01'00'

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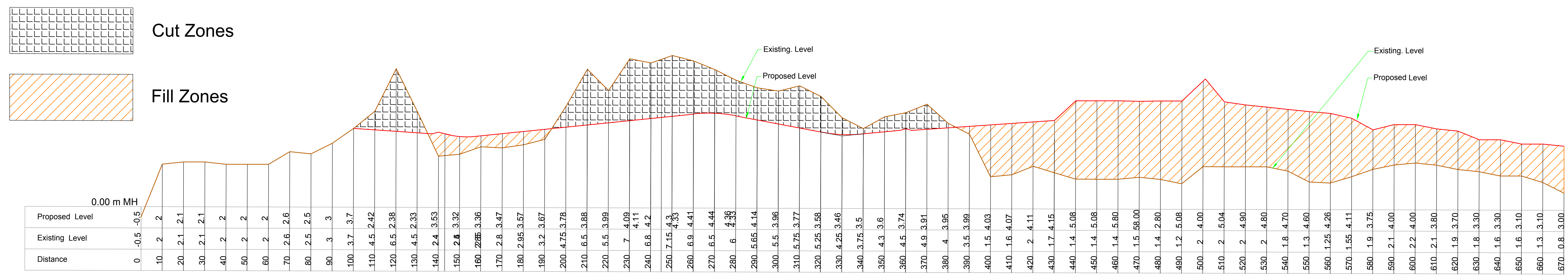
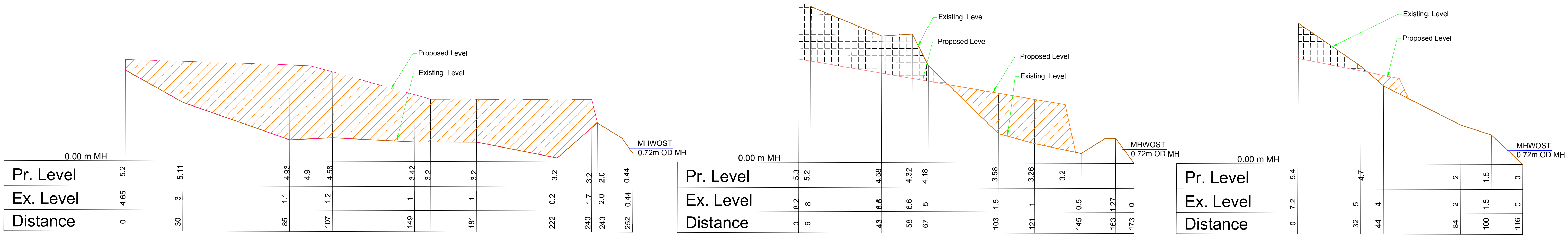
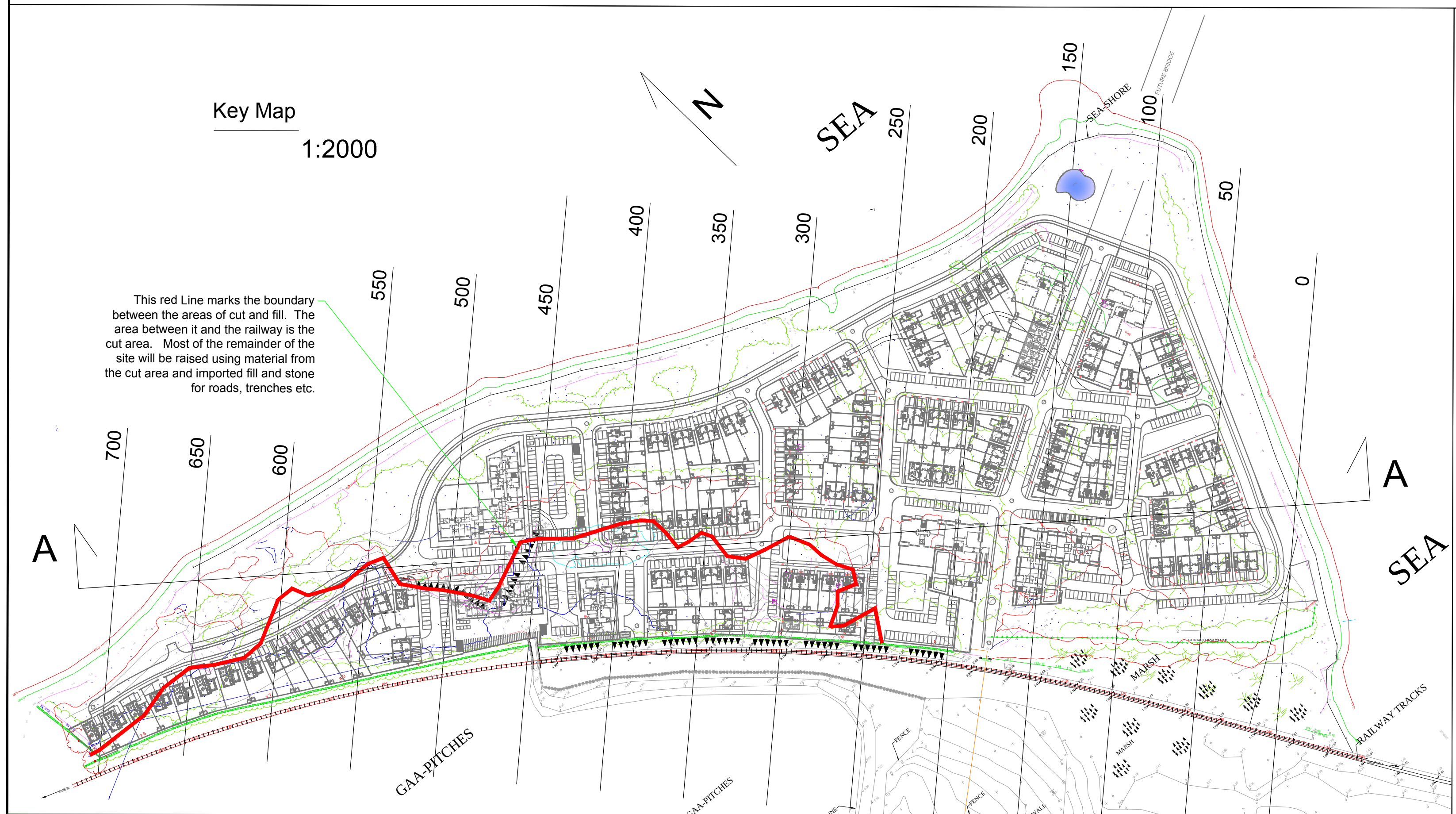
Arthur Murphy B.E., M.Eng.Sc., C.Eng.



Long Section AA Through Site 1:1000H 1:100V



Cross-Setion at 200 1:1000H 1:100V Cross-Section at 400 1:1000H 1:100V Cross-Section at 600



See Appendix C of the Engineering Report document for details of:

1. the cut and fill quantities
2. the nett volume of imported fill
3. a description of the fill
4. management of the fill
5. protection of the otter reserved area and
6. the settlement of site runoff to avoid silt gaining access to the estuary.

ALL DIMENSIONS ARE TO BE CHECKED ON SITE BEFORE COMMENCING AND AT ALL STAGES OF CONSTRUCTION

Do Not Scale. Check for reduction/increase in plotting size

Arthur Murphy & Co.
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Tel: 051 565 565
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Client: William Neville & Sons Ltd. Rockfield House Spawell Road, Wexford

No	Revision Description	Date	By

Project: RESIDENTIAL DEVELOPMENT PARK WEXFORD

Sub Project: Civil Engineering Drawings

Title: SITE CUT AND FILL

First Issue Date:

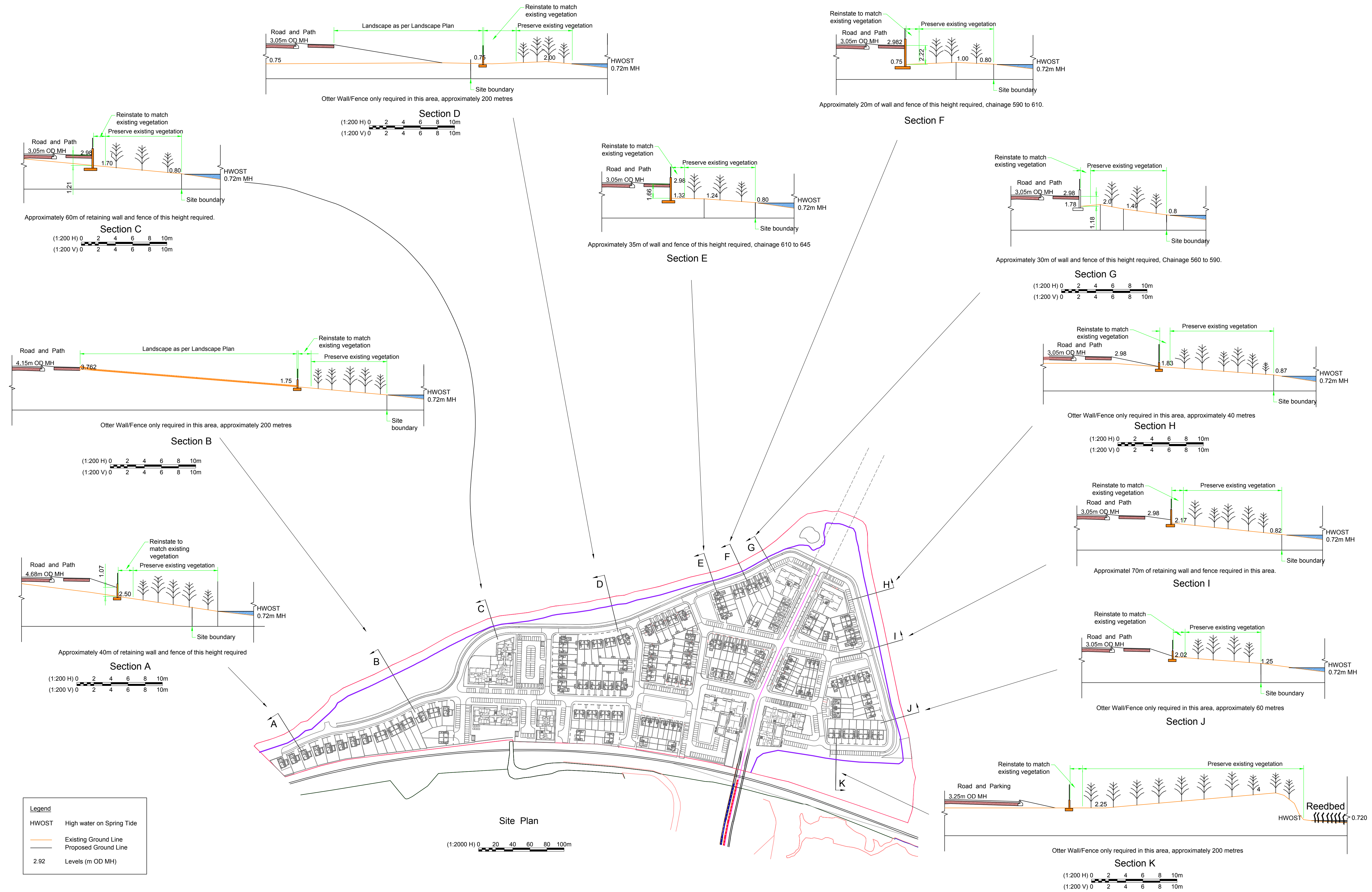
Design: AM

Scale: As Shown

Drawing No.: PL 10

Revision:

Status: Planning



ALL DIMENSIONS ARE TO BE CHECKED ON SITE BEFORE COMMENCING AND AT ALL STAGES OF CONSTRUCTION

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Client
William Neville & Sons Ltd.
Rockfield House
Spawell Road, Wexford

No	Revision Description	Date	By

Project
RESIDENTIAL DEVELOPMENT
PARK
WEXFORD

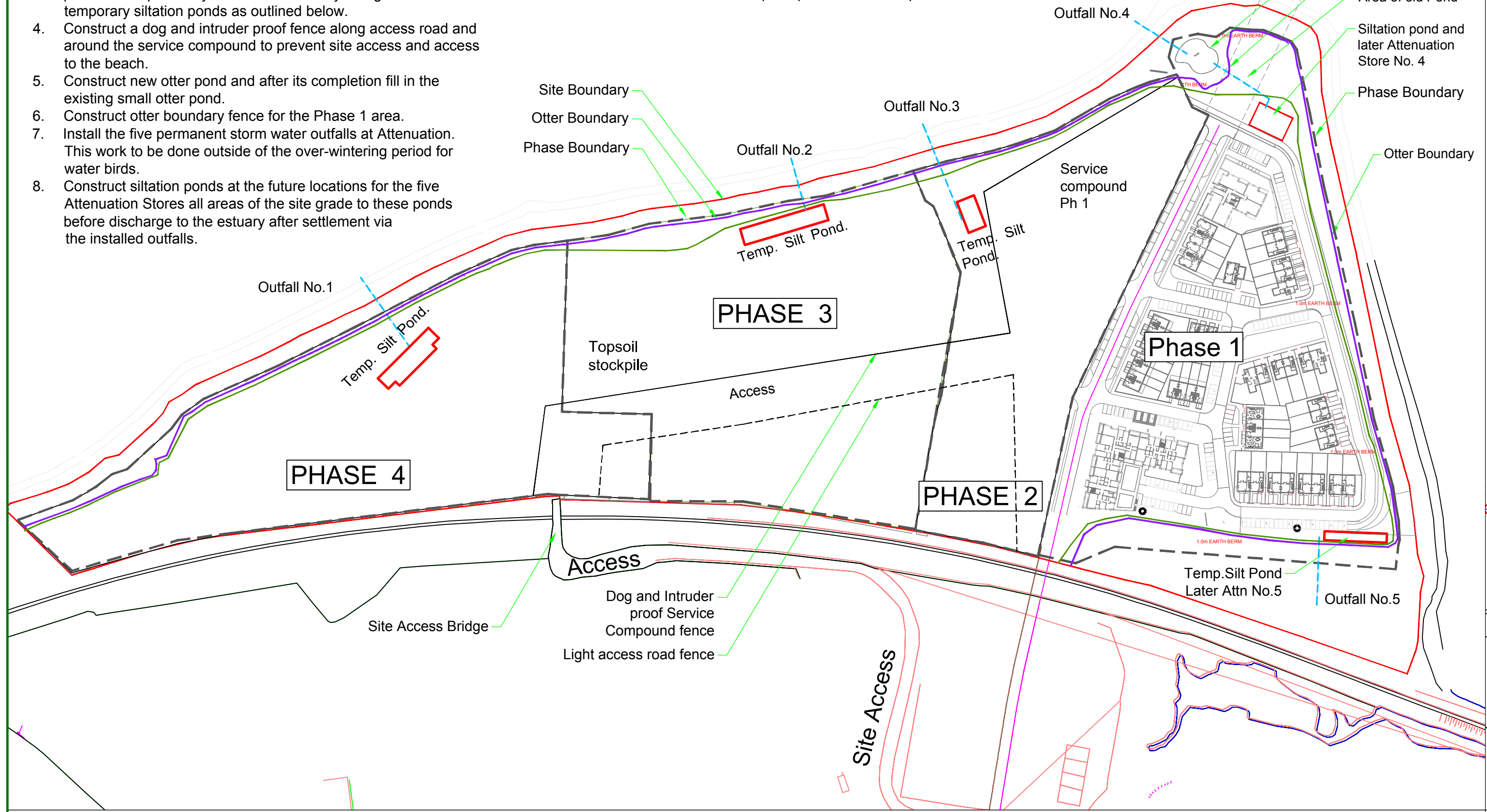
Sub Project
Civil Engineering Drawings

Title
SHORELINE SECTIONS

First Issue Date	Design	Scale
July 2020	AM	As Shown
Drawing No.	**Revision**	**Status**
PL 11		Planning

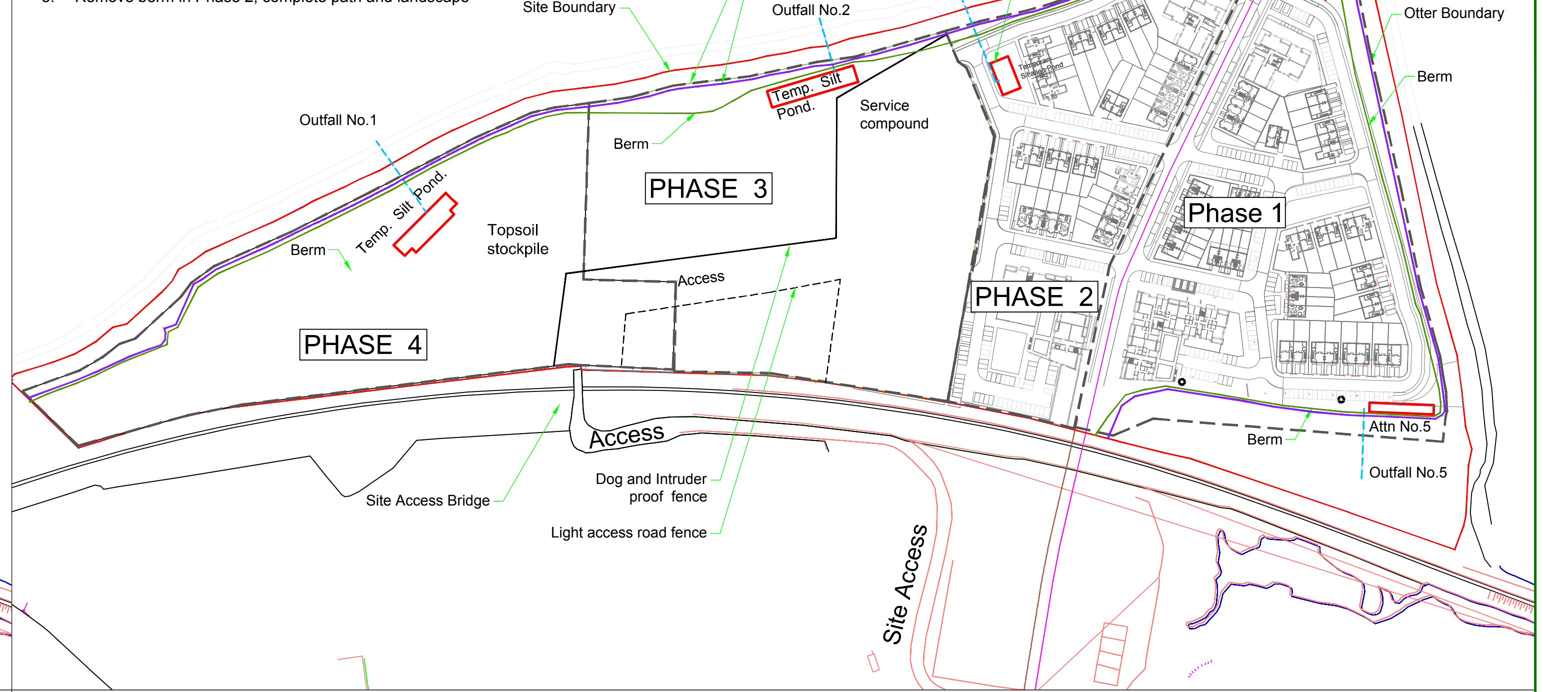
PHASE 1 Preparatory Work.

1. Construct new otter pond 6 months before commencing the main development.
2. After confirmation that otters are using the it fill in the existing small otter pond.
3. Clear the line of the proposed berm for the full length of the berm and construct a 1 metre high berm with a top width of 1m and 1 in 3 side slopes on the line shown for the full extent of the site to prevent escape of silty water to the estuary and guide it to temporary siltation ponds as outlined below.
4. Construct a dog and intruder proof fence along access road and around the service compound to prevent site access and access to the beach.
5. Construct new otter pond and after its completion fill in the existing small otter pond.
6. Construct otter boundary fence for the Phase 1 area.
7. Install the five permanent storm water outfalls at Attenuation. This work to be done outside of the over-wintering period for water birds.
8. Construct siltation ponds at the future locations for the five Attenuation Stores all areas of the site grade to these ponds before discharge to the estuary after settlement via the installed outfalls.
9. Strip topsoil from Phase 1 and 2 areas and stockpile in Phase 3 area.
10. Import and consolidate fill in Phase 1 area.
11. Construct Phase 1 and as needed utilise topsoil from stockpile in Phase 3 area.
12. Replace the temporary siltation ponds in Phase 1 with the permanent attenuation stores and related silt traps and oil/petrol interceptors when most of the construction is completed and the danger of siltation of the stores has passed.
9. Remove berm in Phase 1, complete path and landscape.



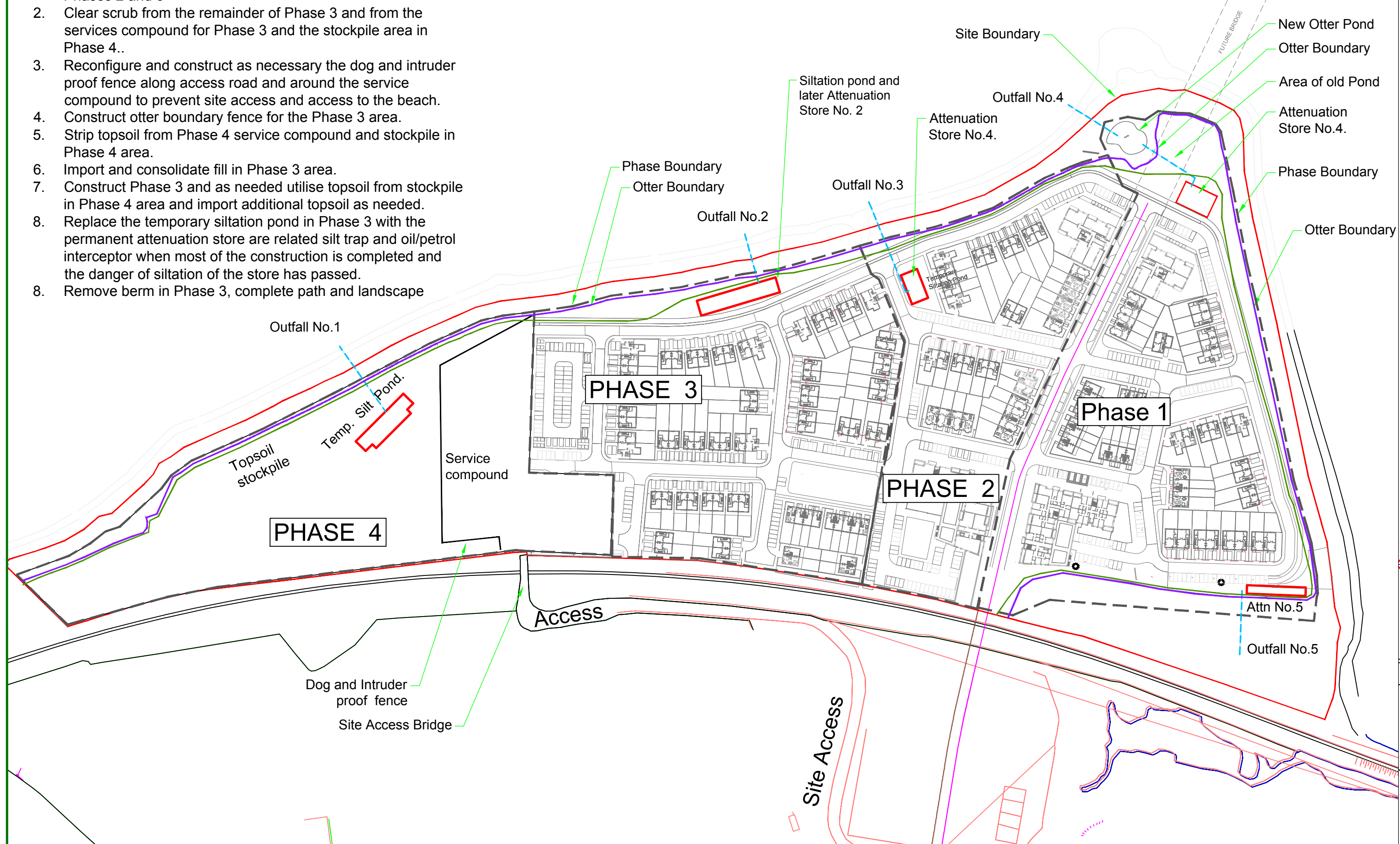
PHASE 2 Preparatory Work.

1. Construct a site security fence on the boundary between Phases 1 and 2
2. Reconfigure, and construct as necessary, the dog and intruder proof fence along access road and around the service compound to prevent site access and access to the beach.
3. Construct otter boundary fence for the Phase 2 area.
4. Strip topsoil from Phase 3 and stockpile in Phase 4 area.
5. Import and consolidate fill in Phase 2 area.
6. Construct Phase 2 and as needed utilise topsoil from stockpile in Phase 4 area and import additional topsoil as needed.
7. Replace the temporary siltation pond in Phase 2 with the permanent attenuation store, Attn. No. 3, and related silt trap and oil/petrol interceptor when most of the construction is completed and the danger of siltation of the store has passed.
8. Remove berm in Phase 2, complete path and landscape



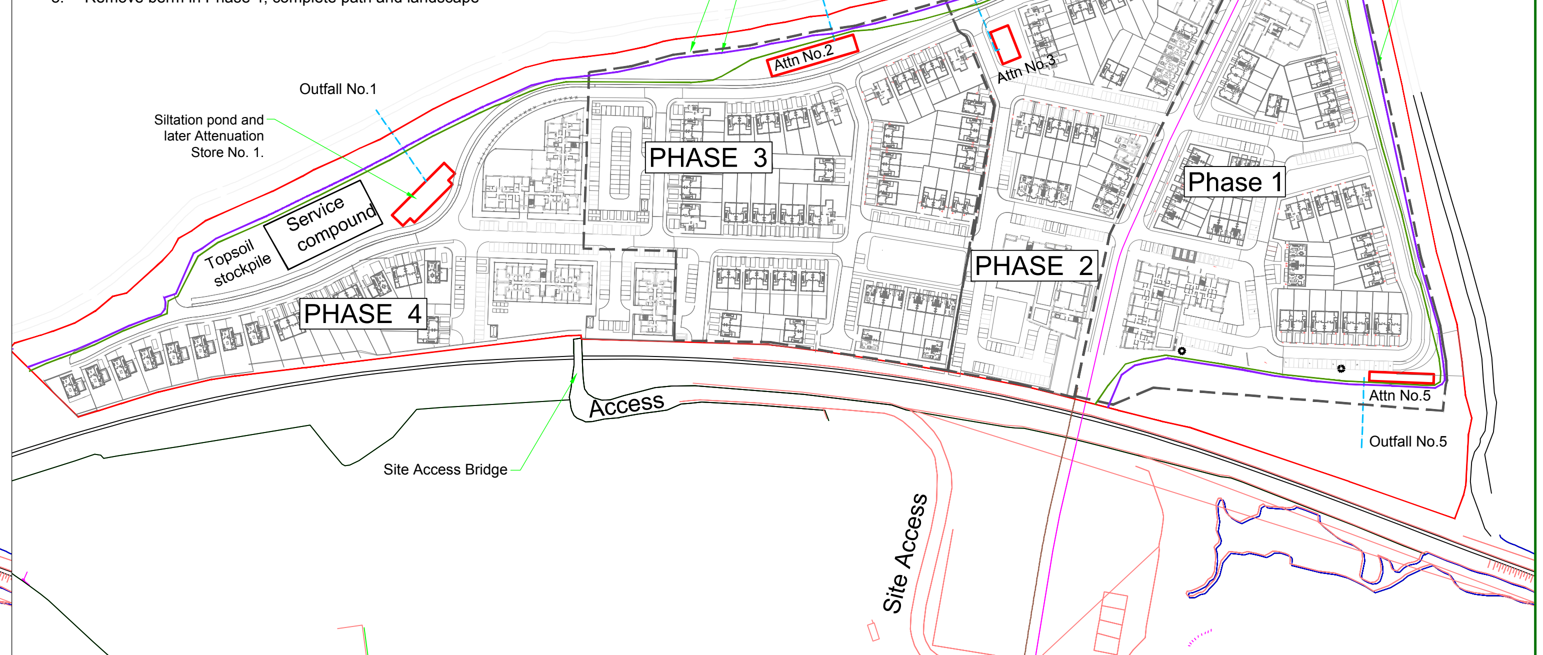
PHASE 3 Preparatory Work.

1. Construct a site security fence on the boundary between Phases 2 and 3
2. Clear scrub from the remainder of Phase 3 and from the services compound for Phase 3 and the stockpile area in Phase 4..
3. Reconfigure and construct as necessary the dog and intruder proof fence along access road and around the service compound to prevent site access and access to the beach.
4. Construct otter boundary fence for the Phase 3 area.
5. Strip topsoil from Phase 4 service compound and stockpile in Phase 4 area.
6. Import and consolidate fill in Phase 3 area.
7. Construct Phase 3 and as needed utilise topsoil from stockpile in Phase 4 area and import additional topsoil as needed.
8. Replace the temporary siltation pond in Phase 3 with the permanent attenuation store are related silt trap and oil/petrol interceptor when most of the construction is completed and the danger of siltation of the store has passed.
8. Remove berm in Phase 3, complete path and landscape



PHASE 4 Preparatory Work.

1. Construct a site security fence on the boundary between Phases 3 and 4.
2. Clear scrub from the remainder of Phase 4.
3. Construct otter boundary fence for the Phase 4 area.
4. Construct the service compound at the location shown. Modification and relocation will be necessary in the later stages. Construction of the buildings closest to the access bridge will take place last and plant and service will be reduced and relocated as necessary in the latter stages.
5. Import and consolidate fill in the local low areas of Phase 4.
6. Construct Phase 4 and import topsoil as needed.
8. Replace the temporary siltation pond in Phase 4 with the permanent attenuation store are related silt trap and oil/petrol interceptor when most of the construction is completed and the danger of siltation of the store has passed.
8. Remove berm in Phase 4, complete path and landscape



ALL DIMENSIONS ARE TO BE CHECKED ON SITE BEFORE COMMENCING AND AT ALL STAGES OF CONSTRUCTION

Do Not Scale. Check for reduction/increase in plotting size

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Client: William Neville & Sons Ltd. Rockfield House Spawell Road, Wexford

No	Revision Description	Date	By

Project: RESIDENTIAL DEVELOPMENT PARK WEXFORD

Sub Project: Civil Engineering Drawings

Title: CONSTRUCTION MANAGEMENT OVERVIEW

First Issue Date	Design	Scale
05/08/2020	AM	1:2000 on A1
Drawing No.	**Revision**	**Status**
PL 12		Planning

Appendix D

Report on Management Future Monitoring

and

Mitigation of Gas Emissions

from

Carcur Landfill Site.

July 31st, 2020.

Appendix D

July 31st, 2020.

Report on Management, Future Monitoring and Mitigation of Gas Emissions from Carcur Landfill Site.

This report is a bringing together of proposals submitted with the 2019 planning application and the refusal of that permission was not based in whole or part on them.

7.3.1 POSSIBLE GAS MIGRATION FROM CARCUR LANDFILL SITE.

7.3.1.1 Landfill Location and History

A land fill site was operated at Carcur south of the railway and largely east of the proposed development site during the mid-twentieth century. The landfill was closed in 1985, 33 years ago now. The closest edge of waste placement in the landfill is 130 metres away from the nearest proposed housing within the development. The development is separated from the landfill by the railway line and by tidal marshes on each side of the railway. This level of separation and the fine and waterlogged nature of the silts in the tidal zone almost certainly prevent gas from the landfill from reaching any dwellings in the proposed development.

7.3.1.2 Monitoring of Gas Levels

Wexford County Council is monitoring the gas levels within the landfill site. As part of the preparation of this planning application 2 gas monitoring wells have been installed by the developer within the development site adjacent to the landfill to assist in determining whether there is any migration of gases under the railway and the intervening mudflats. The positions of the monitoring wells were agreed with Wexford Council and are shown on Arthur Murphy & Co's Drawing PL 01.

An initial set of readings indicated the presence of low levels of methane were present (0.3 to 0.4%). These levels are almost certainly background levels rather than indicating migration from the landfill site. This is as might be expected as the monitoring wells are more than 120 from the edge of the actual landfilled area and the railway and an area of estuarine silt are located in this zone. The silt in particular acts as a barrier to horizontal gas migration into the development site. However due to the need to take a conservative approach with this type of risk, it is proposed to continue monitoring the gas levels before, during and after construction. This will establish a detailed picture of the levels and variations in levels of methane and possible migration routes for the gas.

The Dept. of Environments 'Protection of New Buildings and Occupants from Landfill Gas', published in 1999 recommends that sites within 250m of landfill sites that were used within the last 30 years should be assessed for landfill gas. The Carcur landfill was closed in 1985, 35 years ago, and before any houses are occupied further time will have elapsed.

Nevertheless, it is proposed to continue monitoring the gas levels before, during and after construction to ensure that this conclusion is valid and that there is no unforeseen risk to the development. The results from this further monitoring will determine whether there is a need to take specific measures to protect housing within the development and the nature and extent of any measures that may be advisable.

7.3.1.3 Remedial Measures

Should monitoring indicate that gas migration is occurring it is proposed to finalise the measures to be employed and their areal extent in conjunction with the County Council, and to their final approval, before construction commences on site.

This approach has been agreed with Wexford Council.

The gas remediation and protection measures will be developed and overseen on the development team side by a Chartered Engineer experienced in this work and a list of such experienced consultants has been provided to the applicant by Wexford County Council.

The measures and their extent will be based on the Council's own monitoring of their landfill site and the further monitoring for landfill gas within the proposed development site. A range of protection measures are outlined in Dept. of Environments 'Protection of New Buildings and Occupants from Landfill Gas', published in 1999. The measures to be adopted will comply appropriately with these.

It is proposed to finalise the measures to be employed and their areal extent in conjunction with the County Council, and to their final approval, before construction commences on site. This will be based on the Council's findings and the further monitoring of the gas on the proposed development site. This approach has been agreed with Wexford County Council.

An additional measure that can be considered would be to install an open textured rock filled trench at an agreed location and to an agreed extent to act as a cut-off trench. In view of the existence of the silt barrier mentioned above this trench is not likely to be required but is available as an option should the need arise.

In addition the standard radon barrier in dwellings will be upgraded and the buildings underlain with 200mm of granular fill vented to the open air for houses in any part of the site deemed to be at risk. The areal extent of the site requiring this extra protection will also be agreed with Wexford County Council but all habitations within 250 metres of the landfilled area will have this installed.

7.3.1.4 Residual Impact

The development will have no impact on the landfill and more importantly gas migration from the landfill site is not expected to occur and should gas migration be detected the measures developed and implemented as set out above will ensure that there will be no impact on the buildings and occupants of the development.

Digitally signed by Arthur Murphy
DN: cn=Arthur Murphy, o=Arthur Murphy and Co, ou, email=arthur@ameng.ie, c=IE
Date: 2020.08.11 11:10:07 +01'00'

Arthur Murphy B.E., M.Eng.Sc., C.Eng.

Appendix E

Irish Water Approval and Documents Submitted for that Approval

1. <u>A Statement of Design Acceptance from Irish Water for the Water and Wastewater</u>
2. Calculations of water demand for the project.
3. A mathematical modelling of the water pressure in the water supply network.
4. Pump Hydraulics at one third development completed
5. Pump Hydraulics at Full Development Completed.
6. Wastewater Pumping System Residence Time Calculations
7. Minimum Sump Volume Calculations Under Partial Development
8. Minimum Sump Volume Calculations at Full Development
9. Emergency Storage Volume Calculation
10. Ballast Calculations for Pump Station and Emergency Storage Tank
11. Pump and related equipment specifications from Xylem for Flygt pumps.

Arthur Murphy
Arthur Murphy & Co Consulting Civil and Structural Engineering
Garryrichard,
Foulksmills,
Co. Wexford,
Ireland

UISCE ÉIREANN : IRISH WATER

Uisce Eireann
Bosca OP 860
Oifig Sheachadta
na Cathrach Theas
Cathair Chorcaí

Irish Water
PO Box 860
South City
Delivery Office
Cork City

www.water.ie

22nd November 2018

Dear Mr Murphy

Re: CUST165252 pre-connection enquiry – Subject to contract | Contract denied
Proposed development at Park, Carcur, Wexford.

Irish Water has reviewed your water services designs that were submitted on the 14/11/2018 in relation to water and wastewater connections at Park, Carcur, Wexford and confirm that the proposed water services designs comply with the Irish Water Standard details and codes of practice.

Prior to any works commencing on site, the applicant shall enter into a connection agreement with Irish Water.

You are advised that this correspondence does not constitute an offer in whole or in part to provide a connection to any Irish Water infrastructure and is provided subject to a connection agreement being signed at a later date.

A connection agreement can be applied for by completing the connection application form available at **www.water.ie/connections**. Irish Water's current charges for water and wastewater connections are set out in the Water Charges Plan as approved by the Commission for Energy Regulation.

If you have any further questions, please contact Maurice Feehan from the design team on 022 52284 or email maufeehan@water.ie. For further information, visit **www.water.ie/connections**

Yours sincerely,

Maria O'Dwyer

Connections and Developer Services

Proposed Residential at Park, Wexford by Wm Neville & Sons Ltd

Calculation of Water Demand and Sewage Flows

Full Development

	No of Units	Average occupancy	Daily Vol at 180 l/p/d		
No of 4 bedroom houses	36	5	32,400	l/d	
No of 3 bedroom houses	115	4	82,800		
No of 2 bedroom houses	26	3	14,040		
No of 1 bedroom apartments	15	2	5,400		
No of 2 bedroom apartments	198	3	106,920		
No of 3 bedroom apartments	23	4	16,560		
Total No of Units	413				
Creche (40 Lpd)	60	40	2,400		
Estimated average daily flow		Q_d =	260,520	l/day	= $\sum$(e6:e11)
Percentage of site developed		D_p =	100	%	
At this stage flow is		Q_{da} =	260,520	l/day	= Q_d * D_p / 100
Average flow rate		Q_A =	3.0	l/s	= Q_{da} / 24 / 3600
A peak to average ration of 4 is proposed		R_p =	3		
Estimated maximum flow rate per second		Q_{max} =	9.0	l/s	= R_p * Q_A
Ratio of Normal to average demand		R_{NA} =	2		
Normal water demand		Q_N =	6.0	l/s	

Phasing	Units	Average	Maximum	Normal	
Phase 1	120	0.9	2.6	1.7	
Phase 1 and 2	220	1.6	4.7	3.1	
Phase 1, 2 and 3	295	2.1	6.3	4.2	
Full development	425	3.0	9.0	6.0	

Fire Fighting Requirements					
Fire flow required by Wexford		Q_F =	1,200	litres/min	
		Q_f =	20	l/s	= Q_F / 60

An explicit and conservative mathematical modelling of the pressure in the water supply network based on a 20 l/s fire demand at the most remote point of the site concurrent with 3 times DWF is presented in A4 form herewith.

Proposed Park Development by Wm Neville & Sons Ltd

Review of Watermain Capacities.

This calculation gives the water pressure at key points in the distribution under the following conditions:

Existing Water Pressure at Connection Point at Carcur Roundabout 5.5 bar

Fire flow requirement of 1200 litres per minute or 20 litres per second at the most remote hydrant to west of site.

Concurrent daytime flow of 3 times average flow, that is 3 x 2.97 or 8.91 litres per second.

Peak demand while concurrently providing 20 litres per second is therefore **29 litres per second**.

Friction losses calculated using the Hazen Williams formula. Hazen Williams C 140

$$S = \frac{h_f}{L} = \frac{10.67\, Q^{1.852}}{C^{1.852}\, d^{4.8704}}$$

	Design Flow	Giving Q	Pipe Dia	Giving d	Pipe Length	Friction Loss	D/S Pressure	D/S Presure	D/S Presure
	l/s	m^3/s	mm	m	(L in metres)	h_f	m water	bar	psi
Effect of fire fighting flows on water pressures at Carcur Roundabout									
Present Water Pressure at Carcur Road roundabout while flow in 300mm main from Newtown Reservoir is 170 m^3/hour or 47 l/s							56	5.5	80
Pre Park development flows									
Newtown Road reservoir to Wex. Gen. Hospital	47	0.047	300	0.3	1200	1.7			
From the Hospital to Carcur roundabout the flow is carried in 2 main pipes and the present flow of approx **10 l/s** is split as follows.									
Newtown Road to Carcur Roundabout 180 mm pipe	6.5	0.0065	180	0.18	880	0.4			
Newtown Road to Carcur Roundabout 150 mm pipe	3.5	0.0035	150	0.15	1120	0.4			
Total flow split between 2 pipes	**10**								
Total head loss from reservoir to Carcur roundabout pre development						2.0			
Post Park development flows during fire fighting									
Newtown Road reservoir to Wex. Gen. Hospital	76	0.076	300	0.3	1200	4.0			
From the Hospital to Carcur roundabout the flow is carried in 2 main pipes and the present flow of approx **39 l/s** is split as follows.									
Newtown Road to Carcur Roundabout 180 mm pipe	25.25	0.02525	180	0.18	880	4.6			
Newtown Road to Carcur Roundabout 150 mm pipe	13.75	0.01375	150	0.15	1120	4.7			
Total flow split between 2 pipes	**39**								
Total head loss from reservoir to Carcur roundabout pre development						8.7			
Additional head loss caused by development demand while supplying 20 litres per second for fire fighting						6.7			
Resultant Water Pressure at Carcur Road roundabout							49.4	4.8	70

Assuming an increased flow in the Newtown Road 300mm pipe of 100 litres per second this becomes

	Design Flow	Giving Q	Pipe Dia	Giving d	Pipe Length	Friction Loss	D/S Pressure	D/S Presure	D/S Presure
	l/s	m^3/s	mm	m	(L in metres)	h_f	m water	bar	psi
Present Water Pressure at Carcur Road roundabout while flow in 300mm main from Newtown Reservoir is 170 m^3/hour or 47 l/s							56	5.5	80
Pre Park development flows									
Newtown Road reservoir to Wex. Gen. Hospital	100	0.1	300	0.3	1200	6.7			
From the Hospital to Carcur roundabout the flow is carried in 2 main pipes and the present flow of approx **10 l/s** is split as follows.									
Newtown Road to Carcur Roundabout 180 mm pipe	6.5	0.0065	180	0.18	880	0.4			
Newtown Road to Carcur Roundabout 150 mm pipe	3.5	0.0035	150	0.15	1120	0.4			
Total flow split between 2 pipes	**10**								
Total head loss from reservoir to Carcur roundabout pre development						7.1			
Post Park development flows during fire fighting									
Newtown Road reservoir to Wex. Gen. Hospital	129	0.129	300	0.3	1200	10.8			
From the Hospital to Carcur roundabout the flow is carried in 2 main pipes and the present flow of approx **39 l/s** is split as follows.									
Newtown Road to Carcur Roundabout 180 mm pipe	25.25	0.02525	180	0.18	880	4.6			
Newtown Road to Carcur Roundabout 150 mm pipe	13.75	0.01375	150	0.15	1120	4.7			
Total flow split between 2 pipes	**39**								
Total head loss from reservoir to Carcur roundabout pre development						15.4			
Additional head loss caused by development demand while supplying 20 litres per second for fire fighting						8.3			
Resultant Water Pressure at Carcur Road roundabout							47.8	4.7	68

Using this more stringent condition as a starting point for water pressure at Carcur Roundabout gives the following flows and pressures in the development.

	Design Flow	Giving Q	Pipe Dia	Giving d	Pipe Length	Friction Loss	D/S Pressure	D/S Presure	D/S Presure
	l/s	m^3/s	mm	m	(L in metres)	h_f	m water	bar	psi
Connection at Existing Roundabout							47.8	4.7	68
Rounabout to Railway Bridge (Fire flow of 20 l/s + residential flow of 8.91 l/s)	29	0.029	150	0.15	650	10.8	37.0	3.6	53
Railway Bridge to Junction 17 (Fire flow of 20 l/s + reduced residential flow of 6.5 l/s)	29	0.029	150	0.15	90	1.5	35.5	3.5	51
Junction 17 to Junction 14 (Fire flow of 20 l/s + reduced residential flow of 6.5 l/s)	26.5	0.0265	150	0.15	160	2.2	33.3	3.3	47
Junction 14 to Junction 10 (Fire flow of 20 l/s + reduced residential flow of 4 l/s)	24	0.024	150	0.15	140	1.6	31.7	3.1	45
Junction 10 to Junction 9 (Fire flow of 20 l/s + reduced residential flow of 2 l/s)	22	0.022	150	0.15	80	0.8	30.9	3.0	44
Junction 9 to most westerly hydrant (Fire flow of 20 l/s + reduced residential flow of 1 l/s)	21	0.021	100	0.1	170	11.1	**19.7**	**1.9**	**28**

This analysis shows that under higher than present flows in the Newtown Road that the water pressure during a fire fighting episode that the water pressure is still satisfactory at the very extreme end of the site. The analysis itself is also conservative as there are additional routes for the water both before and within the development which will enhance the water pressures.

Wm Neville & Sons Ltd - Development at Park Wexford at Full Occupancy					
		Average occupancy	Daily Vol at 180 l/p/d		
No of 4 bedroom units	36	5	32,400	l/d	
No of 3 bedroom houses	115	4	82,800		
No of 2 bedroom houses	26	3	14,040		
No of 1 bedroom apartments	15	2	5,400		
No of 2 bedroom apartments	198	3	106,920		
No of 3 bedroom apartments	23	4	16,560		
Creche (40 Lpd)	60	40	2,400		
Estimated average daily flow		Q_{max} =	260,520	l/day	
Percentage of site developed		Dp =	100	%	
At this stage flow is		Q_{da} =	260,520	l/day	$= Q_{max} * D_p / 100$
Average flow rate			3.02		$= Q_{da} / 24 / 3600$
A peak to average ratio of 4 is proposed		R_p =	4		
Estimated maximum flow rate per second		Q_{max} =	12.1	l/s	$= Q_{da} * R_p / 24 / 3600$
Use a pumping rate of		Q_p =	13.3	l/s	$= Q_{max} * 1.1$
Ground level at pump station		L_{gr} =	2.80	m	
Level at pump cut off		L_{co} =	-3.00	m	
Level at top of bridge		L_{br} =	9.00	m	
Level of discharge into gravity sewer		L_{dis} =	0.00	m	
Length of pumping main in the site		D_s =	260.00	m	
Length off site		D_{os} =	600.00	m	
Total pipe length		D_{tot} =	860.00	m	$= D_{os} + D_s$
Manning n		n =	0.011		
Pipe diameter (internal)		ϕ =	150	mm	
Pipe area		A_p =	0.0177	m^2	$= 3.142 * (\phi / 1000)^2 / 4$
Velocity		v =	0.751	m/s	$= Q_p / 1000 / A_p$
Hydraulic R		R =	0.038	m	$= \phi / 4000$
R (2/3)			0.112	m	$= R^{0.667}$
Friction slope		s =	0.005		$= ([v * n] / R^{0.667})^2$
		s_f =	0.54	%	$= s * 100$
Head Loss from pump to bridge		F_{br} =	1.42	m	$= s_f * D_s / 100$
Head loss from bridge onwards		F_{onw} =	3.27	m	$= s_f * D_{os} / 100$
Total Head Loss due to friction		F_{tot} =	4.68	m	$= s_f * D_{tot} / 100$
Static Head to discharge point		H_d =	3	m	$= L_{dis} - L_{co}$
Static Head to top of bridge		H_{br} =	9.00	m	$= L_{br} - L_{co}$
TDH to discharge point			8	m	$= F_{tot} + H_d$
TDH to bridge			10	m	$= H_{br} + F_{br}$

Wm Neville & Sons Ltd - Development at Park Wexford at 27% Occupancy					
		Average occupancy	Daily Vol at 180 l/p/d		
No of 4 bedroom units	36	5	32,400	l/d	
No of 3 bedroom houses	115	4	82,800		
No of 2 bedroom houses	26	3	14,040		
No of 1 bedroom apartments	15	2	5,400		
No of 2 bedroom apartments	198	3	106,920		
No of 3 bedroom apartments	23	4	16,560		
Creche (40 Lpd)	60	40	2,400		
Estimated average daily flow		Q_{max} =	260,520	l/day	
Percentage of site developed		Dp =	27	%	
At this stage flow is		Q_{da} =	69,038	l/day	$= Q_{max} * D_p / 100$
Average flow rate			0.80		$= Q_{da} / 24 / 3600$
A peak to average ratio of 4 is proposed		R_p =	4		
Estimated maximum flow rate per second		Q_{max} =	3.2	l/s	$= Q_{da} * R_p / 24 / 3600$
Use a pumping rate of		Q_p =	3.5	l/s	$= Q_{max} * 1.1$
Ground level at pump station		L_{gr} =	2.80	m	
Level at pump cut off		L_{co} =	-3.00	m	
Level at top of bridge		L_{br} =	9.00	m	
Level of discharge into gravity sewer		L_{dis} =	0.00	m	
Length of pumping main in the site		D_s =	260.00	m	
Length off site		D_{os} =	600.00	m	
Total pipe length		D_{tot} =	860.00	m	$= D_{os} + D_s$
Manning n		n =	0.011		
Pipe diameter (internal)		ϕ =	80	mm	
Pipe area		A_p =	0.0050	m^2	$= 3.142 * (\phi / 1000)^2 / 4$
Velocity		v =	0.699	m/s	$= Q_p / 1000 / A_p$
Hydraulic R		R =	0.020	m	$= \phi / 4000$
R (2/3)			0.074	m	$= R^{0.667}$
Friction slope		s =	0.011		$= ([v * n] / R^{0.667})^2$
		s_f =	1.09	%	$= s * 100$
Head Loss from pump to bridge		F_{br} =	2.84	m	$= s_f * D_s / 100$
Head loss from bridge onwards		F_{onw} =	6.56	m	$= s_f * D_{os} / 100$
Total Head Loss due to friction		F_{tot} =	9.40	m	$= s_f * D_{tot} / 100$
Static Head to discharge point		H_d =	3	m	$= L_{dis} - L_{co}$
Static Head to top of bridge		H_{br} =	9.00	m	$= L_{br} - L_{co}$
TDH to discharge point			12	m	$= F_{tot} + H_d$
TDH to bridge			12	m	$= H_{br} + F_{br}$

Wm Neville & Sons Ltd - Development at Park Wexford					
Pump Hydraulics at One Third Development Complete					
	No of Units	Persons Per unit	Vol at 150 l/p/d		
No of 4 bedroom houses	36	3	16,200	l/d	
No of 3 bedroom houses	115	2.7	46,575	l/d	
No of 2 bedroom houses	26	2.7	10,530	l/d	
No of 1 bedroom apartments	15	2	4,500	l/d	
No of 2 bedroom apartments	198	2.7	80,190	l/d	
No of 3 bedroom apartments	23	2.7	9,315	l/d	
Total Units	413				
Creche (40 Lpd)	71	40	2,840	l/d	
Estimated average daily flow		Q_{dwf} =	170,150	l/d	= $\sum$(e6:e13)
Percentage of site developed		D_p =	30	%	
At this stage the average flow is		Q_{da} =	51,045	l/day	= Q_{dwf} * D_p / 100
Average flow rate			0.59	l/s	= Q_{da} / 24 / 3600
Peak to average ratio for this stage		R_p =	6		= if(D_p<80 , 6 , 5)
Estimated maximum flow rate per second		Q_{max} =	3.5	l/s	= Q_{da} * R_p / 24 / 3600
HPPE Size			90.0		
SDR 17 Wall thickness			3.8		
Pipe diameter (internal)		ϕ =	83	mm	
Self cleansing velocity		V_{SC} =	0.75	m/s	
Pipe area		A_p =	0.005	m^2	= 3.142 * (ϕ / 1000)^2 / 4
Self cleansing flow rate		Q_{sc} =	4.0	l/s	= A_p * V_{SC} * 1000
Use a pumping rate of		Q_p =	4.0	l/s	= max(Q_{sc} , Q_{max})
Ground level at pump station		L_{gr} =	2.8	m	
Level at pump cut off		L_{co} =	-0.8	m	
Level at top of bridge		L_{br} =	9.0	m	
Level of discharge into gravity sewer		L_{dis} =	6.64	m	
Length of pumping main in the site		D_s =	275	m	
Length off site		D_{os} =	440	m	
Total pipe length		D_{tot} =	715	m	= D_{os} + D_s
Manning n		n =	0.011		
Velocity		v =	0.750	m/s	= Q_p / 1000 / A_p
Hydraulic R		R =	0.02	m	= ϕ / 4000
R (2/3)		$R_{2/3}$ =	0.08	m	= $R^{0.667}$
Friction slope		s =	0.01		= ([v * n] / $R^{0.667}$)^2
		s_f =	1.206	%	= s * 100
Head losses in bends and valves		H_{bv} =	0.143	m	= 5 * V_{SC}^2 / (2 * 9.81)
Head Loss from pump to bridge		F_{br} =	3.32	m	= s_f * D_s / 100
Head loss from bridge onwards		F_{onw} =	5.31	m	= s_f * D_{os} / 100
Total Head Loss due to friction		F_{tot} =	8.63	m	= s_f * D_{tot} / 100
Static Head to discharge point		H_d =	7.44	m	= L_{dis} - L_{co}
Static Head to top of bridge		H_{br} =	9	m	
TDH to bridge			12.32	m	= H_{br} + F_{br}
TDH to discharge point			16.07	m	= F_{tot} + H_d

Wm Neville & Sons Ltd - Development at Park Wexford					
Pump Hydraulics at Full Development Complete					
	No of Units	Persons Per unit	Vol at 150 l/p/d		
No of 4 bedroom houses	36	3	16,200	l/d	
No of 3 bedroom houses	115	2.7	46,575	l/d	
No of 2 bedroom houses	26	2.7	10,530	l/d	
No of 1 bedroom apartments	15	2	4,500	l/d	
No of 2 bedroom apartments	198	2.7	80,190	l/d	
No of 3 bedroom apartments	23	2.7	9,315	l/d	
Total Units	413				
Creche (40 Lpd)	71	40	2,840	l/d	
Estimated average daily flow		Q_{dwf} =	170,150	l/d	
Percentage of site developed		D_p =	100	%	
At this stage the average flow is		Q_{da} =	170,150	l/day	$= Q_{dwf} * D_p / 100$
Average flow rate			1.97	l/s	$= Q_{da} / 24 / 3600$
Peak to average ratio for this stage		R_p =	5		= if(D_p<80 , 6 , 5)
Estimated maximum flow rate per second		Q_{max} =	9.8	l/s	$= Q_{da} * R_p / 24 / 3600$
HPPE Size			160.5		= if(D_p>75 , 160.5 , 90)
SDR 17 Wall thickness			10.5		= if(D_p>65 , 10.5 , 3.75)
Pipe diameter (internal)		ϕ =	139.5	mm	
Self cleansing velocity		V_{SC} =	0.75	m/s	
Pipe area		A_p =	0.015	m^2	$= 3.142 * (\phi / 1000)^2 / 4$
Self cleansing flow rate		Q_{sc} =	11.5		$= A_p * V_{SC} * 1000$
Use a pumping rate of		Q_p =	11.5	l/s	= max(Q_{sc} , Q_{max})
Ground level at pump station		L_{gr} =	2.8	m	
Level at pump cut off		L_{co} =	-0.8	m	
Level at top of bridge		L_{br} =	9.0	m	
Level of discharge into gravity sewer		L_{dis} =	6.64	m	
Length of pumping main in the site		D_s =	275	m	
Length off site		D_{os} =	440	m	
Total pipe length		D_{tot} =	715	m	$= D_{os} + D_s$
Manning n		n =	0.011		
Velocity		v =	0.750	m/s	$= Q_p / 1000 / A_p$
Hydraulic R 140 int. dia. pipe		R =	0.035	m	$= \phi / 4000$
R (2/3)		$R_{2/3}$ =	0.107	m	$= R^{0.667}$
Friction slope		s =	0.006		$= ([v * n] / R^{0.667})^2$
		s_f =	0.60	%	= s * 100
Hydraulic R 83 int. dia. pipe		R =	0.021	m	= 83 / 4000
R (2/3)		$R_{2/3}$ =	0.075	m	$= R^{0.667}$
Velocity in 83mm int pipe		V_{83} =	2.144	m/s	$= Q_p / A_p / 1000$
Friction slope		s =	0.098		$= ([V_{83} * n] / R^{0.667})^2$
Length of 83mm int dia pipe		L_{83} =	12	m	
Head loss in 83 pipes			1.174		$= L_{83} * s$
Head losses in bends and valves		H_{bv} =	0.760	m	$= 5 * V_{83}^2 / (2 * 9.81)$
		s_f =	9.78	%	= s * 100
Head Loss from pump to bridge		F_{br} =	1.65	m	$= s_f * D_s / 100$
Head loss from bridge onwards		F_{onw} =	2.63	m	$= s_f * D_{os} / 100$
Total Head Loss due to friction		F_{tot} =	4.28	m	$= s_f * D_{tot} / 100$
Static Head to discharge point		H_d =	7.44	m	$= L_{dis} - L_{co}$
Static Head to top of bridge		H_{br} =	9.00	m	
TDH to bridge			10.65	m	$= H_{br} + F_{br}$
TDH to discharge point			11.72	m	$= F_{tot} + H_d$

Wm Neville & Sons Ltd - Development at Park Wexford

System Residence Time Calculations

Examining residence time at full development					
Estimated average daily flow (dwf)		Q_{dwf} =	170	m^3/d	= Q_{dwf} / 1000
Max. residence time in wet well and main		T_{max} =	6	hours	
Max residence volume at full development		V_{max} =	43	m^3	= Q_{dwf} * T_{max} / 24
Total length of pumping main		L_m =	715	m	= D_{tot}
Volume in HDPE 160 main		V_{160} =	11.0	m^3	= L_m * 3.14 * 0.14^2 / 4
Pumping rate		q_p =	11.5	l/s	
Min. pump run time for 10 starts per hour		T_{pm} =	90	seconds	
Minimum operating volume		V_{om} =	1.04	m^3	= T_{pm} * q_p / 1000
Given a wet well diameter of		W_d =	1.8	m	
Minimum operating depth in wet well is		d_{min} =	0.41	m	= V_{om} / (3.14 * W_d^2 / 4)
Use and operating depth of		d_o =	0.5	m	
Operating Volume used		V_{om} =	1.3	m^3	= V_{om} * d_o / d_{min}
Total Residence volume		V_{tot} =	12.3	m^3	= V_{om} + V_{160}
Is this less than the maximum allowable			YES, OK		= if(V_{tot}<V_{max} , "yes , ok" , "no xxxxxx")
Residence time at partial development of		$\%_{dev}$ =	10	%	
Estimated average daily flow (dwf)		Q_{dwf} =	17	m^3/d	= Q_{dwf} * $\%_{dev}$ / 100
Max. residence time in wet well and main		T_{max} =	6	hours	
Max residence volume at full development		V_{max} =	4.3	m^3	= Q_{dwf} * T_{max} / 24
Total length of pumping main		L_m =	715	m	= L_m
Volume in HDPE 80 main		V_{80} =	3.6	m^3	= L_m * 3.14 * 0.08^2 / 4
Pumping rate		q_p =	4.0	l/s	
Mininum pump run time for 10 starts per hour		T_{pm} =	90	seconds	
Minimum operating volume		V_{om} =	0.36	m^3	= T_{pm} * q_p / 1000
Given a wet well diameter of		W_d =	1.8	m	
Minimum operating depth in wet well is		d_{min} =	0.14	m	= V_{om} / (3.14 * W_d^2 / 4)
Use and operating depth of		d_o =	0.3	m	
Operating Volume used		V_{om} =	0.6	m^3	= V_{om} * d_o / d_{min}
Total Residence volume		V_{tot} =	4.2	m^3	= V_{om} + V_{80}
Is this less than the maximum allowable			YES, OK		= if(V_{tot}<V_{max} , "yes , ok" , "no xxxxxx")

Wm Neville & Sons Ltd - Development at Park Wexford

Minimum Sump Volume Calculations Under Partial Development

Pumping rate	Q_p =	4	l/s	
Maximum no of starts per hour	S_{max} =	10	starts	
Length of each pump cycle	T_{cycle} =	360	seconds	= 3600 / S_{max}
Volume pumped per cycle if pumping continuous	V_{CT} =	1,440	litres	
Trial 0perating volume fraction of this	V_{fr} =	0.25		Lowering this results in shorter cycles. Increasing lengthens the cycle
Trial storage volume	V_{st} =	360	litres	= V_{CT} * V_{fr}
Min. operating depth to achieve this (1.8m ϕ)		0.14	m	
Trial inflow rate	Q_i =	2.0	l/s	Lowering this results in longer cycles and increasing also lengthens the cycle
Pumping time to empty chamber	T_e =	90	seconds	= V_{st} / Q_p
Inflow during pumping	v_{ip} =	180	litres	= T_e * Q_i
Additional time to clear this	t_{inc} =	45	seconds	= v_{ip} / Q_p
Inflow during this pumping	v_{ip} =	90	litres	= t_{inc} * Q_i
Additional time to clear this	t_{inc} =	23	seconds	= v_{ip} / Q_p
Inflow during this pumping	v_{ip} =	45	litres	= t_{inc} * Q_i
Additional time to clear this	t_{inc} =	11	seconds	= v_{ip} / Q_p
Inflow during this pumping	v_{ip} =	23	litres	= t_{inc} * Q_i
Additional time to clear this	t_{inc} =	6	seconds	= v_{ip} / Q_p
Inflow during this pumping	v_{ip} =	11	litres	= t_{inc} * Q_i
Additional time to clear this	t_{inc} =	3	seconds	= v_{ip} / Q_p
Inflow during this pumping	v_{ip} =	6	litres	= t_{inc} * Q_i
Additional time to clear this	t_{inc} =	1	seconds	= v_{ip} / Q_p
Total pumping time	T_p =	179	seconds	= t_{inc} + t_{inc} + T_e + t_{inc} + t_{inc} + t_{inc} + t_{inc}
Pumped volume per cycle	V_p =	714	litres	= T_p * Q_p
Refill time from cut-out to cut-in	t_{ref} =	180	seconds	= V_{st} / Q_i
Total cycle time		359	seconds	**= t_{ref} + T_p**

The Green cells above are varied to achieve the required minimum volume (further iterations would bring the 359 seconds to 360 seconds). Greater than the minimum volumes reduces the number of starts.

Wm Neville & Sons Ltd - Development at Park Wexford				
Minimum Sump Volume Calculations at Full Development				
Pumping rate	Q_p =	11.5	l/s	
Maximum no of starts per hour	S_{max} =	10	starts	
Length of each pump cycle	T_{cycle} =	360	seconds	= 3600 / S_{max}
Volume pumped per cycle if pumping continuous	V_{CT} =	4,140	litres	
Trial operating volume fraction of this	V_{fr} =	0.25		Lowering this results in shorter cycles. Increasing lengthens the cycle
Trial storage volume	V_{st} =	1,035	litres	= V_{CT} * V_{fr}
Min. operating depth to achieve this (1.8m ϕ)		0.41	m	= V_{st} / 1000 / (3.142 * 1.8^2 / 4)
Trial inflow rate	Q_i =	5.75	l/s	Lowering this results in longer cycles and increasing also lengthens the cycle
Pumping time to empty chamber	T_e =	90	seconds	= V_{st} / Q_p
Inflow during pumping	v_{ip} =	518	litres	= T_e * Q_i
Additional time to clear this	t_{inc} =	45	seconds	= v_{ip} / Q_p
Inflow during this pumping	v_{ip} =	259	litres	= t_{inc} * Q_i
Additional time to clear this	t_{inc} =	23	seconds	= v_{ip} / Q_p
Inflow during this pumping	v_{ip} =	129	litres	= t_{inc} * Q_i
Additional time to clear this	t_{inc} =	11	seconds	= v_{ip} / Q_p
Inflow during this pumping	v_{ip} =	65	litres	= t_{inc} * Q_i
Additional time to clear this	t_{inc} =	6	seconds	= v_{ip} / Q_p
Inflow during this pumping	v_{ip} =	32	litres	= t_{inc} * Q_i
Additional time to clear this	t_{inc} =	3	seconds	= v_{ip} / Q_p
Inflow during this pumping	v_{ip} =	16	litres	= t_{inc} * Q_i
Additional time to clear this	t_{inc} =	1	seconds	= v_{ip} / Q_p
Total pumping time	T_p =	179	seconds	= t_{inc} + t_{inc} + T_e + t_{inc} + t_{inc} + t_{inc} + t_{inc}
Pumped volume per cycle	V_p =	2,054	litres	= T_p * Q_p
Refill time from cut-out to cut-in	t_{ref} =	180	seconds	= V_{st} / Q_i
Total cycle time		359	seconds	= t_{ref} + T_p

The Green cells above are varied to achieve the required minimum volume (further iterations would bring the 359 seconds to 360 seconds). Greater than the minimum volumes reduces the number of starts.

Emergency Storage Volume Calculation

Estimated average daily flow (dwf)		Q_{dwf} =	170	l/d	= Q_{dwf}
Required emergency storage capacity agreed		T_{SC} =	20	hours	
Volume Required		V_{sc} =	142	m^3	= Q_{dwf} * T_{SC} / 24
Calculation of volume in sewers and manholes					
Cut in level of pumps		C_l =	-0.58	m O.D.	
Lowest House FFL		F_{FL} =	3.150	m O.D.	
Max. allowable emergency level in sewers		L_{max} =	2.500	m O.D.	
Length of 225 sewers below this level		L_{225} =	1,125	m O.D.	= 91 + 75 + 77 + 286 + 85 + 511
Volume of storage in 225 pipes		V_{224} =	44.7	m^3	= L_{225} * 3.14 * 0.225^2 / 4
Length of 150 sewers below this level		L_{150} =	500	m = 89 +	30 + 45 + 65 + 80 + 40 + 55 + 60 + 21 + 15
Volume of storage in 150 pipes		V_{150} =	8.8	m^3	= L_{150} * 3.14 * 0.15^2 / 4
Total emergency volume in pipes		V_{pi} =	53.5	m^3	= V_{150} + V_{224}
Emergency Vol Manholes & P.Stn (see below)		V_{mhs} =	43.2	m^3	= V_{mhs}
Required storage in overflow tank		V_{sto} =	45.0	m^3	= V_{sc} - V_{pi} - V_{mhs}
Available internal height in tank (average)		H_{sto} =	2.76	m	= 2.86 - 0.24
Tank area required		A_{sto} =	16.3	m^2	= V_{sto} / H_{sto}
Assume width of		W_{sto} =	2.0	m	
Gives a length of		L_{sto} =	8.2	m	= A_{sto} / W_{sto}

Calculation of Emergency Storage in Manholes

MH No.	Available Height	MH Dia.	Available Vol (m^3)
P. Stn.	2.86	1.8	7.3
F1	2.8	1.2	3.2
F2	2.68	1.2	3.0
F3	2.5	1.2	2.8
F4	2.05	1.2	2.3
F5	1.82	1.2	2.1
F6	1.62	1.2	1.8
F7	1.17	1.2	1.3
F8	0.76	1.2	0.9
F9	0.58	1.2	0.7
F10	0.5	1.2	0.6
F13	1.67	1.2	1.9
F14	0.58	1.2	0.7
F15	0	1.2	0.0
F17	1.26	1.2	1.4
F18	0.44	1.2	0.5
F20	0.66	1.2	0.7
F23	0.39	1.2	0.4
F24	2.07	1.2	2.3
F25	1.78	1.2	2.0
F26	1.41	1.2	1.6
F27	0.97	1.2	1.1
F28	0.65	1.2	0.7
F29	0.42	1.2	0.5
F33	1.62	1.2	1.8
F34	0.83	1.2	0.9
F35	0.56	1.2	0.6
		V_{mhs} =	43.2

Ballast Calculations for Pumpstation.

Internal diameter of precast rings	ϕ =	1.8	m	
Clear height of chamber from invert to roof soffit	h_c =	4	m	
Volume of air and benching	V_a =	10.2	m^3	$= h_c * 3.14 * \phi^2 / 4$
Conservatively ignore benching.				
Density of water	γ_w =	10	kN/m^3	
Buoyancey created by the above air volume	B =	102	kN	= Va * γw
Density of concrete	γ_c =	24	kN/m^3	
Nett weight of concrete against buoyancy	γ_{cnt} =	14	kN/m^3	= γc - γw
Volume of concrete required to counteract air buoyancy	V_{cb} =	7.3	m^3	$= B / \gamma_{cnt}$
Increase this by 20% for a safety factor of 1.2	V_{cr} =	8.7	m^3	$= V_{cb} * 1.2$
Depth of roof	t_r =	0.2	m	
Volume of roof	v_r =	0.6	m^3	$= t_r * 3.14 * (\phi + 0.2)^2 / 4$
Reduced volume of roof after openings	v_{rr} =	0.3	m^3	$= v_r / 2$
Thickness of walls of precast rings	t_w =	0.08	m	
Volume of concrete in walls	v_w =	1.9	m^3	$= 3.14 * ([\phi + 2 * t_w]^2 - \phi^2) / 4 * h_c$
Depth of base	d_b =	0.38	m	
Volume of base	v_b =	1.1	m^3	$= d_b * 3.14 * (\phi + 2 * t_w)^2 / 4$
Total volume of concrete in pumpstation before ballast		3.3	m^3	$= v_b + v_w + v_{rr}$
Vol. of concrete ballast for a FOS of 1.2 against uplift	V_A =	5.1	m^3	$= V_{cr} - v_w - v_r - v_b$
Area of external walls of pumpstation	A_w =	26.5	m^2	$= h_c * (\phi + 2 * t_w + 0.15) * 3.14$
Thickness of ballast to outside of full height of walls		191	mm	$= V_A / A_w * 1000$
<u>Use 200mm surround to pumpstation</u>				

Ballast Calculations for Emergency Storage Tank.

Internal length of tank	L_t =	8.2	m	
Internal width of tank	W_t =	2	m	
Average internal height	H_t =	2.8	m	#No formula in cell E35#
Depth of roof concrete	**t_r =**	**0.35**	**m**	
Volume of air and benching	V_a =	45.3	m^3	= Lt * Wt * Ht
Density of water	γ_w =	10	kN/m^3	
Buoyancey created by the above air volume	B =	452.6	kN	$= V_a * \gamma_w$
Level of tank roof soffit	L_{st} =	2.5		
Level of top of tank roof	L_{tt} =	2.85		= Lst + tr
Density of concrete	γ_c =	24.0	kN/m^3	
Nett weight of concrete against buoyancy	γ_{cnt} =	14	kN/m^3	$= \gamma_c - \gamma_w$
Volume of concrete required to counteract air buoyancy	V_{cb} =	32.33	m^3	= B / γcnt
Increase this by 20% for a safety factor of 1.2	V_{cr} =	38.8	m^3	$= V_{cb} * 1.2$
Volume of roof	v_r =	5.74	m^3	= Lt * Wt * tr
Reduced volume of roof after openings	v_{rr} =	5.4	m^3	$= v_r - 0.3$
Thickness of walls	**t_w =**	**0.43**	**m**	
Volume of concrete in walls	v_w =	25.23	m^3	$= t_w * 2 * (L_t + W_t + t_w) * H_t$
Depth of base	**d_b =**	**0.50**	**m**	
Volume of base	v_b =	8.20	m^3	$= v_r * d_b / t_r$
Total volume of concrete in tank structure	V_{ct} =	38.9	m^3	$= v_b + v_w + v_{rr}$
Vol. of concrete ballast for a FOS of 1.2 against uplift	V_{cr} =	-0.074	m^3	$= V_{cr} - V_{ct}$
Check Is tank weight adequate?	YES, OK			= if(Vcr<0 , "yes , ok" , "no xxxx")

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Date: 12 November 2018

Project Ref. QUO-13197-LT5XH2
Quote No. 18-IRL-01114 Alt. 2 Ver. 1
Your Ref. Pump Station at Park, Wexford

Arthur Murphy & Co
Consulting Civil and Structural Engineering
Garryrichard, Foulksmills

For the attention of: Arthur Murphy

Pump Station at Park, Wexford

Further to your recent enquiry we now take pleasure in confirming the attached quotation for the above scheme.

Every endeavour is made to ensure that all major suppliers of equipment and services purchased by Xylem Water Solutions Ireland Ltd have been subject to assessment conforming to the requirements of BS EN, ISO 9001, 2000 and as laid down in the Xylem Water Solutions Ireland Ltd Quality plan.

An excellent after-sales service is operated by our Company and is available on request to all Xylem Water Solutions Ireland Ltd customers.

All FLYGT equipment is backed by an excellent replacement and spares availability service. At your request we would be pleased to supply details of recommended spares for the pumps or equipment covered by this quotation.
We would point out that the pumping equipment offered has a guaranteed minimum 15 years spares availability from the last production model.

For your assistance we enclose explanatory literature and drawings relevant to the type of equipment offered.

Please quote our reference number on any correspondence relating to the enclosed.

Excluded from our offer: All civil work, including cable ducts,plinths sealing of cable ducts etc. Access covers, ESB supply and mini piller by others
Pipework terminates approx 500mm outside the valve chamber in a NP16 Flange, responsibility for connection the rising main by others.
We do not carry out pressure testing of the pipework, pump testing is carried out at the factory only and is an additional charge if required.

Please Note: We have only included for the equipment specifically listed/detailed in our quotation and nothing else. If Xylem engineers are held up on the job for any reason outside of their control these hours will be charged accordingly. This is a budget price & if Xylem are sucessful in being awarded this contract another site visit will be required to confirm all duty details.

We trust our quotation meets with your approval and assure you of utmost attention at all times. Should you require any further information then please do not hesitate in contacting the undersigned by return.

Xylem Water Solutions Ireland Ltd.

Alison O'Reilly

Alison.oReilly@xyleminc.com

Our Sales Representative for your area is:
John Quigley – Tel 087 223 4933
John.Quigley@xyleminc.com
Service / Rental Central No. 0845 707 8012

xylem
Let's Solve Water

80mm option 1

Item#	Qty	Description	Unit Price
1.1	2	Flygt submersible pump NP 3085.160 SH 53-255 2,4 kW With open self cleaning channel impeller, Adaptive N designed for semi permanent wet well installation. NOTE! Guiding claw included, other installation eqiupmen to be ordered separately Main parts are made of Grey cast iron. Impeller material:grey cast iron The pumphouse is prepared for flush-valve. Guide pin 3-phase motor, 50 Hz, 400 VY, Rated power 2,4 kW. Rated current 4,70 A. Rated speed 2,840 RPM Max allowed ambient temperature of 40 dgr Celcius. Thermal contact for stator temp. monitoring. With 1 power cable: 20 m. SUBCAB S3x1,5+3x1,5/3+S(2x0,5) mm2 for direct start. SmartRun possible: No Leakage detector in statorhouse (FLS). Outlet type: Flange 80 mm. Inlet type: 1 80 mm. Impeller material: Cast iron Shaft material: AISI 431	€ 2,904.00
1.2	1	TOPS 150 GRP Packaged Pumps Station c/w valve chamber Drawing No : F2.75224 Size : 1.8m Diameter x 6.0m deep c/w Pipework : 80mm D.I. Cement Lined Pipe Valves : 2 x 80mm Ball Type Check Valve + 2 x 80mm Sluice U.G.R B. : 50mm Galvanised Guide Rail : 50mm Galvanised Brackets : Level control & Ultrasonic - Galvanised Access Cover : 5T S.M.W.L. Galvanised 1350mm x 900mm & : 5T S.M.W.L. Galvanised 1100mm x 900mm	€ 14,343.98
1.3	4	ENM10 Level Regulator Blue 0.95-1.10 g/cm3 c/w 13M Blue PVC cable	€ 98.59
1.4	2	Galvanised mild steel lifting chain 500kg SWL 7m long with lifting rings at each end and every metre	€ 54.26
1.5	2	Galvanised mild steel bow shackle 3.25t SWL	€ 5.53
1.6	2	Alarm unit MiniCAS II 110V Supervision relay for temperature and leakage sensors	€ 239.93
1.7	1	Flygt Control Panel Inclusive of Ultrasonic, Flow meter and mild steel galvanised kiosk.	€ 12,481.00
		Total Discounted Price (NETT)	**€ 33,626.80**

Note
A precast concrete Pumpstation is proposed See Drawing PL 08.

150mm option 2

Item#	Qty	Description	Unit Price

xylem
Let's Solve Water

Item#	Qty	Description	Unit Price
2.1	2	Flygt submersible pump NP 3085.160 SH 53-254 2,4 kW With open self cleaning channel impeller, Adaptive N designed for semi permanent wet well installation. NOTE! Guiding claw included, other installation eqiupmen to be ordered separately Main parts are made of Grey cast iron. Impeller material:grey cast iron The pumphouse is prepared for flush-valve. Guide pin 3-phase motor, 50 Hz, 400 VY, Rated power 2,4 kW. Rated current 4,70 A. Rated speed 2,840 RPM Max allowed ambient temperature of 40 dgr Celcius. Thermal contact for stator temp. monitoring. With 1 power cable: 20 m. SUBCAB S3x1,5+3x1,5/3+S(2x0,5) mm2 for direct start. SmartRun possible: No Leakage detector in statorhouse (FLS). Outlet type: Flange 80 mm. Inlet type: 1 80 mm. Impeller material: Cast iron Shaft material: AISI 431	€ 2,904.00
2.2	1	TOPS 150 GRP Packaged Pumps Station c/w valve chamber Drawing No : F2.75224 Size : 1.8m Diameter x 6.0m deep c/w Pipework : 80mm D.I. Cement Lined Pipe Valves : 2 x 80mm Ball Type Check Valve + 2 x 80mm Sluice U.G.R B. : 50mm Galvanised Guide Rail : 50mm Galvanised Brackets : Level control & Ultrasonic - Galvanised Access Cover : 5T S.M.W.L. Galvanised 1350mm x 900mm & : 5T S.M.W.L. Galvanised 1100mm x 900mm	€ 14,343.98
2.3	4	ENM10 Level Regulator Blue 0.95-1.10 g/cm3 c/w 13M Blue PVC cable	€ 98.59
2.4	2	Galvanised mild steel lifting chain 500kg SWL 7m long with lifting rings at each end and every metre	€ 54.26
2.5	2	Galvanised mild steel bow shackle 3.25t SWL	€ 5.53
2.6	2	Alarm unit MiniCAS II 110V Supervision relay for temperature and leakage sensors	€ 239.93
2.7	1	Flygt Control Panel Inclusive of Ultrasonic, Flow meter and mild steel galvanised kiosk.	€ 12,481.00

Note
A precast
concrete
Pumpstation
is proposed
See Drawing
PL 08.

Total Discounted Price (NETT) € 33,626.80

Installation

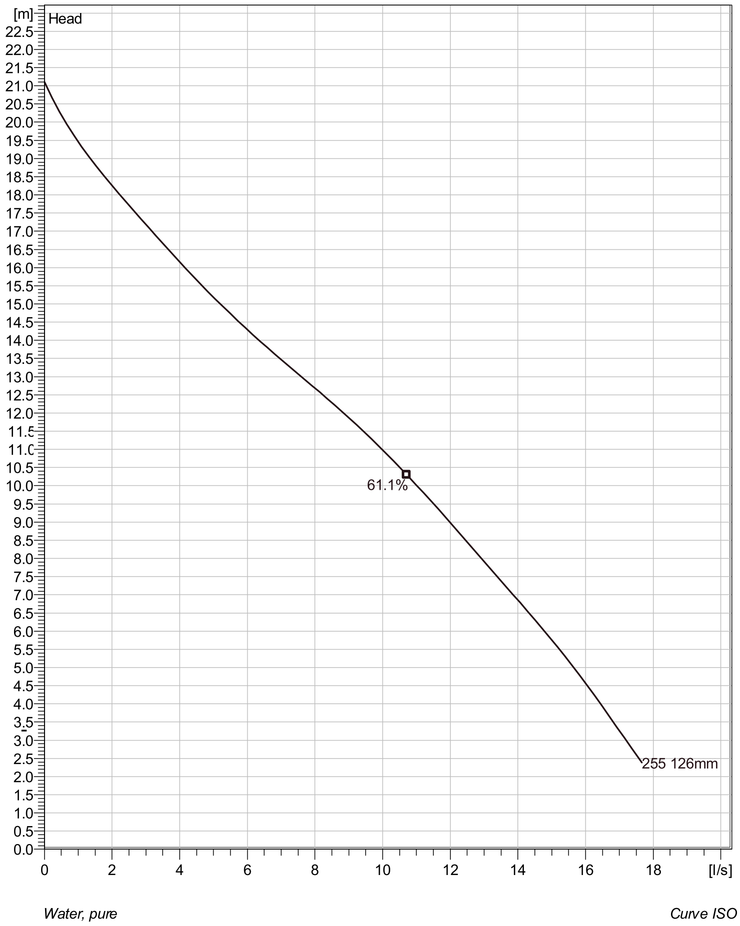
Item#	Qty	Description	Unit Price
3.1	1	Materials	€ 350.00
3.2	1	Labour installation on site 2 x Xylem Engineers	€ 3,900.00

Total Discounted Price (NETT) € 4,250.00

FLYGT godwin LEOPOLD LOWARA multitrode PIMS GROUP SANITAIRE VOGEL PUMPEN WEDECO

NP 3085 SH 3~ Adaptive 255

Technical specification



Note: Picture might not correspond to the current configuration.

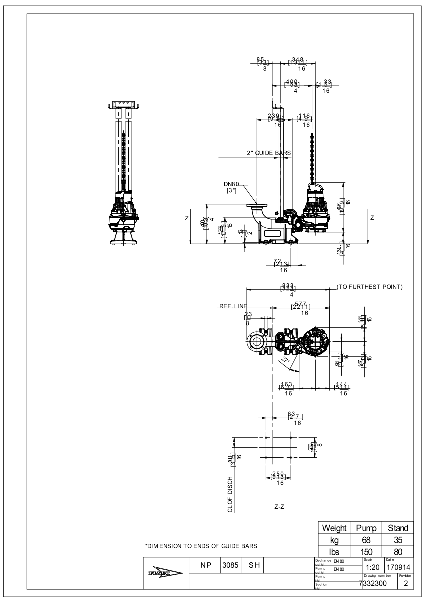
General

Patented self cleaning semi-open channel impeller, ideal for pumping in most waste water applications. Possible to be upgraded with Guide-pin® for even better clogging resistance. Modular based design with high adaptation grade.

Impeller

Impeller material	Grey cast iron
Discharge Flange Diameter	80 mm
Suction Flange Diameter	80 mm
Impeller diameter	126 mm
Number of blades	2

Installation: P - Semi permanent, Wet



Motor

Motor #	N3085.160 15-09-2AL-W 2.4KW Standard
Stator variant	31
Frequency	50 Hz
Rated voltage	400 V
Number of poles	2
Phases	3~
Rated power	2.4 kW
Rated current	4.7 A
Starting current	28 A
Rated speed	2840 rpm
Power factor	
1/1 Load	0.92
3/4 Load	0.89
1/2 Load	0.82
Motor efficiency	
1/1 Load	80.5 %
3/4 Load	82.5 %
1/2 Load	82.0 %

Configuration

Project	Project ID	Created by	Created on	Last update
			11/12/2018	

NP 3085 SH 3~ Adaptive 255

Performance curve

Pump

Discharge Flange Diameter	80 mm
Suction Flange Diameter	80 mm
Impeller diameter	126 mm
Number of blades	2

Motor

Motor #	N3085.160 15-09-2AL-W 2.4KW
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Power factor	
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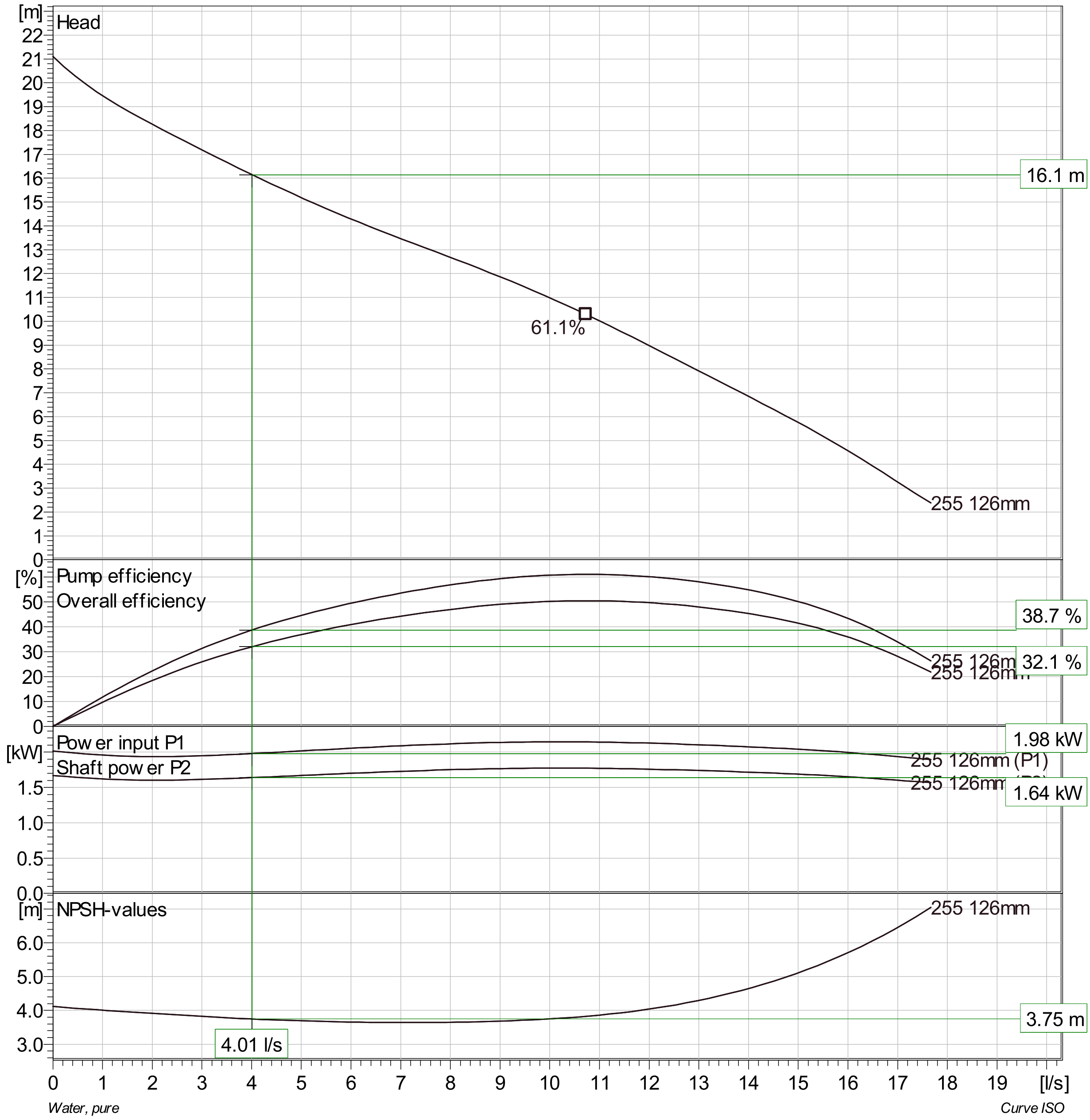
3/4 Load	0.89
1/2 Load	0.82

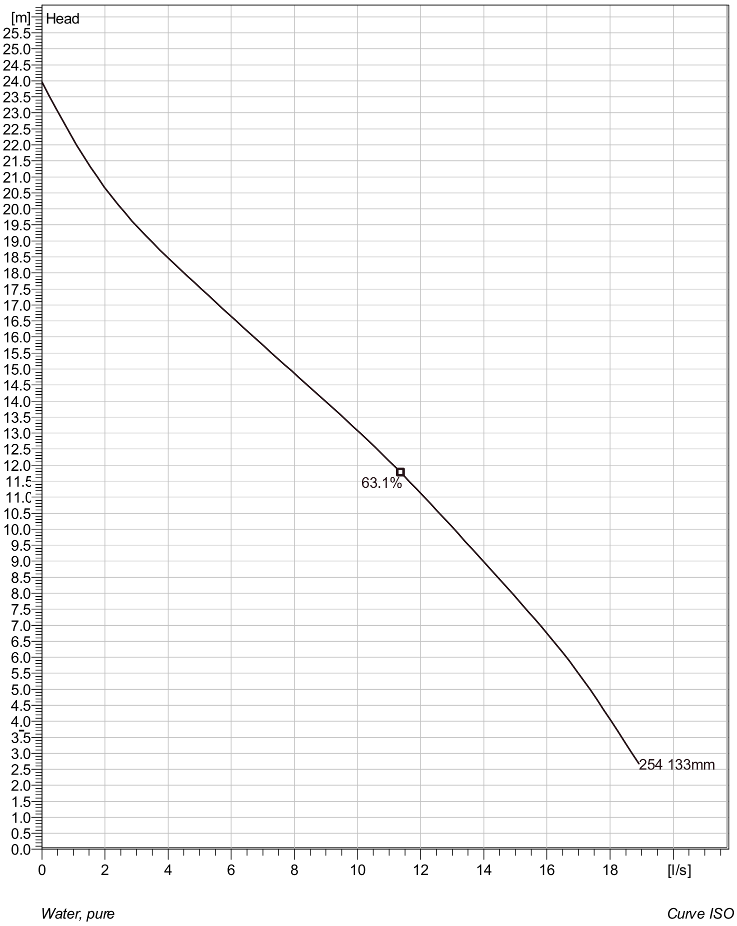
Motor efficiency	
1/1 Load	80.5 %
3/4 Load	82.5 %
1/2 Load	82.0 %



Project	Project ID	Created by	Created on	Last update
			11/12/2018	

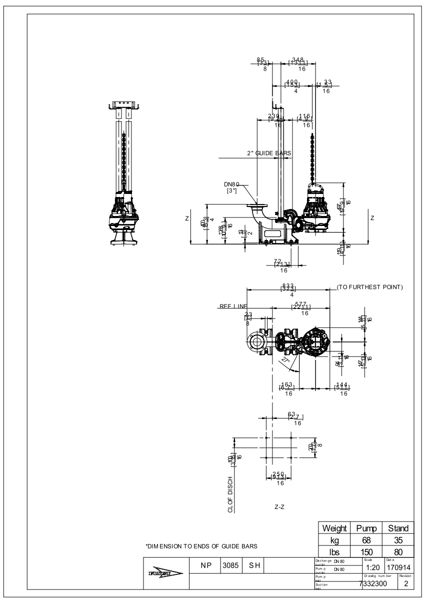
NP 3085 SH 3~ Adaptive 254

Technical specification



Note: Picture might not correspond to the current configuration.

Installation: P - Semi permanent, Wet



General

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Impeller

Impeller material	Grey cast iron
Discharge Flange Diameter	80 mm
Suction Flange Diameter	80 mm
Impeller diameter	133 mm
Number of blades	2

Motor

Motor #	N3085.160 15-09-2AL-W 2.4KW Standard
Stator variant	31
Frequency	50 Hz
Rated voltage	400 V
Number of poles	2
Phases	3~
Rated power	2.4 kW
Rated current	4.7 A
Starting current	28 A
Rated speed	2840 rpm
Power factor	
1/1 Load	0.92
3/4 Load	0.89
1/2 Load	0.82
Motor efficiency	
1/1 Load	80.5 %
3/4 Load	82.5 %
1/2 Load	82.0 %

Configuration

Project	Project ID	Created by	Created on	Last update
			11/12/2018	

NP 3085 SH 3~ Adaptive 254

Performance curve

Pump

Discharge Flange Diameter	80 mm
Suction Flange Diameter	80 mm
Impeller diameter	133 mm
Number of blades	2

Motor

Motor #	N3085.160 15-09-2AL-W 2.4KW
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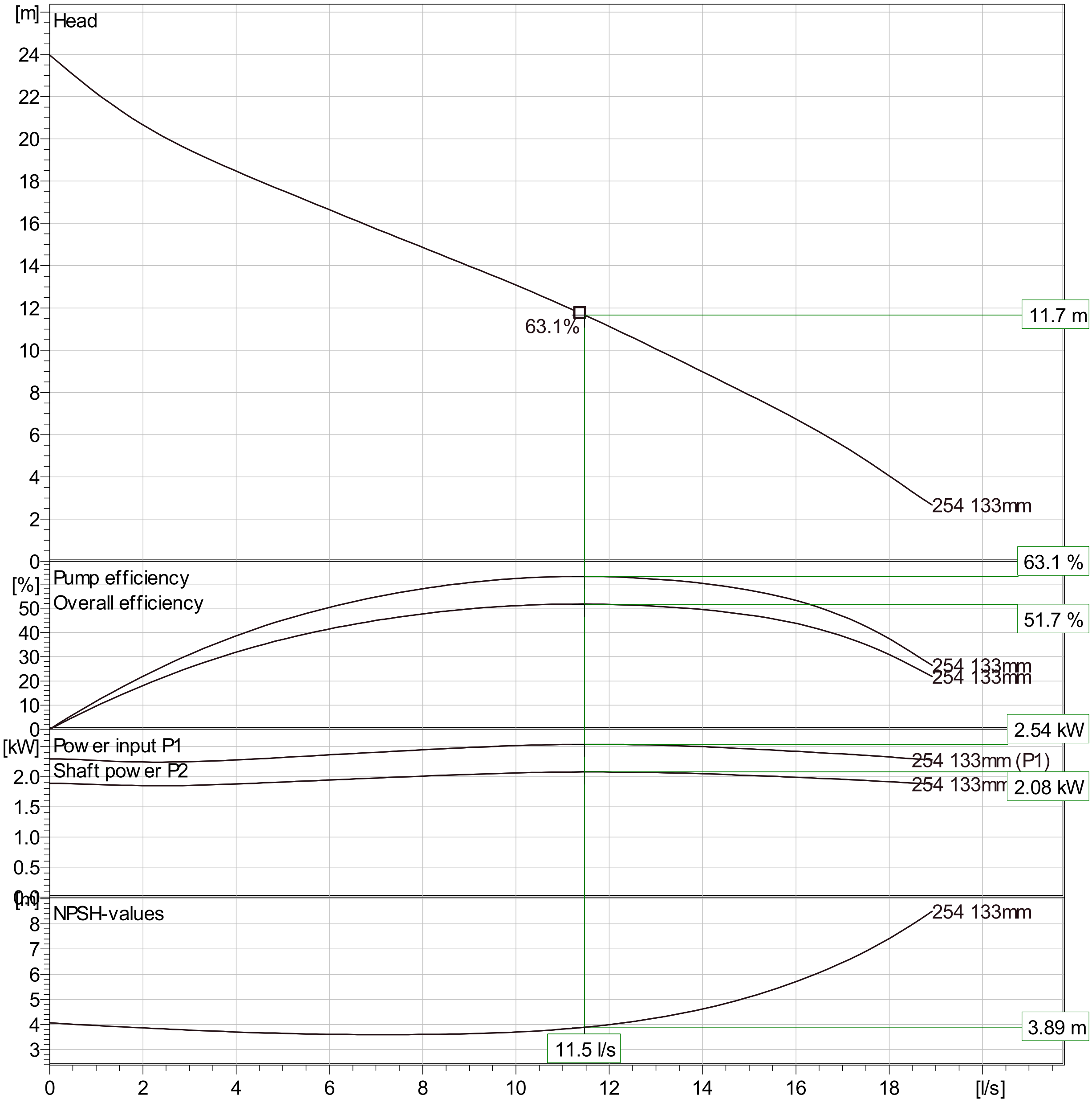

Water, pure *Curve ISO*

Project	Project ID	Created by	Created on	Last update
			11/12/2018	