Action 19.4: Appendix 5 – WML Consulting Preliminary Land Stability Assessment Clod Lane, Haslingden (Appendices 5 to 7)

APPENDIX 05

Coal Authority Report

Issued by:
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Our reference:	**51000241016001**
Your reference:	**44620472**
Date of your enquiry:	**07 March 2013**
Date we received your enquiry:	**07 March 2013**
Date of issue:	**07 March 2013**

This report is for the property described in the address below and the attached plan.

Non-Residential Coal Authority Mining Report

LINDON PARK, HASLINGDEN, LANCASHIRE,

This report is based on and limited to the records held by, the Coal Authority, and the Cheshire Brine Subsidence Compensation Board's records, at the time we answer the search.

Coal mining	See comments below
Brine Compensation District	No

Information from the Coal Authority

Underground coal mining

Past

According to the records in our possession, the property is not within the zone of likely physical influence on the surface from past underground workings.

Present

The property is not in the likely zone of influence of any present underground coal workings.

Future

The property is not in an area for which the Coal Authority is determining whether to grant a licence to remove coal using underground methods.

The property is not in an area for which a licence has been granted to remove or otherwise work coal using underground methods.

The property is not in an area that is likely to be affected at the surface from any planned future workings.

However, reserves of coal exist in the local area which could be worked at some time in the future.

No notice of the risk of the land being affected by subsidence has been given under section 46 of the Coal Mining Subsidence Act 1991.

Mine entries

There are no known coal mine entries within, or within 20 metres of, the boundary of the property.

Coal mining geology

The Authority is not aware of any evidence of damage arising due to geological faults or other lines of weakness that have been affected by coal mining.

Opencast coal mining

Past

The property is not within the boundary of an opencast site from which coal has been removed by opencast methods.

Present

The property does not lie within 200 metres of the boundary of an opencast site from which coal is being removed by opencast methods.

Future

The property is not within 800 metres of the boundary of an opencast site for which the Coal Authority is determining whether to grant a licence to remove coal by opencast methods.

The property is not within 800 metres of the boundary of an opencast site for which a licence to remove coal by opencast methods has been granted.

Coal mining subsidence

The Coal Authority has not received a damage notice or claim for the subject property, or any property within 50 metres, since 31st October 1994.

There is no current Stop Notice delaying the start of remedial works or repairs to the property.

The Authority is not aware of any request having been made to carry out preventive works before coal is worked under section 33 of the Coal Mining Subsidence Act 1991.

Mine gas

There is no record of a mine gas emission requiring action by the Coal Authority within the boundary of the property.

Hazards related to coal mining

The property has not been subject to remedial works, by or on behalf of the Authority, under its Emergency Surface Hazard Call Out procedures.

Withdrawal of support

The property is not in an area for which a notice of entitlement to withdraw support has been published.

The property is not in an area for which a notice has been given under section 41 of the Coal Industry Act 1994, revoking the entitlement to withdraw support.

Working facilities orders

The property is not in an area for which an Order has been made under the provisions of the Mines (Working Facilities and Support) Acts 1923 and 1966 or any statutory modification or amendment thereof.

Payments to owners of former copyhold land

The property is not in an area for which a relevant notice has been published under the Coal Industry Act 1975/Coal Industry Act 1994.

Information from the Cheshire Brine Subsidence Compensation Board

The property lies outside the Cheshire Brine Compensation District.

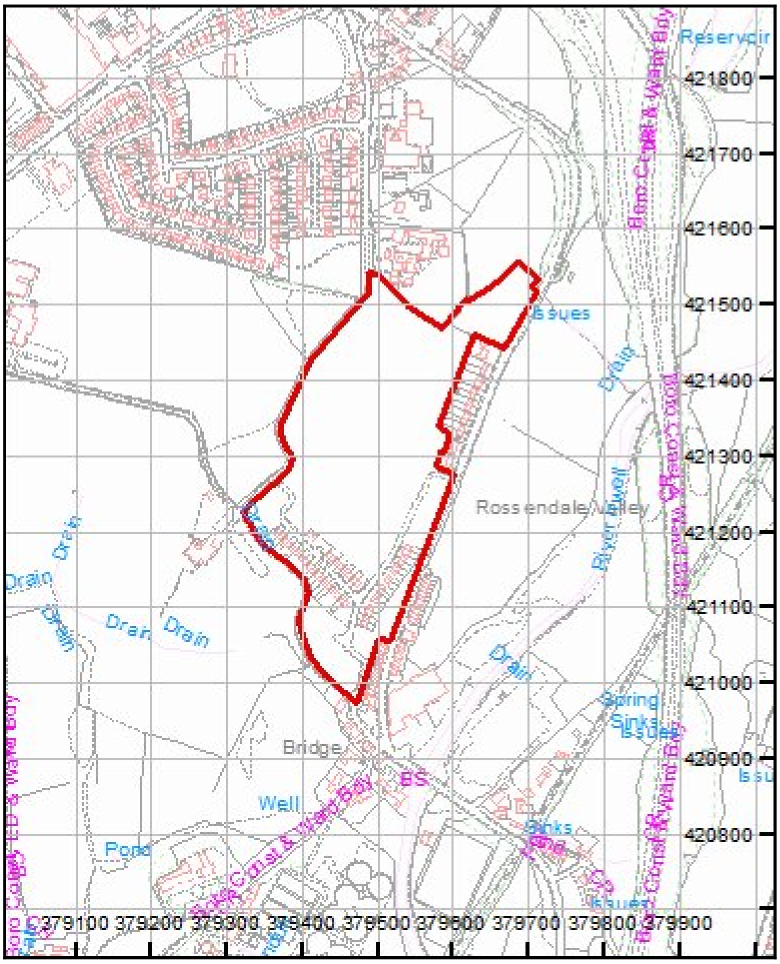
Additional Remarks

This report is prepared in accordance with the Law Society's Guidance Notes 2006, the User Guide 2006 and the Coal Authority and Cheshire Brine Board's Terms and Conditions 2006.

Location map

Approximate position of property

Enquiry boundary



Key

Approximate position of enquiry boundary shown

Mapping sourced from Ordnance Survey

APPENDIX 06

Archive Reports

ROWLINSON CONSTRUCTIONS LIMITED

HOUSING DEVELOPMENT AT HASLINGDEN

FURTHER

REPORT ON THE STABILITY OF THE SITE

ROWLINSON CONSTRUCTIONS LIMITED

HOUSING DEVELOPMENT AT HASLINGDEN

FURTHER REPORT ON THE STABILITY OF THE SITE

OCTOBER, 1980.

Allott & Lomax,
Consulting Engineers,
Fairbairn House,
23 Ashton Lane,
Sale,
Cheshire. M33 1WP.

CONTENTS

1.0 INTRODUCTION

Following completion of a detailed study into the stability of a hillside at Haslingden, a report prepared by Allott & Lomax in April 1978 was submitted to the Borough of Rossendale.

On two subsequent occasions certain comments on the report and requests for further clarification have been made by Dr. J. K. Alderman of Sub-Soil Surveys Ltd., acting in his capacity of adviser to the Borough.

This report answers the points raised by Dr. Alderman, gives the further clarification required and presents additional evidence and discussion in support of the conclusions reached in the April 1978 report.

2.0 GENERAL BACKGROUND

During construction of a housing estate on the site at Haslingden, by Rowlinson Constructions Ltd., serious damage occurred, in September 1973, to the foundations of four houses. Subsequently the Borough of Rossendale instructed Dr. J. K. Alderman of Sub-Soil Surveys Ltd. to report on the stability of the site of the development. The conclusions of his study, given in a letter to the Borough dated 6th August 1976, were that the site was on a relict landslip, which was showing evidence of recent movement and that calculated factors of safety based on the evidence then available were sufficiently low for the hillside to be considered as being potentially unstable until detailed field work could demonstrate otherwise.

In February 1977, Rowlinson Constructions Ltd., commissioned Allott & Lomax to carry out a similar study of the stability of the hillside based on the existing information. The findings of this study were presented in the report entitled "Housing Development at Haslingden - Report on the Stability of the Site" dated May 1977, which concurred with Dr. Alderman's main conclusion that the hillside was a relict landslip and should be considered as potentially unstable until further evidence could be produced. However in coming to this conclusion Allott & Lomax disagreed with Dr. Alderman in respect of recent movement of the overall hillside, concluding, instead, that there was no evidence to support his contention and that in fact strong evidence existed to the contrary.

In May 1977, Allott & Lomax were instructed by Rowlinson Constructions Ltd. to carry out a second stage of the study involving detailed exploratory and investigatory fieldwork to establish the ground conditions at the site. This second stage work was presented as an addendum to the above report, dated April 1978, in which the ground conditions controlling the stability of the hillside were demonstrated. The main conclusions of this report were that the hillside was currently stable and that the factors of safety against reactivation of movement were such that at least partial development of the site was possible provided that it was carried out under the guidance of specialist geotechnical engineers.

The complete report was submitted to the Borough of Rossendale by Rowlinson Constructions Ltd. in May 1978. Dr. Alderman was to consider the report and present his comments and conclusions on the stability of the site to the Borough. He reported in a letter to the Borough dated 20th October 1978, in which it was apparent that he was not fully in agreement with the submitted conclusions. In September 1979 it was agreed between Rowlinson Constructions Ltd. and the Borough of Rossendale, that Allott & Lomax and Dr. Alderman should meet to discuss the differences in their assessment of the site and, if necessary, agree a mutually acceptable course of action to resolve any remaining differences. This meeting was held on 23rd October 1979 when detailed points were discussed and evidence obtained since April 1978 was presented. Dr. Alderman agreed to reconsider his opinion in light of the argument and further evidence presented and to this end Allott & Lomax provided Dr. Alderman with drawings, records and calculations requested by him, under cover of their letter dated 24th November 1979.

In a letter to Allott & Lomax and the Borough dated the 17th April 1980, Dr. Alderman, instead of giving his expected reconsidered assessment of the position in light of all the information available to him, again raised a number of questions and requested a further report justifying the conclusions already submitted to the Borough in the April 1978 report. At a meeting of all parties on 11th July 1980, Allott & Lomax were commissioned to produce such a report for Dr. Alderman's final consideration.

3.0 POINTS RAISED BY DR. ALDERMAN

3.1 General

Dr. Alderman has raised points on the submission to the Borough on two occasions, namely his letter report dated 20th October 1978 and his letter to Allott & Lomax and the Borough dated 17th April 1980. Points raised in the former letter are discussed in Section 3.2 below, except where they have been repeated in the second letter in which case they are discussed with the additional points raised, under Section 3.3.

Copies of the two letters are included as Appendices A and B to this report, but for convenience the relevant portions of text have been reproduced with each individual discussion.

3.2 Letter dated 20th October 1978

3.2.1 "3.1 Ground Investigation

> ...... We also consider that it would be advisable to install a series of inclinometer tubes to establish if there is continuing movement along the former slip planes."

This point was discussed with Dr. Alderman at the meeting held on 23rd October 1979. During this discussion it became clear that, whereas Dr. Alderman had used the term 'inclinometer', he was in fact referring to the simpler and less expensive slip indicator.

Small diameter plastic tubes have been installed and maintained in all boreholes in conjunction with porous filters at their base to act both as piezometer tubes (for water pressure measurement) and slip indicator tubes. Such installations extend to the base of all boreholes and thus penetrate well below the level of the slip planes established from the field investigation work.

The continuity of these tubes, throughout their depth, has been monitored four times since 1977. It is still possible (October 1980) to pass a 0.64m long reference rod to the base of all boreholes, with the exception of No. 107, where the deepest tube is blocked at a high level. This therefore demonstrates that no continuing movement along former slip planes has taken place since these tubes were installed.

3.2.2 "3.2 Ground Conditions

> ...... The underlying rock is stated to have been proved in all the boreholes, except B.H.'s 101 and 106 at depths varying between 26m and 42m below ground level. It should be noted however that the 'rock' was generally only proved for 0.6 to 1.3m so it is possible that in some of the boreholes the rock is a boulder, particularly in B.H.'s 101A and 102."

The possibility that in some of the boreholes material described as rock is a boulder cannot entirely be ruled out. However, considering all the 'rock' levels, a consistent and plausible profile of the rock surface can be produced. This gives adequate assurance that the rock has in fact been proved for the present purposes.

While the rock may have had some bearing on the original failure of the hillside, the geometry of any future landslip will be largely dictated by the existing failure planes in the clay soils above rock level. Thus the precise location or identification of the rock surface is not of major importance.

3.2.3 "3.3 Groundwater Pressures

> (a) ...... From the above it can be seen that there is a large and irregular variation in the water pressures down the slope (Section A) which may require further investigation, depending on the values assumed in the slope stability analysis."

The variation in water pressures and the fact that at depth the pressures are below hydrostatic is consistent with under-drainage to the more permeable solid deposits beneath the drift cover. This behaviour is commonly encountered in the Pennine areas.

Regarding the water pressures used in the stability analysis reported on in April 1978, these were based on the values measured in the field. For analytical reasons, it was necessary to consider a groundwater table or phreatic surface and to model the under-drainage effect by taking stepped reductions in water pressure below the hydrostatic level. The water pressure on the critical slip surface determined by this approach for Section A is plotted on Figure 1 together with the maximum field values as at February 1978, taken from piezometers at or close to the slip surface. Interpolation between readings from piezometers above and below the slip surface was made where no piezometer exists on the slip surface. From the figure it will be seen that the pressures used in the analysis agree closely with the actual maximum measured values at the time of the report. A similar approach was used for Section B. Subsequent to February 1978 continued monitoring of the water levels has permitted a fuller appreciation of the water pressures and their variation. This has led to a re-analysis of the slope the results of which are outlined and discussed in Section 4.4 of this report.

"3.3 (b) In some cases the water pressures plotted in Figs. 13 and 14 of the Allott & Lomax report are lower than the maximum values plotted in Appendix L ......"

At the time of the Addendum report it was considered that some of the piezometers might not have fully stabilised, while others for example B.H. 104 at 133.0m O.D. appeared to be leaking. In preparing Figures 13 and 14 of the Addendum report the maximum water pressure reading has normally been recognised. The principal case where a departure from this was made was for B.H. 101A at 156.2m O.D. At the time it was suspected that the piezometer was malfunctioning, giving an incorrectly high reading. An estimated value based on readings from other piezometers was used in the plot shown on Figure 13. The water pressures used in the analysis were based on the piezometer readings together with the plots on Figures 13 and 14. However in using the modelling techniques discussed above a reasonably conservative approach was taken. In the event the water pressure in the vicinity of borehole 101A used in the analysis closely modelled the maximum recorded value, as can be seen from Figure 1 of this report.

"3.3 (c) The water pressures in B.H. 106 are lower than in similar levels in B.H. 107 although B.H. 106 is higher up the hillside."

It is accepted that the water pressures in boreholes 106 and 107 appear anomalous. The critical piezometer in this respect is in borehole 106 at 163.90m O.D. which gave a comparatively low reading. This point was recognised and a water test carried out on the piezometer at the beginning of March 1978. The piezometer was found to be functioning satisfactorily such that there was no reason to doubt the validity of the readings from this instrument. The difference in water pressures between boreholes 106 and 107 is probably a consequence of the under-drainage effect described earlier.

3.2.4 "3.7 Construction Evidence

Allott & Lomax state that they now consider that the failure of plots 50-53 was caused by sliding of the fill material along the original ground surface thereby causing an imbalance of pressures acting on the rear wall, the failure being probably exacerbated by poor construction.

We consider that calculations should be submitted to substantiate this analysis."

Before entering into a detailed discussion of this failure it is important to restate the relevance of this particular piece of evidence. It was originally invoked by Dr. Alderman as being evidence to support his contention that overall hillside movements were occurring. Examination of the evidence by Allott & Lomax in 1977, led to the conclusion that it was more likely to be a local failure. Examination of the field evidence in the Addendum report confirmed this conclusion and allowed an explanation of the failure to be put forward. The exact cause of failure is however of secondary importance, the fundamental point being whether or not it was local.

The hypothesis of failure put forward in the 1978 report is based on strong field evidence, which inter alia, includes the existence of cracks in the ground surface of the fill material immediately in front of the rear wall of plots 50-51. This shows that, irrespective of analytical substantiation, some form of slip movement had occurred in the fill thereby giving rise to conditions of imbalance of earth pressures on the house foundations.

Simple calculations have been carried out to support this hypothesis and it is possible to demonstrate that failure could occur in this manner although a number of assumptions, albeit not unreasonable, are necessary. Undue weight has therefore not been placed on the calculations other than to indicate that such a mode of failure is possible in the circumstances existing on the site. This gives support to the almost indisputable field evidence that the failure was a local event and not indicative in any way of overall movement of the hillside.

3.2.5 "3.9 <u>Hillside Stability</u>

> 3.9.2 ...... A comparison is made between sections in which it is stated that Section B is approximately 18% more stable than Section A. We question this simple comparison as there is a considerable variation in both the ground conditions and water pressure between the two sections..."

The ground conditions on Sections A and B at and above the existing slip plane have been demonstrated to be similar in that, in each case, the major portion of the slip surface passes through the same type of clay soil which has been given the same strength value and in that the overlying material is similar in both cases. The average slope angle for Section A is only slightly steeper than for Section B. To demonstrate this point an analysis in which water pressure was excluded was carried out. This gave a factor of safety of 1.44 for Section A and 1.50 for Section B, a difference of only 4%, which is attributable to small variations in section geometry. The overall difference of 18% between the two sections is therefore predominantly due to the variation of only one factor, that is, the water pressure.

Although the comparison may be simple the fact remains that at present Section B is demonstrably approximately 18% more stable by comparison with Section A. No evidence exists to suggest that changes in water pressure would be markedly different between sections as only a short distance is involved. As such, it is considered that the approach, albeit simple, of comparing Section A with Section B is valid in giving the order of comparative stability between the two sections. Indeed in discussing recommended factors of safety, the Civil Engineering Code of Practice No. 2 (ref. 30) refers to the approach of making comparisons in slips at the same or on a neighbouring site where the strata are known to be similar.

3.3 Letter dated 17th April 1980

3.3.1 Residual Shear Strength (ϕ_r')

"(a) The Residual Shear Strength

Your computer analysis E5, 8 and 9 indicates that a difference of only one degree between the assumed and actual ϕ_r values (residual shear strength) could lower the factors of safety by about 8.4%, i.e. a lowering of the factor of safety for Section B from 1.34 to 1.23.*"

*The actual computed value was 1.21 from analysis E9.

It is normal to carry out a parametric study of important parameters to ascertain the sensitivity of the factor of safety to theoretical changes in the value of the parameter. In this study, a semi-arbitrary variation of 1° in this parameter was selected purely to allow a graph of variability of factor of safety with residual shear strength to be drawn. A variation of $\pm$ 2° or some other value could equally well have been used. Dr. Alderman's implication that the range selected for illustration of sensitivity can actually occur in practice is not necessarily valid. Any such theoretical variation should not be confused with the likely variation in practice, the assessment of which is a completely separate exercise.

Dr. Alderman has quoted part of Clause 6.3 of the Addendum report out of context and in so doing makes an implication in respect of lower factors of safety that might be misconstrued. The clause goes on to say "It is probable that during the life of the hillside as it exists at the moment, the rainfall has exceeded the recorded maximum. Despite higher rainfall and the resulting unfavourable changes in the groundwater regime there is no evidence to suggest that the hillside in its present form has experienced any movement."

The complete clause invokes the strong evidence which shows that the hillside has not moved for at least 130 years, thereby demonstrating its ability to safely withstand the changes, if any, in the groundwater regime caused by extreme precipitation over that period, irrespective of the value of computed factors of safety.

Regarding the computer analyses referred to by Dr. Alderman, he is quite correct in noting that the difference in factors of safety for Sections A and B is almost entirely due to the difference in groundwater pressures. This point has been discussed earlier in Section 3.2.5.

3.3.3 Site Topography and Assumed Soil Strata

"(c) Site Topography and Assumed Soil Strata

What variations in the factor of safety are considered possible due to variation in the ground contours and soil strata in areas between and away from the boreholes, in particular in the area between Sections A and B?"

Again, referring to Section 3.2.5 it is seen that there is only a 4% variation in the computed factor of safety between Sections A and B arising from variation in the ground contours and soil strata.

From Section A to Section B there is a small and gradual change in the slope angle. In the area between Sections A and B it is therefore reasonable to expect the variation in factor of safety arising from change of ground contours will be small and uniform.

Regarding the soil strata, the high plasticity deposits, which are the soils that principally govern the stability of the hillside, were found on both Section A and Section B at similar levels and were also present at Holme Wood Bend immediately to the north of the site. In view of this it is considered reasonable to expect continuity of the high plasticity deposits between Section A and B. Variation in the factor of safety arising from variation in the soil strata is therefore also likely to be small and uniform.

3.3.4 Assumed Slip Surface

"(d) Assumed Slip Surface

Only one slip surface has been analysed so it is possible that lower factors of safety may exist along other possible slip zones. In particular it appears that a lower factor of safety may be obtained for Section A if alternative slip lines are considered to the west of borehole 102."

Dr. Alderman's statement that only one slip surface has been analysed is not understood. Clause 5.3 of the Addendum report states "All the reasonably postulated failure surfaces on Section A, which are shown on Figure 13, have been analysed to determine the critical one." In fact eight slip surfaces were analysed.

In response to the suggestion by Dr. Alderman that a lower factor of safety may be obtained by considering the slip lines to the west of borehole 102, a further potential failure surface has been analysed. This surface is shown in Figure 3 and was selected to commence at the break in slope to the west of borehole 102. The calculated factor of safety for this surface is 1.10, compared with 1.09 for the critical surface already established. Thus the critical surface is that which was originally identified from a thorough investigation and alternative slip surfaces with lower factors of safety are unlikely to exist.

3.3.5 Effect of Tension Cracks

"(e) Effect of Tension Cracks

It appears that the effects of possible tension cracks in the clay surface have not been allowed for in your stability analysis.

What will be the reduction in the factor of safety if tension cracks are assumed?"

Consideration of the possible effects of tension cracks in the clay surface is more appropriate to undrained stability analysis where the cohesion of the soil, a parameter necessary for the formation of tension cracks, is that which predominantly controls the stability. In the analysis of the long term stability of drained slopes, as is the case at Haslingden, it is generally accepted that the correct conservative approach is to allow for any cohesion element in the soil strength to decay completely with time. In the analyses carried out effective cohesion has been ignored in all cases, thereby precluding tension cracks.

A simple calculation can demonstrate that the assumption of a tension crack filled with water gives an increase, albeit small, in the factor of safety and not a reduction as suggested by Dr. Alderman.

3.3.6 Theoretical Stability Analysis

"(f) Theoretical Stability Analysis

A small factor of safety will be required to allow for possible inaccuracies in the theoretical analysis."

It is understood that Dr. Alderman is implying that in practice failure of a slope may occur when the theoretical factor of safety as calculated is somewhat above unity, with the discrepancy between practice and theory arising from the inaccuracies in the analysis.

The studies of a number of workers based on back-analyses of actual slope failures (whose factors of safety were therefore unity or slightly below) have ascertained that in most cases the more rigorous methods of analysis, based on effective stress parameters for long term stability, will predict failure to within an accuracy of 10%.

In the present case the rigorous solution of Morgenstern and Price (ref. 31) was used in the analysis of both Sections A and B. In addition check analyses of Section B have been carried out using an alternative rigorous solution developed by Janbu (refs. 32, 33). The difference in calculated factors of safety given by the two methods is about 1.5%. Therefore, although an accuracy of $\pm$ 10% may be taken as a general value, this would appear to be somewhat conservative. The similarity in the results given by the two rigorous methods of analysis gives a high level of confidence that, in respect of analytical accuracy, the actual factors of safety are likely to be close to the calculated values.

4.0 FURTHER INFORMATION

4.1 General

Several items of further information relevant to the stability of the hillside have come to hand or been obtained since submission of the Addendum report in 1978. These items are discussed below.

4.2 Structural Surveys

In early 1979, the occupants of a number of properties on the site commissioned an independent survey to be carried out by C. C. Manley Esq., A.R.I.C.S., Chartered Surveyor. A total of seventeen properties on Linden Park Road (Estate Road 4) and Hill Top Drive (Estate Road 1) were surveyed. In the survey, carried out on 28th February 1979, two properties were surveyed in detail the remainder being subjected to a "cursory external inspection". The findings of this survey are summarised below in terms of structural defects to the walls of the properties:-

Defect	Number of Properties Affected
Minor shrinkage (internal)	1
Hairline fractures	10
Settlement cracks or fractures	5

Two properties were found to be free of defects.

In the reports of the survey opinion as to the cause of defects is given only for the two properties surveyed in detail. One of these had no significant defects but the other, No. 35 Linden Park Road as well as having minor internal shrinkage had settlement defects which appear from the report to be broadly similar to those in other properties reported as having settlement defects. The conclusions in the report with respect to No. 35 Linden Park Road are reproduced below:-

> "Of the external defects I would attribute the hairline fractures in face brickwork to be primarily due to either shrinkage or variation in the sub-soil conditions and the supporting foundations ....
>
> In conclusion I do not consider the defects reported are abnormal nor due to structural failure .... "

Dr. Alderman in his original report dated 6th August 1976, considered that cracking of structures erected on the site was evidence of recent movement of the hillside. This more recent information constitutes additional evidence to support the opposite view, concluded in the Addendum report, that damage to the properties erected on the site is not indicative of overall movement of the hillside.

4.3 Water Wells

It has been established that there are wells in the vicinity of the site from which water is either being or has been extracted. These wells have been investigated to establish whether or not they have any bearing on the hillside stability.

At present, as far as can be ascertained, water is only being drawn from one well. This well lies close to grid reference SD 792218, which is just over 0.5 km north-west of the inter-section of Section A, on the site, and Manchester Road. The owner of the well was licensed in 1970, to extract 30 million gallons/year (5,000 gallons/hour) from the well. The North West Water Authority, based on their knowledge of the aquifer, has estimated the drawdown in groundwater level which would result from a continuous extraction of water from this well at the licensed rate, to be approximately 1m at a distance of about 0.7 km from the well. However, it is understood that the actual extraction rate from this well was, until recently, about one-fifth to one-quarter of the licensed rate since when it has been further reduced by 50%. Furthermore it has been established that pumping has not been carried out continuously but has been regularly stopped at weekends and during works holidays.

On the basis of this information the resulting drawdown at the site is unlikely to have exceeded 0.3m and consequently if pumping from this well was discontinued a similar small rise in groundwater levels might occur at the site. The significance of this is discussed in Section 4.4.

From the turn of the century until 1956, water was also abstracted from wells located in the grounds of Rossendale General Hospital, previously Haslingden Union Workhouse and Haslingden Infirmary, which lies just over 1 km north of the site (map reference SD 796225). Details of abstraction rates are not known and it is understood that records were not kept. Nevertheless pumping ceased sufficiently long ago for the effects on groundwater levels at the site at Haslingden to have ceased.

Prior to the construction of wells on the hospital site and between 1956 and 1976 some water was obtained from a well 0.5 km to the north of the hospital. This well, which supplied a relatively small quantity of water to the hospital and supplied an adjacent farm, is distant from the housing site and is located on the opposite side of a major fault. This pumping is therefore considered to have no possible influence on groundwater levels beneath the housing site.

Since 1976, the hospital has obtained all its water from the mains.

4.4 Groundwater Conditions

4.4.1 Further Monitoring

The analyses of stability given in the Addendum report were based on the results of monitoring of groundwater pressures on the slip planes up to February 1978 as discussed in Section 3.2.3 of this report. Since that time monitoring of groundwater pressures took place regularly at approximately monthly intervals over a period of 19 months until September 1979. The results of this monitoring work were made available to Dr. Alderman in November 1979. A single further set of readings has been taken in October 1980 as a check of the situation, the results of which are given in the table in Appendix C.

Most of the piezometers have shown little or no variation in water pressure throughout the overall period, indicating that they are not responsive to short term variations in rainfall. Regarding the piezometers installed either on or close to the slip planes, two of these have shown variations in excess of 0.5m. One of these piezometers, in borehole 107, gave two consecutive readings, in December 1978 and January 1979, which were about 2.7m above the previous almost constant recorded levels. Since that time however the pressures have returned to the original levels and have remained constant. The other piezometer, in borehole 103, has shown variations of a similar order but these involved fluctuations in pressure over the period from September 1978 to June 1979. Concerning the possible malfunction of one of the piezometers in borehole 101A, as mentioned in Section 3.2.3, the steady rise in water level up to April 1978 was followed by a fall in level up to June 1978. Since then the water pressure has remained almost constant.

4.4.2 Re-analysis

In September 1979 it was considered that a sufficiently long term appreciation of the groundwater pressures had been obtained to allow re-analysis of Sections A and B to be carried out. The maximum recorded water levels at that time were used, in spite of, for example, the levelling out of the pressure in borehole 101A to a level below the peak reading. The short term high value in borehole 107 was however excluded. Figures 4 and 5 show the values used in the model for the re-analysis together with the recorded peak water pressures excluding the peak in borehole 107. The results of this re-analysis have also been made available to Dr. Alderman. They show little change in the results presented in the Addendum report, the actual figures being 1.09 for the factor of safety for Section A and 1.32 for Section B.

In October 1980, following the further set of readings, the analysis for Section B was repeated to take account of the short term peak value for borehole 107. In so doing the various peaks which occurred at different times were combined giving a very onerous modelling of the groundwater conditions. The resulting factor of safety for Section B was 1.30.

4.4.3 Understanding of the Regime

The full records now available confirm the interpretation given in the Addendum report indicating a comparable groundwater behaviour for both Sections A and B. The uppermost groundwater table lies in the superficial deposits at a depth of about 0.5m to 1m below ground level and the water pressures decay increasingly below the hydrostatic level with increasing depth below this water table, to an extent which is consistent with a considerable degree of underdrainage to underlying permeable strata. These permeable strata were penetrated by most of the boreholes and the overall records confirm that the water pressures within them are probably connected with water levels in the River Irwell at the toe of the slope. The water pressure profile in the permeable strata lies at or close to hydrostatic with respect to the river water level. On the lower parts of the slope, approaching the River Irwell, the drift cover reduces and the groundwater regime in the drift deposits tends to merge with that of the permeable deposits giving a reduced under-drainage effect.

It is known that the underlying permeable strata form an extensive aquifer and thus in terms of response to rainfall, water levels in them would be expected to be insensitive to short term variations in rainfall. This is confirmed by the groundwater measurements taken from the end of 1977 onwards, from which it appears that even monthly or seasonal variations in rainfall had little or no effect. It is probable therefore that for the water levels in the permeable strata to respond significantly, major changes in rainfall, for example that arising from climatic change, would be required.

The drift strata are comparatively impermeable so that only the uppermost few metres, if any, are likely to experience water pressures that reflect short term variations in rainfall. In effect the critical slip surfaces at some 20 to 25m depth are protected by a thick layer of low permeability material. The water pressures on these slip surfaces are therefore almost certainly controlled by the lower water table and therefore are likely to be sensitive only to major variations in rainfall. Again this is borne out by the field measurements which show little or no fluctuation in the water pressures on the slip surfaces.

In respect of the overall groundwater regime sufficient evidence exists for it to be considered on a regional basis rather than on a local one. The development of the regime and therefore the mechanism controlling the landslip has probably been conditioned by geological and climatic processes.

4.4.4 Future Water Levels and Stability

During the extensive period over which groundwater pressures have been monitored little or no fluctuation has been recorded. Such fluctuations that have been recorded appear to be local and unrelated and in addition it has not been possible to relate these to rainfall. Future short term variations of rainfall are therefore unlikely to result in significant fluctuations in groundwater levels.

Similarly as can be seen from Section 4.3 possible future rise in pressure due to cessation of pumping in the area is of no significance.

Concerning possible longer term fluctuations, rainfall records are available for many years which show, for example, annual variation in rainfall. However it is not possible to predict water pressure variation from the rainfall data unless a direct relationship between the two has been established. This can only be achieved by monitoring water pressures for a long period of time, possibly decades. In the absence of such evidence it would be imprudent to attempt to predict statistically, for example, the 100 year groundwater pressures. In considering longer term fluctuations it is therefore necessary to refer to the indirect evidence that the slope has remained stable for many years in spite of variations in rainfall, as was done in Section 6.3 of the Addendum report. At that time figures were quoted on monthly rainfall levels but in view of the evidence now available the use of monthly returns is not appropriate. Considering annual rainfall in 1954, for example, this was some 32% above the annual returns for the monitoring period. For comparison the statistically predicted 100 year figure is only some 9% above the 1954 value.

In more general terms the Meteorolgical Office reports that in 1979 England and Wales experienced the wettest spring (March to May) since 1727. Similar but slightly less extreme rainfall was recorded in 1782, 1818 and 1947.

If this slope is to be regarded as being in a state of limiting equilibrium under normal rainfall conditions as Dr. Alderman originally suggested and the water pressures are responsive to rainfall, then severe failure of the hillside should have occurred on the occasions of heavy rainfall. The fact that it has not moved is even more significant when compared with other relict landslips which have been reported as having experienced severe movement during the period of the Haslingden investigation, particularly in early 1977 and in the spring of 1979.

Although it is not possible to predict future water pressures in the slope, it is known that the hillside has been subject to extremes of rainfall in the past and has remained stable. Therefore either groundwater pressures have not risen during such periods or, if they have, they have not risen sufficiently to jeopardise the stability of the hillside. There is no reason to suppose that the slope should fail in the future under similar extremes of rainfall.

A parametric study was carried out to determine the pressures required to produce failure of the sections considered. A typical theoretical water pressure line to cause failure is shown on Figures 4 and 5 for Sections A and B respectively. The line chosen is for illustrative purposes only, it being recognised that there are in fact an infinite variety of such lines. From comparison with the actual pressures, it will be seen that a very considerable rise in pressure, almost to hydrostatic for Section B, is required to produce failure. There is no basis upon which to suggest that such pressures could be achieved or even approached in practice.

5.0 ACCEPTABILITY FOR DEVELOPMENT

5.1 Introduction

In discussions with Dr. Alderman, it became clear that one of his principal concerns was demonstration of the acceptability of the numerical factor of safety quoted in the Addendum report. In the discussion which follows an appraisal of risk in geotechnical engineering has been carried out with particular reference to slope stability. This has then been compared with the results from the Haslingden study.

5.2 Appraisal of Acceptability

5.2.1 General

The design, or analysis, of a civil engineering structure involves the following uncertainties:-

(i) uncertainty in dead loads

(ii) uncertainty in live loads

(iii) uncertainty in material properties

(iv) uncertainty in analytical methods

(v) uncertainty arising from construction.

There is therefore a risk of unserviceability or failure in all civil engineering projects.

The relative importance of the uncertainties above depends on the nature of the project. Considering a hillside site which has been subject to detailed investigation, as at Haslingden, uncertainty in dead load is of secondary importance. Uncertainty in live loading is principally variation in groundwater pressure and this has been discussed in Section 4.4.

The only uncertainty in material property of significance is the shearing resistance of the soil through which the failure plane passes. Uncertainty in analytical methods is allowed for as discussed in Section 3.3.6 and construction uncertainties are, in this instance, of secondary importance.

The design of a structure attempts to take into account those uncertainties affecting it to ensure that there is, generally, an acceptably low chance that unsatisfactory behaviour will occur due to likely variations. The term design is used in the text for convenience but the discussion applies equally to analysis of an existing situation.

Three design safety concepts in current use are reviewed below with specific reference to slopes of the type existing at Haslingden. The old and new traditional approaches are considered followed by a more recent approach which has been termed the "modern" approach. The requirements arising from these approaches are then summarised to form a basis for comparison with the results from the Haslingden study.

5.2.2 The Traditional Approach

5.2.2.1 Total Factor of Safety

The safety of a structure is traditionally established during design by determining the applied loads and the ultimate ability of the structure to resist them. The degree of safety is expressed as a ratio of these two values to form a total factor of safety. Thus a single factor is used to provide security against all uncertainties and does not differentiate between unserviceability and collapse modes of behaviour. The value of the factor will vary depending on the particular project.

Design values of total factors of safety have been evolved from experience over a long period to give an unquantified, but acceptable, risk of unsatisfactory behaviour. Where parameters affecting stability have been assessed too onerously very conservative designs have resulted. Conversely where low factors of safety are used there is a consequent increase in the risk of failure. The factor of safety adopted, and the associated level of risk, is determined by a combination of economic and engineering considerations relating to each particular project.

In this country there are no codes of practice governing the choice of a factor of safety for earthworks or slopes. The Civil Engineering Code of Practice No. 2, Earth Retaining Structures (ref. 30) may be considered to come closest to giving an indication of acceptable factors of safety in respect of slope stability. In this Code a factor of safety of 1.5 is required against activation of a first time slide, where soil strength is measured only in the laboratory. Where a slide has occurred the back-analysed strength parameters may be used in the design of remedial works or treatment of a nearby slope together with a factor of safety of 1.25. It is implicit in this Code that first time slides are being considered, reactivation of old slips not being mentioned.

Only relatively recently has the importance of differentiating between residual and peak shear strengths of soils been widely appreciated. The resistance will, on sufficient shearing (sliding) of the soil mass, reduce from a peak to a residual value due to physical changes on the surface of sliding. The residual shear strength is a fundamental and limiting property of a particular soil under a given stress level. It is now accepted that residual shear strengths are applicable to the reactivation of an old slide as at Haslingden. It is therefore important when considering factors of safety that the limiting nature of residual shear strength is appreciated. This allows the acceptance of lower factors of safety than would be appropriate for peak strength conditions, namely first time slides.

Recommendations for factors of safety for the long term (fully drained) stability of slopes have been extensively reviewed (refs. 2, 7-9, 11-26) and the numerical summary below tabulates these recommendations. Some of the references do not specifically refer to residual conditions and have been omitted in the third column.

Recommended Acceptable Total Factor of Safety	Number of Recommendations	
	Undifferentiated	Residual Only
<1.1	3	2
1.1 - 1.2	3	2
1.2 - 1.3	11	3
1.3 - 1.4	5	1
1.4 - 1.5	5	0
>1.5	2	0

Certain organisations produce Standards which give requirements for factors of safety. Examples of such recommendations are listed below:-

Organisation	Required Total Factor of Safety	Reference
National Coal Board	1.2 - 1.35	17
Soviet Union - large earth dams	1.3	18
New York State	1.15	23

A. L. Little of Binnie & Partners (ref. 26) recommends a factor of safety of 1.0 as being an acceptable limit, provided effective cohesion is ignored. From discussions with other leading consultants it is understood that a factor of safety of about 1.25 for residual landslip analysis is generally considered to be acceptable.

It would appear from the foregoing that a total factor of safety between 1.20 and 1.30 would be considered acceptable to a wide range of expert organisations and individuals when considering a relict landslip.

5.2.2.2 Partial Safety Factors

The partial factor concept has been established in Europe for some time and is now generally considered to be a more rational approach. The partial factor concept recognises that the parameters affecting stability have different relative importances. It also recognises the different uncertainties, that is the differences in the variability of individual parameters and in the accuracy to which they may be measured. These differences are therefore taken into account explicitly rather than implicitly as is the case with the total factor concept.

Partial safety factors are applied to the strength parameters (material factors) and to the loads (load factors). There may also be provision for a "social" factor to reflect environmental and social considerations arising from the project. Safety of the structure is then tested by subtracting the factored loads from the factored strength. If a positive difference is obtained then the structure is considered to be safe, conversely a negative difference implies an unsafe situation.

This concept is currently adopted by several British Codes of Practice but no code exists to cover geotechnical work, the Code of Practice No. 2 for example being nearly 30 years old. However, several European codes exist which do apply the partial factor concept to geotechnical work.

Partial factors recommended by some of the European Codes of Practice are summarised below. It should be noted that these factors are intended to be applied to conservative values of the parameter considered.

Country	Partial Safety Factors				Reference
	γL	γw	$\gamma c'$	$\gamma \phi'$	
Norway	1.0	(a)	1.5 (combined)		12
Denmark	1.0	1.2(b)	1.5	1.2	15, 35
Sweden	1.0	-	1.5	1.2	11
Germany	-	-	1.3	1.1	34

Where γL = dead load
γw = pore water pressure
$\gamma c'$ = effective cohesion
$\gamma \phi'$ = effective angle of friction

Note (a) This Code recommends that a parametric study of groundwater pressures is carried out to establish the sensitivity of stability to variations in this parameter. The results of this study should be combined with knowledge of groundwater pressures and used in the assessment of stability.

(b) This is a check factor to be applied with γL = 1.2 and all other partial factors set at unity.

It should be noted that no differentiation is made between peak and residual shear strength parameters, and thus these factors can be considered to be somewhat conservative when applied to a residual problem.

It would appear from the foregoing that, for acceptability of a slope under residual conditions, with effective cohesion assumed as zero, the appropriate material factor would be 1.2 applied to conservative values of the residual angle of friction. In addition the stability should be checked according to note (b) and the sensitivity of stability to groundwater pressure changes should also be established and taken into account in accordance with note (a).

5.2.3 A Modern Approach

5.2.3.1 Introduction

The fundamental limitation of the total and partial factor concepts is that the actual level of risk is unknown. Thus, for example, in spite of two structures both having the same factor of safety the risk to one structure may be different from that to the other. These differences in risk arise from the differences in the uncertainties affecting each structure. Even with a generous factor of safety there may be a high risk of failure and the converse is equally valid.

This limitation has been recognised for some time (refs. 1, 5, 6, 10, 28, 29) but until relatively recently techniques for overcoming it have not been developed.

Development of the techniques is still continuing and the approach is now receiving recognition. However, it is not yet widely used other than in the nuclear industry where it is common to design against extreme hazards on the basis of their probability of occurrence.

5.2.3.2 Assessment of Risk

Methods have been developed whereby it is now practicable to consider stability in terms of probability theory in which parameters may be assigned statistical values representing their measured means and variations (refs. 1, 3, 4, 5, 8, 29). This probabilistic approach also allows the factor of safety to be related to the risk of failure hence the chance of a factor of safety being achieved may be determined for a particular project. The actual relationship between factors of safety and probability of failure are considered in section 5.2.3.4.

5.2.3.3 Consideration of Acceptable Levels of Risk

The determination of an acceptable level of risk is subjective and will be influenced by the nature of the project and its social and environmental context. However reference to the behaviour of structures and common risks experienced in everyday life has provided a basis for establishing acceptable risk levels (refs. 1, 6, 8, 11, 27). The table below summarises some of the general levels of risk which apply to certain events. It should be noted that values quoted represent orders of magnitude rather than precise levels of risk.

Event	Risk	Reference
Daily risk of death from any cause	10^{-5}	11
Annual risk of death in U.K. from any cause - dependent on age	$10^{-2} - 10^{-3}$	1
Annual risk of death in U.K. from motoring accident	10^{-4}	1
Annual risk of death in U.K. from accident in home	10^{-4}	1
Annual failure rate of earth dams	$10^{-3} - 10^{-4}$	8, 21

Note: for example 10^{-4} = 1 in 10,000 chance of occurrence

The above figures show that an annual risk of death of 10^{-4} is generally acceptable to a developed society and this is the conclusion drawn in the CIRIA Report No. 63 (ref. 1). It is reasonable therefore to expect that engineering structures should not have an annual risk of failure of less than 10^{-4} where there is a likelihood of consequential death. DeMello (ref. 11) has concluded similarly that structural risk levels should range from 10^{-4} to 10^{-6}.

5.2.3.4 The Relationship Between Risk and Factor of Safety

The relationship between risk and factor of safety in geotechnical engineering is unique to each soil system. However, there is a general relationship between probability of failure, factor of safety and variability of the strength parameters. For a given probability of failure a parameter having a high variability, e.g. cohesion, will require the use of a higher factor of safety than would a parameter having a low variability, e.g. angle of friction (refs. 3, 4, 7, 8, 9, 29). At Haslingden, the strength parameter governing the stability is the residual friction angle. As this has a very low variability, a comparatively low factor of safety will have an associated high level of confidence.

5.2.4 Summary of the Appraisal

Three design safety approaches have been considered to determine the acceptability of a slope in respect of the likelihood of a failure occurring. For slopes of the type existing at Haslingden at least one of the following should be fulfilled:-

(i) total factor of safety should exceed a figure between 1.20 - 1.30.

(ii) the strength of the hillside should exceed the applied loading when a partial factor of 1.2 is applied to the friction parameter. The notes detailed in Section 5.2.2.2 should also be taken into account.

(iii) the probability of the applied loading exceeding the available strength should not be greater than 10^{-4}.

5.3 Acceptability at Haslingden

5.3.1 Introduction

In considering the acceptability of the slope at Haslingden, the two principal parameters to be taken into account are the groundwater pressure on the slip surface and the shear strength of the soil on the slip surface.

As discussed in section 4.4.4 future short term variations of rainfall are unlikely to result in significant fluctuations in in groundwater levels. For long term variations in groundwater pressures, no information is available and it must therefore be recognised that it is not possible to include definitive values in the analytical work. Any consideration of long term stability must remain a matter of engineering judgement.

For the analytical work the maximum recorded water levels have been recognised so that the calculated factors of safety provide a safeguard against instability arising from these maximum levels being exceeded. Similarly where a probability study has been carried out, the mean factor of safety and exceedence levels are in respect of the maximum water pressures. By taking the maximum water levels therefore, a conservative approach has been used.

Regarding the shear strength of the soil, as discussed in Section 3.3.1, the residual angle of friction has been taken and any cohesion value neglected. As with the water pressures therefore a conservative approach has again been used.

5.3.2 Comparison with the Traditional Approaches

5.3.2.1 Total Factor of Safety

The total factor of safety is deemed to provide measure of safety against "normally" experienced variations in ground which has been adequately investigated. The total factor is therefore considered to take into account variability in soil strength and fluctuations in groundwater pressures. Using combined peak recorded water pressures and ignoring any cohesion for the soil strength the calculated factors of safety are as follows:-

	Calculated Factor of Safety
Section A	1.09
Section B	1.30
Minimum range required	1.20 - 1.30

It may be seen that Section B satisfies the requirements of the total factor of safety approach while Section A does not.

5.3.2.2 Partial Safety Factors

The partial safety factor concept also provides security against soil strength variations but in addition separately allows for an increase occurring to groundwater pressures above the levels measured.

The partial factor concept thus provides a better test of security than the total factor concept in that it recognises and explicitly allows for likely variations in the parameters governing stability. In addition it requires the demonstration of the amount of change in groundwater pressures necessary to bring about failure.

It has been assumed in the stability analysis that the cohesion parameter is zero and therefore the results obtained yield, in effect, a partial material factor with respect to the residual friction parameter, with the load factor equal to unity as required. The material safety factors are shown below for comparison:-

	Material Safety Factor
Section A	1.09
Section B	1.30
Minimum factor required	1.20

In accordance with this approach, a check was carried out with the dead load and pore water pressure partial factors set at 1.2. Under these conditions the minimum factor required is 1.0. A value of 1.16 was obtained for Section B. The parametric study of stability versus groundwater pressures indicated that, for Section B in particular, unrealistically large changes in groundwater levels are necessary to bring the slope to failure. It may be seen that Section B satisfies the requirements of the partial factor of safety approach. Section A however does not.

5.3.3 Comparison with A Modern Approach

5.3.3.1 Introduction

In order to make a comparison with the modern approach a risk analysis was carried out.

The risk analysis was based on the normal Monte Carlo simulation technique where stability trials are generated at random using the statistical data derived from the measured soil parameters. In this case 500 simulations were carried out. The confidence level of the results together with the order of the results was such that it was considered that no significant improvement would be obtained from additional simulations.

5.3.3.2 Risk Analysis

In the risk analysis, the actual measured soil strengths were used in the primary computation so that non-zero values for cohesion were included. Likewise the groundwater pressures taken in September 1979 were considered, that is including some of the peak values but not the combination of peak values. The results of the analysis are shown graphically on Figures 6a and 6b. For Section B the mean value for factor of safety by this analysis is 1.46 with cohesion included against the previously calculated value of 1.30 with cohesion ignored and combined peak water levels. The distribution shown on the above figures was corrected pro rata to centre on a mean value of 1.30. The results achieved are tabulated below.

Factor of Safety Section B	Chance of not Exceeding (%)	Probability of Not Exceeding
1.43	100	1
1.39	98	$\simeq 1$
1.29	41	4×10^{-1}
1.25	8	8×10^{-2}
1.23	2	2×10^{-2}
1.22	1	1×10^{-2}
1.21	0	$10^{-\infty}$

The risk of the applied loading exceeding the available strength is extremely low and less than 10^{-4}, so that Section B satisfies the probabilistic approach.

5.3.4 Discussion

Three approaches to demonstrate the acceptability, in engineering terms, of the hillside at Haslingden have been presented above. For Section A the total and partial factors of safety were considered to be insufficient and therefore a risk analysis was not carried out. By contrast, Section B was found to satisfy both the total and partial factor of safety approaches. Furthermore the calculated probability of failure was less than the maximum acceptable. The calculations are conservative in that cohesion has been excluded from the soil strength and the worst combination of recorded water pressures has been taken. On this basis therefore Section B is acceptably stable.

The major factor that might affect the stability is the long term variation in the groundwater pressures. However, as discussed earlier, the hillside has remained stable under past high annual rainfall as in 1954 and does not appear to be responsive to short duration extremes of rainfall. There is no reason to suppose that this situation should alter. The validity of this negative approach is discussed in reference 1. In addition the water levels required to produce failure, particularly for Section B, are so much higher than the existing levels as to be outside the extremes of credibility, unless a major climatic change occurred.

From the above study, Section B has been demonstrated to be acceptable in terms of overall stability. However Section A, although stable, does not have a sufficiently large margin against instability to allow it to be regarded as acceptable for the purposes of the proposed development. In section 3.3.3 of this report it has been shown that a gradual and uniform change in factor of safety may be expected between Sections A and B. The study of the total and partial factor of safety has determined that a value of 1.20 is the minimum acceptable. The calculated values for Sections A and B are 1.09 and 1.30 respectively so that a value of 1.20 would be achieved approximately midway between these two sections. This represents the limit of acceptability in terms of development . From figure 7, reproduced from the Addendum report, it may be seen that the previously recommended limit for development, that is the northern edge of Zones B, E and F lies approximately midway between Section A and Section B. The above work on acceptability therefore confirms that the recommendations of the Addendum report are acceptable.

6.0 SUMMARY

This report, in addition to answering specific points raised by Dr. Alderman, has reviewed the stability of the hillside at Haslingden in the light of all the information now to hand.

Since early 1977 a considerable amount of effort and money has been expended to justify development of at least some of the site. The investigations have established the geometry of the old landslip on which the site stands to a high level of accuracy. The material properties have been determined by direct measurement and the groundwater conditions have been monitored for an overall period approaching three years.

Analysis of this information, making conservative assumptions where necessary, has indicated that the southern portion of the hillside has a computed safety level which is regarded as acceptable.

No further information, other than from longer term monitoring, can be obtained to assist the judgement of acceptability in terms of theoretical stability. However the overall judgement which has to be made should be based not only on analytical results but should also take account of the engineering context of the problem and all the physical evidence of stability. It should be recognised that the site is not unique but is in fact similar to many other hillside sites in the area which have already been developed.

The additional information available and the further work that has been carried out has confirmed the overall judgements made in the Addendum report and given added confidence to them.

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σ_v' (kN/m²)	LINE 1		LINE 2		LINE 3	
	C_r'	ϕ_r'	C_r'	ϕ_r'	C_r'	ϕ_r'
100 - 200	2	11·5	0	11·5	0	10·5
200 - 300	7	10·5	0	10·5	0	9·5
300 - 400	13	9·5	0	9·5	0	8·5
400 - 500	20	8·5	0	8·5	0	7·5

Line 1 Regression line on test results

Line 2 Line used in analysis

Line 3 Line with a 1° reduction below line 2

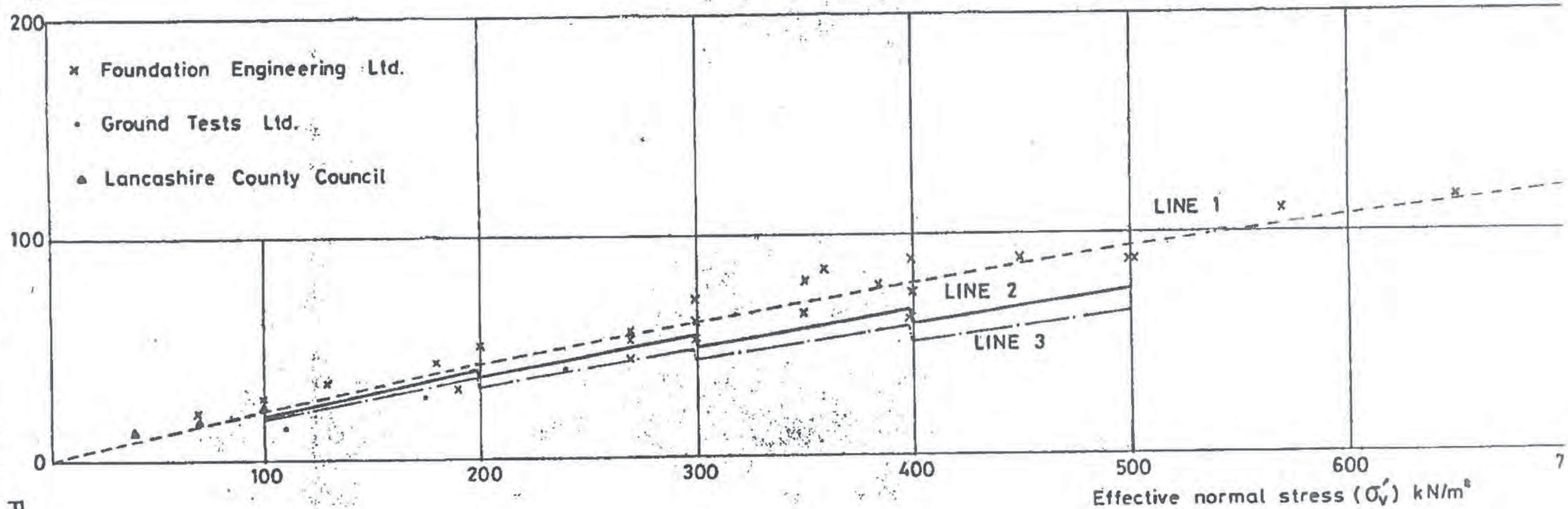

FIG. 2

SUMMARY OF RESIDUAL SHEAR BOX TEST RESULTS FOR HIGH PLASTICITY MATERIALS.

THORNCROFT MANOR SERVICES , DORKING ROAD , LEATHERHEAD KT22 8JB

ALLOTT & LOMAX SLOPE STABILITY RISK ANALYSIS 1.

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 **********
 * F      * FACTOR OF SAFETY
 **********

 VALUES     ------------------ CHANCE OF NOT EXCEEDING ----->
                 0%        20%       40%       60%       80%      100%
 ---------  ---------------------------------------------------------------
    1.325   100% 0         .         .         .         .         .    0%
            ---- 0         .         .         .         .         .  ----
    1.350   100% 0)        .         .         .         .         .    0%
            ---- (0)       .         .         .         .         .  ----
    1.375    97% (0)       .         .         .         .         .    3%
            ---- .(0)      .         .         .         .         .  ----
    1.400    92% . ((0)    .         .         .         .         .    8%
            ---- .    ((0))).        .         .         .         .  ----
    1.425    81% .       ((((0)))    .         .         .         .   19%
            ---- .         . (((0))) .         .         .         .  ----
    1.450    59% .         .        (((((0)))) .         .         .   41%
            ---- .         .         .     (((0)))       .         .  ----
    1.475    34% .         .         .         ((((0)))) .         .   66%
            ---- .         .         .         .      ((0))        .  ----
    1.500    14% .         .         .         .         . (0)     .   86%
            ---- .         .         .         .         .    (0)) .  ----
    1.525     5% .         .         .         .         .      (0). 95%
            ---- .         .         .         .         .        (0.  ----
    1.550     2% .         .         .         .         .         0)  98%
            ---- .         .         .         .         .         0)  ----
    1.575     0% .         .         .         .         .         0  100%
            ---- .         .         .         .         .         0  ----
    1.600     0% .         .         .         .         .         0  100%
            ---- .         .         .         .         .         0  ----
    1.625     0% .         .         .         .         .         0  100%
 ---------  ---------------------------------------------------------------
                100%       80%       60%       40%       20%        0%
                  <----- CHANCE OF EXCEEDING ---------------------

        PROBABILITY DISTRIBUTION DERIVED FROM  500 SIMULATIONS
```

FIG. 6a

```
29 SEP 80 12:03:00                 R I S K                                    PAGE   3
       THORNCROFT MANOR SERVICES , DORKING ROAD , LEATHERHEAD KT22 8JB

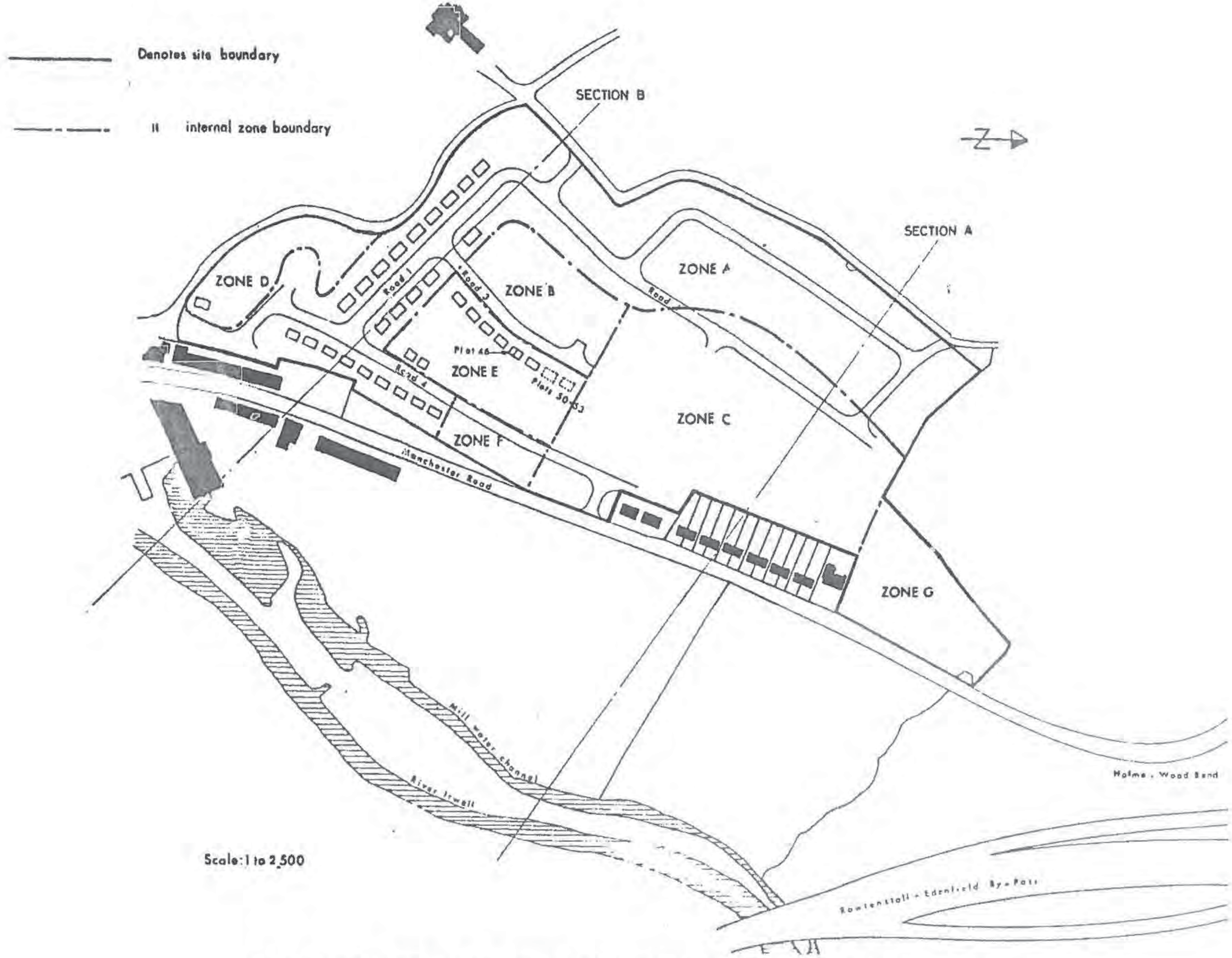
                 ALLOTT & LOMAX SLOPE STABILITY RISK ANALYSIS 1.
=================================================================================

**********
* F        * FACTOR OF SAFETY
**********

VALUES              ------------------ CHANCE OF NOT EXCEEDING ------>
                   0%         20%         40%         60%         80%        100%
---------          ---------------------------------------------------------------
    1.325    100% 0          .           .           .           .          . 0%
             ---- 0          .           .           .           .          . ----
    1.330    100% 0          .           .           .           .          . 0%
             ---- 0          .           .           .           .          . ----
    1.335    100% 0          .           .           .           .          . 0%
             ---- 0          .           .           .           .          . ----
    1.340    100% 0          .           .           .           .          . 0%
             ---- 0          .           .           .           .          . ----
    1.345    100% 0          .           .           .           .          . 0%
             ---- 0          .           .           .           .          . ----
    1.350    100% 0          .           .           .           .          . 0%
             ---- 0          .           .           .           .          . ----
    1.355    100% 0)         .           .           .           .          . 0%
             ---- 0)         .           .           .           .          . ----
    1.360     99% 0)         .           .           .           .          . 1%
             ---- (0         .           .           .           .          . ----
    1.365     98% (0         .           .           .           .          . 2%
             ---- (0)        .           .           .           .          . ----
    1.370     98% (0)        .           .           .           .          . 2%
             ---- .0)        .           .           .           .          . ----
    1.375     97% .0)        .           .           .           .          . 3%
             ---- (0)        .           .           .           .          . ----
    1.380     97% (0)        .           .           .           .          . 3%
             ---- .(0)       .           .           .           .          . ----
    1.385     96% .(0)       .           .           .           .          . 4%
             ---- .(0)       .           .           .           .          . ----
    1.390     95% .(0)       .           .           .           .          . 5%
             ---- . (0)      .           .           .           .          . ----
    1.395     94% . (0)      .           .           .           .          . 6%
             ---- . (0)      .           .           .           .          . ----
    1.400     92% .  (0)     .           .           .           .          . 8%
---------  -----------------------------------------------------------------------
                 100%        80%         60%         40%         20%         0%
                      <------ CHANCE OF EXCEEDING ---------------------

           PROBABILITY DISTRIBUTION DERIVED FROM 500 SIMULATIONS
```

FIG. 6b



ZONING OF SITE FOR DEVELOPMENT (AFTER 1978 REPORT - FIG.18)

FIG.7

APPENDIX A

SUB SOIL Surveys Ltd

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OUR REF JKA/FP

YOUR REF

20th October, 1978

Borough of Rossendale,
6 St. James Square,
Bacup,
Lancashire OL13 9AA

Dear Sirs,

Housing Development, Manchester Road, Haslingden

Further to your letter of the 1st June 1978 we have now examined and analysed the reports submitted by Allott and Lomax relating to the ground stability at the above site, and have the following comments to make:

1. Introduction

On the 6th August 1976 we submitted our report on the stability of the above site based on the available information, and in particular on the results of the two investigations carried out by Ground Tests Limited (See their reports 1248 B and 1326). A summary of our conclusions is given below:

1.1 The majority of the site, except possibly for the north-west end, to the west of the western scar (See Fig. 1) has in the past been subjected to large scale slipping. It is not known if this slipping had ceased prior to the recent development but evidence of excessive settlements and structural damage indicate that ground movements had occurred during the construction period which, were possibly due to a recent slipping of the slope.

1.2 We carried out a stability analysis of the hillside along section 1 (See Ground Test report - May 1975) based on the following main assumptions.

1.2.1 The ground surface profile is given by the contours on drawing No. PD 242/4A.

1.2.2 The water table profile in the lower part of the site is as given by profile "A" on drawing No. 1248B/8 (Ground Tests Ltd.). This water table varies from about 4 - 8m below ground surface level (Mean about 6m).

1.2.3 The Boulder Clay/Laminated Clay interface is planar across the whole of the hillside and we have carried out analyses assuming inclination of 3^{o} and $4\frac{1}{2}^{o}$.

1.2.4 The angles of residual shear strength for the clay deposits are as follows:

$\phi r = 26^{o}$ (Boulder Clay)
$\phi r = 9^{o}$ (Laminated Clay)

These stability analysis gave factors of safety at, or close to unity, indicating that if the assumptions are correct then there is a risk of slope instability, in particular when the ground surface levels and/or groundwater conditions are affected by construction work.

1.3 We recommended that further development of the site should not be carried out until it can be either

i) confirmed that there is an adequate factor of safety in particular areas, or

ii) adequate remedial measures are taken to improve the stability of the slope and produce an adequate factor of safety.

In order to establish accurately the stability we recommended that:

a) further boreholes be sunk to obtain more detailed information on the variation of soil strata below the site

and b) piezometers be established at various depths and locations within the glacial clays to establish the variation of groundwater pressures below the site.

2. Desk Study by Allott and Lomax

Following a request by Rowlinson Construction Limited on the 27th January, 1977 Allott and Lomax carried out a desk study of the currently available evidence and submitted their report on this investigation in May 1977. Our comments on this report are as follows:

2.1 In their clause 2.1 (conclusions) they state "that the hillside, on which the site is located has been subjected to large scale landslipping". This is in agreement with the first part of our clause 1.1 above. In their clause 2.4, however, they state that they are of the opinion that "there is no evidence arising from the developement of the site to date to suggest continuing large scale movement of the hillside".

2.2 In their clause 2.2 they state that "stability analyses based on the information available give very low factors of safety against reactivating movement of the hillside as a mass and nearby construction experience shows that reactivation is likely if major disturbance is caused". This is in agreement with our clause 1.2 above.

2.3 A large part of the report covers the investigation of the evidence of ground-movements and associated structural damage. In general attempts are made to indicate that these ground-movements, etc. have been produced by localised instability and not by an overall movement of the hillside. Stability analysis based on the proved soil by Ground Tests Limited have not, however, been carried out to substantiate this argument.

2.4 In conclusion Allott and Lomax recommended that the following further work be carried out, in particular to confirm or otherwise the "overall stability of the hillside", but also "to provide detailed conclusions on the local stability conditions in the presently developed areas of the site and firm proposals for methods of stabilising the site appropriate to these areas".

2.4.1 To carry out a site investigation to provide more detailed information regarding the ground conditions, and in particular

a) To prove the geology and to establish accurately the depth and profile of the primary interfaces down to bedrock.

b) To locate and confirm the presence of existing slip planes in order to understand the likely mechanism of previous hillside movements.

c) To obtain a better appreciation of the shear strength parameters of the superficial deposits, particularly of the laminated clay.

d) To establish the groundwater levels and water pressures existing in the hillside, together with their variations with time.

e) If found to be appropriate, to obtain a measure of the mass permeabilities of the various deposits, and in particular, of the bedrock.

and f) To carry out detailed geotechnical mapping of the hillside.

2.4.2 To c arry out a detailed topographical survey of the site, and

2.4.3 Investigation of the evidence of ground-movements in certain areas.

3. Site Investigation

In May 1977 Rowlinson Construction Limited instructed Allott and Lomax to carry out, as a second stage study, the further work recommended in their report (See clause 2.4 above). Details of this investigation are given in the "Report on the Stability of the Site" which was submitted in April 1978. Our comments on this investigation and report are as follows:-

3.1 Ground Investigation

The ground investigations generally consisted of the following:

a) Excavation of nineteen trial pits.

b) Sinking of nine shell and auger boreholes to depths varying between 25.8 – 49.2m below ground level. This borehole investigation was carried out by Foundation Engineering Limited between 1st September 1977 and 16th January 1978.

and c) The installation of 25 piezometers and 4 standpipes for recording the groundwater pressures.

Items b) and c) above are similar to our recommendations to Rowlinson Construction Ltd. on the 12th January 1977. However in this letter we also stated that "We also consider that it would be advisable to install a series of inclinometer tubes to establish if there is continuing movement along the former slip planes". These inclinometers do not appear to have been installed. I cannot understand this omission as these inclinometer tubes would immediately indicate any measurable movements along any former slip planes.

3.2 Ground Conditions

The boreholes generally revealed ground conditions similar to those assumed in the previous analysis and reports. However there were important differences in detail which indicate that the ground conditions are more complex than previously assumed.

Glacial Till is stated to have been proved in all the boreholes to depths of between 19m and 41m below ground level and overlying the Lacustrine Deposits which were up to 6m thick. The Lacustrine Deposits were underlain by gravels, silts and clays except in BH's 101A and 102 where these were absent.

The underlying rock is stated to have been proved in all the boreholes, except BH's 101 and 106 at depths varying between 26m and 42m below ground level. It should be noted, however, that the "rock" was generally only proved for 0.6 - 1.3m so it is possible that in some of the boreholes the rock is a boulder and not bedrock, in particular in BH's 101A and 102.

3.3 Groundwater Pressures

The piezometer and standpipe readings indicate that generally the water pressures are close to hydrostatic at upper levels. However below these upper levels the water pressures generally fall increasingly below hydrostatic pressure with increasing depth through the glacial till and lacustrine deposits, except for BH's 104 and 105. In these two boreholes which are relatively close to the river the recorded water pressures remained approximately hydrostatic in all the piezometers.

The above groundwater observations indicate that the water pressures at depth are lower than originally assumed from the results of the Ground Test investigation, in particular within the lacustrine deposits. This reduction in the water pressures will tend to increase the factor of safety and stability of the slope. There are, however, a number of anomalies in the groundwater observation which require clarification, in particular regarding their application to the stability analysis calculations.

These are as follows:

a) The water pressures within the Lacustrine Clay along section A (BH's 101-105) are stated to be as follows:

BH.	Tip Level (m) A.O.D.	Water Pressure Level (m) A.O.D.
101	154.8	165.6
101 A	156.2	176.8
102	152.6	165.0
103	144.8	151.1
104	141.1	160.7

From the above it can be seen that there is a large and irregular variation in water pressures down the slope which may require further investigations, depending on the values assumed in the slope stability analysis.

b) In some cases the water pressures plotted on Figs. 13 and 14 of the Allott and Lomax report are lower than the maximum values plotted in Appendix L. (See BH. 101-170.8m tip level, BH. 101A-156.2m, BH. 104-133.0m, etc.). Which of the water pressures have been used in the slope stability analysis? Clause 5.5 of the Allott and Lomax report indicates that the water pressure profiles shown on Figs13 and 14 have been used, and not the highest observed values.

c) The water pressures in BH. 106 are lower than in similar levels in BH. 107 although BH. 106 is higher up the hillside.

3.4 Large Scale Landslipping

Discontinuities, which show evidence of shearing have been observed in the soil samples. A study by Allott and Lomax of the discontinuities shear zones confirms that the hillside has been subject to past large scale slipping and the planes along which such slipping is evident are deep and in general occur in the laminated clay above the lacustrine deposits.

3.5 Residual Shear Strengths

A summary of residual shear box test results for high plasticity clays have been plotted by Allott and Lomax on Fig. 15. These results include the values obtained from the recent investigation together with values from the previous investigation of the site by Ground Tests Limited and a nearby investigation by Lancashire County Council.

The results show a reduction in the angle of residual shearing resistance ($\phi r'$) with an increase in the effective normal stress, with values varying from about

$$8 - 12^{\circ}$$

with a mean value along the possible deep slip failure zone of about

$$9 - 10^{\circ}$$

This angle of residual shearing resistance is similar to the value assumed in our original calculations (See clause 1.2.3 above).

3.6 Slope Stability

Non circular stability analyses have been carried out by Allott and Lomax on section A (northern half of site - BH's 101-105) and section B (southern end of site - BH's 106-108) for the failure surfaces shown on Figs. 13 and 14.

They state that from these analyses the critical failure surfaces have been shown to be those passing through the laminated clay at, or just above the glacial till/lacustrine deposit interface. The minimum factors of safety determined by the analytical work are

1.14 for section A

and 1.34 for section B

and they also state that the analyses indicated that the stabiltiy fo the hillside is virtually unaffected by the house loading.

3.7 Construction Evidence

Allott and Lomax state that they now consider that the failure of plots 50 - 53 was caused by sliding of the fill material along the original ground surface thereby causing an *imbalance* of pressures acting on the rear wall, the failure being probably exacerbated by poor construction.

We consider that calculations should be submitted to substantiate this analysis.

3.8 Monitoring

The results of monitoring work show that within the accuracy of the survey methods used, no movements have been recorded during the eight month period over which records have been taken.

In our opinion it would have been advisable to instal inclinometer tubes through the former slip planes so that observations of any deep slipping could have been recorded.

3.9 Hillside Stability and Recommendations for the development of the Site.

3.9.1 Allott and Lomax have estimated the minimum factors of safety of the slope to be

1.14 for section A

1.34 for section B

and in clause 6.3 of their report they state that

a) "a factor of safety can generally be determined to within 10% of the actual value when, as in this case, the analysis is based on accurate ground information."

b) "Groundwater pressures, which are of fundamental importance in any assessment of stability are known to be influenced by rainfall". They state that since 1934 the rainfall at New Hall Filter Station in 1977 (i.e. period of investigation) has been exceeded by up to 50%. Higher groundwater pressures probably existed during these heavier rainfall periods resulting in lower factors of safety than those estimated from the recent investigation.

3.9.2 Allott and Lomax consider that the margin of safety for the central part of the hillside, represented by section A is not sufficient to allow development of that area of the site without unacceptable risk. The northern part of the hillside, being steeper, will have less margin.

They consider that the southern part of the hillside which is currently developed and represented by section B has a shallower slope and a factor of safety comfortably in excess of unity.

A comparison is made between two sections in which it is stated that section B is approximately 18% more stable than section A. We question this simple comparison as there is a considerable variation in both the ground conditions and water pressures between the two sections.

3.9.3 Allott and Lomax consider that the site is capable of selective housing development provided care is exercised in respect of local instability. They have sub-divided the site into six zones (See Fig. 18) and given comments on the stability of each of the zones.

They consider that in terms of overall stability of the hillside the existing development in Zone A is acceptable and that development can continue with care providing adequate provision is made for construction on a sloping site where appropriate.

4. Conclusions

In conclusion we have the following comments to make:

4.1 The recent investigation, and in particular the proving of shear planes at depth within the glacial deposits, has confirmed that the hillside, on which the site is located, has been subjected to large scale landslipping in the geological past.

These former landslips have produced weak zones within the laminated and lacustrine clays so that slope instability will now tend to develop at lower ground slopes and/or groundwater pressures than for hillsides which have not been subjected to landslips.

4.2 Allott and Lomax do not consider that there has been any recent major deep seated landslips and state that there is strong topographical evidence to show that the hillside has remained stable over the past 130 years.

They may be corrected but we consider that they should submit more detailed evidence, together with calculations, to confirm that the failures in plots 50 - 53 have been produced by local instability.

We also consider that a series of inclinometer tubes should have been placed into the hillside to establish if there are continuing movements along the former slip planes.

4.3 The recent investigation has generally confirmed that the basic assumptions made in calculating the stability of the hillside (ex. ground conditions, residual shear strength) are approximately correct, except for the assumed groundwater pressures.

The piezometers indicate that the water pressures along the deep slip zone during the period of the recent investigation were lower than those assumed in the original analysis. There will, therefore, be higher factors of safety against sliding so the hillside may have been stable during this period.

Allott and Lomax state that the minimum factors of safety determined by the analytical work are

1.14 for Section A
1.34 for Section B

The factor of safety, however, varies rapidly with changes of groundwater pressures and there are a number of anomalies in the groundwater pressure observations (See Clause 3.3 a) - b) above). Consequently it is not possible to comment on the accuracy of the above values until these anomalies have been clarified.

We consider that detailed stability analysis calculation and drawings for the critical slip surfaces should be submitted to the Local Authority so that a reliable estimate of the possible variations in the factor of safety can be obtained. These calculations and drawings should clearly show the following:

a) Scaled sections of the analysed slopes (Section A & B) showing the assumed soil conditions and the estimated location of the critical slip surface and b) Details of the assumed groundwater pressures and angles of shearing resistance along the critical slip surface.

4.4 We believe that the rainfall in the area (New Hall Filter Station) during 1977 (i.e. period of the recent investigation) has been exceeded by up to 50% since 1934. During these heavier periods of rainfall the groundwater pressures along the slip surface may have been higher than during the period of the recent investigation thus producing a lower factor of safety.

A further study of the groundwater pressures, including those taken recently since the completion of the investigation, should be carried out to see if there is any indication of the variation of groundwater pressures with rainfall values. In particular as about six of the piezometers were still rising up to the last recorded levels given in the report.

4.5 From the above it can be seen that Allott and Lomax have estimated that during the period of this investigation the factor of safety against hillside instability varied from about 1.14 in the centre of the northern half of the site (Section A) to about 1.34 at the southern end of the site (Section B). Lower factors of safety may, however, develop if there is a build-up of groundwater pressures along the slip failure zones.

These factors of safety are relatively low and do not give one confidence in giving a reasonable guarantee on the long-term stability of slope. We are of the opinion, therefore, based on the available information that the Local Authority should not accept responsibility for the long-term stability of the slope, although it is possible that the hillside may remain stable during the life of existing structutres placed on the site.

Yours faithfully,
p.p. Sub Soil Surveys Ltd.,

[redacted]

Dr. J. K. Alderman
Director

SUB SOIL Surveys Ltd

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17th April, 1980

21 APR 1980

Allott & Lomax,
Consulting Engineers,
Fairbairn House,
23 Ashton Lane,
Sale,
Cheshire.

6123/1 DRC

B.Taylor+Co.

22/4/80

6123/1

Dear Sirs,

Housing Development, Manchester Road, Haslingden

Further to my phone discussions yesterday with your Mr.Broomhead I write to confirm the following points:

1. At our meeting on the 23rd October 1979 I requested further factual information and design calculations, in particular those detailed in clause 4.3 of my report to the Borough of Rossendale (20th October, 1978). This information was supplied by yourselves on the 26th November, 1979.

2. The main point of discussion at the meeting was regarding the long term stability of the slope and the various factors which will tend to produce a difference between the estimated and actual factors of safety for the slope.

I stressed that in my opinion Allott & Lomax had not, in their report (April, 1978) given adequate proof to the Local Authority so that both Allott & Lomax and the Local Authority could give a reasonable guarantee on the long term stability of the slope, in particular during the life span of any structures built or to be built on the part of the site where Allott and Lomax stated in their report that they considered that development could continue (i.e. Zone's A,B,D & E and possibly F).

A report on the above has not yet been received by either the Local Authority or myself.

I suggest that you now submit a report on the above to the Local Authority and comment on how possible variations to the assumed parameters used in estimating the stability of the slope will produce lower factors of safety for the sites than the two values quoted in your report which are only for two particular sections, i.e.

1.14 for Section A
1.34 for Section B

This will then enable a more accurate assessment to be made of the actual factor of safety and future stability of the sloping sites and the risk of future ground slips. In particular,I suggest you comment on the variation of the following assumed parameters used in the stability analysis of the slope.

a) <u>Residual Shear Strength</u> - Your computer analysis E5,8 and 9 indicates that a difference of only 1^{o} between the assumed and actual Ør values (residual shear strength) could lower the factors of safety by about 8.4%, i.e. a lowering of the F.O.S. for Section B from 1.34 to 1.23.

b) <u>Groundwater Pressures</u> - Your analysis is based on piezometric readings taken over the winter 1977/8 (clause 5.5). In clause 6.3 of your report state that since 1934 the rainfall in 1977 has been exceeded by up to 50%. Higher groundwater pressures are, therefore, to be expected during these very wet weather periods resulting in lower factors of safety.

It is also noted that your computer analysis E3 and E7 indicate that the difference between the factors of safety for sections A & B is almost entirely due to the difference in the assumed groundwater pressures.

c) Site Topography and Assumed Soil Strata -

What variations in the F.O.S. are considered possible due to variation in the ground contours and soil strata in areas between and away from the boreholes, in particular in the area between sections A and B ?

d) Assumed Slip Surface - Only one slip surface has been analysed so it is possible that lower factors of safety may exist along other possible slip zones. In particular it appears that a lower F.O.S. may be obtained for Section A if alternative slip lines are considered to the west of BH.102.

e) Effect of Tension Cracks - It appears that the effect of possible tension cracks at the clay surface have not been allowed for in your stability analysis.

What will be the reduction in the F.O.S. if tension cracks are assumed ?

d) Theoretical Stability Analysis - A small factor of safety will be required to allow for possible inaccuracies in the theoretical analysis.

Yours faithfully,
p.p. Sub Soil Surveys Ltd.,

[redacted]

Dr. J.K. Alderman
Director

c.c. Planning Officer, Borough of Rossendale (Ref.BPC/HMV)
A.W. Mawer & Co.

APPENDIX C

Results of Borehole Monitoring on 14th/15th October, 1980

Borehole No.	Piezometer (tip depth)	Water Level (m.bgl)	Dipped Depth* (m.bgl)	Depth of Failure Plane (m.bgl)
101	28.35)	destroyed		
	44.35)			
101A	4.85	0.85		
	10.55	3.37		
	19.15	8.71		
	33.15	19.62	33.2	25.0
102	26.45	17.67		
	30.15	30.06	29.6	26.0
103	13.40	6.00		
	27.75	17.91		
	35.30	21.07	35.3	26.0
104	12.75	1.08		
	22.30	3.05		
	30.35	1.01	28.6	22.0
105	19.20	3.13		
	27.80	2.42	16.5	10.0
106	4.85	0.58		
	12.00	5.07		
	23.10	22.30		
	42.50	40.54	40.5	23.5
107	4.85	0.93		
	15.50	8.26		
	26.85	21.26	26.9	25.0
	42.15	22.14	+	
108	4.85	0.48		
	11.50	6.64		
	17.50	8.99		
	22.50	18.21		
	33.00	19.34	32.7	18.0

* Using 0.64m long reference rod

+ Tube blocked to reference rod at 6m bgl

bgl - below ground level

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2nd April, 1981

Ian N. Goldthorpe, DIPL ARCH DIP TP ARIBA FRTPI,
Chief Planning Officer,
Borough of Rossendale,
6 St. James Square,
Bacup,
Lancashire OL13 9AA

Dear Sir,

HOUSING DEVELOPMENT, MANCHESTER ROAD, HASLINGDEN

We have now examinated and analysed the "Further Report on the Stability of the Site" submitted by Allott and Lomax (Dated October 1980) together with the factual information previously supplied by them on the 26th November, 1979 and have the following comments to make:

1. INTRODUCTION

1.1 The site was purchased by Rowlinson Construction Ltd. for housing purposes and work commenced on site in June 1972. Work commenced on services and roads followed by the erection of semi-detached houses, several were sold and occupied.

We have no knowledge of, or have received any information on ground investigations having been carried out on the site prior to construction.

1.2 Towards the end of 1973 it was noted that defects were appearing in houses under construction. The foundations to two pairs of houses appeared to have failed and other signs of settlement were evident in the majority of other buildings.

2. GEOTECHNICAL INVESTIGATION BY GROUND TESTS LTD.

2.1 Rowlinson Construction commissioned Ground Tests Ltd. in March 1974 to carry out a geotechnical investigation of the site and they produced at least three reports dated:

October 1974	(Ref. No. 1248B)
November 1974	(Ref. No. 1248B)
May 1975	(Ref. No. 1326)

The latter report was for the relatively level, high lying, western area of the site and consisted of the sinking of nine boreholes (BH's 14 - 22) to depths of 6.05m - 7.45m below ground level. This report states that "it is considered that the development of plots 113 - 195 inclusive will not be influenced by slope stability considerations".

2.2 The reports (Ref. 1248B - Oct./Nov. 1974) gave details and comments on thirteen boreholes (BH's 1-13) which were sunk to depths varying from 10.6m - 23.6m below ground level. These proved the succession of the virgin strata to be

Glacial Stony Clays
overlying
Glacial Laminated Clays

Piezometers were established in BH's 5 - 13 to establish the groundwater levels but the response zones for these are not given.

2.3 Ground Tests carried out stability analysis for three sections through the sloping site and state that the following minimum factors were obtained:

Section No.1 (Southern End):-	F.O.S. = 1.58	(Effective Stress Analysis)
Section No.2 (Centre):-	F.O.S. = 2.12	(Total Stress Analysis)
Section No.3 (Northern End):-	F.O.S. = 1.76	

Their wording regarding the required factor of safety is difficult to understand but our interpretation is that they consider that the Minimum Factor of Safety should be greater than

1.25 for a slope which will not be loaded by buildings

and 1.50 for a slope which will be loaded by buildings

Therefore they are stating that for this housing development site the minimum factor of safety should be greater than

1.50

As the estimated minimum factors of safety for the three sections are above this value and vary from 1.58 - 2.12 they state that

"Whereas analysis indicates that the hillside is at present in a stable condition, it should be recognised that there is only a small margin of safety. Indiscriminate interference with the natural slope and/or excessive or badly located loading could easily create a potentially hazardous condition".

2.4 It should be noted, however, that in the stability analysis the following shear strength parameters were used:

Glacial Stony Clay	Ød	$= 26^{\circ}$
Glacial Laminated Clay	Ød	$= 19^{\circ}$

The residual shear strength value was used for the Glacial Stony Clay but not for the Glacial Laminated Clay which according to the test results is

Ør $= 9^{\circ}$

If this value, which is only about a half of the adopted value, had been used then the minimum factors of safety would have been greatly reduced.

In the addendum Report No. 1248B (Nov.'74) lower factors of safety (1.28 - 1.44) were obtained for shallow slips through the Glacial Stony Clay but these were for localised slips in areas where the slopes were relatively steep and would not be supporting buildings.

3. During 1974 and 1975 additional further cracks continued to appear in the buildings and in January 1976 a confidential report on the site was obtained from the Geotechnical Section of the Surveyors Department of the Lancashire County Council's. This report considered the available information and in particular

carried out simple stability analysis assuming that the angle of residual shear strength for the Laminated Clay was 9° as given above. Factors of Safety of from 0.99 - 1.03 were obtained for two of the sections considered.

4. REPORT BY SUB SOIL SURVEYS (6.8.76)

In April 1976 we were commissioned by Rossendale Borough Council to undertake a detailed appraisal of the information available in respect of the stability of the site.

We submitted our report on the 6th August 1976 and our conclusions can be summarised as follows:

a) The majority of the site, except possibly for the north-west end, to the west of the western scar (See Fig.1) has in the past been subjected to large scale slipping. It is not known if this slipping had ceased prior to the development but we gave seven examples of ground movements and/or structural damage which indicated possible recent slipping of the slope.

b) Our preliminary stability analysis based on the available information indicated that the slope to the east of the western scar may have a factor of safety close to unity.

c) We stated that it would not be advisable to carry out further development of any part of the site until it could be either

i) confirmed that there is an adequate factor of safety against slope instability

or ii) adequate remedial measures are taken to improve the stability of the slope and produce an adequate factor of safety.

In order to establish accurately the stability of various parts of the site it would be necessary to carry out a detailed investigation and in particular to a) sink further boreholes in order to obtain more detailed information on the variation of soil strata below the site and b) instal piezometers at various depths and locations to establish the variation of the groundwater pressures below the site.

5. <u>REPORT BY ALLOTT & LOMAX (May 1977)</u>

<u>5.1</u> Following a request by Rowlinson Construction Limited on the 27th January 1977 Allott and Lomax carried out a desk study of the currently available evidence and submitted their report in May 1977. In this report they agree that, based on the available information

a) the hillside, on which the site is located has been subjected to large scale landslipping

and b) Stability analysis give very low factors of safety against reactivating movement of the hillside as a mass. They state that nearby construction experience shows that reactivation is likely if major distrubance is caused

but they considered that

a) the hillside has remained stable over the past 130 years.

b) there is no evidence arising from the development of the site to date to suggest continuing large scale movement of the hillside.

and c) there is strong evidence of local instability on the site.

They suggested that further site investigation works should be carried out and state that

"<u>Until this further work has been carried out the hillside, and hence the site, should be considered as potentially unstable.</u>"

5.2 In the report Allott and Lomax state that:

i) the hillside may be underdrained by the underlying bedrock, further complicating the groundwater conditions.

ii) Cut and Fill has taken place on the site during the development and up to 3.5m of hillside has been cut away in the area prepared for the plots on the west side of Road 4.

iii) A study of aerial photographs indicate that during the last 30 years the site has not altered in position or deviated from line over this period, although some re-alignment work to Manchester Road at Holme Wood Bend is discernible.

iv) In the Rossendale area it is local experience that slopes less than 8^{o} to the horizontal are generally stable, whilst steeper slopes may be subject to movements in unfavourable ground conditions. Since slope angles on the site, with the exception of the southern part, generally exceed this angle it would appear that the site generally has not reached the same stage of stabilisation as other areas .... it therefore seems probable that the hillside could, still, on a geological time scale, be experiencing movements.

6. REPORT BY ALLOTT & LOMAX (April 1978)

6.1 In May 1977 Rowlinson Construction Ltd. instructed Allott and Lomax to carry out, as a second stage study, the further work recommended in their previous report. Details of this investigation are given in their "Report on the Stability of the Site" which was submitted in April, 1978.

6.2 The ground investigation carried out consisted of

a) Excavation of nineteen trial pits

b) Sinking of nine shell and auger boreholes to depths varying from 25.8 - 49.2m below ground level.

and c) The installation of 25 piezometers and standpipes for recording the groundwater pressures.

The boreholes generally revealed ground conditions similar to those assumed in the previous analysis and reports, but there were important differences in detail which indicated that the ground conditions are more complex than previously assumed.

6.3 Groundwater pressure observations indicate that generally the water pressures are close to hydrostatic at upper levels, i.e. similar to that assumed in the previous reports. However below these upper levels the water pressures generally fall increasigly below hydrostatic with increasing depth through the glacial clays, except for BH's 104 and 105 which are relatively close to the river.

This reduction is considered to be due to the hillside being underdrained to a significant extent by the underlying bedrock (Millstone Grit) and possibly the overlying glacial gravels.

This reduction in the water pressures will tend to increase the factor of safety and stability of the slope.

6.4 Discontinuities, which show evidence of shearing have been observed in the soil samples. Allott and Lomax state that this confirms that the hillside has been subject to past large scale slipping and the planes along which such slipping is evident are deep and in general occur in the laminated clay above the lacustrine deposits.

6.5 The results of monitoring work show that within the accuracy of the survey methods used, no movements have been recorded during the eight months which records have been taken.

6.6 Non circular stability analyses were carried out on two sections. Section A was in the northern half of the site through BH's 101 - 105 whilst Section B was in the southern half of the site close to Road 1 and through BH's 106 - 108. From this analysis Allott and Lomax have estimated the minimum factors of safety to be

1.14 for Section A

and 1.34 for Section B

They consider that the margin of safety for the central part of the hillside, represented by section A is not sufficient to allow development of that area of the site, without unacceptable risk.

They consider that the southern part of the hillside which is currently developed and represented by section B has a shallower slope and a factor of safety comfortably in excess of unity.

6.7 From the results of their investigation Allott and Lomax consider that the site is capable of selective housing development provided care is exercised in respect of local instability. They have sub-divided the site into six zones (See Fig.18) and given comments on the stability of each of the zones. This selection has been carried out based on the assumption that the results from the stability analysis indicate that the area covered by Section B is 18% safer than the area of Section A.

They consider that in terms of overall stability of the hillside the existing development of Zone A is acceptable and that development can continue with care providing adequate provision is made for construction on a sloping site where appropriate.

7. REPORT BY SUB SOIL SURVEYS (20th Oct. 1978)

Following our examination and analysis of the reports by Allott and Lomax relating to the ground stability we had the following comments to make:

7.1 In our opinion it would have been advisable to instal a series of inclinometer tubes to establish if there is continuing movement along the former slip planes. These inclinometers did not appear to have been installed.

7.2 The groundwater pressures at depth, and in particular along the deep slip surface, are lower than originally assumed from the results of the Ground Test investigation. This reduction in water pressures will tend to increase the factor of safety and stability of the slope.

There were, however, a number of anomalies in the groundwater observations which required clarification, in particular regarding their application to the stability analysis calculations. These were detailed in our report.

7.3 Allott and Lomax stated that they now consider that the failure of plots 50 - 53 was caused by sliding of the fill material along the original ground surface. We considered that calculations should be submitted to substantiate this analysis.

7.4 We recommended that:

a) Detailed stability analysis calculation and drawings for the critical slip surface should be submitted to the Local Authority.

b) These drawings should give details of the assumed groundwater pressures and assumed shearing resistances along the critical slip surface.

c) A further study of the groundwater pressures should be carried out in particular, as the rainfall in the area (New Hall Filter Station) during 1977 (i.e. period of the investigation) has been exceeded by up to 50% since 1934. Consequently the groundwater pressures along the slip surface may have been higher during these heavy rainfall periods, thus reducing the factor of safety.

7.5 In conclusion we stated that:

"From the above it can be seen that Allott and Lomax have estimated that during the period of their investigation the factor of safety against hillside instability varied from about 1.14 in the centre of the northern half of the site (Section A) to about 1.34 at the southern end of the site (Section B). Lower factors of safety may, however, develop if there is a build-up of groundwater pressures along the slip failure zones.

These factors of safety are relatively low and do not give one confidence in giving a reasonable guarantee on the long-term stability of slope. We are of the opinion, therefore, based on the available information that the Local Authority should not accept responsibility for the long-term stability of the slope, although it is possible that the hillside may remain stable during the life of existing structures placed on the site."

8. On the 19th September 1979 we were requested by the solicitors acting on behalf of the Rossendale District Council to meet and have discussion with Allott and Lomax to see if the obvious conflict of opinion between us could be resolved. This meeting took place on the 23rd October 1979. At this meeting we requested further factual information and design calculations, in particular those detailed in clause 4.3 of my report to the Borough of Rossendale (20th October 1978). This information was supplied by Allott and Lomax on the 26th November 1979.

The main point of discussion at the meeting was regarding the long term stability of the slope and the various factors which will tend to produce a difference between the estimated and actual factors of safety for the slope. I stressed that in my opinion Allott and Lomax had not, in their report, given adequate proof to the Local Authority so that both Allott and Lomax and the Local Authority could give a reasonable guarantee on the long term stability of the slope, in particular, during the life span of any structures built or to be built on the parts of the site where Allott and Lomax stated in their report that they considered that development could continue (i.e. Zones A, B, D & E and possibly F).

9. A report on the above was not received by either the Local Authority or myself so following phone discussions I wrote to Allott and Lomax on the 17th April, 1980 clarifying my requirements. I suggested that they now submit a report on the above and in particular comment on how possible variations to the assumed parameters used in estimating the stability of the slope could produce lower factors of safety for the sites than the two values quoted in their report which were only for two particular sections, i.e.

1.14 for Section A
1.34 for Section B

I suggested that, in particualr they comment on possible changes in the Factor of Safety due to variations of the following assumed parameters and conditions used in the stability analysis of the slope

a) Site Topography and Assumed Soil Strata

b) Assumed Slip Surface

c) Effect of Tension Cracks

d) Theoretical Stability Analysis

e) Residual Shear Strength

f) Groundwater Pressures

The above report was not received so a meeting was arranged on the 11th July 1980 at the Offices of Allott and Lomax. After some considerable discussion it was agreed that Messrs. Allott and Lomax would supply by the end of August their comments upon the variations in the factor of safety on the site as requested in our letter of the 17th April 1980. This report was received by ourselves on the 13th November, 1980.

10. FURTHER REPORT ON THE STABILITY OF THE SLOPE BY ALLOT & LOMAX (October 1980)

10.1 In clause 3.2.1 Allott and Lomax state that the small diameter plastic tubes which are being used as piezometers are acting as slip indicators (i.e. simple inclinometer tube) in particular, the deep tubes which pass below the slip line.

They state that continuity of these tubes, throughout their depth has been monitored four times since 1977 thus demonstrating that no continuing movement along former slip planes has taken place since these tubes were installed (i.e. 1977). We agree with this statement.

10.2 SITE TOPOGRAPHY AND ASSUMED SOIL STRATA

In clauses 3.2.5 and 3.3.3 Allott & Lomax state that

a) the average slope angle for Section A is only slightly steeper than for Section B.

b) there is only a 4% variation in the computed factor of safety between Sections A and B arising from variations in the ground contours and soil strata between these sections. This is demonstrated by analysing the slopes in which the water pressures are excluded. This gave factors of safety of

1.44 for section A
and 1.50 for section B

One factor, however, that appears to have been ignored is the fact that Section B is <u>not along the line of maximum slope,</u> i.e. at right angles to the contours. In order to be along the line of maximum slope the section would have to be rotated about 20° to the north (See site layout drg. 242/33B).

We have carried out a computer stability analysis of this modified section B and the original section B and obtained the following values :-

Section B:- F.O.S. = 1.323 (i.e. equal to Allott & Lomax Value)

Modified Section B :- F.O.S. = 1.302

The modified section B is therefore, giving a lower factor of safety than that given by Allott and Lomax.

10.3 ASSUMED SLIP SURFACE

In clause 3.3.4 Allott and Lomax state that the critical surface is that which was originally identified from a thorough investigation and alternative slip surfaces with lower factors of safety are unlikely to exist.

They may be correct but insufficient evidence has been supplied to confirm this. The information supplied, however, indicates that only seven slip surfaces were analysed for Section A and one slip surface for Section B, i.e. the eight quoted in their report. There appears to have been only one slip surface analysed for both sections in the Laminated Clay so unless they have exactly located the "old slip surface in this deposit it would have been advisable to have considered alternative locations of the slip surface within the Laminated Clay".

10.4 EFFECT OF TENSION CRACKS

I do not agree with their statement in clause 3.3.5 that tension cracks should only be considered when carrying out an undrained stability analysis. Slope and excavation instabilities frequently occur when tension cracks are suddenly filled with water during rainy periods, in particular, as they enable a rapid build-up of pore-water pressures within the clay.

I would be interested to see "the simple calculations" which demonstrates that a tension crack filled with water increases the factor of safety.

10.5 THEORETICAL STABILITY ANALYSIS

In clause 3.3.6 Allott and Lomax state that in most cases the more rigorous methods of analysis, based on effective stress parameters for long term stability will predict failure to within an accuracy of 10%.

The two methods used in the analysis of Section A gave a difference of 1½%.

10.6 RESIDUAL SHEAR STRENGTH ($\emptyset_r$ value)

In clause 3.3.1 Allott and Lomax comment on the fact that a difference of only one degree between the assumed and actual $\emptyset_r$ values could lower the factor of safety from 1.34 to 1.21, i.e. a difference of about 10%. They state that the variation of 1^o was purely arbitrary and that a variation of $\pm 2^o$ or some other value could well have been used.

This is not absolutely correct because the choice of $\pm 1^o$ variation appears reasonable when the extreme variations as shown on the residual shear strength plot on Fig.15 (Allott and Lomax Report April '78) is about $\pm 1\frac{1}{2} - 2^o$.

We agree, however, with the Allott and Lomax statement that generally they have not used the mean $\emptyset_r$ value as indicated by the above Fig.15 but have used a more conservative value (See Fig.2), in particular, for effective normal stresses in excess of 300 kN/m^2.

10.7 GROUNDWATER PRESSURES

10.7.1 In clause 3.2.3 Allott & Lomax state that "The variation in water pressures and the fact that at depth the pressures are below hydrostatic is consistent with under-drainage to the more permeable solid deposits beneath the drift cover".

We agree with this statement in particular, with regard to the lowering of the water table due to under-drainage into the bedrock. This under-drainage may however, produce wide variations in the groundwater pressures at the base of the Laminated Clays because of variations in the permeability of the underlying drift deposits and/or bedrock. For example the Millstone Grits in this area could vary from shales (See BH's 103, 104 & 107) to sandstones (See BH's 101A, 102 & 105) and grits with a thick zone of gritstone (Brooks bottom Grit) possibly outcropping below the north-west corner of the site. The shales will generally have low permeabilities so in areas where the Laminated Clays rest directly on the shales there will tend to be reduced underdrainage.

This variable undrainage may partially explain the wide variations in the assumed ground-water pressures on the slip plane which are as follows :

BH	Assumed Pressure Head (m)
101A	15.4
102	8.1
103	12.5
104	18.1
105	7.4
106	0
107	8.4
108	8.0

The above variations do not give one confidence that the actual groundwater pressures along the slip plane can be estimated to an accuracy of say ± 10% in particular when one considers the wide variation between BH's 106 and 107.

We have calculated by computer, stability analysis of Section B that a variation of 1m. in the average groundwater pressures along the slip plane could reduce the F.O.S. by about 0.04 (i.e. about 3%).

10.7.2 In clause 3.3.2 Allott and Lomax agree that the difference in factors of safety for Sections A and B is almost entirely due to the difference in groundwater pressures.

10.7.3 The analysis of stability in the Allott and Lomax Report (April '78) were based on the results of monitoring of ground-water pressures on the slip planes up to February 1978. Since then the monitoring of groundwater pressures took place regularly at approximately monthly intervals over a period of 19 months until September 1979. A further set of readings was taken in October 1980 and we, together with Allott and Lomax took a set of readings on the 26th March, 1981.

Most of the piezometers have shown little or no variation in water pressure throughout the overall period, indicating that they are not responsive to short term variations in rainfall. Two of the piezometers have, however, shown short term rises in the water pressure, with piezometer 107/151.10m showing a temporary rise of about 2.7m and piezometer 103/144.75m showing a temporary rise of about 2.1m. Both of these piezometers have response zones either close to or on the slip planes.

Allott and Lomax have carried out a re-analysis of the slope stability based on the additional groundwater observations to October 1980 and have consequently reduced their estimated Factors of Safety to

1.09 - Section A
1.32 - Section B

If the short term peak observed in BH.107 is considered then the factor of safety is further reduced to

1.30

It should be noted that if the groundwater pressures within the Laminated Clay is responsive to long term rainfall values then higher water pressures are to be expected because between 1973 and 1980 the rainfall, as indicated in three nearby stations has been less than the average, thus indicating lower factors of safety in the future.

10.8 ACCEPTABILITY FOR DEVELOPMENT - FACTOR OF SAFETY

Section 5 of the report discusses the acceptability of certain factors of safety values for the hillside and gives arguments on why factors of safety lower than those traditionally quoted should be acceptable for this site.

In clause 5. 2.2.1 it is stated that "The Civil Engineering Code of Practice No.2, Earth Retaining Strutures may be considered to come closest to giving an indication of acceptable factors of safety in respect of slope stability". In this Code of Practice it is stated that a factor of safety should not be less

1.5 where the strength properties have been obtained from laboratory tests. This is the condition applicable to this site.

The Code states that the Factor of Safety should not be less than 1.25 where the strength properties of the soil have been obtained by analysis of a previous failure in the same strata at the same or a neighbouring site. This case is not applicable to this site.

We accept that factors of safety less than 1.5 are possible for some sites providing the various parameters used in the stability analysis (i.e. shear strength, groundwater pressures, etc.) are known to a high degree of accuracy. The investigations carried out at this site, however, indicate very variable groundwater pressures along the slip surface which are only known reasonably accurately at the piezometer positions. There is also an irregular variation in the thickness and depth of the various soil strata below the site.

It is stated that lower factors of safety may be used with residual shear strength values than when normal drained shear strength values are applicable. We would not accept this assumption because there is far more knowledge on the values and variation of the drained shear strength parameters for glacial clays than there is on the values of residual shear strengths.

It is stated that"from discussions with other leading consultants it is understood that a factor of safety of about 1.25 for residual landslip analysis is generally considered to be acceptable" and then continues by stating that "It would appear from the foregoing that a total factor of safety between 1.20 and 1.30 would be considered acceptable to a wide range of expert organisations and individuals when considering a relict landslip."

Can Allott and Lomax submit examples of hillside sites with similar variable strata and groundwater conditions where factors of safety of 1.2 - 1.3 have been considered acceptable, and where the site is to be used for housing development?

In clause 5.2.3.3. there is a discussion on the "Acceptable Levels of Risk" following which it is stated that "It is reasonable therefore, to expect that engineering structures should not have an annual risk failure of less than 10^{-4}".

It is interesting to note that a recent analysis of this kind has related the mechanistic value of factor of safety to risk (Meyerhoff 1970) and (Alonso 1976). See Fig.16 below

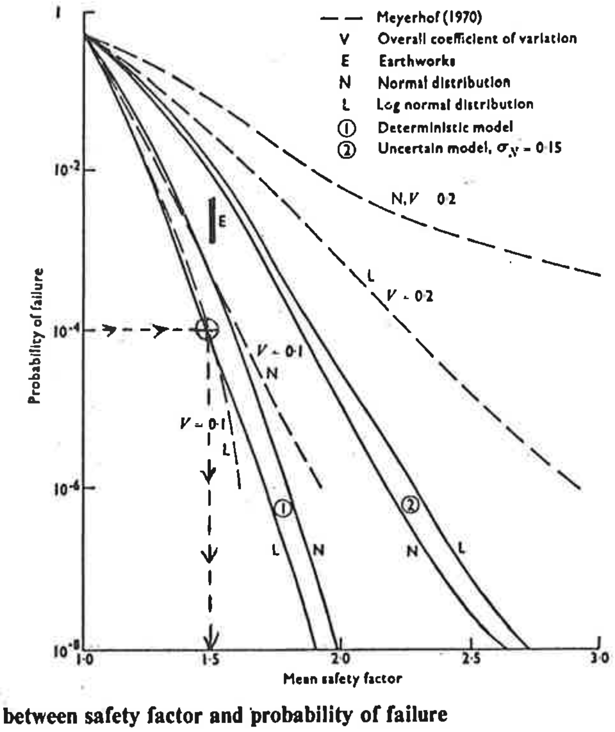

Fig. 16. Relationship between safety factor and probability of failure

From this it can be seen that for the level of risk quoted by Allott and Lomax (i.e. $< 10^{-4}$) the mean value of the factor of safety is 1.5.

11. COMMENTS AND CONCLUSIONS

11.1 Allott and Lomax have now modified their estimated factors of safety to the following lower values

1.09 - Section A
1.30 - Section B

If Section B is corrected so that it runs along the line of maximum slope in this area then the above factor of safety is reduced to about

1.28 - Section B

11.2 The following are the main parameters which can affect the estimated factor of safety of the hillside.

a) Site Topography and Assumed Soil Strata - There is a variation of about 4% in the Factor of Safety due to variations in ground level and assumed soil strata between Sections A and B.

b) Assumed Slip Circle - Insufficient slip circles have been analysed to comment on possible variations in the position of the assumed slip circle.

c) Theoretical Stability Analysis - Allott and Lomax state that the more rigorous method of analysis can predict failure to within an accuracy of 10%.

d) Residual Shear Strength - A difference of only one degree between the assumed and actual ϕ_r values could lower the factor of safety by about 10%.

e) Groundwater Pressures - There is a considerable variation and irregularity in the measured groundwater pressures along the slip surface. The factor of safety should, therefore, be sufficiently high to allow for the possible difference between the assumed and actual groundwater pressures along the slip circle.

11.3 Allott and Lomax have assumed that because the estimated factor of safety for Section B is higher than for Section A then it has a higher actual factor of safety and therefore safe for building. It has been demonstrated however, that the difference in the factors of safety between the two sections is predominantly due to the variation of only one factor, i.e. the groundwater pressure. The two sections may, therefore, have a similar actual factor of safety with the estimated difference being due to variations between the actual and assumed groundwater pressures along the slip surface. The mean value of the two estimated factors of safety is

1.185

a value which would be considered too low by Allott and Lomax according to their statement in clause 5.2.2.1.

11.4 In conclusion it can be seen from the above that the estimated factors of safety of 1.09 for Section A and 1.28 for Section B of the hillside slopes are now less, by about 5%, than those previously estimated by Allott and Lomax.

These factors of safety are relatively low, in particular when considered with respect to the variation of the groundwater pressures along the slip surface, and do not give one confidence in giving a reasonable guarantee on the long-term stability of the slope. We are of the opinion, therefore, based on the available information that the Local Authority should not accept responsibility for the long-term stability of the slope when considered for the support of housing development, although it is possible that the hillside may remain stable during the life of existing structures placed on the site.

Yours faithfully,
p.p. Sub Soil Surveys Ltd.,

[redacted]

Dr. J.K. Alderman
Director

Consultant in Soil Mechanics
and Foundation Engineering

APPENDIX 07

Geo-Ventures Ground Investigation

Geo-Ventures (UK) Limited
Geotechnical and Environmental Services

Site Linden Park Road, Haslingden				**Borehole Number** BH 1
Boring Method Cable Percussion	**Casing Diameter** 200mm cased to 12.00m 150mm cased to 30.00m	**Ground Level (mOD)**	**Client**	**Job Number** 13-684
	Location	**Dates** 09/04/2013-15/04/2013	**Engineer** Robert E Fry & Associates Limited	**Sheet** 1/3

Depth (m)	Sample / Tests	Casing Depth (m)	Water Depth (m)	Field Records	Level (mOD)	Depth (m) (Thickness)	Description
						(0.20)	Black TOPSOIL
0.20	D					0.20	MADE GROUND : firm / stiff yellow / brown sandy clay fill
0.70	D						
						(1.80)	
1.50-1.95	U			22 blows			
2.00 2.00-2.45	D U			49 blows		2.00	Stiff grey / brown CLAY with occasional fine sub-rounded gravel
2.50 2.50-2.95	D U			34 blows		(1.50)	
3.00 3.00-3.45	D U			27 blows			
3.50 3.50-3.95	D U			28 blows		3.50 (0.50)	Firm to firm / stiff grey / brown slightly sandy CLAY
4.00 4.00-4.45	D U			43 blows		4.00 (0.50)	Firm / stiff to stiff grey / brown CLAY
4.50 4.50-4.95	D U			42 blows		4.50 (0.50)	Firm to firm / stiff grey / brown CLAY with occasional fine sub-rounded gravel
5.00 5.00-5.45	D U			26 blows		5.00	Firm / stiff grey / brown CLAY with occasional fine sub-rounded gravel
5.50 5.50-5.95	D U			24 blows		(1.00)	
6.00 6.00-6.45	D U			22 blows		6.00	Firm / stiff to stiff grey / brown CLAY with occasional fine sub-rounded gravel
6.50 6.50-6.95	D U			26 blows		(1.00)	
7.00 7.00-7.45	D U			27 blows		7.00	Stiff grey / brown CLAY with occasional fine sub-rounded gravel
7.50 7.50-7.95	D U			25 blows		(1.00)	
8.00 8.00-8.45	D U			34 blows		8.00 (0.50)	Firm / stiff to stiff grey / brown CLAY with occasional fine sub-rounded gravel
8.50 8.50-8.95	D U			25 blows		8.50 (0.50)	Stiff grey / brown CLAY with occasional fine sub-rounded gravel
9.00 9.00-9.45	D U			34 blows		9.00	Firm / stiff to stiff grey / brown CLAY with occasional fine sub-rounded gravel
9.50 9.50-9.95	D U			29 blows		(1.00)	
						10.00	

Legend | Water | Instr

Remarks Services inspection pit excavated by hand to 1.20m	**Scale (approx)** 1:50	**Logged By** Drill Crew
	Figure No. 13-684.BH 1	

Geo-Ventures (UK) Limited
Geotechnical and Environmental Services

Site Linden Park Road, Haslingden				**Borehole Number** BH 1
Boring Method Cable Percussion	**Casing Diameter** 200mm cased to 12.00m 150mm cased to 30.00m	**Ground Level (mOD)**	**Client**	**Job Number** 13-684
	Location	**Dates** 09/04/2013-15/04/2013	**Engineer** Robert E Fry & Associates Limited	**Sheet** 2/3

Depth (m)	Sample / Tests	Casing Depth (m)	Water Depth (m)	Field Records	Level (mOD)	Depth (m) (Thickness)	Description	Legend	Water	Instr
10.00	D						Stiff grey / brown CLAY with occasional fine sub-rounded gravel			
10.00-10.45	U			36 blows						
10.50	D									
10.50-10.95	U			40 blows						
11.00	D									
11.00-11.45	U			40 blows						
11.50	D			09/04/2013:DRY						
				10/04/2013:11.30m						
11.50-11.95	U			24 blows						
12.00	D									
12.00-12.45	U			25 blows						
12.50	D					(5.00)				
12.50-12.95	U			35 blows						
13.00	D									
13.00-13.45	U			46 blows						
13.50	D									
13.50-13.95	U			40 blows						
14.00	D									
14.00-14.45	U			37 blows						
14.50	D									
14.50-14.95	U			59 blows						
15.00	D					15.00	Firm to firm / stiff grey / brown CL:AY with occasional fine sub-rounded gravel			
15.00-15.45	U			26 blows						
15.50	D									
15.50-15.95	U			60 blows						
16.00	D					(2.00)				
16.00-16.45	U			61 blows						
16.50	D			10/04/2013:DRY						
				11/04/2013:15.55m						
16.50-16.95	U			79 blows						
17.00	D					17.00	Stiff grey / brown CLAY with occasional fine sub-rounded gravel			
17.00-17.45	U			82 blows						
17.50	D									
17.50-17.95	U			62 blows						
18.00	D					(2.00)				
18.00-18.45	U			85 blows						
18.50	D									
18.50-18.95	U			87 blows						
19.00	D					19.00	Firm / stiff grey / brown CLAY with occasional fine sub-rounded gravel			
19.00-19.35	U			100 blows		(0.50)				
19.40	D					19.50	Stiff grey / brown CLAY with occasional fine sub-rounded gravel			
19.50-19.95	U			64 blows						

Remarks	**Scale (approx)** 1:50	**Logged By** Drill Crew
	Figure No. 13-684.BH 1	

Geo-Ventures (UK) Limited
Geotechnical and Environmental Services

Site	Linden Park Road, Haslingden	**Borehole Number**	**BH 1**
Boring Method	Cable Percussion	**Casing Diameter**	200mm cased to 12.00m 150mm cased to 30.00m
Ground Level (mOD)		**Client**	
Job Number	13-684	**Location**	
Dates	09/04/2013- 15/04/2013	**Engineer**	Robert E Fry & Associates Limited
Sheet	3/3		

Depth (m)	Sample / Tests	Casing Depth (m)	Water Depth (m)	Field Records	Level (mOD)	Depth (m) (Thickness)	Description	Legend	Water	Instr
20.00	D									
20.00-20.45	U			58 blows						
20.50	D									
20.50-20.95	U			50 blows						
21.00	D									
21.00-21.45	U			59 blows						
21.50	D									
21.50-21.95	U			67 blows						
22.00	D			11/04/2013:DRY						
				15/04/2013:21.34m						
22.00-22.45	U			71 blows						
22.50	D									
22.50-22.95	U			50 blows						
						(6.50)				
23.00	D									
23.00-23.45	U			47 blows						
23.50	D									
23.50-23.95	U			62 blows						
24.00	D									
24.00-24.45	U			60 blows						
24.50	D									
24.50-24.95	U			40 blows						
25.00	D									
25.00-25.45	U			55 blows						
25.50	D									
25.50-25.95	U			100 blows						
						26.00				
26.00	D						Firm / stiff grey / brown CLAY with occasional fine sub-rounded gravel			
26.00-26.45	U			49 blows		(0.50)				
						26.50				
26.50	D						Stiff grey / brown CLAY with occasional fine sub-rounded gravel			
26.50-26.95	U			57 blows						
27.00	D									
27.00-27.45	U			61 blows						
27.50	D									
27.50-27.95	U			65 blows						
28.00	D									
28.00-28.45	U			62 blows		(3.50)				
28.50	D									
28.50-28.95	U			69 blows						
29.00	D									
29.00-29.45	U			75 blows						
29.50	D									
29.50-29.95	U			78 blows						
				15/04/2013:DRY						
30.00	D					30.00				

Remarks	**Scale (approx)** 1:50	**Logged By** Drill Crew
	Figure No. 13-684.BH 1	

Geo-Ventures (UK) Limited
Geotechnical and Environmental Services

Site	Linden Park Road, Haslingden		**Borehole Number**	BH 2
Boring Method	Cable Percussion	**Casing Diameter** 200mm cased to 12.00m 150mm cased to 30.00m	**Ground Level (mOD)**	
Client			**Job Number**	13-684
Location		**Dates** 16/04/2013-19/04/2013	**Engineer** Robert E Fry & Associates Limited	**Sheet** 1/3

Depth (m)	Sample / Tests	Casing Depth (m)	Water Depth (m)	Field Records	Level (mOD)	Depth (m) (Thickness)	Description	Legend	Water	Instr
						(0.20) 0.20	Black TOPSOIL			
0.10	D						Firm / stiff grey / brown CLAY			
0.20	D					(1.80)				
1.50-1.95	UB NR			60 blows						
						2.00	Stiff grey / brown CLAY with occasional fine sub-rounded gravel			
2.00	D									
2.00-2.45	U			28 blows		(1.00)				
2.50	D									
2.50-2.95	UB NR			22 blows						
						3.00	Firm grey / brown CLAY			
3.00	D									
3.00-3.45	U			28 blows						
3.50	D									
3.50-3.95	U			32 blows						
4.00	D									
4.00-4.45	U			32 blows		(3.00)				
4.50	D									
4.50-4.95	UB NR			30 blows						
5.00	D									
5.00-5.45	U			28 blows						
5.50	D									
5.50-5.95	U			25 blows						
						6.00 (0.50)	Firm / stiff to stiff grey / brown CLAY with occasional fine sub-rounded gravel			
6.00	D									
6.00-6.45	U			26 blows						
						6.50 (0.50)	Firm to firm / stiff grey / brown CLAY with occasional fine sub-rounded gravel			
6.50	D									
6.50-6.95	U			37 blows						
						7.00	Firm / stiff to stiff grey / brown CLAY with occasional fine sub-rounded gravel			
7.00	D									
7.00-7.45	U			33 blows						
7.50	D					(1.50)				
7.50-7.95	U			35 blows						
8.00	D									
8.00-8.45	U			31 blows						
						8.50	Stiff grey / brown CLAY with occasional fine sub-rounded gravel			
8.50	D									
8.50-8.95	U			30 blows						
9.00	D									
9.00-9.45	U			29 blows						
9.50	D									
9.50-9.95	U			35 blows						

Remarks
Drillers descriptions of strata encountered at this stage
Services inspection pit excavated by hand to 1.20m

Scale (approx) 1:50
Logged By Drill Crew
Figure No. 13-684.BH 1

Geo-Ventures (UK) Limited
Geotechnical and Environmental Services

Site: Linden Park Road, Haslingden					**Borehole Number**: BH 2
Boring Method: Cable Percussion	**Casing Diameter**: 200mm cased to 12.00m, 150mm cased to 30.00m	**Ground Level (mOD)**:	**Client**:		**Job Number**: 13-684
	Location:	**Dates**: 16/04/2013-19/04/2013	**Engineer**: Robert E Fry & Associates Limited		**Sheet**: 2/3

Depth (m)	Sample / Tests	Casing Depth (m)	Water Depth (m)	Field Records	Level (mOD)	Depth (m) (Thickness)	Description
10.00	D						
10.00-10.45	U			27 blows			
10.50	D						
10.50-10.95	U			25 blows			
						(3.50)	
11.00	D						
11.00-11.45	U			40 blows			
11.50	D						
11.50-11.95	U			39 blows			
						12.00	Fiirm / stiff to stiff grey / brown CLAY with occasional fine sub-rounded gravel
12.00	D						
12.00-12.45	U			28 blows		(0.50)	
						12.50	Stiff grey / brown CLAY with occasional fine sub-rounded gravel
12.50	D						
12.50-12.95	U			30 blows			
13.00	D						
13.00-13.45	U			32 blows			
						(2.00)	
13.50	D						
13.50-13.95	U			31 blows			
14.00	D						
14.00-14.45	U			29 blows			
						14.50	Firm / stiff to stiff grey / brown CLAY with occasional fine sub-rounded gravel
14.50	D						
14.50-14.95	U			30 blows		(0.50)	
						15.00	Stiff grey / brown CLAY with occasional fine sub-rounded gravel
15.00	D						
15.00-15.45	U			33 blows			
15.50	D						
15.50-15.95	U			35 blows			
16.00	D						
16.00-16.45	U			43 blows			
16.50	D						
16.50-16.95	U			64 blows		(3.50)	
17.00	D						
17.00-17.45	U			74 blows			
17.50	D						
17.50-17.95	U			73 blows			
18.00	D						
18.00-18.45	U			42 blows			
						18.50	Grey / brown SILT
18.50	D						
18.50-18.95	U			54 blows		(0.50)	
						19.00	Firm / stiff grey / brown CLAY with occasional fine sub-rounded gravel
19.00	D						
19.00-19.35	UB NR			59 blows		(0.50)	
19.40	D					19.50	Stiff grey / brown CLAY with occasional fine sub-rounded gravel
19.50-19.95	U			62 blows 10/04/2013:DRY			
				11/04/2013:DRY			

Remarks
Chiselling from 20.00m to 20.80m for 1.50 hours.

Scale (approx)	Logged By
1:50	Drill Crew

Figure No. 13-684.BH 1

Geo-Ventures (UK) Limited
Geotechnical and Environmental Services

Site	Borehole Number
Linden Park Road, Haslingden	BH 2

Boring Method	Casing Diameter	Ground Level (mOD)	Client	Job Number
Cable Percussion	200mm cased to 12.00m 150mm cased to 30.00m			13-684
	Location	**Dates** 16/04/2013-19/04/2013	**Engineer** Robert E Fry & Associates Limited	**Sheet** 3/3

Depth (m)	Sample / Tests	Casing Depth (m)	Water Depth (m)	Field Records	Level (mOD)	Depth (m) (Thickness)	Description	Legend	Water	Instr
20.00	D									
20.00-20.80	B									
21.00-21.45	U			60 blows						
21.50	D									
21.50-21.95	U			40 blows						
						(4.50)				
22.00	D									
22.00-22.45	U			41 blows						
22.50	D									
22.50-22.95	U			56 blows						
23.00	D									
23.00-23.45	U			38 blows						
23.50	D									
23.50-23.95	U			59 blows						
						24.00	Firm / stiff to stiff grey / brown CLAY with occasional fine sub-rounded gravel			
24.00	D									
24.00-24.45	U			61 blows						
						(1.00)				
24.50	D									
24.50-24.80	U			100 blows						
						25.00	Stiff grey / brown CLAY with occasional fine sub-rounded gravel			
25.00	D									
25.00-25.45	U			42 blows						
25.50	D									
25.50-25.95	U			34 blows						
26.00	D									
26.00-26.90	B									
						(2.50)				
27.00-27.45	U			42 blows						
						27.50	Firm / stiff to stiff grey / brown CLAY with opccasional fine sub-rounded gravel			
27.50	D			11/04/2013:DRY						
				15/04/2013:DRY		(0.50)				
27.50-27.95	U			71 blows						
28.00	D					28.00	Stiff grey / brown CLAY with occasional fine sub-rounded gravel			
28.00-28.45	U			66 blows						
28.50	D									
28.50-28.95	U			62 blows						
						(2.00)				
29.00	D									
29.00-29.45	U			56 blows						
29.50	D			15/04/2013:DRY						
29.50-29.95	U			69 blows						
30.00	D			16/04/2013:0.00m		30.00				

Remarks
Chiselling from 20.00m to 20.80m for 1.50 hours. Chiselling from 26.00m to 26.90m for 1.00 hour.

Scale (approx)	Logged By
1:50	Drill Crew

Figure No. 13-684.BH 1

Geo-Ventures (UK) Limited
Geotechnical and Environmental Services

Site: Linden Park Road, Haslingden

Borehole Number: BH 3

Boring Method: Cable Percussion

Casing Diameter: 200mm cased to 12.00m; 150mm cased to 30.00m

Ground Level (mOD):

Client:

Job Number: 13-684

Location:

Dates: 23/04/2013-25/04/2013

Engineer: Robert E Fry & Associates Limited

Sheet: 1/3

Depth (m)	Sample / Tests	Casing Depth (m)	Water Depth (m)	Field Records	Level (mOD)	Depth (m) (Thickness)	Description
						(0.40)	MADE GROUND : brown / black soil and broken concrete fill
0.20	D					0.40	
							Stiff grey slightly sandy CLAY
1.50-1.95	U			17 blows			
2.00-2.45	SPT N=14			2,3/4,3,3,4			
2.00	D						
2.00-2.45	D						
2.50-2.95	U			37 blows			
3.00-3.45	SPT N=11			2,2/3,2,3,3			
3.00	D						
3.00-3.45	D						
3.50-3.95	U			27 blows			
4.00-4.45	SPT N=11			2,2/3,3,2,3			
4.00	D					(7.60)	
4.00-4.45	D						
4.50-4.95	U			21 blows			
5.00-5.45	SPT N=14			1,2/2,3,3,6			
5.00	D						
5.00-5.45	D						
5.50-5.95	U			41 blows			
6.00-6.45	SPT N=12			1,2/2,3,3,4			
6.00	D						
6.00-6.45	D						
6.00-7.20	B						
7.50-7.95	U			21 blows			
8.00	D					8.00	Firm / stiff to stiff grey / brown CLAY with occasional fine sub-rounded gravel
8.50-8.95	SPT N=11			2,2/3,2,3,3			
8.50-8.95	D						
9.50-9.95	U			30 blows			

Remarks
Services inspection pit excavated by hand to 1.20m
Chiselling from 6.00m to 7.20m for 1.00 hour.

Scale (approx): 1:50

Logged By: J. Crook

Figure No.: 13-684.BH 1

Geo-Ventures (UK) Limited
Geotechnical and Environmental Services

Site	Borehole Number
Linden Park Road, Haslingden	BH 3

Boring Method	Casing Diameter	Ground Level (mOD)	Client	Job Number
Cable Percussion	200mm cased to 12.00m 150mm cased to 30.00m			13-684

Location	Dates	Engineer	Sheet
	23/04/2013- 25/04/2013	Robert E Fry & Associates Limited	2/3

Depth (m)	Sample / Tests	Casing Depth (m)	Water Depth (m)	Field Records	Level (mOD)	Depth (m) (Thickness)	Description	Legend	Water	Instr
10.00	D									
10.50-10.95 10.50-10.95	SPT N=13 D			1,2/2,3,4,4						
11.50-11.95	U			39 blows						
						(6.00)				
12.00	D									
12.50-12.95 12.50-12.95	SPT N=18 D			2,2/4,4,5,5						
13.50-13.95	U			66 blows						
						14.00	Stiff grey / brown slightly sandy CLAY			
14.00	D									
14.50-14.95 14.50-14.95	SPT N=25 D			3,4/4,6,6,9						
15.50-15.95	U			48 blows						
16.00	D									
16.50-16.95 16.50-16.95	SPT N=22 D			3,4/4,6,6,6						
						(6.00)				
17.20-17.70	B									
18.50-18.95 18.50-18.95	SPT N=27 D			2,5/6,6,7,8						
19.50-19.95	U			42 blows 10/04/2013:DRY						
				11/04/2013:DRY		20.00				

Remarks
Chiselling from 17.00m to 17.50m for 0.50 hours.

Scale (approx)	Logged By
1:50	J. Crook

Figure No.
13-684.BH 1

Geo-Ventures (UK) Limited
Geotechnical and Environmental Services

Site	Borehole Number
Linden Park Road, Haslingden	BH 3

Boring Method	Casing Diameter	Ground Level (mOD)	Client	Job Number
Cable Percussion	200mm cased to 12.00m 150mm cased to 30.00m			13-684

Location	Dates	Engineer	Sheet
	23/04/2013- 25/04/2013	Robert E Fry & Associates Limited	3/3

Depth (m)	Sample / Tests	Casing Depth (m)	Water Depth (m)	Field Records	Level (mOD)	Depth (m) (Thickness)	Description	Legend	Water	Instr
20.00	D						Firm / stiff to stiff grey / brown CLAY			
20.50-20.95 20.50-20.95	SPT N=24 D			3,3/5,5,6,8						
						(2.00)				
21.50-21.95	U			47 blows						
						22.00	Stiff grey slightly sandy CLAY with occasional fine sub-rounded gravel			
22.00	D									
22.50-22.95 22.50-22.95	SPT N=29 D			4,5/6,6,8,9						
23.50-23.95	U			64 blows						
24.00	D									
						(4.50)				
24.50-24.95 24.50-24.80	SPT N=40 D			4,6/8,9,10,13						
25.50-25.95	U			50 blows						
26.00	D									
						26.50	Firm / stiff to stiff grey / brown slightly sandy CLAY			
26.50-26.90 26.50-26.95	SPT 50/250 D			5,7/11,12,15,12						
						(1.50)				
				11/04/2013:DRY						
27.50-27.95 28.00	U D			15/04/2013:DRY 62 blows						
						28.00	Stiff grey / brown slightly sandy CLAY with occasional fine sub-rounded gravel			
28.50-28.95 28.50-28.95	SPT N=50 D			5,7/9,12,13,16						
						(2.00)				
29.50-29.95	U			62 blows 15/04/2013:DRY						
30.00	D			16/04/2013:0.00m		30.00				

Remarks	Scale (approx)	Logged By
	1:50	J. Crook
	Figure No. 13-684.BH 1	