# Appendix 16.2
# Geotechnical Interpretative Report

# M74 Junction 5, Raith

## Geotechnical Interpretative Report

## On Construction of Underpass

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# Contents

# 1 Introduction and Brief

Mouchel Fairhurst Joint Venture (MFJV) was appointed by Transport Scotland as Employer's Representative for the proposed upgrading of the M74 Junction 5, Raith.. Part of this upgrading proposes the option (Option 'D') of a major underpass to carry traffic flows from the A725 beneath the existing M74. As part of their brief, MFJV designed and supervised a ground investigation to identify geotechnical constraints to the construction of the proposed scheme, under a future Design and Build contract. In addition to this general requirement, MFJV was specifically tasked to provide a geotechnical interpretative report for the proposed underpass which would inform the Specimen Design.

# 2 Summary of Previous Work

## 2.1 Fieldwork

A number of ground investigations have been undertaken at the site in the past, principally for the construction of the existing M74. These investigations were relatively limited in coverage and in the depth of ground investigated. A total of 36 Cable Percussive and 7 Rotary hole records were available. The exploratory holes from the various investigations are included on Drawing Number M8MFJV.ST3.G.1313.

A preliminary phase of investigation, known as Contract 1 – Phase 1, was undertaken by MFJV during 2004, to examine a number of route options. This comprised 19 Cable Percussive boreholes with 2 Rotary follow-ons. These exploratory holes are also included on Drawing Number M8MFJV.ST3.G.1313 and are identified by the end-suffix "-04". The works were undertaken by Raeburn Drilling Ltd and reported in their report dated 18 March 2005[Ref 1].

## 2.2 Background Information

Other historical information pertaining to the site and utilised in this report is listed below:

(i) Geotechnical Desk Study Report – M74 Junct 5, Raith, by MFJV [Ref 2].
(ii) BGS Sheet NS 75 NW, Drift, Bedrock and Environmental, Scale 1:10,000
(iii) BGS Sheet 31 (W), Airdrie, Drift Edition, Scale 1:50,000
(iv) BGS Sheet 31 (W), Airdrie, Solid Edition, Scale 1:50,000
(v) BGS Sheet Memoir 31W, The Geology of the Airdrie District, 1996 [Ref 3].

# 3 Location and Site Description

## 3.1 Location, Site Description and Proposed Scheme

The site is located approximately 1.0km west of Bothwell at Junction 5 on the M74 Motorway. The River Clyde lies around 600m to the south of the site. A site location plan is given in Drawing Number M8MFJV.ST3.G.1311.

The site is generally flat except for the existing road infrastructure and lies within the floodplain of the River Clyde at a general level of 22m AOD. The proposed scheme is to be constructed beneath the existing Raith Junction, broadly following the line of the current A725 which runs in a SW-NE direction. The area to the south of Raith Junction is designated a Site of Special Scientific Interest (SSSI).

The proposed underpass is approximately 625m long, 35m wide and will take the form of a road box 5.8m high. The underpass extends between Ch 425 and Ch 1050. The structure attains a maximum depth of 14.0m to finished road level at its midpoint, which lies immediately below the centre of the existing Raith Junction. The proposed structure is crossed by three roads, the M74 motorway and twice by the roundabout forming part of the Raith Junction. The location and layout of the proposed underpass is given on Drawing Number M8MFJV.ST3.G.1313. A long section showing the proposed road profile is included on Drawing Number M8MFJV.ST3.G.1314.

## 3.2 Site History

Historical Maps

Historical Ordnance Survey maps dating back to 1864 are described in detail in the Desk Study Report[Ref 2]. The site has been largely undeveloped over its charted history apart from the A725, which appeared on the first edition maps, and the construction of the M74 and associated junctions which first appeared on the 1971 editions. However, early mineral railway lines crossed the site prior to the modern road construction.

## 3.3 Geology

The solid and drift geology for the site is derived from the sources listed in Section 2.2. Extracts from the 1:50,000 solid and drift geology BGS maps are reproduced in Drawing Number M8MFJV.ST3.G.1312.

Drift Geology

The drift geology of the Clyde Valley is complex. It is reported [Ref 3] that during the last ice age, some 10,000 to 28,000 years ago, an ice sheet, known as the Dimlington covered Central Scotland. It is estimated to have been up to 1km thick. The erosive force carved out any existing deposits within the area creating a deep channel which was later infilled. Initially this area was filled by glacial meltwater deposits. As the glacier slowly retreated glacial meltwaters were ponded by westward retreating ice, which created a dam in the Glasgow area, forming a large lake knows as Lake Clydesdale. Deltaic sand and lacustrine clays and silts were deposited within the lake. With further ice recession and down-wasting, the natural dam in the valley collapsed and the sea flooded Lake Clydesdale, whereupon marine sediments were laid down. It is conjectured that marine inundation rose to an elevation of about 45m AOD, some 23m above the present site level. Isostatic rise and sea level lowering subsequently took place and marine deposition was later replaced by alluvial deposition.

The drift geology at the site contains lithologies which reflect all of these geological events. The complexity is well illustrated by a diagrammatic cross section reproduced from the BGS 1:50,000 drift map in Drawing Number M8MFJV.ST3.G.1315. This section through Bothwell runs approximately 250m north of the site.

Solid Geology

The proposed underpass is indicated to be underlain principally by rocks of the Upper Coal Measures. Typically the Coal Measures comprise sandstones with interbedded siltstone, mudstone and coals. Below the site the strata may be expected to dip at around 5° to the south.

Two significant faults, with downthrows to the north, are indicated. Both faults run in a NW-SE direction in the vicinity of the underpass. One passes directly beneath the Raith Junction whilst the other crosses the route at around Ch 200 i.e. outwith the proposed underpass.

Mining

There are no known records of any shallow coal workings in the Upper Coal Measures. No evidence of shallow mining was found during the recent investigation. Extensive deep mining within seams belonging to the Middle and Lower Coal Measures has taken place around the study area in the distant past. Any residual subsidence effects from deep early mining will have long ceased according to the authoritative statements below.

One feature of extensive mining is the possibility of long term regional rebound of deep groundwater within abandoned workings as a result of the cessation of pumping. However White Young Green (as consultants to the Coal Authority) have reported that although the groundwater sampled deep within the rock had possibly been in contact with Coal Measures strata, there was no evidence of direct connection with old workings [Ref 4]. The geotechnical implications are discussed in Section 7.

The Coal Authority made the following statements within their Report for North Raith [Ref 5]:

*'The property is within the likely zone of influence on the surface from workings in 7 seams of coal at 170m to 340m depth, the last date of working being 1968.'*

*'Ground movement from the above mentioned past coal workings should by now have ceased.'*

*'We have no knowledge of any mine entries within, or within 20 metres of, the boundary of the property.'*

The Coal Authority made the following statements within their Report for South Raith [Ref 6]:

*'The property is within the likely zone of influence on the surface from workings in 8 seams of coal at 150m to 290m depth, the last date of working being 1968.'*

*'Ground movement from the above mentioned past coal workings should by now have ceased.'*

*'We have no knowledge of any mine entries within, or within 20 metres of, the boundary of the property.'*

MFJV have also received a Mining Subsidence Report from the Mineral Valuer [Ref 7]. Specific reference was not made to the proposed Underpass Construction (Option D) upon which this report is based. However, the Mineral Valuer reiterates the comments within the Coal Authority Reports and made the following concluding comment:

*'….I am of the opinion that the risk of damage to the subjects from underground mining subsidence is minimal.'*

## 3.4 Hydrology

The River Clyde flows west some 600m south of Raith Junction. Strathclyde Loch is a large, man-made body of water located approximately 500m SE of the junction.

Hydrological maps for the area included in the Desk Study Report [Ref 2] show the areas adjacent to the River Clyde and Strathclyde Loch, may be inundated by up to 2m of water during a 1 in 100 year flood event. An independent flood study by MFJV has resulted in flood protection measures, in the form of earth bunds, being incorporated in the scheme.

Localised flood plain features in the form of large ponds lie to the north, south and west of the junction.

## 3.5 Hydrogeology

The BGS Hydrogeological Map of Scotland shows the site to overlie a locally important aquifer in rocks of the Carboniferous period: Westphalian Series. This is a moderately permeable aquifer with flow predominantly in fissures and other discontinuities.

# 4 Recent Ground Investigation

## 4.1 Field Investigations

The recent site work (termed Phase 4 of a wider investigation of the M8 Corridor and Consequential Works), was designed and supervised by MFJV and carried out in 2005/2006 by Raeburn Drilling Ltd. Earlier work had indicated that high groundwater within very dense, water bearing sand and gravels was likely and could have a significant impact on the design and construction of the proposed underpass. Early on in the investigation it became apparent that artesian groundwater pressures were persistent in the north of the site. This resulted in a review of the detailed hydrogeological study which formed part of the investigation. With the advice of specialist groundwater consultancy Oxford Geotechnica International (OGI), the pumping test regime, comprising six separate clusters of boreholes with remote holes to monitor the regional effects on ground and surface waters, was further refined. Both soils and rock were targeted and the impact of the geological faults investigated. In total, the investigation comprised the following:

- 130 Cable Percussive boreholes (including ‘shell and auger’ and ‘terrier rig’ methods)
- 23 Rotary holes (including ‘open-hole’ and ‘coring’ methods)
- 28 Trial Pits
- 33 Cone Penetration Probes

The results of the investigation are contained in the Raeburn Draft Factual Report dated May 2006[Ref 8]. All the recent and past exploratory holes and monitoring positions are indicated on Drawing Number M8MFJV.ST3.G.1313. The most recent have the suffix “4BH”, and the hydrogeological boreholes are further designated using colour coding.

## 4.2 Laboratory Testing

The following geotechnical laboratory tests were undertaken during the most recent investigation:

- Classification
- Acceptability
- One Dimensional Consolidation
- Undrained shear strength.
- pH, Sulphate and organic content tests
- Uniaxial compressive strength tests on rock
- Point load tests on rock

The results of the laboratory testing are contained within the Raeburn Draft Factual Report dated May 2006 [Ref 8] and are summarised in Section 6.

# 5 Ground and Groundwater Conditions

The assessment of ground and groundwater influences on the underpass is based upon selected exploratory holes that lie within close proximity to the existing alignment of the underpass. Information from other exploratory holes shown on Drawing Number M8MFJV.ST3.G.1313 has been considered in relation to the wider ground model

The general ground conditions encountered during the investigation are presented on Drawing Number M8MFJV.ST3.G.1314. This drawing groups the materials into their respective lithologies as identified by the BGS [Ref 3].

It is apparent that the ground conditions at Raith Junction differ from those further north at Bothwell - compare Drawing Number M8MFJV.ST3.G.1315 and Drawing Number M8MFJV.ST3.G.1314. At Raith Junction the lateral extent of the Broomhouse Formation is greater and less consistent and the Ross Formation is thicker. However, both formations are open to different interpretation and may not exactly correspond to the BGS model as the basis of their interpretation is not reported. It is further conjectured that the granular layers beyond Ch 800 to Ch 1150, beneath the glacial till, do not belong to the Broomhouse Formation but comprise sands and gravels caught up within the lodgement till or a pre-glacial sand and gravel later overlain by till. For design purposes however, lithological descriptions are limited and, as such, the various formations have been grouped and renamed to reflect their likely geotechnical properties as follows:

| **BGS Lithology Grouping** | **Geotechnical Grouping** |
|---|---|
| Law Formation: sand and gravel; alluvial deposits | Upper Sand and Gravel |
| Paisley Formation: mainly silt and clay; intertidal and subtidal deposits | Marine Silt/Clay |
| Bellshill Formation: mainly silt and clay; glaciolacustrine deposits | Glaciolacustrine Silt/Clay |
| Ross Formation: mainly sand and gravel; glaciolacustrine deltaic deposits | Lower Sand and Gravel |
| Broomhouse Formation: glacial sand and gravel; subglacial deposits | Lower Sand and Gravel |
| ‘Sand and Gravel’ below till beneath the northern section | Lower Sand and Gravel |
| Wilderness Till Formation: lodgement till | Glacial Till |

A revised cross section has been prepared on the basis of the above groupings and is presented on Drawing Number M8MFJV.ST3.G.1316.

A summary of the ground conditions encountered using the above geotechnical groupings is given below:

### 5.1.1 Made Ground

Made ground generally blankets the site and is principally associated with the existing road infrastructure. It typically comprises granular and cohesive material in a medium dense/firm state and ranges in thickness from 1.0 to 4.0m reaching a maximum depth in the vicinity of Raith Junction, where thicknesses of between 7.0 to 12.8m (M74 embankment) were encountered.

### 5.1.2 Upper Sand and Gravel

This stratum is relatively thin and discontinuous across the length of the site. It extends from Ch 560 onwards, and varies in thickness from 1.0m to a maximum 5.8m. It comprises a mixture of granular materials including sand, sand and gravels, and silty sands. It is typically in a medium dense to dense state although discrete layers of silt and clay are often present. The deposit typically occurs above the formation of the proposed underpass although beyond Ch 940, it will form the formation soils.

An approximately 2.0 to 2.5m thick seam of Peat was found to overly the Upper Sand and Gravel between Ch 900 to Ch 950.

### 5.1.3 Glaciolacustrine Silt/Clay

This deposit appears to differ from the Marine Silt/Clay in that it is generally weaker, being soft/very soft, and the silts tend to be cohesive and less frequent, often absent within the deposit. The extent of this stratum is confined to each end of the underpass and can be anticipated in the vicinity of the proposed formation between Ch 420 to Ch 510 and locally between Ch 935 to Ch 990.

### 5.1.4 Marine Silt/Clay

This stratum occurs extensively beneath the length of the underpass between Ch 500 to Ch 1000. It can be expected close to the formation over some 320m. The deposit typically occurs as an upper layer of clay overlying silts. In some cases the clay may be absent altogether whereas clay, without a silt horizon is rare. The thickness of the stratum varies considerably. At either end it thins to between 1.0 to 3.0m reaching thicknesses of 9.0 to 10.8m over the central 250m of the site. The maximum thickness of this material underlying formation level is 7.7m at Ch 790. The clay is typically in a firm state of consistency, although locally it may be soft or stiff. The silt is described as compact and is sandy, suggesting it will behave more granular than cohesive. The silt is in a medium dense state of compaction, and occasionally loose. Laminations of silt and sand are common within the clays whilst bands of clay often occur within the silt.

### 5.1.5 Lower Sand and Gravel

The Lower Sand and Gravels are extensive and form the thickest stratum beneath the site. Over the southern section they can be up to 17.5m thick. Over the north these deposits underlie the till immediately above rockhead, and range from 0.5 to 3.5m thick.

The stratum is derived from two main lithological formations: the Ross and the Broomhouse. The Ross Formation, which overlies the Broomhouse typically comprise sand. Minor proportions of silt or gravel may be present and occasionally the deposit may comprise sand and gravel or gravel. It is generally in a medium dense to dense state. Discrete isolated layers of clay up to 1.6m thick and cohesive lenses can occur, although these are not common.

The underlying Broomhouse materials are generally in a dense to very dense state and typically, although not always, occur as sand and gravels. Cohesive bands are generally absent.

The Lower Sand and Gravel beneath the till over the northern section comprises medium dense to very dense granular deposits with varying amounts of sand and gravel.

### 5.1.6 Glacial Till

Glacial till occurs generally beneath the entire route, although it is relatively thin, and sometimes absent over the southern section. Over the northern section it approaches thicknesses of between 3 to 6.5m and is underlain by granular deposits.

The till is distinguished by its reddish brown colour and is typically a stiff or very stiff sandy gravelly clay.

### 5.1.7 Rock

Rock comprises interbedded sandstones, siltstones and mudstones. The siltstone and mudstone are typically weak and the sandstone moderately strong. Core recovery was generally good except in the vicinity of the fault identified at Ch 710, where all rock types were highly fractured and non-intact, especially within the upper 10m of rockhead. Bedrock level approaches 0m AOD over the south, deepening to a minimum level of -8.5m AOD beneath Raith Junction, becoming progressively shallower moving north where it reached a highest level of +8.5m AOD. This represents a range in bedrock level of some 17m.

## 5.2 Groundwater Conditions

During drilling operations groundwater was encountered in the majority of the boreholes and multiple strikes were common, with three and even four separate groundwater strikes recorded. Typically, the first strike occurred within the upper sand and gravel between 2 to 3m depth, rising to 1.0m bgl. The second was within the upper sand and gravel anywhere between 5 to 10m rising to levels approaching the upper sand and gravels. A third strike was often encountered lower down within the lower sand and gravel and/or glacial till and could rise by varying amounts from zero to ground level. The fourth strike was at rockhead. The strike at rockhead was often very high, particularly over the northern section where artesian conditions were commonly encountered. Not all the boreholes followed this pattern of strikes, as many would inevitably have been obscured by the drilling process. A list of all the boreholes where artesian groundwater was encountered during drilling is given in Appendix A. The boreholes shown on the long sections on

Drawing Numbers M8MFJV.ST3.G.1314 and M8MFJV.ST3.G.1316 indicate their various groundwater strikes and subsequent rises.

Standpipe piezometers were installed in approximately 52 holes during the most recent investigation. These installations were located in discrete zones within the principal strata types. Where the installations are included on the geological long section - refer to Drawing Numbers M8MFJV.ST3.G.1314 and M8MFJV.ST3.G.1316 - they show the relevant response zone together with the highest recorded levels. The complete set of monitoring results is included in the Raeburn Factual Report dated May 2006 [Ref 8].

Two vibrating wire piezometers were also installed within the Marine Silt/Clay zone in boreholes 4BH230A and 4BH235.

Given the importance of the regional hydrology of the area, its SSSI status, and that groundwater control will play an essential part in successfully constructing the works, groundwater specialists OGI, were sub-consulted by MFJV to advise and report separately on this aspect of the investigation. The OGI report [Ref 9] describes the hydrogeological investigative works undertaken. It includes full analysis of pumping tests, ground and surface monitoring, and provides conclusions and recommendations for necessary groundwater control measures, covering construction, design and the regional impact in relation to proposed dewatering measures.

# 6 Laboratory Testing Results

Our review of the laboratory results is based upon a selection of exploratory holes that lie within close proximity to the existing alignment of the underpass. It does not consider in detail test data sourced from all the exploratory holes contained within Drawing Number M8MFJV.ST3.G.1313.

## 6.1 Made Ground

Tables 6.1a & 6.1b summarise the testing carried out on the made ground.

Table 6.1a: Granular Made Ground

| Geotechnical Property | Range | Average |
|---|---|---|
| Moisture content | 8 – 32% | 15% |
| Bulk density | 2.16 – 2.27 ($Mg/m^3$) | 2.21 ($Mg/m^3$) |
| CBR (remoulded) | 0.5 – 42.6% | 13.7% |
| Optimum Moisture Content | 7.3 – 20.6% | 11% |
| Moisture Condition Value | 3.8 – 9.0% | 6.4% |

Test data and borehole logs indicate the granular Made Ground typically comprises non plastic, medium dense reddish brown clayey Sands and Gravels with inclusions of ash, shale, slag and cobbles.

Table 6.1b: Cohesive Made Ground

| Geotechnical Property | Range | Average |
|---|---|---|
| Moisture content | 5 – 32% | 17% |
| Liquid limit | 21 – 47% | 31% |
| Plastic limit | 12 – 29% | 17% |
| Plasticity index | 6 – 25% | 14% |
| Bulk density | 2.05 (Mg/m$^3$)* | 2.05 (Mg/m$^3$) |
| CBR (remoulded) | 0.2 – 122% | 18.6% |
| Undrained shear strength | 9 – 139 (kN/m$^2$)* | Not Applicable |
| Optimum Moisture Content | 6.9 - 20.6% | 10.4% |
| Moisture Condition Value | 2 - 10.1% | 6.2% |

* Limited number of tests available.

Test data and borehole logs indicate the cohesive Made Ground typically comprises low to intermediate plasticity, firm reddish brown sandy gravely Clay with ash and shale fragments.

## 6.2 Upper Sand and Gravel

Tables 6.2a and 6.2b summarise the testing carried out on the Upper Sand and Gravel.

Table 6.2a: Upper Sand and Gravel - Granular Deposits

| Geotechnical Property | Range | Average |
|---|---|---|
| Moisture content | 4.4 – 32% | 17% |
| Bulk density | 1.89 – 2.1 (Mg/m$^3$) | 1.96 (Mg/m$^3$) |
| CBR | 0.1 – 1.6% | 0.6% |
| Optimum Moisture Content | 7.9 – 8.9% | 8.2% |

Test data and borehole logs indicate the material typically comprises non plastic, medium dense to dense, greyish brown clayey Sands and Gravels.

Table 6.2b: Upper Sand and Gravel - Cohesive Deposits

| Geotechnical Property | Range | Average |
|---|---|---|
| Moisture content | 3.4 – 64 % | 26% |
| Liquid limit | 32 – 39% | 35% |
| Plastic limit | 20% | 20% |
| Plasticity index | 12 – 19 % | 15% |
| Bulk density | N/A* | 2.11 (Mg/m$^3$) |
| CBR | 0.24 – 0.55% | 0.4% |
| Undrained shear strength - single test | N/A* | 17 (kN/m$^2$) |
| Optimum Moisture Content | No data available | - |
| Moisture Condition Value | No data available | - |

* Only one test result was available.

Test data and borehole logs indicate the material typically comprises brown sandy/gravely or silty Clays.

## 6.3 Glaciolacustrine Silt/ Clay

Table 6.3 summarises the testing carried out on the Glaciolacustrine Silt/ Clay.

Table 6.3: Glaciolacustrine Silt/ Clay

| Geotechnical Property | Range | Average |
|---|---|---|
| Moisture content | 17 – 48% | 32.5% |
| Liquid limit | 21 – 53% | 38% |
| Plastic limit | 12 – 28% | 20% |
| Plasticity index | 8 – 32% | 17.5% |
| Bulk density | 1.72 – 2.24 ($Mg/m^3$) | 1.91 ($Mg/m^3$) |
| Undrained shear strength | 11 – 75 ($kN/m^2$) | 41.5 ($kN/m^2$) |
| Consolidation testing (mv value at 100kPa) | 0.105 – 0.43 ($m^2/MN$) | 0.286 ($m^2/MN$) |
| Consolidation testing (Cv value at 100kPa) | 1.43 – 6.67 ($m^2/yr$) | 0.369 ($m^2/yr$) |

Test data and borehole logs indicate the material typically comprises intermediate to high plasticity, medium to high compressibility, very soft to soft, brown, slightly sandy laminated Clay.

## 6.4 Marine Silt/ Clay

Table 6.4 summarises the testing carried out on the Marine Silt/ Clay.

Table 6.4: Marine Silt/ Clay

| Geotechnical Property | Range | Average |
|---|---|---|
| Moisture content | 13 – 40% | 30.6% |
| Liquid limit | 28 – 47% | 36% |
| Plastic limit | 18 – 26% | 21% |
| Plasticity index | 6 – 22% | 15% |
| Bulk density | 1.83 – 2.15 ($Mg/m^3$) | 1.96 ($Mg/m^3$) |
| CBR | 0.1 – 0.61% | 0.26% |
| Undrained shear strength | 9 – 47 ($kN/m^2$) | 29.9 ($kN/m^2$)* |
| Consolidation testing (mv value at 100kPa) | 0.052 – 0.136 ($m^2/MN$) | 0.076 ($m^2/MN$) |
| Consolidation testing (Cv value at 100kPa) | 6.935 – 34.766 ($m^2/yr$) | 21.53 ($m^2/yr$) |
| Optimum Moisture Content | 7.0 - 18.9% | 11.3% |
| Moisture Condition Value | 2.1 – 14.6% | 6.5% |

* Treat with knowledge that stiff material was also encountered in the CPT exploratory holes.

Test data and borehole logs indicate the material typically comprises low to intermediate plasticity, low to medium compressibility, uncompact and compact, greyish brown, laminated slightly sandy Silt (with regular inclusions of firm sandy silty Clay).

## 6.5 Lower Sand and Gravel

Table 6.5 summarises the testing carried out on the Lower Sand and Gravel.

Table 6.5: Lower Sand and Gravel

| Geotechnical Property | Range | Average |
|---|---|---|
| Moisture content | 6.8 – 33% | 16.5% |
| Bulk density | 2.04 – 2.33 ($Mg/m^3$) | 2.18 ($Mg/m^3$) |
| CBR | 0.3 – 37.5% | 9% |

Test data and borehole logs indicate the material typically comprises non plastic, medium dense to dense, grey and brown Sands and Gravels.

## 6.6 Glacial Till

Table 6.6 summarises the testing carried out on the glacial till.

Table 6.6: Glacial Till

| Geotechnical Property | Range | Average |
|---|---|---|
| Moisture content | 6.4 – 17% | 10.5% |
| Liquid limit | 19 – 26% | 22.5% |
| Plastic limit | 12 – 14% | 12.3% |
| Plasticity index | 7 – 14 % | 10% |
| Bulk density | 2.21 – 2.42 (Mg/m$^3$) | 2.32 (Mg/m$^3$) |
| Undrained shear strength | 171 – 349 (kN/m$^2$) | 260 (kN/m$^2$) |
| Consolidation testing (mv value at 100kPa) | 0.038 – 0.05 (m$^2$/MN)* | 0.042 (m$^2$/MN) |
| Consolidation testing (Cv value at 100kPa) | 3.02 – 20.776 (m$^2$/yr) | 11.9 (m$^2$/yr) |

* Only two tests results were available.

Test data and borehole logs indicate the material typically comprises low plasticity, very low compressibility, firm to stiff, reddish brown sandy gravely Clay.

## 6.7 Rock

Table 6.7 summarises the testing carried out on the rock.

Table 6.7: Rock

| Rock Type | Unconfined Compressive Strength (MPa)* | |
|---|---|---|
| | Range | Average |
| Sandstone | 0.96 – 94.56 | 48.71 |
| Mudstone | 0.48 – 21.84 | 7.96 |
| Siltstone | - | - |

* As taken from UCS test data or factored from Point Load

Index data (UCS = 24 x $Is_{(50)}$ )

Test data and borehole logs indicates the grey Sandstone is typically moderately strong and unweathered with very close to closely spaced discontinuities. The grey Mudstone and Siltstone is typically weak and unweathered with medium to closely spaced discontinuities.

## 6.8 Chemical Testing

Table 6.8 summarises the pH and Sulphate testing carried out on soil and groundwater samples.

Table 6.8 : Chemical Testing

| Material | pH | | Sulphate SO$_4$, 2:1 water:soil mg/l | |
|---|---|---|---|---|
| | Range | Average | Range | Average |
| Granular M.G. | 7 – 10.2 | 8 | 10 – 1680 | 388 |
| Cohesive MG | 7.2 – 8.4 | 7.9 | 30 – 2040 | 345 |
| Upper S&G | 6.3 – 8.2 | 7.0 | 60 – 270 | 142 |
| Glaciolacustrine S/C | 5.7 – 8.3 | 7.4 | 30 – 180 | 80 |
| Marine S/C | 6 – 8.2 | 7.5 | 30 – 190 | 100 |
| Glacial Till | 8 – 8.7 | 8.3 | 20 – 20 | 20 |
| Groundwater | 6.5 – 8.3 | 7.3 | 40.8 – 400 | 202 |

For comment refer to Section 7.9 – Chemical Attack

# 7 Geotechnical Assessment

## 7.1 General

The base slab of the proposed underpass will be formed at depths ranging from near ground level to typically 10m deep and locally up to 16.0m beneath the Raith Junction. It is expected that structural considerations will necessitate a base slab with an overall construction thickness in the order of 2.0m. The underpass structure (excluding deep cut-off walls) will be formed entirely within the superficial soils. One of the principal aims of the field investigation was to establish whether or not the Marine Silt/ Clay layer was consistent and reliable to the extent that construction problems were much reduced. This has been proved not to be the case. The soil types above and below formation are highly variable, as is the groundwater regime, which is adverse in engineering terms and essential to control for safe construction. Rockhead level varies over a range of 17m and rockhead conditions differ along the length of the underpass. A geological fault has been proved beneath Raith Junction and is considered to have a significant effect on the local hydrogeology. In summary, the geotechnical conditions at the site are complex and problematic however it has been concluded, nevertheless, that the scheme can be successfully engineered given a thorough understanding of these conditions.

A geotechnical assessment of the scheme is given below. To allow a full appreciation of the factors involved, the assessment first provides a geotechnical overview of the scheme under the following headings:

- Ground Conditions
- Groundwater and Groundwater Control
- Mine Water Rebound
- Structure Type and Construction Methodology
- Bearing Capacity/ Settlement/ Swelling/ Ground Movements
- Earth Pressures and Hydrostatic Pressures
- Materials Acceptability
- Chemical Attack

This overview is then followed by a specific geotechnical assessment including recommendations for the proposed structure along the full length of the route.

Effective groundwater control is essential to the design and construction of the scheme given the pressure heads anticipated. Dewatering aspects have been separately assessed by OGI and are reported in the OGI report [Ref 9], which confirms the feasibility of such a process.

Contamination and groundwater chemistry are other important features of the design and these are reported separately in the MFJV report titled M74 Junction 5 Raith, Contamination Assessment Report, [Ref 10].

## 7.2 Ground Conditions

The attached Drawing titled 'Diagrammatic Engineering Geological Long Section' – Reference M8MFJV.ST3.G.1316 illustrates the interpretation of the ground conditions.

### 7.2.1 Soil

*Upper Sand and Gravels*

These are anticipated to occur above the base of the proposed formation throughout the length of the underpass. High groundwater, close to 1.0m below ground level, can be expected but in most cases should be effectively cut-off by piles penetrating cohesive strata beneath.

*Glaciolacustrine Silt/ Clay*

These deposits are not extensive but will occur at formation level at the start of the underpass and between Ch 860 to Ch 900. They are typically very soft to soft, of intermediate to high plasticity and as a result will be prone to softening if disturbed or exposed. They may require localised removal and replacement. At Ch 860 to Ch 900 they will undergo considerable stress relief on the removal of up to 12m of overburden. However, here the deposit is relatively thin and with some replacement almost inevitable, subsequent swelling and consequent settlement on loading of the slab is unlikely – see Section 7.6 Nevertheless, these deposits will be susceptible to base instability owing to their low strength and if subjected to high groundwater pressures from within underlying strata.

*Marine Silt/ Clay*

These deposits are extensive and are expected to form the majority of the formation soils (between Ch 570 to Ch 880). They are typically firm clays or medium dense sandy silts, with sand laminations common. Soft deposits occur locally. They are underlain by sands and gravels which are sometimes under sub-artesian and artesian groundwater pressures. Groundwater was recorded within the silts and given the presence of sand laminations and sand bands, water movement from the underlying gravels is likely. The clay deposits will be susceptible to base instability owing to their relatively low strength and substantial depth in relation to the width of excavation. Thus it is considered that both the silts and clays will be at risk from base instability. At Ch 860 to Ch 900 they will undergo considerable stress relief on the removal of up to 16m of overburden. However, as discussed later in Section 7.6 their swelling potential is considered low.

*Lower Sand and Gravel*

These deposits generally lie beneath the formation, the only exception between Ch 560 to Ch 640 where they may intrude at formation. They are water bearing throughout, often under high water pressure heads which are variable, owing in part to the variability of the strata. Groundwater is high within the strata, typically sub-artesian over the south and artesian over the northern section. Pressure heads above the base slab are anticipated throughout; up to 8.8m over the south of the route and a maximum 13.2m over the north. High uplift forces are therefore anticipated on the structure and on any overlying soils.

Although not conclusively proved in every case, it is suspected this stratum will be in hydraulic continuity with the underlying rock which is artesian over the northern section.

Borehole progress through the lowermost sand and gravels was slow and arduous during the investigation fieldwork owing to their very dense state as well as to the presence of cobbles and boulders and groundwater. Construction methods must take account of these difficulties.

*Glacial Till*

Till occurs at depth beneath most of the length of the underpass but lacks consistency in thickness and composition. Over the north it typically overlies deposits of sands and gravels which are believed to be in hydraulic continuity with the underlying artesian bedrock. The greatest thickness of till encountered without the presence of any significant granular inclusions was 6.4m and as little as 1.5m. This lack of consistency combined with the threat of encountering high sub/ artesian groundwater pressure reduces the likelihood of this stratum providing a reliable cut-off, or suitable foundation for the base slab, or for supporting friction piles.

### 7.2.2 Rock

Rockhead is highly variable between the north and south sections, with a range of up to 17.0m. The level progressively shallows moving north, being at -8.5m AOD at Ch 825 reducing to +8.5m AOD by Ch 1050.

The interbedded nature of the rock dictates that any of the three main rock types; sandstone, siltstone or mudstone should be anticipated at rockhead. The rock is typically intact but broken rock should be anticipated within the faulted zone in the vicinity of Raith Junction.

## 7.3 Groundwater and Groundwater Control

A summary of the monitoring of the groundwater piezometric levels measured within the different strata is as follows:

Table 7.3 Highest Level Groundwater Records

| Stratum | Southern Section | Northern Section |
|---|---|---|
| | Highest Level (m AOD) | Highest Level (m AOD) |
| Made Ground | N/A | 25.8 |
| Upper Sand and Gravel | 19.5 | 22.4 |
| Glaciolacustrine Silt/ Clay | N/A | 19.8 |
| Marine Silt/ Clay | N/A | 21.2 |
| Lower Sand and Gravel | 20.4 | 26.3 |
| Glacial Till | N/A | 20.5 |
| Rock (Sandstone/Mudstone/Siltstone) | 20.6 | 28.0 |

N/A – Indicates no data currently available.

Selected piezometric levels across the site are indicated on the long section in Drawing Number M8MFJV.ST3.G.1316.

The results indicated in Table 7.3 above give rise to the following conclusions:

a. Groundwater within the rock is artesian over the northern section but sub-artesian over the southern part. Even at depth over the southern section, the more permeable sandstones are not artesian. It is conjectured that the presence of the geological fault and/or the thick sand and gravel aquifer is dissipating the artesian pressure at this boundary.
b. The lower sand and gravel and the rock are suspected of being in hydraulic continuity across the site.
c. The piezometric levels within the lower sand and gravel are equivalent to those in the upper sand and gravel, except over the northern section where artesian levels are recorded within the lower deposits.
d. The glacial till has a lower piezometric level than the underlying sand and gravel. Artesian pressures tend to exhibit on puncturing through the till.
e. High uplift pressure and buoyancy effects will be experienced by buried structures over the majority of the site. Pressures up to 145kN/m$^2$, and typically of the order of 90kN/m$^2$ are predicted at the base of the foundation slab.
f. Base instability of excavations is a high risk as a result of high piezometric levels acting below strata forming the formation soils. Active dewatering during construction is essential to ensure safe construction.
g. A high groundwater table is generally present within the upper sand and gravels.

Clearly groundwater control will be an essential feature of the design and construction given the pressure heads anticipated. These aspects are covered in the OGI report [Ref 9].

## 7.4 Mine Water Rebound

The possibility of minewater rebound was raised in Section 3.3. Due to the scale of mining in the wider area, the effect of its cessation may be to increase the groundwater pressures within the rock beneath the site. In view of this enquiries have been made of the Coal Authority (CA) to obtain further information in this regard. The CA's consultants carried out an assessment of the chemical characteristics of the groundwater samples taken by Raeburn and reported that the groundwater had been in contact with coal measures strata but not old coal mine workings. Their report also states that, although they have not carried out a full hydrogeological assessment, they consider that mine water in this area will have fully recovered. Further discussion with the Coal Authority and their engineering advisors is desirable along with the further groundwater monitoring which is recommended by OGI in their report [Ref 9].

## 7.5 Structure Type and Construction Methodology

The high groundwater regime dictates that a watertight structure, capable of resisting high net uplift forces and buoyancy effects and large lateral earth and water pressures must be provided.

Parallel secant pile, or diaphragm walls, are considered the only types capable of providing a watertight structure cost effectively for the scale of construction envisaged. Where internal propping of the walls is not possible they will require to be designed as embedded cantilevers, with possible external anchoring. Either wall type is feasible in such a case. Where internal props can be accommodated (i.e. over the majority of the underpass), the walls will utilise a top prop constructed via top-down construction with the base slab acting as a bottom prop. In order to provide the necessary degree of support in both cases, the walls will require to penetrate the lower sand and gravels or deeper, depending on the degree of uplift anticipated. Where sub-artesian groundwater conditions prevail, wall construction could proceed without groundwater lowering measures. Where high sub-artesian or artesian groundwater pressure is expected, either method requires active control of groundwater. The choice between secant piles and diaphragm wall for the propped section is essentially a construction-related one. Diaphragm walls are most efficient in design terms but more difficult to construct under the envisaged groundwater pressures. Given an effective dewatering scheme, diaphragm walls may well be preferred. However, secant piles can be constructed by employing local dewatering techniques at pile bases (from adjacent pile shafts) although piled walls are less effective with regard to water-tightness.

For base slab construction groundwater control will be essential throughout the entire length of underpass to prevent base instability during construction regardless of the method of wall construction.

In the long term, the structure will require to resist large uplift forces arising from the underlying groundwater pressure. Since it is necessary for the wall to penetrate the water bearing strata to ensure initial stability, water pressures present at depth will ultimately be exerted at the base of the slab via 'worm holes' and through interconnected granular bands, lenses and laminations. Two design issues result:

a. the slab will require to be integral with the wall to resist uplift.

b. the walls will require to be sufficiently long to provide friction to prevent the structure from floating.

Ensuring that the base slab is competently integral with the walls requires a robust shear key connection. It is generally recognised that such a connection is easier to form on a diaphragm wall, where preformed cut out details can be inserted in the reinforcement lowered into the wall. It is possible but more awkward and costly to construct a shear key on a secant wall.

To prevent floatation, it is envisaged that walls of the order of 5 to 10m in length below the base of the excavation should be sufficient to provide frictional restraint. However, over the northern half of the structure, where a combination of artesian pressure and shallow rock exists, it is envisaged that the walls will need to be founded within rock. Construction in rock would also favour the use of secant piles and effective groundwater control is essential to the success of any operation.

A central parallel wall (or discrete piles) may be required to restrain the slab against uplift and to limit bending stress over the large span. A cost comparison between a heavily reinforced slab and additional tension elements will likely dictate the choice.

## 7.6 Bearing Capacity/ Settlement/ Swelling/ Ground Movements

Bearing Capacity

Over most of the length of the underpass, the formation will be subject to a permanent decrease in loading of up to around 100 kN/m$^2$ as a result of excavation to form the box structure. The structure will also be subject to large hydrostatic uplift forces. The main forces on the structure will therefore be negative due to buoyancy effects, apart from at the three locations which support carriageways above the underpass, where end bearing considerations might be required. In this case, both the Lower Sands and Gravels and the underlying bedrock will provide a suitable bearing stratum, as will the Glacial Till although to a lesser extent given its limited thickness and variability. With the likelihood of the base slab being integral with the walls to resist uplift forces, and with net loadings negligible on account of overburden reduction, bearing capacity considerations become largely redundant.

Settlement

Where net downward forces may develop (e.g. supporting the carriageways), the walls will be founded within very dense granular soils, very stiff glacial till or bedrock. In any case, taking account of the expected small net increases in loading, settlement is anticipated to be negligible.

Given that uplift pressure may constitute the principal loading on the structure it would be advisable to include a permeable layer (e.g. granular blanket, lean mix or low fines concrete) at the base of the slab to evenly distribute hydraulic pressures. This will limit any potential for differential movement.

Settlement can result from the increases in vertical effective stress caused by dewatering operations. The magnitude and extent of drawdown are reported in the OGI report [Ref 9] and indicate a drawdown of 6m in the vicinity of the interchange and up to 4.0m in the vicinity of the nearby restaurant and hotel to the SE of the junction. Finite difference analysis using the PLAXIS program predicts a possible total settlement of 40mm within the underlying soils at the location of the existing junction and 30 mm at the nearby restaurant and hotel. The differential settlement calculated is only 3 mm for the restaurant and 5mm for the hotel, which is relatively minor although despite the higher potential total settlements.

At the existing Raith junction record drawings [Ref 11] indicate that the current overbridges located north and south are supported on 50-70 ton capacity piles taken to depths of 7.62 to 8.53m (south bridge) and 11.89 to 16.46m (north bridge) i.e. within the superficial soils. These piles lie 65m distant from the proposed underpass and appear to be end bearing founded in the lower sand and gravels. At their location a drawdown of 6m is predicted as a result of dewatering operations. Based on these levels, this could result in an increase in effective stress of 60kPa. Finite difference analysis using the PLAXIS program predicts a total settlement of 40mm within the underlying soils. The induced level of strain would be sufficient to mobilise negative skin friction on the piles. However, assuming the case where the piles were formed in advance of their associated approach embankments, the base of the piles will have experienced a higher loading from the effects of negative skin friction immediately following construction, given that the pressure imposed by the 8m bank would have been of the order of 160kPa. Over time this will have reduced to zero as the soil consolidated. Hence the projected drawdown should not result in an increase in loading over that previously experienced and therefore no further settlement of the piles would be expected. However, this would not be the case if the piles had been formed after construction of the approach embankments and the underlying soils had already undergone consolidation. In this case the 60kPa resulting from the drawdown would effectively be a new loading as far as the pile was concerned. This scenario is considered less likely but will require to be confirmed otherwise settlements of the order of 15mm below the pile could be expected. The net result is that differential settlement of between 25 to 40mm could be pessimistically predicted to occur between embankment and abutment. The implications for existing services, drainage and carriageway will require to be fully assessed and suitable protection/remedial measures (including possible resurfacing) might be required.

Because of the lack of location-specific ground investigation data in the vicinity of the restaurant and hotel, fairly pessimistic ground conditions have been assumed. Moreover, the OGI report [Ref 9] indicates a number of ways in which the effects of drawdown can be mitigated by recharging the dewatered aquifers or by lengthening the flow paths towards the excavation by taking the underpass walls deeper. There is also recognition that the north of the underpass is distinctly different from the south, with its much lower transmissivity and interlayering of soils. As a result of these factors, the reported drawdowns in the vicinity of the restaurant and hotel are considered to be sensibly cautious.

Settlement can also result from the construction process of forming the walls. This is discussed separately below.

Swelling

A net reduction in loading can result in swelling of clay soils. Excavations of up to 16m are anticipated which could result in stress relief of the order of 320 kN/$m^2$. The walls will be formed almost certainly by excavation/boring methods and taken down to relatively incompressible strata in relation to future loading. As a result there will be little reconsolidation of material in the vicinity of the walls. Normally, that would increase the loading potential on the base since swelling would exceed the net downward movement. However, where cohesive soils are expected at formation, either the overburden stresses are low (around Ch 450) or the clays and silts are firm/ medium dense, and of low plasticity or non plastic, and therefore their swelling potential is considered low. Moreover, the initial layer will likely be removed and replaced by a granular blanket in order to distribute evenly hydrostatic pressures developing at the base of the slab. It is therefore concluded that the risk of swelling pressure developing on the underside of the slab in the long term is low.

Ground Movements (during construction)

Ground movement induced by construction of the retaining walls and base slab could be significant if not properly controlled. Buildings, infrastructure and services located close to the works could be adversely affected. During construction ground movement may be caused by:

- Method of forming the wall
- Excavation between the walls
- Flow of water through or under the walls causing loss of ground

Ground movement (both vertical and horizontal) caused by forming diaphragm walls are quoted to be of the order of 0.05% of the excavation depth for stiff clays [Ref 12]. The bulk of the movement takes place within a horizontal distance 1.5 times the depth of the excavation. Slightly smaller movements are quoted for contiguous walls, with higher lateral movements (up to 0.08%) quoted for secant walls. For bored pile walls settlement effects extend further out, up to twice the depth of the wall.

Ground movement during excavation is highly dependant on the materials excavated and the stiffness of the wall system. Settlements of the ground in the vicinity of supported excavations of the order of 0.04 to 0.35% of the excavation depth are quoted for stiff clays, with greater movement, up to 0.1 to 0.3% for sands and much greater movements of 1 to 2% for soft/ firm clays. The stiffness of the wall has a major effect, with stiffer wall systems tending towards the lower end of the ranges quoted. For the proposed underpass, lateral movement of the order of 0.25 to 0.5% of the excavation depth can be expected. These values could increase significantly if the flexibility of the wall is reduced and/or weaker materials at the base of the excavation are encountered. There is recognition that if the ground at the base of the excavation is rigid, then the area of influence outside the excavation is small. It is also recognised that unless the walls are very stiff and/or keyed into stiff strata the additional support provided by propping may not control ground movement effectively.

At the site, stiff formation either in its natural state or resulting from groundwater control, is anticipated at excavation depth. As the stiffness of the wall can be controlled by its structural design and propping, it should be possible to design a wall system and a construction method to limit ground movement to within acceptable serviceability limits.

Significant movement is expected to be confined to local areas of very loose granular or soft cohesive deposits. The movement is likely to be generated at these locations during boring below the water table. CFA piles forming secant walls are particularly susceptible. Such problems can be avoided by adopting strict procedural methods.

The current overbridges located north and south of Raith Junction lie 65m from the proposed underpass. At this distance the construction of the wall (excluding any effects from dewatering) should have no adverse impact on the pile foundations.

Where ground movement is considered critical, monitoring of surface and subsurface movement should form an integral part of the construction works.

## 7.7 Earth Pressures and Hydrostatic Pressures

Earth pressures will vary depending on the method of construction, wall stiffness, the materials adjacent to the structure and the geometry of the situation. At this preliminary stage the following general remarks can be made:

- For purposes of preliminary design the following effective angles of friction for the various strata are advised:

Table 7.7: Typical Effective Angles of Friction

| Engineering Material Type | $\phi'$ (degrees) |
|---|---|
| Granular Made Ground | 30 |
| Cohesive Made Ground | 28 |
| Upper Sand and Gravel, granular deposits | 33 |
| Upper Sand and Gravel, cohesive deposits | 28 |
| Glaciolacustrine Silt/ Clay | 25 |
| Marine Silt/ Clay | 27 |
| Lower Sand and Gravel | 36 |
| Glacial Till | 31 |

- The action of forming the wall will result in active pressures developing. Higher pressures may develop in the long term given the stiff nature of the structure although such increases should be limited. The relative width of the excavation is likely to shed load during excavation onto deeper, stiffer strata where it will absorb the load redistribution, thus protecting the structure.
- The rough surface of bored piles and diaphragm walls will result in mobilisation of wall friction.
- It is current UK practice to use wall friction values of 2/3 $\phi'$ on the active side and 1/2 $\phi'$ on the passive side.
- Pile to soil adhesion values for the materials concerned should be taken as zero.

The walls will be subjected to considerable hydrostatic pressures. During construction base uplift pressures will require to be controlled in order to ensure stability and thus their magnitude will be limited at the base of the excavation. High hydrostatic levels on the side of the walls should be assumed during construction, with no reduction taken into account due to drawdown effects owing to the possibility of the need for recharging the aquifer to limit the impact of drawdown.

In the long term, the design should cater for high base uplift and high hydrostatic pressures as it is anticipated that deep water pressures will force a route via worm holes etc up the sides of the structure. Existing artesian pressures in the north will flow and ultimately relieve at the current hydrostatic level of the uppermost groundwater level. Artesian and sub-artesian pressures will be confined by the underside of the base slab. Measures may be considered at the base of the slab to relieve such pressure but would require an associated permanent drainage outlet. The alternative is to design the slab to accommodate significant uplift pressure.

## 7.8 Material Acceptability

There is limited opportunity for re-use of excavated soils elsewhere in the scheme therefore, it is anticipated that the majority of the soils will be disposed off-site – refer to the Contamination Report [ref 10].

A preliminary assessment of the proportions of acceptability has been completed for the Made Ground, Upper Gravels and Marine Silt Clay. The Made Ground and Upper Gravels were further subdivided into cohesive and granular type deposits and the bulk of the excavated material will comprise these five soil types. Section 6.0 includes a series of tables which review the data used to assess acceptability.

The assessment of acceptability was based upon the following criteria:

1) Natural moisture content within the range of 90% maximum dry density (DMRB HA44/91).
2) MCV value's between 8.5 and 15 (DMRB 4.1.4 SH7/83).
3) Undrained Shear Strength >45 kPa (DMRB HA44/91).

Table 7.8 provides a summary of the assessment of the proportions of acceptability for the various material types.

Table 7.8 Proportions of Acceptable Material

| Engineering Material Type | Proportion of Acceptable Material[2] |
|---|---|
| Engineered Made Ground (Granular) [1] | 70% as Class 1 |
| Engineered Made Ground (Cohesive)[1] | 20% as Class 2C |
| Non-Engineered Made Ground (Granluar & Cohesive) [1] | 0 |
| Upper Sand and Gravel, granular deposits[2] | 50% |
| Upper Sand and Gravel, cohesive deposits[2] | 0 |
| Marine Silt/ Clay | 0 |

[1] It is expected that a significant proportion of the Made Ground will comprise Engineered Fill sourced from the existing M74 and associated embankments.

[2] In practice, the intermixed nature of the Upper Sands & Gravels might make it difficult to selectively excavate the acceptable soils. The suggested proportion reflects this difficulty.

[3] The proportions of acceptable material have been limited by the high moisture content of the in situ soils. Drying of the materials will improve the proportions of acceptability for all types.

## 7.9 Chemical Attack

The vast majority of the soil samples tested recorded low sulphate results and near neutral pH's. This suggests the concrete should be designed for a DS-1 sulphate class and an ACEC Class AC-1 in accordance with Table C2 of BRE Special Digest 1:2005, Third Edition.

Exceptionally there appears to be a localised region from Ch 800 to Ch 900 where a consistently high level of sulphate content was recorded. Values ranging from 710 to 2040 mg/l were returned from samples taken within 4TP213, 4BH233, 4BH235 and 4BH269. Concrete in this region should be designed for DS-3 sulphate class and AC-3.

## 7.10 Geotechnical Assessment along the Route

The following geotechnical assessment covers the full length of the underpass and assumes a 2m thick base slab construction. An indication of the resulting probable design is presented on Drawing Number M8MFJV.ST3.G.1317.

<u>Ch 425 to Ch 590</u>

Progressively increasing excavation up to a maximum 9.0m to the underside of the base slab is anticipated. A cantilever secant pile wall terminated within the lower gravels should provide the necessary support. A diaphragm wall is also technically feasible but the simpler and cheaper option of using a secant pile wall is likely to be preferred.

Maximum hydrostatic uplift pressure of the order of 55 kN/m$^2$ can be expected on the underside of the slab from within the Lower Sand and Gravels which will form the majority of the formation soils.

Active groundwater control within the Lower Sand and Gravels will be required during construction of the base slab to limit large base inflows. Active groundwater control is not envisaged as being necessary to form the walls.

Stress relief of the order of 180 kN/m$^2$ as a result of overburden removal is predicted. However, the granular formation should have an immediate response to this reduction with no long term impact on the base of the slab.

Beyond Ch 540 a structural deck is required to carry the roundabout. Walls founded in the Lower Sand and Gravels should provide the necessary support.

<u>Ch 590 to Ch 775</u>
Excavations in excess of 10m and a maximum of 16.2m to the underside of the base slab beneath Raith Junction are anticipated. A top prop is proposed throughout this length of the underpass.

Maximum hydrostatic uplift pressure of the order of 90 kN/m$^2$ can be expected on the underside of the slab from within the Lower Sand and Gravels which will act on the Marine Silt/ Clays forming the formation soils. A granular blanket should be provided beneath the slab to balance the hydrostatic pressures.

Groundwater control within the Lower Sand and Gravels will be required during construction to eliminate the risk of base instability of the Marine Silt/ Clays. Active groundwater control is not envisaged to form the walls.

Stress relief of the order of a maximum 320 kN/m$^2$ as a result of overburden removal is predicted. However, as discussed above the risk of swelling is considered low.

Where the underpass crosses the M74 (which is carried on embankment) a structural deck is required to support the carriageway. Walls founded in the Lower Sand and Gravels should provide the necessary support. The net loading is likely to be much reduced given the effects of stress relief and hydrostatic uplift forces.

<u>Ch 775 to Ch 940</u>
Excavations of the order of 10 to 12m to the underside of the base slab are anticipated. A top prop is proposed throughout this length of the underpass. Artesian groundwater pressures are anticipated which will require the wall to penetrate the underlying rock to provide the necessary uplift resistance.

Maximum hydrostatic uplift pressure of up to 145 kN/m$^2$ can be expected on the underside of the slab from within the Lower Sand and Gravels or rock and will act on the Marine Silt/ Clays forming the formation soils. A granular blanket should be provided beneath the slab to balance the hydrostatic pressures acting.

Active groundwater control within the Lower Sand and Gravels and within the rock will be required during construction to eliminate the risk of base instability in the Marine Silt/ Clays. Active groundwater control is also envisaged to form the walls.

Stress relief of the order of 240 kN/$m^2$ as a result of overburden removal is predicted. However, as discussed above the risk of swelling is considered low.

Around Ch 850 a structural deck is required to carry the roundabout. Walls founded in the Lower Sand and Gravels or rock should provide the necessary support.

Ch 940 to Ch 1050

Excavation, up to a maximum up to 10.0m to the underside of the base slab is anticipated although excavations of the order of 5m are likely to be more typical. Cantilever walls terminated within the rock for the deeper excavations, or glacial till for lesser depths should provide the necessary support. Temporary props may be required to support the cantilever wall prior to the construction of the base slab. It may even be necessary to provide permanent support, external to the wall i.e. anchors, for the deeper excavations.

Maximum hydrostatic uplift pressure of the order of 145 kN/$m^2$ can be expected on the underside of the slab from within the rock and lower sand and gravels which will ultimately act on the upper sand and gravels forming the majority of the formation soils.

Active groundwater control within the rock will be required during construction of the base slab to limit large base inflows. Groundwater control is also envisaged to form the walls should these penetrate the lower sand and gravels where artesian groundwater is anticipated.

Stress relief to a maximum 120 kN/$m^2$ as a result of overburden removal is predicted. However, the granular formation should have an immediate response to this reduction with no long term impact on the base of the slab.

# 8 Summary

**8.1** A comprehensive ground and hydrogeological investigation has been completed for the proposed road network improvements at Raith. This report has interpreted the investigation findings in order to specifically address how the prevailing ground conditions influence the design and construction of the deep underpass which is proposed to carry the A725 beneath the existing junction with the M74.

**8.2** One of the principal aims of the field investigation was to establish whether or not a Marine Silt/ Clay layer was consistent and reliable enough to permit conventional, straightforward methods of construction. This has proved not to be the case. The soil types above and below formation are highly variable and differ significantly between the north and south of the interchange. The groundwater regime is similarly highly variable, and is particularly adverse in engineering terms due to the presence of artesian groundwater pressures. Rockhead level varies over a range of 17m while rockhead conditions differ throughout the length of the underpass, becoming shallower towards the north. A geological fault, confirmed beneath the site, is considered to have a significant influence on the local hydrogeology. In summary, the geotechnical conditions at the site are complex and problematic. Nevertheless, it is considered that the scheme can be engineered successfully based on a detailed understanding of these conditions.

**8.3** As a result of the complexity of the groundwater conditions, the importance of the regional hydrogeology of the area and its SSSI status, specialist groundwater consultancy Oxford Geotechnica International (OGI) were sub-consulted by MFJV to advise and report separately on the investigation and feasibility of groundwater dewatering during construction.

**8.4** The underpass will be constructed wholly within superficial soils and to a maximum depth of 16.2m. Either a Secant Pile or Diaphragm Wall form of construction is considered suitable to accommodate the anticipated combined earth pressures and hydrostatic forces and to ensure a stable and water tight structure. Active groundwater control will play an essential part in successfully constructing the works and an array of dewatering wells will be required to construct the walls and the base slab for the underpass.

**8.5** High groundwater pressures are also expected to exhibit on the underside of the base slab in the long term. To deal with these pressures the base slab will need to be integral with the walls and a shear key connection will be necessary. The importance of such a connection may favour the use of diaphragm wall construction with its simpler shear key constructability However, secant piles can be constructed by employing local dewatering techniques at pile bases (from adjacent pile shafts) although piled walls are less effective with regard to water-tightness. The choice is essentially construction related.

**8.6** High groundwater pressures will also subject the base slab to very high structural loading. In order to design the slab against uplift, intermediate walls or discrete piles may be required. A cost comparison between a heavily reinforced slab and additional tension elements will dictate the final choice.

**8.7** The OGI analysis predicts drawdowns of between 3.5 to 6.0m occurring in close proximity to the underpass as a result of dewatering operations during construction. This could cause an increase in the effective stress in the surrounding soils and has the potential to cause settlement of nearby structures. It is calculated that up to 40mm settlement could result although differential settlement will be much lower. This figure is considered pessimistic and further groundwater modelling perhaps with some targeted GI in vulnerable areas would be required to refine this estimate. A further reduction would also be possible through implementation of the groundwater recharge measures suggested by OGI to ameliorate drawdown .

**8.8** Other surface and sub-surface ground movement is inevitable with deep wall construction. The amount and extent of movement can be controlled by properly informed choice of construction method and quality workmanship. The presence of stiff strata at depth should assist and, as the stiffness of the wall can be controlled by its structural design and effective propping, it is considered possible to design a wall system to limit ground movement to within acceptable serviceability limits. Nevertheless, the calculation of ground movement is not straightforward and beneficial use can be made of precedent and case history data. In addition, observational monitoring of ground movements must form an integral part of the design and construction works.

**8.9** An outline design for the wall construction along the length of the underpass to suit the variable ground/ groundwater regime has been proposed and is presented in Section 7 above.

# 9 References

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6. The Coal Authority, *'Coal Mining Report, M74 Junction 5, Strathclyde'* - Reference 00157775-04 - 07 April 2004.
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14. Manual of Contract Documents for Highway Works Volume 7*, 'Pavement Design and Maintenance',* Section 2 Foundations.
15. Tomlinson, Foundation Design & Construction, 6$^{th}$ Edition, 1995.
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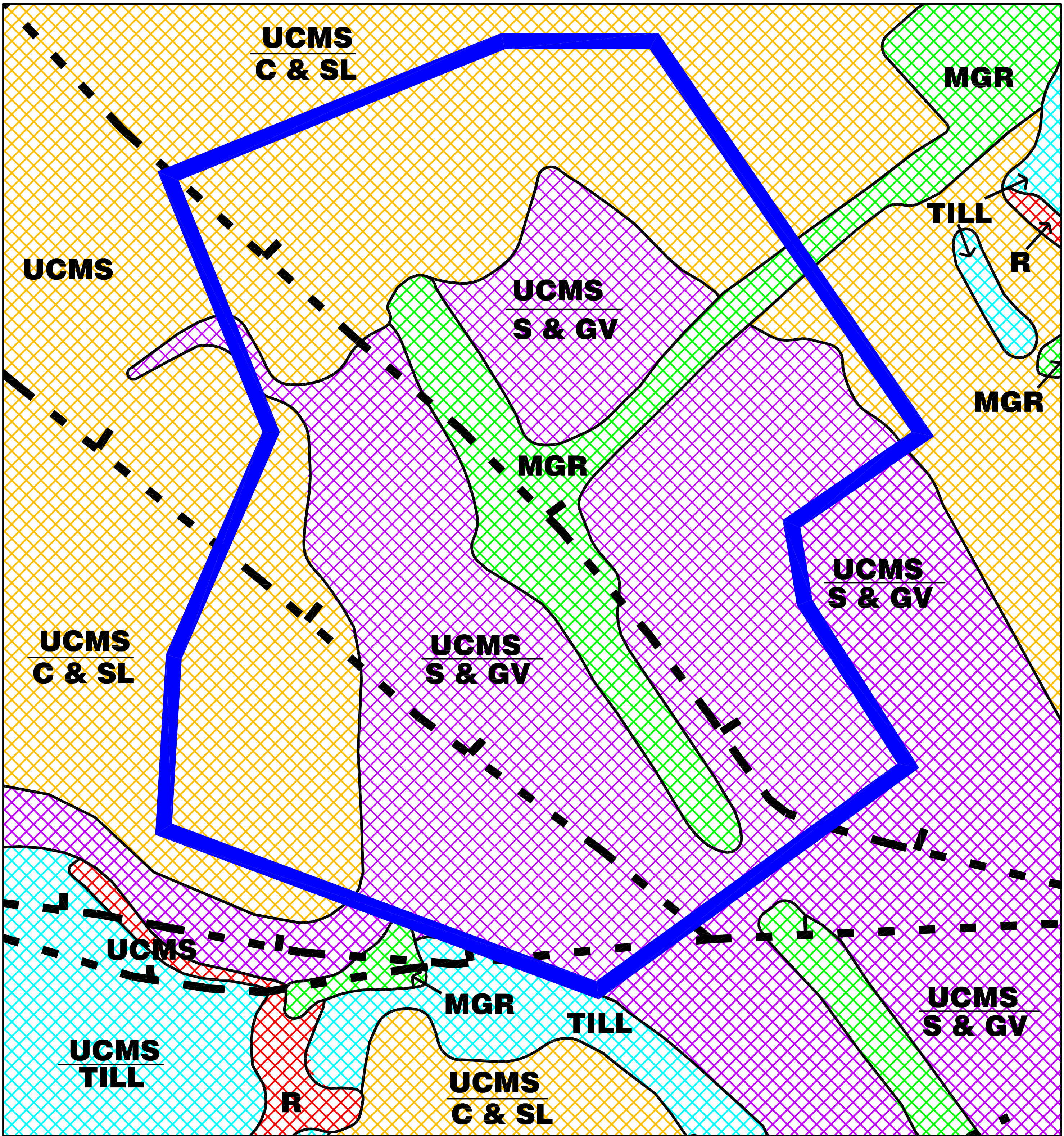
# Drawings




| | | | |
|---|---|---|---|
| CLIENT: SCOTTISH EXECUTIVE | Mouchel FAIRHURST IN ASSOCIATION WITH SIAS YA | PROJECT M8 Baillieston - Newhouse and Associated Improvements | DRAWING NO: M8MFJV.ST3.G.1311 |
| | | TITLE SITE LOCATION PLAN | SCALE: 1:10000 / REV - |
| | | | DRAWN BY: MP / DATE: 07/11/06 |




KEY

DRIFT

| | |
|---|---|
| MGR | MADE GROUND: MAN-MADE DEPOSITS ON ORIGINAL GROUND SURFACE |
| S & GV | SAND AND GRAVEL OF FLOOD PLAINS AND VALLEY FLOORS (GENERALLY NEAR GROUNDWATER LEVEL) |
| C & SL | CLAY AND SILT ("BRICK CLAY"): FLATTISH OR UNDULATING SPREADS |
| TILL | TILL (BOULDER CLAY): CLAY WITH STONES, COMMONLY STIFF AND ILL-SORTED |
| R | BEDROCK AT OR NEAR SURFACE |
| | GEOLOGICAL BOUNDARY (DRIFT) |

SOLID

| | |
|---|---|
| UCMS | UPPER COAL MEASURES: RED, PURPLE, YELLOW & GREEN SILTSTONES, MUDSTONES & SEATCLAYS, WITH THIN SANDSTONES; THICK CROSS-BEDDED SANDSTONES IN LOWER PART |
| | FAULT, CROSSMARK ON DOWNTHROW SIDE |

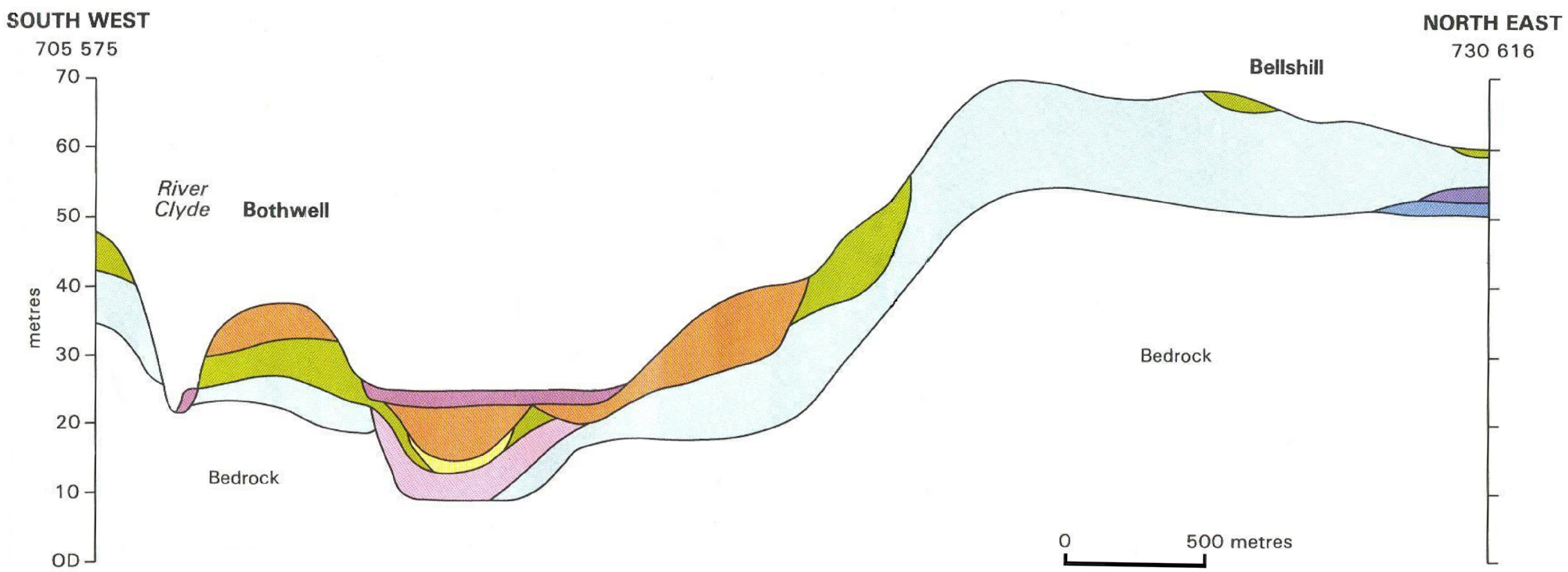


| | | |
|---|---|---|
| CLIENT: SCOTTISH EXECUTIVE | Mouchel FAIRHURST IN ASSOCIATION WITH SIAS YA | PROJECT M8 Baillieston - Newhouse and Associated Improvements |
| | | TITLE SOLID & DRIFT GEOLOGY OF THE STUDY AREA |

| | |
|---|---|
| DRAWING NO: | M8MFJV.ST3.G.1312 |
| SCALE: | 1:10000 |
| REV | - |
| DRAWN BY: | MP |
| DATE: | 07/11/06 |

| | |
|---|---|
| | Law Formation; sand and gravel; alluvial deposits (shown on generalised sections only) |
| | Paisley and Linwood formations; mainly silt and clay; intertidal and subtidal deposits |
| | Bellshill Formation; mainly silt and clay; glaciolacustrine deposits |
| | Ross Formation; mainly sand and gravel; glaciolacustrine deltaic deposits |
| | Broomhouse Formation; glacial sand and gravel; subglacial deposits |
| | Wilderness Till Formation; lodgement till (shown on generalised sections only) |
| | Broomhill Formation; clay and silt; glaciolacustrine deposits (shown on generalised sections only) |
| | Baillieston Till Formation; lodgement till |

GENERALISED SECTION ACROSS THE VALLEY OF THE RIVER CLYDE AT BOTHWELL




British Geological Survey, Geological Sheet 31W Airdrie (Scotland) – 1:50 000 Series

| | | | | | | |
|---|---|---|---|---|---|---|
| CLIENT: Transport Scotland (An agency of Scottish Executive) | PROJECT TITLE: M74 Junction 5, Raith | | | | | |
| A | Final Version | MP | MMcL | RI | 03/07 | |
| REV | REVISIONS | BY | CHKD | APP'D | DATE | |
| AMENDMENTS | | | | | | |



| | | | |
|---|---|---|---|
| DESIGN BY: | JJH | CHECKED BY: | MMcL |
| DATE: | JULY 06 | DATE: | NOV 06 |
| DRAWN BY: | MP | APPROVED BY: | RI |
| DATE: | JULY 06 | DATE: | NOV 06 |
| DRAFT | | X | FINAL REPORT |

ENGINEER: Mouchel FAIRHURST
IN ASSOCIATION WITH SIAS YA

DRAWING TITLE: BGS Diagrammatic Geological Section at Bothwell

| SCALE | DRAWING NO | REV |
|---|---|---|
| 1:20,000 AT A3 | M8MFJV.ST3.G.1315 | A |

# Appendix A - List of Boreholes Where Artesian Conditions Encountered:

4BH233

4BH234

4BH236

4BH237B

4BH237C

4BH238

4BH238B

4BH241

4BH251

4BH251A

4BH252

4BH252A

4BH255

4BH270

4BH270A

4BH272

4BH273

4BH274

4BH290

4BH292

4BH293