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UNITED KINGDOM NIREX LIMITED

Rock Characterisation Facility

Longlands Farm, Gosforth, Cumbria

SUPPLEMENTARY PROOF OF EVIDENCE

OF

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ROLE OF THE RCF

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1. SUMMARY

1.1 In this Supplementary Proof of Evidence I provide responses to a number of points raised:

i. Methods of Construction

ii. Headgear Height

iii. Alternative Schemes for the RCF

iv. Reuse of RCF Shafts in a Repository

v. Points Raised by the RWMAC

vi. One Shaft Rather Than Two

1.2 I deal with these matters because:

i. Methods of Construction

- Dr Green and Dr Western assert that the impact of excavation damage on fluid flow is not generally understood. Until it is, information from a sub-surface facility will be of little or no value to the establishment of a PCPA (PE/FOE/1, paragraph 9.8).
- Dr Salmon (PE/FOE/5, paragraph 4.11) and Dr Hencher (PE/FOE/6, paragraph 6.16) provide misleading views of the extent of the hydraulic conductivity enhancement of excavation disturbance.
- Dr Allison argues that Nirex evidence does not describe the proposed method of rock support (PE/FOE/7, paragraph 4.66).
- Dr Allison asserts that the Nirex proposals rely only on drill and blast and that there are no identified plans to examine alternative methods of construction which could result in less damage to the rock mass (PE/FOE/7, paragraphs 4.37 and 4.55 to 4.60).
- The extent of blast-induced fracturing of the rock mass during construction of the shafts by the drill and blast method was raised by Mr Walker on Day 2 of the Inquiry.
- The extent to which tolerances are required in the operation of drill and blast excavation of the shafts was raised by the Assistant Inspector on Day 2 of the Inquiry.
- The nature of relaxation of rocks around underground shafts and galleries was raised by the Inspector in questions put to Dr Chambers on day 6 of the Inquiry.
- The methods for sealing the shafts against groundwater inflow were examined by the Inspector on Day 6 of the Inquiry.

ii. Headgear Height

- Mr Spendlove indicates that the RCF headgear height exceeds that of the 1991 Sellafield repository concept. Nirex has not designed the RCF to minimise any adverse effect on the surrounding area (PE/SPD/1, paragraph 5.4).

iii. Alternative Schemes for the RCF

- Mr Spendlove makes a case for relocating the proposed South Shaft site to the south west, thereby reducing the visual impact of the headgear by siting it on lower ground and further away from the Lake District National Park (PE/SPD/1, paragraph 5.5).

iv. Reuse of RCF Shafts in a Repository

- Dr Allison maintains that Nirex proposals do not make clear the future use of the RCF (PE/FOE/7, paragraph 4.36).
- On Day 26 Mr Brodie put to Mr Folger that, to cut costs and save unnecessary time delays, the RCF will need to be constructed so as to allow it to be adapted to form part of the deep repository.

v. Points Raised by the RWMAC

- In his Supplementary Proof of Evidence, Mr Folger makes reference (**PE/NRX/12/S1**, paragraph 7.5) to *the September 1995 RWMAC Report* [GOV/414], and explains that a number of issues raised by the RWMAC have been addressed in the period since March 1994. I provide details of how these issues have been addressed since that time.

vi. One Shaft Rather Than Two

- The operational limitations, in terms of safety, of constructing just one shaft were examined by Mr Weston and by the Inspector on Day 2 of the Inquiry.
- In his evidence Mr Spendlove makes about a case for sinking only one shaft (PE/SPD/1, Section 6).

1.3 I now address each matter in turn.

i. Methods of Construction (Section 3)

Disturbance Caused by Construction

1.4 I make two main points concerning the disturbance of the rock mass that would be created by excavating the RCF:

i. that there is a general understanding of excavation disturbance. I show a general understanding in Appendix 1, and redress the misleading views put forward by Dr Salmon and Dr Hencher; and

ii. that Nirex has a pragmatic and conservative assumption regarding the magnitude and extent of excavation disturbance. The RCF is required to test this assumption. It is not necessary to await the outcome of an international research programme which Dr Green and Dr Western seem to require.

Construction by Drill and Blast

1.5 I make five main points concerning the drill and blast method:

i. drill and blast is a tried and tested method for shaft sinking to the depths required for the RCF. Dr Allison accepts that drill and blast may be an appropriate method to construct at least one of the shafts;

ii. Nirex has used an international blast consultant to develop blast designs for the RCF shafts which will keep blast damage to a practicable minimum;

iii. Nirex's blast consultant has calculated the extent of blast damage likely to be encountered in sinking the RCF shafts;

iv. contrary to Dr Allison's assertions I show that, because the blast designs take account of the strength and variations of the rocks through which the shafts will be sunk, the blast damage can be expected to be minimal and sensibly constant; and

v. I provide estimates of the extent of measured blast damage from the Swedish and Canadian underground facilities, and show these to be similar to the extents of blast damage expected for the rocks at Sellafield, contrary to Dr Allison's assertion.

Alternative Methods to Drill and Blast

1.6 I make four main points concerning alternative methods of construction. These show Dr Allison's suggestion to construct one of the shafts by a machine boring method to be impractical:

i. as a partner in the ZEDEX project, Nirex has direct experience of measuring and comparing the excavation disturbance associated with construction by drill and blast and with machine boring methods. Dr Allison fails to recognise this point;

ii. preliminary and so far unpublished results show that there is little or no significant difference in the extent of damage caused by machine boring methods and that caused by the drill and blast method. I accept that Dr Allison is unlikely to be aware of this;

iii. data from the only manufacturer of proven boring machines suitable for shaft sinking, and consultation with the largest shaft sinking contractor to use shaft boring, confirms that the depths

of the RCF shafts are at the upper limit of practical experience in sinking by the boring method. Dr Allison apparently fails to recognise the practical significance of this point;

iv. in addition to working at the upper limit of current experience, Dr Allison's proposed alternative method of construction would result in practical difficulties in maintaining verticality of the shaft. No shaft has previously been sunk to the depth required for the RCF using the method proposed by Dr Allison whilst maintaining verticality; and

v. the proposed alternative method proposed by Dr Allison is potentially unsafe. Connecting a pilot hole via an adit through water bearing sandstones to the bottom of a shaft in which persons are required to work presents a safety hazard which cannot exist in the method proposed by Nirex.

Rock Support in the RCF

1.7 I make two main points concerning rock support in the RCF:

i. Dr Allison states that I say that the cover rocks through which the RCF shafts pass will be lined with a hydrostatic concrete lining in order to provide structural support for the openings. This is wrong. I do not say this anywhere in my evidence. The purpose of the lining is to reduce inflows of water into the completed shaft; and

ii. Dr Allison states that Nirex's proposals contain no evidence concerning rock support for the RCF. To address this point I provide a brief account of the development of support designs and an outline of the permanent support requirements for the RCF shafts and galleries.

Groundwater Control

1.8 In response to questions raised by the Inspector on Day 6 I provide further details of the method for controlling groundwater inflows during and after the sinking of the RCF shafts.

ii. Headgear Height (Section 4)

1.9 In Section 4 I make three main points:

i. the purpose of the RCF shafts is scientific. The shafts will provide access for observations and measurements during shaft sinking. They will also provide access to the galleries for observations and measurements during Phases 2 and 3. The height of the temporary headgear used for construction will be in the range 25 to 30 metres is required for construction. In order to provide safe decking at ground level and safe overwind protection for persons riding the shaft and in order to provide an underslinging allowance for equipment, a headgear height of 29.2 metres is required, once construction of the shafts has been completed;

ii. the purposes of the Preferred Conceptual Design and the Alternative Option for the December 1991 repository concept were not scientific. The height of the temporary headgear used for construction would have been in the range 25 to 30 metres. In other words, it would have been the same as that now proposed for the RCF. In the Preferred Conceptual Design a permanent headgear height of 15 metres was achievable because the shafts were not intended for manriding and materials conveyancing. In the Alternative Option, the larger decking heights, underslinging allowance and overwind protection required would have been achieved by below ground decking. Nirex subsequently abandoned the Alternative Option in favour of the Preferred Conceptual Design; and

iii. For these reasons Mr Spendlove is wrong to claim that:
"It was therefore clearly the intention at that time by Nirex to limit the height of protruding structures to 15 metres above ground level in order to meet public concerns. The RCF proposals for construction headgears of 25 to 30 metres and permanent headgears of 29.2 metres above ground level clearly do not meet those concerns and therefore it is not correct to say that 'the development has been located and designed to minimise any adverse physical effects on the surrounding area' (Ref. ***PE/NRX/2****, paragraph 2.33)."*

iii. Alternative Schemes for the RCF (Section 5)

1.10 In Section 5 I deal with two points made by Mr Spendlove. These are:

i. Mr Spendlove proposes an option to re engineer the surface works by excavating into the south west side of the existing drilling platform (PE/SPD/1, paragraph 5.5, (i)). Unfortunately Mr Spendlove does not make plain in his evidence to what extent he considers it might be appropriate to excavate into the existing drilling platform. I am therefore, as yet, unable to help the Inquiry by responding in detail to this point. Nirex has written to Mr Spendlove asking for better details of this, and his other point, which I now move onto;

ii. Mr Spendlove's other point is to move the shaft locations a short distance towards the south west. To move the locations of the shafts as Mr Spendlove proposes would be substantially disadvantageous to the scientific purpose of the RCF for four reasons:

- Firstly, Mr Spendlove's proposal would put the shafts outside the volume of rock which Nirex has characterised in detail and reported in *Nirex Report S/95/007* [COR/524]. The significance of this is that a shaft position outside this volume, as Mr Spendlove proposes, would make the study of continuity between fractures mapped in the shafts and flow zones observed in the boreholes significantly more difficult.
- Secondly, Mr Spendlove's proposal would put the shafts outside the centre of the groundwater pressure monitoring system which is described in Dr Chaplow's evidence. This would significantly impair the effectiveness of the monitoring system to detect the drawdown caused by shaft sinking. This is an important point because the single most important scientific aspect of Phase 1 will be the information which can be gathered regarding the hydrogeological performance of the rockmass, as noted by the RWMAC (paragraph 16) [COR/414]. To redress this, one or more additional monitoring boreholes would need to be drilled and instrumented.
- Thirdly, Mr Spendlove's proposal would put the collar and foreshaft construction works for the South Shaft into a water saturated, drift-filled hollow which is part of a buried valley feature cut into rockhead. This would entail construction through up to 22 metres of water bearing, unconsolidated ground which would require redesign work to be undertaken and would render the collar and foreshaft more difficult and expensive to construct. The associated environmental impact of this is not addressed by Mr Spendlove.
- finally, Nirex has drilled borehole RCF as a centreline borehole for the South Shaft and has planning permission to drill borehole RCF4 as a centre line borehole for the North Shaft. Mr Spendlove's relocation of the shafts would therefore either require the South Shaft to be sunk without the benefit of a centreline borehole to aid design and construction, or else require a new and additional borehole to be drilled.

iv. Reuse of RCF Shafts in a Repository (Section 6)

1.11 On Day 26 of the Inquiry Mr Brodie put to Mr Folger that, in order to save unnecessary time delays, the RCF will need to be constructed so as to allow it to be adapted to form part of the deep repository.

1.12 As I explain in Section 4 of my Supplementary Proof of Evidence, the purpose of the RCF is scientific. In designing the shafts to achieve scientific goals, there are five design aspects which have been explicitly considered in developing the design of the RCF shafts in respect of possible future incorporation into a repository. These are:

- Firstly, the location of the shafts within the PRZ. This is a matter which explained in Dr Hooper's Proof of Evidence (**PE/NRX/15**, paragraphs 7.3 to 7.6).
- Secondly, the diameter of the shafts have been sized to meet the needs of the scientists working in the shaft and to provide efficient methods of spoil clearance. The RCF shafts have not been sized to achieve any particular purpose or function in a future repository.

- Thirdly, no special precautions are being taken to provide 'as built' records of the shaft. It is expected that the scientific works carried out during, and as in integral part of, construction will be adequate for any future change of use of the RCF shaft which might be proposed.
- Fourthly, the design life of the shafts is based on standard civil and mining engineering principles. No special design features have been adopted for the RCF shafts.
- Finally, as far as sealing of the shafts is concerned, Nirex has taken the precaution of ensuring that no grouts or sealing materials are deployed in construction of the shafts without Nirex's prior approval.

v. Points Raised by the RWMAC (Section 7)

1.13 The September 1995 RWMAC Report entitled *"Review of aspects of the scientific mission and role of the Nirex Rock Characterisation Facility at Sellafield"* suggests a number of technical matters to which Nirex should give particular attention in its planning and execution of the project (paragraph 32) [GOV/414]. In Section 7 of my Supplementary Proof of Evidence I report that all of the matters raised by the RWMAC in their September 1995 Report have been addressed.

vi. One Shaft Rather Than Two (Section 8)

1.14 In his evidence Mr Spendlove makes out a case for initially sinking only one shaft for the RCF (PE/SPD/1, Section 6). There are four reasons why Mr Spendlove's case is impractical:

- Firstly, he does not demonstrate what the benefits would be of having one intrusion (shaft) into the PRZ rather than two.
- Secondly, it would not be possible to proceed to Phases 2 and 3 with only one shaft as this would not be safe.
- Thirdly, there are scientific benefits to be gained from initially sinking a second shaft.
- Finally, Mr Spendlove's case appears to rest on the point that all of Phase 1 together with a small part of Phase 2 would enable a definite point of decision to be reached as to whether to abandon the project or continue (PE/SPD/1, paragraph 6.7). He provides no evidence to support this view. In contrast, Nirex has said that such a definite point of decision cannot be planned with precision (as stated, for example, in **PE/NRX/12**, paragraph 6.27; COR/101, paragraphs 1.39 and 1.40).

2. INTRODUCTION

2.1 In this Supplementary Proof of Evidence I provide responses to a number of points raised:

i. Methods of Construction
ii. Headgear Height
iii. Alternative Schemes for the RCF
iv. Reuse of RCF Shafts in a Repository
v. Points Raised by the RWMAC
vi. One Shaft Rather Than Two

2.2 I deal with these matters because:

i. Methods of Construction

- Dr Green and Dr Western assert that the impact of excavation damage on fluid flow is not generally understood. Until it is, information from a sub-surface facility will be of little or no value to the establishment of a PCPA (PE/FOE/1, paragraph 9.8).
- Dr Salmon (PE/FOE/5, paragraph 4.11) and Dr Hencher (PE/FOE/6, paragraph 6.16) provide misleading views of the extent of the hydraulic conductivity enhancement of excavation disturbance.
- Dr Allison argues that Nirex evidence does not describe the proposed method of rock support (PE/FOE/7, paragraph 4.66).

- Dr Allison asserts that the Nirex proposals rely only on drill and blast and that there are no identified plans to examine alternative methods of construction which could result in less damage to the rock mass (PE/FOE/7, paragraphs 4.37 and 4.55 to 4.60).
- The extent of blast-induced fracturing of the rock mass during construction of the shafts by the drill and blast method was raised by Mr Walker on Day 2 of the Inquiry.
- The extent to which tolerances are required in the operation of drill and blast excavation of the shafts was raised by the Assistant Inspector on Day 2 of the Inquiry.
- The nature of relaxation of rocks around underground shafts and galleries was raised by the Inspector in questions put to Dr Chambers on day 6 of the Inquiry.
- The methods for sealing the shafts against groundwater inflow were examined by the Inspector on Day 6 of the Inquiry.

ii. Headgear Height

- RCF headgear height exceeds that of the 1991 Sellafield repository concept. Nirex has not designed the RCF to minimise any adverse effect on the surrounding area (PE/SPD/1, paragraph 5.4).

iii. Alternative Schemes for the RCF

- Mr Spendlove makes a case for relocating the proposed South Shaft site to the south west, thereby reducing the visual impact of the headgear by siting it on lower ground and further away from the Lake District National Park (PE/SPD/1, paragraph 5.5).

iv. Reuse of RCF Shafts in a Repository

- Dr Allison maintains that Nirex proposals do not make clear the future use of the RCF (PE/FOE/7, paragraph 4.36).
- On Day 26 Mr Brodie put to Mr Folger that, to cut costs and save unnecessary time delays, the RCF will need to be constructed so as to allow it to be adapted to form part of the deep repository.

v. Points Raised by the RWMAC

- In his Supplementary Proof of Evidence, Mr Folger makes reference (**PE/NRX/12/S1**, paragraph 7.5) to *the September 1995 RWMAC Report* [GOV/414], and explains that a number of issues raised by the RWMAC have been addressed in the period since March 1994. I provide details of how these issues have been addressed since that time.

vi. One Shaft Rather Than Two

- The operational limitations, in terms of safety, of constructing just one shaft were examined by Mr Weston and by the Inspector on Day 2 of the Inquiry.
- In his evidence Mr Spendlove makes about a case for sinking only one shaft (PE/SPD/1, Section 6).

3. METHODS OF CONSTRUCTION

Disturbance Caused by Construction

3.1 In their evidence (PE/FOE/1, paragraph 9.8), Dr Green and Dr Western say:

> "*In addition to the creation of a void within the Potential Repository Zone (PRZ), excavation of an RCF would also damage the surrounding host rock. The impact of excavation damage on fluid flow is not generally understood, and is the subject of an international scientific research programme. [COR/605] Until this issue has been resolved, the information from a sub-surface facility will be of little or no value to the establishment of a PCPA.*"

3.2 I disagree. Before setting out my reasons it is helpful to provide context. Creating a void in rock at depth, including boreholes, removes material which was previously bearing some of the load of the adjacent and

overlying rocks. Removal of this material therefore causes a redistribution of stresses around the excavation. This may be accompanied by some physical movement of the rocks around the excavation, including movements on existing fractures and the creation of new fractures. This phenomenon is termed 'excavation disturbance', and is recognised by Nirex (**PE/NRX/15**, paragraph 6.46) and taken into account by Nirex (**PE/NRX/15**, paragraph 6.47). I disagree with Dr Green and Dr Western because:

i. There is a general understanding of excavation disturbance and its impact on fluid flow as I show in Appendix 1 of this Supplementary Proof of Evidence.

ii. As explained in Dr Hooper's evidence (**PE/NRX/15**, paragraph 6.47) the pragmatic and conservative assumption which is made by Nirex is that the hydraulic conductivity within the zone of excavation disturbance may increase by a factor of up to a hundred over a distance equivalent to twice the diameter of the excavation. It is not necessary to await an outcome of an international scientific research programme if measurements carried during shaft sinking can test, and subsequently confirm, this pragmatic and conservative assumption. The methods for measuring the hydraulic conductivity and the extent of the excavation disturbed zone are set out in my evidence (**PE/NRX/16**, paragraphs 4.21 to 4.24 and paragraphs 5.24 to 5.26). These methods are based on an international research programme, as stated in my evidence (**PE/NRX/16**, paragraph 4.21, fourth and fifth bullet points).

Construction by Drill and Blast

3.3 The extent of blast-induced fracturing of the rock mass during construction of the shafts by the drill and blast method was raised by Mr Walker in cross examination of Mr Smith on Day 2. The extent to which tolerances are required in the operation of drill and blast excavation of the shafts was raised by the Assistant Inspector in questions put to Mr Smith, also on Day 2.

3.4 In paragraphs 3.7 to 3.13 of this Supplementary Proof of Evidence I explain that the extent of blast induced fracturing of the rock mass is small, provided care is taken in the proper design and execution of the blast, and provided that due care is paid to tolerances. Details of the blast patterns which have been developed for the RCF shafts and from which estimates of the extent of blast-induced damage to the rock mass have been derived are given in Appendix 2 of this Supplementary Proof of Evidence. Within that Appendix, paragraphs A2.12 and A2.22 deal with tolerances.

3.5 In his evidence (PE/FOE/7), Dr Allison argues that one of the purposes of the RCF should be to test the effects of different methods of excavation (PE/FOE/7, paragraph 4.55). He lends weight to his argument by saying that the cover rocks at Sellafield are "*relatively weak*" (PE/FOE/7, paragraph 4.55). He goes on to say that, whilst blast damage in the hard, relatively massive and uniform crystalline rocks of the Swedish and Canadian underground research laboratories would be expected to be minimal and sensibly constant, this cannot be said for the rock sequence at Sellafield (PE/FOE/7, paragraph 4.57). He further goes on to say that it would be possible to construct one of the RCF shafts using a shaft boring machine method (PE/FOE/7, paragraphs 4.59 and 4.60).

3.6 I disagree with Dr Allison for four reasons:

i. it is misleading to describe the cover rocks of Sellafield as "*relatively weak*", as I show in paragraphs 3.14 to 3.20 below;

ii. it is misleading to say that blast damage in the rocks at Sellafield could not be expected to be minimal and sensibly constant, as I also show in paragraphs 3.21 and 3.22 below;

iii. Nirex has carried out research in conjunction with SKB and ANDRA at Äspö to compare the measured extents of damage to the peripheral rock mass caused by the drill and blast method and the Tunnel Boring Machine method. Results of this work show that there is little or no significant difference in the extent of the damage caused by using these two methods, as I show in paragraphs 3.23 and 3.24 below; and

iv. Dr Allison's proposed alternative shaft construction method is disadvantageous, as I show in paragraphs 3.25 to 3.27 below.

3.7 The drill and blast method has been tried and tested in the construction of many hundreds of shafts in the United Kingdom and many thousands of shafts around the world. It is an industry standard method for constructing shafts to provide access to the depths required for the RCF.

3.8 Blasting will cause some limited damage to the rockmass during construction of the shafts. The amount of damage can be controlled and kept to a practical minimum by using a technique called cautious blasting (also known as controlled blasting, or smooth blasting). This technique takes into account the strength of the rocks in which the excavation is to be created. Typically, the extent of damage to the rockmass caused by blasting can be kept to several tens of centimetres from the excavation wall. The cautious blasting technique has been used in the construction of the Canadian Underground Research Laboratory (URL).

3.9 Nirex will construct the RCF shafts using cautious blasting methods, and has used a consultant with experience of cautious blasting on the URL Project to:

i. develop blast designs specific to the Sellafield sequence of rocks with the objective of excavating the shafts with the least practicable amount of blast damage to the rock mass, taking into account their strength and variations in strength; and

ii. calculate estimates of the extent of blast damage to the rock mass likely to be encountered in using these blast designs in the Sellafield sequence of rocks.

3.10 A series of blast designs for the RCF shafts has been developed by Nirex's blast design consultant. These blast designs have been prepared based on site-specific Sellafield rock strength data and using the design principles set out in Appendix 2 of this Supplementary Proof of Evidence.

3.11 For the sandstones through which the RCF shafts will be sunk, the preliminary calculated average value of the extent of blast damage is estimated to be 38 centimetres into the rock from the shaft wall.

3.12 For the Brockram, the preliminary calculated average value of the extent of blast damage is estimated to be 36 centimetres into the rock from the shaft wall.

3.13 For the BVG, the preliminary calculated average value of the extent of blast damage is estimated to be 50 centimetres into the rock from the shaft wall.

Rock Strength in Relation to Drill and Blast

3.14 Dr Allison says:

> "*the cover rocks at Sellafield are relatively weak*" (PE/FOE/7, paragraph 4.55).

3.15 This is somewhat misleading. The term "*relatively weak*" is not a standard term. Standard terms for the strength of rocks are recommended for use by the *Geological Society Engineering Group Working Party* (NRX/16/7). These recommendations take the form of a standardised set of descriptive terms and associated quantitative ranges of compressive strengths. This rock strength scale is reproduced in Table 3.1.

3.16 In *Nirex Report 801*, (FOE/5/15), Table 4.2 provides details of strength and deformability test results for the BVG and St Bees Sandstone. Data for the St Bees Sandstone are provided for measurements of uniaxial compressive strengths performed on 28 samples from Borehole 2 ('BH2') and 14 samples from Borehole 7 ('BH7').

3.17 The data from Table 4.2 of FOE/5/15 shows the mean uniaxial compressive strength of St Bees Sandstone from the BH2 samples to be 65 MPa. According to the rock strength scale (Table 3.1), this mean value corresponds with the term 'strong'.

3.18 The data from Table 4.2 of FOE/5/15 shows the mean uniaxial compressive strength of St Bees Sandstone from the BH7 samples to be 67 MPa. This also corresponds to the term 'strong'.

3.19 The range in measured uniaxial strengths for St Bees Sandstone for the BH2 samples is reported (in FOE/5/15, Table 4.2) as 24 to 124 MPa. This range corresponds to 'moderately strong' through 'strong' to 'very strong' according to the rock strength scale.

3.20 The range in measured uniaxial strengths for St Bees Sandstone for the BH7 samples is reported (FOE/5/15, Table 4.2) as 33 - 90 MPa. This range corresponds to 'moderately strong' to 'strong'. It is therefore more accurate and meaningful to describe the cover rocks of Sellafield through which the RCF shafts will be sunk as 'strong' rather than as "*relatively weak*" (PE/FOE/7, paragraph 4.55).

3.21 Dr Allison makes a point concerning Sellafield strengths in comparison with those encountered in the Swedish and Canadian underground laboratories. He says (PE/FOE/7, paragraph 4.57):

"*However, the underground research laboratories in Sweden and Canada are sited in plutonic, massive, hard, crystalline rocks. The compressive strengths of these rocks are generally greater and substantially more uniform than those associated with the rock sequence at Sellafield. Blast damage in these hard, relatively massive and uniform crystalline rocks, particularly where careful blasting techniques are employed, would be expected to be minimal, and sensibly constant; but this cannot be said for the rock sequence at Sellafield*."

3.22 This is misleading. He states correctly that blast damage in the hard, relatively massive and uniform crystalline rocks of the underground laboratories in Sweden and Canada would be expected to be minimal and sensibly constant, particularly where careful blasting techniques are employed. However, I disagree that the same cannot be said for the rock sequence at Sellafield for three reasons:

i. Nirex's blast consultant, who has experience of cautious blasting in the URL, has developed blast designs for the rock sequence at Sellafield. These designs utilise the available rock strength data from boreholes including the South Shaft centre line borehole (RCF3) and were developed to maintain minimal damage to the rock mass (estimates of average blast damage expected for the sandstones, Brockram and BVG are given above in paragraphs 3.11, 3.12 and 3.13 respectively).

ii. Nirex is a partner in the joint SKB/ANDRA/Nirex excavation damage research project (ZEDEX) underway in Äspö, Sweden (**PE/NRX/16**, paragraph 4.21, fourth and fifth bullet points, and paragraph 4.23). As part of this project, the extent of blast-induced damage to the rock mass has been measured by a number of techniques (which I describe in Appendix 3 of this Supplementary Proof of Evidence). Figure 3.1 gives preliminary and so far unpublished results from the ZEDEX project which show that blast-induced damage in the Swedish rocks ranges from 30 centimetres to 80 centimetres. This compares with Nirex's blast design consultant's calculated average value for the extent of damage of 50 centimetres for the BVG, as I explain in paragraph 3.13, demonstrating that measured blast damage extents in the rocks of the Swedish facility are comparable to expected blast damage extents in the Sellafield rocks.

iii. The extent of measured blast induced damage in the rocks of the Canadian URL has been reported by Martin et al. (NRX/16/8). Martin et al. describe changes in the in situ hydraulic properties to be confined to a zone less than 0.5 metres deep. They also report that most of the flow occurs in the excavation-disturbed zone in the fractured rock immediately around the excavated opening. The extent of this fractured rock is less than 300 millimetres, as reported by Martin et al. (NRX/16/8, Conclusions, page 8). This range of a zone less than 50 centimetres thickness for change in hydraulic properties, and less than 30 centimetres for the extent of fractured rock, is broadly consistent with Nirex's blast design consultant's calculated average value for the extent of damage of 50 centimetres for the BVG, as I explain in paragraph 3.13, demonstrating that measured blast damage extents in the rocks of the Canadian facility are comparable to expected blast damage extents in the Sellafield rocks.

Alternative Methods to Drill and Blast

3.23 To obtain a better understanding of the extent to which the excavation disturbed zone depends upon the method of excavation, Nirex is a partner in the ZEDEX project. One of the objectives of the ZEDEX project is to understand the mechanical behaviour of the excavation disturbed zone with respect to its origin, character, magnitude of property change, extent, and dependence on excavation method (NRX/16/1, paragraph 2, page 1).

3.24 The preliminary and so far unpublished ZEDEX results given in Figure 3.1 show that there is little or no significant difference in the measured extent of damage to the rock mass caused by the Tunnel Boring Machine (about 35 to 50 centimetres) compared with the drill and blast method (about 30 to 80 centimetres).

3.25 The possibility of constructing the RCF shafts by machine boring was given consideration by Nirex before the planning application for the RCF was submitted. Several consultants were used to evaluate the practicalities of this boring method. In particular, Wirth, as the only manufacturers of a proven shaft boring machine (the 'V' Mole) and RUCBOR, the South African company with the most experience of operating the 'V' Mole, were consulted. Technical papers from the literature on shaft boring were also examined to establish the extent to which any shaft boring projects so far carried out could be considered to be comparable to the RCF shafts.

3.26 The conclusion which was reached, as stated in my evidence (**PE/NRX/16**, paragraph 6.16), is that shaft boring is not an established method of construction for shafts at these depths where there are no pre-existing underground workings for spoil clearance via a pilot hole. Even with the benefit of a pilot hole, as suggested by Dr Allison, there is only one known example of a project carried out to the full depth of the Variant Case. In 1989 the Oryx Mine in South Africa successfully bored a shaft to 1000 metres, effectively defining an upper limit to current experience of shaft construction by machine boring methods.

3.27 To illustrate this point graphically, Figure 3.2 shows the numbers of bored shafts around the world in terms of the diameters and depths of bored shafts. The data for Figure 3.2 are reproduced in Appendix 4 of this Supplementary Proof of Evidence.

3.28 In addition to the practical difficulties of working at the upper limit of current experience, there are two additional reasons why Dr Allison's proposal is disadvantageous:

i. Verticality. For shafts used purely for ventilation or services (e.g. power, water), verticality of the shaft is often not a requirement. For the RCF shafts verticality is a requirement, in order to wind men and materials. Maintaining verticality with a machine bore and pilot hole is difficult. In constructing the Oryx Mine bored shaft, for example, verticality was not maintained.

ii. Safety. Connecting a pilot hole via an adit for muck (spoil) clearance through water bearing sandstones to the bottom of another shaft in which persons are working is a potential safety hazard, which would not exist if the shafts were constructed by the method proposed by Nirex.

Rock Support in the RCF

3.29 Dr Allison says:

> "*In evidence presented by Dr Mellor [paragraph 6.23, **PE/NRX/16**, et seq], it is stated that the upper portion of the shafts, within the cover rocks above the BVG will be lined with a hydrostatic concrete lining, in order to provide structural support for the openings*" (PE/FOE/7, paragraph 4.62) .

3.30 This is wrong. I do not say this anywhere in my evidence. The purpose of the hydrostatic lining is to reduce the inflows of water into the completed shaft, as described in my evidence (**PE/NRX/16**, paragraph 4.18) and as explained more fully in paragraph 3.48 of this Supplementary Proof of Evidence.

3.31 Dr Allison says:

> "*Evidence in support of Nirex's proposals contains no evidence concerning rock support for excavations within the BVG*" (PE/FOE/7, paragraph 4.66).

3.32 To address this point, the following paragraphs explain the general principles adopted for rock support in the RCF, together with a brief account of the development of support designs and an outline of the permanent support requirements for the RCF shafts and galleries.

3.33 Most of the rocks through which the RCF shafts will be sunk will be self supporting. That is to say that no additional support (to restrain rock mass deformations) will need to be installed over most of the shaft depth. However, Nirex's consultants responsible for developing preliminary designs for the rock support systems for the RCF shafts and galleries have designed additional support systems which will where necessary be installed as soon as practicable. For the shafts, such installation will take place during the mucking out (i.e. spoil clearance) operations.

3.34 The forms of support which have been designed for the RCF range from self-supporting, through to rock bolts only (in the form of spot or pattern bolting), to sprayed concrete only and to rock bolts and fibre reinforced sprayed concrete. Mesh will be utilised where appropriate for safety reasons and not for rock support.

3.35 To ensure that the installed support systems perform as required in the design and that excavation disturbance is kept to a practicable minimum, the support systems will be instrumented and monitored. The optimisation of support and its timing of installation will be achieved by monitoring the installed support

system itself and the excavation profile at frequent intervals until the support system and the excavation have reached equilibrium. Thereafter monitoring will continue at less frequent intervals.

3.36 The intervals and types of monitoring will be dependant upon the location of the support with respect to the sump, face or excavation intersections and the observed rates of change of deformation of either the support systems or the excavation profile.

3.37 If data from monitoring, or any other indicators such as routine visual inspection, suggest that the support system is not performing in a satisfactory manner, remedial measures in the form of additional support will be carried out immediately to ensure that equilibrium is restored.

3.38 The general principles of rock support, and the forms of support which I have outlined for the RCF, are standard principles and forms in both civil and mining engineering. By means of frequent monitoring, inspection, maintenance and where necessary remediation, the support systems could if necessary be assured well beyond the life of the RCF.

Groundwater Control

3.39 On Day 6, the methods for sealing the shafts against groundwater inflow were examined by the Inspector in questions put to Dr Chambers. In this section, I explain those design aspects of the RCF shafts which control groundwater inflows to the shafts.

3.40 The designs and methods for controlling groundwater inflow in the shafts have been developed for Nirex by consultants associated with, and responsible for designing, most of the shafts built in the United Kingdom in the last 20 years, and numerous shafts overseas. The general principles of design for the lining are standard principles in both civil and mining engineering. By means of frequent monitoring, inspection, maintenance and where necessary remediation, the lining may be assured well beyond the life of the RCF.

3.41 The control of groundwater inflow to the shafts will be achieved by two methods. These are:

i. probe hole testing and cover grouting; and

ii. hydrostatic lining.

Details of these two methods are given in paragraphs 3.42 to 3.57 below.

Method i: Probe Hole Testing and Cover Grouting

3.42 As explained in my Proof of Evidence (**PE/NRX/16**, paragraph 4.10) in the course of sinking the shaft, the shaft sinker will drill probe holes down ahead of construction. These probe holes will be hydraulically tested in order to identify sources of groundwater which will require pre-treatment (cover grouting) in order to control any inflows into the excavation to safe, acceptable levels.

3.43 The probe holes will be drilled in sets of four in a spiral pattern from the shaft quaterlines. The probe hole lengths will be about 40 metres. In the course of sinking the shaft successive sets of probe holes will be drilled so that they overlap each other by about 10 metres.

3.44 The use of probe holes allows in situ testing of the ground beneath the shaft floor, during shaft sinking operations. Such testing is required by the shaft sinker in order to enable operational decisions to be made about grout injection from the shaft floor into the rock mass through which the shaft will subsequently be sunk. However, a preliminary design of patterns of grout injection holes has already been completed by Nirex's consultants, based upon the geological, hydrogeological and geotechnical data from boreholes described in Dr Chaplow's Proof of Evidence (**PE/NRX/14**). The test data from probe holes is used to refine this preliminary design once the position and hydrogeological properties of particular potential flowing zones have been confirmed by in situ measurements obtained as part of normal sinking operations.

3.45 Once a zone requiring ground treatment is confirmed the shaft will be sunk to about 10 metres above the potential flowing zone. A ring of holes will then be drilled and a set of valved standpipes cemented in and pressure tested. Once satisfactorily pressure tested, a set of injection holes will be drilled through the valved standpipes cemented into the shaft sump, and fanned out around the proposed excavation line of the shaft. Grout is subsequently injected into the water bearing ground through the injection holes. This pattern of fanning out of injection holes gives rise to the general shape, or the distribution, of grout injected through them. The pattern is called 'cover grouting'.

3.46 A typical grout cover for the RCF will extend vertically to between 30 and 40 metres and, if necessary, two or more overlapping covers will be deployed in order to achieve a full grout treatment.

3.47 If the zone can be treated with a single cover, this will take the approximate shape or form of a hollow cone or curtain. If multiple covers are needed, the grouted zone will comprise a series of overlapping and interlocking cones as shown in Figure 3.3.

Method ii: The Hydrostatic Lining

3.48 The purpose of the hydrostatic lining is to reduce inflows of groundwater to the completed shaft. The concrete hydrostatic lining has been designed to withstand the hydrostatic head in accordance with United Kingdom practice for sinking shafts through water-bearing sedimentary rocks. The lining has been designed to a high standard in order to provide a durable structure which will give a watertight, dry shaft through the lined section.

3.49 The term 'watertight' is here taken to mean that some groundwater is expected to pass through the lining forming damp patches and small weeps or seeps, but that the total quantity so passing through the lining would be less than 25 litres per minute for the entire lining of one shaft. In practice this amount would be unlikely to be noticed as water running down the lining, because much of the water would be removed by evaporation from the ventilation. As part of routine inspection of the hydrostatic lining, damp patches and weeps are monitored and may if necessary be remediated by further backwall grouting to improve the watertightness of the lining (backwall grouting is explained in more detail in paragraphs 3.51 to 3.53 of this Supplementary Proof of Evidence). Because hydrostatic pressure increases linearly with depth, the shaft lining thickness also increases with depth, in a series of steps. This in turn means that the excavated diameter is also increased with depth, in a series of steps, in order to accommodate the lining at a constant finished internal diameter of 5 metres. In addition to backwall grouting, a shaft seal is emplaced at the base of the shaft as explained in paragraph 3.54 to 3.57 of this Supplementary Proof of Evidence.

How the Permanent Lining is Installed

3.50 The permanent concrete lining is cast in situ from the top of the shaft downwards as excavation of the shaft proceeds. Generally the lining is cast some 5 to 15 metres behind the shaft sump and is cast in 5 to 6 metre sections. Once the hydrostatic lining has been cast in situ, it is possible that there may be remnant voids behind the lining (i.e. between the lining and the rocks) which may still be able to transmit water into the voids. It is important to the performance of the lining to shut these voids off, and this is accomplished by a technique known as backwall grouting which is explained in the next few paragraphs.

How Backwall Grouting Makes the Lining Watertight

3.51 The backwall grouting process can be undertaken when the concrete of which the lining is made has cured sufficiently to provide adequate strength to resist backwall grouting injection pressures. Backwall grouting makes the lining watertight in two ways:

i. It prevents the passage of water behind the lining by infilling voids, including joints and fractures which may have opened around the opening of the excavation (see Appendix 1 for a description of rock 'relaxation' around the excavation). In the case of the RCF it will prevent vertical mixing of the lower saline water with that of the main aquifer; and

ii. It seals the construction joints of the in situ lining (i.e. the junctions between successive pours of the concrete which have formed the lining). This improves the overall integrity of the lining.

3.52 Backwall grouting will be completed in stages as soon as the concrete has cured sufficiently to provide adequate strength to resist backwall grouting pressures.

3.53 Backwall grout injections proceed upwards from the lowest groutable level (determined by the lowest point to which the concrete is known to have cured to acceptable strength). Grout injection behind the lining takes place through purpose-built grout injection pipes cast into the lining. Injections are carried out in successive passes using progressively increasing grout pressures until leakages are eliminated over the particular length of lining. The injection is terminated at a pre-determined finishing pressure. This pressure

is the maximum required to enable the passage of the grout a sufficient distance into the rock mass without hydrofracturing the rock mass or over-pressuring the hydrostatic lining.

Shaft Seal

3.54 Close to the base of the hydrostatic lining, a lining seal is placed. The purpose of this seal is to prevent groundwater from the sandstones which might move down behind the lining from escaping below the lining and entering the unlined section of the shaft below. For the RCF shafts, this seal will comprise a combination of two cementitious seals and one chemical seal. Once these are placed, the intervening lining between them is pressure backwall grouted using superfine cementitious mixes through grout pipes set in the hydrostatic lining at half the normal interval. The purpose of this construction is to provide continuous annular seals and to fully displace voids and pathways with both cementitious and chemical grouts. The seals can be engineered to penetrate and seal very small void spaces. The lining seals will be emplaced in either the North Head Member or the Brockram, depending on ground conditions as found.

3.55 The method of construction is slightly different for each seal type. To emplace the chemical seal (which is the uppermost of the seals within the seal combination), the shaft excavation is carefully over excavated (for example by hand) to form an annular excavation in addition to that required to accommodate the lining. This annular excavation may be up to 400 millimetres thick and between 4 and 6 metres in height. Once excavated, a series of pre-cast concrete segments are placed within this over excavation so as to provide an annulus of approximately 250 millimetres between the rear of the segments and the rock face. The lower 1 metre in length of annulus is then filled with an expanding concrete, on top of which 2 metres of chemical grout and gravel are placed. This is finally topped with approximately 1 metre of expanding concrete and retained by a temporary rolled steel ring anchored to the rock face. Once set, the hydrostatic lining is cast, incorporating several rings of grout pipes. Once the lining has cured the grout pipes are drilled out and the chemical seal re-injected under pressure to ensure an effective seal.

3.56 The cementitious seals, of which there are two located below the chemical seal, are formed in a similar manner to the chemical seal except that the over-excavation is some 100 to 200 millimetres, the length is of the order of 1 metre and the method of restraining the gravel pack material is by the temporary utilisation of steel sheeting. Once the gravel pack is set the lining is cast with two rings of grout tubes. Once the lining is cured the gravel pack is injected with cementitious grout.

3.57 Once all the seals are in place and the lining has fully cured the intervening sections of shaft lining are backwalled grouted to the maximum allowable pressure with grout mixes utilising superfine cements. This combination of one chemical seal, two cementitious seals and backwall grouting with superfine cements forms an extensively sealed section of the shaft some 30 metres in length close to the shaft base.

4. HEADGEAR HEIGHT

4.1 On Day 2 and Day 26, Mr Spendlove asked whether or not the RCF shafts were the same shafts as those which had been intended for the revised conceptual design of the Sellafield repository published in 1991 (in SPD/1/1). In his evidence, Mr Spendlove draws a comparison between the headgear heights for the RCF shafts and those of the repository conceptual design and concludes that (PE/SPD/1, paragraph 5.4):

> "*It was therefore clearly the intention at that time by Nirex to limit the height of protruding structures to 15 metres above ground level in order to meet public concerns. The RCF proposals for construction headgears of 25 to 30 metres and permanent headgears of 29.2 metres above ground level clearly do not meet those concerns and therefore it is not correct to say that 'the development has been located and designed to minimise any adverse physical effects on the surrounding area' (Ref.* ***PE/NRX/2****, paragraph 2.33).*"

4.2 I disagree with Mr Spendlove because the height of a headgear is determined by its purpose. In paragraphs 4.3 to 4.20 of this Supplementary Proof of Evidence I show that the purpose of the RCF shafts is very different to the purposes of the repository conceptual design to which Mr Spendlove is referring. In paragraphs 4.21 to 4.23 I show why Mr Spendlove's comparison is ill founded.

The Purpose of the RCF Shafts

4.3 The purpose of the RCF shafts is scientific. The shafts will enable Nirex to acquire data to address the uncertainties which Dr Chaplow identifies (**PE/NRX/14**, paragraphs 6.83 to 6.88, 7.31 to 7.34 and 8.17 to 8.19) and provide the information which Dr Hooper identifies (**PE/NRX/15**, Section 6).

4.4 The ways in which the RCF shafts will enable Nirex to acquire data to address these uncertainties are:

i. to provide access for observations and measurements during sinking of the shafts in Phase 1 as I explain in my evidence (**PE/NRX/16**, Section 4); and

ii. to provide access to the galleries for observations and measurements during Phases 2 and 3 as I explain in my evidence (**PE/NRX/16**, Section 5).

The Purposes of Shafts in the 1991 Repository Concept

4.5 A description of the revised conceptual design of the Sellafield repository was published in December 1991 as:

> "*The Repository Project - an engineering progress report describing the preferred design concept*" (SPD/1/1).

4.6 This engineering project report contains details of two different conceptual designs for a repository at Sellafield. The two different conceptual designs are called:

i. the Preferred Conceptual Design (SPD/1/1, page 4); and

ii. the Alternative Option (SPD/1/1, page 6).

4.7 The differences between the Preferred Conceptual Design and the Alternative Option are clearly set out in SPD/1/1, (on pages 6 and 9) and I do not reproduce them here. However, what I do draw attention to is a description of the shafts and the use which was envisaged for them at that time within the framework of the conceptual design for the repository.

The Preferred Conceptual Design

4.8 Two ventilation shafts were envisaged in the Preferred Conceptual Design. Each of the shafts was of about 4.5 metres diameter (SPD/1/1, page 6, third bullet point).

4.9 The two shafts were envisaged as having emergency access facilities for men and equipment (SPD/1/1, page 6, third bullet point). They were not envisaged as providing man riding facilities or materials access.

4.10 The two shafts and their permanent headgear were envisaged as being contained within two buildings about 15 metres high (SPD/1/1, page 6, third bullet point).

4.11 The two shafts did not prove a means of access for constructing other parts of the repository.

4.12 The temporary headgear which was envisaged for sinking the two shafts was 25 to 30 metres in height. After sinking the shafts the temporary headgear would have been dismantled and the permanent headgear would have been installed.

The Alternative Option

4.13 Three shafts were envisaged in the Alternative Option. Two were of 4.5 metre diameter, and the third was of 8.5 metre diameter (SPD/1/1, page 6, last bullet point).

4.14 Unlike the shafts of the Preferred Conceptual Design, the two 4.5 metre diameter ventilation shafts of the Alternative Option were envisaged as providing man riding facilities and materials access (SPD/1/1, page 6, last bullet point).

4.15 The two 4.5 metre diameter shafts of the Alternative Option were envisaged as having headgear with overwind protection, underslinging allowance and significantly larger decking heights than would be required of the shafts for the Preferred Conceptual Design. These features would be essential to allow for

the safe riding of the shaft by persons and for the transport of large items to support construction of the repository.

4.16 The third shaft for the Alternative Option was envisaged for transfer of waste packages from the surface (SPD/1/1, page 6, last bullet point).

4.17 All three shafts of the Alternative Option were envisaged as having headgear which could be accommodated within 15 metre high buildings (SPD/1/1, page 9, first paragraph).

4.18 To accommodate the larger decking heights, underslinging allowances and overwind protection, the Alternative Option envisaged below ground decking for all three shafts.

4.19 In the Alternative Option, below ground decking was envisaged as providing access to all three shafts in the headworks site. Access to the headworks site was envisaged as being from the waste receipt area on the BNFL Sellafield site via bored tunnels running about 50 metres below ground level (SPD/1/1, page 9, first complete bullet point). Access from the bored tunnels into the shafts was envisaged as being via insets in each shaft at about 50 metres below ground level.

4.20 The Alternative Option was subsequently set aside by Nirex. Subsequent work has raised confidence in the feasibility of the Preferred conceptual Design and the alternative would not offer the same advantages in terms of operational simplicity as the Preferred Conceptual Design, as stated in the report (last paragraph of Advantages of the Preferred Scheme, SPD/1/1, page 9). A major aspect of this lack of operational simplicity stemmed from the reliance on below ground decking.

Mr Spendlove's Comparison

4.21 Mr Spendlove's point (which he makes in PE/SPD/1, paragraph 5.4), in respect of construction headgears and permanent headgears, is that the heights of the headgears in the repository conceptual designs which predate the RCF were limited to 15 metres to meet public concerns and that this has not been done for the RCF.

4.22 He is wrong. The construction headgear heights in the Preferred Conceptual Design and the Alternative Option and the RCF were all in the same range of 25 to 30 metres.

4.23 The reason that the RCF permanent headgear height is 29.2 metres above ground level is to provide ground level decking, and underslinging allowance and overwind and detaching capability for reasons of safety. No such requirements existed in the Preferred Conceptual Design for the reasons I explain in paragraphs 4.9 to 4.11 above. Such requirements as did exist in the Alternative Option were envisaged as being accommodated by means of below ground decking at an horizon 50 metres below ground level as I explain in paragraphs 4.15 to 4.19. The Alternative Option was subsequently set aside, as I explain in paragraph 4.20.

5. ALTERNATIVE SCHEMES FOR THE RCF SHAFTS

5.1 Mr Spendlove suggests (PE/SPD/1, paragraph 5.5) two alternative schemes for the RCF as options which might further minimise adverse physical effects on the surrounding area:

> "*An examination of the contours depicted in* ***PE/NRX/1****Figs. 6.2 and 9.1 shows that the 67.0 metre above OD contour is within 125 metres of the proposed South Shaft site and to the South West. It would therefore be possible to construct the Shaft Collar (i.e. landing level) at a height of, say, 69 metres above OD with a 'level' approach, retain the drainage facility, and as a result the top of the permanent headgear would be at 97.2 metres above OD. There would then be a height of 12.7 metres above the proposed shaft platform level of 84.5 metres above OD. These figures are put forward to indicate that there are options available to meet the previously expressed intention in December 1991*

i. by excavating into the South West side of the existing drilling platform; or

ii. by moving the shaft locations a short distance towards the South West.

5.2 In the following paragraphs of this Section I comment on the implications of Mr Spendlove's alternative options i and ii as set out in paragraph 5.5 of his Proof of Evidence (PE/SPD/1).

Option i: By Excavating into the South West Side of the Existing Drilling Platform.

5.3 Mr Spendlove does not make plain in paragraph 5.5 of his evidence (PE/SPD/1) to what extent he considers that it might be appropriate to excavate into the south west side of the existing drilling platform. Without better details, I am unable to address this alternative option. Mr Spendlove has been asked to provide better details and when they have been considered, Nirex may wish to address them in a Supplementary Proof of Evidence.

Option ii: By Moving the Shaft Locations a Short Distance Towards the South West.

5.4 To move the locations of the shafts as Mr Spendlove suggests would be substantially disadvantageous to the scientific purpose of the RCF, as I explain in paragraphs 5.5 to 5.10 of this Supplementary Proof of Evidence.

5.5 There are five fundamental disadvantages to Mr Spendlove's suggestion to move the shaft locations. Taken together, they are sufficiently disadvantageous as to render Mr Spendlove's proposals impractical. The fundamental disadvantages are:

i: Geology

5.6 Mr Spendlove's proposal would put the shafts outside the volume of rock within which Nirex has characterised the geology in detail and which is described in *Nirex Report S/95/007* [COR/524]. This volume of rock is approximately 50 metres in diameter extending to about 900 metres bOD (COR/524, paragraph 1.3). The boreholes which provide the data used in this detailed study comprise boreholes RCF3, RCM1 and RCM2. These boreholes lie approximately 30 metres apart at the apices of a near isosceles triangle, as clearly shown in Figures 1.1 and 1.2 of COR/524. A position outside this volume would make the study of continuity between connected fractures mapped in the shafts and flow zones observed in the boreholes significantly more difficult. The need to extend our knowledge and understanding of these connections in three dimensions beyond the limits of the shafts is important and is described in my evidence (**PE/NRX/16**, paragraphs 4.5 and 4.6).

ii: Monitoring

5.7 Mr Spendlove's proposal would put the shafts outside the centre of the groundwater pressure monitoring system which is described in Dr Chaplow's evidence (**PE/NRX/14**, Table A.11). This would impair the effectiveness of this monitoring system to detect the drawdown caused by shaft sinking. This is an important point because a key scientific outcome of Phase 1 will be the information which can be gathered regarding the hydrogeological performance of the rock mass as noted by the RWMAC (paragraph 16) [GOV/414]. The

boreholes of the groundwater monitoring system in and around the Potential Repository Zone have been carefully designed and their locations are shown in Figure 5.1. This figure shows the locations of boreholes on the surface of the ground as open circles. However, some of the boreholes are intentionally deviated from a vertical trajectory, and therefore it is also important to show the subsurface trajectories of the boreholes in addition to their surface locations. This has been done in Figure 5.1 by marking the surveyed subsurface borehole trajectories as green lines from the borehole surface positions. The North and South Shaft locations as proposed by Nirex are shown in Figure 5.1 as green squares. I show my interpretation of Mr Spendlove's alternative shaft locations in Figure 5.1 as red squares. It is evident from Figure 5.1 that Mr Spendlove's proposal would put the shafts outside the central portion of the roughly triangular shaped area of five monitoring boreholes (RCM1, RCM2, RCM3, RCF1 and RCF2). The drilling of one or more new and additional boreholes would be required in order to redress this lack of adequate monitoring capability.

iii: Depth to Rock Head

5.8 Mr Spendlove's proposal would put the collar and foreshaft construction work for the South Shaft into a water saturated, drift-filled hollow which is part of a buried valley feature cut into rockhead. This would entail construction through up to 22 metres of unconsolidated ground (this being the difference between the 67 metre aOD surface elevation and the base of the hollow at 45 metre aOD), as shown in Figure 5.2. This would make construction works more difficult and time consuming, with associated environmental impacts, and more expensive. It would also require the collar and foreshaft to be redesigned.

iv: Shaft Centre-line Boreholes

5.9 Nirex has drilled Borehole RCF3 as a centre-line borehole for the South Shaft and has planning permission to drill Borehole RCF4 as a centre line borehole for the North Shaft. Mr Spendlove's relocation for the shafts would therefore either require the South Shaft to be sunk without the benefit of a centre line borehole to aid construction of the shaft or else require a new and additional borehole to be drilled.

6. REUSE OF RCF SHAFTS IN A REPOSITORY

6.1 In cross examination on Day 26, Mr Brodie put to Mr Folger that to cut costs and save unnecessary time delays, the RCF will need to be constructed so as to allow it to be adapted to form part of the deep repository. Mr Folger stated that I would deal with this matter.

6.2 The RCF shafts have been designed for scientific purposes, as I have explained in paragraphs 4.3 and 4.4. Nirex has given some consideration to possible future incorporation of the RCF shafts into the repository. There are five aspects which have been explicitly considered in developing the design of the RCF shafts in respect of possible future incorporation into a repository. These are:

i. The location of the shafts, which is explained in Dr Hooper's Proof of Evidence (**PE/NRX/15**, paragraphs 7.3 to 7.6);

ii. The diameter of the shafts has been sized to meet the needs of the scientists working in the shafts and to provide efficient methods of spoil clearance. The spoil clearance method which has been adopted for the RCF shafts is the twin kibble method, which is more efficient than spoil clearance using a single kibble. In practical engineering terms it would be inefficient to operate a twin kibble system in a shaft with a diameter much less than 5 metres. The RCF shafts have not been sized to achieve any particular purpose or function in a future repository.

iii. Records of departures from the detailed design drawings for the RCF shafts which may arise as a result of the actual ground conditions as found, and the construction as actually completed. These are known as the 'as-built' records. The scientific works which are planned to be carried out during, and as an integral part of, construction of the RCF are expected by Nirex to provide adequate as-built records for any future change of use of the RCF shafts which might be proposed;

iv. The design life of the shafts, which I have explained in paragraphs 3.38 and 3.40 above is based on standard civil and mining engineering principles. No special design features have been adopted for the RCF shafts.

v. Sealing of the shafts. Nirex has taken the precaution of ensuring that no grouts or sealing materials are deployed in construction of the shaft without Nirex's prior approval. Before giving such approval we will consider the potential impact on the performance of a repository.

7. POINTS RAISED BY THE RWMAC

7.1 In his Supplementary Proof of Evidence, Mr Folger makes reference (**PE/NRX/12/S1**, paragraph 7.5) to the September 1995 RWMAC Report [GOV/414], and in particular, to paragraph 32 of that report which suggests a number of technical matters to which Nirex should give particular attention in its planning and execution of the project. In this Section I describe the current position on each of the matters identified by the RWMAC.

7.2 The technical matters which the RWMAC has identified and which it has suggested the Company should give particular attention to (paragraph 32) [GOV/414] are:

> " *provision of an adequate period of time before shaft sinking begins in the Sherwood Sandstone to ensure that the base hydrogeological conditions have been established; prior confirmation that a baseline has been achieved should be provided by independent peer review;*
>
> - *prior publication of the predicted drawdown associated with shaft excavation;*
> - *the need for shaft lining in the BVG;*
> - *monitoring of microseismic activity, natural gases and thermal regime;*
> - *prototype trial of use of friable grout backfill;*
> - *distribution of dewatered ground around the underground excavations."*

I deal with each of the following points under separate headings in the following paragraph.

Point i: Baseline

7.3 The issue of confirmation that a hydrogeological baseline has been achieved is addressed by Dr Chaplow in his Supplementary Proof of Evidence (**PE/NRX/14/S1**, Section 10).

Point ii: Prior Publication of Predictions

7.4 Nirex's policy on publication is addressed by Mr Folger (**PE/NRX/12/S1**, Section 8). In giving his evidence in chief on Day 25, Mr Folger confirmed that Nirex will publish in advance its predictions of significant findings expected to arise during shaft sinking. These publications of predictions will include projections of the hydrogeological effects of shaft sinking and the responses which we expect to arise on the groundwater pressure monitoring system described in Dr Chaplow's Proof of Evidence (**PE/NRX/14**, Table A.11).

7.5 In giving his evidence in chief on Day 25, Mr Folger confirmed that Nirex will publish the results of Phase 1 of the RCF sector by sector. This will generate a sequence of twelve separate publications of results, one for each sector, for the Central Case (both shafts to access the 650 metres bOD horizon). The sequence may run up to sixteen separate publications for the full extent of the Variant Case (both shafts to access the 900 metres bOD horizon), consistent with Table 7.1 and Figures 7.1 and 7.2 of my Proof of Evidence (**PE/NRX/16**).

Point iii: Shaft Lining in the BVG

7.6 In terms of the design of the RCF shafts, the term 'lining' is used to describe structures to withstand hydrostatic pressures and to reduce groundwater inflows into the shaft, as explained in paragraph 3.48 of this Supplementary Proof of Evidence and in paragraph 4.18 of my Proof of Evidence (**PE/NRX/16**).

7.7 Rock support systems are described as methods and materials deployed to provide structural support to the rock mass and to ensure the health and safety of persons within the facility.

7.8 For the reason stated by the RWMAC (paragraph 20) [GOV/414] it is not the intention to line the BVG section of the RCF shafts :

> "*The objective of leaving the BVG section of shaft unlined is to permit free drainage of water into the shaft as part of the large-scale drawdown test*".

7.9 However, the RWMAC have expressed the view that the shaft may, in the long term, form part of a radioactive waste repository and that the condition of the rock around the shaft may deteriorate with time (paragraph 20) [GOV/414]. It is an objective of the Science Programme for the RCF to instrument, monitor and inspect the condition of the shafts and support systems as explained in paragraphs 3.35 to 3.37 of this Supplementary Proof of Evidence.

7.10 This instrumentation, monitoring and inspection will be maintained throughout the life of the shafts. Deterioration in the form of disequilibrium in the rock mass forming the walls of the shaft will be remediated to ensure that equilibrium is restored, and such deterioration stopped, as soon as is practicable.

7.11 The general principles of rock support for the RCF are set out in paragraphs 3.33 to 3.38 of this Supplementary Proof of Evidence. These principles accommodate the use of sprayed concrete and fibre reinforced sprayed concrete. If, following a period of instrumentation, monitoring and inspection of the shafts it was decided to add additional support to the BVG sections, then this would be accomplished by means of fibre reinforce sprayed concrete. This would be a continuous support system, not a hydrostatic lining, and would be installed with openings and drainage holes left along its length in order to permit free groundwater inflow as suggested by the RWMAC (paragraph 20) [GOV/414].

Point iv: Monitoring of Microseismic Activity, Natural Gases and Thermal Regime

7.12 All of these monitoring activities will be carried out in the RCF.

7.13 The microseismic activity monitoring will comprise:

- Acoustic emission and microseismic sondes installed in some of the boreholes of the groundwater pressure monitoring system described in Dr Chaplow's evidence (**PE/NRX/14**, Table A.11). To date these have already been installed in Boreholes RCF1 and RCM3 and Borehole 5; and
- Acoustic emission monitoring in short underground boreholes as part of the programme monitoring of excavation disturbance and blast induced damage described in my Proof of Evidence (**PE/NRX/16**paragraph 4.21, fifth bullet point).

Geophone and vibration sensors as part of blast monitoring, as outlined in paragraph 4.26 of this Supplementary Proof of Evidence.

7.14 In addition to this instrumentation, an additional seismic monitoring station with automatic triggering and recording systems will be installed towards the end of Phase 1. The final location of this station has yet to be decided. The current favoured location is in the telemetry room show in Figures 6.2 and 6.3 and described in paragraph 6.7 of my Proof of Evidence (**PE/NRX/16**).

7.15 The monitoring for natural gases including radon, toxic and combustible gases will be carried out in accordance with Health and Safety Executive (HSE) guidelines utilising a number of methods. In addition, routine monitoring for blast gases will be carried out. Monitoring will be by:

- Fixed Monitoring Underground - which will send signals to the surface. This is needed where rapid action needs to be taken in respect of identified hazards. e.g. carbon monoxide, smoke, hydrocarbon gases;
- Underground Measurements - these will be carried out with hand held equipment which enables the operator to check the airways in a more detailed manner prior to the commencement of and during his working period; and
- Air Sampling - where samples are taken from the airway on a regular basis for laboratory analysis.

Point v: Friable Grout Backfill

7.16 Nirex's strategy for undertaking sealing studies proposed for the RCF is set out in Dr Hooper's Supplementary Proof of Evidence (**PE/NRX/15/S1**, paragraph 6.33).

Point vi: Distribution of Dewatered Ground

7.17 The extent of drawdown associated with the excavation of shafts will be monitored on the groundwater pressure monitoring system described in Dr Chaplow's evidence (**PE/NRX/14**, Table A.11). Additionally, the extent of dewatering of the rock mass around the excavation will be continuously monitored in a number of underground boreholes drilled out from the RCF, including:

- Underground boreholes drilled out from galleries constructed at the end of Phase 1 as part of the peripheral drilling described in paragraph 4.14 of my Proof of Evidence (**PE/NRX/16**). These will be subsequently continuously monitored for groundwater pressure.
- Underground boreholes drilled out from one shaft towards the second shaft to monitor 'before and after' hydraulic conductivity changes within the excavation disturbance zone (**PE/NRX/16**, paragraphs 4.23 and 4.24). Some of these will be subsequently monitored for groundwater pressure.
- Underground boreholes drilled and continuously monitored as part of the SCD experiment (**PE/NRX/16**, paragraphs 5.7 and 5.8).
- Measurements of two phase flow characteristics of the fracture system as part of gas migration studies (**PE/NRX/16**, paragraph 5.20).
- Underground boreholes drilled out from the galleries in Phases 2 and 3 for the EDZ experiments described in my Proof of Evidence (**PE/NRX/16**, paragraphs 5.25 and 5.26).

7.18 In summary, all of the matters raised by the RWMAC in their September 1995 report (paragraph 32) [GOV/414], have been addressed.

8. ONE SHAFT RATHER THAN TWO

8.1 In his evidence Mr Spendlove makes out a case for initially sinking only one shaft for the RCF (PE/SPD/1, Section 6). In this Section Evidence I explain that Mr Spendlove's case is impractical.

8.2 Mr Spendlove's case is that Nirex should minimise the number of intrusions (shafts) into the PRZ until we understand the system better. I believe Mr Spendlove's case is impractical for the following reasons:-

i. He does not demonstrate what benefits would flow from making the second intrusion (shaft) into the PRZ later rather than sooner.

ii. It is not possible to proceed to Phases 2 and 3 with only one shaft as this would not be safe. Nirex are advised that the Mines (Safety of Exit) Regulation do not apply, as the RCF would not fall within the definition of a mine. Nirex are also advised that it would be under a statutory and common law duty to ensure the health, safety and welfare of its employees and contractors, and to provide safe means of access and egress to their place of work. These duties would require the same access and egress provisions as if it were a mine. Nirex would, in any event, wish to ensure that the RCF was no less safe than if it were a mine. If the RCF were a mine, at least two shafts or outlets providing at least two separate exits to the surface would be required to progress to Phase 2.

iii. There are scientific benefits to be gained from sinking the second shaft at an early stage as planned. These include the acquisition of data on the excavation disturbed zone measured at one shaft using instruments installed in boreholes from the other shaft, as I explain in my evidence (**PE/NRX/16**, paragraph 4.21, fourth bullet point). This under-excavation method is also recognised by Friends of the Earth as a method for providing valuable information on the control of peripheral rock disturbance (PE/FOE/7, paragraph 4.60). The benefits also include information on continuity between connected fractures mapped in both shafts and fractures observed in existing boreholes, as I explain in my evidence (**PE/NRX/16**, paragraph 4.5).

iv. Mr Spendlove's case appears to rest on the point that execution of Phase 1 together with a small part of Phase 2 would enable a definite point of decision to be reached as to whether to abandon the project or continue (PE/SPD/1, paragraph 6.7). He provides no evidence to support this view. In contrast, Nirex has said that such a definite point of decision cannot be planned with precision (**PE/NRX/12**, paragraph

6.27, COR/101, paragraphs 1.39 and 1.40). The RWMAC state (at paragraph 34 of GOV/414) that in their view "*the information from Phases 2 and 3 may be important to demonstrate at the public inquiry [into a DWR] that a satisfactory safety assessment ..... can in due course be proven.*"

9. REFERENCES

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SUPPLEMENTARY PROOF OF EVIDENCE Dr DAVID MELLOR

TABLES, FIGURES AND APPENDICES

TABLE 3.1 - SCALE OF ROCK STRENGTHS

Strength MN/m^2 ($1MN/m^2 = 146\ lb/in^2$)	Term
<1.25	very weak
1.25 to 5	weak
5 to 12.5	moderately weak

12.5 to 50	moderately strong
50 to 100	strong
100 to 200	very strong
>200	extremely strong

Recommended by the Geological Society Engineering Group Working Party (NRX/16/7)

FIGURE 3.1 : ZEDEX - PRELIMINARY RESULTS OF MEASUREMENTS OF PLASTIC DEFORMATION/BLAST INDUCED DAMAGE FROM TUNNEL BORING MACHINE (TBM) DRIFT AND DRILL-AND-BLAST DRIFT. "R" DENOTES BLAST ROUND NUMBER.
Click on image to see larger picture

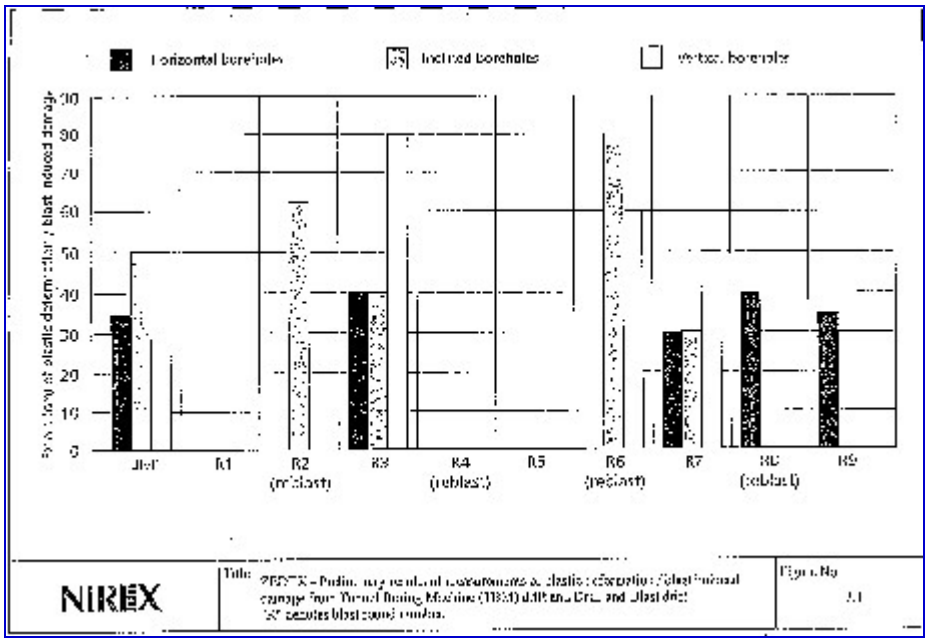


FIGURE 3.2: DIAMETER VS SHAFT DEPTH FOR EXISTING BORED SHAFTS (UPDATED 1995) (SHAFTS LESS THAN 2m IN DIAMETER OR 100m IN DEPTH NOT INCLUDED)
Click on image to see larger picture

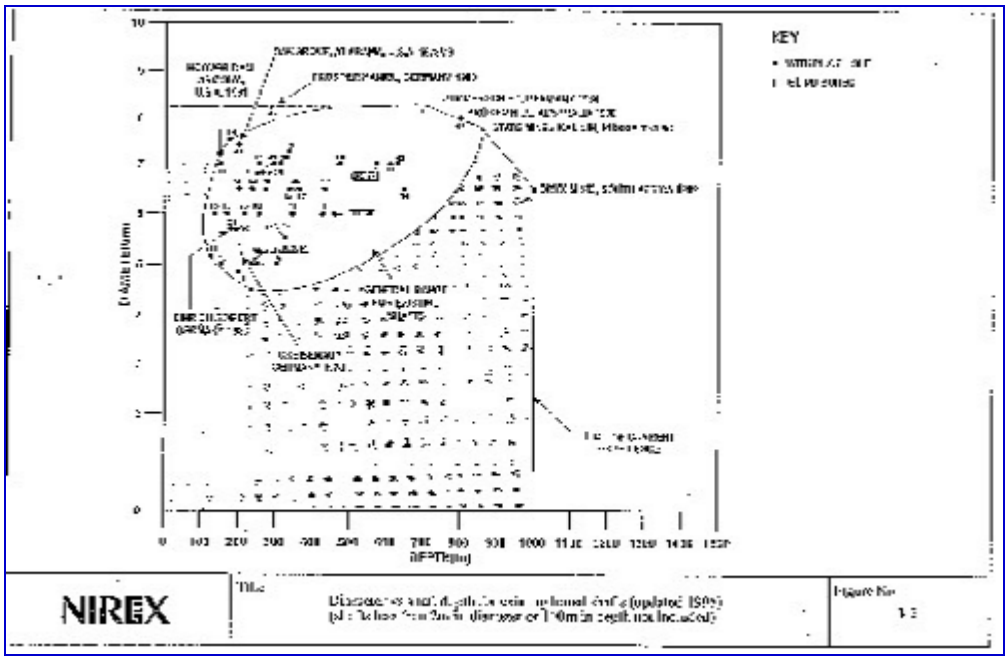


FIGURE 3.3: SKETCHES ILLUSTRATING VERTICAL SECTION THROUGH SHAFT WITH SINGLE AND MULTIPLE GROUT COVERS

Click on image to see larger picture

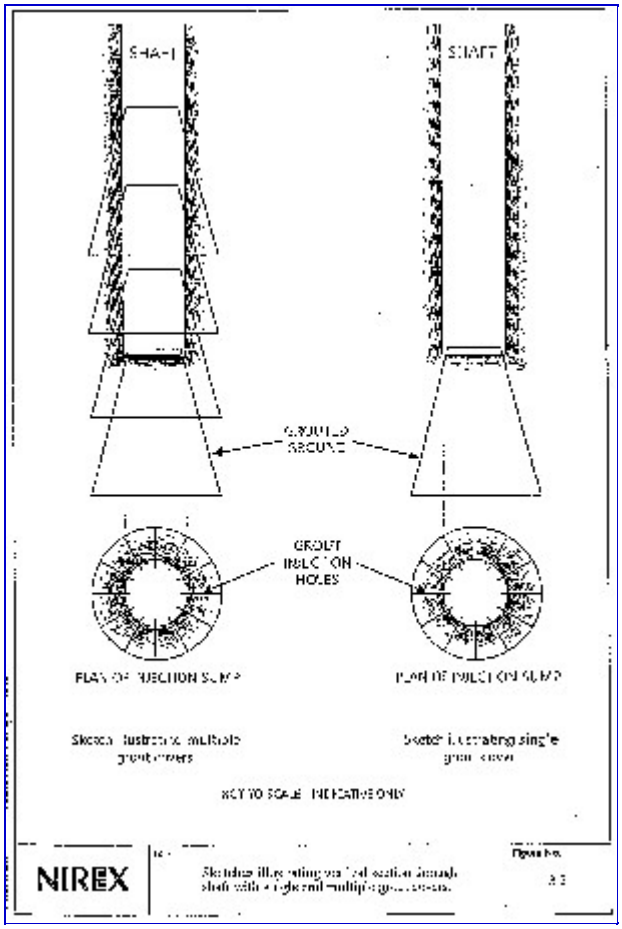


FIGURE 5.1: PRZ: NIREX MONITORING BOREHOLE LOCATIONS

Click on image to see larger picture

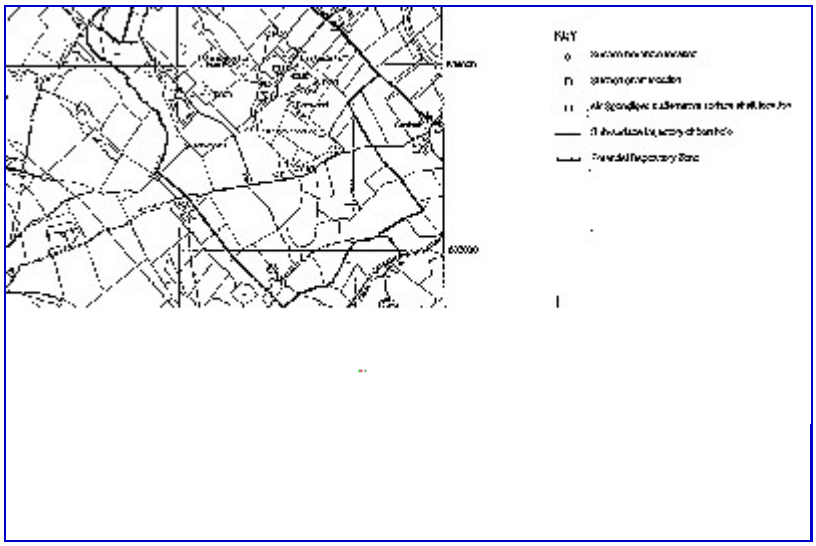
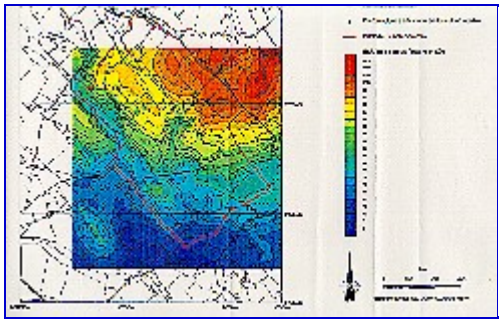


FIGURE 5.2: PRZ: ROCKHEAD CONTOUR MAP (REPRODUCED FROM DRAWING 010087 IN COR/518)
Click on image to see larger picture



APPENDIX 1

A General Understanding of Excavation Disturbance

The General Case: Elastic Deformation

A1.1 Creating an excavation in rock at depth by any method will cause a redistribution of stresses in rock in the vicinity of the excavation. This is sometimes referred to in simple terms as 'relaxation' of the rock around the excavation or opening. Strictly speaking, the redistribution of stress may introduce regions of increased compressive stresses around the perimeter of the excavation as well as regions of 'relaxation', or tensile stresses, in the vicinity of the excavation.

A1.2 In general terms, the stresses acting in towards the excavation tend to be reduced by comparison with the undisturbed (i.e. pre-existing) stress field. This may produce a tendency for existing fractures which are roughly circumferential to the shaft or tunnel (the so-called 'onion skin' or tangential fractures of Kelsall et al. FOE/5/14, page 128) to be opened up as shown in Figure A1.1. Close to the excavation, these tangential fractures will tend to increase the rock mass hydraulic conductivity in the axial direction i.e. parallel to the excavation, as stated by Kelsall et al. (FOE/5/14, page 127). Such increases in axial hydraulic conductivity have been measured in field tests and are recognised, for example, by Pusch and Stanfors (FOE/6/16, page 456). The tendency for pre existing fractures that are tangential to the excavation to be opened up is also noted by Long et al. (FOE/6/10, page 563).

A1.3 Also in general terms, stresses acting around the periphery of the opening tend to become concentrated and this results in a tendency for fractures which were initially approximately radial to the excavation to be closed down (as shown by Kelsall et al. FOE/5/14, in Figure 3). These radial fractures will tend, therefore to undergo a reduction in hydraulic conductivity, as stated by Kelsall et al. (FOE/5/14, page 128), and as noted by Pusch and Stanfors (FOE/6/16, page 456) and as noted by Long et al. (FOE/6/10, page 563), and as also shown in Figure A1.1.

A1.4 Paragraphs A1.1 to A1.3 of this Supplementary Proof of Evidence describe in general terms what is known as the elastic behaviour of rocks around an excavation. In simple terms, the description 'elastic' means that the deformation of the rock (i.e. the strain, or change in length of the rock) is linearly proportional to the change in stress, and that the deformation is (theoretically at least) reversible. In order to understand the changes in rock around an excavation more fully, it is essential to consider how rocks behave when they become broken and loosened around the excavation. The zone of broken and loosened rock around an excavation is sometimes called the plastic zone, and may be defined in basic terms as the zone in which the stresses around the opening have increased sufficiently to overcome the strength of the rock which subsequently becomes broken and loosened. It is important to note that a small (a few tens of centimetres) plastic zone may exist around an excavation at depths of several hundreds of metres no matter how carefully the excavation is made, and no matter what technique is used to create the excavation.

The Plastic Zone

A1.5 The significance of the plastic zone is made clear by Kelsall et al. (FOE/5/14, page 126) which should be read with reference to Figure A1.2 of this Supplementary Proof of Evidence:

> "*The distinction between the elastic and plastic zones around an underground opening is important with respect to stress distributions and the resultant effects on fracture permeability. In the case of elastic deformations adjacent to an opening, the radial stress is reduced close to the shaft tunnel wall whereas the tangential stress is increased. In this case, it is expected that the permeability of fractures tangential to the opening (perpendicular to the radial stress) should be increased, whereas the permeability of radial fractures should be reduced. In the case of plastic deformations adjacent to an opening, both the radial and tangential stresses are reduced close to the wall in the plastic zone so that the permeability of both tangential and radial fractures should be increased.*".

This clear statement gives a useful and general understanding of the impact of excavation damage on fluid flow for both the elastic and the plastic behaviour of the rock mass in response to the creation of an excavation within the rock mass. Kelsall et al. also provide a useful reference and a graphical example (FOE/5/14, Figure 2) for estimating the likely extent of the plastic zone as a function of excavation depth.

Blast Induced Damage

A1.6 In addition to the small extent of broken and loosened rock of the plastic zone, there will be some breakage and loosening of rock caused by blasting if that method is used to create the excavation. This further breakage and loosening may result in large increases in hydraulic conductivity immediately adjacent to an opening. However, as stated by Kelsall et al. (FOE/5/14, part of the abstract):

> "*Blasting may result in large increases in hydraulic conductivity immediately adjacent to an opening* ***but blast damage may be limited to within 0.3m of the opening using controlled blasting techniques with low perimeter charge weights***." (my emphasis)

A1.7 Kelsall et al.'s estimate of limiting blast damage to within 0.3 metres of the opening using controlled blasting techniques compares well with estimates of blast induced damage for the RCF shafts, which I provide in paragraphs 3.11 to 3.13 of this Supplementary Proof of Evidence, and with the results from the ZEDEX experiment which I describe in point (ii) of paragraph 3.22 and in Figure 3.1 of this Supplementary Proof of Evidence. It is also important to note from Figure 3.1 that there is little or no significant difference between the extent of plastic deformation which has been measured in the wall of the Tunnel Boring Machine excavated tunnel and the blast-induced and plastic deformation which has been measured in the wall of the drill and blast tunnel nearby.

A1.8 The estimate (by Kelsall et al.) of limiting blast damage to within 0.3 metre of the opening using controlled blasting techniques also compares well with the later work carried out in the Canadian URL and reported by Martin et al. (NRX/16/8). In particular, Martin et al. conclude that:

> "*Both single-borehole and large-scale hydraulic testing indicated that the excavation-disturbed zone for the tunnel investigated is less than 0.5m deep and that* ***most of the flow in the excavation-disturbed zone occurs in the fractured rock immediately around the excavation opening, which was less than 300 mm thick****. It would appear from the limited testing carried out that the excavation-induced fractures do not form a continuous path and that flows in the excavation-disturbed zone can be reduced significantly by simply increasing the flow path to greater than one blast round*" (my emphasis) (NRX/16/8, Conclusions page 9).

Magnitude of changes in rock mass hydraulic conductivity

A1.9 Nirex is aware of the significance of the excavation disturbed zone and are taking the practical steps to ensure that the Company will not have to rely on the detailed characterisation of it to make the Post Closure Safety Case. It is Nirex's position that it will be sufficient to demonstrate that the excavation disturbance has in reality no more enhanced axial hydraulic conductivity than the magnitude and extent currently assumed (**PE/NRX/15**, paragraph 6.47). This approach means that it is not necessary to ensure that excavation disturbance has to be eliminated or reduced to an insignificant value even if that were a practical possibility. However, this approach does need to be tested and the RCF is an essential part of testing our judgements about excavation disturbance.

A1.10 In his evidence, Dr Salmon says:

> "*Kelsall (FOE/5/14) considered that the stress changes caused by excavation may increase permeability by several orders of magnitude*". (PE/FOE/5, paragraph 4.11).

A1.11 This statement is potentially misleading because it may lead the reader to the erroneous view that such high increases in hydraulic conductivity are typical or representative of a significant volume of the excavation disturbed zone. They are not. As Kelsall et al. say in the abstract of their own work (FOE/5/14, part of abstract):

> "*The analysis predicts that hydraulic conductivity may be increased,* ***close to the wall of the opening****, by two or three orders of magnitude over the far field value solely in response to stress relief. The zone in which hydraulic conductivity is increased by at least one order of magnitude over the far-field value* ***is limited to within one or two radii from the opening****".* (my emphasis)

A1.12 In simple terms, Kelsall et al. (FOE/5/14) do not provide any general evidence for increases in hydraulic conductivity caused by the excavation disturbance zone larger than the two orders of magnitude over two excavation diameters which are currently assumed as an upper limit by Nirex, and which has been plainly stated in Nirex's evidence (**PE/NRX/14**, paragraphs 7.27 to 7.30, and **PE/NRX/15**, paragraphs 6.45 to 6.49) .

A1.13 In his evidence (PE/FOE/6, paragraph 6.16), Dr Hencher says:

> "*Similarly Pusch (1992) [FOE/6/16], with reference to work carried out at the Stripa Mine and at Äspö in Sweden, reports on the* ***severe disturbance*** *caused by tunnel construction in granite. The authors note that adjacent to excavation, bulk hydraulic conductivity differed in axial and radial directions by up to 10,000 times*." (my emphasis).

A1.14 This is misleading. The full abstract for the paper by Pusch and Stanfors referred to by Dr Hencher (FOE/6/16, abstract in full) clearly states a different perspective on net axial conductivity, and in particular on radial conductivity which they state as reducing and not increasing:

> "*Two large-scale field tests in granite for identifying possible excavation-induced rock disturbance have been performed in Sweden as part of the on-going research for developing techniques for disposal of highly-radioactive waste from Swedish nuclear reactors. The tests involved determination of the effect of blasting and stress changes on the near-field rock and have lead to a preliminary model of excavation damage.*
>
> *Blasting produces a* ***disturbed zone with enhanced axial hydraulic conductivity from a few decimetres to more than 1m from the tunnel periphery, depending on the charge. The axial hydraulic conductivity is increased by 2-3 orders of magnitude in this zone****, a net value being about 10^{-8}m/sec in rock with a virgin conductivity of 10^{-11}-10^{-10}m/sec. The increase in conductivity is partly due to stress changes, which raise the axial conductivity also further out in the rock. Such influence may take place to a distance from the tunnel periphery of* ***up to 50% of the tunnel radius and the net axial conductivity of the zone is estimated to be about 10 times higher than the average virgin conductivity if the tunnel is suitably oriented with respect to the fractures sets. This change is associated with a reduction in radial conductivity of around 5 times***." (my emphasis)

A1.15 Nowhere in their paper do Pusch and Stanfors (FOE/6/16) use the expression 'severe disturbance'. Dr Hencher is ascribing a weight and significance to the description of the disturbance which is unreasonable in the context of the work reported by Pusch and Stanfors. What Pusch and Stanfors do say about the disturbance is:

> "*The experience from the Äspö tests is in fair agreement with the conclusions from the Stripa experiments that even rather careful blasting of tunnels causes considerable disturbance down to at least 1m depth in the floor and to approximately 0.5m in the walls and roof*."
>
> ***General Preliminary Model of Disturbance of Rock around Blasted Drifts in Granitic Rock at Depth***
>
> *The field tests confirm that both blasting and stress changes cause very significant disturbance in the nearfield and the Stripa experiments demonstrate that this results in a substantial increase in axial hydraulic conductivity close to the periphery of drifts. Even very careful blasting increases the average conductivity of initially low-pervious rock by 2-3 orders of magnitude within 0.5-1.0m distance from the periphery, primarily through the creation of a shallow network of fractures, most of which are parallel to the periphery and produced by the combined effect of high gas pressures and high tangential stresses*." (FOE/6/16, page 455 and 456).

A1.16 Dr Hencher may have misquoted the paper by Pusch and Stanfors (FOE/6/16) which on page 79 (this being the first page of the paper) refers back to an earlier paper by Pusch (NRX/16/9). In this earlier paper by

Pusch, increases in axial permeability from the Buffer Mass Test in Stripa are described as may be having been as high as 1,000 to 10,000 times higher than that of the undisturbed rock, as shown in the abstract to Pusch's paper (NRX/16/9, abstract in full):

> "*Theory as well as small- and large-scale experiments indicate that tunnel excavations in rock result in an increased axial permeability close to the tunnels. This is due to stress relief as well as to blasting, of which the former is generally assumed to be most important, at least when smooth blasting is applied. The Buffer Mass Test in Stripa offered a possibility of quantifying the increase in axial permeability and it was concluded form this experiment that the increase may have been as high as 1,000 - 10,000 times* ***in a 0.5-1.0m wide zone adjacent to the excavation****, assuming that the value* 10^{-10}*m/sec derived from the preceding, large-scale "Macropermeability Test" is representative of the hydraulic conductivity of the undisturbed rock. If this figure is correct, disturbance by blasting would be the major cause of the increased perviousness*." (my emphasis)

A1.17 It is important to put the extent of these increased axial permeabilities into context. They apply to a zone of only 0.5 to 1 metre away from the shaft wall as is clearly stated by Pusch in the abstract above.

A1.18 Useful generalisations about the disturbed zone may be made by examining data from experiments in different places. However, the characteristics and the extent of the excavation disturbed zone are likely to be very site specific, as noted by Martin et al. (NRX/16/8, conclusions, page 9):

> "*The characteristics and extent of the excavation-disturbed zone are very site specific and depend on the shape of the excavation, its orientation relative to the stress field, the frequency and spacing of natural fractures, the properties of the rock and the natural fractures, the excavation method, and the magnitude and anisotropy of the stress field*".

A1.19 There is no evidence presented by or on behalf of Friends of the Earth, or in references relied upon in evidence given by Friends of the Earth, to indicate that increases in hydraulic conductivity greater than two orders of magnitude over an extent of two excavation diameters might be exceeded by excavating the RCF shafts or galleries.

FIGURE A1.1: PREDICTED EFFECTS OF STRESS CHANGESON ROCK MASS HYDRAULIC CONDUCTIVITY (ELASTIC STRESS ANALYSIS FOR FRACTURED BASALT - ISOTROPIC INITIAL STRESSCONDITIONS). AFTER KELSALL ET AL (FOE/5/14 FIGURE 3)

Click on image to see larger picture

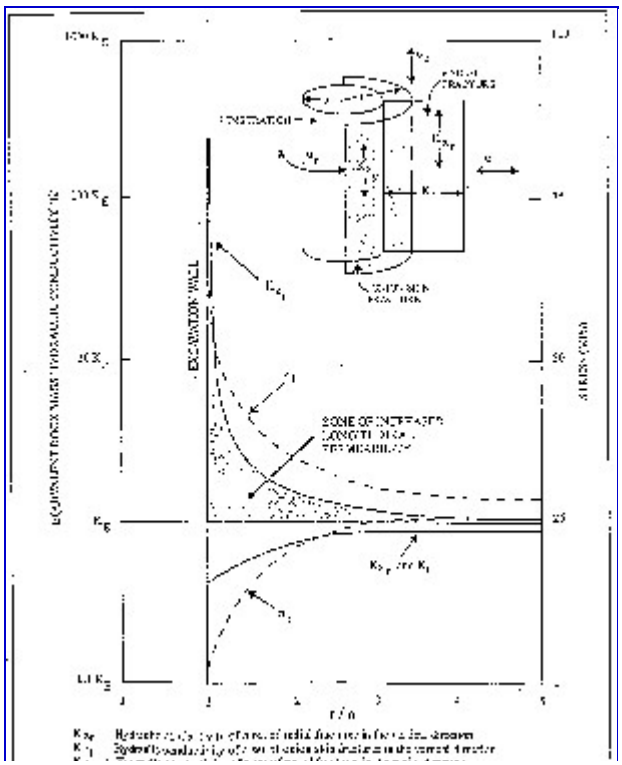
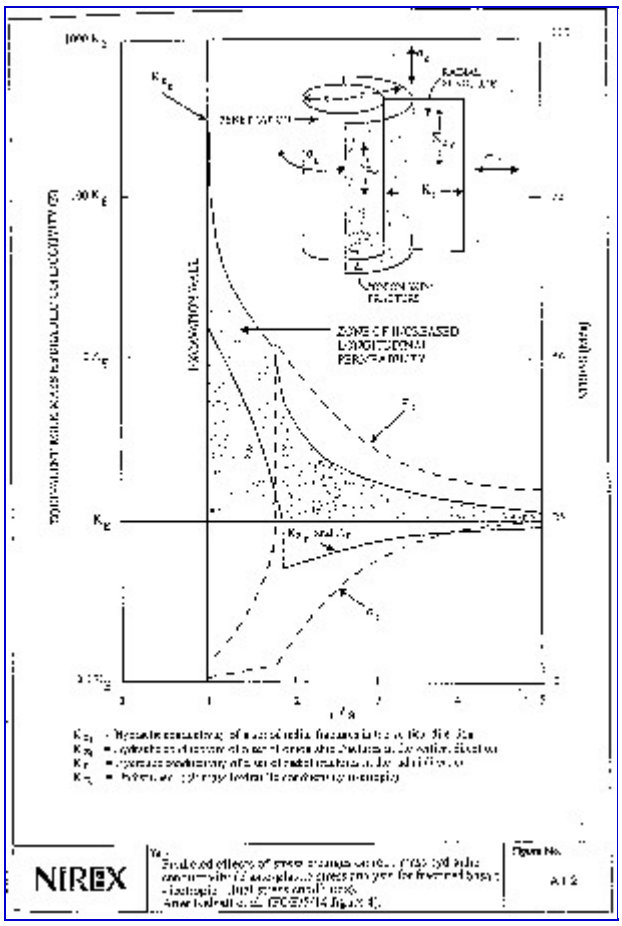


FIGURE A1.2: PREDICTED EFFECTS OF STRESS CHANGESON ROCK MASS HYDRAULIC CONDUCTIVITY (ELASTO-PLASTIC
STRESS ANALYSIS FOR FRACTURED BASALT - ISOTROPIC INITIALSTRESS CONDITIONS). AFTER KELSALL ET AL (FOE/5/14 FIGURE 4)
Click on image to see larger picture



APPENDIX 2

Cautious Blasting Technique Applied to the Construction of the RCF Shafts

Fundamentals of Shaft Blast Design

A2.1 Shaft blasting is a specialised form of blasting, made difficult by the narrow, confined geometry of the shaft excavation. The only naturally available free face toward which material may be displaced by blasting is that provided by the horizontal floor of the shaft. The confinement within the shaft is the most fundamental factor which must be addressed in the blast design.

A2.2 Because of the confinement, shaft blasting is characterised by relatively high specific drilling (defined as the linear drill metres per cubic metre of rock broken) and relatively high blast energy factors (expressed as the quantity of explosive energy employed to break a unit volume or mass of rock, e.g.kg/m^3).

A2.3 Fragmentation requirements of shaft blasting also tend to be greater than those of most other blasting operations because of the relatively small scale of spoil removal equipment able to be employed within the confines of the shaft. High blast energy factors do not however correspond to increased damage to the rock mass. The blast energy factor in itself is not a reliable index of the damage potential from blasting. In a

well designed blast, the dual objectives of fine fragmentation and minimum damage to the rockmass from blasting are mutually compatible.

The Nirex Approach to Constructing the RCF shafts

A2.4 Shaft blasting for the RCF will be performed using full face blasting, utilising blast designs similar to those used in the circular shaft extension of the Canadian Underground Research Laboratory (URL), as stated in my evidence (**PE/NRX/16**, paragraph 6.29). Nirex has used an international blast consultant, whose experience includes the URL project, to review blast damage mechanisms relevant to construction of the RCF shafts, the influence of blast design parameters on damage to the rock mass and to estimate the likely extent and degree of damage to the rock mass from blasting in shaft construction. A range of preliminary blast designs for the different rock types and conditions which will be encountered during shaft sinking has been prepared for Nirex by the blast consultant. A peer review of these blast designs will be completed before shaft sinking starts. By these means, Nirex intends to achieve its objective of keeping blast-induced damage to the rock mass as low as is practicable, as stated in my evidence (**PE/NRX/16**, paragraph 6.28).

Full Face Shaft Blast with Burn Cut

A2.5 Full-face shaft blasting is designed to uniformly advance the complete cross-sectional area of the face in the one blast, as opposed to benching, where alternative sides of the shaft face are progressively advanced ahead of one another. Full-face blasts are more efficient than benching blasts in terms of the advance per round. Mechanisation of the drilling of blast holes also lends itself to full-face blasting through the use of parallel hole drilling within the cut. Importantly, within a given rock type and condition, full-face blasting permits a consistent drill pattern geometry to be implemented, without the need to radically adjust drill-hole angles and collar positions according to the depth of round, as is the case for benching. These alternative options for shaft blasting by full-face and benching methods are illustrated in Figure A2.1.

A2.6 In full face blasting there are two main methods for removing rock by the blast. These are:

i. the wedge cut in which the existing free face (i.e. the floor or sump of the shaft) is used to break a wedge of rock from the near surface of the face; or

ii. the burn cut (also known as a parallel hole cut) in which sufficient void space and free face is created by drilling holes which are closely spaced and not charged with explosive. These so called 'relief holes' enable expansion and displacement of the initial blasted material to break a cylindrical volume of rock along the entire length of the drilled pattern. The initial blast holes which fire into the relief holes are known as 'cut holes'.

These two types of cut are illustrated in Figure A2.2.

A2.7 The depth of drilling, for blast holes in a blast pattern tends to be determined by the cross-sectional area of the shaft, with larger diameter shafts lending themselves to increased drill depths. Accurate drilling is important, initially in the burn-cut where successful breakage is critical to the overall performance of the blast, and also at the shaft perimeter where drill-hole alignment will influence spacing between the holes and the look-out angle. A mechanised drilling cycle is preferred to manual methods of blast hole drilling, because it enables better overall levels of control and accuracy to be imposed on the drilling operation. To achieve a mechanised drilling cycle, the shaft sinking contractor will be required, as part of the contract, to undertake drilling of blast holes using a specialised piece of equipment known as a shaft drill jumbo.

Design of the Burn Cut Drill Pattern for the RCF Shafts

A2.8 The full-face blast design is characterised by a series of concentric rings of drilled blastholes, with a burn-cut located within the central part of the shaft, although typically slightly offset, as shown in Figure A2.3. This offset arrangement allows the position of the cut and relief holes to be moved slightly between successive rounds to avoid drilling into possible hole butts in the cut from the previous round (i.e. for safety reasons). All blast holes are drilled vertically, except the perimeter holes which feature a slight look-out in order to accommodate the mast of the drill jumbo and enable the next round of holes to be collared in the correct position, thereby maintaining the designed shaft diameter.

A2.9 The overall design of the drill pattern may be considered in terms of the shaft geometry, with the shaft divided into three basic blast areas, as shown in Figure A2.3. These blast areas reflect the variable conditions of confinement (or available free face) within the blast, as the blast proceeds:

i. *Cut zone*, consisting of both charged blastholes and empty relief holes, immediately surrounded by the easer holes. The cut is the initial point of the blast. The function of the easer holes is to expand the initial void created by the cut.

ii. *Production holes* within the body of the blast, so-called because their main function is to achieve the most efficient breakage and advance within the round, making use of the void previously provided by the cut and subsequently developed by the easer holes.

iii. *Contour holes*, which include the outer two rings of holes (perimeter and cushion rings, as shown in Figure A2.3). The dual objectives of these outermost rings of holes is to achieve fragmentation within the blast zone whilst minimising damage to the surrounding rock mass.

A2.10 The depth of drilling may be adjusted in accordance with the shaft diameter and the rock mass conditions. As in all blasting, holes must be drilled slightly deeper than the planned depth of excavation (pull), since the advance achieved in a finely tuned shaft blast typically ranges between 90 and 95% of the drilled depth. For the RCF shafts, a drill depth of around 2.75 metres is necessary to achieve the designed advance of 2.5 metres, with the empty relief holes drilled 0.25 metres deeper that the rest of the round to provide greater relief for the cut to break into.

The Burn Cut

A2.11 The purpose of the cut is to create an initial opening within the rock to act as a free face so that subsequent production holes are able to slash into the void. The cut holes are the first holes in the round to detonate. Because of the initial high confinement, the burn-cut features the highest concentration of drill-holes and explosive charge of the pattern. The geometry of the cut holes and adjacent easer holes are such that the detonation sequence progressively develops increased void and increased free face until the production blastholes are able to break efficiently.

A2.12 The geometry of the burn-cut is ultimately critical to its success. The success of the cut relies very much on accuracy of drilling and the incorporation of sufficient void space (i.e. uncharged relief holes) into which rock fragments may be ejected during the blast. The tolerances on relief hole and cut hole numbers, diameters, positions and verticality are therefore very important. These tolerances will form an important part of the blast designs used in sinking the RCF, and the tolerances will be contractually enforced by monitoring and recording every drilled pattern prior to firing.

Detonation Sequence

A2.13 A controlled sequential detonation sequence is used in the blast. With reference to Figure A2.3 the sequence of firing is:

i. the cut holes are fired first. These are fired individually in sequence, with typically 100 millisecond delays;

ii. the easer holes of the expanded cut are then fired individually in sequence;

iii. next the inner ring of production holes is detonated;

iv. the outer ring of production holes is then detonated; and

v. finally the cushion holes and then the perimeter holes are detonated.

A2.14 Because total shaft blast duration may extend upwards of 10 seconds, the influence of delay scatter (inherent manufacturing error in the timing of pyrotechnic delay firing elements) must be allowed for in design to prevent out of sequence detonations.

Cautious Blasting Technique and Design of Blast Hole Charge

A2.15 The implementation of design measures in the RCF shaft blast pattern and detonation sequence to minimise damage to the rock mass is termed the 'cautious blasting' technique or method. The method may

also properly be described as one of smooth blasting, in that the blasthole initiation commences within the centrally located cut and progressively moves outwards with the perimeter holes firing last. Holes essentially fire ring-by-ring, although not all holes within a ring fire simultaneously.

A2.16 The cautious blasting method relies on the following elements for minimisation of damage to the rock mass:

i. Reduced mass of explosive in the blastholes located against the designed excavation boundary (perimeter holes), and also in the penultimate ring of holes (cushion holes).

ii. Reduced blasthole pressure toward the perimeter of the blast zone, using decoupled blastholes charges, to reduce the potential for shattering/fracturing of rock material. The term decoupled means that the diameter of the explosive cartridge is very much smaller than the diameter of the blast hole.

iii. Large numbers of close spaced drill-holes for the perimeter and cushion rings to compensate for the reduced breakage capability of the decoupled blasthole charges.

iv. Timing of blasthole initiation within the perimeter row to enhance interaction between adjacent blastholes.

A2.17 Detonation sequences begin near the centre of the pattern, and end at the outermost ring of blastholes. However, the process of designing the blast pattern deals first with the outermost ring of blastholes and proceeds inwards. The design process begins with explosive charge calculations for the perimeter holes to control and minimise the extent of damage to the rock mass. Equally important, all subsequent blasthole charge calculations in the pattern ensure that the damage zone from any individual blasthole will not exceed the damage zone which will be created by the perimeter hole. This is the design basis for the cautious blasting technique, and is illustrated schematically in Figure A2.4.

A2.18 The charge for the perimeter holes is calculated to achieve the practicable limit to rock mass damage at the surface of the excavation. The practicable minimum limit of damage from blasting is provided when the blasthole pressure in the perimeter holes is constrained to a value less than the (dynamic) crushing strength of the rock mass. This demands a greatly reduced blasthole pressure through the application of a greatly reduced linear charge concentration in the blasthole and charge decoupling. The blasthole pressure is a calculable parameter, and it is related to the performance characteristics of the explosive, primarily the velocity of detonation.

A2.19 In practical terms, the most visible and commonly applied measure of rock mass damage minimisation is an assessment of the amount of overbreak, particularly the percentage of remnant perimeter holes (called half-holes or half-barrels) which remain standing in the excavation surface after the blast. The retention of half-barrels in the exposed shaft wall is an indication that blasthole pressures are adequately contained below the dynamic compressive strength of the rock. It should not, however, be assumed that an excavation boundary (e.g. shaft wall) which displays a large proportion of half holes represents the effective limit of blast damage. Some degree of damage inevitably extends beyond the boundary irrespective of the retention of half-barrels. The objective of achieving half hole finish does, however, impose practical constraints on blast hole charging practice which is beneficial to the minimisation of damage to the rock mass. An example of a clean excavation surface with a high percentage of half barrels is shown in Figure A2.5. This is a photograph of a gallery excavated by the drill and blast method at the Canadian URL.

A2.20 Having designed a charge for the perimeter ring of holes which keeps damage to a minimum, the next step is to design an appropriate charge and drill-hole distances for the penultimate, or cushion, ring of holes such that damage to the rock mass from firing the cushion holes does not extend beyond that incurred by firing the perimeter holes. The extent of damage to the rock mass from the cushion holes is estimable in terms of the levels of vibration which will be caused by the explosive. Very close to the blasthole the vibration levels will be sufficiently high to induce fracturing in the rock. In simple terms this amounts to disruption and fracturing of the rock by shock waves, a point made by Mr Walker in cross examination of Mr Smith on Day 2 of the Inquiry. Mathematical expressions can be used to relate the peak particle vibrational velocity to the strain (deformation) of the rock mass around an individual blasthole. By these means it is common practice in designing blast patterns to calculate the maximum particle velocity (PPV_{max}) which can be withstood by the rock before disruption and fracturing occurs. This allows the

linear charge concentration of explosives in a particular blasthole to be designed such that shock-induced blast damage to the rock mass around that blasthole is controlled and confined to a known distance. For the cushion holes, this distance should be such as to not induce additional damage to the rock mass which has been calculated to be caused by the perimeter holes.

A2.21 The production hole, easer hole and cut hole linear charge concentrations of explosive are similarly calculated to produce shock-induced blast damage which does not extend into the rock mass to a greater extent than that the rock mass to a greater extent than that which has been calculated for the perimeter holes.

A2.22 The accuracy of the final excavation shape, and the quality of the surface finish, is also dependant on the accuracy of drilling of the perimeter holes. The tolerance allowable on the perimeter holes will be specified as part of the shaft sinking contract. A typical value for such a tolerance in drilling the perimeter blast holes would be an allowable angular deviation of up to two degrees on specified look-out angle.

FIGURE A2.1: ALTERNATIVE SHAFT BLAST OPTIONS, OF FULL-FACE AND BENCHING
Click on image to see larger picture

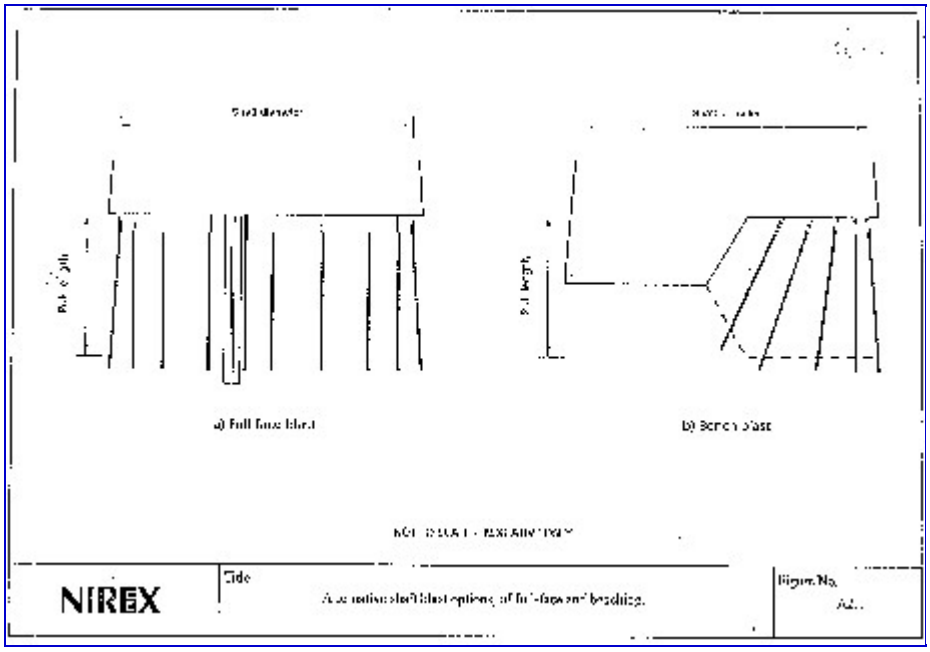


FIGURE A2.2: PRINCIPLE OF BURN-CUT AND WEDGE-CUT DESIGNS IN FULL-FACE SHAFT BLASTING
Click on image to see larger picture

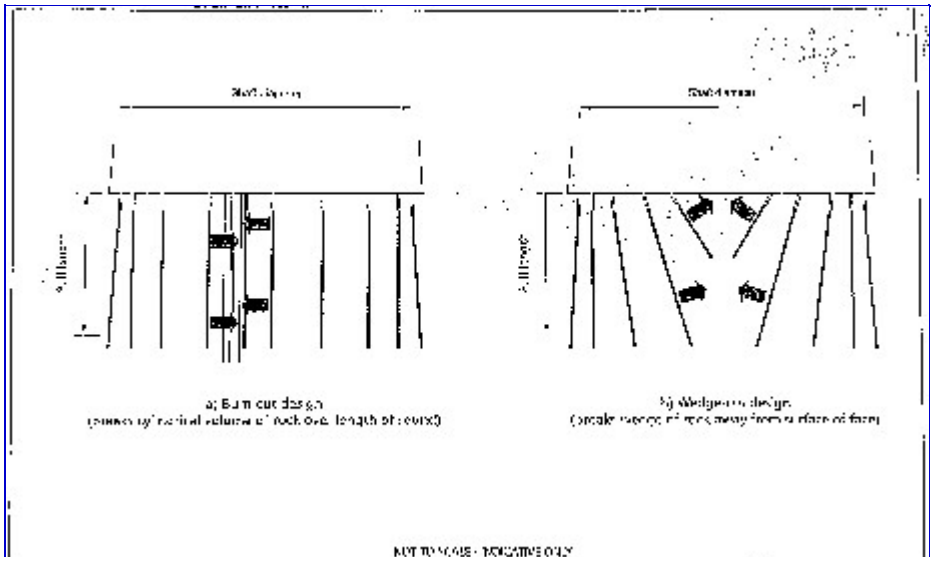


FIGURE A2.3: SCHEMATIC PLAN VIEW ILLUSTRATING A SHAFT BLAST PATTERN
Click on image to see larger picture

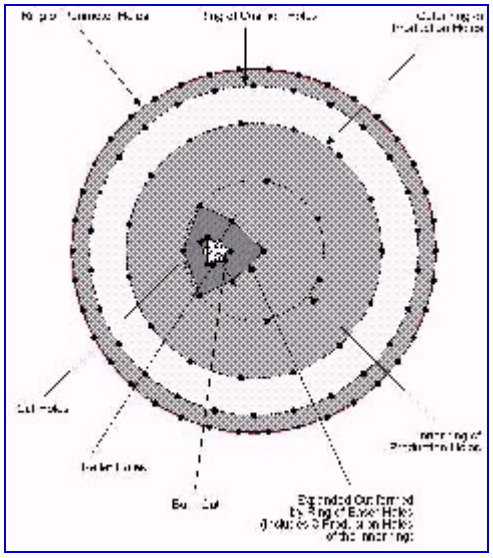


FIGURE A2.4: CAUTIOUS BLASTING TECHNIQUE, IN CONCEPT, SHOWING EXTENT OF DAMAGE PRODUCED BY A PERIMETER HOLE, CUSHION HOLE AND PRODUCTION HOLE
Click on image to see larger picture

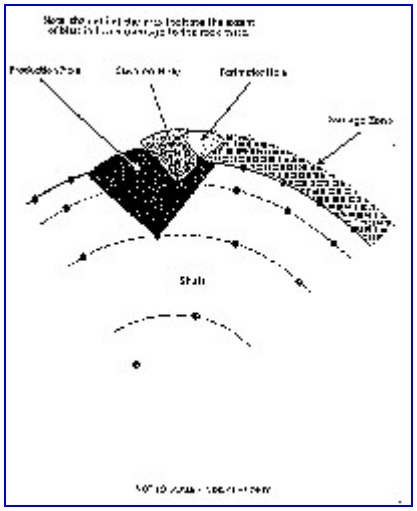


FIGURE A2.5: PHOTOGRAPH OF TUNNEL CREATED IN THE UNDERGROUND ROCK LABORATORY IN CANADA USING THE DRILL AND BLAST METHOD
Click on image to see larger picture

APPENDIX 3

Description of Measurement Techniques Used to Characterise the Extent of Plastic Deformation/Blast Induced Damage Around the TBM and Drill and Blast Drifts of the ZEDEX Experiment at Äspö.

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A3.1 This appendix provides a preliminary, and outline summary description of techniques used to estimate damage to the rock mass associated with the TBM tunnel and the drill and blast (drill and blast) drift of the ZEDEX project. All the techniques used were conducted using specialised instruments in boreholes drilled into or from the drifts.

Borehole Resonance Measurements

A3.2 These were conducted in 4 boreholes in the TBM drift and 17 boreholes in the drill and blast drift using the Borehole Resonance Tool (BORET) of BGR which measures in situ seismic impedance at the borehole wall. Changes in the material properties are identifiable as decreased (or increased) levels in impedance function magnitude and a sudden jump in impedance function phase. The measurement interval in the borehole is variable, ranging (in the case of ZEDEX) from 2 centimetres to 10 centimetres.

High Resolution Seismic Down-Hole Measurements

A3.3 These were conducted in 4 boreholes in the TBM drift and 17 boreholes in the drill and blast using BGR equipment.

A3.4 For measurements a seismic acceleration transducer is used. The probe is clamped to the borehole wall by 2 pneumatic cylinders. The seismic signals are generated by a pendulum hammer, bolted to the wall or floor of the drift. The hammer carries a further accelerometer for measuring the exact impact time on the wall to trigger the recording of the accelerograms. The accelerometer has a built in amplifier which has an external power source. The signals are recorded by a signal analyser in time domain. Travel time and frequency content are analysed. Measurements were taken every 50 millimetres between 0.05 and 1.0 metre, and every 100 millimetres between 1 and 2 metres.

High Resolution Permeability Measurements

A3.5 These were performed in 3 TBM boreholes and 9 drill and blast boreholes by Laboratoire de Geomecanique (ENSG) using the SEPPI (System Experimental de mesures de Permeabilite in situ par Pulse) probe designed to measure permeability values as a function of depth from the excavation wall. The probe contains:

- a small injection chamber (50 millimetres);
- 2 mechanical packers;
- 2 chambers of leakage control connected to graduated capillary tubes of different diameters to estimate volumetric flow rate; and
- 2 standard packers which ensure watertightness.

A3.6 The tool allows the rock to be saturated before measurements are taken. Tests are made as pulse tests conducted by increasing the pressure in the middle chambers once isolated. Pressure versus time is automatically recorded.

A3.7 Measurements are taken every 50 millimetres between 0.1 metres and 0.5 metres, every 100 millimetres between 0.5 metres and 1 metre, and every 200 millimetre between 1 metre and 2 metres.

Laboratory Investigations on Core Samples Form 3 Metre Radial Boreholes

A3.8 The purpose of this study is to quantify the microcracking induced by excavation in the near field (plastic zone) as a function of distance from the drift wall.

A3.9 Methods used in the laboratory on core samples to help characterise microcracking including:

- ultrasonic wave velocities;
- isotropic compression tests;
- permeability measurements on oriented cubic samples;
- crack porosity and free porosity; and
- index of birefringence.

APPENDIX 4

Bored Shafts Completed with the Wirth 'V' Mole. (Data kindly supplied for Nirex's use by Wirth Maschinen und Bohrgeröte Fabrik GmbH)

No	Project Company	Year	Machine	Depth m	Dia. m	Rock formation
1	Emil Mayrisch, FRG Thyssen Schachtbau Deilmann Haniel	1971	GSB 450/500	231	4.50	Shale Sandstone
2	Walsum, FRG Thyssen Schachtbau Deilmann Haniel	1972	GSB 450/500	243	5.00	Shale Sandstone
3	Zollverein, FRG Thyssen Schachtbau Deilmann Haniel	1973	GSB 450/500	227	500	Shale Sandstone
4	Sterkrade, FRG Thyssen Schachtbau Deilmann Haniel	1974	GSB 450/500	196	4.90	Shale Sandstone
5	Carl Alexander, FRG Thyssen Schachtbau Deilmann Haniel	1974/5	GSB 450/500	228	5.00	Shale Sandstone

6	Minister Stein, FRG Thyssen Schachtbau Deilmann Haniel	1975	GSB 450/500	302	5.00	Shale Sandstone
7	Emil Mayrisch, FRG Thyssen Schachtbau Deilmann Haniel	1976	GSB 450/500	226	5.00	Shale Sandstone
8	Rossenray, FRG Thyssen Schachtbau Deilmann Haniel	1976/7	GSB 450/500	224	5.00	Shale Sandstone
9	Emil Mayrisch, FRG Thyssen Schachtbau Deilmann Haniel	1977	GSB 450/500	227	5.00	Shale Sandstone
10	Ibbenbuhren, FRG Thyssen Schachtbau Deilmann Haniel	1977	SB VI 500/650	463	6.00	Shale Sandstone
11	Gneisenau Thyssen Schachtbau Deilmann Haniel	1978	GSB 450/500	204	5.10	Shale Sandstone
12	Gottelborn Thyssen Schachtbau Deilmann Haniel	1978	SB VI 500/650	430	6.50	Coal Shale Sandstone Conglomerate
13	Viktoria, FRG Thyssen Schachtbau Deilmann Haniel	1978	SB VI 500/650	106	5.00	Coal Shale Sandstone
14	Walsum, FRG Thyssen Schachtbau Deilmann Haniel	1979	SB VI 500/650	230	5.00	Shale
15	Erin, FRG Thyssen Schachtbau Deilmann Haniel	1979	SB VI 500/650	170	6.00	Sandstone Brick-lining
16	Prosper IV, FRG Thyssen Schachtbau Deilmann Haniel	1979	SB VI 500/650	124	6.00	Shale Sandstone
17	Prosper IV, FRG Thyssen Schachtbau Deilmann Haniel	1980	SB VI 500/650	345	5.50	Shale Sandstone
18	Luisenthal I, FRG Thyssen Schachtbau Deilmann Haniel Saarberg Interplan	1980	SB VII 650/850	262	7.00	Shale Sandstone
19	Luisenthal II, FRG Thyssen Schachtbau Deilmann Haniel	1981	SB VII 650/850	337	7.20	Shale Sandstone Conglomerate
20	Gneisenau, Karl III, FRG Thyssen Schachtbau Deilmann Haniel	1981	SB VI 500/650	283	6.50	Shale Sandstone
21	Alabama I, USA Thyssen Mining Construction	1981	SB VII 650/850 E/ Sch	472	7.00	Sandstone

22	Prosper 61 WN 2 Thyssen Schachtbau Deilmann Haniel	1981/2	SB VI 500/650 E	340	5.50	Shale Sandstone
23	Alabama II, USA Thyssen Mining Construction	1982	SB VII 650/850 E/ Sch	603	7.00	Sandstone
24	Alabama III, USA Thyssen Mining Construction	1982	SB VII 650/850 E/ Sch	575	7.00	Sandstone
25	Monopol, FRG Thyssen Schachtbau Deilmann Haniel	1982	SB VI 500/650 E	290	6.80	Shale Sandstone
26	Luisenthal III, FRG Thyssen Schachtbau Deilmann Haniel Saarberg Interplan	1982	SB VII 650/850	170	7.50	Shale Sandstone
27	Alabama IV, USA Thyssen Mining Construction	1983	SB VII 650/850 E/ Sch	588	7.00	Sandstone
28	Rheinpreussen, FRG Thyssen Schachtbau Deilmann Haniel Saarberg Interplan	1982	SB VII 650/850	200	6.50	Shale Sandstone
29	Heinrich Robert, FRB Thyssen Schachtbau Deilmann Haniel	1983	VSB VI 580/750 E/ Sch	170	5.80	Shale Sandstone
30	Friedrich Heinrich, FRG Thyssen Schachtbau Deilmann Haniel	1984	VSB VI 580/750 E/ Sch	320	7.00	Shale Sandstone
31	Neu Monopol, FRG Thyssen Schachtbau Deilmann Haniel	1984/5	SB VI 500/650	240	6.80	Shale Sandstone
32	Prosper Haniel, FRG Thyssen Schachtbau Deilmann Haniel	1984/5	SB VI 500/650	250	6.00	Shale Sandstone
33	Lohberg, FRG Thyssen Schachtbau Deilmann Haniel	1985	VSB VI 580/750 E/ Sch	451	6.00	Shale Sandstone
34	Lummerschied, FRG Thyssen Schachtbau Deilmann Haniel Saarberg Interplan	1985	SB VII 650/850	733	8.20	Shale Sandstone
35	Hlubina, CSFR VOKD	1985	VSB VI 580/750 E/ Sch	180	5.80	Shale Sandstone
36	Prosper Haniel, FRG Thyssen Schachtbau Deilmann Haniel	1985	SB VI 500/650	251	6.80	Shale Sandstone
37	Friedrich Heinrich, FRG Thyssen Schachtbau Deilmann Haniel	1985/6	VSB VI 580/750 E/ Sch	380	6.50	Shale Sandstone

38	Dukla, CSFR VOKD	1986	VSB VI 580/750 E/ Sch	348	6.50	Sandstone
39	Vitzny, CSFR VOKD	1987/8	VSB VI 580/750 E/ Sch	620	6.50	Shale Sandstone
40	Niederberg BS 0242, FRG Thyssen Schachtbau Deilmann Haniel	1987	SB VI 500/650	150	5.00	Shale Sandstone
41	Sophia Jacoba, FRG Thyssen Schachtbau Deilmann Haniel	1987/8	SB VII 650/850	420	6.00	Shale Sandstone
42	Prosper Haniel, FRG Thyssen Schachtbau Deilmann Haniel Saarberg Interplan	1988	SB VII 650/850	287	8.20	Shale Sandstone
43	Lohberg, FRG Thyssen Schachtbau Deilmann Haniel	1988	VSB VI 580/750	330	6.00	Shale Sandstone
44	Monopol, FRG Thyssen Schachtbau Deilmann Haniel	1988/9	SB VI	240	6.50	Shale Sandstone
45	ORYX MINE South Africa RUC Thyssen Schachtbau	1989	SB VII	1000	6.50	Shale Quartz
46	Bezruc, CSFR VOKD	1989/0	VSB VI 580/650 E/ Sch	365	6.50	Shale Sandstone
47	Australia RUC Thyssen Schachtbau	1990	SB VII	810	8.00	Granite Amphibolite
48	Hoover Dam, elevator shaft USA Frontier Kemper (Deilmann Haniel)	1991	VSB VI	140	7.00	Volcanics
49	Niederberg FRG Thyssen Schachtbau	1993	SB VII	280	7.00	Shale
50	Lohberg FRG Thyssen Schachtbau	1994/5	SB VI	220	6.00	Coal

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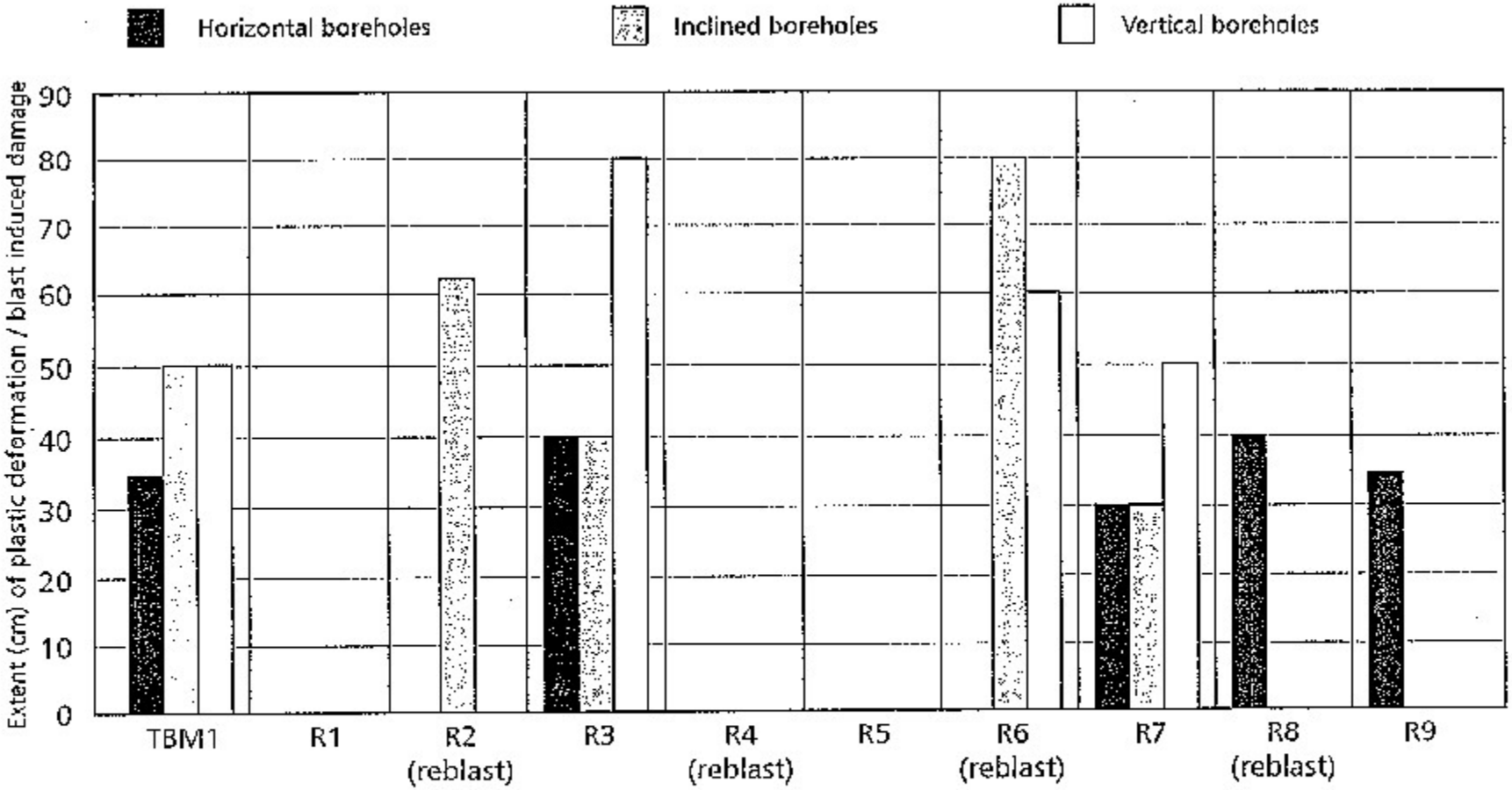
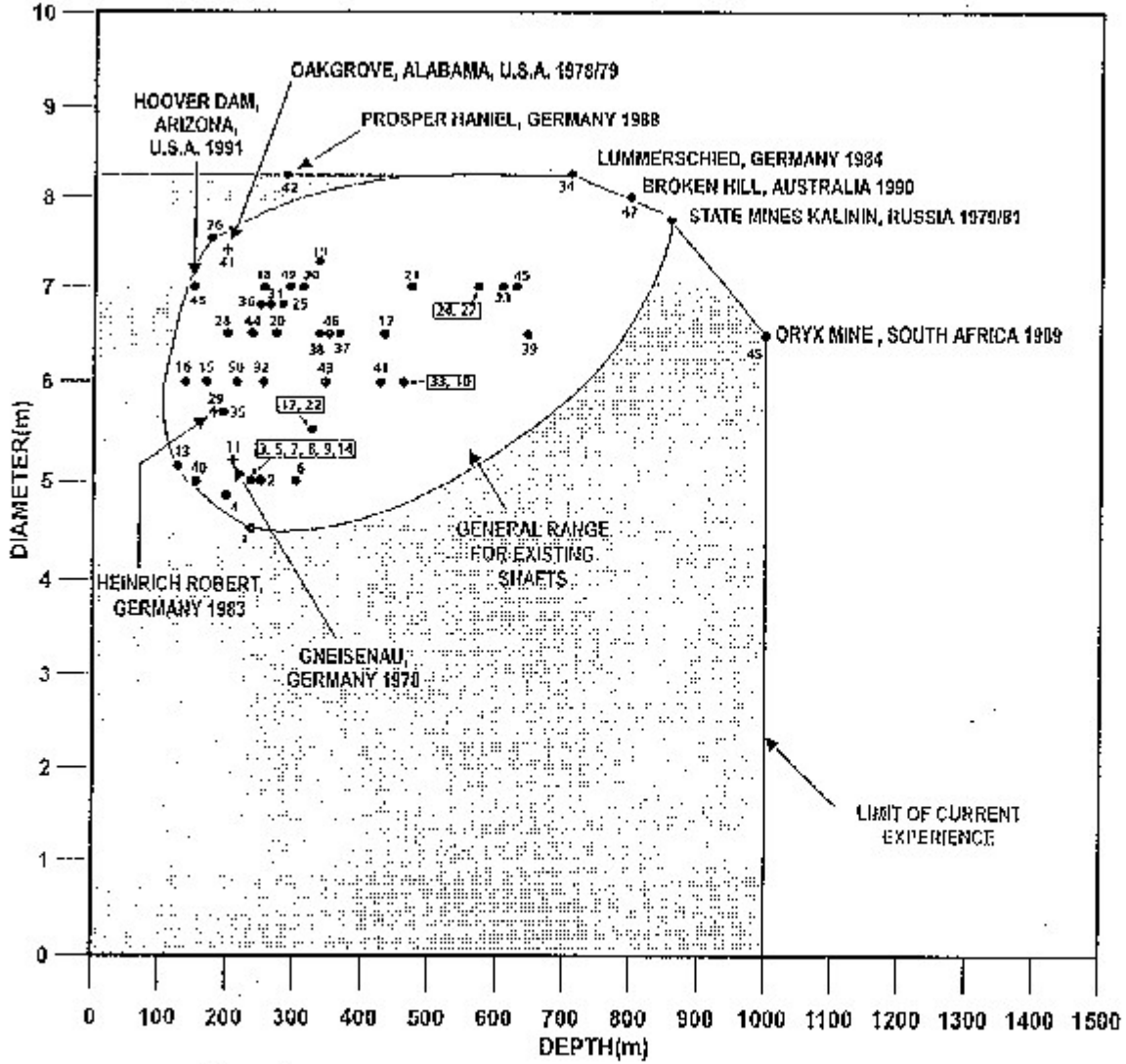


NIREX	Title ZEDEX - Preliminary results of measurements of plastic deformation / blast induced damage from Tunnel Boring Machine (TBM) drift and Drill-and-Blast drift. "R" denotes blast round number.	Figure No. 3.1

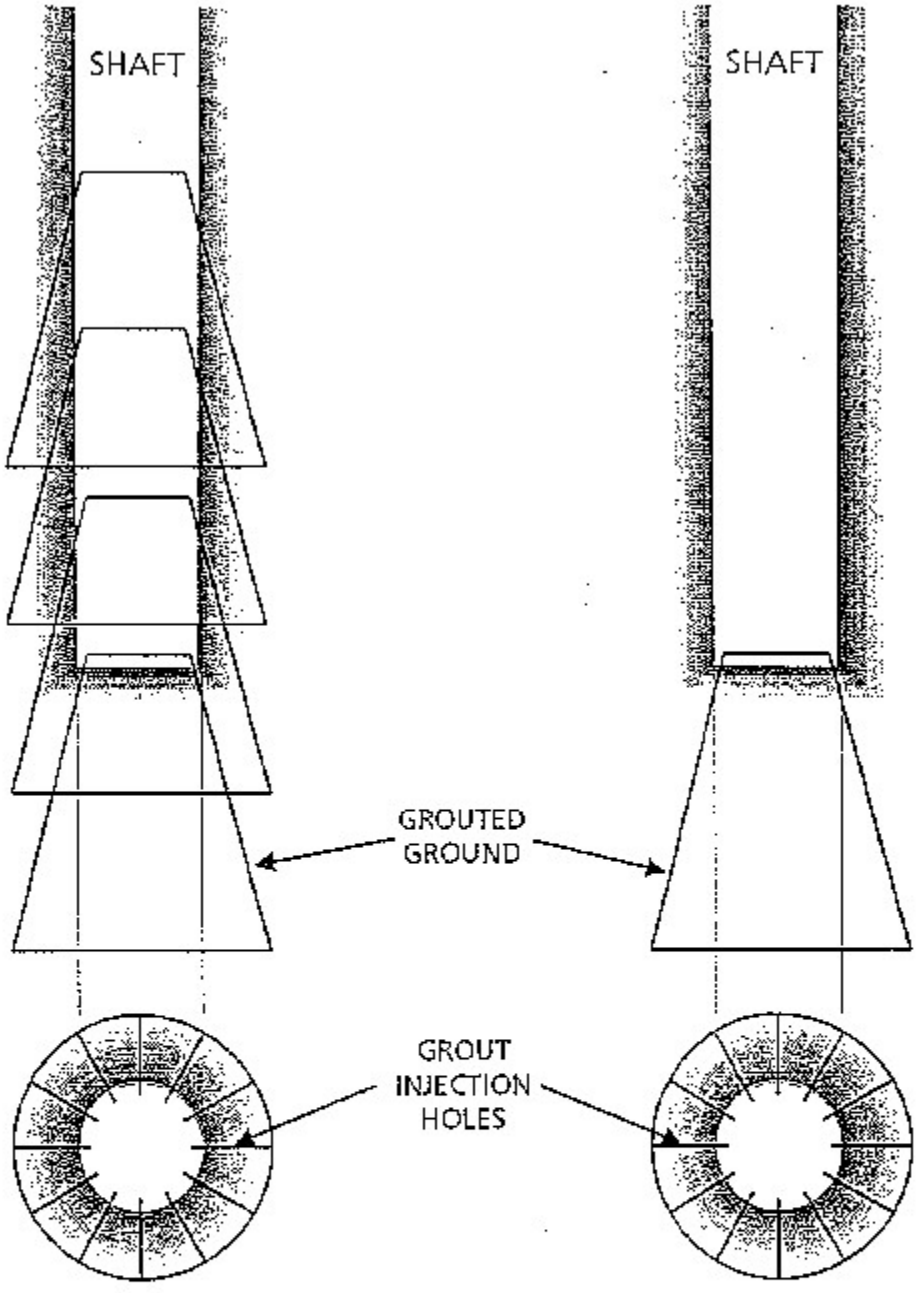


NIREX

Title

Diameter vs shaft depth for existing bored shafts (updated 1995)
(shafts less than 2m in diameter or 100m in depth not included)

Figure No.
3.2



Sketch illustrating multiple grout covers

Sketch illustrating single grout cover

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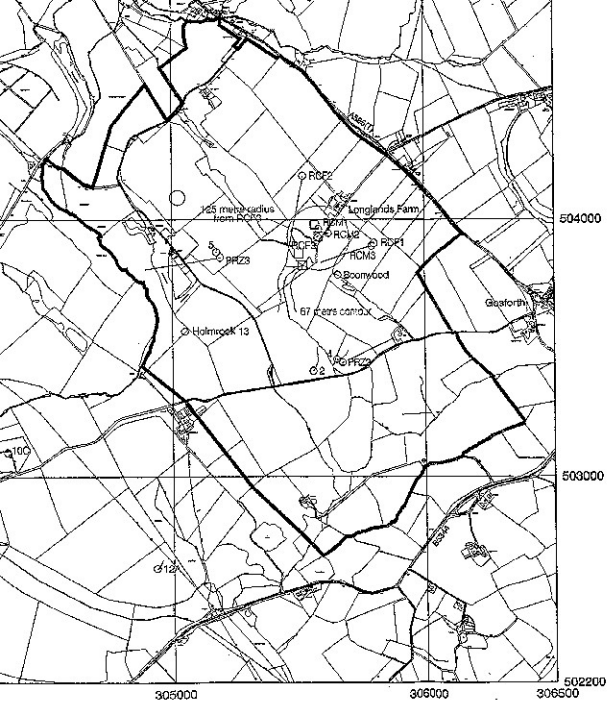
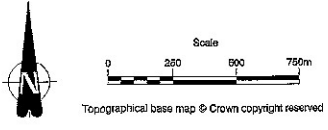
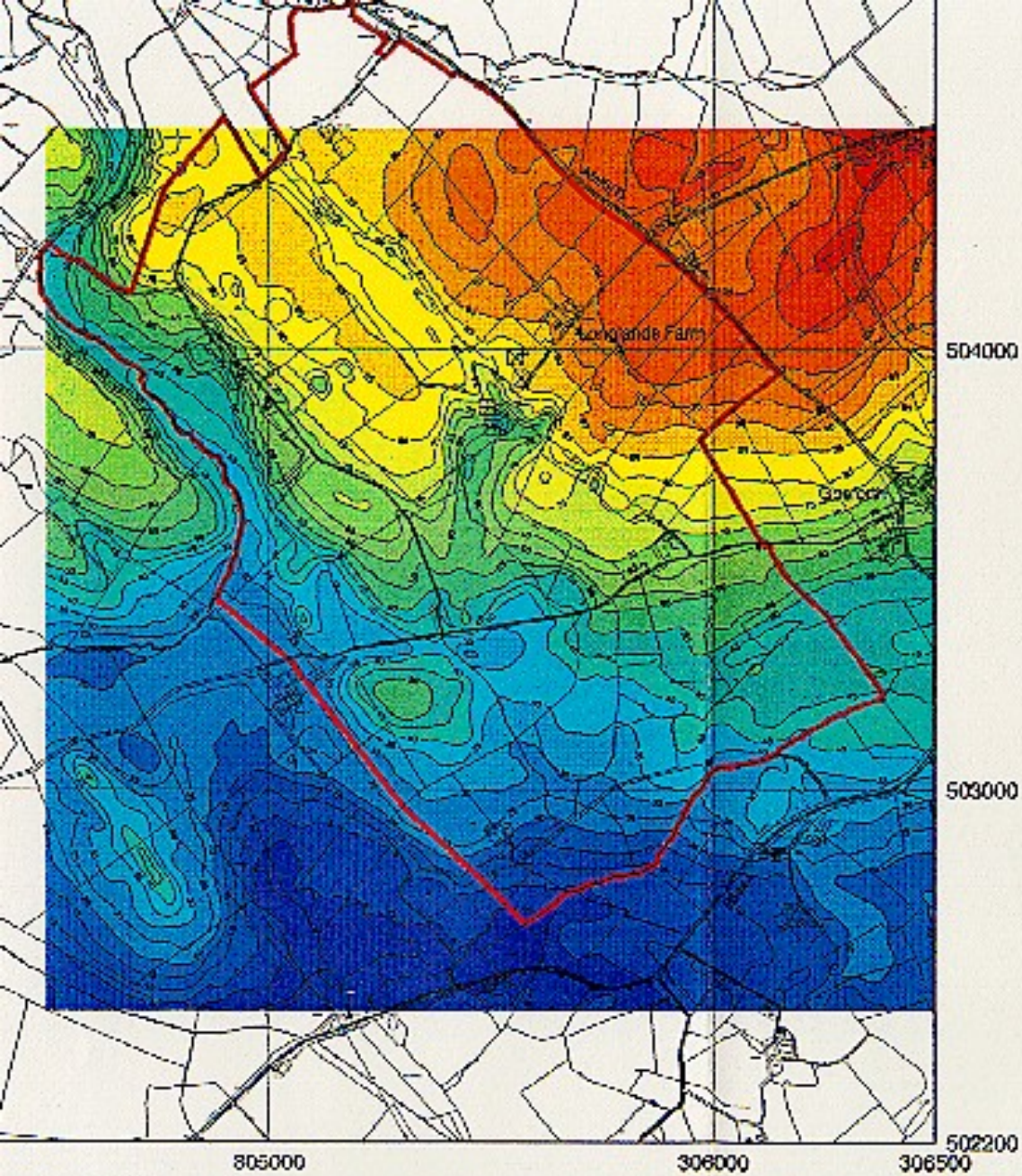
NIREX	Title Sketches illustrating vertical section through shaft with single and multiple grout covers.	Figure No. 3.3



KEY

- Surface borehole location
- Surface shaft location
- Mr Spendlove's alternative surface shaft location
- Sub-surface trajectory of borehole
- Potential Repository Zone





Mr Spendlove's alternative surface shaft location

Potential Repository Zone

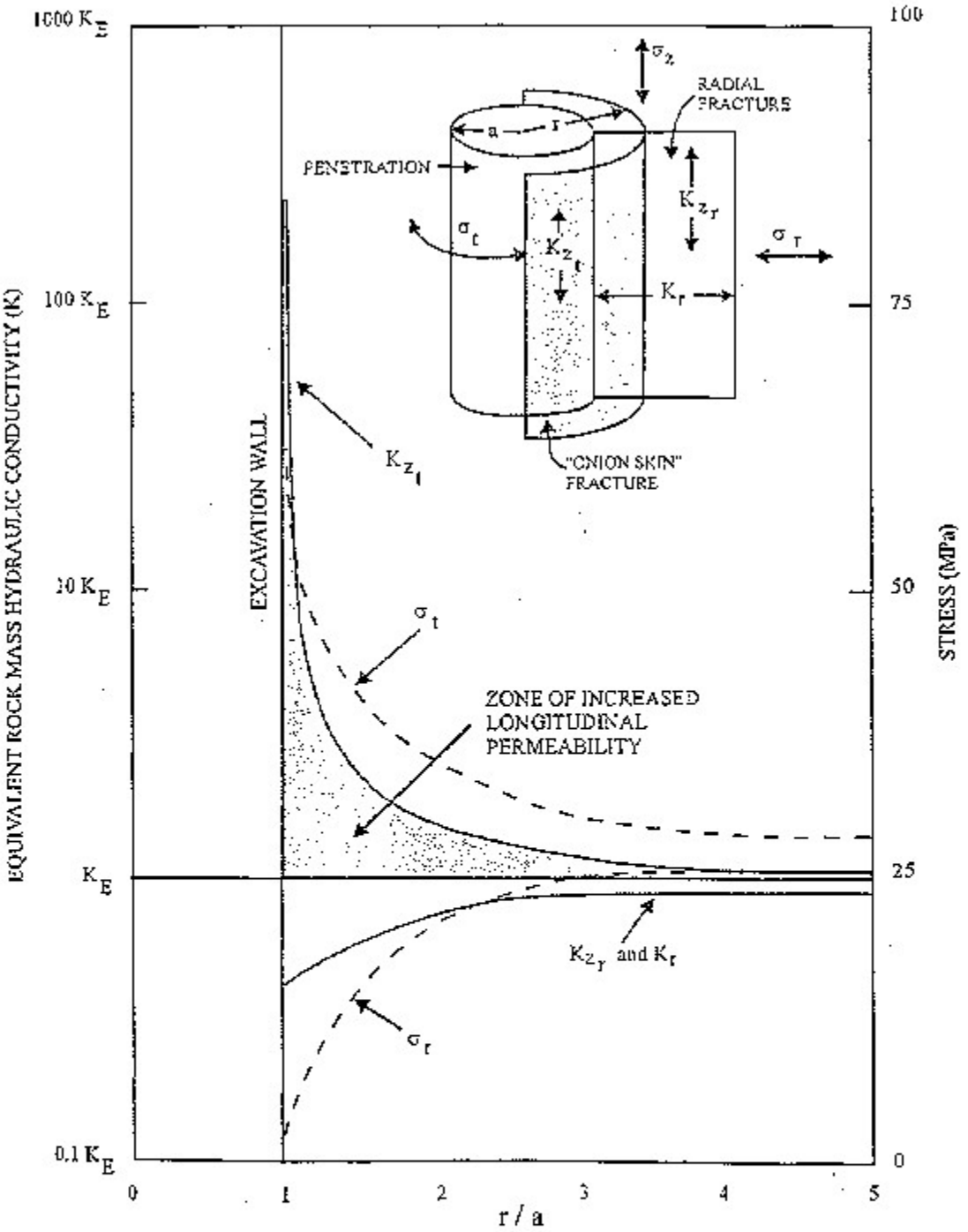
Heights in metres (relative to OD)

115
110
105
100
95
90
85
80
75
70
65
60
55
50
45
40
35
30
25
20
15
10
5
0

N

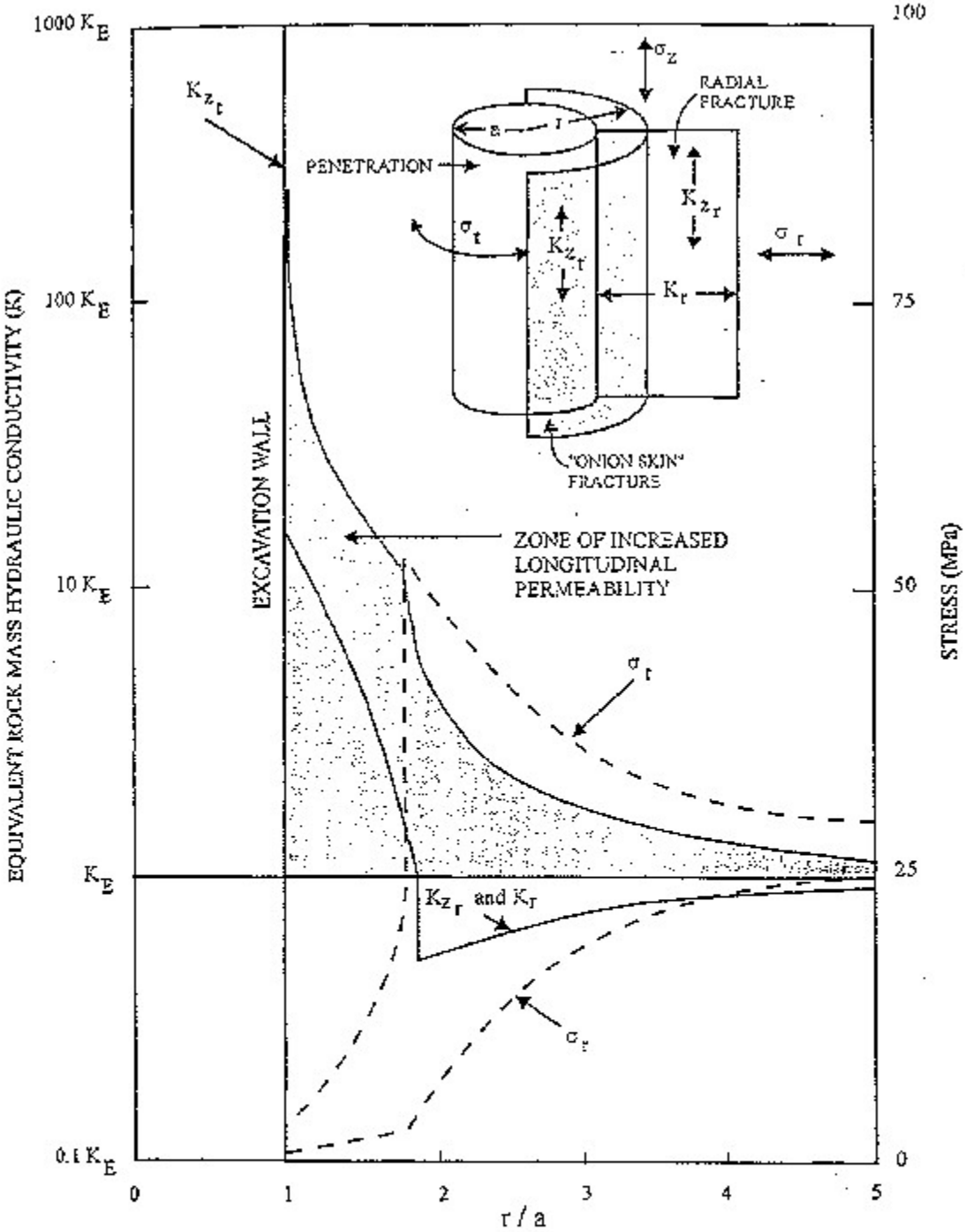
Scale

0 250 500 750m



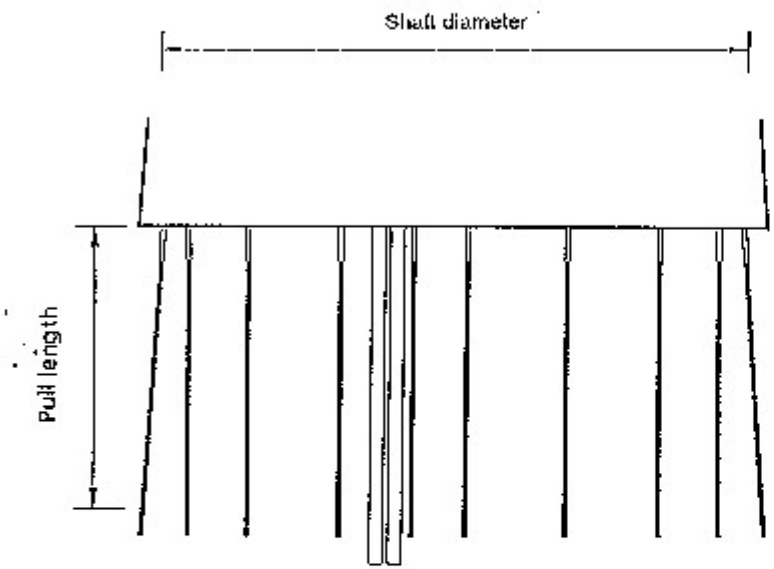
K_{zr} = Hydraulic conductivity of a set of radial fractures in the vertical direction
K_{zt} = Hydraulic conductivity of a set of onion skin fractures in the vertical direction
K_r = Hydraulic conductivity of a set of radial fractures in the radial direction
K_E = Undisturbed rock mass hydraulic conductivity (isotropic)

NIREX	Title: Predicted effects of stress changes on rock mass hydraulic conductivity (elastic stress analysis for fractured basalt - isotropic initial stress conditions). After Kelsall et al. (FOE/5/14 figure 3).	Figure No. A1.1

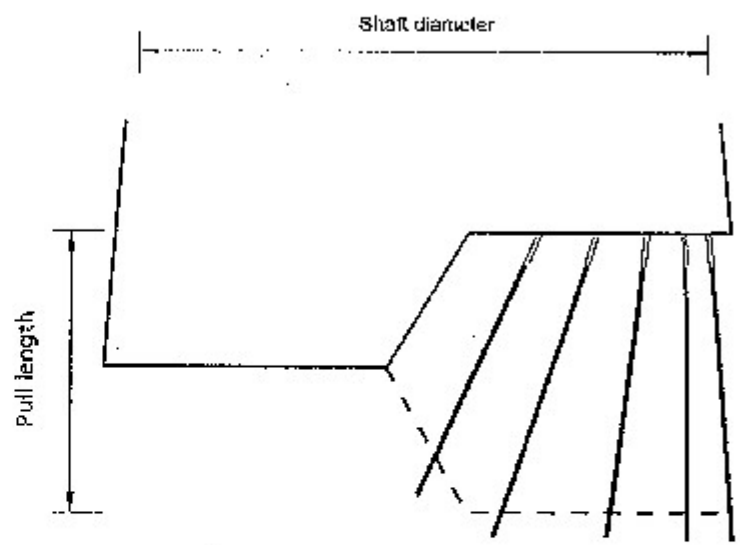


K_{z_r} = Hydraulic conductivity of a set of radial fractures in the vertical direction
K_{z_t} = Hydraulic conductivity of a set of onion skin fractures in the vertical direction
K_r = Hydraulic conductivity of a set of radial fractures in the radial direction
K_E = Undisturbed rock mass hydraulic conductivity (isotropic)

NIREX	Title Predicted effects of stress changes on rock mass hydraulic conductivity (elasto-plastic stress analysis for fractured basalt - isotropic initial stress conditions). After Kelsall et al. (FOE/5/14 figure 4).	Figure No. A1.2



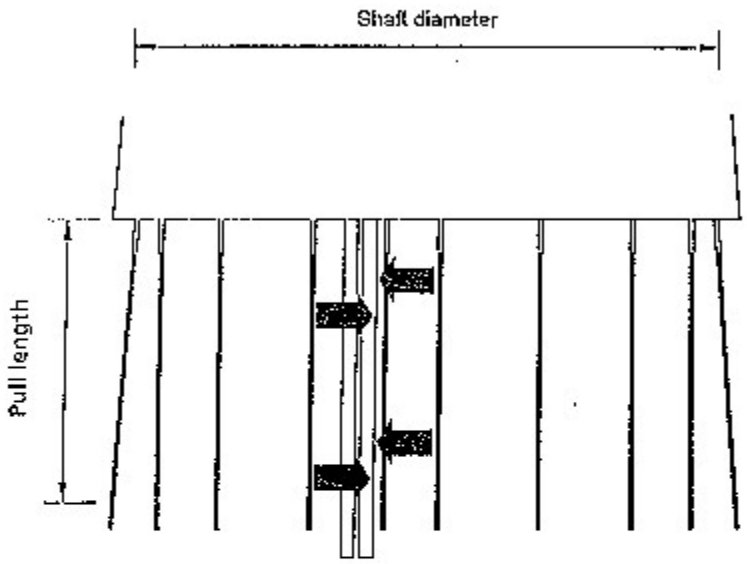
a) Full-face blast



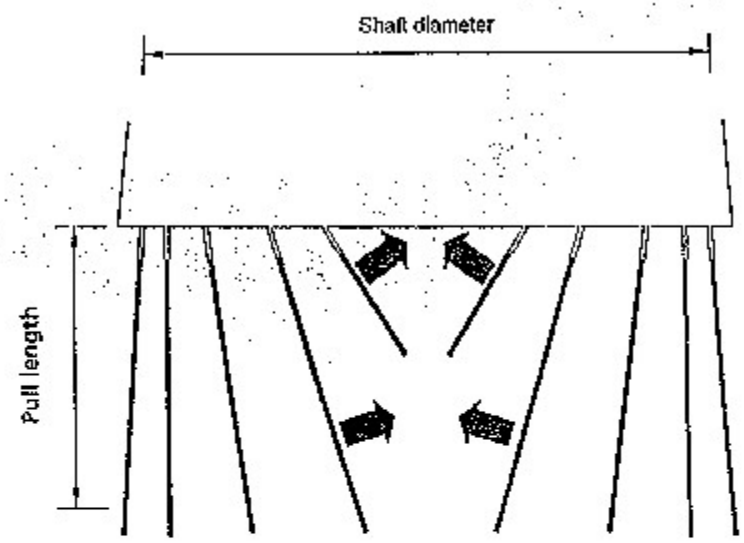
b) Bench blast

NOT TO SCALE - INDICATIVE ONLY

NIREX	Title Alternative shaft blast options, of full-face and benching.	Figure No. A2.1



a) Burn-cut design
(breaks cylindrical volume of rock over length of round)



b) Wedge-cut design
(breaks wedge of rock away from surface of face)

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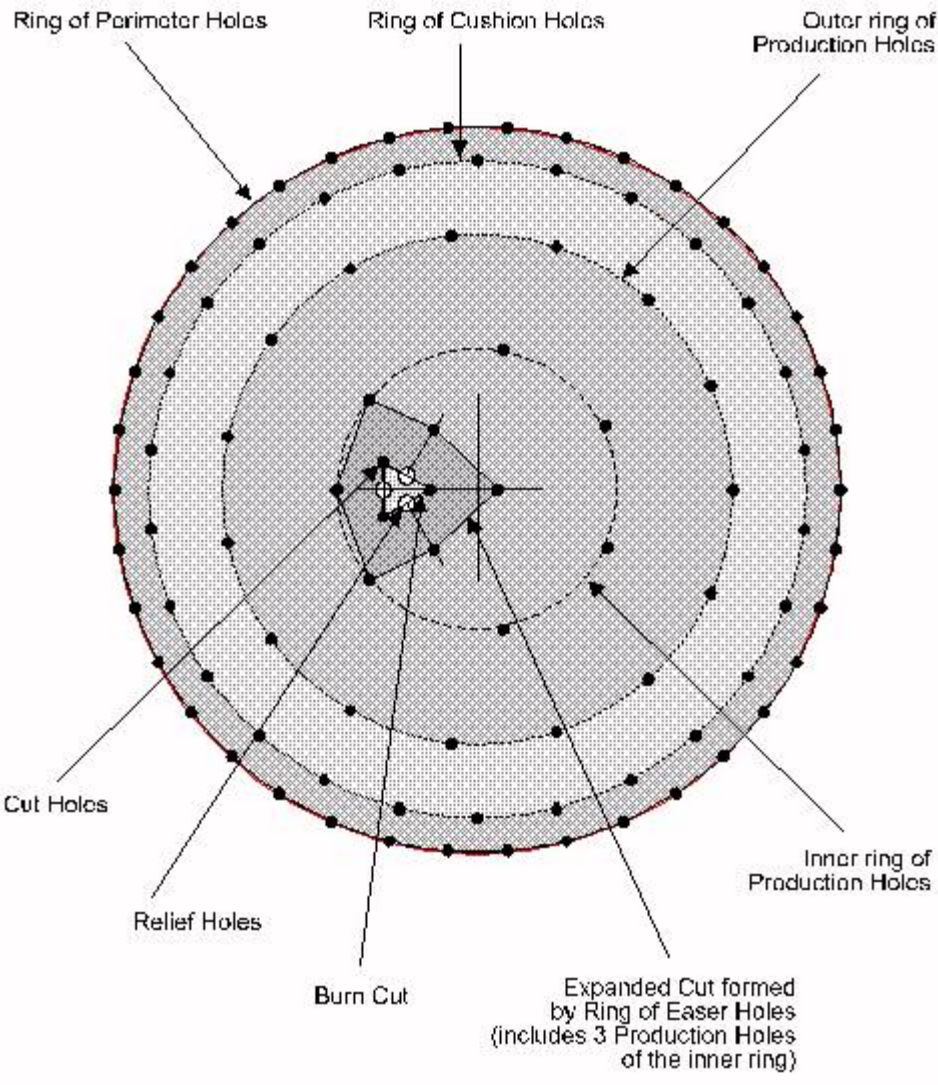
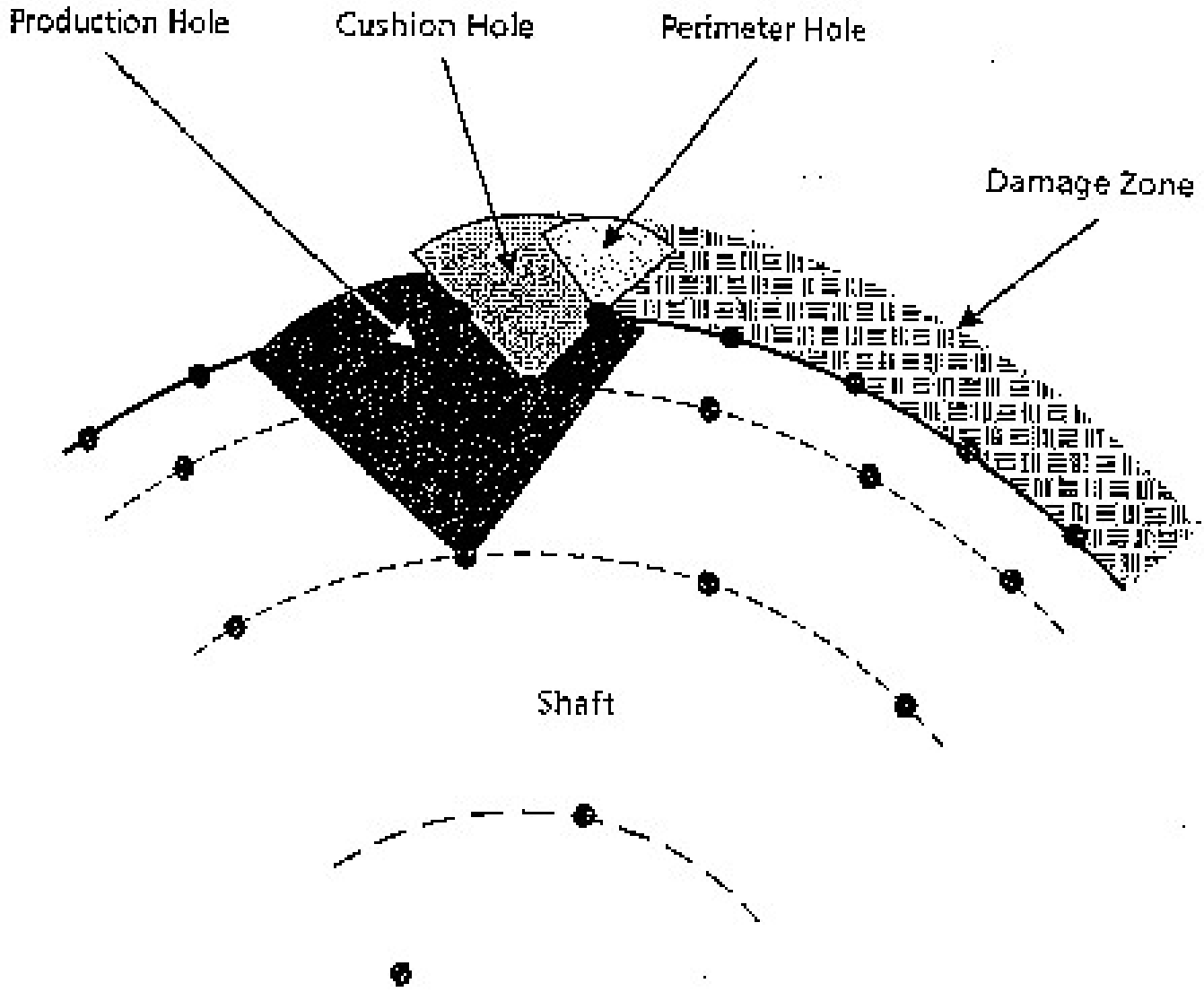
NIREX	Title Principle of burn-cut and wedge-cut designs in full-face shaft blasting	Figure No. A2 2


Ring of Perimeter Holes
Ring of Cushion Holes
Outer ring of Production Holes
Cut Holes
Relief Holes
Burn Cut
Expanded Cut formed by Ring of Easer Holes (includes 3 Production Holes of the inner ring)
Inner ring of Production Holes

Note: the radii of the arcs indicate the extent of blast induced damage to the rock mass.



NOT TO SCALE - INDICATIVE ONLY