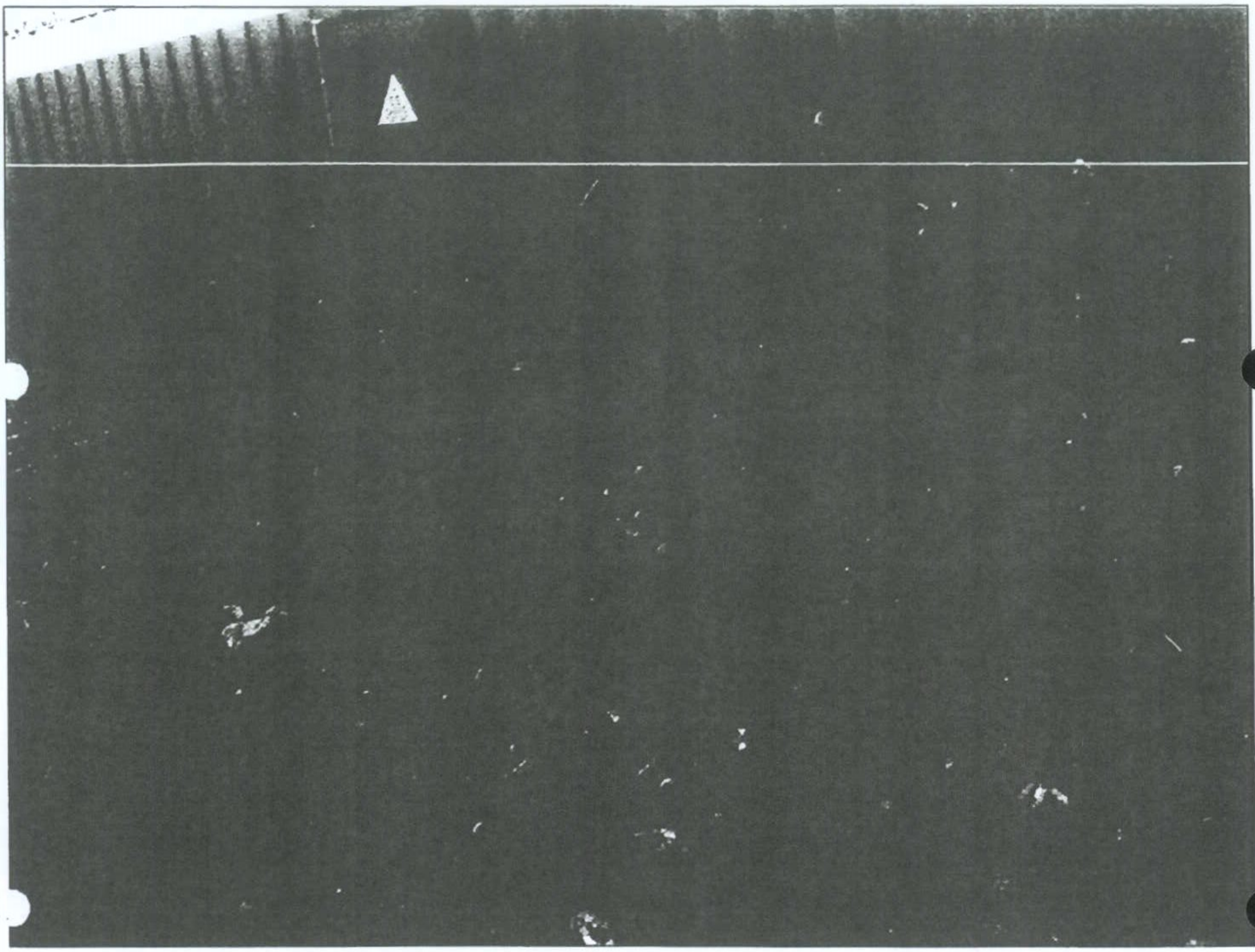
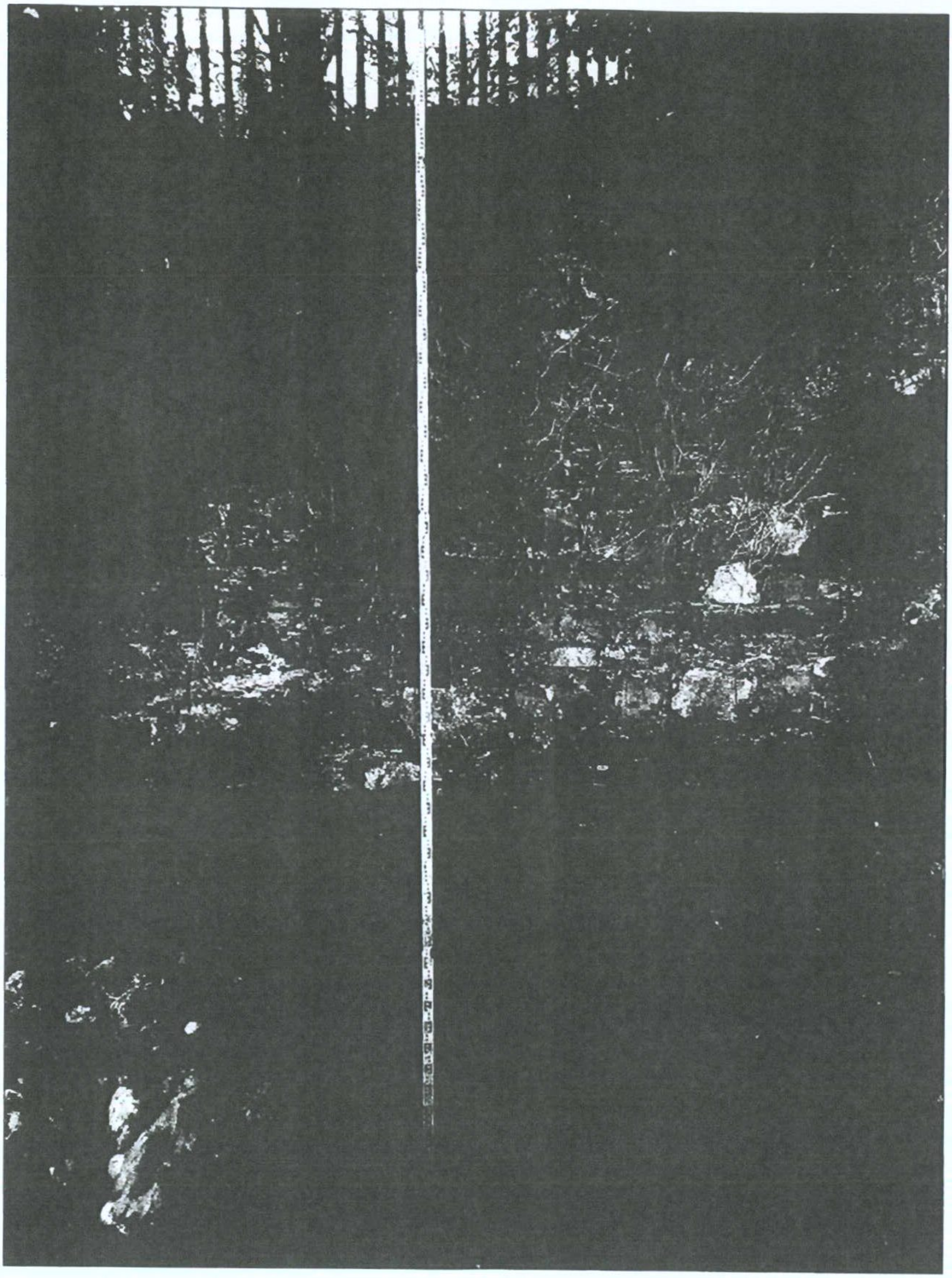


BYRNE LOOBY PARTNERS
CONSULTING ENGINEERS
22 CANAL WALK
PARK WEST
DUBLIN 12
TEL : 01-6301593
FAX : 01-6301594
E-MAIL : info@blp.ie

Client	O'CONNORS SUTTON CRONIN
Project	B462 - WESTGATE DEVELOPMENT
Drawing Title	TRIAL PIT 3
Date	12.12.05
Scale	NTS
Drawing number	PHOTO F
Revision	



BLP CONSULTING ENGINEERS

BYRNE LOOBY PARTNERS
CONSULTING ENGINEERS
22 CANAL WALK
PARK WEST
DUBLIN 12
TEL : 01-6301593
FAX : 01-6301594
E-MAIL : info@blp.ie

Client: O'CONNORS SUTTON CRONIN	
Project: B462 – WESTGATE DEVELOPMENT	
Drawing Title: SLIT TRENCH 1	
Date: 12.12.05	Scale: NTS
Drawing number: PHOTO G	Revision:

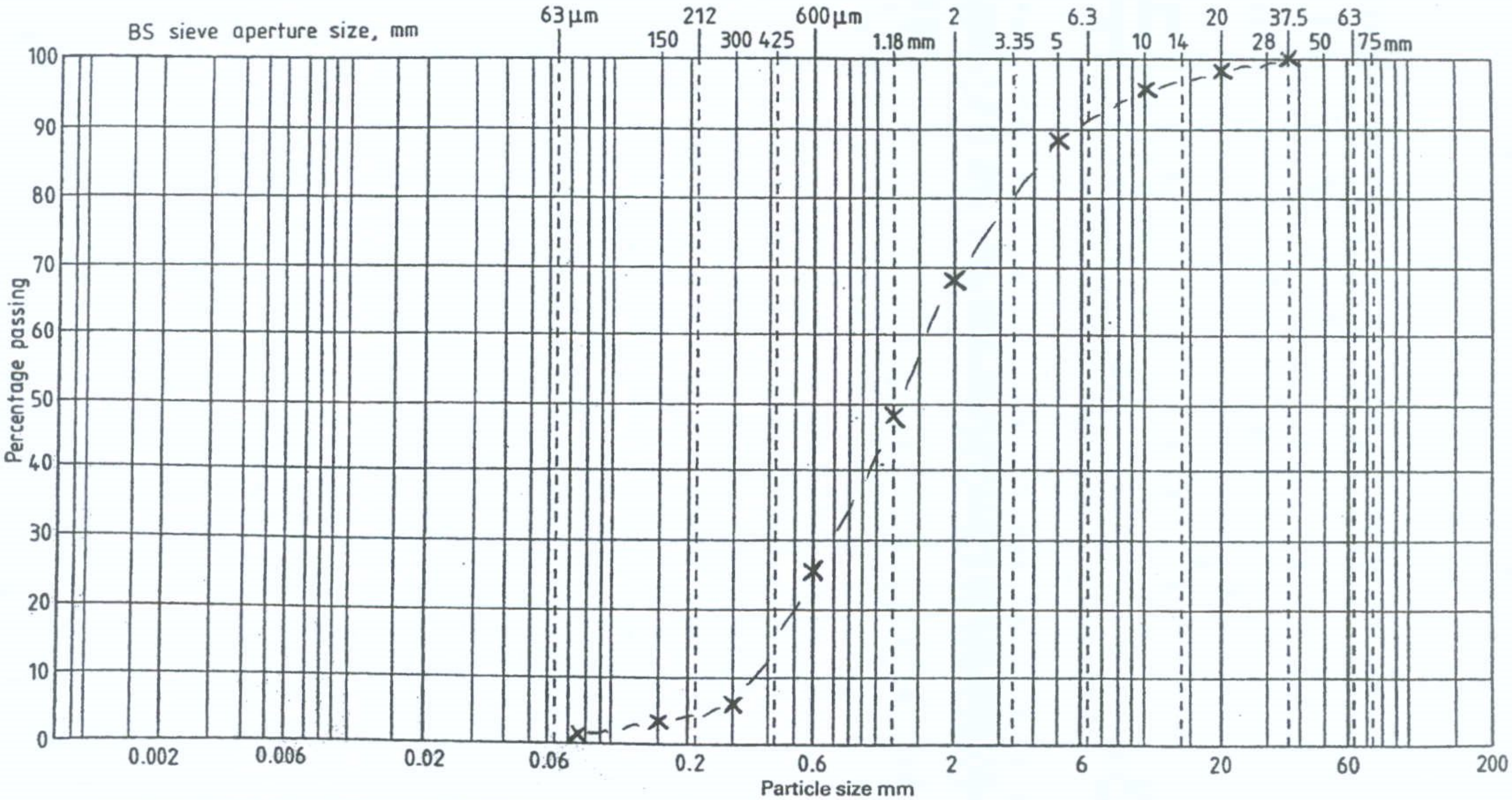
Appendix G

Laboratory Testing

LABORATORY TEST RESULTS - WESTGATE

BH No	Depth mBGL	MC %	LC %	PL %	PI %	Remarks
4	8.0	9.8	25	18	7	
6	10.0	7.6	25	15	10	
13	7.0	9.3	27	16	11	
14	8.0	7.8	25	17	8	
15	9.0	9.1	25	14	11	
15	10.0	8.7	23	17	6	

Location WESTGATE	Soil desc. ,.tion	Job ref.		Sample no.	
		Borehole/~~Pit no.~~	3	Depth	10·0 m
Test method	BS 1377 : Part 2 : 1990 : **9.2/9.3/9.4/9.6/9.7 ***			Date	11/1/06



CLAY	Fine	Medium	Coarse	Fine	Medium	Coarse	Fine	Medium	Coarse	COBBLES	BOULDERS
	SILT			SAND			GRAVEL				

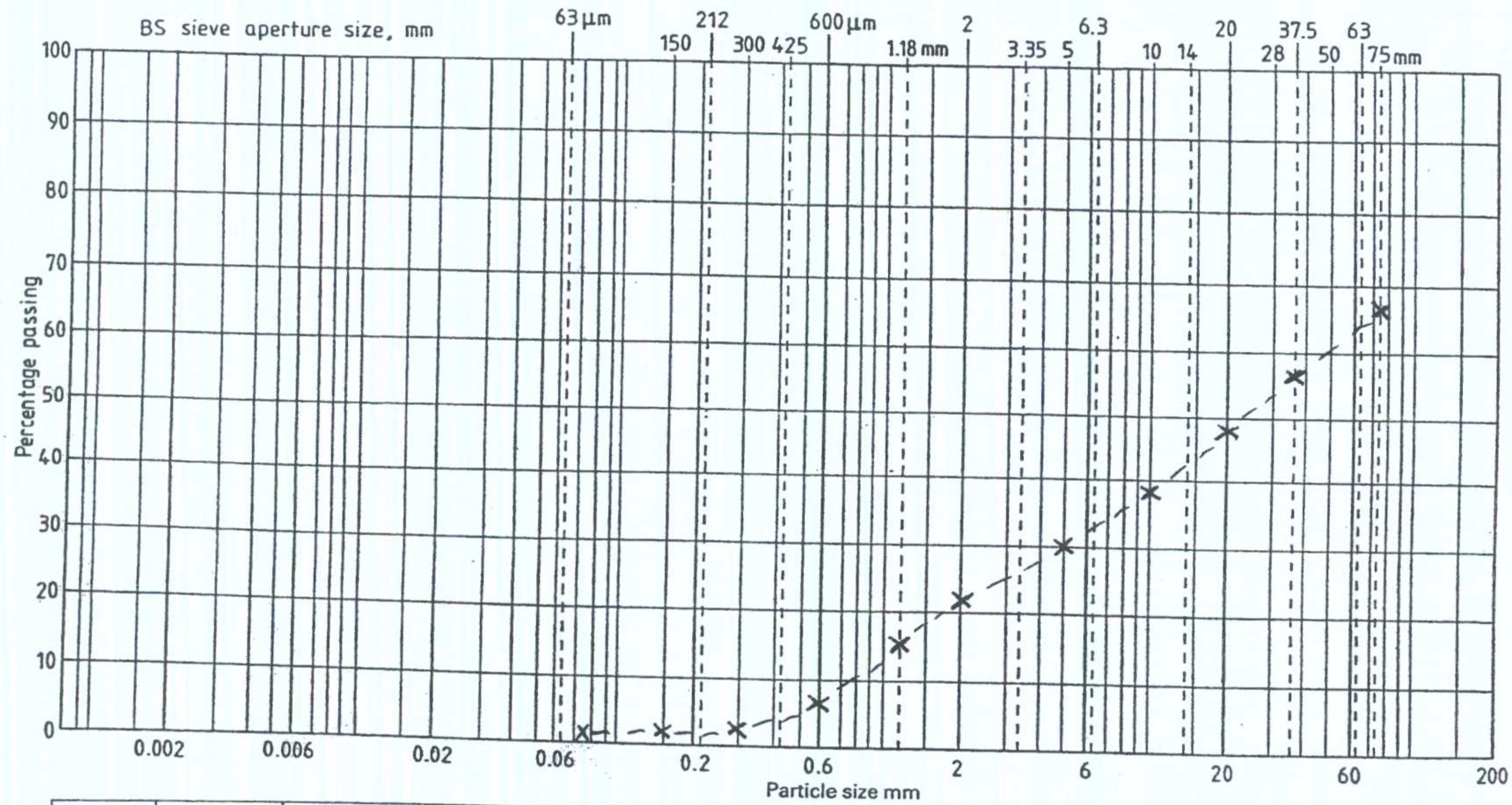
* Delete as appropriate	Operator	Checked	Approved
	SM		

BS 1377 : Part 2 : 1990

Particle size distribution chart

m 2.

Location WESTGATE	Soil description	Job ref.		Sample no.	
		Borehole/~~Pit no.~~	7A	Depth	5.0 m
Test method	BS 1377 : Part 2 : 1990 : **9.2/9.3/9.4/9.6/9.7** *			Date	11/1/06



CLAY	Fine	Medium	Coarse	Fine	Medium	Coarse	Fine	Medium	Coarse	COBBLES	BOULDERS
	SILT			SAND			GRAVEL				

* Delete as appropriate

Operator	Checked	Approved
SM		

BS 1377 : Part 2 : 1990

☑ Interim

☐ Validated

ALcontrol Laboratories Ireland

Table Of Results

Ref Number: 06-B00102/01

Client: Ground Investigations Ireland Limited

Date of Receipt: 10/01/2006
(of first sample)

Sample Type: SOIL

Location:

Client Contact: James Lombard

Client Ref: 1032 WEST GATE

	Detection Method		GRAVIMETRIC	KONE	METER
	Method Detection Limit		<0.1%	<0.003g/l	napH Units
	UKAS Accredited			✓	✓
ALcontrol Reference	Sample Identity	Other ID	Natural Moisture Content	Soluble Sulphate 2:1 Extract	pH
			%	g/l	pH Units
06-B00102-S0003	BH 8 9.0M	UNKNOWN	5.6	0.038	8.63
06-B00102-S0004	BH 14 8.0M	UNKNOWN	7.8	0.015	8.03
06-B00102-S0005	BH 15 10.0M	UNKNOWN	8.7	0.050	7.90

Notes : METHOD DETECTION LIMITS ARE NOT ALWAYS ACHIEVABLE DUE TO VARIOUS CIRCUMSTANCES BEYOND OUR CONTROL.

THE DATA ON THIS PRELIMINARY REPORT IS NOT VALIDATED AND MAY BE SUBJECT TO CHANGE.

NDP = NO DETERMINATION POSSIBLE

Checked By : Anne Kelly

Printed at 10:01 on 12/01/2006

* SUBCONTRACTED TO OTHER LABORATORY / ** SAMPLES ANALYSED AT THE CHESTER LABORATORY

INTERPRETATIVE GEOTECHNICAL REPORT

FOR

SITE INVESTIGATION

AT

WESTGATE, DUBLIN 8

FOR

O'CONNOR SUTTON CRONIN

Byrne Looby Partners
Consultant Engineers
February 2006
B462
Rev 1

REPORT ISSUE

Report Title: Interpretative Geotechnical Report for Site Investigation at Westgate, Dublin 8 for O'Connor Sutton Cronin.

Issue No.	Date	Checked	Passed
1	January 2006	MB	MAL
2	February 2006	MAL	MAL

Byrne Looby Partners, Consulting Engineers
22, Canal Walk, Park West, Dublin 12, Ireland
Tel 01 6301593, Fax 01 6301594, e-mail: info@blp.ie

CONTENTS

1.0 INTRODUCTION

Byrne Looby Partners have been requested by O'Connor Sutton Cronin to carry out a Geotechnical site investigation at the Westgate Site in Dublin 8.

The proposed development is understood to consist of multi-storey offices and apartment structures. A double level basement shall be located across the total site area.

The site investigation was carried out in November/December 2005 in accordance with BS 5930 "British Standard for Site Investigations".

The brief of the site investigation was to determine suitable foundation options, soil retention options and groundwater control measures for the proposed development. The following report provides geotechnical recommendations for the proposed development based on the ground conditions encountered during the site investigation.

The scope of the site investigation consisted of

- 6 No. rotary coreholes
- 11 No. boreholes
- 4 No. trial pits.
- Insitu soil and permeability testing
- Laboratory testing

The report is presented in 6 sections. Section 2.0 presents a description of the site including the proposed development and a summary of the 2 No. previous site investigations that were carried out by URS (2002) and Site Investigations Ltd (2000). Section 3.0 and 4.0 summarises the site investigation and ground conditions. Section 5.0 presents the recommendations for the proposed development. Section 6.0 provides a summary of the report.

2.0 SITE DESCRIPTION

2.1 Site Location

The site is located approximately 2.5km to the west of Dublin City and approximately 40m south west of Heuston Station. Access to the site is via St. John's Road.

Figure 1 presents the site location.

2.2 Site Boundary

The site is bounded to the north by St. John's Road to the south and west by The Royal Hospital/Museum of Modern Art and to the east by Military Road.

The approximate area of the site is 3.5 hectares.

2.3 Site Topography

Topographical levels at the site vary between 7.1m OD and 14.2m OD. In general the site slopes moderately from north to south at approximately 1:20.

Figure 2 presents the topographical survey of the site.

2.4 Proposed Development

The proposed development is to consist of a multi-storey development consisting of residential apartments and office units. A double level basement is to underlay the super structure across the complete site area. The level of the basement is to be +0.5m OD north of GL W and +3.0m OD south of GL W.

Figure 3 presents the proposed development.

2.5 Desk Study

2 No. site investigations were carried out by Site Investigations Ltd in 2000 and by URS in 2002. A review of the 2 No. site investigations as part of the desk studies.

The 2000 site investigation consisted of 5 No. shell and auger boreholes. Boreholes were carried out to between 5.7m bgl and 18.0m bgl. 2 No. boreholes were cored (open-hole) from shallow depths so that the deeper strata may be described. In general the boreholes revealed that ground conditions consisted of stiff brown boulder clay. However in BH 3 and BH 5 ground conditions consisted of 5m to 10.5m of stiff brown boulder clay over 2.6m to 4.5m of sandy gravel over stiff brown boulder clay.

The 2000 site investigation locations are presented in Figure 4.

The 2002 site investigation consisted of 5 No. shell and auger boreholes. Boreholes were carried out to between 5.5m bgl and 15.5m bgl. The ground conditions generally consisted of 1m to 2m of fill over 3.5m to 5.0m of gravel over brown boulder clay.

In both site investigations the Shell and Auger boreholes refused at shallow depths due to most likely the density of the gravels and the presence of boulder and boulder layers.

The 2002 site investigation locations are presented in Figure 4.

The 2002 URS boreholes were instrumented with standpipes so that groundwater monitoring could be carried out. Table 1 below presents the static water levels recorded by URS.

Location	Water Level (m OD)
MW101	2.0
MW102	<1.4
MW103	8.1
MW104	6.7
MW105	1.3

Table 1 – Groundwater Level 2002 – Site Investigation

*Note in the southwest corner of the site investigation a water level of 9.1m OD was indicated.

Appendix A presents the borehole logs from 2000 and 2002 site investigations.

3.0 SITE INVESTIGATION

3.1 General

The site investigation was carried out in November/December 2005 in accordance with BS 5930 British Standard for Site Investigations. BLP supervised all site works.

The brief of the site investigation was as follows:

a) Determine suitable foundation options.
b) Provide retention options for basement excavation.
c) Provide recommendations for the control of groundwater ingress into the basement excavation.

The site investigation consisted of 6 No. rotary coreholes, 11 No. shell and auger boreholes and 4 No. trial pits, Insitu Field Testing and Geotechnical Laboratory Testing.

3.2 Rotary Coreholes

6 No. Rotary coreholes were carried out by Ground Investigations Ireland (GII). Rotary coreholes were carried out to between 21.3m bgl and 33.6m bgl.

Total Core Recovery (TCR), Solid Core Recovery (SCR), Rock Quality Designation (RQD) and Fracture Index values were recorded during corings so that the consistency of the rock may be determined.

Rotary coreholes consisted of a 70mm ϕ corehole. In the rotary cores the overburden was drilled by the "Open Hole" method which does not provide any recovery of the subsoils and soil descriptions are based on flush returns and are therefore not that accurate. This approach was employed to ensure that the holes achieve the required total depth. At depths greater than 15m rotary coring was introduced so that the overburden could be investigated at depths below where the shell and auger borehole refused. It was also employed to define rock head on the site.

Standard Penetration Tests were carried out in the rotary coreholes so that a profile of the strength/consistency of the subsoils could be determined.

Rotary coreholes were instrumented with standpipes so that groundwater levels could be monitored across the site.

Appendix B presents the rotary corehole logs.

3.3 Boreholes

11 No. boreholes were completed as part of the site investigation. Boreholes were carried out between 6.5m bgl and 20.5m bgl. In general the initial boreholes refused at shallow depths (i.e. in the upper 4m). This refusal was attributed to the high quantity of cobbles encountered in the upper gravel stratum. In order to avoid the larger boulders at shallow depths, trial pits were excavated at the borehole locations and backfilled with sand.

Bulk disturbed soil samples were removed from the boreholes at regular 1.0m intervals. Standard Penetration tests were also carried out at 1.0m intervals so that a profile of the soil strength and consistency could be determined.

All boreholes were instrumented with a standpipe so that future groundwater monitoring and rising head tests could be carried out.

Appendix C presents the borehole logs.

Appendix E presents the rising head test results.

3.4 Trial Pits

3 No. trial pits were excavated to determine the cobble and boulder size and their percentage in the upper gravel stratum.

1 No. slit trench was carried out perpendicular to the existing boundary wall so that the foundation level and dimensions of the wall foundations could be determined.

Appendix D presents the trial pit logs.

Photos A to G illustrate the trial pits and slit trench.

.5 Laboratory Testing

Appendix G contains laboratory testing carried out as part of the investigation. Laboratory testing included atterburg tests, moisture content tests, PSD grading curves and sulphate/ pH testing.

4.0 GROUND CONDITIONS

4.1 General

In general ground conditions across the site consist of made ground over gravel over stratas consisting of interlayering gravel, brown and black boulder clays over bedrock.

Table 2 below presents a summary of the ground conditions.

Stratum	Depth to Top of Stratum (m bgl)	Thickness (m)
Sand Fill / Made Ground	0	0.8 – 4.8
Gravel	2.7 – 4.8	0.3 – 9.0
Brown Boulder Clay /Black Boulder Clay/Gravel	1.7 – 5.3	-
Bedrock	26.1 – 27.6	-

Table 2 – Summary of Ground Conditions

Figure 5 and Figure 6 illustrate geological cross sections through the site indicating the proposed basement levels.

It can be seen from the cross sections that at the formation level (+0.5m OD) of the basement north of GL W ground conditions consist of dense gravels or very stiff brown/black boulder clay. In general at formation level (+3.0m OD) south of GL W ground conditions consist of brown or black boulder clay.

4.2 Fill / Made Ground

Fill/made ground was encountered in all boreholes, rotary coreholes and trial pits at ground level. In the case of the boreholes the upper 3m of material generally consisted of backfilled sand.

The original fill/made ground material extended to between 0.8m bgl and 4.8m bgl.

4.3 Gravel

Gravel was generally encountered beneath the fill/made ground stratum between 2.7m bgl and 4.8m bgl except in BH1, BH13 and BH15. The thickness of this stratum varied between 0.3m and 9.0m. Gravel was also encountered at deeper depths interlayered with the brown and black boulder clay in boreholes BH3, BH4, BH6, BH14 and BH15 and in rotary coreholes RC1, RC2, RC3, RC4, RC5 and RC6. This lower gravel stratum was encountered between 9.5m bgl and 26.6m bgl. The thickness of this stratum varied between 1.5m and 6.9m where penetrated.

This material is generally described as dense to very dense sand gravel with cobbles and boulders. The upper gravel stratum contained a large quantity of cobble size material and occasional boulders (c. 500mm). SPT "N" VALUES for this material vary between 21 and refusal. This material has a ϕ' value of approximately 40°.

Rising head tests have been carried out in the standpipes to determine the permeability of the gravel stratum. The rising head tests reveal that the permeability of the gravel material varies from in excess of 5×10^{-4} to 4×10^{-6} m/s.

Rising head test results are presented in Appendix E.

4.4 Brown Boulder Clay

Brown boulder clay was encountered in boreholes BH1, BH3, BH13, BH14 and BH15 and in rotary coreholes RC1, RC2, RC3 and RC4. This material was encountered between 0.8m bgl and 23.6m bgl. The thickness of the stratum varied between 0.4m and 4.5m.

In general the upper layers of this material were described as stiff brown boulder clay and the deeper layers were described as very stiff to hard brown boulder clay. SPT "N" values for this material vary between 12 and refusal.

The upper brown boulder clay strata have therefore an undrained shear strength of approximately 75kPa to 125kPa. The lower brown boulder clay found underlying the hard black boulder clay layer would have an undrained shear strength of 150kPa to 200kPa.

4.5 Black Boulder Clay

Black boulder clay was encountered in boreholes BH3, BH4, BH6, BH3, BH14 and BH15 and in rotary coreholes RC1, RC2, RC3, RC4 and RC6. This material was encountered between 4.2m bgl and 16.1m bgl and varied in thickness between 1.8m and 8.2m.

In general this material is described as very stiff to hard black boulder clay. SPT "N" values for this material were refusal values. This material has an undrained shear strength of approximately 150kPa to 200kPa.

4.6 Bedrock

Bedrock was encountered in all rotary coreholes except RC5 (22.0m bgl) and RC6 (21.3m bgl). The bedrock was encountered between -13.7m OD and -19.2m OD. The bedrock was proven to be between 4.0m and 7.5m.

In general the bedrock was described as strong dark grey and black LIMESTONE, fresh to highly weathered. In RC2 5m of weak black calcareous MUDSTONE was encountered above limestone.

Total Core Recovery (TCR) values for the limestone bedrock varied between 63% and 100%. Solid Core Recovery (SCR) values varied between 41% and 100% and Rock Quality Designation (RQD) values varied between 9% to 69%. Fracture Spacing Index values varied between NIL and 12.

TCR and SCR values for the mudstone were 0%. RQD values for the mudstone varied between 30% and 33%. Fracture Spacing Index values for the mudstone were NIL.

4.7 Groundwater

Groundwater strikes were recorded in the gravel strata during drilling of the boreholes. Table 3 below presents a summary of the groundwater strikes during drilling.

Location	Depth (m bgl)
BH3	9.5
BH6	11.7
BH7	3.7
BH8	9.7
BH14	12.2
BH15	12.5

Table 3 – Summary of Groundwater Strikes

Groundwater levels in the boreholes and rotary coreholes were monitored on 9th December 2005 to 3rd January 2006. Table 4 below presents the recorded groundwater levels.

Location	9th December 2005		13th December 2005		19th December 2005		3rd January 2006	
	Water Level (m bgl)	Water Level (m OD)	Water Level (m bgl)	Water Level (m OD)	Water Level (m OD)	Water Level (m bgl)	Water Level (m bgl)	Water Level (m OD)
BH1	-	-	-	-	4.2	4.9	4.6	4.4
BH2	-	-	DRY	DRY	DRY	DRY	DRY	DRY
BH3	-	-	4.1	8.3	4.1	8.3	4.9	7.5
BH4	-	-	7.8	3.5	8.2	3.1	8.7	2.6
BH5	-	-	-	-	DRY	DRY	DRY	DRY
BH6	-	-	7.5	2.9	7.7	2.7	8.0	2.4
BH7	1.9	7.2	1.9	7.2	1.9	7.2	1.9	7.1
BH8	7.6	1.1	7.6	1.1	7.7	1.0	7.9	0.8
BH13	-	-	-	-	-	-	-	-
BH14	9.5	2.5	9.5	2.5	9.6	2.4	9.8	2.2
BH15	8.7	4.6	8.2	5.1	8.9	4.4	8.9	3.7
RC1	11.8	1.6	11.7	1.7	11.8	1.6	12.0	1.4
RC3	4.4	8.4	4.4	8.3	4.5	8.2	4.7	8.1
RC4	-	-	6.4	1.0	6.5	0.6	6.7	0.7
RC5	-	-	7.3	0.8	7.5	0.6	7.7	0.4
RC6	-	-	-	-	4.0	5.3	6.3	3.0

Table 4 Summary of Groundwater Monitoring

5.0 RECOMMENDATIONS

5.1 General

The following section provides geotechnical recommendations for the proposed development.

The proposed development is to consist of multi-storey retail and residential structures. A double level basement is to be constructed across the whole site area. The proposed basement level north of GL W is +0.5m OD and south of GL W is +4.0m OD.

5.2 Foundations

The recommended foundation option for the proposed development are pad foundations. Pad foundations may be founded of the very stiff to hard brown or black boulder clay or the dense to very dense gravel.

A nett allowable bearing capacity of 350kPa may be assumed for the dense gravel stratum at the formation indicated. A nett allowable bearing capacity of 350kPa may be assumed for the black boulder clay and hard brown boulder clay underlying this strata at the formation indicated.

In the area of BH15 and BH14 formations are likely to consist of stiff to hard brown boulder clay. In the area of BH4 and BH6 formations will consist of very stiff to hard black boulder clay. In the area of RC2, RC4, RC5, RC6, BH2 and BH8 formations will consist of dense gravel, indicating that 350 kPa is acceptable.

In all remaining areas formations may consist of dense gravels or the brown or black boulder clay strata. All formations should be thoroughly inspected to ensure that all material of stiff consistency or less has been removed.

Figure 5 and Figure 6 illustrate cross sections through boreholes and rotary coreholes indicating the material at the basement formation level.

Deep foundations such as pile foundations are also an option in areas of high column loads. For example 600mm ϕ and 900mm ϕ bored piles would typically support Safe Working Loads of 1750kN and 3000kN respectively with 6m to 8m embedment into the dense gravel or hard boulder clay strata.

.3 Soil Retention / Water Cut-Off

The proposed basement formation level north of GL W is +0.5m OD. The ground levels in this area vary between 6.6m OD and 11.5m OD. This equates to a dig depth of between approximately 6.5m and 11.5m. Monitored groundwater levels in this area vary between 0.8m OD and 8.1m OD. In the area of BH1, BH3, BH6, BH7 and RC3 the groundwater level varies between 2.5m OD and 8.5m OD. The variation in monitored water levels is attributed to the fact that the groundwater in the gravel strata in this area is thought to be under artesian pressure conditions and also due to the permeability difference in the clay and gravel stratas.

The proposed basement formation level south of GL W is +3.0m OD. Ground levels in this area vary between 12.0m OD and 14.2m OD. This therefore equates to a dig depth between 9.0m and 11.2m approximately. Monitored water levels in this area vary between 1.6m OD and 9m OD. In general however water levels are below formation level except in BH 15 (6.0m OD) and the previous URS water monitoring point at location gridline W/2.

The key issue in dewatering the site is to be able to control groundwater within the discharge constraints that Dublin City Council (DCC) will allow to the public services in the proximity of the site. Typically Dublin City Council will only allow in the order of 5 to 15 l/s to be discharged.

It is therefore imperative that a cut-off wall substantially limit groundwater ingress, so that the site pumping can control groundwater ingress within the constraints of the relevant discharge licenses.

It is therefore recommended that a secant pile wall be employed in the proposed development in order to provide both groundwater cut off and permanent/temporary soil retention. It is recommended that a propped bored 900mm or 600mm ϕ hard/soft secant pile wall be provided depending on the retained height of the wall. The secant pile wall should extend to –3m OD to –5m OD to achieve a 1.5m embedment into the underlying clay layers (i.e. max pile length of 19m long piles). This will provide sufficient embedment for overall stability of the pile wall and also *most probably* sufficient water cut off so that groundwater ingress may be controlled by conventional pumping regimes.

As part of the pile wall design a dewatering analysis should be carried out to determine secant pile toe levels required to achieve reasonable ground water cut off to allow conventional site pumping control groundwater ingress. Therefore the above suggested toe levels may vary dependant on the result of this analysis.

The toe level of the wall will be dependent on two major factors, firstly the overall stability of the wall which will determine the required embedment to resist over turning of the wall and secondly the groundwater ingress beneath the wall.

The computer package STAWAL from OASYS Geo has been used to determine the required toe level for the overall stability of the wall. The required toe levels are between –4.5m OD and –6.0m OD to the north of GL W and –3.0m OD to the south of GL W. The pile reinforcement can be terminated at these levels however the pile (concrete) should be extended to achieve at depth of 1.5m into the underlying deep boulder clay layers.

The depth to boulder clay in rotary core No. 5 dips off to 18m bgl.

The driller described it as gravel with clay returns. We would recommend that for groundwater cut-off the piling contractor be requested to provide a 2m embedment into boulder clay. The proposed method of piling should ensure that the piling contractor can achieve the embedment and be able to detect the presence of the layer.

The analysis also indicates that a intermediate row of props/anchors will be required to along GL 01 as per attached sketch is reduce wall deflections and pile bending moment if pile diameter of 900mm or less is employed. Pile wall deflections should be limited to approximately 25mm along the road boundary and 15mm to 20mm along all boundaries bounded by the existing stonewall, subject to reviewing the existing boundary wall.

The following design parameters may be employed for the design of the secant pile wall.

Stratum	Condition	γ (kN/m^3)	C'	Cu (kPa)	ϕ (°)	E (kPa)
Made Ground	Drained	18	0	-	30°	45,000
Dense Gravel	Drained	19	0	-	40°	90,000
Brown Boulder Clay	Drained Undrained	19 19	0 -	- 100	35° -	80,000 80,000
Black Boulder Clay	Drained Undrained	19 19	0 -	- 175	37° -	95,000 95,000

Table 5 – Design Parameters for Retaining Wall Design

.4 Pile Installation

3 No. Trial pits were excavated to determine the cobble and boulder size, shape and content in the upper gravel stratum by visual inspection. The cobles/boulders were both rounded and sub angular in shape and typically varied between 100mm and 350mm with maximum size of 500mm. The Appendix includes photographs of the same. Also a significant amount of the Shell and Auger boreholes refused at shallow depths and those that achieved deeper depths recorded significant chiselling hours due to hard drilling and boulders.

Piling contractors should be made aware of the above in their appraisal of the drillable of the ground. The following should also be considered by the piling contractor

- Recommended minimum piling plant-drilling torque of 25 Tm (Particularly for CFA) should be provided. Approx. 20Tm is the minimum that should be employed however drilling rates may be slow and the possibility exists of not achieving the required depths.

- Piling contractor to provide methodology and costs of overcoming refusal in the event that its occurs with rotary bored or CFA techniques on the boulders noted.

A trial piling operation should be carried out with the preferred piling contractor, with the proposed plant to be employed prior to award of contract. This should consist of drilling 8 bores to design toe level at various locations across the site.

5.5 Uplift

The structure and podium areas should be reviewed in the permanent condition to determine if hydrostatic water pressures will result in a net uplift pressure. It is recommended that at the south end of the site that hydrostatic level of +9m OD while at the northern end a level of +2m OD be employed. The proposed levels take into consideration hydrostatic levels recorded by URS on previous site investigation works. The variation of ±0.5m between URS and current monitored levels can be attributed to seasonal variations.

It is recommended that the following uplift pressures be designed for:

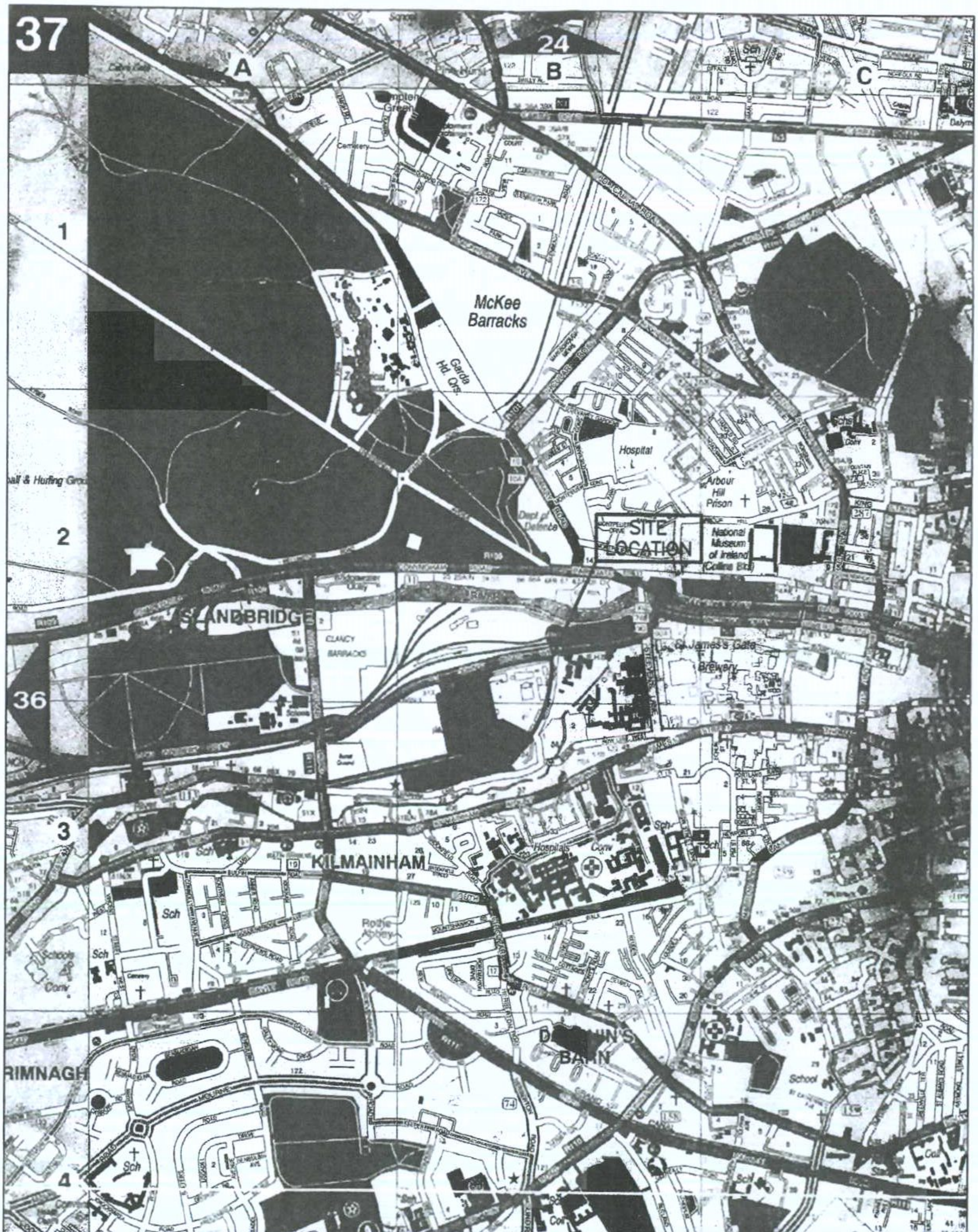
- North of Gridline H +2m OD
- North of Gridline U +8.5m OD

- South of Gridline U +9m OD

The construction sequence should also be reviewed to insure that in the temporary condition that hydrostatic water pressure are allowed for i.e. provide dewatering to maintain water levels at a depressed level to insure no uplift occurs on temporary anchors. It is recommended that if permanent or temporary anchors are employed that they be passive anchors rather than pre-stressed as it will make corrosion protection and the slab water proofing detail made easy.

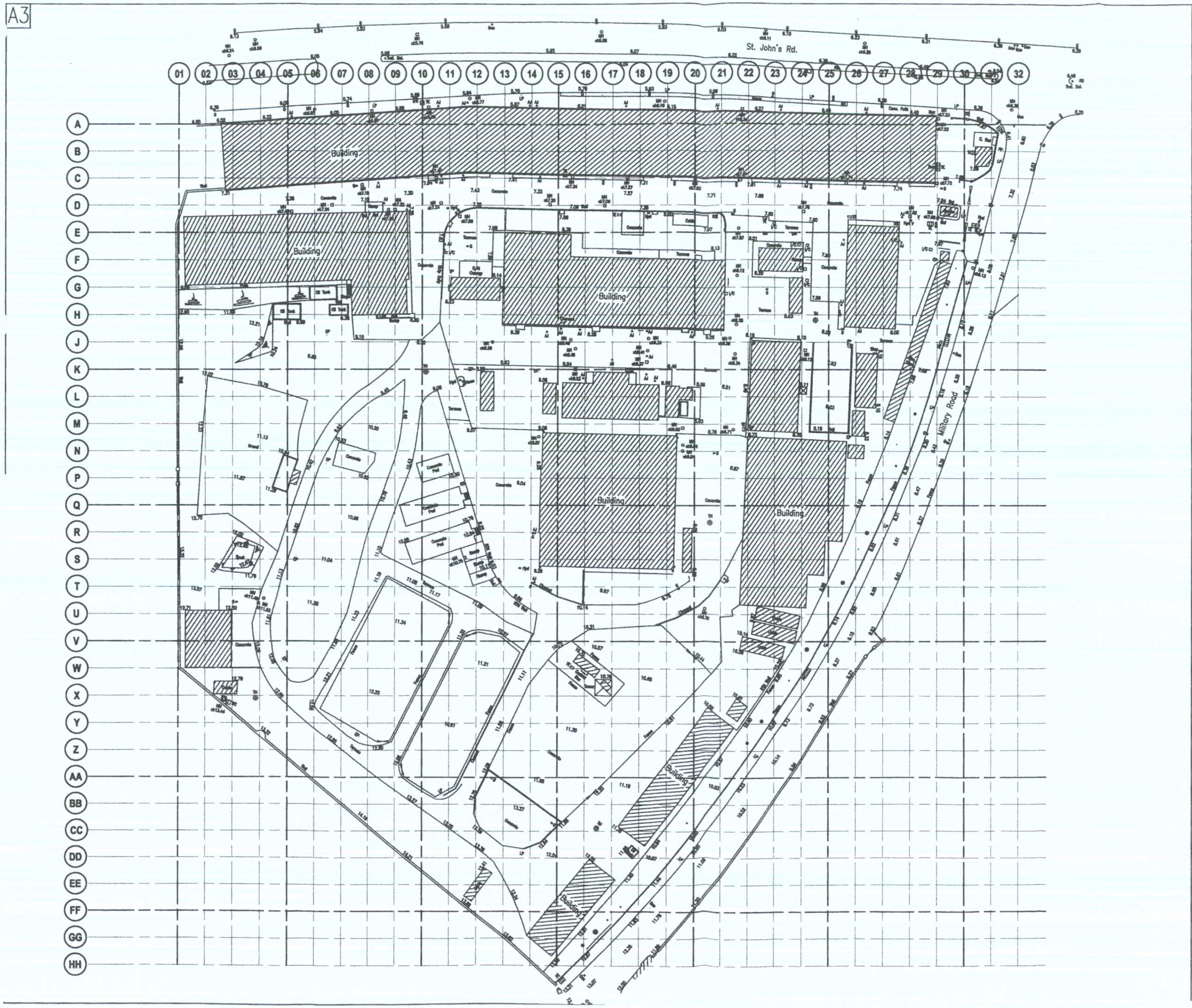
5.6 Sub-surface Concrete

pH and SO3 testing has been specified to allow concrete mix design of sub-surface concrete be carried out. Appendix G contains the data. The testing indicated that XAI grade concrete is acceptable.



BYRNE LOOBY PARTNERS
CONSULTING ENGINEERS
22 CANAL WALK
PARK WEST
DUBLIN 12
TEL : 01-6301593
FAX : 01-6301594
E-MAIL : info@blp.ie

Client: O'CONNORS SUTTON CRONIN	
Project: B462 – WESTGATE DEVELOPMENT	
Drawing Title: SITE LOCATION	
Date: 14.12.05	Scale: NTS
Drawing number: FIGURE 1	Revision:



GENERAL NOTES:

Rev	Date	Revision	by	chkd

BYRNE LOOBY PARTNERS
CONSULTING ENGINEERS
22 CANAL WALK
PARK WEST
DUBLIN 12

TEL : 01-6301593
FAX : 01-6301594
E-MAIL: INFO@BLP.IE

BLP
CONSULTING ENGINEERS

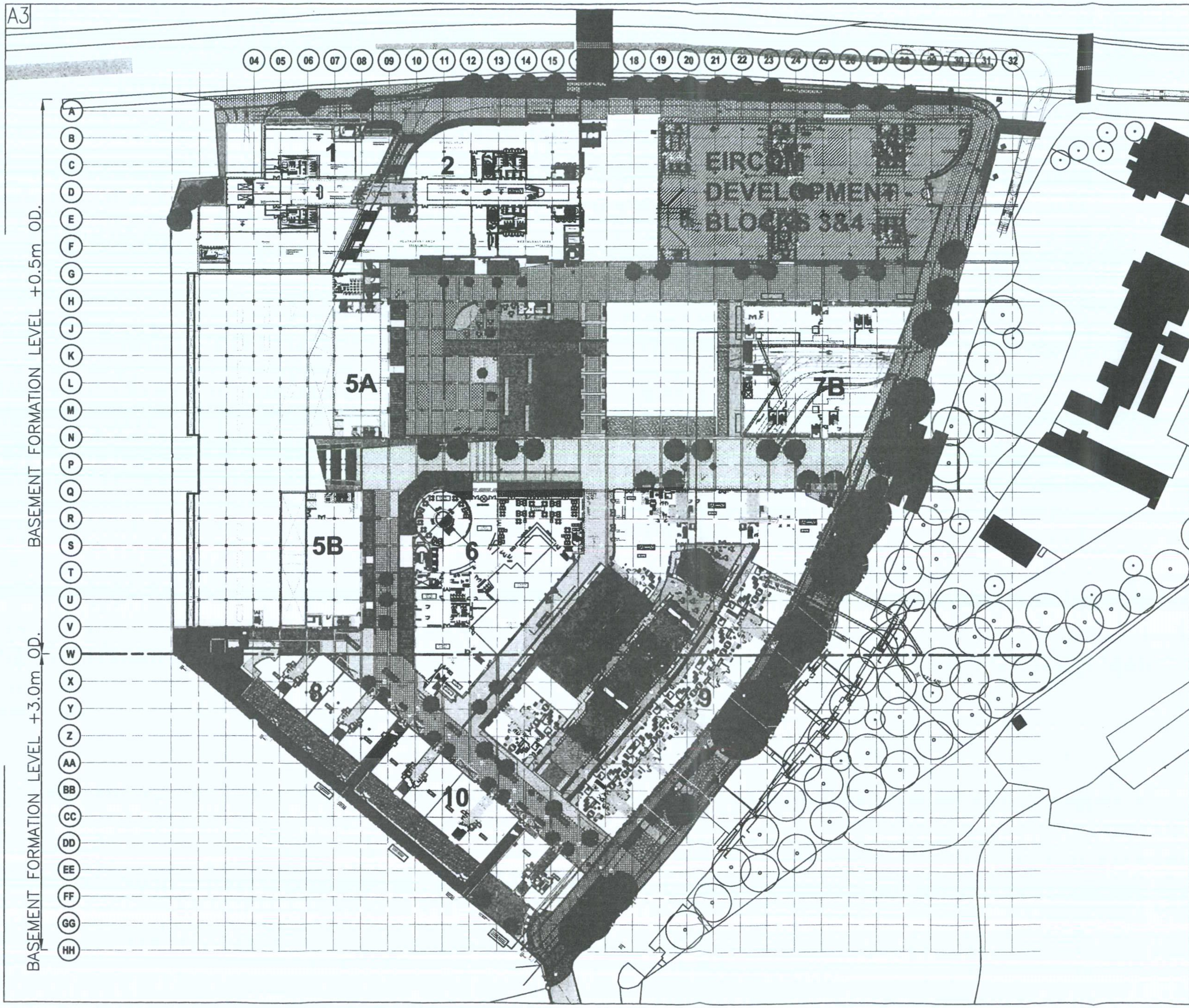
CLIENT
O'CONNOR SUTTON CRONIN

PROJECT
WESTGATE

DRAWING TITLE
TOPOGRAPHICAL SURVEY

STATUS

drawn by: LT	date: 12.12.05	scale: 1:1000	chk: ML
project no		drg.no.	rev.
B462		FIGURE 2	–



GENERAL NOTES:

Rev	Date	Revision	by	chkd

BYRNE LOOBY PARTNERS
CONSULTING ENGINEERS
22 CANAL WALK
PARK WEST
DUBLIN 12

TEL : 01-6301593
FAX : 01-6301594
E-MAIL: INFO@BLP.IE

CONSULTING ENGINEERS

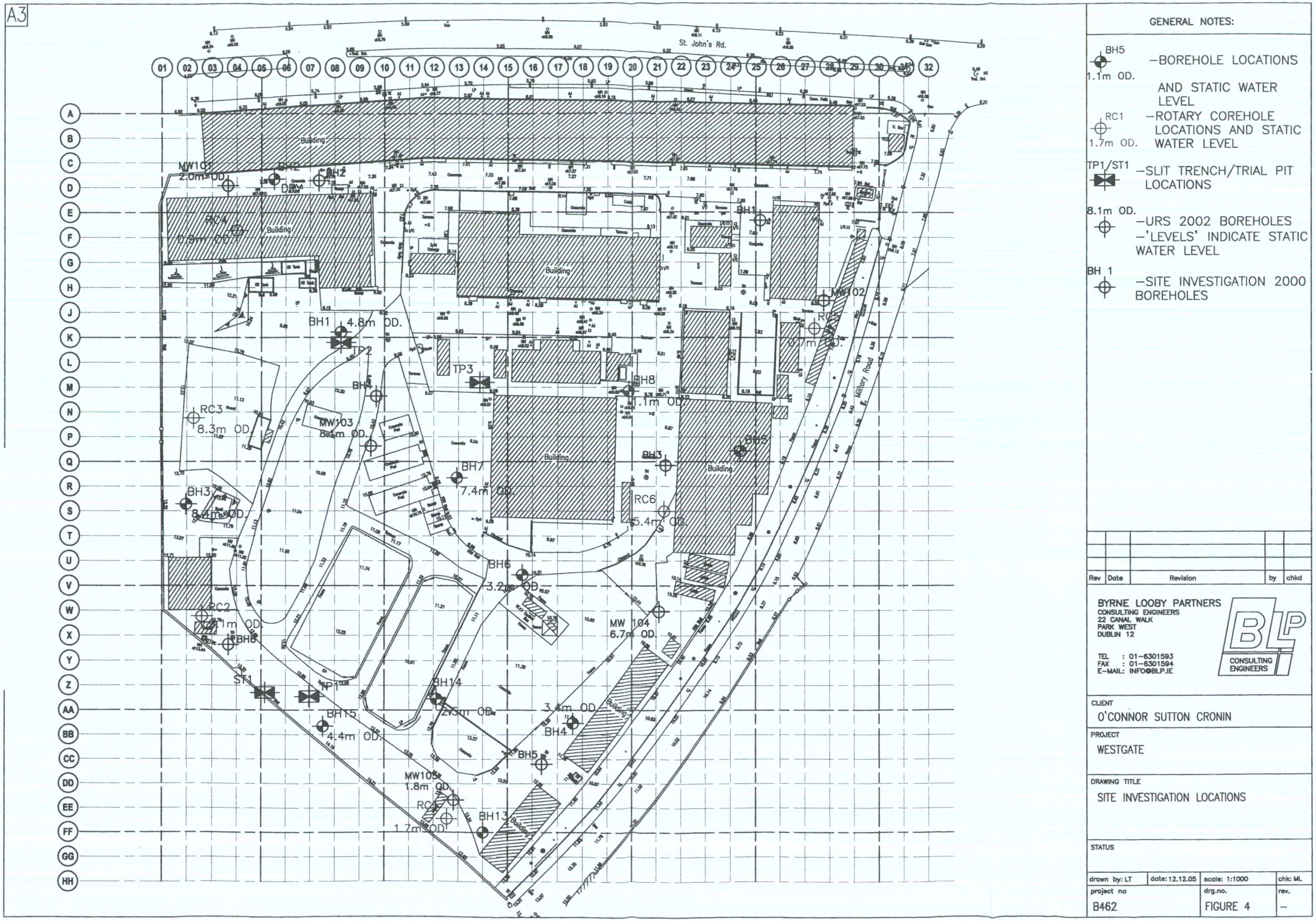
CLIENT
O'CONNOR SUTTON CRONIN

PROJECT
WESTGATE

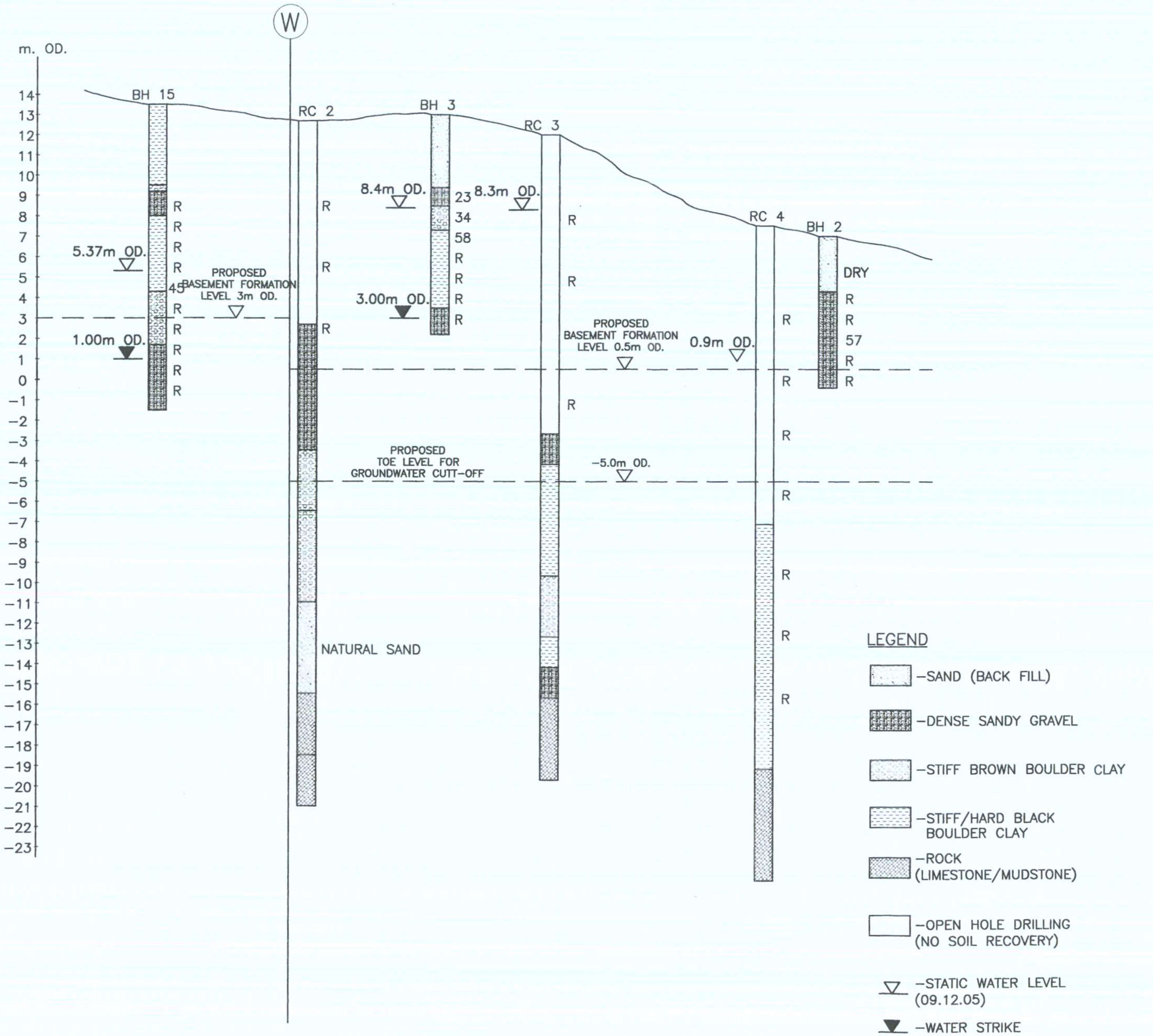
DRAWING TITLE
PROPOSED DEVELOPMENT

STATUS

drawn by: LT	date: 12.12.05	scale: 1:1000	chk: ML
project no		drg.no.	rev.
B462		FIGURE 3	–


A3
St. John's Rd.
Military Road
Building
BH1 4.8m OD.
TP2
TP3
BH4
BH8 1.1m OD.
RC3 8.3m OD.
MW103 8.1m OD.
BH7 7.4m OD.
BH3
BH5
BH3 8.4m OD.
RC6 5.4m OD.
BH6 3.2m OD.
RC2 7.1m OD.
BH6
MW 104 6.7m OD.
ST1
TP1
BH15 4.4m OD.
BH14 2.3m OD.
3.4m OD. BH4
BH5
MW105 1.8m OD.
RC1 1.7m OD.
BH13
MW101 2.0m OD.
BH2
BH12
RC4 0.9m OD.
BH1
MW102
RC5 0.7m OD.
GENERAL NOTES:
BH5 1.1m OD. –BOREHOLE LOCATIONS AND STATIC WATER LEVEL
RC1 1.7m OD. –ROTARY COREHOLE LOCATIONS AND STATIC WATER LEVEL
TP1/ST1 –SLIT TRENCH/TRIAL PIT LOCATIONS
8.1m OD. –URS 2002 BOREHOLES –'LEVELS' INDICATE STATIC WATER LEVEL
BH 1 –SITE INVESTIGATION 2000 BOREHOLES
Rev Date Revision by chkd
BYRNE LOOBY PARTNERS
CONSULTING ENGINEERS
22 CANAL WALK
PARK WEST
DUBLIN 12
TEL : 01-6301593
FAX : 01-6301594
E-MAIL: INFO@BLP.IE
BLP CONSULTING ENGINEERS
CLIENT
O'CONNOR SUTTON CRONIN
PROJECT
WESTGATE
DRAWING TITLE
SITE INVESTIGATION LOCATIONS
STATUS
drawn by: LT
date: 12.12.05
scale: 1:1000
chk: ML
project no
B462
drg.no.
FIGURE 4
rev.
–

A3



GENERAL NOTES:

Rev	Date	Revision	by	chkd
A	20.12		LT	ML

BYRNE LOOBY PARTNERS
CONSULTING ENGINEERS
22 CANAL WALK
PARK WEST
DUBLIN 12

TEL : 01−6301593
FAX : 01−6301594
E−MAIL: INFO@BLP.IE

BLP CONSULTING ENGINEERS

CLIENT
O'CONNOR SUTTON CRONIN

PROJECT
WESTGATE

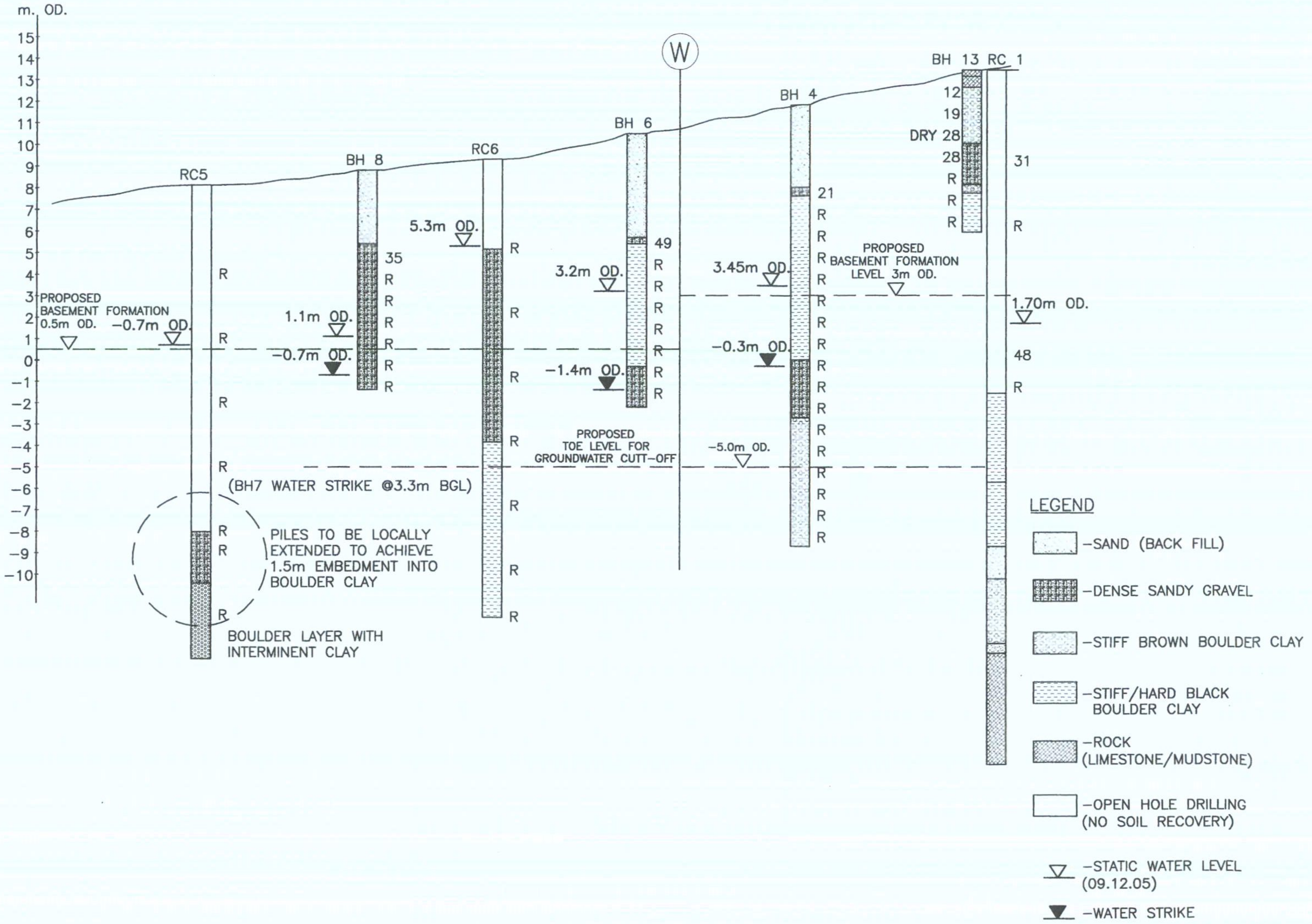
DRAWING TITLE
SECTION 1−1

STATUS

drawn by: LT	date: 12.12.05	scale: 1:200H/1000V	chk: ML
project no B462		drg.no. FIGURE 5	rev. A

A3

GENERAL NOTES:



Rev	Date	Revision	by	chkd
A	20.12		LT	ML

BYRNE LOOBY PARTNERS
CONSULTING ENGINEERS
22 CANAL WALK
PARK WEST
DUBLIN 12

TEL : 01−6301593
FAX : 01−6301594
E−MAIL: INFO@BLP.IE

BLP CONSULTING ENGINEERS

CLIENT
O'CONNOR SUTTON CRONIN

PROJECT
WESTGATE

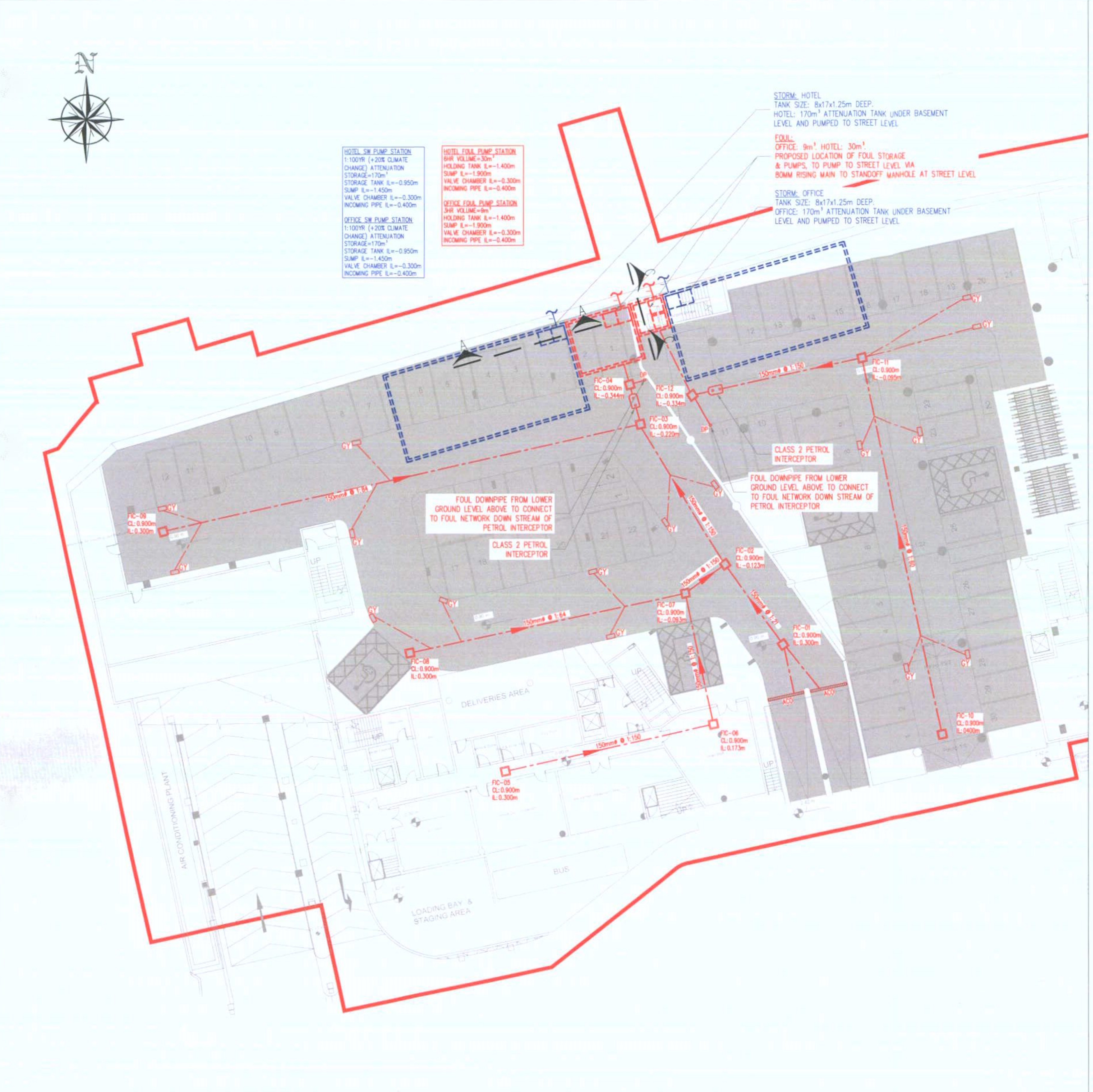
DRAWING TITLE
SECTION 2−2

STATUS

drawn by: LT	date: 12.12.05	scale: 1:200H/1000V	chk: ML
project no		drg.no.	rev.
B462		FIGURE 6	A

APPENDIX 8A

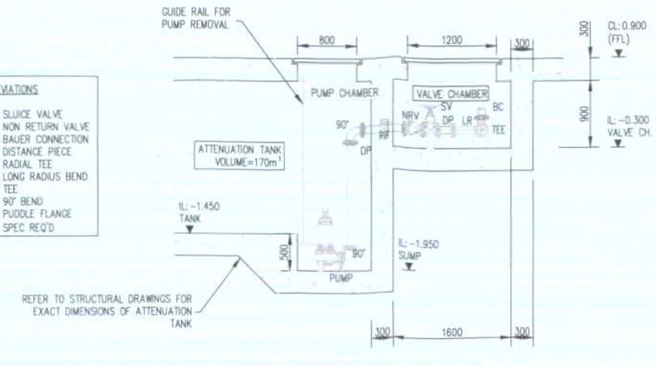
PROPOSED STORM WATER SYSTEM



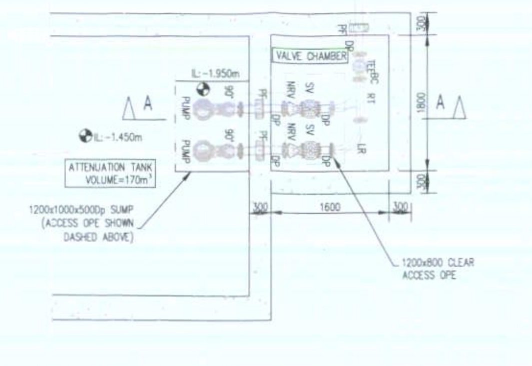
PROPOSED BASEMENT -1 DRAINAGE LAYOUT.
SCALE 1:250

ABBREVIATIONS

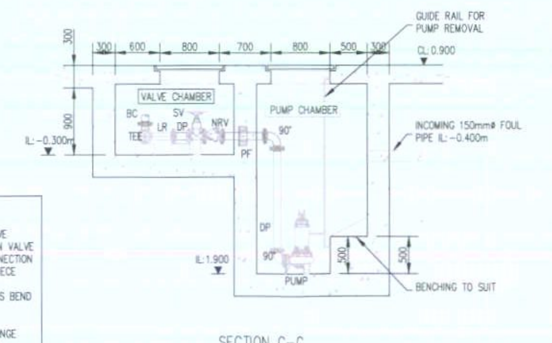
SV	SLUICE VALVE
NRV	NON RETURN VALVE
BC	BAUER CONNECTION
DP	DISTANCE PIECE
RT	RADIAL TEE
LR	LONG RADIUS BEND
TEE	TEE
90°	90° BEND
PF	PUDDLE FLANGE
PUMP	PUMP SPEC REQ'D



SURFACE WATER ATTENUATION SECTION A-A
SCALE 1:50



PLAN OF ATTENUATION TANK, PUMP SUMP AND VALVE CHAMBER
SCALE 1:50



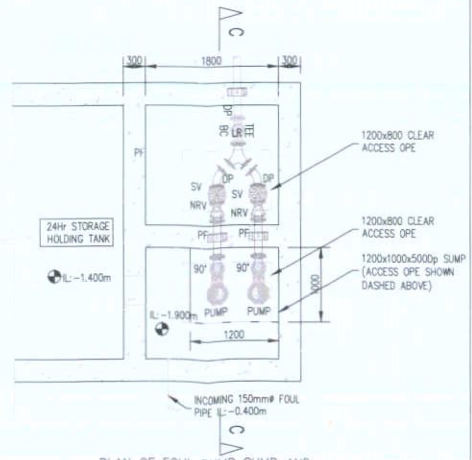
SECTION C-C
SCALE 1:50

ABBREVIATIONS

SV	SLUICE VALVE
NRV	NON RETURN VALVE
BC	BAUER CONNECTION
DP	DISTANCE PIECE
RT	RADIAL TEE
LR	LONG RADIUS BEND
TEE	TEE
90°	90° BEND
PF	PUDDLE FLANGE
PUMP	PUMP SPEC REQ'D

NOTE:

1. ALL PUMPING STATION PIPE WORK, FITTINGS AND VALVES ETC. TO BE ALL FLANGED DUCTILE IRON IS EN 598 WITH PN-16 FLANGES TO BE EN 1092-1.
2. ALL PUMP, VALVE AND TANK COVERS TO BE D400 LOCKABLE SEALED COVERS FLUSH WITH ROOF SLAB.
3. REFER TO STRUCTURAL DRAWINGS FOR EXACT POSITION OF WALLS, RC, THICKNESSES ETC.



PLAN OF FOUL PUMP SUMP AND VALVE CHAMBER
SCALE 1:50

LEGEND:

- PROPOSED FOUL RISING MAIN
- PROPOSED SURFACE WATER RISING MAIN
- PROPOSED ROAD GULLY
- PROPOSED FOUL INSPECTION CHAMBER
- PROPOSED CLASS 2 PETROL INTERCEPTOR

PUMPING PLANT: -
SPECIFIC MINIMUM REQUIREMENTS FOR PUMPING PLANT ARE AS FOLLOWS:

1. PUMPING PLANT SHOULD BE OF FAILSAFE DESIGN.
2. ALL PLANT AND EQUIPMENT TO BE SUITABLY EX-RATED IN ACCORDANCE WITH THE HAZARDOUS AREA CLASSIFICATION FOR THE PUMPING STATION SITE.
3. PUMPING PLANT TO BE DUTY AND STANDBY ARRANGEMENT OR DUTY/ASSIST AND STANDBY ARRANGEMENT;
4. PUMPS TO BE SUBMERSIBLE PUMPS WITH AUTOMATIC DECOUPLING ARRANGEMENTS COMPLETE WITH TWIN GUIDE RAILS, EASY LIFT, ETC.;
5. PUMPING PLANT TO BE OF PROVEN TRACK RECORD; INCLUDING COOLING JACKETS.
6. AUTOMATIC SELECTION ROTATION OF THE DUTY/STANDBY OR DUTY/ASSIST/STAND-BY PUMPS TO BE PROVIDED ON AN HOURS RUN BASIS WITH MANUAL OVER-RIDE;
7. PUMPS TO BE SIZED FOR A MINIMUM OF 3 TIMES DWF, IF STORAGE PROVIDED, AND 6 TIMES DWF OTHERWISE;
8. PUMPS TO BE SUITABLE FOR PUMPING UNSCREENED WASTEWATER CONTAINING FIBROUS MATERIAL WITH A MINIMUM SOLIDS PASSAGE SIZE OF 100MM. PUMPS CONNECTED TO SMALL DIAMETER RISING MAINS TO BE FITTED WITH AN ANTI-BLOCKAGE/ANTI-RAGGING SYSTEM LINKED TO THE POWER AMP RECORDING SYSTEM;
9. PUMPS TO HAVE A MINIMUM DISCHARGE SIZE OF 80MM;
10. PUMP CONTROL TO BE VIA ULTRASONIC LEVEL TRANSDUCERS, LOCATED ABOVE LIQUID LEVEL, IN AN EASILY ACCESSIBLE LOCATION;
11. THE PUMP GUIDE SYSTEM TO ALLOW THE PUMP UNITS TO BE AUTOMATICALLY COUPLED TO THE OUTLET PIPEWORK AND HELD IN PLACE BY ITS OWN WEIGHT;
12. THE GUIDE SYSTEM TO ALLOW THE PUMP UNITS TO BE LIFTED TO THE TOP OF THE WET WELL WITHOUT THE NEED TO UNDO ANY FIXING ARRANGEMENTS OR TO ENTER THE WET WELL;
13. ANCHOR BOLTS SHALL BE STAINLESS STEEL, STAINLESS STEEL AND GALVANISED STEEL SURFACES SHALL NOT COME INTO CONTACT WITH EACH OTHER;
14. PUMPS TO BE MOUNTED ON A CAST IRON COUPLING/DUCK-FOOT PEDESTAL, WITH AUTOMATIC DECOUPLING ARRANGEMENTS;
15. PUMP ARRANGEMENT TO ALLOW EASY INSTALLATION AND SPEEDY REMOVAL FROM THE SUMP WITHOUT NEED FOR OPERATOR ENTRY TO THE SUMP;
16. PUMPS GUIDE RAILS TO BE OF GALVANISED MILD STEEL OR STAINLESS STEEL (GRADE 316);
17. PUMPS TO BE PROVIDED WITH CERTIFIED, STAINLESS STEEL LIFTING CHAIN (DESIGNED TO BS4942), SUITABLY SIZED AND FIT FOR PURPOSE, WITH 8MM THICK LINKS, AT LEAST, AND LARGE LINKS A NOT MORE THAN 1M INTERVALS;
18. SPARE CERTIFIED STAINLESS STEEL CHAINS, OF SIMILAR CAPACITY TO THE INSTALLED CHAIN UNIT, SHALL BE PROVIDED TO FACILITATE REGULAR INSPECTION/REPLACEMENT OF THE LIFTING CHAIN;
19. ANCHOR BOLTS TO BE OF STAINLESS STEEL OR GALVANISED STEEL SUITABLE FOR THE MATERIAL BEING RETAINED (NO CONTACT BETWEEN STAINLESS STEEL AND GALVANISED STEEL);
20. DISCHARGE PIPEWORK WITHIN THE WET WELL TO BE COMPLETE WITH BENDS, TEE-PIECES, FITTINGS, ETC. TO LINK THE WET WELL PIPEWORK TO THE VALVE CHAMBER PIPEWORK;
21. PIPEWORK WITHIN THE VALVE CHAMBER TO INCORPORATE ISOLATION VALVES (ONE PER PUMP INSTALLED), NON-RETURN VALVES (ONE PER PUMP INSTALLED), BENDS, TEE-PIECES, ETC.;
22. NON-RETURN VALVES TO HAVE REMOVABLE COVERS, DUCTILE IRON BODY WITH RESILIENT SEATED DISC AND STAINLESS STEEL HINGE PIN, COMPLETE WITH EITHER A BALL WEIGHT OR LEVER ARM AND WEIGHT;
23. BENDS TO BE SWEPT/SLOW BENDS TO MINIMISE BLOCKAGES AND PIPE FRICTION LOSSES; SLUICE VALVES TO BE PROVIDED WITH REMOVABLE HAND-WHEELS;
24. FLANGE ADAPTORS TO BE PROVIDED TO PERMIT EASE OF REMOVAL OF VALVES FROM THE PIPEWORK;
25. ALL PIPEWORK AND VALVES TO BE OF DUCTILE IRON, PN-16, TO 85 4772 AND EN 598, SUITABLE FOR USE WITH SEWAGE;
26. PUMP MOTORS TO BE HIGH EFFICIENCY WITH CLASS F INSULATION AND LP68 RATING; PUMP EFFICIENCY SHALL BE MAINTAINED WITHIN 15% OF ITS MAXIMUM EFFICIENCY OVER THE WHOLE OF THE SPECIFIED DUTY RANGE.
27. MOTOR AND MOTOR HOUSING TO BE BOLTED TO THE PUMP HOUSING, SHRINK OR PRESS FIT ASSEMBLIES WILL NOT BE ACCEPTED;
28. MOTORS MUST INCLUDE STATOR OVER-TEMPERATURE PROTECTION IN THE FORM OF THERMISTORS EMBEDDED IN EACH PHASE OF THE WINDINGS, OVER-TEMPERATURE PROTECTION SHOULD AUTOMATICALLY RE-SET WHEN THE TEMPERATURE RETURNS TO NORMAL;
29. PUMPS SHALL HAVE A MAXIMUM SPEED OF 1500RPM. PUMP CHARACTERISTICS SHALL BE STABLE, NON-OVERLOADING AND SHALL BE SUCH THAT THE PUMPS SHALL OPERATE AS CLOSE TO MAXIMUM EFFICIENCY AT THE DESIGN POINT.
30. PUMPS TO BE PROVIDED WITH INDICATOR PLATES PROVIDING INFORMATION FOR THE PUMP, MOTOR, ETC. A DUPLICATE STAINLESS STEEL PLATE TO BE PROVIDED AND MOUNTED IN THE CONTROL PANEL.
31. WHERE VALVES ARE LOCATED IN DRYWELL SUMP, VALVE EXTENSIONS HANDLES TO BE SUPPLIED (INCLUDING THE CORING OF THE CONCRETE COVER ABOVE) WITH 80MM HOLE.
32. PUMP CHAIN TO BE STAINLESS GRADE STAMPED AND CERTIFIED WITH OVAL SHAPED MASTER RINGS EVERY ONE METER IN LENGTH.
33. ALL EMERGENCY STOPS TO BE LOCATED OVER GROUND ON STAINLESS STEEL PILLARS IN THE LOCATION /AREA OF PUMP SUMPS.
34. ALL PANELS TO BE LABELLED ACCORDINGLY.
35. ALL PIPE WORK TO BE LABELLED WITH DIRECTIONAL ARROWS AND LABELS STATING WHAT IS IN PIPE WORK .E.G. FERRIC/FOUL ETC.
36. ALL CHAMBERS TO HAVE SIGNAGE STAMPED ON THEM STATING WHAT THEY ARE E.G. FOUL SUMP/STORM SUMP.
37. NON RETURN VALVES TO BE GOOD QUALITY FLAP TYPE FOR SEWAGE NOT WATER.

DCC PLAN NO: 4610/22
RECEIVED: 04/08/2022

H087

PLANNING DRAWING.
NOT FOR CONSTRUCTION.
ALL LEVELS GIVEN ARE RELATIVE TO ORDNANCE DATUM.
THIS DRAWING HAS BEEN ISSUED FOR INFORMATION PURPOSES ONLY AND MUST NOT BE USED FOR CONSTRUCTION UNDER ANY CIRCUMSTANCES

NOTES
1. For setting out refer to Architect's drawings.
2. This drawing to be read in conjunction with all other Architectural and Engineering drawings and all other relevant drawings and Specifications.
3. DO NOT SCALE THIS DRAWING. Use figured dimensions only.
4. No part of this document may be reproduced or transmitted in any form or stored in any retrieval system of any nature without the written permission as copyright holder except as agreed for use on the project for which the document was originally issued.
5. Ordnance Survey Ireland Licence Number EN 0074022

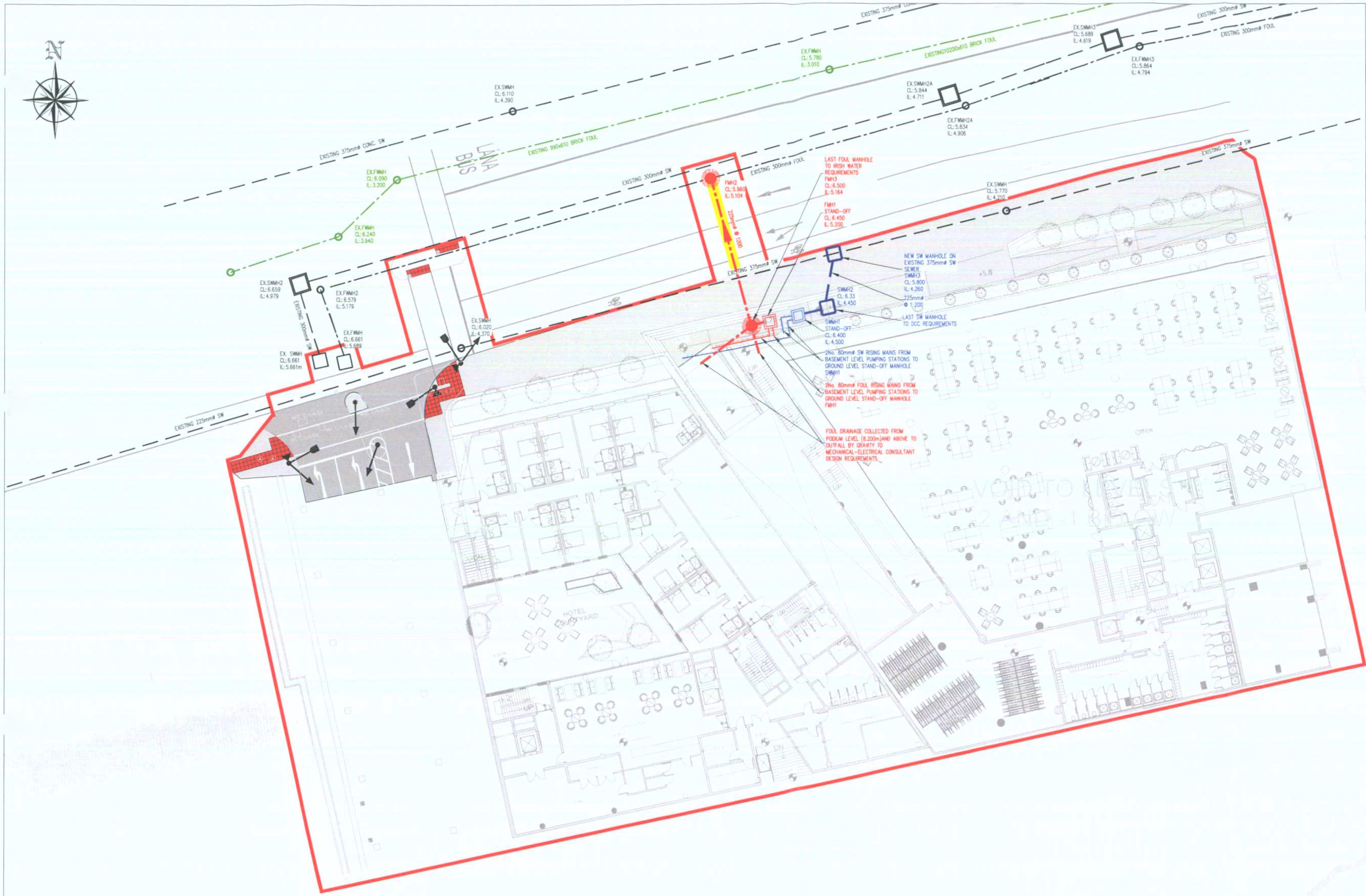
Rev. No.	Date	Revision Note	Drn. By	Chkd. By
P1	13.05.2022	ISSUED FOR PLANNING	JS	GF
P2	20.07.2022	ARCHITECT'S LAYOUT REVISED	JS	GF

Architect	Reddy Architecture
Project	HSQ COMMERCIAL SITE DEVELOPMENT.
Title	PROPOSED DRAINAGE LAYOUT BASEMENT LEVEL
Dwg. No.	HSQ-CSC-XX-XX-DR-C-0201

Date	Drn by	Chkd by	Aprvd by	Scale	Revision
FEB 2022	DD	OS	OS	AS SHOWN @ A1	P2

CS Consulting Group
DUBLIN | LONDON | LIMERICK
Head Office
19-22 Dame Street, Dublin 2
T: +353 (0)1 5480863 F: +353 (0)1 9011355
e: info@csconsulting.ie
w: www.csconsulting.ie

NSAI Certified
Quality I.S. EN ISO 9001:2008
Environment I.S. EN ISO 14001:2004
Energy I.S. EN ISO 50001:2011
Health & Safety OHSAS 18001:2007



LEGEND:

EXISTING FOUL SEWER	EX. FMH
EXISTING SURFACE WATER SEWER	EX. SWMH
EXISTING COMBINED SEWER	
PROPOSED FOUL SEWER	FMH1
PROPOSED FOUL SEWER CONCRETE SURROUNDED	
PROPOSED FOUL SEWER STAND-OFF MANHOLE	
PROPOSED SURFACE WATER SEWER	SWMH1
PROPOSED STORM WATER STAND-OFF MANHOLE	
PROPOSED FOUL RISING MAIN	
PROPOSED SURFACE WATER RISING MAIN	
EXISTING GROUND LEVEL	
PROPOSED LEVEL	12.000m

PROPOSED LOWER GROUND DRAINAGE LAYOUT.

SCALE 1:200

H087

PLANNING DRAWING.
NOT FOR CONSTRUCTION.
ALL LEVELS GIVEN ARE RELATIVE TO ORDNANCE DATUM.
THIS DRAWING HAS BEEN ISSUED FOR INFORMATION PURPOSES ONLY AND MUST NOT BE USED FOR CONSTRUCTION UNDER ANY CIRCUMSTANCES

NOTES
1. For setting out refer to Architect's drawings.
2. This drawing to be read in conjunction with all other Architectural and Engineering drawings and all other relevant drawings and Specifications.
3. DO NOT SCALE THIS DRAWING. Use figured dimensions only.
4. No part of this document may be reproduced or transmitted in any form or stored in any retrieval system of any nature without the written permission as copyright holder except as agreed for use on the project for which the document was originally issued
5. Ordnance Survey Ireland Licence Number EN 0074022

Rev. No	Date	REVISION NOTE	Drn. By	Chkd. By
P1	13.05.2022	ISSUED FOR PLANNING	JS	GF
P2	20.07.2022	ARCHITECT'S LAYOUT REVISED	JS	GF

Architect	Reddy Architecture
Project	HSQ COMMERCIAL SITE DEVELOPMENT.
Title	PROPOSED DRAINAGE LAYOUT LOWER GROUND FLOOR
Dwg. No.	HSQ-CSC-XX-XX-DR-C-0202

Date	Drn by	Chkd by	Apprvd by	Scale	Revision
FEB 2022	DD	OS	OS	1:200 @ A1	P2

CS Consulting Group
DUBLIN | LONDON | LIMERICK
Head Office
19-22 Dame Street, Dublin 2
T: +353 (0)1 5480863 F: +353 (0)1 9011355
e: info@csconsulting.ie
w: www.csconsulting.ie

NSAI Certified

Quality	I.S. EN ISO 9001:2008
Environment	I.S. EN ISO 14001:2004
Energy	I.S. EN ISO 50001:2011
Health & Safety	OHSAS 18001:2007

APPENDIX 8B

SITE SPECIFIC FLOOD RISK ASSESSMENT

CS CONSULTING

GROUP

DUBLIN | LONDON | LIMERICK

Site-Specific Flood Risk Assessment

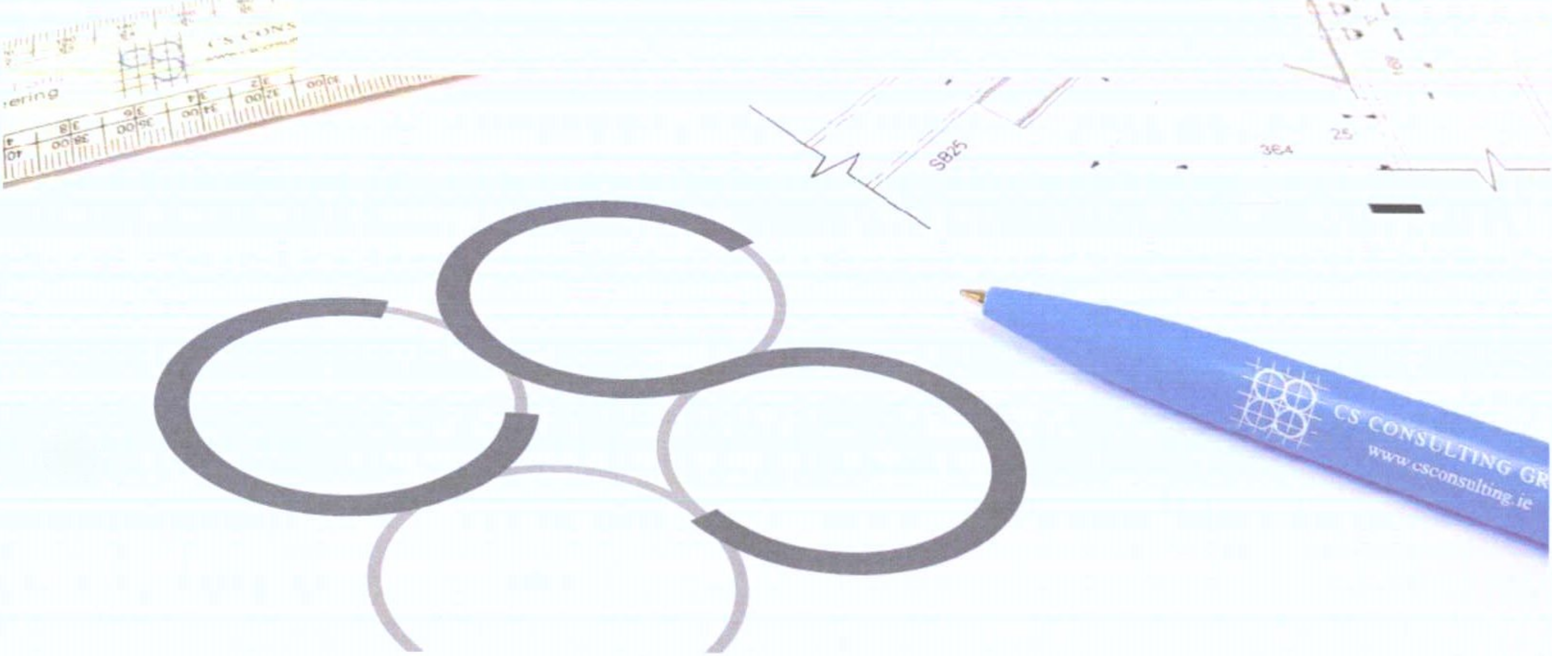
Office and Hotel Development

Heuston South Quarter, St. John's Road West, Kilmainham, Dublin 8

Client: HPREF HSQ Investments Ltd.

Job No. H087

July 2022



CS CONSULTING
GROUP
DUBLIN - LONDON - LIMERICK

SITE-SPECIFIC FLOOD RISK ASSESSMENT

OFFICE AND HOTEL DEVELOPMENT

HEUSTON SOUTH QUARTER, ST. JOHN'S ROAD WEST, KILMAINHAM, DUBLIN 8

CONTENTS

File Location: Job-H087\B_Documents\C_Civil\A_CS Reports\SSFRA

BS 1192 FIELD	**HSQ–CSC–ZZ–XX–RP–C–0202-P3**				
Job Ref.	Author	Reviewed By	Authorised By	Issue Date	Rev. No.
H087	PS	GF	RFM	19.07.2022	P3
H087	PS	GF	RFM	05.04.2022	P2
H087	PS	GF	RFM	17.02.2022	P1
H087	PS	GF	RFM	02.02.2022	P0

1.0 INTRODUCTION

Cronin & Sutton Consulting Engineers (CS Consulting) have been commissioned by HPREF HSQ Investments Ltd. to prepare a Site-Specific Flood Risk Assessment (SSFRA) to accompany a planning application for a proposed office and hotel development at Heuston South Quarter, St. John's Road West, Kilmainham, Dublin 8.

This report reviews the site's vulnerability to a variety of flooding mechanisms, in accordance with published guidelines, and the need for any flood risk mitigation measures. The sequential approach, as outlined by the national flood guidelines, was followed in the preparation of this report.

In preparing this report, CS Consulting has made reference to the following:

- Dublin City Development Plan 2016–2022
  (including Strategic Flood Risk Assessment)
- Greater Dublin Regional Code of Practice for Works
- Office of Public Works Flood Maps
- Department of the Environment Flooding Guidelines
- Geological Survey of Ireland Maps
- Local Authority Drainage Records

The SSFRA is to be read in conjunction with the engineering drawings and documents submitted by CS Consulting and with all relevant further documentation submitted by other members of the design team as part of this planning submission.

Note on sources:

- The Office of Public Works flood database (www.floodinfo.ie) was accessed in September 2021. All historic, fluvial, and tidal flooding information from this source that is employed in this report is based on the data from this time.

- The GSI website (www.gsi.ie), from which geological and hydrogeology data has been sourced for use in this report, was accessed in September 2021.

2.0 SITE LOCATION AND PROPOSED DEVELOPMENT

2.1 Site Location

The proposed development is located on St. John's Road West at the Heuston South Quarter (HSQ) complex in Dublin 8, within the administrative jurisdiction of Dublin City Council. The planning application boundary encloses an area of approx. 0.62ha. The development site is bounded to the west by the gardens of the Royal Hospital Kilmainham, to the north by St. John's Road West, to the east by an existing office building, and to the south by a further partially developed section of the wider HSQ complex that is also in the applicant's ownership.

Figure 1 – Location of proposed development site
(map data & imagery: EPA, OSi, NTA, OSM Contributors, Google)

The location of the proposed development site is shown in **Figure 1**; the indicative extents of the development site, as well as relevant elements of the surrounding road network, are shown in more detail in **Figure 2**.

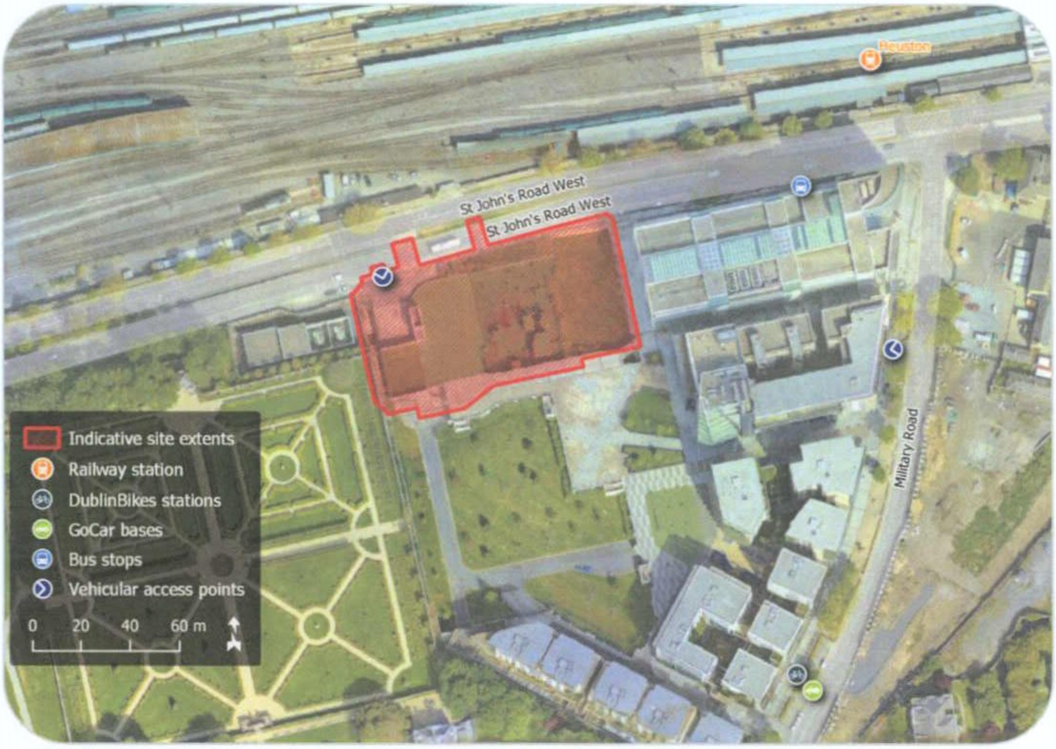


Figure 2 – Site extents and environs
(map data & imagery: NTA, DCC, OSi, OSM Contributors, Google)

2.2 Existing Land Use

The subject site is brownfield, comprising a partially developed section of the Heuston South Quarter (HSQ) complex. Some surface level internal roads are present on the site, which benefits from the existing established HSQ vehicular accesses on St. John's Road West (R148) and Military Road. The site has been landscaped as an interim measure to improve its aesthetics pending its complete development. There is already an established road, pedestrian and cycle network in the vicinity of the site.

2.3 Description of Proposed Development

The proposed development will provide a mixed use commercial development comprising of a hotel (241 no. bedrooms) and an office block delivering a cumulative Gross Floor Area (GFA) of 32,602, inclusive of basement area. The proposed development consists of:

- Site clearance and localised demolitions to remove part of the podium and Basement Level -1 reinforced concrete slabs at the interface of the proposed hotel and office blocks, together with the incorporation of part of the existing basement level structure extending to approximately 4,228 sq.m (GFA).
- The proposed basement will be integrated within the existing basement levels serving the wider HSQ development and will be accessed from the existing vehicular ramped accesses/egresses onto/off St. John's Road West and Military Road to the north and east, respectively. The proposed basement area is split into two areas to provide a dedicated Hotel Basement area of approximately 2,132 sq.m (GFA) and an Office basement area of 2,096 sq.m (GFA).
- The construction of a 5-storey hotel (over lower ground and basement levels) to provide 238 no. bedrooms. At basement level provision is made for 24 no. car parking spaces; 2 no. motorcycle spaces together with plant and storage rooms. A waste storage area with dedicated loading bay / staging area is provided along with dedicated set-down area for deliveries. A dual-purpose service bay is also provided at basement level with modifications to existing line markings to the basement parking area to accommodate the development. At Lower Ground floor level provision is made for 14 no. Bedrooms; Conference Room; Kitchen and Staff facilities and Changing Rooms / WCs plus ancillary Gym. This floor is arranged around an internal courtyard space. Provision is made at Podium level for 19 no. Bedrooms; Dining Area and Foyer with entrance at the South-Eastern corner of the

building onto a new laneway separating the proposed hotel and office building. Provision is made at the south-western corner at podium level for an ESB sub-station / switch room and 15 no Sheffield type bicycle stands are provided for the hotel and the retail / café unit, providing storage space for 30 no. bicycles. A total of 205 no. bedrooms are provided at the upper levels (above podium level). The top floor of the hotel (4th floor) has a splayed setback to provide a west facing roof terrace. An ancillary hotel bar (118 sq.m) opens onto this roof terrace.

- The construction of a 12-storey (over lower ground and basement levels) office building to the east of the proposed hotel building to provide 19,474 sq.m of office floorspace (GFA) from lower ground floor level and above. Provision is made at basement level for 30 no. car parking spaces; 2 motorcycle spaces and 120 no. bicycle storage spaces together with plant and storage rooms. Provision is made for a further 196 no. bicycle storage spaces at Lower Ground floor level plus changing rooms (including showers). At podium level 2 no. ESB sub-stations and switch rooms are proposed. The foyer and entrance is provided at the southern end of the building at Podium level along with a Retail/Café unit of 208 sq.m at the South-Western corner of the building. The building is setback at 4th floor level to provide a west facing roof terrace. Splayed setbacks to the southern and eastern elevations at the 11th floor level forms a roof terrace that wraps around the South-Eastern corner of the building. Plant is provided at rooftop level that is enclosed by curved louvred screens and PV panels.
- Works proposed along the St John's Road West frontage include the omission of the existing left-turn filter lane to the vehicular ramped access to the HSQ development and re-configuration of the pedestrian crossings at the existing junction together with the re-configuration of the existing pedestrian crossing over the westbound lanes of St. John's Road West leading to an existing pedestrian refuge

island and re-alignment of the existing footpath along the site frontage onto St John's Road West to tie into the reconfigured junction arrangement.

- Drainage works proposed include the provision of 2 no. below basement surface water attenuation tanks with duty/stand-by arrangement pump sumps and associated valve chambers, and 2 no. below basement foul pump sumps with duty/stand-by arrangement and 24hr emergency storage and associated valve chambers. New foul drainage and stormwater drainage connections are proposed to existing foul and storm sewers in St. John's Road West with associated site works.
- Hard and soft landscaping works are proposed at lower ground level along St John's Road West and at podium level to provide for the extension and completion of the public plaza to the south of the proposed Office Block and the provision of a new pedestrian laneway connecting St John's Road West with the public plaza at podium level.

3.0 EXISTING INTERNAL DEVELOPMENT INFRASTRUCTURE

The original masterplan for the entire HSQ development was granted planning permission by Dublin City Council in 2003 (DCC Ref 2656/03). As part of this planning grant, the developer was obliged to constructed infrastructure to serve the entire development at the outset. This included new foul and surface water sewers along St. John's Road West and Military Road. The new 300mm foul sewer connected to a public combined sewer at Dr. Steeven's Hospital. The new 300mm surface water sewer connected to the existing Camac Culvert, also adjacent to Dr. Steeven's Hospital. Finally, a new 450mm watermain was extended down Military Road, from an existing line at Bow Lane, as part of these initial infrastructure works (see **Appendix H**).

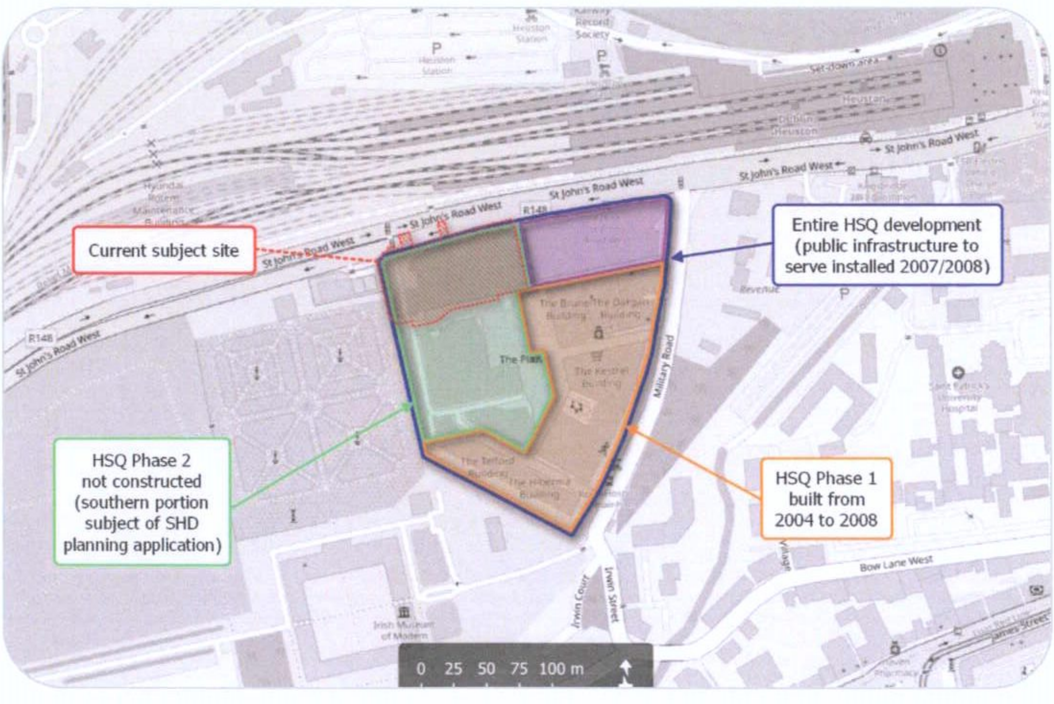


Figure 3 – Indicative location of early works
(background map imagery: OSM Contributors)

A number of subsequent applications were approved by DCC, based on connecting into the infrastructure noted in the 2003 masterplan. The majority of the east of the site was constructed prior to the financial crash in 2008.

During the basement construction works for the completed portions of the development, much of the foul, surface and watermain infrastructure was constructed to connect to the newly installed public infrastructure.

4.0 LEVEL OF SERVICE

There is an existing inherent risk of any flood event occurring during any given year. Typically, this likelihood of occurrence was traditionally expressed as a 1-in-100 chance of a 100-year storm event happening in any given year.

A less ambiguous expression of probability is the Annual Exceedance Probability (AEP), which may be defined as the probability of a flood event being exceeded in any given year. Therefore a 1-in-100-year event has a 1% AEP; similarly, a 100% AEP can be expressed as a 1-in-1-year event.

The Planning System and Flood Risk Management, Guidelines for Planning Authorities (Flood Risk Management Guidelines), published in 2009, set out the best practice standards for flood risk assessment in Ireland. These are summarised in **Table 1** below (Table 8.1 from Flood Risk Management Guidelines document).

Table 1 – Summary of Level of Service: Flooding Source

Development Category	Flooding Source		
	Drainage	River	Tidal/Coastal
Residential	1% AEP	0.1% AEP	0.1% AEP
Commercial	1% AEP	1% AEP	0.5% AEP
Water-compatible (docks, marinas)	-	>1% AEP	>0.5% AEP

Under these guidelines, a proposed development site has first to be assessed to determine the flood zone category it falls under.

It is a requirement of Dublin City Council, the *Greater Dublin Strategic Drainage Study* (DCC 2005), and the Flood Risk Management Guidelines that the predicted effects of climate change be incorporated into any proposed design. **Table 2** below indicates the predicted climate change variations.

Table 2 – Predicted climate change variations

Design Category	Predicted Impact of Climate Change
Drainage	20% Increase in rainfall
Fluvial (river flows)	20% Increase in flood flow
Tidal / Coastal	Minimum Finished Floor Level 4.0 – 4.15m AOD

The flooding guidelines define three distinct areas of combined flood risk: Zones A, B, and C. These are described below.

- **Zone A** – High Probability of Flooding. Where the average probability of flooding from rivers and sea is highest (greater than 1% AEP for fluvial flooding or 0.5% AEP for tidal flooding).
- **Zone B** – Moderate Probability of Flooding. Where the average probability of flooding from rivers and sea is moderate (between 0.1% AEP and 1% AEP for fluvial flooding, and between 0.1% AEP and 0.5% AEP for tidal flooding).
- **Zone C** – Low Probability of Flooding. Where the probability of flooding from rivers and sea is lowest (less than 0.1% AEP for both fluvial and coastal flooding).

A review of DCC flood risk mapping shows the subject site to be located in **Flood Zone C**. See **Appendix B** and **Figure 4**.

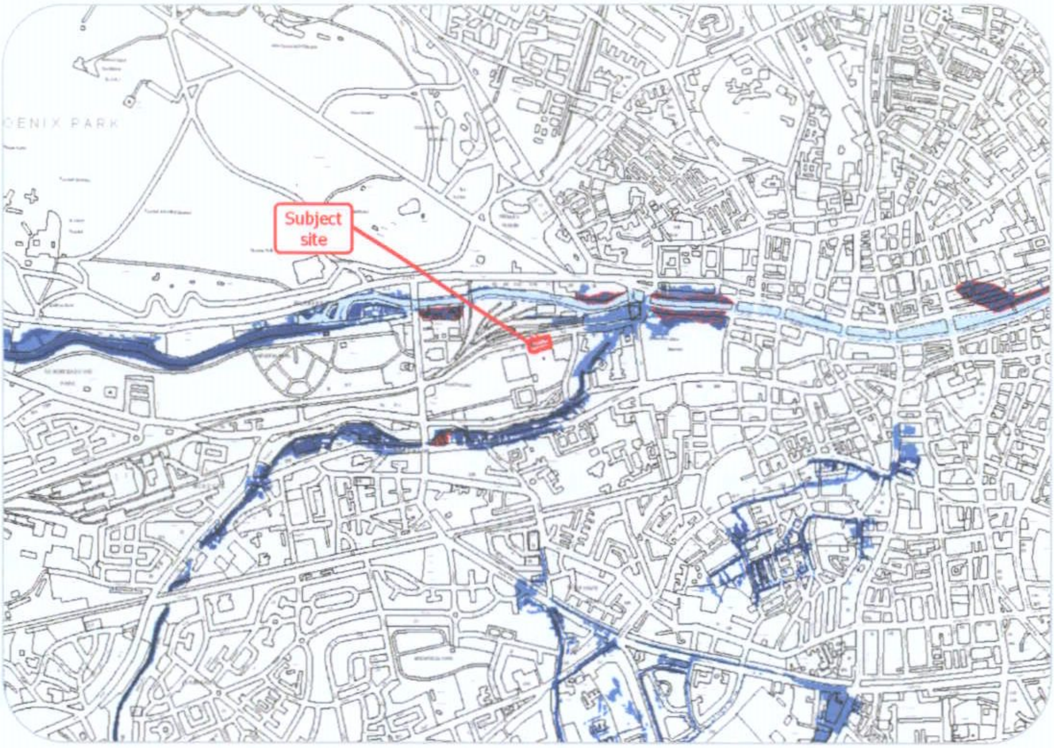


Figure 4 – Extract of DCC composite flood risk mapping
(background imagery source: Dublin City Council)

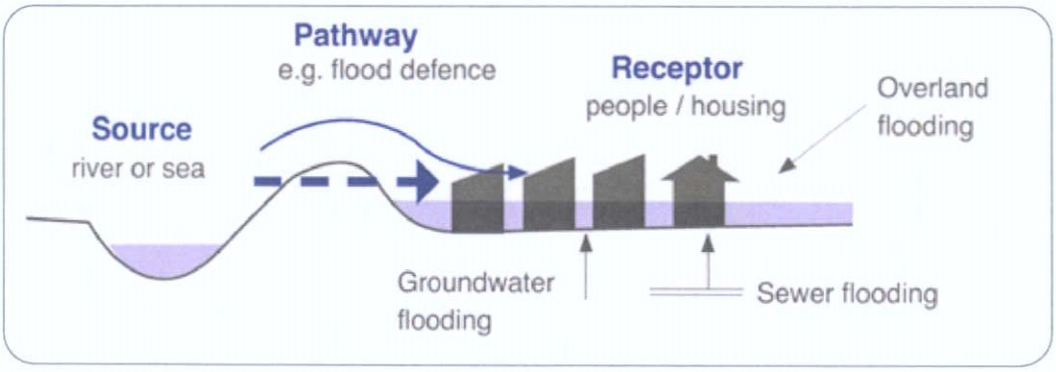


Figure 5 – Source-pathway-receptor model
(The Planning System and Flood Risk Management Guidelines)

The Flood Risk Management Guidelines include an 'appropriateness' matrix for various developments and their potential risk factors. This matrix, reproduced in **Table 3** below, indicates whether a proposed development requires further analysis in the form of a justification test. The Flood Risk

Management Guidelines generally classify commercial buildings as 'less vulnerable developments'.

Table 3 – Flood Zone vs. Justification Test Matrix

Development Category	Flood Zone A	Flood Zone B	Flood Zone C
Highly Vulnerable Development	***Justification Test Required***	***Justification Test Required***	Appropriate
Less Vulnerable Development	***Justification Test Required***	Appropriate	Appropriate
Water-compatible Development	Appropriate	Appropriate	Appropriate

As previously noted, the subject site is located within **Flood Zone C**. As such, a justification test is not required.

5.0 HISTORIC FLOODING

A review of the Office of Public works historical flooding database (www.floodmaps.ie) does not indicate any previous recorded incidents of flooding on the subject site. See **Appendix A** for a copy of the Past Flood Event Local Area Summary Report for this area. An extract of the associated mapping is shown in **Figure 6** below.

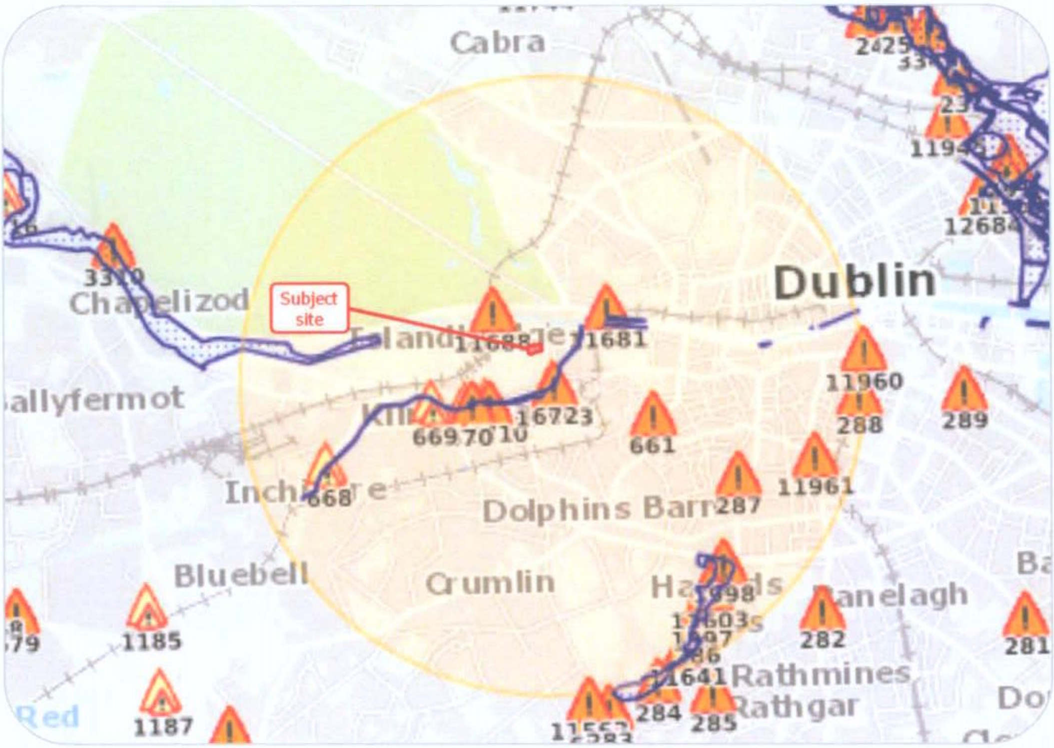


Figure 6 – Extract of Past Flood Event Local Area Summary Report map
(background imagery source: OPW)

6.0 FLUVIAL FLOODING

A review of the Office of Public Works fluvial flood risk maps developed as part of the CFRAM initiative indicates that the subject site is located outside of the zone at risk from a predicted 1-in-1000-year fluvial flood event. See **Appendix C** for the relevant CFRAM map of predicted fluvial flooding extents, an extract of which is shown in **Figure 7**.

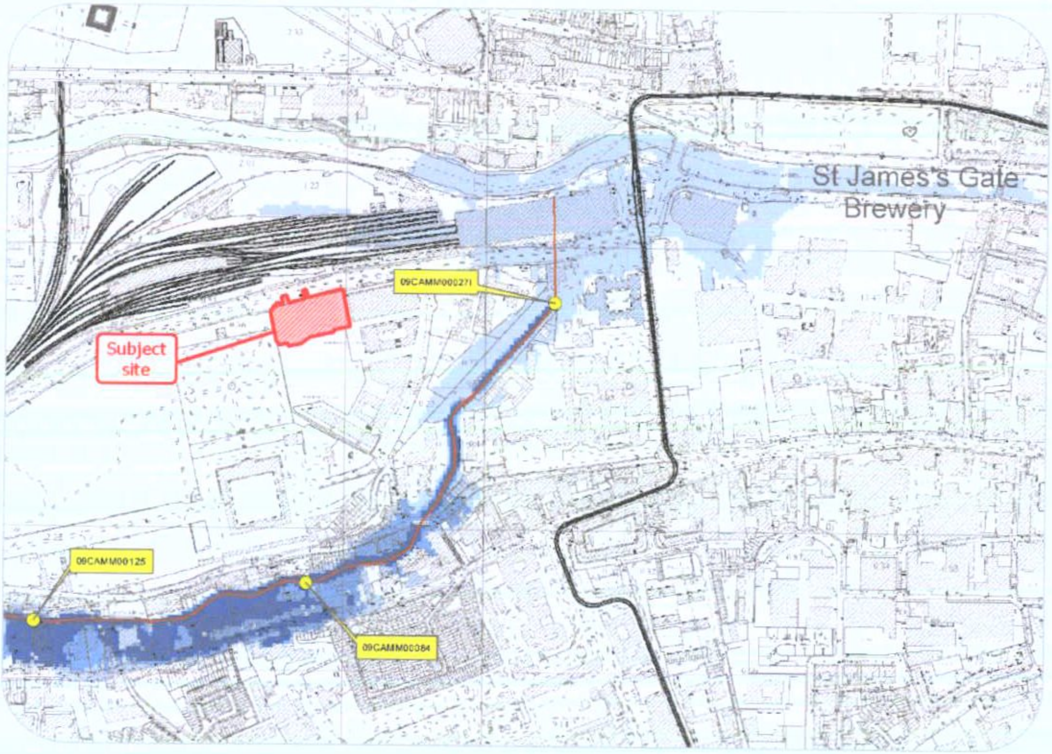


Figure 7 – Extract of CFRAM fluvial flood extents mapping
(background imagery source: OPW)

7.0 PLUVIAL FLOODING

Pluvial flooding is flooding which has originated from overland flow resulting from high intensity rain fall. A high-level pluvial flood map has been produced by both Dublin City Council & the OPW, but it is of benefit as a high-level tool rather than for a specific site. Previous flood events in the area can be reviewed on the Office of Public Works web site (www.floodmaps.ie). Refer to **Figure 6** and to **Appendix A** for the historical flood mapping in the area, and to **Appendix D** for the DCC pluvial flood map.

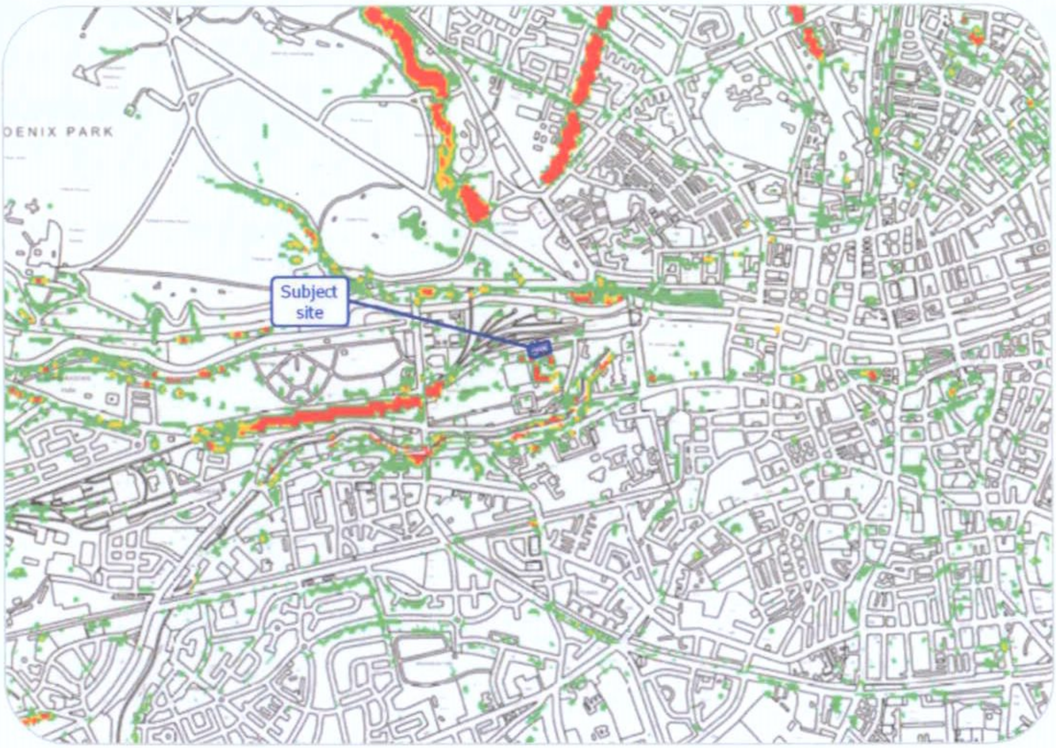


Figure 8 – Extract of DCC pluvial flood hazard mapping
(background imagery source: DCC)