Generally, water affected material should be removed from the site. Depending on organic content, it may be possible to re-used this material as engineered fill provided the material is dried and moisture conditioned to the optimum moisture content for compaction.

In identified landslip areas, all water-affected material and failed material should be removed and taken off site. As described above, in proposed fill areas, where the stripped surface is steeper than 6° (10% grade) the exposed soil surface should be stepped to key the fill into the natural ground. Within the boggy areas and where surface water seepage is present, subsurface drains should be constructed prior to backfilling, as described in Section 3.4.

3.4 Drainage

Adequate and appropriate drainage will be crucial in maintaining the stability of cut and fill batters and natural slopes over the site. It is recommended that site drainage include the following elements:

- Prior to commencing the main earthworks and construction program, drains should be constructed to dry out the swampy areas around and downslope of landslips on the western side of the site and in the area of the cattle dam in the southern area of the site. The drains should be not greater than 10m apart at the head of the landslips. In the area of the landslips, the drains should be constructed in a radial, fan-shaped pattern terminating at or immediately adjacent to the head of each slip. Open trench drains may be used provided they are no deeper than approximately 1.0m and are orientated at between 60° and 90° to the natural contours. If problems are encountered with excavation or collapse of the trench sides, subsurface, gravel-filled drains may be used.

  Once the swampy areas have largely dried out, the water-affected, slipped material may be excavated and permanent, subsurface drains installed prior to backfilling to the required profile. These drains should consist of gravel-filled box or V-shaped trenches lined with geotextile and backfilled with gravel and aggie-type slotted plastic pipes. The drains should be extended upslope beyond the slip scarps by approximately 5m, excavated to sufficient depth to intersect the basalt-metamorphic interface layer.

  As for the temporary drains, in the area of the landslips, the permanent drains should be constructed in a radial, fan-shaped pattern and should utilise and connect with any remaining temporary drains. A typical drainage pattern is shown on Figure 1. The permanent drains should be constructed after all failed and/or water-softened material has been removed from the boggy or landslide areas and should be excavated to a depth of approximately 0.6m into undisturbed natural ground.

  The discharge from the subsurface drains should be collected and channelled into the stormwater reticulation system.

  Construction of footings or pools, etc. in the fill should not be affected by the drainage trenches. Where footings or excavations extend below natural ground, excavations should include provision for locally diverting any subsoil drain intersected.

  Alternatively, a gravel blanket-type drain could be used, covering the area of the slip, provided a geotextile barrier is placed above and below the geotextile layer.

- In areas of fill, subsurface drains should be constructed along the interface between the fill and natural ground, excavated approximately 0.6m into the natural ground, to collect subsurface seepage which may discharge out of the natural ground during and immediately after periods of heavy rainfall. The drains should be laid out in a herringbone pattern at around 45° to the contours of the underlaying natural ground with a grade of not less than 100H:1V and should be collected and channelled into the stormwater reticulation system.

- All discharge from gutter downpipes and overland water flow from roads, driveways, paved areas and subsurface drains for allotments and retaining walls is to be collected and channelled into the piped stormwater reticulation system.
- Where possible, the ground surface on allotments should be graded away from cut or fill batters and retaining structures to direct surface water flow towards roadways and stormwater reticulation systems. If allotments cannot be graded in this manner, lined collection drains should be constructed along the top of cut and fill batters and retaining structures to collect surface water flow and direct it into the stormwater reticulation system. Aggie-type drains to be constructed at the rear of the base of retaining structures and directed into the stormwater reticulation system (refer Figure 2 and 3).

3.5 Excavations and Earthworks

3.5.1 Excavatability of Materials

An assessment of the excavatability of materials encountered on site during the field investigation is presented in report B17439/1-B and is repeated here in the interests of limiting cross-referencing between the two reports.

The excavatability of soil and weathered rock is dependent on a number of material parameters and methods of excavation. These may be summarised as follows:

- Material type (eg granite, mudstone etc).
- Weathering Grade (highly weathered, slightly weathered etc).
- Compressive strength and strength profile (uniformly high strength, high strength with low strength bands etc).
- Grain Size (fine grained as for a shale or basalt, coarse grained as for a granite or conglomerate)
- Rock Mass Structure (foliated, bedded, blocky, massive etc).
- Discontinuity properties (orientation with respect to excavation direction, separation, infilling etc).
- Groundwater.
- Mode of Excavation (eg. trenches, benching etc).
- Type, Model, Condition of Equipment and Power Rating.

A number of studies have been carried out to relate the rock mass strength and structure parameters to excavatability (or rippability) using different sizes of tractors and excavators.

The test pits carried out on site were excavated using a 30 tonne excavator fitted with 1m wide bucket and spade teeth. In general, the test pits excavated on site encountered residual clay soils and extremely weathered, very low strength rock. In practice, it is considered that these materials could be excavated using conventional earthmoving equipment such as hydraulic excavators, dozers and scrapers. The presence of discontinuous layers of boulders in the residual basalt soils may cause some difficulty for scrapers in localised areas.

A number of test pits and boreholes encountered nominal "refusal" on weathered rock materials. The test pit numbers and the RL and material descriptions in which "refusal" was encountered are summarised in Table 1.

TABLE 1: SUMMARY OF EXCAVATOR AND DRILLING RIG REFUSAL DEPTHS

Test Pit/Borehole No.	Refusal depth and RL	Material Description
TPC25	1.3m, RL13.39m	XW-HW metasiltstone
TPC27	3.3m, RL34.46m	XW basalt with boulders
TPC29	1.2m, RL19.67m	XW-HW metasiltstone
TPC35	3.6m, RL39.64m	XW basalt with boulders
BH5	3.9m, RL26.1m	XW basalt
BH6	7.5m, RL23.5m	XW basalt

It is considered that bulk excavations below the "refusal" depth for the excavator could require the use of a medium sized dozer (D7-size) with single ripping tyne. It should be noted that the presence of layers of boulders in the confines of the test pits may result in an overly-pessimistic refusal depth when compared to bulk excavations. "Refusal" depths for the drilling rig were generally deeper that those for the excavator and typically occurred on extremely weathered, low strength basalt. It is considered that bulk excavations below the "refusal" depth for the drilling rig could require the use of a medium to large sized dozer (D8 to D9-size) with single ripping tyne.

It is expected that medium to high strength metasandstone may be encountered in excavations near the road cuttings on Fraser Drive. Bulk excavations in this material will require a large dozer (D9-size) fitted with a single tyne ripper and a large excavator (30 tonne or greater) fitted with a rock bucket and ripping attachment in confined excavations, with some use of hydraulic rock breakers.

3.5.2 Earthworks and Fill Batters

In general, all earthworks should be carried out accordance with the AS3798 – 1996.

Prior to placing fill, site preparation should be carried out and subsurface drains installed as discussed in Section 3.4.

Engineered fill should be placed in layers not exceeding 300mm loose thickness with each layer compacted to a dry density ratio of not less than 100% of Standard maximum dry density.

Permanent, unsupported fill batters in engineered fill and natural soils should be formed no steeper than 1.5H:1V for batters with a vertical height of not greater than 1.5m and 2H:1V for batters with a vertical height of not greater than 5.0m. For cut or fill batters greater than 5.0m in vertical height, specific engineering advice should be obtained. Large batter may have specific drainage requirements to prevent raised groundwater levels and may require additional support such as soil nails or retaining structures.

For unsupported cut or fill batters in the area of the basalt – metamorphic interface zone notably in the central area of the north-facing hillside, observations should be carried out by an experienced Geotechnical Engineer or Engineering Geologist during excavation to assess the dip angle and direction of the interface. In general, it is considered that batters excavated at a gradient of no steeper than 2H:1V in this area should have an adequate factor of safety against failure. Local ground support measures may be required where the interface dips out of the batter face. These measures could include local soil nailing and installation of gravel-filled, counterfort drains in the batter face to intercept seepage along the interface zone.

Similarly, the interface zone should not "daylight" within approximately 20m horizontally downslope of unsupported fill batters nor be present at a depth of less than 2m under the toe of unsupported fill batters.

3.6 Cut Batters and Retaining Structures Adjacent to Property Boundary

It is understood that cut batters are to be formed along the rear boundary of the site adjacent to the existing residential properties along the western and central north-facing site boundaries, with the cut batters to be supported by crib-type retaining walls. A typical cross-section through this boundary after construction is shown on Figure 4. The cross-section shows a 1.0m bench at the property boundary above a cut batter formed at no steeper than 2H:1V (26°) falling from the property boundary to the top of the retaining wall. The height of the retaining wall will vary depending on location but will typically be in the range of 2m to 5m.

In general the retaining structures along the rear boundary of the site should be no higher than approximately 5m. As outlined in Section 3.2, where cuttings exceed 5m, this may be achieved by forming a 26° (2H:1V) batter between the top of the retaining structure and a point no closer than 1.0 from the property boundary. A 1m wide strip of natural ground should be left between the batter and adjacent properties.

The majority of the upslope properties have some form of fill embankment or retaining structure along or adjacent to the property boundary, surcharging the natural ground adjacent to the subject site. Retaining wall design will need to take into account the surcharge conditions produced by the 2H:1V batter above the wall and the existing retaining structure and embankments along or adjacent to the property boundary. Table 2 summarises site observations regarding the existing embankments and retaining structures along the site boundary.

It should be noted that the heights and set-backs presented in Table 2 were measured from the site boundary fence with a tape measure and may not be sufficiently accurate for design purposes. It is recommended that accurate survey be carried out to provide this information. It should be also be noted that some the structures on adjacent allotments, such as blockwork walls and swimming pools, may be sensitive to ground movements caused by excavations for cut batter along the property boundary and may require underpinning or additional support. The designs of the footings of these structures are not known.

TABLE 2: SUMMARY OF EXISTING EMBANKMENTS AND RETAINING STRUCTURES ALONG WESTERN AND CENTRAL NORTH-FACING BOUNDARY

Lot No.	Description	Lot No.	Description	Lot No.	Description
Hillcrest Avenue		22	1.8m blockwork wall	14	1.8m boulder wall
1	1.0m fill	23	1.5m rock wall, 5m from fence	15	1.6m boulder wall
2	1.0m fill	24	1.0m fill	16	Garden/lawn
3	Garden/lawn	25	1.8m fill	17	Garden/lawn
4	Garden/lawn	26	1.8m fill	**Seaview Street**	
5	Garden/lawn	27	Vacant	1	2.4m blockwork wall
6	Garden/lawn	28	Vacant	2	2.8m blockwork wall
7	Garden/lawn	29	Vacant	3	2.4m junk wall

Lot No.	Description	Lot No.		Description	Lot No.
8	Garden/lawn	Hillcrest Ave/Ocean Street		4	2.2m fill
9	Garden/lawn	1	Lawn/garden	5	1.5m-2.0m fill
10	1.0m sleeper wall	2	Lawn/garden	6	1.5m fill
11	0.8m rock wall	3	Lawn/garden	7	1.5m fill
12	1.0m-1.5m fill	4	Lawn/garden	8	1.5m fill
13	1.0m fill	5	Lawn/garden	9	1.0m fill
14	1.0m fill	6	Lawn/garden	10	1.6m rock/boulder wall
15	1.0m fill	7	1.8m boulder wall, 7m from fence.	11	1.2m sleeper wall
16	1.5m fill	8	Garden/lawn	12	1.8m boulder wall
17	1.5m fill	9	1.0m fill, pool	13	Garden/lawn
18	None	10	Garden/lawn	14	1.0m sleeper wall
19	None	11	1.0m fill	15	Up to 1.8m boulder wall
20	1.8m blockwork wall	12	1.4m fill, set back 1.4m	16	Vacant
21	0.5m fill	13	1.0m fill	17	Vacant

The area of the adjacent hillside between and downslope of approximately Lot 13 on Ridgeway Crescent and Lot 26 on Hillcrest Avenue (ref. Drawing 1) requires specific consideration due to the presence of three landslips downslope of these allotments. As discussed in Section 3.2, the water-softened soils, failed material and surface of rupture (slip plane) will be excavated out of the slip areas and permanent drains installed. The slip areas and surrounding area will then be backfilled with engineered fill, burying the former landslip areas.

It is recommended that any large retaining structures be located downslope of the base of the basalt/metamorphic interface layer and that backfill be formed no steeper than approximately 5H:1V. Retaining structures located near the property boundary should be limited in height to 2.0m as described in Section 3.2.

It is considered that the proposed remedial measures in the area of the existing landslips will substantially improve the stability of these areas by improving drainage, removing existing failed-material and the surfaces of rupture, backfilling with engineered fill and supporting of the lower areas with appropriate engineered retaining structures.

3.7 Design Parameters for Retaining Structures

It is understood that the preferred designs for retaining structures on this site are concrete crib-type walls and boulder walls. Geotechnical design parameters for retaining structures constructed on this site are presented in Table 3.

TABLE 3: CHARACTERISTIC DESIGN PARAMETERS FOR RETAINING STRUCTURES

Material Type	Unit Weight (kN/m^3)	Effective Cohesion, C′ (kPa)	Effective Internal Angle of Friction, ϕ'	Unfactored Earth Pressure Coefficient, K_a+
Residual Clay	20	5	27	0.38
Engineered Clay Fill	18	5	25	0.42
Sand Backfill	18	0	30	0.33
Gravel Backfill	20	0	35	0.27

\+ please note that correction factors should be applied to the earth pressure coefficient to allow for sloping backfill and/or sloping rear wall surface.

Material strength and uncertainty reduction factors on the values listed in Table 3 should be applied in accordance with AS 4678 - 2002.

Table 4 summarises the allowable bearing pressures for design of retaining wall footings founded on different materials.

TABLE 4: ALLOWABLE BEARING PRESSURES FOR RETAINING WALL FOOTINGS

Founding Material	Allowable Bearing Pressure (kPa)
Stiff CLAY	150
Very Stiff CLAY	200
Hard CLAY/Very Low Strength Rock	300
Engineered Fill	100

It should be noted that the allowable bearing pressure for engineered clay fill is relatively low. This may limit the height of retaining structures founded in engineered clay fill to approximately 2m. Alternatively, footings may be deepened to found in natural soils using mass concrete or piered footings. This may have implications on the location and design of retaining structures, particularly where they cross infilled gullies.

For retaining structures or buildings founded on fill, some settlement of the fill and retaining structure may be experienced due to long-term consolidation of the fill under its self-weight and applied structural loads. Settlement due to long-term consolidation is typically around 0.01 times the vertical fill thickness. Provided the design bearing pressure does not exceed the recommended values presented in Table 4, vertical settlement under the toe of the footing due to structural loads is expected to be less than 25mm. As an example, for a footing founded on 2m of engineered fill, a vertical settlement of the order of 20mm may be expected under the toe of the footing. A building founded on 10m of fill may experience 100mm of long-term settlement.

3.8 Retaining Wall Design

3.8.1 Structure Classification

Preliminary designs of crib-type retaining walls have been undertaken in accordance with AS 4678-2002. For the purposes of the Standard, these walls have been classified as Type 2 structures in accordance with Section 1 of AS 4678. Provided occupied structures such as houses are to be limited to not closer than 3 m to the top face of the wall, or 3m to the bottom face if they are below the wall, then only moderate damage and negligible to low risk to life is anticipated in the event of a failure.

Crib walls, being internally interlocked, will tend to fail in a mode not dissimilar to reinforced soil walls. In accordance with Appendix A of AS 4678 therefore, the distance beyond which structures will have no affect on these wall classifications is 3m for walls up to 5m high, and 5m for walls from 5m to 8m high.

For houses and structures located up to 3m from a 5m to 8m high wall, it is recommended they be founded on piers or piles as described below. This will reduce the likelihood of damage in the event of a collapse.

3.8.2 Design

Table 5 presents the results of these preliminary designs for walls of various heights along the rear boundary of the site. The designs for these walls assume a 26° (2H:1V) slope between the property boundary and the crest of the wall. The "internal" walls, located within the site, assume a 6° slope above and below the wall and a 5kPa surcharge above the wall, with a 20 kPa surcharge (structural load) no closer than 5m from the rear of the wall. Where buildings are to be located closer than 5m to the rear of the wall, they should be founded on deep footings or piers such that the founding level is below a line drawn up at 2V:1H from the rear of the base of the wall. The designs of the walls have used the characteristic design parameters presented in Table 3. Some details of the general design and layout of crib walls is shown on Figure 2.

TABLE 5: SUMMARY OF CRIB-TYPE WALL DESIGN

Crib-Type Walls on Rear Property Boundary					
Typical Wall Height	Width of Top of Wall	Width of Base of Wall	Height of First Row of Cribs	Height of Second Row of Cribs	Height of Third Row of Cribs
2m	1.2m	1.2m	2.0m	--	--
3m	1.2m	1.2m	3.0m	--	--
4m	1.2m	2.4m	4.0m	2.0m	--
5m	1.2m	2.4m	5.0m	2.5m	--
Crib-Type Walls on Internal Boundaries					
6m	1.2m	2.4m	6.0m	3.0m	--
7m	1.2m	2.4m	7.0m	3.5m	--
8m*	2.4m	3.6m	8.0m	5.5m	3.0m

* All the above wall designs, with the exception of the 8m high wall, limit the allowable bearing pressure under the wall footing to 200kPa. It should be noted that 8m high walls require an allowable bearing

pressure of 250kPa, corresponding to a very stiff to hard clay. If materials of these types are not present under the wall footing, alternative footing designs incorporating deepened strip footings or piered footings may be required to achieve the required allowable bearing pressure under the wall.

Concrete-type crib walls should be constructed on concrete footings reinforced with heavy gauge weld mesh. In order to resist water infiltration and softening of the foundation soils, it is recommended that the footings for crib-type walls be excavated to a depth of at least 0.6m into the founding soils. The outer edge of the concrete should be poured to ground level and the footing should include drains to prevent water ponding in the footing. To resist sliding, the lowermost structural members of the crib wall should be embedded into the concrete.

The area between the cribs and natural soils or engineered fill should be backfilled with free-draining material (eg. drainage gravel). Geotextile should be placed between the granular backfill and the natural soils or engineered fill. An aggie-type drain should be constructed along the rear of the wall to collect groundwater seepage and channel it into the stormwater reticulation system.

Where the basalt/metamorphic interface is present immediately to the rear of the retaining structure (ie exposed on the face of the temporary cutting) it is recommended that the design of the retaining wall be reviewed by an experienced Geotechnical Engineer or Engineering Geologist after visiting the site. The design of the retaining wall may require modification based on the site observations.

3.9 Construction of Boulder Walls

It is understood that boulder walls are to be used as retaining structures in areas other than along the property boundary and where the wall height will not exceed approximately 3m.

All boulder walls should be designed by a qualified Civil or Geotechnical Engineer for the local surface and subsurface conditions. The design should take into account subsurface conditions, surcharge and structural loads and local slope angles above and below the wall.

Some general guidelines for boulder wall construction area shown on Figure 3. Some construction recommendations are outlined as follows:

- The base row of boulders should be embedded in a mass concrete footing to resist sliding or rotation of the boulders. The mass concrete should be poured to ground level with the top of the concrete surface graded downslope to prevent water ponding within the footing and potentially softening the soils downslope or under the footing.
- The area between the boulder wall and natural soils or engineered fill should be backfilled with free-draining material (eg. drainage gravel). Geotextile should be placed between the granular backfill and the natural soils or engineered fill.
- Interlocking between the boulders is critical in maintaining the strength and performance of the boulder wall. The boulders should be selected to be tabular to rectangular prism in shape (ie not rounded or sub-spherical) to allow close contact and minimal voids between the boulders. Boulders should be placed with at least 30% of the boulders as "headers". In general, the boulders should interlock in such a way as to form a single, massive structure.
- Dental concrete may be used if preferred to infill voids between the boulders and increase the interlocking at boulder contacts.

Coffey can provide a more detailed specification for boulder wall construction if required.

For and on behalf of
COFFEY GEOSCIENCES PTY LTD

Information

Important information about your **Coffey** Report

As a client of Coffey you should know that site subsurface conditions cause more construction problems than any other factor. These notes have been prepared by Coffey to help you interpret and understand the limitations of your report.

Your report is based on project specific criteria

Your report has been developed on the basis of your unique project specific requirements as understood by Coffey and applies only to the site investigated. Project criteria typically include the general nature of the project; its size and configuration; the location of any structures on the site; other site improvements; the presence of underground utilities; and the additional risk imposed by scope-of-service limitations imposed by the client. Your report should not be used if there are any changes to the project without first asking Coffey to assess how factors that changed subsequent to the date of the report affect the report's recommendations. Coffey cannot accept responsibility for problems that may occur due to changed factors if they are not consulted.

Subsurface conditions can change

Subsurface conditions are created by natural processes and the activity of man. For example, water levels can vary with time, fill may be placed on a site and pollutants may migrate with time. Because a report is based on conditions which existed at the time of the subsurface exploration, decisions should not be based on a report whose adequacy may have been affected by time. Consult Coffey to be advised how time may have impacted on the project.

Interpretation of factual data

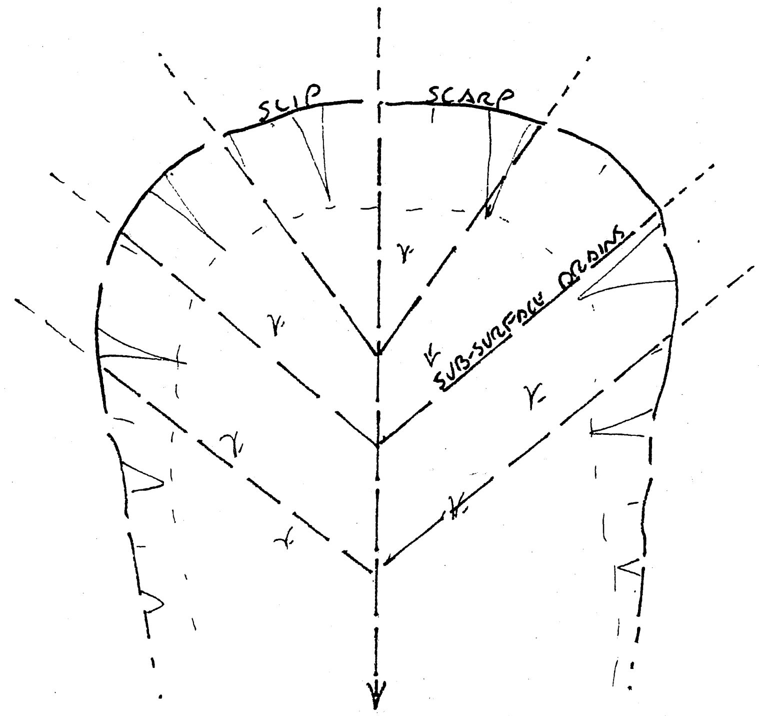
Site assessment identifies actual subsurface conditions only at those points where samples are taken and when they are taken. Data derived from literature and external data source review, sampling and subsequent laboratory testing are interpreted by geologists, engineers or scientists to provide an opinion about overall site conditions, their likely impact on the proposed development and recommended actions. Actual conditions may differ from those inferred to exist, because no professional, no matter how qualified, can reveal what is hidden by earth, rock and time. The actual interface between materials may be far more gradual or abrupt than assumed based on the facts obtained. Nothing can be done to change the actual site conditions which exist, but steps can be taken to reduce the impact of unexpected conditions. For this reason, owners should retain the services of Coffey through the development stage, to identify variances, conduct additional tests if required, and recommend solutions to problems encountered on site.

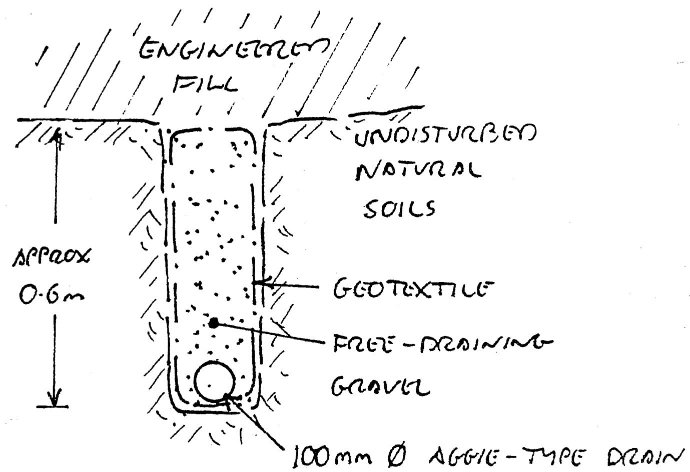
Your report will only give preliminary recommendations

Your report is based on the assumption that the site conditions as revealed through selective point sampling are indicative of actual conditions throughout an area. This assumption cannot be substantiated until project implementation has commenced and therefore your report recommendations can only be regarded as preliminary. Only Coffey, who prepared the report, is fully familiar with the background information needed to assess whether or not the report's recommendations are valid and whether or not changes should be considered as the project develops. If another party undertakes the implementation of the recommendations of this report there is a risk that the report will be misinterpreted and Coffey cannot be held responsible for such misinterpretation.

Your report is prepared for specific purposes and persons

To avoid misuse of the information contained in your report it is recommended that you confer with Coffey before passing your report on to another party who may not be familiar with the background and the purpose of the report. Your report should not be applied to any project other than that originally specified at the time the report was issued.





Coffey Geosciences Pty Ltd ACN 056 335 516		Geotechnical I Resources I Environmental I Technical I Project Management	
Drawn	KU	MARTIN FINDLATER AND ASSOCIATES PTY LTD Greenview Developments Pty Ltd Residential Development Fraser Drive, Tweed Heads South Some Details of Subsurface Drainage in Landslide Areas	FIGURE 1
Approved	IM		
Date	July 2002		
Scale	NTS		Job no: B17439/1-F

Coffey