Report No: WR4446/SRA
Date: September 2004

STABILITY RISK ASSESSMENT

for

PEN-Y-BONT LANDFILL SITE
PENTRE, CHIRK, WREXHAM
NORTH WALES

Prepared for
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Project Quality Assurance
Information Sheet

Stability Risk Assessment
Pen-y-Bont Landfill Site, Pentre, Chirk, Wrexham, North Wales
WRG WASTE SERVICES LIMITED

Report Number	:	WR4446/SRA	***Report Status***	:	FINAL

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1. INTRODUCTION

1.1 Report Context

In August 2004, **Encia Consulting Limited** (ECL) were commissioned by **WRG Waste Services Limited** (WRG) to prepare a Stability Risk Assessment for the PPC Permit Application for Pen-y-Bont Landfill Site. This stability Risk Assessment has been prepared using guidance contained within the **Environment Agency R&D Technical Report P1-385/TR2**.

The PPC Permit Application is required in order to secure the continuity of waste disposal activities beyond **9th November 2004**, after which date the remaining side-slope lining, capping, and restoration will be progressed to completion, in accordance with the proposed landfilling operations.

This Stability Risk Assessment (SRA) Report has been compiled as a reference document to accompany the **PPC Application Form Part B for Landfill**, particularly sections 1.2.11 to 1.2.23.

As part of the **PPC Permit Application**, Encia has undertaken a geo-technical Stability Risk Assessment (**SRA**). This document describes the manner in which the assessment has been carried out and presents the overall findings of the work. The relevant background information describing the site setting (including geological, geo-technical and engineering information, site monitoring data, and development proposals) is detailed within the site's **Environmental Setting and Installation Design Report** (**ESID Report**).

The methodology adopted for this Stability Risk Assessment largely follows the principles outlined in the *Environment Agency R&D Technical Report P-385 volumes TR1 and TR2* and, while not representing official Environment Agency Guidance, will be hereon referred to as the Guidance. Where additional analytical techniques have been used, these are described within the text.

1.1.1 Outline of the Installation

The location and detailed environmental setting of the existing landfill site is described in the accompanying **Environmental Setting and Installation Design** (**ESID) Report**. An outline of the proposed installation is included in this report.

The landfill site has been developed, from its inception, as an engineered containment facility, under the principle of hydraulic containment. Cells 1 to 3 were constructed with basal lining systems comprising:

- An **artificial sealing liner** on the base of the site comprising a 1m minimum thickness engineered clay liner with $k=5.5 \times 10^{-10}$ ms^{-1}. (The exception to this is Cell 3, where a 2mm HDPE flexible membrane liner (FML) and protection geo-textile was **also** used across the base.);

- An **artificially established geological barrier** on the side-slopes of the site comprising a 1m minimum thickness (perpendicular to side-slope) engineered compacted clay liner (CCL) with $k=5.5x10^{-10}$ ms^{-1} **and**, above 45mAOD (or 1m below upper surface of the Ruabon Marl) a 2mm HDPE flexible membrane liner (FML) with protection geo-textile. The exception to this is the majority of the side slopes to Cell 1, where **only** a CCL was used. The exact extent of the HDPE FML side-slope liner is shown on **Drawing ESID 6A**;

- A **geological barrier** below the artificial sealing liner on the base comprising in excess of 40m of Ruabon Marl.

Together, these elements form the containment engineering required to meet the requirements of the Hydro-geological Risk Assessment (HRA). These elements also conform to the requirements of the **Landfill (England & Wales) Regulations 2002**, and **Regulatory Guidance Note 6**. A note is also made in this report with regard to the HRA for the assessment of the lining system integrity. This is because the integrity of the containment lining system is linked to the assumptions about permeability in the HRA.

Upon the completion of landfilling in each cell, the waste will be capped with a 1mm thick linear low-density polyethylene (LLDPE) geo-membrane cap, with appropriate bedding and protection layers, and restoration soils.

These systems are shown on **Drawing ESID6A & 6B**. The following assessment has been compiled with reference to the most-recently published versions of all Environment Agency guidance documents.

1.2 Conceptual Stability Site Model

The following sub-sections present a summary of the natural geological, geo-synthetic, or fill materials (including engineered fill and waste infill) used in the model, relating specifically to the components identified in **Form IPPC Landfill Part B**, and from the guidance contained within the *Environment Agency R&D Technical Report P1385/TR2*.

The sub-grade model comprises Upper Coal Measures (Ruabon Marl) overlain by Glacial Till (Boulder Clay) on the western and southwestern margins of the quarry of the site and Terrace Deposits (Alluvium) in the flood plain of the River Dee. This has been further characterised into the following layers (from the top):

However, the site drift geology is more complex than this and comprises the following:

Made Ground

- Coal washings, previously located at the base of the quarry;
- Colliery shale, outside the main quarry to the south and east;
- Brickworks waste, encountered below the colliery shale;
- Gravel, till and weathered and contaminated clay to the northwest and southeast of the site.

Superficial Deposits

- River terrace and alluvial clay, sand, and gravel located in the floodplain area of the River Dee;

- Glacial till comprising gravely sandy silty clay and sandy silty clayey gravel has also been identified on the western and south-western margins of the quarry.

Upper Coal Measures, Ruabon Marl Formation

- Red brown mudstone and occasional light grey siltstones and fine sandstones. The Ruabon Marl has been identified across the site with the exception of the northwest corner and the central portion of the site;

Middle Coal Measures Series

- These are reported to consist of sandstone, mudstone, siltstone and coal. This series is brought adjacent to the Ruabon Marl formation in the central portion of the site by the faulting.

Formation	Thickness	Description
Superficial Deposits:		
Alluvium	5m	Firm red brown very sandy silty clay with sub-angular to sub-rounded fine to medium gravel
Terrace Deposits	2m	Red brown mottled sandy silty clay with fine to coarse gravel
Glacial Till (Boulder Clay)	11m	Soft to firm and stiff brown grey silty clay with gravel
Upper Coal Measures:		
Ruabon Marl	>40m	Red brown and grey green mudstone
Middle Coal Measures:		
	>45m	Grey green silty mudstone with coal, siltstone and sandstone

However, with the exception of topsoil (which was removed prior to development) and made ground, this represents the solid & drift geology of the site. The solid geology, beneath the drift, comprises Middle Coal Measures (sandstones, siltstones & mudstones) that have been faulted into position by two NNE to SSW and NE to SW trending faults. Notwithstanding the above, it must be noted that, although the perimeter side slope lining system is founded partly upon the Alluvium (and made ground in places) the basal lining system is founded entirely upon the Ruabon Marl.

The model of the containment basal and side-slope lining systems assessed comprise a 1m thick layer of re-compacted Ruabon Marl combined with a 2mm HDPE FML over the base of Cell 3 and a large part of the side-slopes above 45mAOD. (This additional FML has been utilised in the parts of the site where there is a potential for landfill gas migration.)

<u>Mining Activity</u>

The site lies within an area of previous coal mining activity. More specifically, the site lies within the likely zone of influence on the surface from workings in 4 seams of coal at 100m to 330m depths. The 1:25,000 Series Ordnance Survey map for Llangollen and Wrexham South (Sheet SJ 24/34) identifies disused pits, shafts, and open-cast workings approximately 2.5km to the north and west of the site, indicating that mining has taken place in the vicinity of the site.

The Working Plan for this site states that 'in the event of an adit being uncovered, the Coal Board will be notified and the adit sealed to the Coal Board specification in the agreement with the Waste Regulation Authority.' It should be noted that although coal workings were identified to the east and west of the site, at Plaskynaston and Wynnstay collieries, no adits have ever been uncovered at the site to date.

In addition, the **Coal Authority Report** for the site (dated **September 2004**) states that the last date of working was 1927 and that ground movement from the above mentioned coal workings should by now have ceased. This report also states that the site is not within the zone of likely influence on the surface from any present underground coal workings, and is not within a geographical area for which a licence to extract coal by underground methods is awaiting determination by the Coal Authority.

The Coal Authority add that they have no knowledge of any mine entries within, or within 20 metres of, the boundary of the site, and that the property is not located within the geographical boundary of an opencast site from which coal has been extracted by opencast methods. A full copy of this report is included in **Appendix SRA1**.

Existing Outer Perimeter Slopes

When assessing the stability, and integrity, of the containment lining systems at Pen-y-Bont Landfill Site, it should be noted that there are a number of existing outer perimeter slopes, which (although **not** proposed to be amended superficially) could influence the stability and integrity of the containment lining system of the landfill. In addition to this, the landfill proposals for the site could also affect the stability of these existing outer perimeter slopes.

These slopes have historically been listed numerically, in the order in which concerns were first raised about their stability. These are:

- **Slope 1** – Located on the western boundary overlooking the B5605;
- **Slope 2** – Located on the northern boundary overlooking the River Dee;
- **Slope 3** – Located adjacent to, and north of, Slope 2, also overlooking the River Dee;
- **Slope 4** – The tree-covered slope adjacent to, and south of, Slope 1, also overlooking the B5605.

A significant amount of work has already been undertaken with regard to the assessment (and monitoring) of these slopes since November 1999, when Wrexham County Borough Development Services Directorate first raised concerns about these slopes. Indeed, following remediation in the form of toe loading, **Slope 1** is no longer a significant concern. However, monitoring work continues with **Slope 2** and **Slope 3** and the reports detailing the evaluation, remediation, and monitoring work associated with these slopes is referenced in **Appendix SRA1**. (Copies of these reports are also included in the **ESID Report**.) Notwithstanding the above, the primary focus of this assessment is to determine the potential impact of these slopes upon the integrity of the landfill containment system, and the potential impact of the landfill proposals on the stability of these slopes.

1.2.1 Basal Sub-Grade Model

The geology beneath the proposed basal and side-slope lining system is described in the **ESID Report**. The base lies at an elevation between **40.5mAOD** in the west and **37.5mAOD** in the southeast, and the basal sub-grade immediately beneath the basal lining system comprises un-weathered Ruabon Marl. From the site investigations undertaken to date, this provides a firm unyielding platform for the basal lining system. An assessment of the potential consolidation in the Ruabon Marl (as a result of waste loading) is made later in the assessment. The findings of the site investigations and laboratory testing are shown in **Appendix SRA1.**

1.2.2 Side Slope Sub-Grade Model

The excavated quarry face has been re-graded to form the proposed side-slope sub-grade. From site investigations undertaken to date (and summarised in the **ESID Report**) these side-slopes comprised predominantly the stiff Ruabon Marl overlain by Glacial Till (Boulder Clay) and River Terrace Deposits (Alluvium) with some made ground towards the surface.

For the majority of the site, the internal faces of the existing quarry perimeter have been re-profiled on the landfill side at an **overall** slope angle of 1 in 2.5. This is made up of 10m (vertical height) slope sections at 1 in 2, in combination with 4m to 5m horizontal benches, as shown on **Drawing ESID6A & 6B**.

For the section of the site on the landfill (inner) side of **Slope 4**, the upper lift of the side-slope sub-grade will be formed with engineered fill (placed against the existing steep embankment after the removal of existing vegetation and re-grading) to create the 1 in 2 batter required for the 1m CCL and 2mm FML side-slope lining system.

The stability of these perimeter batters will be analysed for the case of the landfill side-slope without confinement of the side-slope liner, or waste, under the side-slope sub-grade analysis and assessment section.

1.2.3 Basal Lining System Model

The existing basal lining system for the cells at Pen-y-Bont: (from the Ruabon Marl formation upwards) comprises:

- 1m thick re-engineered Ruabon Marl compacted clay liner (CCL) with a maximum permeability of $k=5.5x10^{-10}$ m/s;
- 300mm thick leachate drainage stone (20mm, clean, non-calcareous gravel) with 160mm diameter HDPE preferential leachate pipework.

The exception to this is Cell 1 where only a herringbone pattern of preferential leachate pipework was utilised for the leachate drainage layer to leachate chamber **LC01**.

Details of the geo-technical testing of samples of materials recovered from the existing lining system in Cells 1 to 3 (inclusive) are presented in previous **CQA Validation Reports** for the basal and side-slope lining systems, referenced in the **ESID Report**.

The base of the landfill is divided into 3 cells by bunds that hydraulically isolate leachate within each cell. The bund between Cell 1 & 2 is 10m above the base of the landfill whilst the bund between Cell 2 & 3 is 3m above the base of the landfill.

1.2.4 Side Slope Lining System Model

The internal prepared face of the perimeter quarry side-slope has provided, and will continue to provide, the sub-grade formation surface for the 1m thick (perpendicular to the side-slope) compacted clay side-slope liner and (for sensitive areas of the site with regard to landfill gas migration) the additional 2mm HDPE FML side-slope liner and protection geo-textile.

The side-slope lining system models are the previously described sub-grade side-slopes, coupled with the side-slope lining system outlined above. The critical models for analysis will be the slopes highlighted in 1.2.2, shown on **Drawing ESID6A & 6B**.

The side-slope liner will be laid at an **overall** slope angle of approximately **1 in 2.5**. The side-slope liner proposed for Pen-y-Bont Landfill will be constructed in a uniform, surface linear arrangement, in a series of 10m (vertical height) lifts at **1 in 2**, with the waste. The stability of the side-slope lining system will be assessed when un-confined, and when confined with waste. The interaction between the consolidating (and de-grading) waste and the side-slope liner (long-term integrity as a result of settling waste) will be addressed in the risk screening section.

The stability of the proposed side-slope liner will be analysed in accordance with the sections shown on **Drawing ESID6A** & **6B**. The results of the analyses will be presented in **Appendix SRA3**. A summary of the results is presented in **Table SRA15** later in this report.

1.2.5 Waste Mass Model

The site is currently permitted to accept inert soils, household, commercial, and industrial wastes. It has been assumed that temporary waste slopes will not exceed a gradient of **1 in 2.5**, to ensure stability. At this gradient, slope failure within the waste itself is considered unlikely for the slope lengths possible at Pen-y-Bont. However, the risk of a non-circular translational waste slip along one of the side-slope geo-synthetic interfaces is a potential failure mode, which will be examined for the critical side-slope sections.

The leachate level within the cell will typically be maintained at a level **2m** above the top of the basal liner sump positions, but not less than **0.5m** above these basal sump levels, to allow for practical pumping considerations. Details of the parameters used in the waste analysis are shown in Section 2.6.

1.2.6 Capping System Model

A permanent cap comprising a 1mm LLDPE geo-membrane with suitable bedding and protection layers, restoration soils, (1m soil making materials) and preferential surface water pathways have been assumed. This capping system is described in the **ESID Report**. Both smooth and textured geo-membrane caps were considered at the conceptual design stage. A textured geo-membrane was selected as a result of the analyses highlighted later in this report. The pre-settlement restoration contours indicate that the slope angles will not exceed an angle of **19.65 degrees** (**1 in 2.8**) over a slope distance of approximately **45m**, or an angle of **10.49 degrees** (**1 in 5.4**) over a slope distance of approximately **105m**. These worst-case situations have been modelled.

2. STABILITY RISK ASSESSMENT

The six principal components of the conceptual stability site model have been considered and the various elements of that component have been assessed with regard to stability, and integrity.

The principal components considered are:

- The basal sub-grade;
- The side slope sub-grade;
- The basal lining system;
- The side-slope lining system;
- The waste;
- The capping system.

2.1 Risk Screening

Issues relating to stability and integrity (as defined in the **Part B PPC Application Form** for the Landfill Sector) for each principal component of the proposed development have been subject to a preliminary review to determine the need to undertake further detailed geotechnical analyses. The following sections present the results of this screening exercise.

2.1.1 Basal Sub-Grade Screening

The basal sub-grade for the whole site comprises Ruabon Marl (Upper Coal Measures). Details of the inter-cell bunds are shown on **Drawing ESID6A & ESID6B.** These documents are referenced in **Appendix SRA1**. A summary of these findings is also included in the **ESID Report**.

The total and differential settlement experienced as a result of construction of the basal lining system, and the subsequent in-filling of the cells, are unlikely to compromise the stability, or integrity, of the basal lining system. However, the consolidation range will be calculated, to aid an assessment of the proposed basal lining system integrity, based on the work of Edelmann et al (1999) and Arch et al (1996). During construction of the site basal liner, any soft areas identified were removed as part of the construction process. Evidence of this practice was also documented in the **CQA Validation Reports**. Therefore, the potential for any significant differential settlement in the basal lining system is very low.

The key considerations for the basal sub-grade, and the implications for stability and integrity are presented in **Table SRA 1** below:

Table SRA1: Stability Components for Basal Sub-grade

Excessive Deformation	Compressible sub-grade	The basal sub-grade for all the existing cells is the Ruabon Marl sequence of the Upper Coal Measures. As this material comprises a significant thickness of over-consolidated firm stiff clay, an analysis of the potential strains that could be imposed at any stage of the infilling process does not need to be undertaken.
	Basal heave	Due to the thickness of Ruabon Marl above any cohesion-less water bearing strata, the risk associated with basal heave during construction of the basal lining system does not need to be addressed.
	Cavities in sub-grade	The potential for cavities in the basal sub-grade is very low and as any soft areas were removed as part of the CQA process, they are unlikely to threaten the integrity of the basal lining system.
Filling on Waste	Compressible Waste	Not Applicable
	Cavities in waste	Not Applicable

2.1.2 Side Slope Sub-Grade Screening

The existing internal face of the perimeter batter (side-slope liner) will be re-profiled at an **overall** slope angle of 1 in 2.5. The stability of these slopes will be considered in the short-term (immediately after any further re-grading has taken place) and in the long-term, together with the proposed sidewall lining system, and the in-filled waste.

The key considerations for the side slope sub-grade and the implications for stability, and integrity, are presented in **Table SRA 2** below:

Table SRA2: Stability Components for Side Slope Sub-grade

Cut Slope (Un-confined & Confined)	Rock	Stability	Not Applicable
		Cavities in sub-grade	Not Applicable
		Groundwater	Not Applicable
		Deformability	Not Applicable
	Cohesive Soils	Stability	The remaining side slope sub-grades to be constructed at 1 in 2, will require assessment.
		Deformability	Consolidation and bearing capacity of the existing batters will be assessed but compression is unlikely to be significant.
		Time Dependent stability	Effective stress parameters will be used for the western bund stability assessment. This situation will be important for the long-term situation of the perimeter side slopes, as increased pore water pressures may influence upon the stability of Slope 4.
		Groundwater	The piezometric groundwater head will be taken into account when the remaining side-slope sub-grades are assessed.
	Granular Soils	Stability	Not Applicable
		Deformability	Not Applicable
		Groundwater	Not Applicable
Fill Slope (Un-confined & Confined)	Cohesive Soils	Stability	Effective stress parameters will be used for the western bund (in-fill) stability assessment. This situation will be important for the long-term situation of the perimeter side slopes, as increased pore water pressures may impact upon the stability of Slope 4.
		Time dependent stability	The stability of the confined (following restoration) perimeter bunds in preventing an embankment slip will be checked again, using effective long-term parameters.
		Groundwater	The latest drift groundwater levels are shown on Drawing ESID11. These levels have been interpolated from the latest perimeter monitoring borehole information.
	Granular Soils	Stability	Not Applicable
		Time dependency	Not Applicable
		Groundwater	Not Applicable
Natural Slopes		Stability	The outer perimeter slopes identified above (Slopes 1, 2 & 3) have already been assessed as part of previous work. Slope 1 and Slope 2 have also undergone remedial works, and are being monitored. Slopes 2 & 3 are currently being monitored, with survey equipment, on a regular basis. Although a review of the work associated with Slopes 1, 2 & 3 will be required, only Slope 4 requires additional modelling. The stability of this existing perimeter tree covered slope on the western boundary overlooking the road will need to be checked, allowing for waste with relatively steep restoration profiles to be placed against it.
		Groundwater	Rising groundwater levels (as a result of very wet periods) has affected the existing perimeter slopes. Slopes 2 & 3 are currently monitored, after regarding works and improved surface water drainage works were undertaken.

2.1.3 Basal Lining System Screening

As stated in 2.1.1 above, an analysis of basal lining movement will be presented to demonstrate the factor of safety against liner strain during the waste deposition period.

The controlling factors that influence the stability, and integrity, in the basal lining system are given in **Table SRA 3**, below:

Table SRA3: Stability Components for Basal Lining System

Artificial Sealing Liner	Stability and Integrity	The basal sub-grade is the Ruabon Marl. This material will consolidate slightly under the proposed loading from the lining system and waste infill. However, only a brief analysis will only be required, as the slight compression expected is unlikely threaten the integrity of the basal lining system.
	Compressible sub-grade	A brief analysis of the potential strains that could be imposed at any stage of the infilling process will be undertaken. This analysis will address the construction and waste deposition phases, as well as the situation following waste placement.
	Cavities	The implications of mining activity have already been addressed above.
	Basal Heave	Although the base of the landfill is significantly below the piezometric head of the groundwater, the thickness of the Ruabon Marl removes the possibility of basal heave.
Artificially Established Geological Barrier	Stability and Integrity	Not Applicable
	Compressible sub-grade	Not Applicable
	Cavities	Not Applicable
	Basal Heave	Not Applicable
Basal Leachate Control System (Drainage medium & Preferential Pipework)	Stability and Integrity of Pipework	Although the waste loads are unlikely to cause complete failure of the pipework (with appropriate bedding and surround to enable redistribution of point loads) the factor of safety against **buckling** will be calculated for the preferential pipework proposed for the base. In addition, the percentage deformation as a result of long term loading (**creep**) will be assessed using the **modified IOWA formula**. The maximum anticipated longitudinal bends are well within the tolerance of the proposed pipework, which can tolerate a **3-degree movement** at each **6m joint**.
	Sub-grade movement from compressible sub-grade or cavities	As sated above, the small amount of consolidation is unlikely to impact upon the basal leachate control system.
	Pipework Capacity and Longevity	The volume carrying capacity of this system is based on the fall, and pipe diameter of the primary pipework. Using the Colebrook White equation, and assuming a nominal bore of 150mm, and an hydraulic gradient of 0.02 (1 in 50) this individual pipe has a capacity of over 50m3 per hour. By inspection of the worst case 1 in 100 year storm event, this provides a significant level of redundancy in the short term before waste infilling commences. (Long-term leachate flow rates, where silting up may become problematic, will be substantially less.)

2.1.4 Side Slope Lining System Screening

Barriers built to full height (in addition to overcoming stability considerations) need to be protected from the environment, since cohesive materials are prone to desiccation cracking, and weathering. The side-slope lining system proposed for Pen-y-Bont Landfill is to be installed in a uniform, surface linear, lift arrangement against the existing prepared cut face, or filled batter, in the case of the upper lift of the south-western embankment. The face for the side-slope liner in Cells 1 to 3 has been, and will continue to be, re-profiled at an **overall** slope angle of 1 in 2.5. Installation of a 2mm HDPE side-slope FML liner and protection layer (typically a non-woven needle punched protection geo-textile with a CBR puncture resistance >19kN) will remove the potential for deterioration as a result of weathering.

The key considerations for the side-slope lining system and the implications for stability, and integrity, are presented **Table SRA4** below:

Table SRA4: Stability Components for Side Slope Lining System

Un-confined	Stability (Mineral System)	Stability of the **1 in 2** mineral element of the side-slope lining system will require further assessment.
	Stability (Geo-synthetic System)	In order to ensure an FOS >1.3 for stability of the un-confined FML side slope liner against a translational slip, a double textured FML should be utilised, <u>unless</u> reliance is made upon an anchor trench. The stability (as opposed to the integrity) of the unconfined FML (or protection geo-textile) does not need further assessment beyond this.
Confined	Stability (Mineral System)	Confinement of the mineral element will increase the factor of safety from that of the un-confined system as even new waste at shallow depths will have some stiffness, and will provide added lateral stability for the mineral system.
	Integrity (Geo-synthetic System)	Confinement of the side slope FML will lead to axial forces (and subsequent tensile strains) because of consolidation of the side slope sub-grade, and the compressible waste infill. An assessment of the likely strains will be made.
	Integrity (Mineral System)	Confinement of the side slope lining system will lead to an increase in tensile and shear strains as a result of consolidation of the side slope sub-grade, created by the lateral waste loading. Although these are unlikely to affect the integrity of the side slope liner, a brief analysis will be made. For the side-slope lining system at Pen-y-Bont (and based on our experience of finite difference modelling undertaken at similar sites) the tensile strains will not be significant enough to warrant the use of a low friction slip zone.

2.1.5 Waste Mass Screening

As waste is in-filled into the cells in lifts, temporary waste slopes will be present. Analysis is required in terms of the stability of these temporary waste slopes. The most critical situation will be when the waste is providing lateral restraint against the side-slope liner, and when waste is deposited to full height on one side of the liner and these situations will be analysed in more detail.

The controlling factors that influence the stability of the waste mass are presented in **Table SRA 5** below:

Table SRA5: Stability Components for Waste Slopes

Failure wholly in waste	Stability		Temporary waste slopes will exist during the development of the site. Although it is proposed to limit these slopes to 1 in 2.5, a brief assessment of the longest temporary waste slopes will be made to assess the sensitivity of the conclusions to the range of likely waste parameters to be encountered. Stability of the waste mass upon completion of landfill has not been analysed, as the restored slopes are shallower than 1 in 2.5. However, the risk of a translational slope failure of the capping system will be assessed.
Failure involving liner and waste	Mineral Clay/Geo-synthetics	Stability	Given that the waste mass with a temporary outer slope will be formed against the side slope, the potential for the waste to move along this interface needs to be considered further within this report.
		Integrity	An assessment of the integrity of all elements of the side slope lining system as waste is placed will be required.

2.1.6 Capping System Screening

The site will be capped with a 1mm LLDPE geo-membrane cap with suitable bedding and protection layers, and restoration soils above as shown on **Drawings ESID6A & 6B**. As the final restoration contours are relatively steep, a detailed analysis of the stability of the cover soils, with consideration of the effect of seepage forces in the cover soil, and plant movement during installation, is proposed.

The key considerations for the capping system, and the implications for stability and integrity, are presented in **Table SRA6** below:

Table SRA6: Stability Components for Capping System

Geo-synthetic Material	Stability	Pre-restoration Slope inclination	The capping system is to be placed on pre-settlement slopes exceeding a maximum inclination of 1 in 2.8. It is considered necessary to undertake further assessment of the capping system for this scenario.
		Plant Movements	Plant movements (acceleration & deceleration) on the capping system during construction, and following restoration, will have an impact upon the stability of the slope.
		Gas Pressure beneath capping system	The methodology proposed in the paper by Thiel (1999) examines a failure of the gas extraction system leading to build up of positive gas pressure underneath the capping membrane across a large enough area to cause local instability, by lowering the normal stress (from the capping system bulk load) providing shear resistance to sliding. This paper recommends a gas drainage layer beneath the capping membrane, to provide a mechanism to prevent such a build up. However, the placement of a gas control layer in this vicinity brings with it a significant number of problems, which can actually exacerbate the failure mechanism it is designed to mitigate. The reasons for this are addressed in the assessment section of this report.
	Integrity	Compressible waste	No external factors will be present to cause anything other than deformations normally associated with waste settlement. Further investigation is not considered to be required.
		Slope deformation	No external factors will be present to cause anything other than deformations normally associated with waste settlement. Further investigation is not considered to be required.
		Plant Movements	Wheel and track loads on the Plant travelling on top of the capping system will impose stresses, which induce strains in the capping system. These strains will need to be analysed in more detail.
		Cavities in waste	All bulky low-density items will be flattened prior to burial. This will avoid the presence of subsurface cavities that may give rise to unstable ground conditions during filling. In addition, it is standard practice for the final layers of waste to be selected and inspected to ensure that these do not cause damage to the final capping. This aspect is therefore not considered to require further assessment.

2.2 Lifecycle Cells

The history of the site is documented in the **ESID Report**. In common with other sites that have been opened in recent years, the site has been developed as a containment landfill, under the principal of hydraulic containment, from the outset.

Landfilling proceeded from Cell 1 to Cell 3 in sequential order, with the side-slope lining lifts prepared in advance of each waste lift. Although the basal lining systems for each cell were constructed in one stage, the side-slope lining systems were, and will continue to be, constructed in sectional 10m (vertical height) lifts.

The critical states (design scenarios) for consideration of the basal and side-slope lining system stability, and integrity, will therefore be:

- During construction of the side-slope lining system;
- During, and following completion of, the land-filling in each cell lift, where a differential load will exist across the side-slope lining system;
- Following the completion of land-filling in all cells, where a significant load will exist right across the basal lining system;
- Following the completion of capping, and restoration.

The critical states (design scenarios) for consideration of the capping system stability, and integrity, will therefore be:

- Following the completion of filling with waste to pre-settlement levels (for capping stability analyses);
- During construction of the capping system, and maintenance of the restored landfill.

2.2.1 Leachate Management

Leachate management within the site comprises a granular leachate drainage blanket, with piped preferential pathways, laid across the base of each cell, as shown on **Drawing ESID7**. The exception to this is Cell 1, where only preferential pipework has been installed. The leachate head is controlled using vertical risers, positioned at the cell sumps, as shown on **Drawing ESID7**. (Up-slope risers have been trialled at the site.) The existing, previously installed, basal leachate pipework has **not** been designed to accommodate a maximum 10% deflection (under long-term creep to resist the static gravity loading of the waste) or achieve a factor of safety of 2.5 against buckling. Leachate management (beyond stability considerations) is described in the **ESID Report**.

2.2.2 Landfill Gas Management

Landfill gas will be managed using an active gas extraction system. The system will incorporate drilled vertical gas extraction wells, and will be connected to the power generation engines (and stand-by flare) using wellheads, and suitably sized lateral pipework. The effectiveness of the extraction system will be affected by differential settlement of the waste leading to low spots along the gas carrier mains across previously filled areas. These low spots could lead to the collection of condensate, which in turn will lead to blockages in the collection system. To minimise the effect of waste settlement on the effectiveness of the gas collection system, gas extraction mains will be installed to suitable gradients across filled areas, and condensate sumps will be installed at strategic locations. These measures will ensure that the effectiveness of the collection system will not be affected by settlement of the waste mass. Landfill gas management (beyond stability considerations) is described in the **ESID Report**.

2.2.3 Daily Cover Characteristics

Daily cover will be used on site consisting of suitable inert material, or other suitable material as agreed with the Environment Agency. It will be placed to a depth to ensure adequate covering of the waste, whilst ensuring sufficient traction to delivering vehicles.

2.3 Data Summary

Geo-technical data for the analysis of the stability of Pen-y-Bont Landfill has been obtained from a number of sources. These include previous borehole investigations, trial pits, laboratory testing, CQA validation reports, and published data applicable to the analyses.

The expectation is that, post-completion, there will be a **25%** reduction in waste volumes, due to settlement (consolidation) and degradation of the organic waste fraction. Therefore, a surcharge of **33%** has been allowed for in the derivation of pre-settlement contour levels for the waste, as far as reasonable practicable, taking into account the steep perimeter restoration slopes.

The following data are required as input for the analyses undertaken for this Stability Risk Assessment:

- Material unit weights;
- Un-drained (total stress analysis) and effective (peak and residual) shear strengths of soils and waste;
- Peak and post-peak interface friction angles of interfaces and structural elements used to represent the basal, side-slope, and capping system geo-synthetics;
- Elastic properties of the soils, and waste;
- Elastic properties of the interfaces.

There is a significant amount of site-specific data on geo-technical properties. Accordingly, values have been adopted from previous geo-technical risk assessment work for the site. However, it is pointed out that there are published data relating to the geo-technical properties of the materials present within the stability models and parameters have been adopted both from published sources and Encia's database of properties for such materials. Conservative parameters have been adopted such that a satisfactory level of confidence in the analytical results is achieved.

2.4 Selection of Appropriate Factors of Safety

The factor of safety is the numerical expression of the degree of confidence that exists for a given set of conditions, against a particular failure mechanism occurring. It is commonly expressed as the ratio of the load or action that would cause failure against the actual load or actions likely to be applied during service. This is readily determined for some types of analysis, for example limit equilibrium slope stability analyses. However, greater consideration must be given to analyses that do not report factors of safety directly. For example, a finite difference analysis of shear strains within a steep side-slope lining system would not usually indicate overall failure of the model even though the strains could be high enough to indicate a failure of the integrity of the lining system. In such cases, it is necessary to define an upper limit for shear strains and to express the factor of safety as the ratio of allowable strain to actual strain.

Before determining appropriate factors of safety for the various components of the model, it is necessary to identify key receptors, and evaluate the consequences in the event of a failure relating to both stability, and integrity. Consideration of the following receptors is required:

• Groundwater;
• Property - Site infrastructure & third party property in particular;
• Human beings at direct risk.

The factor of safety adopted for each component of the model would be related to the consequences of a failure.

BS6031 - Code of Practice for Earthworks (Clause 6.5.1.2 Safety Factors) states that suitable safety factors in a particular case can only be arrived at after careful consideration of all the relevant factors, and the exercise of sound engineering judgement. The factors to be considered include:

a) The complexity of the soil conditions;
b) The adequacy of the site investigation;
c) The certainty with which the design parameters represent the actual in-situ conditions;
d) The length of time over which the stability has to be assured;
e) The likelihood of unfavourable changes in groundwater regime in the future;
f) The likelihood of unfavourable changes in the surface profile in the future;
g) The speed of any movement which might take place;
h) The consequences of any failure.

2.4.1 Factor of Safety for Basal Sub-Grade

Although consolidation of the basal sub-grade will influence the integrity of the basal lining system, this particular aspect will be addressed in Section 2.4.3.

2.4.2 Factor of Safety for Side Slopes Sub-Grade

A factor of safety of 1.3 is considered appropriate for the slopes likely to exist in the short, or long-term. However, in the assessment of long-term stability, consideration needs to be given to the time taken for the equalisation of pore water pressures (after excavation) and the time for which the slopes will be left exposed. Therefore, where *residual* (as opposed to *peak*) effective strength parameters are utilised, a lower factor of safety (> unity for low risk situations) would be considered acceptable, in accordance with the Guidance.

The exception to this is the tree covered outer sub-grade slope on the western boundary overlooking the B5605 (Slope 4). For this slope, consideration needs to be given to the most likely failure mechanism, in order to allow sufficient notice to be achieved before a slip that may block the road. Previous work has required a factor of safety in excess of 1.4 for this slope. However, where no allowance is made for the existing well-established trees, a lower factor of safety would be satisfactory.

2.4.3 Factor of Safety for Basal Lining System

Assessing the short and long-term integrity of the basal lining system will be based on the work of Edelmann et al (1999), Jessberger and Stone (1991), and Arch at al (1996), as well as the Guidance. The full references for all these papers are included at the end of this report, but a summary of their conclusions (and their applicability to the situation at Pen-y-Bont) will be documented in the assessment section.

Considering the above references, a factor of safety in excess of 1.3 is considered acceptable. However, it is proposed (for this element of the risk assessment) to present the maximum strains determined from the analysis, and compare these with the conclusions of the latest research relating to this aspect of landfill design, in order to determine acceptability.

2.4.4 Factor of Safety for Side Slope Lining System (Unconfined & Confined)

A minimum factor of safety of 1.3 is considered acceptable for the short-term un-drained condition. In the assessment of the un-confined long-term stability, consideration needs to be given to the time taken for the equalisation of pore pressures within the side slope sub-grade and the time for which the slope will be exposed. Taking into account the time before placement of waste against the side-slope liner (providing lateral support) a lower factor of safety for the long-term (effective stress) condition would be considered acceptable.

Assessment of the confined side-slope liner integrity will be based on the work of Edelmann et al (1999), Jessberger and Stone (1991), and Arch at al (1996), as well as the Guidance. The full references for all these papers are included at the end of this report, but a summary of their conclusions (and their applicability to the situation at Pen-y-Bont) will be documented in the assessment section.

2.4.5 Factor of Safety for Waste Mass

A minimum factor of safety of 1.3 is considered acceptable for stability, if reasonably conservative values are used. However, as the waste shear strength parameters presented within the Guidance are considered conservative and can be considered to already include an element of partial factoring, it is considered appropriate to adopt a factor of safety of 1.2, if adopting these shear strength parameters in combination with the traditional approach (Section 2.2.4 of the Guidance).

2.4.6 Factor of Safety for Capping System

A minimum factor of safety against sliding (translational slip) failure of any component of the capping system of 1.3 is considered appropriate and has been adopted where peak (effective stress) shear strength conditions are applied for the pre-settlement slopes. Factors of safety of greater than unity are considered appropriate where residual shear strengths are applied. It is pointed out that the calculated factors of safety will increase as the waste settles.

In addition to the stability analyses, an assessment of the effects of construction plant on the integrity of the geo-membrane component of the cap has also been undertaken. For the purposes of this analysis, a factor of safety of 1.5 against tensile rupture of the geo-membrane has been adopted. (The interface friction angles assumed must also be confirmed on site as still being suitable, during the

CQA process.) However, it is also proposed to present the maximum strains determined from the analysis, and compare these with the conclusions of the latest research relating to this aspect of landfill design, in order to determine acceptability.

2.5 Justification for Modelling Approach and Software

In order to perform a comprehensive Stability Risk Assessment, the components of the landfill development have to be considered not only individually but also in conjunction with one another, where relevant. Any analytical techniques adopted for such an assessment should adequately represent all of the considered scenarios (the different modelled Cells of the lifecycle) for both confined and unconfined conditions (where appropriate). The methodology and the software should also achieve the desired output parameters for the assessment. This equates to the determination of limit equilibrium factors of safety, or the calculation of strains within liner components.

The analytical methods used in this stability risk assessment include:

- Limit equilibrium stability analyses for the calculation of factors of safety for the un-confined side-slope sub-grade and side-slope lining system, the confined perimeter bund stability, and the waste mass, with temporary waste slopes adjacent to the inter-cell bund;
- Finite element analyses for the determination of axial loads (and resultant strains) in the geo-synthetic components, and tensile and shear strains within the mineral lining systems, where appropriate;
- Closed-form analyses for the capping system stability, and for the leachate pipework integrity.

The limit equilibrium analyses are to be assessed using the **GEO** suite of computer programs written and developed by **OASYS** (SLOPE: Version 17). This suite of programs is routinely used by Encia Consulting Limited for the analysis of slope stability problems, and has been found to be completely satisfactory.

Slope stability analyses were carried out for the outer perimeter slopes, the side-slope sub-grade, and the side-slope liner using the following methods:

- **Bishop Slip Circle Analysis** with slip circles passing through the toe of any slope or change of slope;
- **Bishop Slip Circle Analysis** with circles at increasing radii;
- **Janbu Non-Circular Analysis** through the slope.

Translational slips along a combination of geo-synthetic elements are analysed by modelling a material with nominal thickness along the plane under consideration. The lowest interface friction angle for a group of geo-synthetic and non geo-synthetic materials can then be used as the internal angle of friction for this nominal thickness material, to calculate the factor of safety against failure along this plane.

The proprietary software **PLAXIS** (Version 8.2) has been used for the side-slope, basal liner, and capping system strain assessment during construction of the lining system, and during all phases of waste placement. This is a two-dimensional finite element programme intended for the analysis of deformation and stability in geotechnical engineering. It is equipped for the simulation of non-linear, time dependent and anisotropic behaviour of soils and rock. In addition, since soil is multi-phase material, special procedures are required to deal with hydrostatic and non-hydrostatic pore pressures in the soil. **PLAXIS** was originally developed for geotechnical engineers studying river embankments on the soft soils of the lowlands of Holland. In subsequent years, **PLAXIS** has been extended to cover most other areas of geotechnical engineering. It is therefore well suited for application to the Pen-y-Bont Landfill containment lining system assessment.

The stability assessments of the capping system soils have been undertaken using the methods proposed by Jones and Dixon (1998). The analysis of the effects of construction plant on the geo-membrane component of the capping system was undertaken using the method proposed by Koerner & Daniel (1997).

Side lining systems constructed in stages (and reliant on waste for some stability) can deform during the construction process. This is because the layers of waste initially placed against the lining system provide relatively low levels of support, and because, at shallow depths, the waste has a low stiffness and shear strength. (These parameters increase with depth through the increased vertical stress.)

The finite element work models the emplacement of the waste infill in stages, to enable an assessment of the integrity of the side-slope, and basal, lining systems.

To summarise, stability and integrity assessments have been carried out to assess the following:

- Stability of the Side-Slope Sub-grade;
- Stability of the Side-Slope Liner Pre-Waste Placement;
- Integrity of the Basal Liner Post-Waste Placement;
- Integrity of the Side-Slope Liner Post-Waste Placement;
- Stability of the Temporary Waste Slopes;
- Stability of the Capping System During Construction & Post Closure (Including Plant & Vehicle Movements);
- Integrity of the Capping system During Construction & Post-Closure (Including Plant & Vehicle Movements).

2.6 Justification for Geo-technical Parameters Selected for Analyses

The parameters selected for material properties take into account the analyses undertaken, and where there was uncertainty, a sensitivity analysis was used to assess the potential for instability, or loss of integrity, due to excessive levels of strain.

Cut slopes in cohesive soils are kept stable in the short-term by pore water suctions (negative pore water pressure), which increase the effective shear strength of the soil. However, as these suctions dissipate (time depending upon the permeability of the soil) stability decreases. Where embankment fills are placed, excess (positive) pore water pressures are created in cohesive soils, which lowers the effective shear strength of the soil. Therefore, consideration has been given to both the short-term (un-drained total stress) and long-term (effective stress) states for each scenario, throughout development of the site.

Both the artificial sealing liner (ASL) for the basal lining system and the artificially established geological barrier (AEGB) for the side-slope lining system have been constructed from the site-won Ruabon Marl to form the mineral compacted clay liner (CCL). Therefore, parameters consistent with these materials have been utilised within the analyses. These parameters were refined using a back analysis of an existing slope formed in these materials, and compared with previous laboratory test results.

For the finite element analysis undertaken to assess the differential and total settlements (and resulting strains) in the lining system, the previous site investigation and laboratory work undertaken was used to apportion stiffness values to the sub-grade for each depth band.

The critical states (design scenarios) for consideration of the basal and side-slope liner stability will therefore be:

- During construction of the side-slope lining system;
- During the deposition and compaction of waste against side-slope liner;
- Following the completion of land-filling in each cell, where a differential load will exist across the basal and side-slope lining system;
- Following the completion of waste deposition during capping installation, and following restoration.

According to Reddy et al (1996), the behaviour of the waste body itself will control the ultimate performance of the side-slope lining system in the long-term. As waste is typically a heterogeneous material with engineering properties that change over time, a range of waste material properties have been used, as part of a sensitivity analysis. This range of values is proposed to reflect the volumetric strain hardening behaviour of waste (increasing stiffness and shear strength) resulting from the decreasing volume of the waste over time. As Jones and Dixon (2001) demonstrate (and what common sense would dictate) waste in a landfill becomes stiffer with age, and burial depth. Therefore, for the assessment of vehicle loading on the capping system, conservative waste values have been used to reflect the situation with new waste close to the surface.

In ***long-term*** (effective stress) analyses, the materials are reliant on their frictional properties (ie ϕ') for shear strength, and little from their apparent cohesion (c').

In terms of non-hazardous waste strength, Encia adopts conservative values of effective shear strength parameters as derived from a study of geotechnical properties of municipal waste by Van Impe and Bouazza (1995) these values being backed up in later work by Kavazanjian et al (1996) and later confirmed in a research summary by Jotisankasa (2001). The values for c' and ø' adopted throughout the modelling were 5kPa and 25 degrees, respectively. The unit weight of waste was taken as 11kN/m3, a value slightly higher than that generally adopted of 10kN/m3. This is based upon experience gained from some of Encia's most recent modelling and stability work.

Table SRA7 below summarises the parameters utilised in the analyses:

Table SRA7: Summary of Material Parameters

	Long term parameters		Short term parameters		
Material	**Cohesion with respect to effective stress** **c' kN/m²**	**Angle of friction with respect to effective stresses** **ϕ' °**	**Un-drained shear strength** **C_u kN/m²**	**ϕ_u °**	**Bulk density** **γ'/γ kN/m³**
Typical Non-Hazardous Waste (Jessberger 1994)	5.00	25.00	-	-	11.00
Waste (Sensitivity Range)	5.00 to 28.00	15.00 to 41.00	-	-	10.00 to 11.00
Leachate Drainage Stone	0.00	30.00	-	-	18.00
Temporary Spine Road (DoT Type 1)	0.00	33.00 to 35.00	-	-	18.00
Re-compacted Engineered Clay Liner (Ruabon Marl)	0.00 to 5.00	25.00 to 30.00	50.00 to 75.00	-	20.00
Alluvial Sand & Gravel	0.00	30.00	-	-	19.00
Made Ground	0.00	28.00 to 29.00	20.00 to 40.00	-	20.00
Weathered Marl & Boulder Clay	5.00 to 10.00	25.00 to 30.00	50.00 to 100.00	-	20.00
Un-weathered Ruabon Marl	30.00	35.00	400.00	-	20.00

The material parameters utilised for the finite element analysis to assess consolidation in the side and basal sub-grade, and the resulting strains in the side and basal lining system, are shown below.

Table SRA8 below summarises the parameters utilised in the analyses:

Table SRA8: Summary of Material Parameters for Finite Element Analyses

Material	Bulk Density	Cohesion with respect to effective stress	Angle of friction with respect to effective stress	Water Permeability	Poisson's Ratio	Young's Modulus
	kN/m^3	kN/m^2	°	m/s	-	MN/m^2
Non-Hazardous Waste	11.00	5.00	25.00	$1x10^{-5}$	0.35	0.5
Waste (Sensitivity Range)	10.00 to 11.00	5.00 to 28.00	15.00 to 41.00	$1x10^{-4}$ to $1x10^{-8}$	0.30 to 0.35	0.5 to 6.0
Leachate Drainage Stone	18.00	0.00	30.00	$1x10^{-3}$	0.30	3.0
Temporary Spine Road	18.00	0.00	33.00 to 35.00	$1x10^{-5}$	0.30	3.0
Engineered Clay Liner (Ruabon Marl)	20.00	0.00 to 5.00	25.00 to 30.00	$5.5x10^{-10}$	0.35	3.0
Alluvial Sand & Gravel	19.00	0.00	30.00	$1x10^{-4}$	0.35	3.0
Made Ground	20.00	0.00	28.00 to 29.00	$5x10^{-8}$	0.35	2.0 to 3.0
Weathered Marl & Boulder Clay	20.00	5.00 to 10.00	25.00 to 30.00	$5x10^{-9}$	0.35	10.0
Un-weathered Ruabon Marl	20.00	30.00	35.00	$5x10^{-10}$	0.35	10.0 to 100.0

Table SRA9 below summarises the extensional stiffness values utilised for the geo-synthetic elements of the composite basal lining system in the analyses:

Table SRA9: Summary of Geo-synthetic Material Parameters for Finite Element Analyses

Geo-synthetic Material	Product Specification Assumed	Extensional Stiffness, J kN/m
2mm High Density Polyethylene (HDPE) Flexible Membrane Liner (FML) Side Slope Liner	*'2mm GSE HD Friction Flex'*	275
Geo-textile Protection Layer (For 2mm HDPE FML Side-Slope Liner)	*'Geofabrics HP19'*	150
1mm Linear Low Density Polyethylene (LLDPE) Flexible Membrane Cap	*'1mm GSE Multi Friction Flex'*	175

The above parameters for the analysis of stability were obtained from a combination of published literature and site-specific laboratory testing. Engineering properties for the waste mass were obtained using guidance from *Environment Agency R&D Technical Report P1385/TR1*. Figures used for the interface friction angles for the various geo-synthetic and mineral interfaces have been taken from **Table 7.2**, **Table 7.3**, & **Table 7.4** of Report No.1 of the Guidance. These tables are not repeated here, but the figures should be confirmed as being accurate during preparation of the **CQA Method Statement**, and validated on site, before preparation of the **CQA Validation Report**.

Table SRA10 below summarises the plant loadings used in the finite element analyses.

Table SRA10: Summary of Plant Loadings for Finite Element Strain Analyses in the Capping System

Critical Plant Scenario	Un-factored Total Plant Load (Axle Loads) kN	Contact Area m^2	Un-factored Maximum Ground Bearing Pressures kN/m^2	Actual Width of Load m	*3D Corrected UDL (Wheel or Track) Load Per Unit Length for Capping Depth kN/m
20 Tonne D6 Dozer/20 Tonne 360 Degree Excavator on Tracks	200kN (100+100)	2 Tracks of $1.75m^2$	$60kN/m^2$	400mm to 500mm	29kN/m over 3.5m Track
JCB 3CX Backhoe (2 Wheel Axles) Excavator on Capping System	80kN (30+50)	2 Wheels of 0.25m2 & 2 Wheels of $0.38m^2$	$60kN/m^2$ & $65kN/m^2$	300mm & 450mm	30kN/m for 500mm & 33kN/m over 750mm
Fully Laden Dumper (3 Wheel Axles) on Stone Access Road Above Capping System	380kN (100+ 140+140)	6 Wheels of $0.375m^2$	$135kN/m^2$ & $185kN/m^2$	500mm	67kN/m & 2x 93kN/m over 3 sets of 750mm

A** ***Boussinesq Analysis *was used based on research work of Poulos & Davis (1974)*

2.7 Analyses

2.7.1 Basal Sub-Grade Analyses

An analysis of the sub-grade consolidation, and its impact upon the integrity of the basal lining system, is included in Section 2.7.3.

2.7.2 Side Slope Sub-Grade Analyses

The side-slope sub-grade analyses required are the landfill side of the un-confined re-graded quarry face, the landfill side face of the existing western embankment following in-filling with engineered fill to achieve the required 1 in 2 batter, and an analysis of the outside slope of the existing western embankment when confined (and un-confined) on the waste side, following restoration.

Un-confined Inner Perimeter Side Slope Sub-grade

Analysis of the side-slope sub-grade stability was carried out using the following methods:

- **Bishop Slip Circle Analysis** with slip circles passing through the toe of any slope or change of slope;
- **Bishop Slip Circle Analysis** with circles at increasing radii;
- **Janbu Non-Circular Analysis** through the slope.

A summary of the results of the SLOPE runs for the side slope sub-grade are presented in **Table SRA11** below:

Table SRA11: Summary of SLOPE Runs for Un-confined Side-Slope Sub Grade – Landfill Side

Reference	Description	Lowest Factor of Safety
1	Short-term Stability of Side Slope Sub-Grade, Boulder Clay / Made Ground 1 in 2, Cu=50kPa, Phi=0^0	1.85
2	Short-term Stability of Side Slope Sub-Grade, Ruabon Marl 1 in 2, Cu=400kPa, Phi=0^0	3.24
3	Long-term Stability of Side Slope Sub-Grade, 1 in 2, C'=10kPa, Phi=31^0 (Single 10m Lift)	2.14
4	Long-term Stability of Side Slope Sub-Grade, Ruabon Marl 1 in 2, Cu=11kPa, Phi=31^0 (Double 20m Lift)	1.72

The factors of safety in the short-term using the un-drained values (total stress analysis) are higher than the factors of safety in the long-term using effective stress values. This is because the cohesive soils, with un-drained shear strengths in excess of 40kN/m2, receive a reduction in shear strength with the transition to effective stress values.

However, the factors of safety for the long-term situation (using effective stress values) were all in excess of 1.3. The key factor in the short and long-term was shown to be the thickness of any soft cohesive made ground with un-drained shear strengths less than 40kN/m2. A graphical representation of the results is shown in **Appendix SRA3**.

Confined (On One Side) Perimeter Embankment – Outer Batter – Slope 4

Analysis of the sub-grade slope stability was carried out using the following methods:

- **Bishop Slip Circle Analysis** with slip circles passing through the toe of any slope or change of slope;
- **Bishop Slip Circle Analysis** with circles at increasing radii;
- **Janbu Non-Circular Analysis** through the slope.

A summary of the results of the SLOPE runs for the side slope sub-grade are presented in **Table SRA12** below:

Table SRA12: Summary of SLOPE Runs for Perimeter Embankment – Non-Landfill (Outer) Side – With & Without Landfill

Reference	Description	Lowest Factor of Safety
Slope 4	Stability of Un-Confined Outer Slope, Short Term, Cu=50kPa, Phi'=0^0 (Un-drained Values)	1.85
Slope 4	Stability of Confined (1 in 2.8) Outer Slope, Short Term, Cu=50kPa, Phi=0^0 (Un-drained Values)	1.85
Slope 4	Stability of Un-Confined Outer Slope, Long Term, C'=5kPa, Phi'=30^0 (*peak* Effective Stress Values)	1.35
Slope 4	Stability of Confined Outer Slope (1 in 2.8) Long Term, C'=5kPa, Phi'=30^0 (*peak* Effective Stress Values)	1.35
Slope 4	Stability of Un-Confined Outer Slope, Long Term, C'=5kPa, Phi=25^0 (*Residual* Effective Stress Values)	1.08
Slope 4	Stability of Confined Outer Slope, Long Term, C'=5kPa, Phi'=25^0 (*Residual* Effective Stress Values)	1.08
Slope 4	Stability of Un-Confined Outer Slope (1 in 2), Long Term, C'=5kPa, Phi'=30^0 (*peak* Effective Stress Values)	1.22
Slope 4	Stability of Confined Outer Slope (1 in 2) Long Term, C'=5kPa, Phi'=30^0 (*peak* Effective Stress Values)	1.22

The factors of safety for the un-drained (short-term) conditions were all in excess of 1.3. The limiting factor in the short and long-term was shown to be from any rogue bands of cohesionless strata within the perimeter embankment. The factors of safety for the long-term situation (using peak and residual effective stress values) were not in excess of 1.3. However, the factors of safety **before** and **after** confinement with the landfill with different waste densities (ranging from 11kN/m^3 to 15kN/m^3) different perimeter waste slopes (up to 1 in 2) and different effective shear strength parameters **were the same**. This situation is addressed under the assessment section. A graphical representation of the results is shown in **Appendix SRA3**.

<u>Un-Confined Following Fill Placement – Inner Side-Slope Sub-grade – Western Embankment</u>

Analysis of the sub-grade slope stability was carried out using the following methods:

- **Bishop Slip Circle Analysis** with slip circles passing through the toe of any slope or change of slope;
- **Bishop Slip Circle Analysis** with circles at increasing radii;
- **Janbu Non-Circular Analysis** through the slope.

A summary of the results of the SLOPE runs for the side slope sub-grade are presented in **Table SRA13** below:

Table SRA13: Summary of SLOPE Runs for Un-Confined Western Perimeter Embankment – Landfill (Inner) Side – Following Fill Placement

Reference	Description	Lowest Factor of Safety
Western Side-Slope Sub-grade	Stability of Un-Confined Side Slope Sub-grade, Short Term, Following Placement of Engineered Fill Cu=50kPa, Phi'=0^0 (Un-drained Values)	1.95
Western Side-Slope Sub-grade	Stability of Un-Confined Side Slope Sub-grade (1 in 2) Long Term, Following Placement of Engineered Fill C'=5kPa, Phi'=25^0 (Peak Effective Stress Values)	1.43

The factors of safety for the short-term (un-drained) and long-term (effective stress) conditions were in excess of 1.3. The limiting factor in the long-term was shown to be from the creation on excess pore water pressures (reduction in shear strength) in the cohesive fill, should the reliance on cohesion be reduced.

2.7.3 Basal Liner Analyses

The only area requiring analysis is the integrity of the basal lining system as a result of consolidation within the sub-grade during, and following completion of, the waste deposition process. A two-dimensional finite element model for the critical section, shown in **Appendix SRA2** has been used to determine the maximum strains.

A summary of the calculation results of the deformations in the mineral liner resulting from the waste in-fill regime modelled is shown in **Table SRA13**. The maximum consolidation expected at the top of the Ruabon Marl is predicted to occur after final tipping.

A summary of the maximum strains in the basal lining system is shown in **Table SRA14** below:

Table SRA14: Summary of Maximum Strains & Lowest Factors of Safety for Basal Lining System Elements

Scenario	1m Compacted Clay Liner (Basal Liner)	
Activity/Factor of Safety	Maximum Horizontal (Tensile) Strain (%)	Maximum Vertical (Shear) Strain (%)
Basal Liner - Worst Case (Intermediate)	0.50%	5.50%
Basal Liner - Worst Case (Final)	0.75%	7.50%
Permeability & Strain Guidance Limit	**1.3%** (Edelmann et al 1999)	**10%** (Arch et al (1996)
Lowest Factor of Safety****	**1.73**	**1.33**

* The strain values have been derived from the tensile strength values according to the manufacturer's QC test results.
** Based on research work referenced in the Guidance.

Analysis of the sub-grade bearing capacity and settlement potential of the sub-grade in the remaining Cells was also undertaken, to compare against the work of Edelmann et al. The maximum likely settlement range is not likely to compromise the liner integrity, or permeability. No further geo-technical analysis was deemed necessary. A graphical representation of the results is presented in **Appendix SRA3**.

2.7.4 Side Slope Liner Analyses (un-confined and Confined)

Following on from an analysis of the side slope sub-grade in Section 2.7.1, an assessment of the un-confined side slope liner was undertaken.

Un-confined Side Slope Liner (Stability)

Analysis of the unconfined side slope stability was carried out using the following methods:

- **Bishop Slip Circle Analysis** with slip circles passing through the toe of any slope or change of slope;
- **Bishop Slip Circle Analysis** with circles at increasing radii;
- **Janbu Non-Circular Analysis** through the slope;

A summary of the results of the SLOPE runs for the unconfined side slope liner are presented in **Table SRA15** below:

Table SRA15: Summary of SLOPE Runs for Un-confined Side Slope Liner

Reference	Description	Lowest Factor of Safety
1	Stability of Un-confined Side-Slope Liner, Total Stress Analysis, Cu=50kPa, Phi=0^0	1.72
2	Stability of Un-confined Side-Slope Liner, Total Stress Analysis, Cu=75kPa, Phi=0^0	2.18
3	Stability of Un-confined Side-Slope Liner, Effective Stress Values, C'=5kPa, Phi'=25^0	1.43
4	Stability of Un-confined Side-Slope Liner, Effective Stress Values, C'=0kPa, Phi'=25^0	1.35

The factors of safety in the short-term using the un-drained values (total stress analysis) are higher than the factors of safety using effective stress (long-term) values, in contrast to the side-slope sub-grade. This is because the very soft Lower Alluvium sub-grade slope stability benefits from side-slope liner placement in the short-term. However, when undertaking an effective stress analysis, the installation of the side-slope liner lowers the shear strength of the sub-grade (as a result of the increase in pore water pressure) lowering the factor of safety.

Notwithstanding this, the factors of safety for the short and long-term situation were all in excess of 1.3. The key factor in the short and long-term was shown to be the sub-grade strength. A graphical representation of the results is shown in **Appendix SRA2**.

<u>Confined Side Slope Liner (Integrity)</u>

A summary of the maximum strains in each element of the side slope lining system are shown in **Table SRA16** below:

Table SRA16: Summary of Maximum Strains & Lowest Factors of Safety for Side Slope Lining System Elements

Scenario	**2mm HDPE Geo-membrane Side Slope FML Liner**		**1m Compacted Clay Liner (Side Slope Mineral Liner)**	
Activity/Factor of Safety	Maximum Tensile Stress (kN/m)	Maximum Tensile Strain* (%)	Maximum Horizontal (Tensile) Strain (%)	Maximum Vertical (Shear) Strain (%)
Side-Slope Liner - Worst Case (Intermediate)	1.37kN/m	0.57%	0.25%	4.50%
Side Lope Liner - Worst Case (Final)	2.05kN/m	0.85%	0.50%	6.50%
Permeability & Strain Guidance Limit	**-**	**3%** (Peggs et al)	**1.3%** (Edelmann et al 1999)	**10%** (Arch et al (1996)
Lowest Factor of Safety**	**-**	**3.53**	**2.60**	**1.54**

* The strain values have been derived from the tensile strength values according to the manufacturer's QC test results.
** Based on research work referenced in the Guidance.

2.7.5 Waste Analyses

When considering the stability of the waste mass and temporary waste slopes, the stability of the confined side-slope lining system, and the confined perimeter bunds, must also be addressed.

The side-slope lining system is also considered under this heading because the risk of a non-circular translational slip along the interface with the sidewall lining system exists. In order to undertake the stability assessment for the waste mass, three potential modes of failure have been considered, namely:

- Failure Mode 1 - Critical slip surfaces passing solely through the waste;
- Failure Mode 2 - Critical slip surfaces passing through the waste and along the side liner;

All the potential failure modes highlighted above have been analysed for the scenario of the temporary waste batter. The analysis has considered the stability of the components in terms of circular and non-circular 2-D limit equilibrium using the computer program SLOPE.

<u>Failure Mode 1</u>

The section analysed is based upon a worst-case scenario, which is located within Cell 3 for a south facing temporary waste slope formed as a result of placing waste against the perimeter side-slope. It has been assumed that the waste slope will be constructed at an overall slope of 1 in 2.5. The distribution of pore fluid pressure may vary within the waste mass, due to a number of factors. For non-hazardous waste, the most likely reason for increased pore fluid pressures is where saturated low permeability waste is placed and subsequently loaded by future waste. Two pore fluid pressure conditions have been assigned to the waste mass in the analyses.

The first condition assumes that the pore water pressure within the waste is represented by the phreatic surface of the leachate, at 2m above the top of the basal liner. The second condition adopts a r_u value of 0.1, which essentially represents a more conservative condition, which describes the pore fluid pressure regime as a function of the slope height at any given point. This can be used to reflect the development of excess pore pressures, as the waste is raised.

The results presented in **Table SRA14** below represent the calculated factors of safety for a critical slip surface that passes solely through the waste (Failure Mode 1) assuming a circular slip plane and effective stress parameters. A graphical representation of the failure mode is presented in **Appendix SRA3**. Additional cases were used to assess the reduction in the factor of safety between the anticipated effective stresses for varying pore fluid pressure conditions within the waste mass. As can be seen, the factor of safety exceeds the acceptable level for all cases considered.

Failure Mode 2

The section analysed was also based upon a worst-case scenario, which is located within Cell 3 for a south facing temporary waste slope formed as a result of placing waste against the perimeter side-slope. At this location, it has been assumed that the waste slope will be constructed at an overall slope of 1 in 2.5, from the toe at the inter-cell bund to a height of 20m.

Failure Mode 2 considers a critical slip surface that passes along the side-slope lining system and through the waste. The critical interface within the side-slope lining system is that between the waste and the FML geo-textile protection layer. Both peak and residual shear strength conditions for this interface were examined. The variation of pore water pressures in the waste previously used for the investigation of Failure Mode 1 (critical slip surfaces occurring solely within the waste) were applied to the investigation of Failure Mode 2. The results presented in **Table SRA17** represent the calculated factors of safety for the Failure Mode 2 analyses, assuming a non-circular slip plane and effective stress parameters.

The slope was examined following placement of waste in lifts. Values for the factor of safety (FOS) against slip failure for the proposed system are shown in **Table SRA17** below:

Table SRA17: Summary of SLOPE Runs for Waste Mass Stability

Reference	Analysis	Slip	Lowest Factor of Safety
Stability of Waste – Failure Mode 1 ru=0.0	Bishop	Circular	1.68
Stability of Waste - Failure Mode 1, ru=0.1	Bishop	Circular	1.49
Stability of Waste – Failure Mode 2, Peak Values, ru=0.0	Janbu	Non-Circular	2.52
Stability of Waste – Failure Mode 2, Post Peak Values, ru=0.0	Janbu	Non-Circular	2.20

Stability of Waste – Failure Mode 2, Peak Values, ru=0.1	Janbu	Non-Circular	2.32
Stability of Waste – Failure Mode 2, Post Peak Values, ru=0.1	Janbu	Non-Circular	2.04

Only the lowest factors of safety achieved are shown in the summary table above. Increasing the effective cohesion of the interface increases the factor of safety further.

Temporary Waste Slope Stability

In order to minimise strains in the side-slope lining system, it is proposed that all temporary waste batters will be 1 in 2.5. Following the analysis outlined above, a detailed analysis of all the temporary waste slopes was not believed to be necessary.

2.7.6 Capping Analyses

Stability of Capping & Restoration System - Incorporation of Equipment Loads

The key areas requiring analysis are the **stability** of the capping system (and cover soils) as a result of critical plant movements, and the **integrity** of each element of the capping system as a result of the compression within the sub-grade during construction, and the subsequent restoration period.

The placement of cover soil on a slope with a relatively low shear strength inclusion (such as a geo-membrane) should **always** be from the toe upward, towards the crest. This way, the gravitational forces of the cover soil and live load of the construction equipment are compacting previously placed soil and working with an ever present passive wedge and stable lower portion beneath the active wedge. While it is prudent to specify low ground pressure equipment to place the soil, the reduction of the FOS value from no equipment load while up the slope will be seen to be minimal.

For soil placement down the slope, however, a stability analysis must add an additional dynamic stress into the solution. This stress decreases the FOS value, and in some cases, to a great extent. Unless absolutely necessary, the design must consider the dynamic force of the construction placement equipment.

Static or Constant Velocity Plant Loads on the Capping System

For the case of a *D6 Bulldozer* pushing cover soil up from the toe of the slope to the crest, the calculation uses the free body diagram in **Appendix SRA4**, as the basis for the assessment.

This analysis adds the specific piece of plant (characterized by the weight and subsequent ground bearing pressure) and dissipates this force (or stress) through the cover soil thickness, to the interface of the geo-membrane.

Upon determining the additional equipment load at the cover soil-to-geo-membrane interface, the analysis proceeds as shown in **Appendix SRA4**, but with an additional force down (and parallel to) the slope. This additional force is equivalent to the weight of the plant load resolved parallel to the slope, and adjusted to reflect the reduction of this load on the interface in question, as a result of distribution through the cover soil.

By resolving the plant load into forces parallel and perpendicular to the slope, it can be seen there is an additional load, which increases the frictional resistance to movement, or sliding. It is a well-documented proof that, if the analyses were undertaken in an infinite slope situation, the net effect of additional loads acting vertically is **neutral**, as far as translational slope stability is concerned. However, as the passive wedge at the base of the slope (finite slope analysis) remains the same, the factor of safety is reduced, although only slightly.

Accelerating (or Decelerating) Plant Loads on the Capping System

For the case of a *D6 Bulldozer* pushing cover soil down from the crest of the slope to the toe, the analysis again uses the force diagram **Appendix SRA4**. However, this time an additional force (on top of the forces from the static load) must be included, resulting from the acceleration or deceleration of the equipment.

The magnitude of this force is equipment operator dependent and related to both the equipment speed and the time to reach the speed (or time to stop). Again, this additional force from accelerating (or decelerating) must be distributed through the cover soil thickness, to determine the force per unit width at the interface in question.

Analysis of the stability of the system has been assessed for the following scenarios:

- Without plant movements;
- With static plant on the surface (or plant travelling at a constant velocity);
- With construction plant pushing cover soil up the slope (from the toe of the slope, to the crest) and;
- With construction plant (accelerating, decelerating or stopping) pushing cover soil down from the crest of the slope, to the toe.

A summary of the stability calculations for the cap is presented in **Table SRA18**.

Table SRA18: Summary of Stability Analyses for Plant on Capping System – Constant Velocity & Accelerating/Decelerating

Reference	Description	Lowest Factor of Safety
105m Long Slope at 1 in 5.4	Stability of Capping System, Long Term, Cu=0.5kPa, Phi'=25, No Additional Plant Loading	2.29
45m Long Slope at 1 in 2.8	Stability of Capping Slope, Long-Term, C'=0.5kPa, Phi'=25, No Additional Plant Loading	1.29
105m Long Slope at 1 in 5.4	Stability of Capping System, Long Term, Cu=0.5kPa, Phi'=25, Worst Case Plant Loading Static or Travelling at Constant Velocity	2.24
45m Long Slope at 1 in 2.8	Stability of Capping Slope, Long-Term, C'=0.5kPa, Phi'=25, Worst Case Plant Loading Static or Travelling at Constant Velocity	1.27
105m Long Slope at 1 in 5.4	Stability of Capping System, Long-Term, Cu=0.5kPa, Phi'=25, Worst Case Plant Accelerating (or Decelerating) Down the Slope from zero to 20km/hour (or vice versa) in 3 seconds	2.02
45m Long Slope at 1 in 2.8	Stability of Capping System, Long-Term, Cu=0.5kPa, Phi'=25, Worst Case Plant Accelerating (or Decelerating) Down the Slope from zero to 20km/hour (or vice versa) in 3 seconds	1.09

Integrity of Capping System – Plant & Equipment Loads

A summary of the maximum strains in each element of the composite capping system is shown in **Table SRA19** below. A two-dimensional finite element model for each of the critical sections shown on the attached figures has been used for the determination the maximum strains.

Table SRA19: Summary of Maximum Strains & Lowest Factors of Safety for Capping System Elements

Low Permeability Capping Material	**1mm Linear Low Density Polyethylene Flexible Membrane Cap**	
Activity/Factor of Safety	Maximum Tensile Stress (kN/m)	Maximum Tensile Strain (%)*
20 Tonne D6 Dozer/20 Tonne 360 Degree Excavator on Tracks	0.27kN/m	0.33%
JCB 3CX Backhoe (2 Wheel Axles) Excavator on Capping System	0.27kN/m	0.33%
Fully Laden Dumper (3 Wheel Axles) on Stone Access Road Above Capping System	0.89kN/m	1.07%
Permeability & Strain Guidance Limit	-	**10%** (Peggs et al 2003)
Lowest Factor of Safety**	**-**	**9.35**

** The strain values have been derived from the tensile strength values according to the manufacturer's QC test results for the stress to strain relationship.*
*** Based on research work referenced in the Guidance.*

A summary of the calculation results of the deformations of the cap and the geo-synthetic components resulting from the loading regime modelled are shown in **Appendix SRA5.**

2.8 Assessment

2.8.1 Basal Sub-Grade Assessment

Assessment of the stability of the basal sub-grade is considered under **Section 2.8.3** below.

2.8.2 Side Slopes Sub-Grade Assessment

The factors of safety for the side-slope sub-grade stability achieve the required value of 1.3, except for the existing perimeter outer side slope (Slope 4), which is noticeably less.

<u>Assessment of Inner Side Slope Sub-grades</u>

By inspection, if the un-confined side slope sub-grade is stable, then it might be concluded that the confined side-slope, following placement of the side-slope liner, will also be stable. This is certainly true when considering short-term (un-drained) values, as placement of the side-slope liner increases the factor of safety. However, care must be exercised when considering the side-slope stability using effective stress values, as installation of the side-slope liner increases pore water pressures within the cohesive side slope sub-grade, and actually reduces the factor of safety. Notwithstanding the above, the factors of safety are adequate, and although the placement of waste will increase pore water pressures further, the lateral load of the waste increases the factor of safety.

<u>Assessment of Slope 4</u>

Although the factors of safety for Slope 4 do not achieve the required values, the factors of safety (and the most likely failure mechanism) remain the same before and after landfill is placed on the inner side of this prepared slope. This failure mechanism is also unlikely to have a significant impact upon the containment lining system, as long as a sufficient standoff is maintained. In addition, the analyses undertaken ignore negative pore water pressures in the cohesive embankment, and the effect of the existing trees and tree roots, which have arguably ensured stability in the past, and should ensure stability in the foreseeable future.

It is now well understood that that vegetation (and trees in particular) aid slope stability in cohesive soils through the removal of water by evapo-transpiration. This gives an increase in soil pore water suctions in the slope, increasing the soil shear strength. However, the effect of vegetation suctions and the impact of cycles of seasonal wetting and drying on slope stability is currently ignored in engineering practice due to a lack of real data on the modification of soil moisture regimes by different types of vegetation.

Trees will generate substantial leaf litter, even in poor soils. Leaf litter may tend to inhibit the development of an herbaceous under storey, but provides cover against raindrop erosion. The effects of weathering, possible splash erosion from direct precipitation at the edge of the under storey, and raindrops penetrating through foliage, as well as stem flow down the tree trunk during heavy rainfall may have a cumulative erosional effect on the bank soil. However, compared to an un-vegetated, or fallow state, slopes covered by a good stand of close growing vegetation experience an increase in erosion resistance of between one and two orders of magnitude. This statement is based on the work of Carson & Kirkby (1972) and Kirkby & Morgan (1980). Vegetation not only protects the soil surface directly, but the roots and rhizomes of plants bind the soil and introduce extra cohesion over and above any intrinsic cohesion that the slope may have.

Therefore, in light of the above findings, the following precautionary recommendation is made - The distance between the top of the steep tree-covered batter and the landfill containment engineering used in the analyses should be maintained right along the tree-covered batter. This should ensure that any future (long-term) slope failure does not interfere with the landfill, and the landfill does not interfere with any likely failure mechanism.

As this precaution will affect only a small section of the western perimeter, the recommendation is unlikely to impact significantly upon the conceptual design, or the void. However, should this prove unsatisfactory, further location-specific soil parameters shall be gained, and the safety factor quantified as a result of the established trees, as part of the detailed design of the side-slope lining system adjacent to this area.

2.8.3 Basal Liner Assessment

From the results presented above, the basal liner integrity is deemed satisfactory for the scenarios considered. In the following sections, the maximum strain results presented in **Table SRA14** and the acceptable strain limits (from peer reviewed published papers) utilised to calculate the factors of safety, are discussed.

The finite element analyses undertaken have examined the basal liner under a number of scenarios judged to represent the most critical stages of development for the lining system as a whole. A summary of the maximum horizontal (tensile) strains and vertical (shear) strains for the basal lining system is presented in **Table SRA14** above.

For the purposes of assessing the integrity of the mineral liner, it is necessary to select a suitable criterion, which can relate the model's reported output to permeability. This design criterion requires some understanding of the permeability-strain relationship of cohesive materials. While no exact site specific data exist with respect to this, research by Arch et al (1996) has shown that permeability of compacted clays (normally and over-consolidated) tends to decrease for strains up to the yield point of the material (typically 6%) after which increases in permeability are exhibited. However, values above the original permeability of the compacted clay are only indicated after much larger strains (around 11%).

For the purposes of this report, a design criterion value of 10% shear strain has been adopted, since this represents a point at which permeability remains within the as-compacted specification. As the maximum expected vertical shear strain in the CCL is 7.5%, a factor of safety greater than 1.3 exists for the mineral liner integrity.

Edelmann et al (1999) undertook an assessment to determine the effect of permeability of horizontal tensile strain. The details of this experiment are detailed in the paper presented in the Journal of Geotechnical Engineering, also outlined in the Guidance. At horizontal strains of 1.3% for a clay with very similar Atterburg limits (and the same plasticity index classification to the proposed mineral lining material at Pen-y-Bont) no measurable increases in permeability were found. Jessberger & Stone (1991) also outline the benefits of confinement in minimising tension cracking within mineral liners subjected to differential loading. Due to the in-filling methodology proposed, this situation also occurs in the final stage of land-filling, where confinement of the mineral lining system is greatest.

The maximum horizontal strain calculated in the CCL, as part of the numerical analyses, was 0.75%. This also equates to a factor of safety in excess of 1.3.

To conclude, the factors of safety for integrity of the composite basal lining system are all in excess of 1.3. For this reason, it is concluded that the basal lining system proposed meets the requirements for long-term stability, and integrity. It is also concluded that the assumptions made in the hydro-geological risk assessment may be relied upon.

2.8.4 Side Slope Liner Assessment (Un-confined & Confined)

Stability of Unconfined & Confined Side Slope Liner

The factors of safety for the side-slope liner stability achieve the required value. The use of conservative values, and site observations, ensures confidence in the results. By inspection, if the un-confined side slope liner is stable, then the confined side-slope, following waste deposition, will also be stable.

However, care must be exercised when considering the placement of the side-slope liner using effective stress values, as installation of the side-slope liner increases pore water pressures within the cohesive side slope sub-grade, and actually reduces the factor of safety. Notwithstanding the above, the factors of safety are still in excess of 1.3 and although the placement of waste will increase pore water pressures further, the lateral load of the waste increases the factor of safety.)

The factors of safety against a translational side-slope liner slip when unconfined are all acceptable in the short and long-term. The factors of safety calculated are, for the most part, significantly greater than those required. This assessment demonstrates that the side slope lining system proposed for Pen-y-Bont Landfill can meet the basic requirements for stability in the short, and more importantly, long-term.

From the results presented above, the side slope liner (and side slope sub-grade) is deemed stable for the scenarios considered. However, it is the integrity of the confined side slope lining system, which is the critical aspect in this situation. In the following sections, the maximum strain results presented in **Table SRA15** and the acceptable strain limits (from peer reviewed published papers) utilised to calculate the factors of safety, are discussed.

The finite element analyses undertaken have examined the side slope liner under a number of scenarios judged to represent the most critical stages of development for the lining system as a whole. A summary of the maximum horizontal (tensile) strains and vertical (shear) strains the side slope lining system is presented in **Table SRA15** above.

Integrity of Confined Mineral Side-Slope Liner

For the purposes of assessing the integrity of the clay element of the side-slope liner, it is necessary to select a suitable criterion, which can relate the model's reported output to permeability. This design criterion requires some understanding of the permeability-strain relationship of cohesive materials. While no exact site specific data exist with respect to this, research by Arch et al (1996) has shown that permeability of compacted clays (normally and over-consolidated) tends to decrease for strains up to the yield point of the material (typically 6%) after which increases in permeability are exhibited. However, values above the original permeability of the compacted clay are only indicated after much larger strains (around 11%).

For the purposes of this report, a design criterion value of 10% shear strain has been adopted, since this represents a point at which permeability remains within the as-compacted specification. As the maximum expected vertical shear strain in the mineral side-slope liner is 6.50%, a factor of safety greater than 1.3 exists for mineral liner integrity.

Edelmann et al (1999) undertook an assessment to determine the effect of permeability of horizontal tensile strain. The details of this experiment are detailed in the paper presented in the Journal of Geotechnical Engineering, also outlined in the Guidance. At horizontal strains of 1.3% for a clay with very similar Atterburg limits (and the same plasticity index classification to the proposed mineral lining material at Pen-y-Bont) no measurable increases in permeability were found. Jessberger & Stone (1991) also outline the benefits of confinement in minimising tension cracking within mineral liners subjected to differential loading. Due to the in-filling methodology proposed, this situation also occurs in the final stage of land-filling, where confinement of the mineral lining system is greatest.

The maximum tensile strain calculated in the side slope liner, as part of the numerical analyses, was 0.50%. This also equates to a factor of safety in excess of 1.3.

Integrity of Confined Geo-synthetic (FML) Side Slope Liner

The placement of compressible waste against the side-slope liner will induce tensile stresses in the system. Although the percentage horizontal strain at yield and subsequently break, is typically in excess of 20% for FMLs, BAM in Germany place a limit of **3%** long-term strain on HDPE geo-membrane liners to avoid stress-cracking problems for a period of at least 100 years. This requirement is based on the work of Seeger and Müller (2003). Peggs et al (2003) has recommended maximum strains for different materials as follows:

· HDPE smooth SCR <1500 hr - 6%
· HDPE smooth SCR >1500 hr - 8%
· HDPE random texturing - 4%*
· HDPE structured profile - 6%*
· LLDPE density <0.935 g/cm3 - 12%
· LLDPE density >0.935 g/cm3 - 10%
· LLDPE random texture - 8%
· LLDPE structured profile - 10%
· PP un-reinforced - 15%

* *Synthetic side slope systems proposed for Pen-y-Bont Landfill Site*

The measurement of strain is used as an indirect measure of the stress that exists in a geo-membrane that might result in stress cracking. While this is clearly important for HDPE, it is not as significant for other materials that are not susceptible to stress cracking, unless oxidised. However, the objective is to limit stress to a sub-critical value where stress cracking will not be a practical problem.

As can be seen from the analysis results outlined above, the strains in the geo-synthetic side slope FML are all less than the 3% limit currently used for HDPE.

It should be noted that (as far as **leachate** management is concerned) sidewall lining systems function to ensure that any perched leachate is discouraged from lateral migration, and to ensure that unacceptable leachate discharges are prevented. (An artificial sealing liner (ASL) is **not** required to extend up the sidewall of the landfill.)

To conclude, the factors of safety for integrity of the side-slope lining system are all in excess of 1.3. For this reason, it is concluded that the side slope lining system proposed meets the requirements for long-term stability, and integrity. It is also concluded that the assumptions made in the hydro-geological risk assessment may be relied upon.

2.8.5 Waste Assessment

The lowest factor of safety for the most critical temporary waste slope was 1.58. This was for a temporary waste slope immediately adjacent to the side-slope lining system. Although the situation modelled is unlikely to occur in reality, as the waste is proposed to be laid in horizontal lifts, the modelling of this more conservative scenario (see **Appendix SRA4**) was much simpler. Temporary waste slopes of steeper than 1 in 3 (up to 1 in 2.5) would be acceptable for the slope heights considered at Pen-y-Bont. However, waste slopes steeper than 1 in 2.5 have not been considered in the finite element analysis, and it is recommended that waste batters of approximately 1 in 2.5 are adhered to.

Leachate re-circulation should not affect the stability of waste mass, as the leachate is allowed to drain through the waste by gravity, therefore not affecting pore water pressures. However, an allowance for increased pore pressures due to the variable permeability of the waste mass, and the resultant loading, has been allowed for.

Analysis of the waste mass using pre and post-settlement contours is not required as the steepest slope experienced (on the site perimeter) is no greater than 1 in 2.8. An analysis of the slope gradient using the same parameters put into the temporary waste slope analysis would give factors of safety in excess of the required 1.3.

2.8.6 Capping Assessment

The factors of safety obtained for all aspects of the capping stability using conservative parameters are adequate. Settlements within the waste mass will have some effect on the integrity of the cap but the differential settlement expected around these edges of the cap is not envisaged to be detrimental in the long-term. Settlement in the adjacent waste mass will increase the factors of safety for stability, assuming even settlement. The factor of safety achieved is also judged to provide sufficient allowance for differential settlement.

From the analyses undertaken, it can be demonstrated that a smooth geo-membrane would be suitable for significant parts of the cap, without the need for the incorporation of a textured geo-membrane. However, when the forces resulting from any dynamic equipment loading are taken into consideration, a textured geo-membrane would be required for any slopes steeper than 1 in 6, with the assumed conservative parameters for these materials. (A textured geo-membrane may be utilised for the whole cap, as an additional safety factor.)

<u>Plant & Vehicle Movements - Stability of the Capping & Restoration System</u>

As can be seen from the analysis section above, the factors of safety for plant and vehicles on the capping system are in excess of 1.3, except for plant accelerating or decelerating down the very steep perimeter sections of the cap. As can be seen from the analysis section, there is an inherent danger with plant moving down the slope. It is also worth noting that the same result comes about by plant decelerating, instead of accelerating. The sharp breaking action is arguably the more severe condition due to the extremely short times involved when stopping forward motion. Clearly, only in unavoidable situations should cover soil placement equipment be allowed to work down the slope. If it is unavoidable, an analysis should be made of the specific situation and the construction specifications should reflect the exact conditions made in the analysis. As a minimum, the ground contact pressure of the equipment should be stated, along with suggested operator control of the cover soil placement operations.

Plant & Vehicle Movements - Integrity of the Geo-synthetic Cap (1mm LLDPE)

The placement of cover soils against any capped waste will induce tensile stresses in the capping system. However, it is the size of the differential settlements and angular distortion (differential settlement/distance) that are important to the tensile stresses in the cap. Although percentage horizontal strain at yield and subsequently break, is typically in excess of 20% for FMLs, BAM in Germany place a limit of 3% long-term strain on HDPE geo-membrane liners to avoid stress-cracking problems for a period of at least 100 years. This requirement is based on the work of Seeger and Müller (2003). Peggs et al (2003) has recommended maximum strains for different materials as follows:

· HDPE smooth SCR <1500 hr - 6%
· HDPE smooth SCR >1500 hr - 8%
· HDPE random texturing - 4%
· HDPE structured profile - 6%
· LLDPE density <0.935 g/cm3 - 12%*
· LLDPE density >0.935 g/cm3 - 10%
· LLDPE random texture - 8%
· **LLDPE structured profile - 10%***
· PP un-reinforced - 15%

* *Synthetic systems proposed for Pen-y-Bont Landfill Site*

The measurement of strain is used as an indirect measure of the stress that exists in a geo-membrane that might result in stress cracking. While this is clearly important for HDPE, it is not as significant for other materials that are not susceptible to stress cracking, unless oxidised. However, the objective is to limit stress to a sub-critical value where stress cracking will not be a practical problem.

As can be seen from the analysis results outlined above, the strains in the geo-synthetic cap are all significantly less than the 10% limit currently used for LLDPE.

Even after designing for plant movement, it is still advisable to stress that it is preferable for the protection material, and subsequent cover material, to be deposited by plant moving up the slope, as opposed to plant moving down the slope. The factors of safety for **integrity** of the capping (mineral or geo-synthetic) system are all in excess of 1.3. In addition, the factors of safety for **stability** of the capping system under plant loading are also all in excess of 1.3, with the exception of plant decelerating whilst travelling down the steepest parts of the capping system. The worst case is determined to be a vehicle pushing material down the slope during construction, whilst decelerating. For this reason, a number of recommendations are made in the monitoring section.

Landfill Gas Pressures – Stability of the Capping System

Gas pressures will be prevented from building up by the gas extraction system. Therefore, gas pressures are unlikely to affect the stability, or integrity, of the cap.

The methodology proposed in the paper by Thiel (1999) examines a failure of the gas extraction system leading to build up of positive gas pressure underneath the capping membrane across a large enough area to cause local instability, by lowering the normal stress (from the capping system protection layer and restoration soils) providing shear resistance to sliding. This paper recommends a gas drainage layer beneath the capping membrane, to provide a mechanism to prevent such a build up. However, the placement of a gas control layer in this vicinity brings with it a significant number of problems, which can actually exacerbate the failure mechanism it is designed to mitigate. These are listed below:

- The geo-composite materials utilised for gas drainage layers typically have lower interface friction angles between an overlying FML and than those which would normally exist between a cohesive bedding material and an overlying FML, as proposed at Pen-y-Bont. Indeed, the incorporation of a layer with a potentially low interface friction angle, coupled with the likelihood of positive gas pressure right across this interface (as would be possible with a layer such as this) is an aspect which it is wiser to design out.

- In utilising a cohesive material for the FML bedding layer, as proposed at Pen-y-Bont, positive gas pressure build up directly beneath large sections of the FML would be far less likely than with a gas drainage layer, in intimate contact with the FML, directly beneath the cap. In effect, the critical slip surface for analysis (where a positive gas pressure could reduce the normal loading (and the subsequent frictional resistance to sliding) would be at the interface between the waste mass and the bedding layer. By observation, the interface friction angle between these two materials would, arguably, be far greater than that between an FML (even if textured) and a non-woven needle punched geo-textile.

There are other important reasons for not incorporating a gas drainage layer, but these are not relevant to the stability assessment. However, the control of landfill gas at Pen-y-Bont will undertaken using vertical wells designed to create a slight negative pressure within the landfill, with a pressure gradient designed to draw landfill gas away from the perimeter of the landfill. The likelihood of failure of the gas extraction system for a period long enough for positive gas pressures to build up at the perimeter is also considered to be very low, due to management controls already in place at the site.

These are the reasons a gas control layer beneath the cap is not proposed and the same reasons explain why the methodology proposed by Thiel (1999) does not need to be addressed.

3. MONITORING

3.1 The Risk Based Monitoring Scheme

The risk of instability for Pen-y-Bont Landfill is generally considered to be low. However, given that monitoring is still being undertaken on the outer perimeter slope known locally as **Slope 2**, the recommendations relating to **Slope 2** and **Slope 3**, outlined in the conclusions of the **Entec Monitoring Data Review Report** for **Chirk Landfill Slope 2 Stability Remedial Works** dated **25th February 2004**, should be undertaken.

In addition to these recommendations, the **detailed design** of the side-slope lining system on the landfill side of **Slope 4** (engineered fill adjacent to the existing slope) should ensure that a sufficient stand-off is maintained, in accordance with the section analysed. This is required to ensure that the existing tree-covered outer slope is not adversely affected by the landfill proposals, and that any future (long-term) signs of instability (adjacent to the road) can be remedied without the risk of damage to the integrity of the adjacent engineered containment systems.

3.1.1 Basal Sub-Grade Monitoring

The basal sub-grade does not require any post-construction monitoring.

3.1.2 Side Slope Sub-Grade Monitoring

The side slope sub-grade does not require any post-construction monitoring.

3.1.3 Basal Lining System Monitoring

The basal lining system does not require any post-construction monitoring.

3.1.4 Side Slope Lining System Monitoring

The side-slope lining system does not require any post-construction monitoring.

3.1.5 Waste Mass Monitoring

The waste mass does not require any specific monitoring during the waste deposition phase, other than ensuring that safe waste profiles are maintained.

3.1.6 Capping System Monitoring

As the proposed final (pre-settlement) restoration contours at Pen-y-Bont are very steep, in comparison to other landfills, the following recommendation is made, in order to ensure stability of the capping system:

- **The placement of cover soil on a slope with a relatively low shear strength inclusion (such as a geo-membrane) should always be from the toe upward, towards the crest. This way, the gravitational forces of the cover soil and live load of the construction equipment are compacting previously placed soil, and working with an ever present passive wedge and stable lower portion beneath the active wedge. While it is prudent to specify low ground pressure equipment to place the soil, the reduction of the factor of safety value from no equipment load while working up the slope has been shown to be nominal.**

In addition to this recommendation, it must be noted that for the case of a fully loaded dump truck, a stone access road (or alternative loading spreading solution) will be required to keep strains (from the significant wheel loads) to within acceptable limits. For this reason, the following recommendation is also made:

- A fully loaded dump truck should not be allowed to travel over the capping system, or restoration soils, without a stone access road (or alternative load spreading solution) in order to avoid a loss of integrity to the geo-synthetic capping membrane.

It is considered that, although settlement may affect leachate and landfill gas extraction pipework, the drilling and installation of new facilities can replace these items. This means that the effect of waste settlement on such pipework is not critical to the management of leachate and landfill gas at the site.

Other than the above recommendations, the capping system does not require any monitoring, except for observational reporting of any significant erosion (before the stabilising effect of vegetation has taken over) so that any necessary repairs can be made.

4.0 References

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APPENDIX SRA1

SUMMARY OF PREVIOUS SITE INVESTIGATIONS & LABORATORY TEST RESULTS

ALLIED EXPLORATION AND GEOTECHNICS LIMITED

Unit 25 Stella Gill Industrial Estate, Pelton Fell
Chester-le-Street, Co.Durham. DH2 2RJ
a NAMAS LABORATORY Testing No. 1367

Consolidated Drained Shear Box Test

BS 1377 : PART 7 : 1990 Clause 4

Site: Chirk Landfill Slope Stability Site Investigation
Client: Shanks Waste Services — **Job No.** PR1115

Borehole: BH-02 — **Sample:** U2 — **Depth:** 1.20m

For sample description please refer to sample description sheet

Stage Number		1	2	3
Specific Depth	m	1.21	1.25	1.27
Length	mm	60	60	60
Height	mm	19.9	19.9	19.9
Initial Moisture Content	%	21.6	21.6	21.6
Initial Wet density	mg/m^3	2.04	2.07	2.09
Initial Dry density	mg/m^3	1.68	1.70	1.72
CONSOLIDATION				
Normal Stress	kPa	25	50	100
Height at end of Stage	mm	19.3	18.9	18.5
Duration	Day(s)	1	1	1
SHEARING				
Rate of Strain	mm/min	0.036	0.031	0.030
Peak Shear Stress	kPa	14.78	29.56	51.33
Displacement at Peak Stress	mm	6.45	4.84	3.70
Rate for Residual Runs	mm/min	N/A	N/A	N/A
Residual Shear Stress	kPa	N/A	N/A	N/A
Duration	Day(s)	1	1	1
Final Moisture Content	%	19.2	18.2	17.3
Final Wet Density	mg/m^3	2.06	2.12	2.17
Final Dry Density	mg/m^3	1.73	1.79	1.85

PEAK SHEAR STRESS PARAMETERS

Apparent Cohesion	c'	5
Angle of Shearing Resistance	ø'	26

RESIDUAL PARAMETERS

Apparent Cohesion		N/A
Angle of Shearing Resistance		N/A

REMARKS

DATE TESTED [illegible] — NAME [illegible]BELL
DATE OF ISSUE [illegible] — APPROVED BY

Pen-y-Bont SRA 1

Certificate No. : PR111503

ALLIED EXPLORATION AND GEOTECHNICS LIMITED

Unit 25 Stella Gill Industrial Estate, Pelton Fell
Chester-le-Street, Co.Durham. DH2 2RJ
a NAMAS LABORATORY Testing No. 1367

Consolidated Drained Shear Box Test

BS 1377 : PART 7 : 1990 Clause 4

Site: Chirk Landfill Slope Stability Site Investigation
Client: Shanks Waste Services **Job No.** PR1115

Borehole: BH-03 **Sample:** U9 **Depth:** 4.00m

For sample description please refer to sample description sheet

Stage Number		1	2	3
Specific Depth	m	4.01	4.03	4.05
Length	mm	60	60	60
Height	mm	20.0	20.0	20.0
Initial Moisture Content	%	17.6	17.6	17.6
Initial Wet density	mg/m^3	2.01	1.97	2.00
Initial Dry density	mg/m^3	1.70	1.68	1.70
CONSOLIDATION				
Normal Stress	kPa	80	160	320
Height at end of Stage	mm	18.9	17.8	16.4
Duration	Day(s)	1	1	1
SHEARING				
Rate of Strain	mm/min	0.024	0.023	0.024
Peak Shear Stress	kPa	49.78	89.83	194.44
Displacement at Peak Stress	mm	7.92	7.54	7.39
Rate for Residual Runs	mm/min	N/A	N/A	N/A
Residual Shear Stress	kPa	N/A	N/A	N/A
Duration	Day(s)	1	1	1
Final Moisture Content	%	19.2	18.8	16.6
Final Wet Density	mg/m^3	2.13	2.24	2.42
Final Dry Density	mg/m^3	1.79	1.89	2.08

PEAK SHEAR STRESS PARAMETERS

Apparent Cohesion	c'	1
Angle of Shearing Resistance	σ'	30

RESIDUAL PARAMETERS

Apparent Cohesion		N/A
Angle of Shearing Resistance		N/A

REMARKS

DATE TESTED 04.04 [illegible] NAME [illegible] McCONNELL
DATE OF ISSUE [illegible] 04 [illegible] APPROVED BY

Pen-y-Bont SRA 1

ALLIED EXPLORATION AND GEOTECHNICS LIMITED

Unit 25 Stella Gill Industrial Estate, Pelton Fell
Chester-le-Street, Co.Durham. DH2 2RJ
a NAMAS LABORATORY Testing No. 1367

Consolidated Drained Shear Box Test

BS 1377 : PART 7 : 1990 Clause 4

Site:	Chirk Landfill Slope Stability Site Investigation		
Client:	Shanks Waste Services	Job No.	PR1115
Borehole:	BH-04	Sample:	U1
Depth:	1.50m		

For sample description please refer to sample description sheet

Stage Number		1	2	3
Specific Depth	m.	1.52	1.55	1.58
Length	mm	60	60	60
Height	mm	20.0	20.0	20.0
Initial Moisture Content	%	22.4	22.4	22.4
Initial Wet density	mg/m^3	2.04	2.02	1.92
Initial Dry density	mg/m^3	1.67	1.65	1.57

CONSOLIDATION

Normal Stress	kPa	30	60	120
Height at end of Stage	mm	19.21	18.87	18.17
Duration	Day(s)	1	1	1

SHEARING

Rate of Strain	mm/min	0.014	0.012	0.011
Peak Shear Stress	kPa	19.50	37.13	59.11
Displacement at Peak Stress	mm	6.57	4.25	6.42
Rate for Residual Runs	mm/min	0.013	0.011	0.014
Residual Shear Stress	kPa	19.13	31.13	52.89
Duration	Day(s)	2	2	2
Final Moisture Content	%	22.8	22.9	22.7
Final Wet Density	mg/m^3	2.13	2.15	2.12
Final Dry Density	mg/m^3	1.74	1.75	1.73

PEAK SHEAR STRESS PARAMETERS

Apparent Cohesion	c'	9
Angle of Shearing Resistance	σ	24

RESIDUAL PARAMETERS

Apparent Cohesion	c'	[illegible]
Angle of Shearing Resistance	[illegible]	[illegible]

REMARKS [illegible]

DATE TESTED	[illegible]	NAME	[illegible]
DATE OF ISSUE	[illegible]	APPROVED BY	[illegible]

Pen-y-Bont SRA1

ALLIED EXPLORATION AND GEOTECHNICS LIMITED

Unit 25 Stella Gill Industrial Estate, Pelton Fell
Chester-le-Street, Co.Durham. DH2 2RJ
a NAMAS LABORATORY Testing No. 1367

Consolidated Drained Shear Box Test

BS 1377 : PART 7 : 1990 Clause 4

Site: Chirk Landfill Slope Stability Site Investigation
Client: Shanks Waste Services   **Job No.** PR1115

Borehole: BH-04   **Sample:** U8   **Depth:** 4.50m

For sample description please refer to sample description sheet

Stage Number		1	2	3
Specific Depth	m	4.52	4.54	4.56
Length	mm	60	60	60
Height	mm	20.0	20.0	20.0
Initial Moisture Content	%	13.4	13.4	13.4
Initial Wet density	mg/m^3	2.02	1.99	1.99
Initial Dry density	mg/m^3	1.78	1.76	1.76

CONSOLIDATION

Normal Stress	kPa	90	180	360
Height at end of Stage	mm	18.7	17.9	18.1
Duration	Day(s)	1	1	1

SHEARING

Rate of Strain	mm/min	0.026	0.025	0.024
Peak Shear Stress	kPa	63.00	106.17	238.00
Displacement at Peak Stress	mm	5.81	5.65	7.70
Rate for Residual Runs	mm/min	N/A	N/A	N/A
Residual Shear Stress	kPa	N/A	N/A	N/A
Duration	Day(s)	1	1	1
Final Moisture Content	%	16.5	16.2	12.1
Final Wet Density	mg/m^3	2.22	2.28	2.17
Final Dry Density	mg/m^3	1.91	1.96	1.94

PEAK SHEAR STRESS PARAMETERS

Apparent Cohesion	C'	3
Angle of Shearing Resistance	Ø	31

RESIDUAL PARAMETERS

Apparent Cohesion		N/A
Angle of Shearing Resistance		N/A

REMARKS

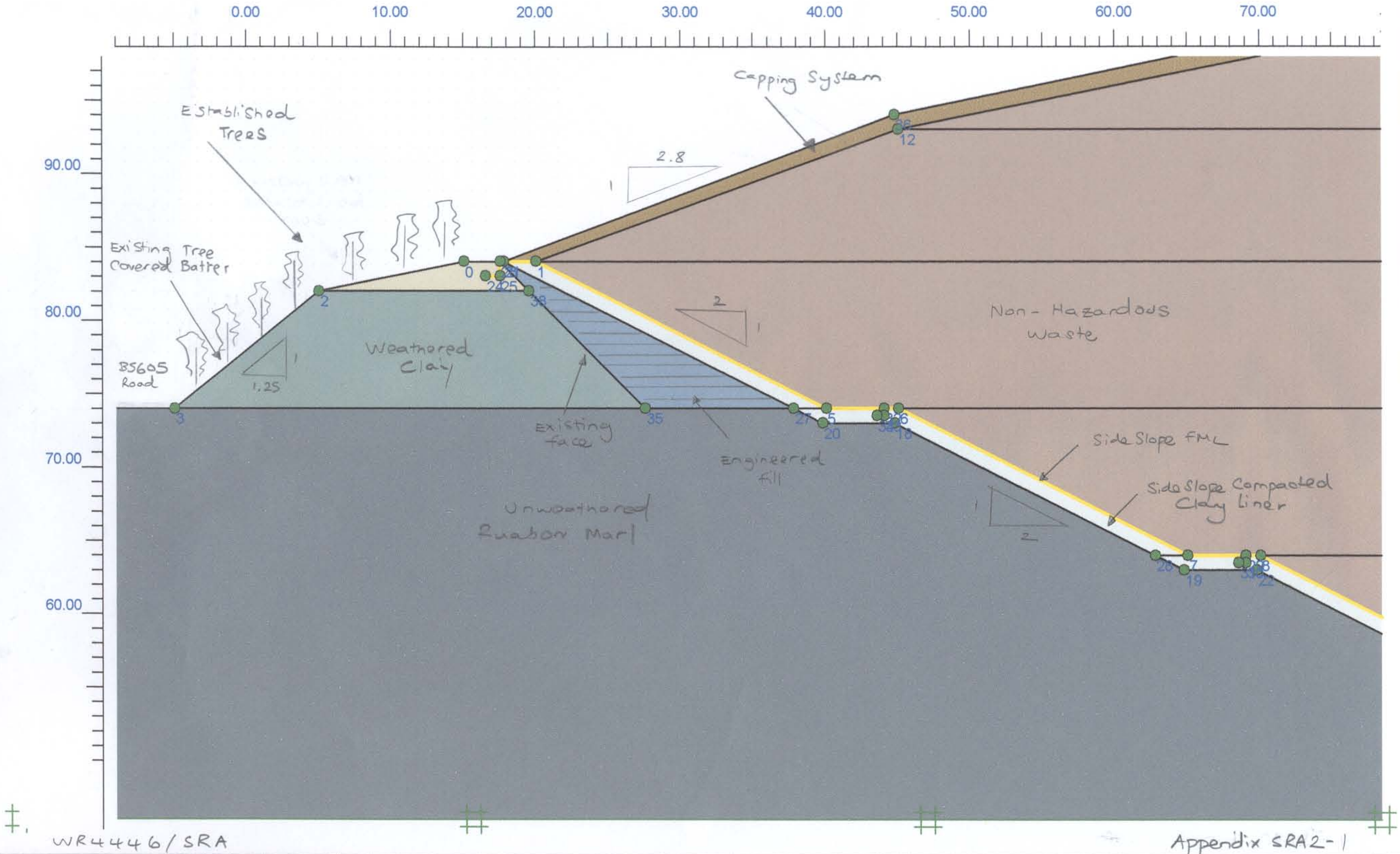
DATE TESTED
DATE OF ISSUE

NAME
APPROVED BY

Pen-y-Bont SRA 1

APPENDIX SRA2

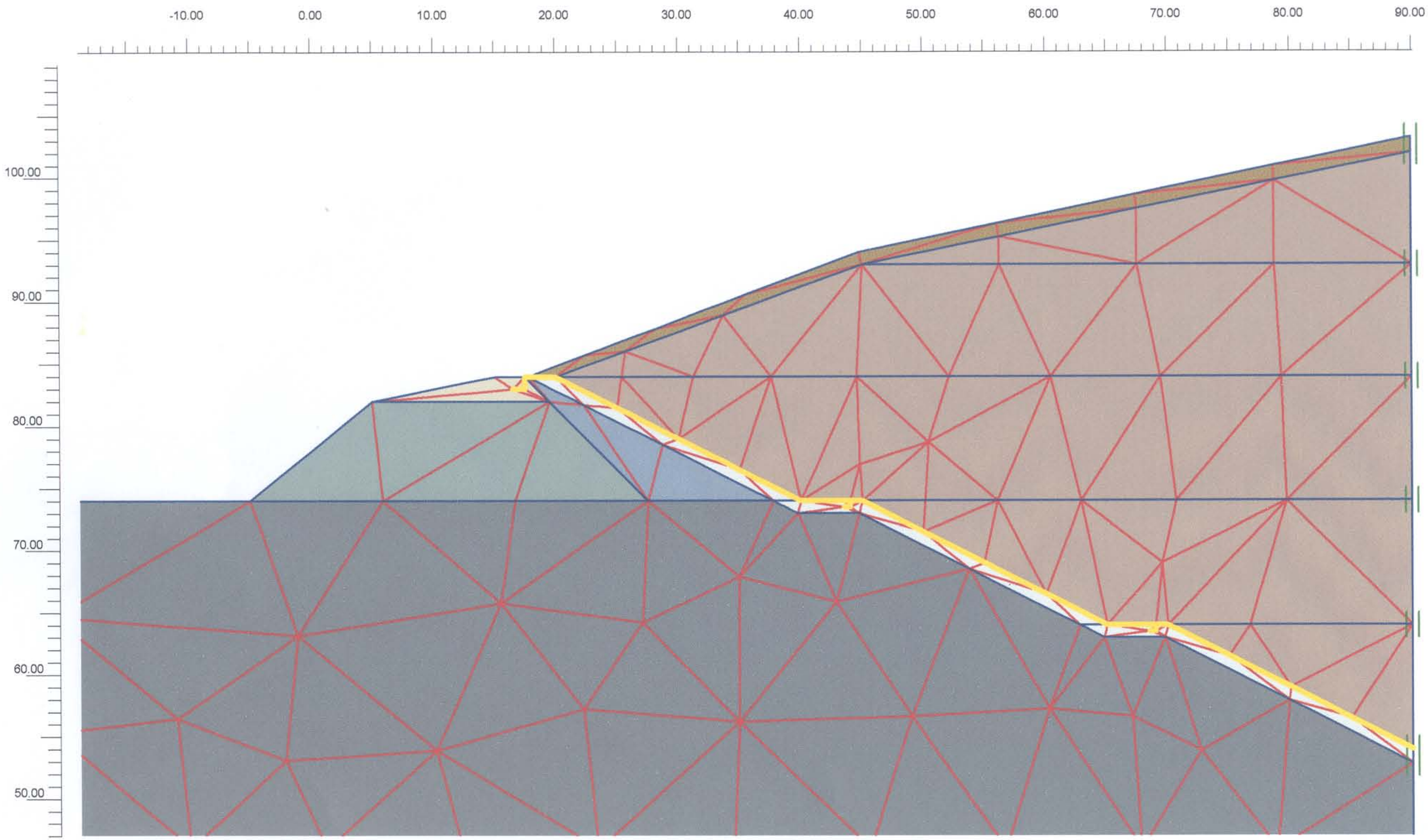
SIDE-SLOPE SUB-GRADE AND SIDE-SLOPE LINER STABILITY ANALYSES



PLAXIS Finite Element Code for Soil and Rock Analyses	Project description Western Perimeter overlooking B5605		
	Project name Side Slope.plx	Date 27/10/2004	User name Encia Consulting Limited

Version 8.2.4.811

PLAXIS V8



Connectivities

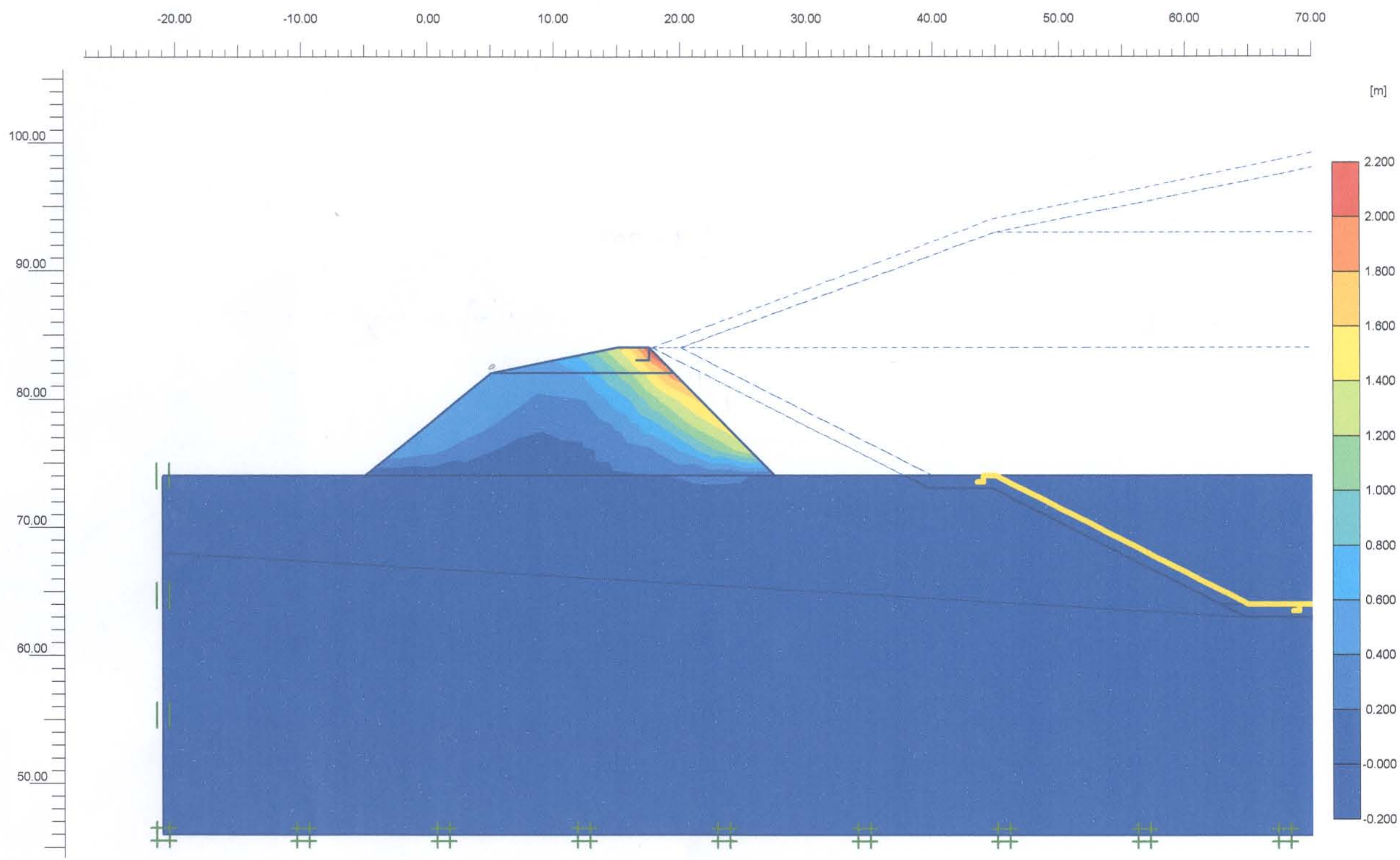
WR4446/SRA

Appendix SRA 2-2

PLAXIS Finite Element Code for Soil and Rock Analyses	Project description: South Eastern Side Slope Lining System Finite Element Mesh		
	Project name	Date: 27/10/04	User name: Encia Consulting Limited

Version 8.2.4.133

PLAXIS V8



Total displacements (Utot)
Extreme Utot 2.14 m

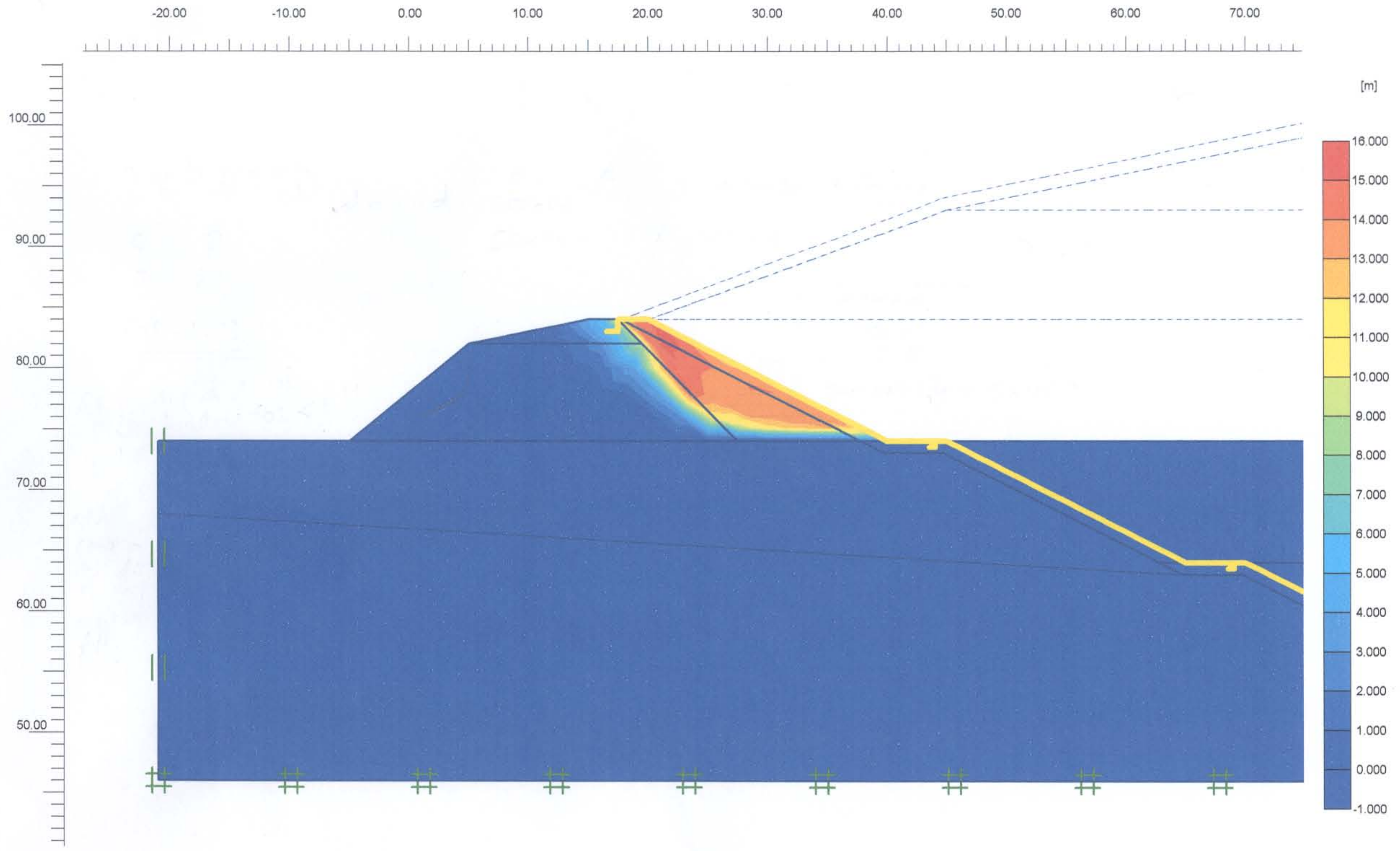
WR4446 / SRA

Appendix SRA 2-3

PLAXIS Finite Element Code for Soil and Rock Analyses	Project description: Existing Side Slope Subgrade Prior to Engineered Fill		
Project name: Side Slope	Step: 100	Date: 27/10/04	User name: Encia Consulting Limited

Version 8.2.4.133

PLAXIS V8



Total displacements (Utot)
Extreme Utot 15.22 m

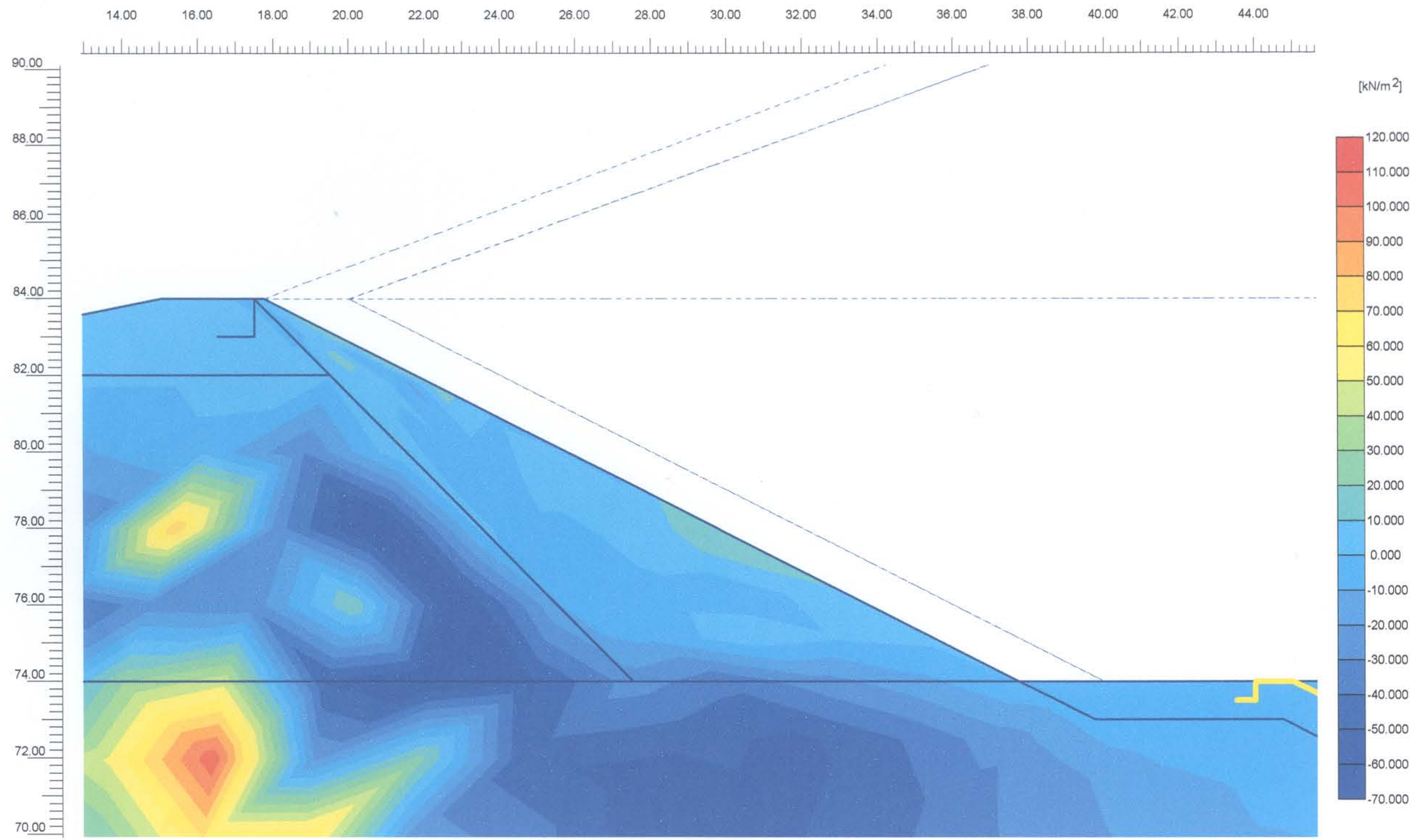
WR4446/SRA

Appendix SRA 2-4

PLAXIS Finite Element Code for Soil and Rock Analyses	Project description: Side Slope Engineered Fill - Failure Mechanism		
	Project name: Side Slope	Step: 97	Date: 27/10/04	User name: Encia Consulting Limited

Version 8.2.4.133

PLAXIS V8



Excess pore pressures
Extreme excess pore pressure 111.25 kN/m^2
(pressure = negative)

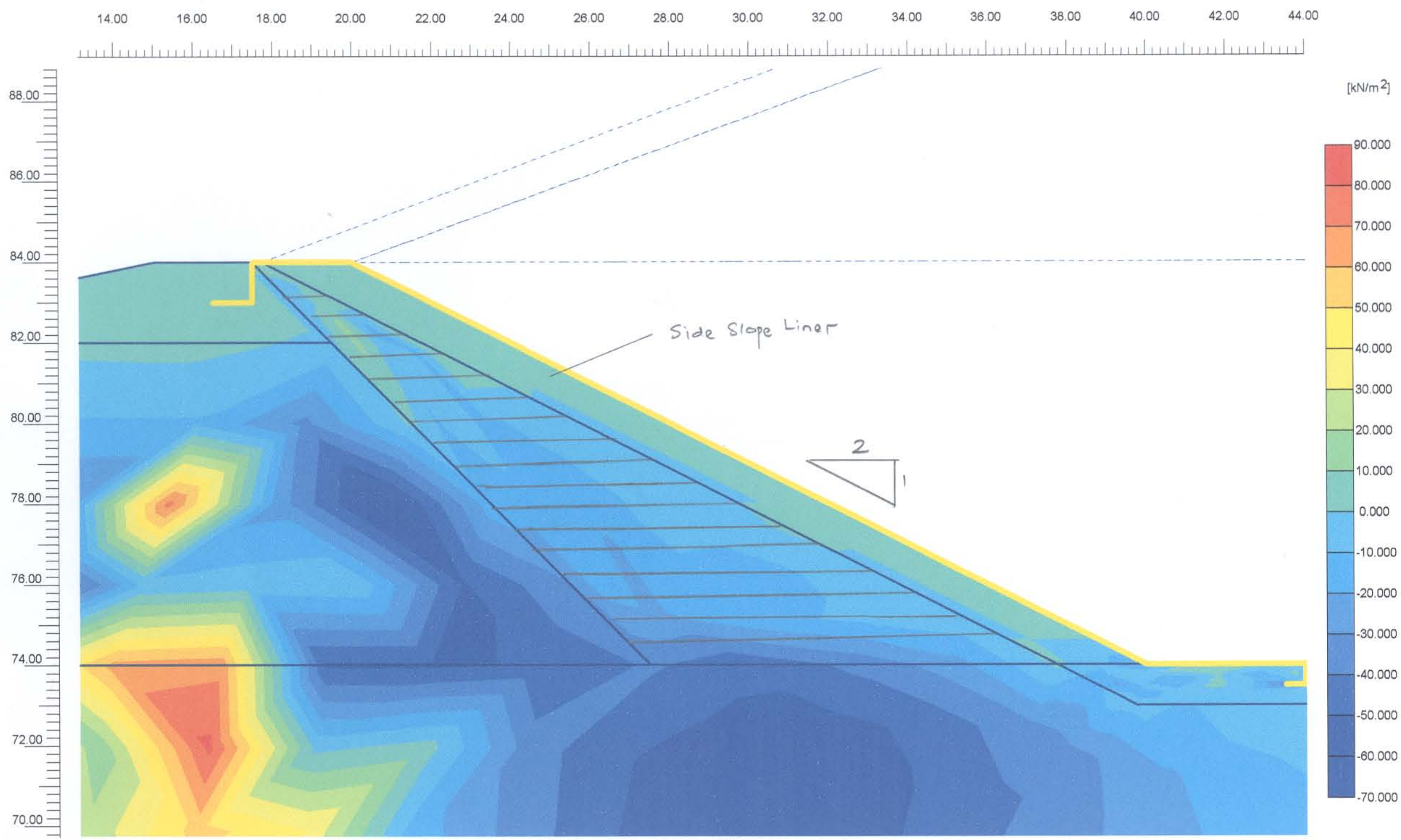
WR4446/SRA

Appendix SRA 2-5

PLAXIS Finite Element Code for Soil and Rock Analyses	Project description Excess Pore Water Pressures Prior to Installation of Side Slope Liner			
	Project name Side Slope	Step 134	Date 27/10/04	User name Encia Consulting Limited

Version 8.2.4.133

PLAXIS V8



Excess pore pressures
Extreme excess pore pressure 89.05 kN/m^2
(pressure = negative)

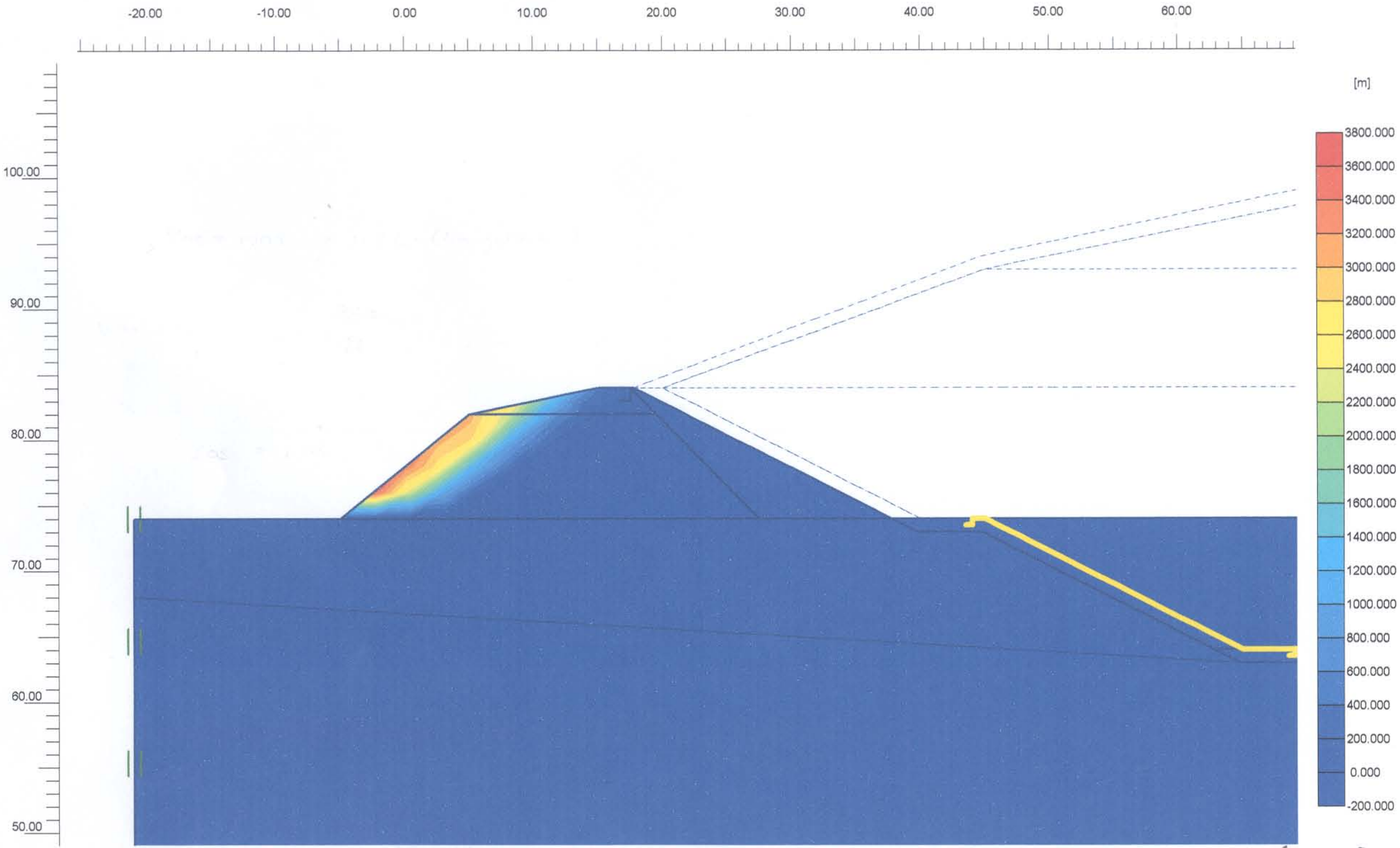
WR 444G / SRA

Appendix SRA 2-6

PLAXIS Finite Element Code for Soil and Rock Analyses	Project description: Excess Pore Water Pressures Following Installation of Side Liner			
	Project name: Side Slope	Step: 97	Date: 27/10/04	User name: Encia Consulting Limited

Version 8.2.4.133

PLAXIS V8



FoS = 1.08 to 1.22 (Long-Term)

Total displacements (Utot)
Extreme Utot $3.62*10^3$ m

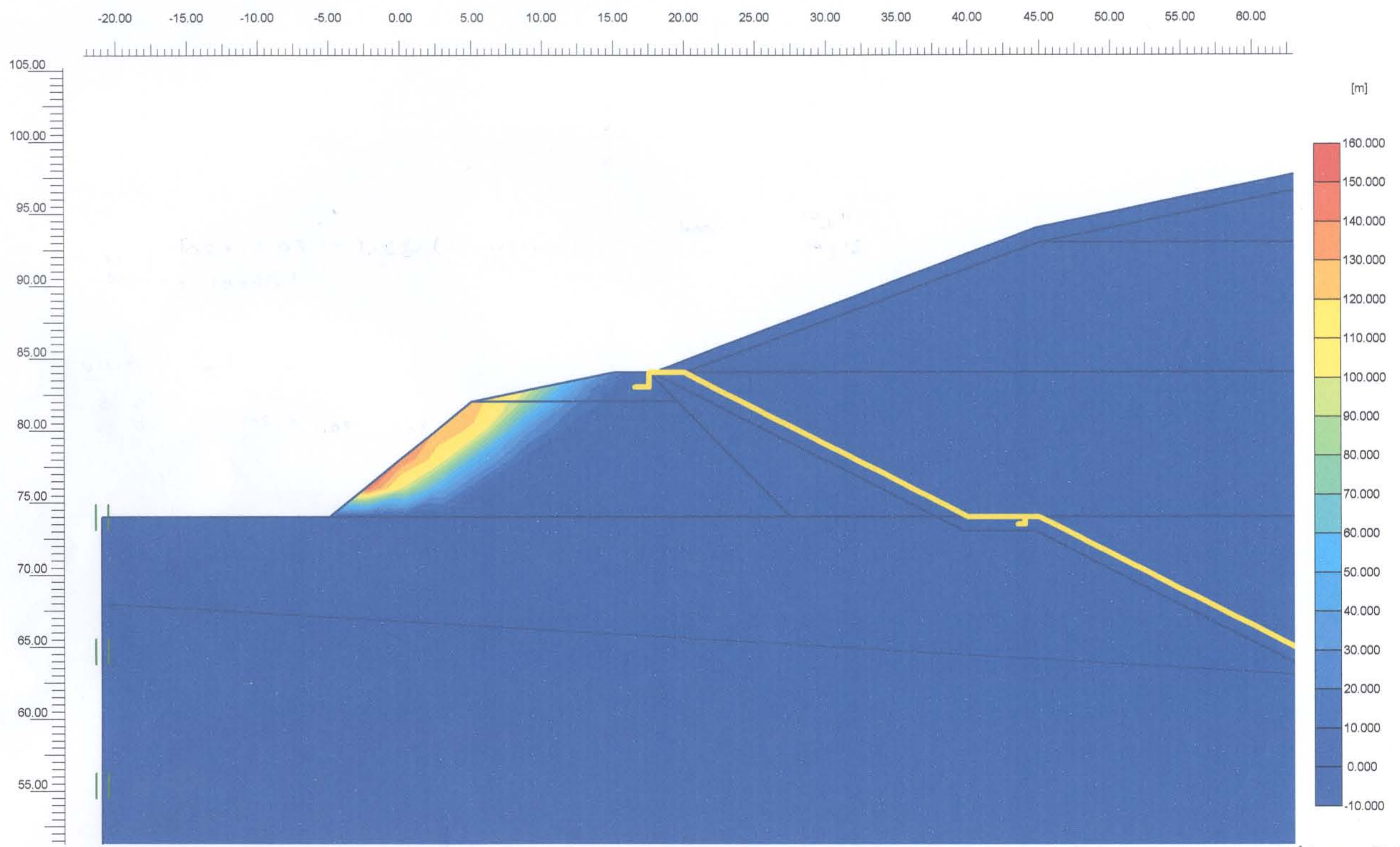
WR4446/SRA

Appendix SRA 2 - 7

PLAXIS Finite Element Code for Soil and Rock Analyses	Project description: South East Perimeter Outer Slope - Failure Mechanism		
	Project name: Side Slope	Step: 197	Date: 27/10/04	User name: Encia Consulting Limited

Version 8.2.4.133

PLAXIS V8



FOS = 1.08 to 1.22 (Long-Term)

Total displacements (Utot)
Extreme Utot 151.61 m

WR446 / SRA

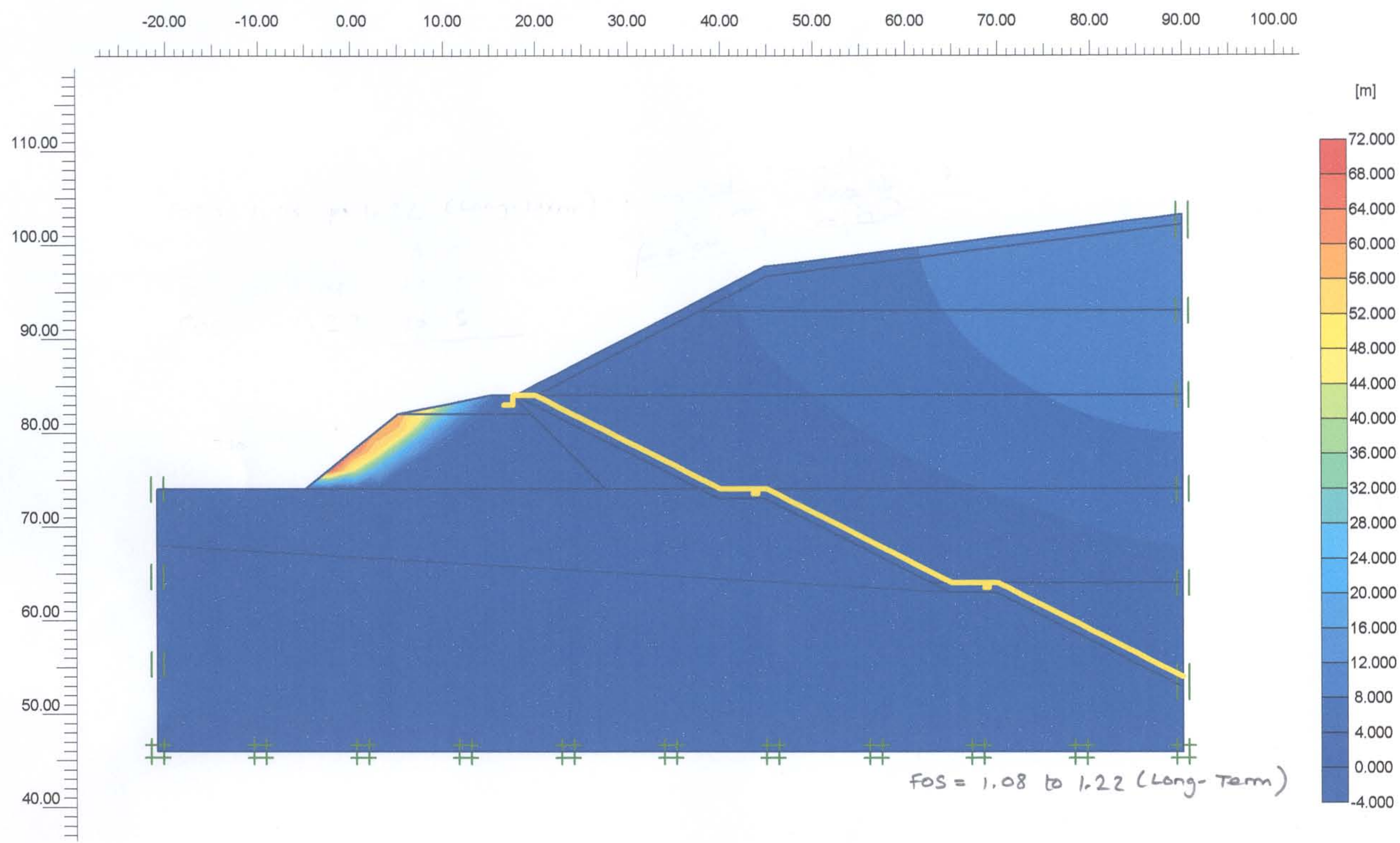
Appendix SRA 2-8

PLAXIS
Finite Element Code for Soil and Rock Analyses

Project description			
South East Perimeter Outer Slope - Failure Mechanism			
Project name	Step	Date	User name
Side Slope	197	27/10/04	Encia Consulting Limited

Version 8.2.4.133

PLAXIS V8



Total displacements (Utot)
Extreme Utot 70.17 m

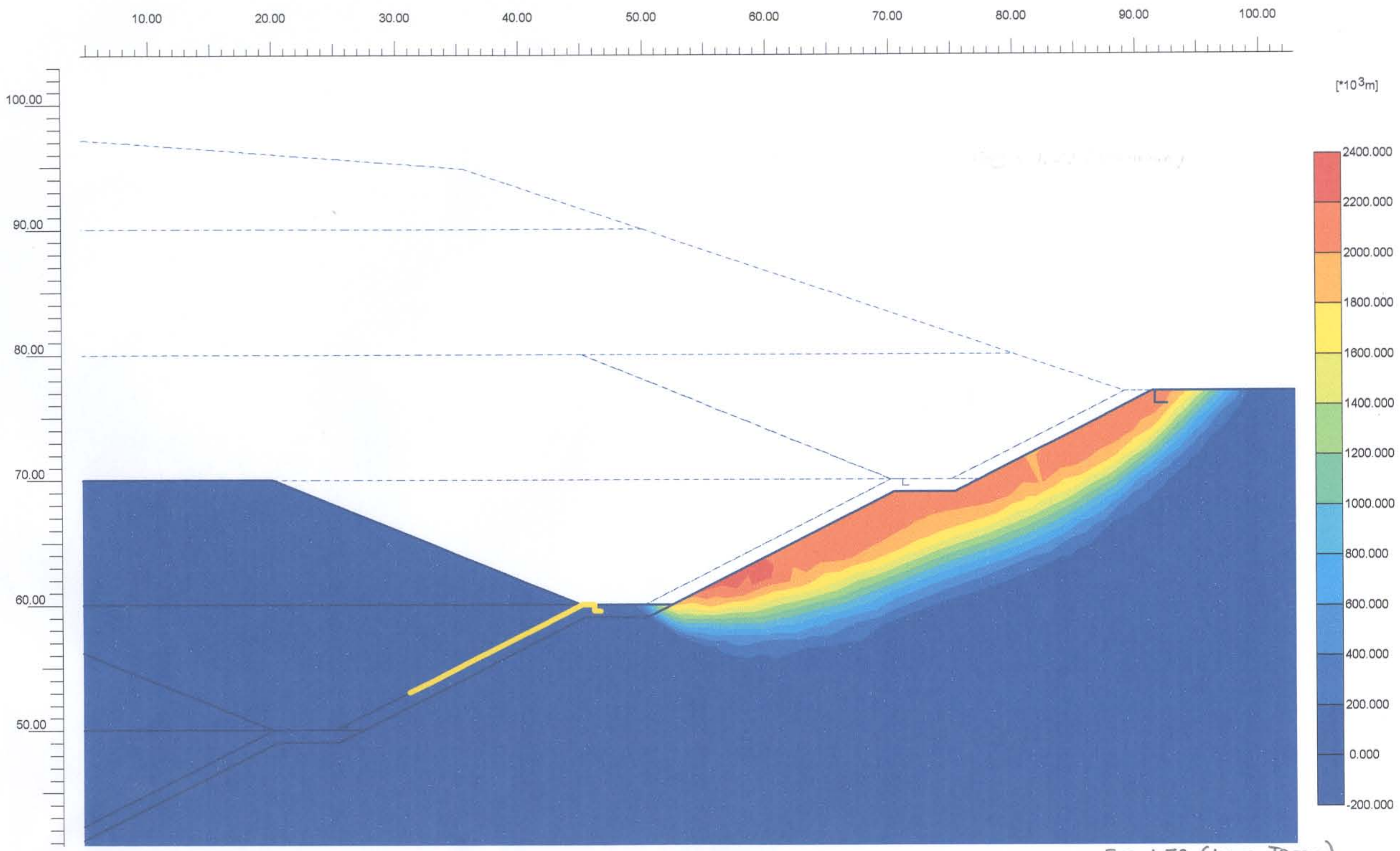
WR4446/SRA

Appendix SRA 2-9

PLAXIS Finite Element Code for Soil and Rock Analyses	Project description: South East Perimeter Outer Slope - Failure Mechanism (Waste 1 in 2)		
	Project name: Side Slope (Inc...	Step: 135	Date: 27/10/04	User name: Encia Consulting Limited

Version 8.2.4.133

PLAXIS V8



FOS=1.72 (Long-Term)

Total displacements (Utot)
Extreme Utot $2.35*10^6$ m

WR446/SRA

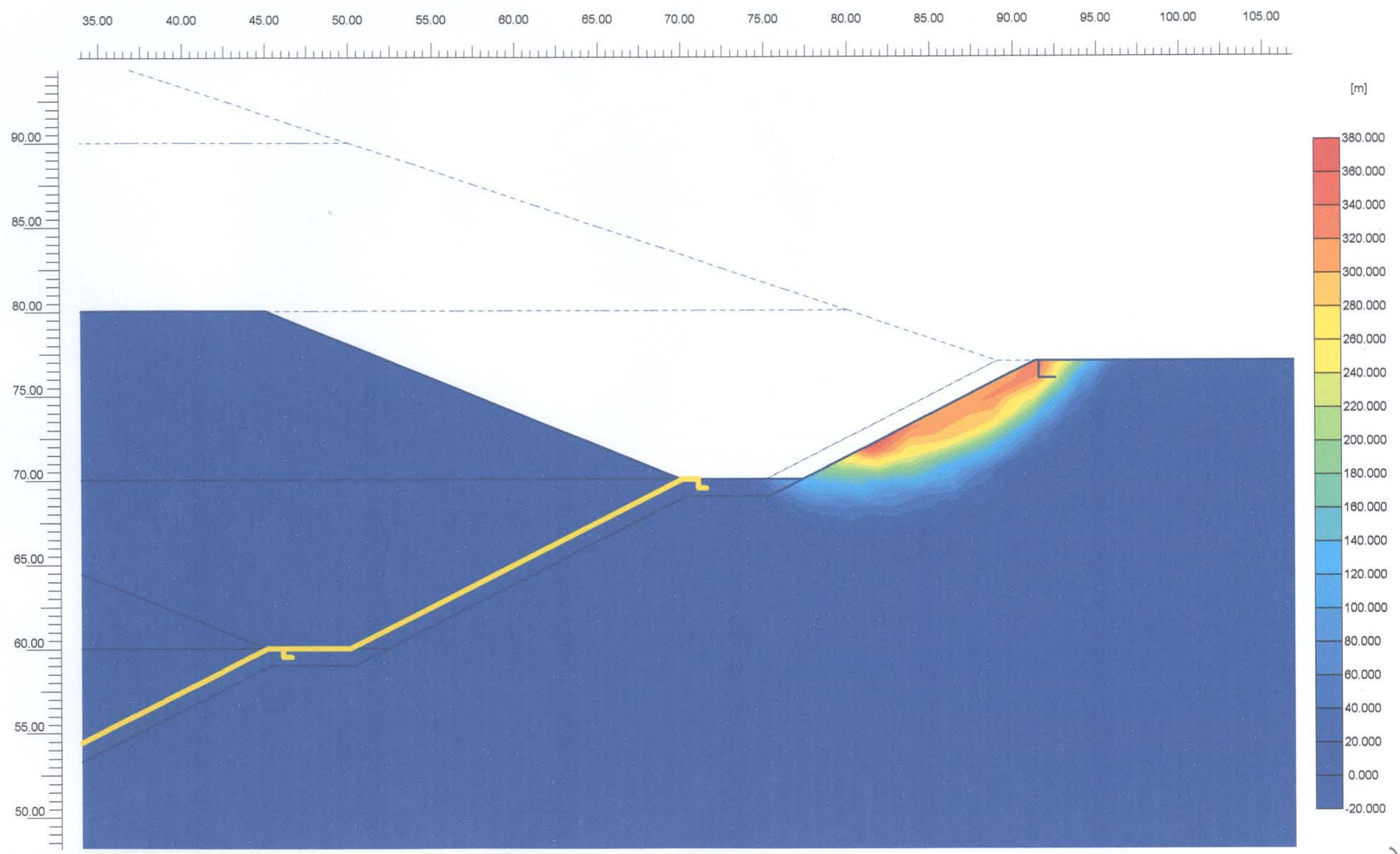
Appendix SRA 2-10

PLAXIS
Finite Element Code for Soil and Rock Analyses

Project description			
Side Slope Subgrade Failure Mechanism			
Project name: Pen-y-Bont	Step: 150	Date: 25/10/04	User name: Encia Consulting Limited

Version 8.2.4.133

PLAXIS V8



FoS = 2.14 (Long-Term)

Total displacements (Utot)
Extreme Utot 377.28 m

WR446 / SRA

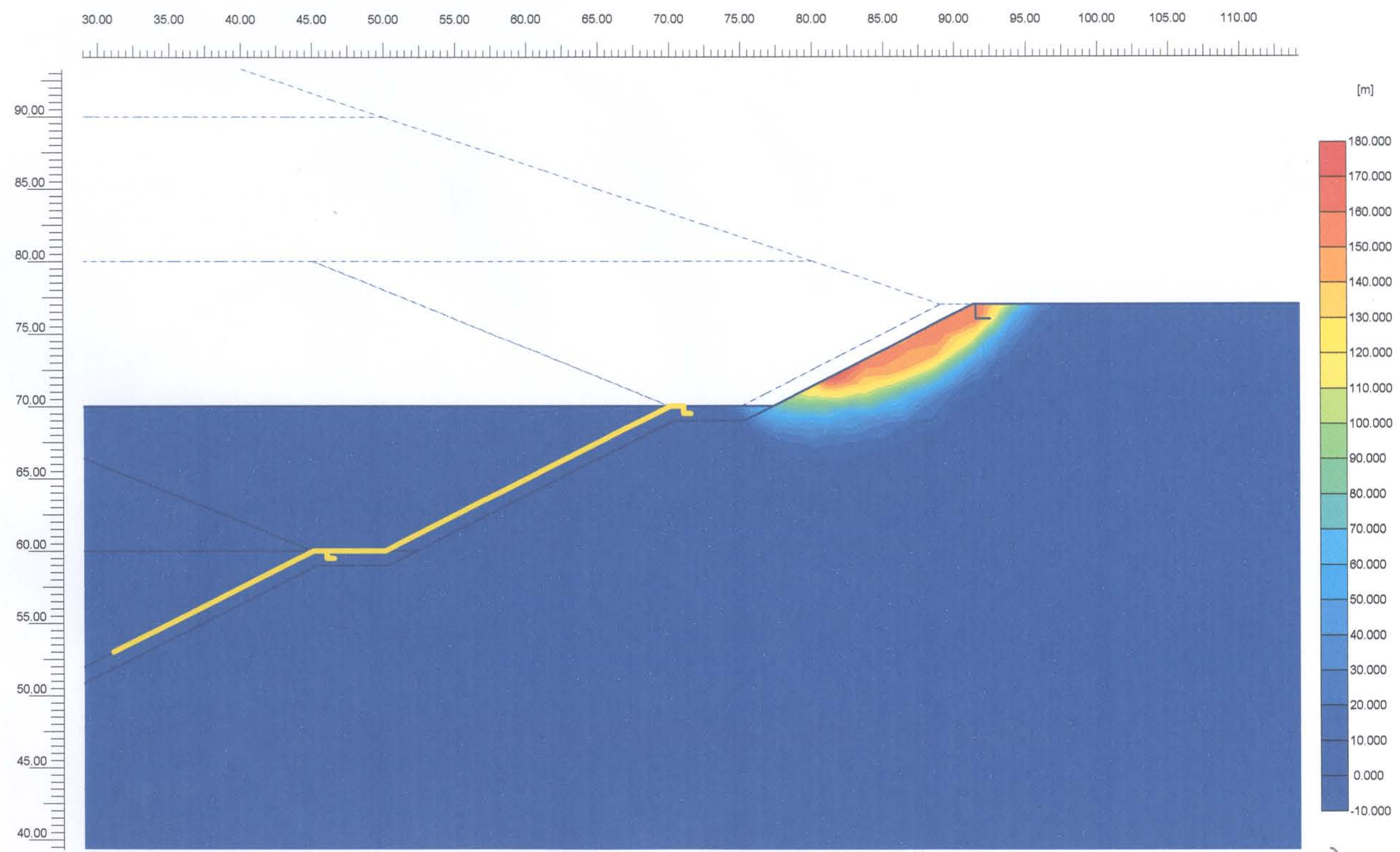
Appendix SRA 2 - 11

PLAXIS
Finite Element Code for Soil and Rock Analyses

Project description			
Upper Side Slope Failure Mechanism			
Project name: Pen-y-Bont	Step: 397	Date: 25/10/04	User name: Encia Consulting Limited

Version 8.2.4.133

PLAXIS V8



FOS = 2.18 (Long-Term)

Total displacements (Utot)
Extreme Utot 179.44 m

WR4446/ SRA

Appendix SRA 2-12

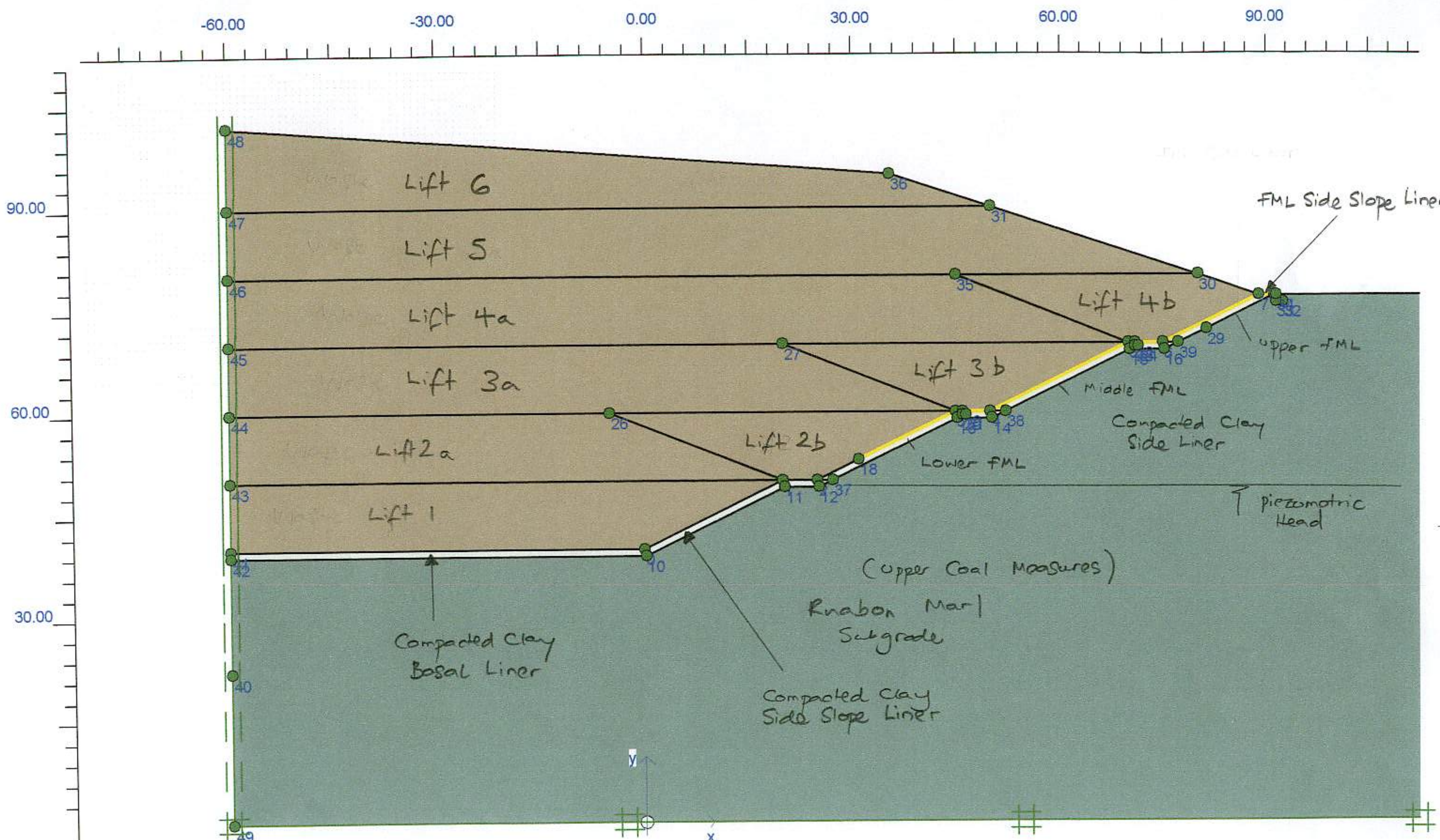
PLAXIS
Finite Element Code for Soil and Rock Analyses

Project description			
Upper Side Slope Subgrade Failure Mechanism			
Project name: Pen-y-Bont	Step: 284	Date: 25/10/04	User name: Encia Consulting Limited

Version 8.2.4.133

APPENDIX SRA3

BASAL LINER
AND SIDE-SLOPE LINER
INTEGRITY ANALYSES



WR4446/SRA

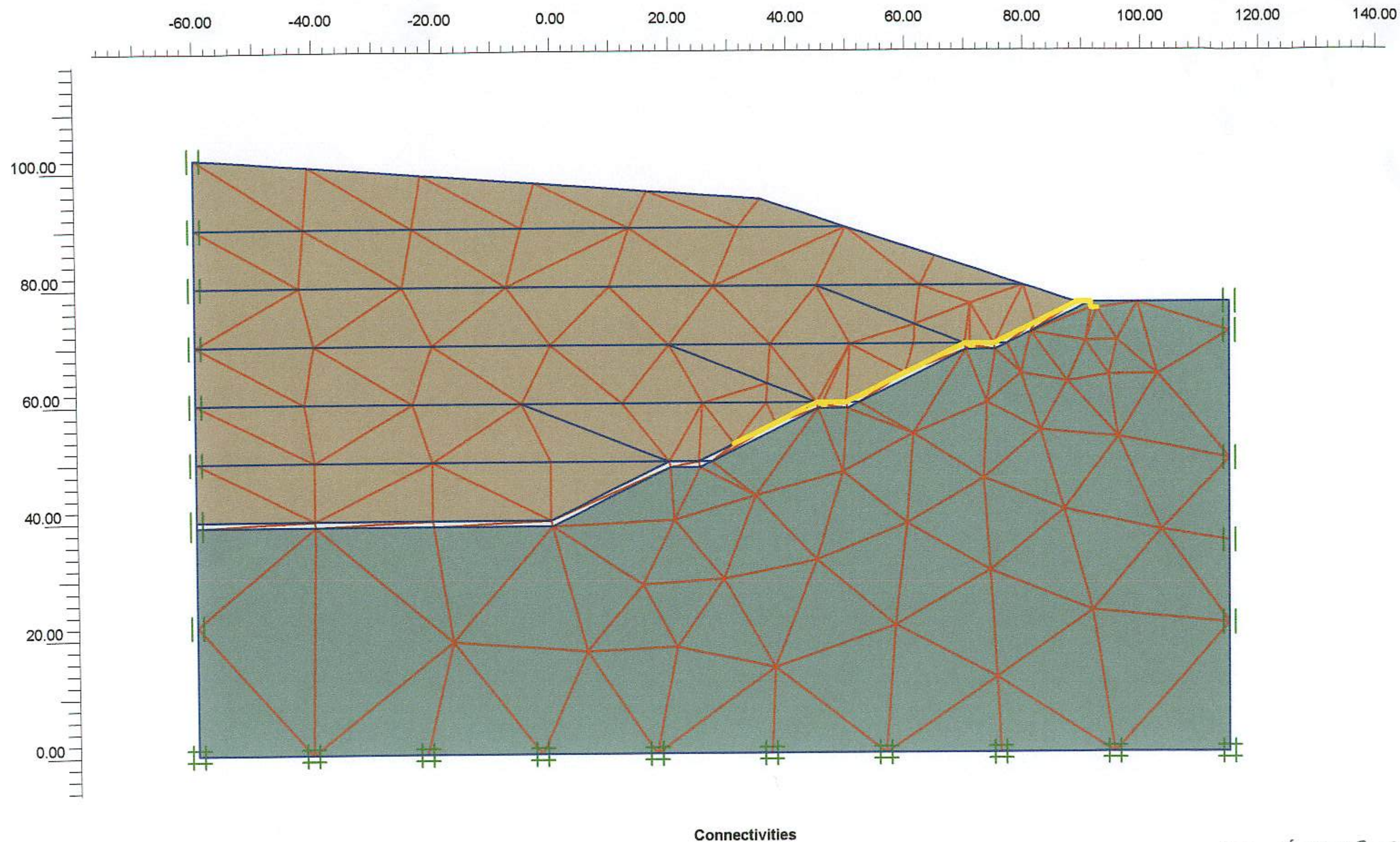
Appendix SRA3-1

PLAXIS
Finite Element Code for Soil and Rock Analyses

Version 8.2.4.811

Project description		
Side Slope Lining System - Worse Case - Geometry		
Project name: Pen-y-Bont.plx	Date: 27/10/2004	User name: Encia Consulting Limited

PLAXIS V8

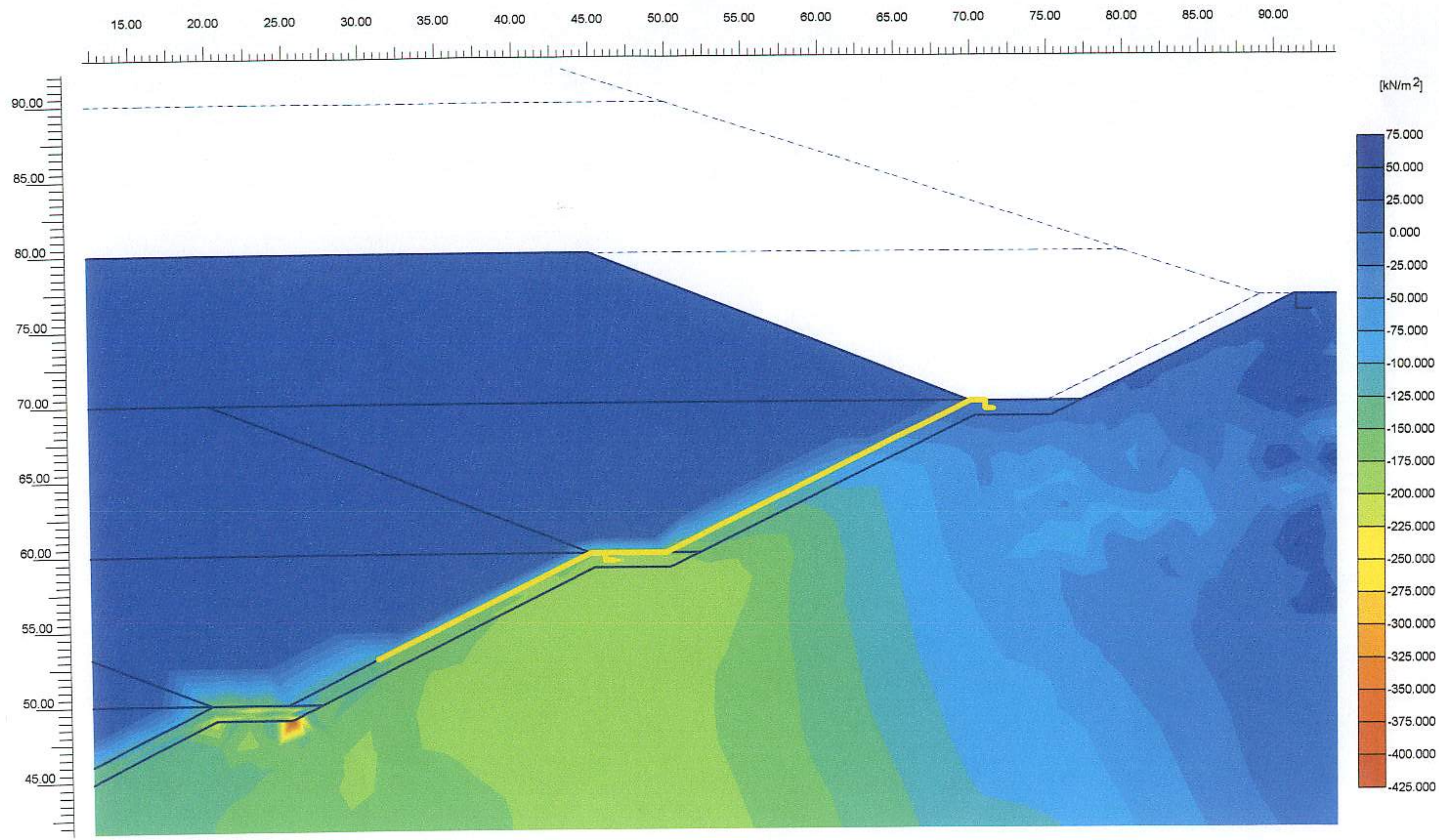


Appendix SRA 3-2

PLAXIS Finite Element Code for Soil and Rock Analyses	*Project description* Finite Element Mesh For Containment Lining System		
	Project name	*Date* 27/10/04	*User name* Encia Consulting Limited

Version 8.2.4.133

PLAXIS V8



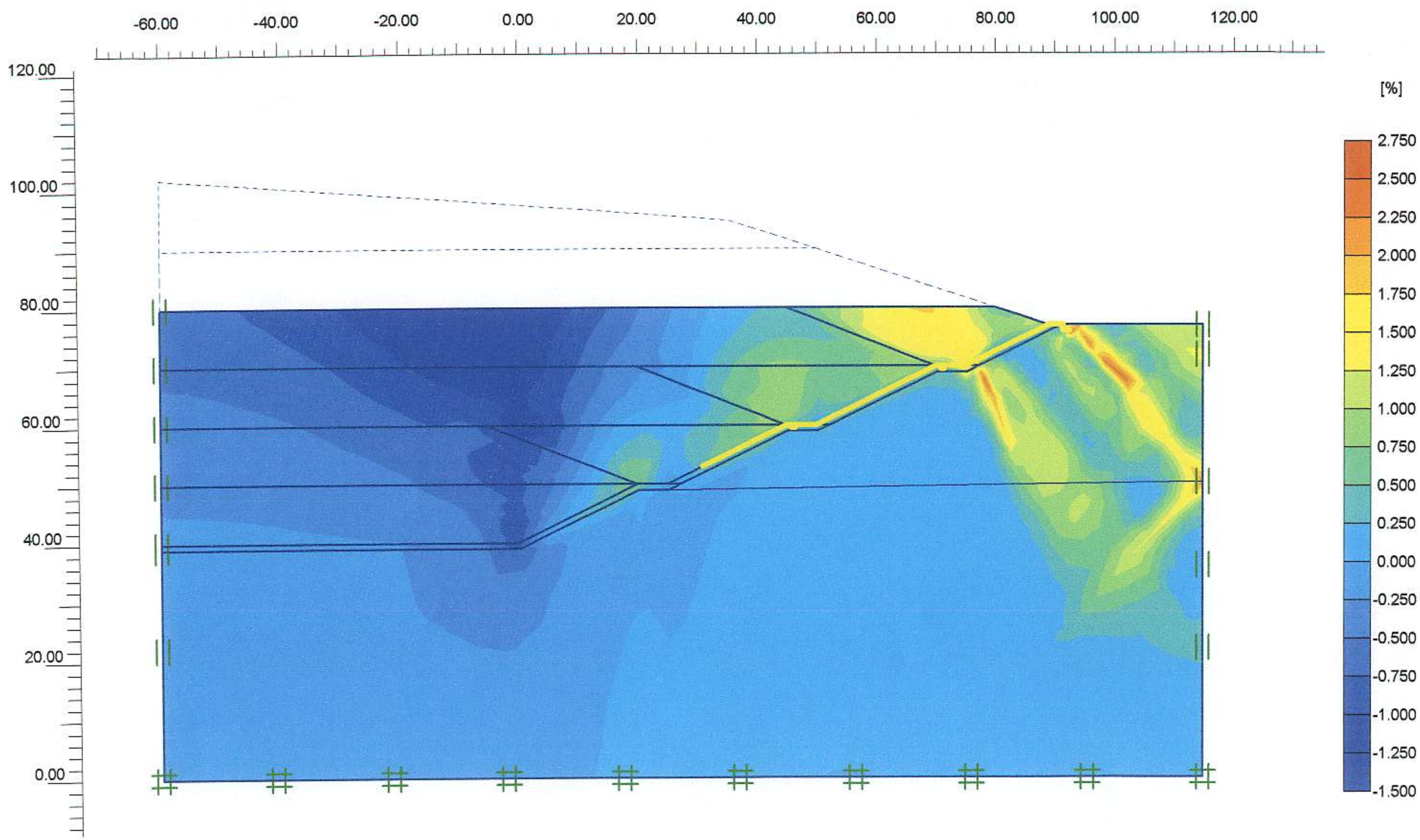
Excess pore pressures
Extreme excess pore pressure -402.89 kN/m^2
(pressure = negative)

Appendix SRA3 – 3

PLAXIS Finite Element Code for Soil and Rock Analyses	*Project description* Excess Pore Water Pressures Following Waste Lift 6			
	Project name Pen-y-Bont	*Step* 397	*Date* 25/10/04	*User name* Encia Consulting Limited

Version 8.2.4.133

PLAXIS V8



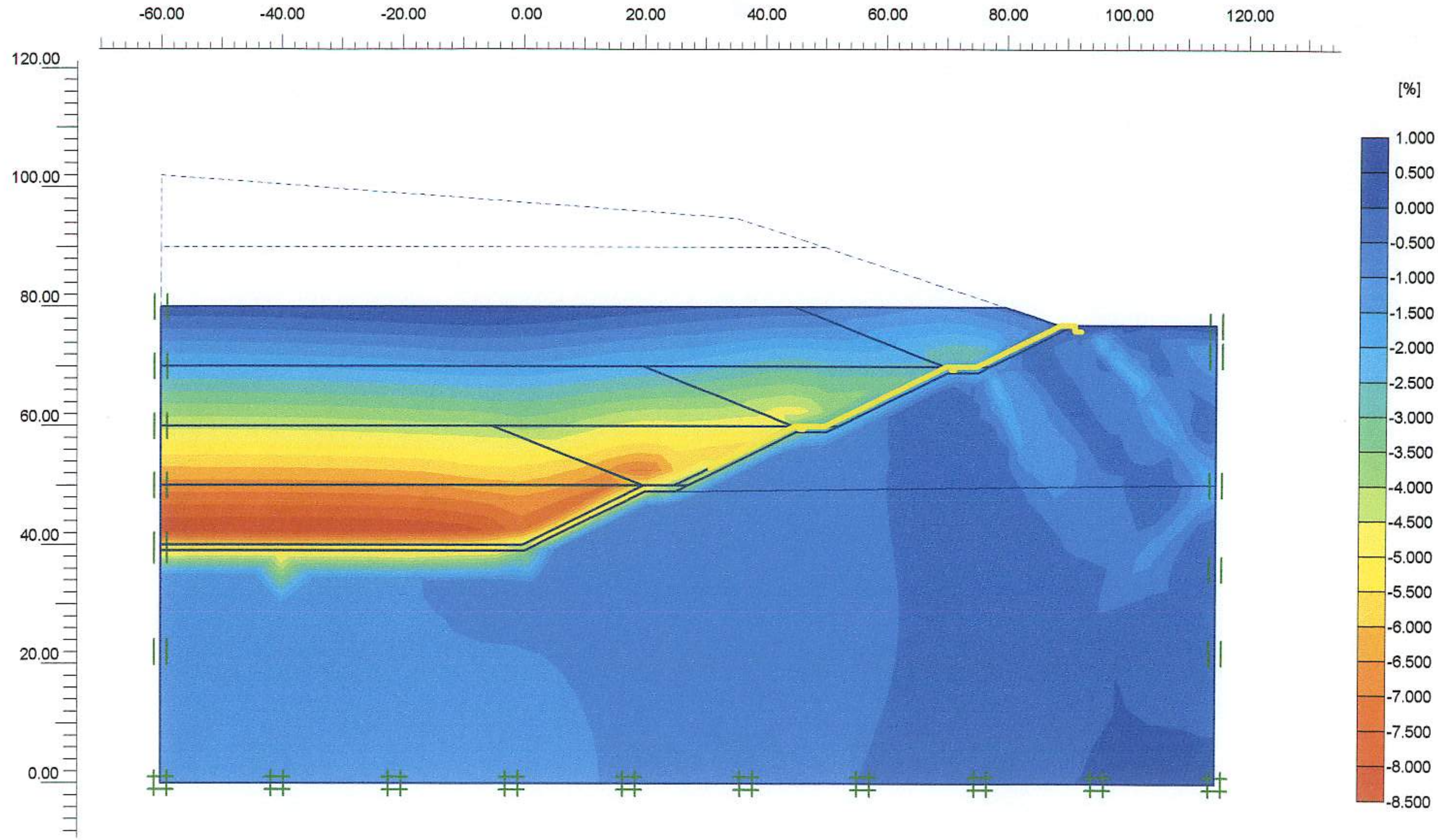
Horizontal strains (Eps-xx)
Extreme Eps-xx 2.61%

Appendix SRA 3 - 4

PLAXIS Finite Element Code for Soil and Rock Analyses	Project description			
	Horizontal Strains After Partial Waste Infill			
	Project name: Pen-y-Bont	Step: 86	Date: 27/10/04	User name: Encia Consulting Limited

Version 8.2.4.133

PLAXIS V8



Vertical strains (Eps-yy)
Extreme Eps-yy -8.49%

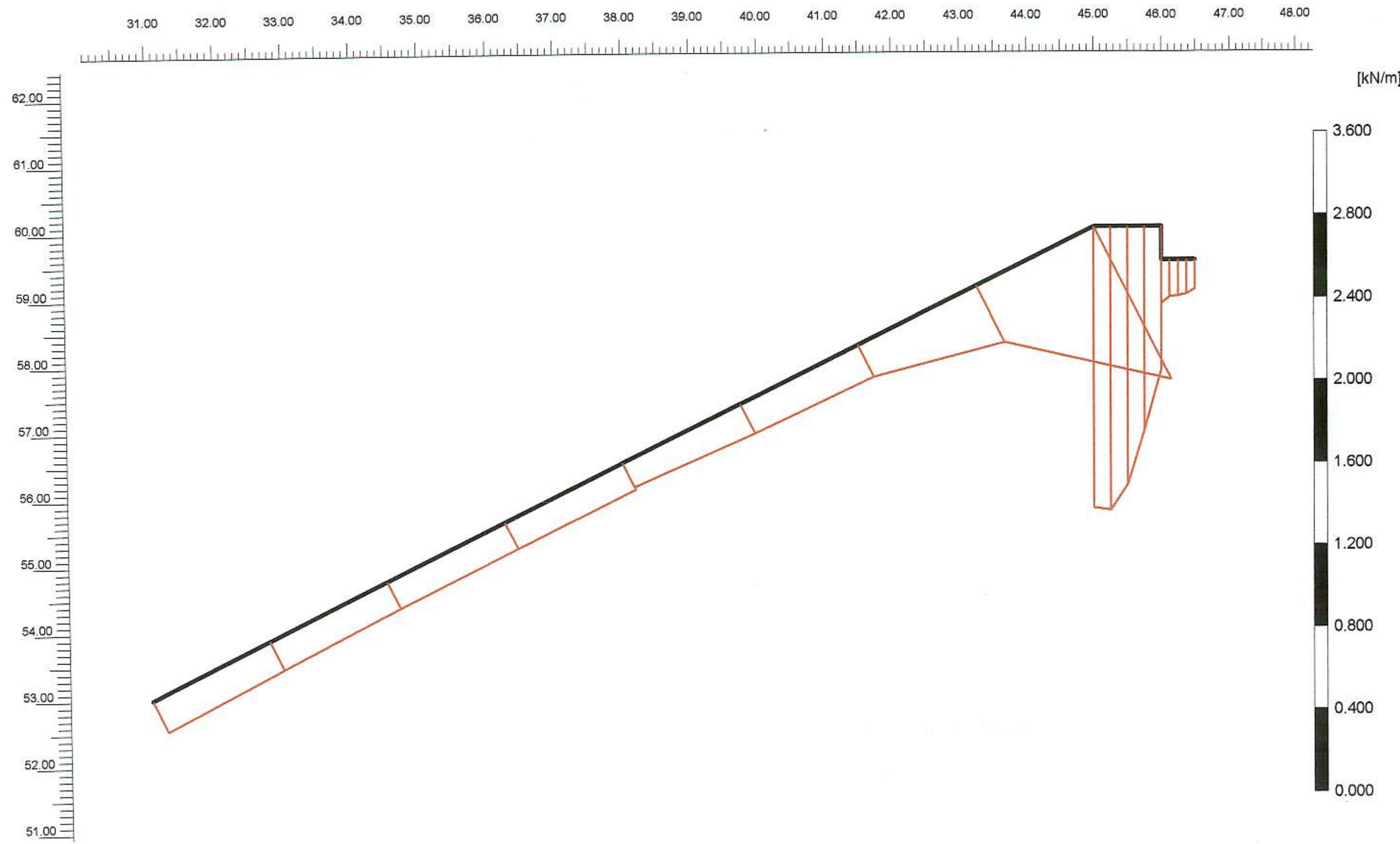
WR4446/SRA

Appendix SRA3-5

PLAXIS
Finite Element Code for Soil and Rock Analyses
Version 8.2.4.133

Project description			
Vertical Strains Following Partial Waste Infill			
Project name	Step	Date	User name
Pen-y-Bont	86	27/10/04	Encia Consulting Limited

PLAXIS 8.0



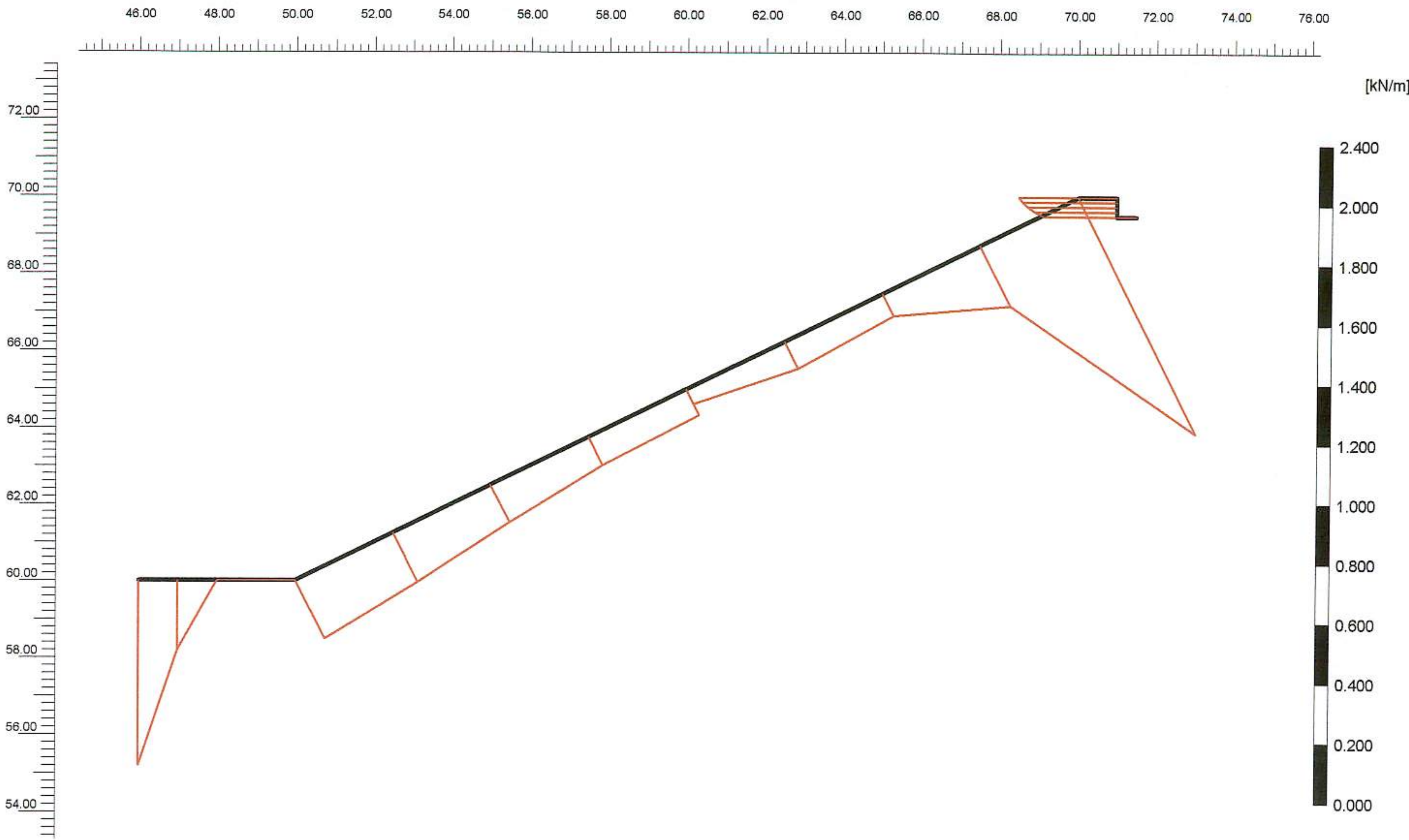
Axial forces
Extreme axial force 1.37 kN/m

Appendix SRA 3-6

PLAXIS Finite Element Code for Soil and Rock Analyses	*Project description* Axial Forces In Lower FML After Partial Waste Lift		
	Project name Pen-y-Bont	*Step* 90	*Date* 27/10/04	*User name* Encia Consulting Limited

Version 8.2.4.133

PLAXIS 8.0



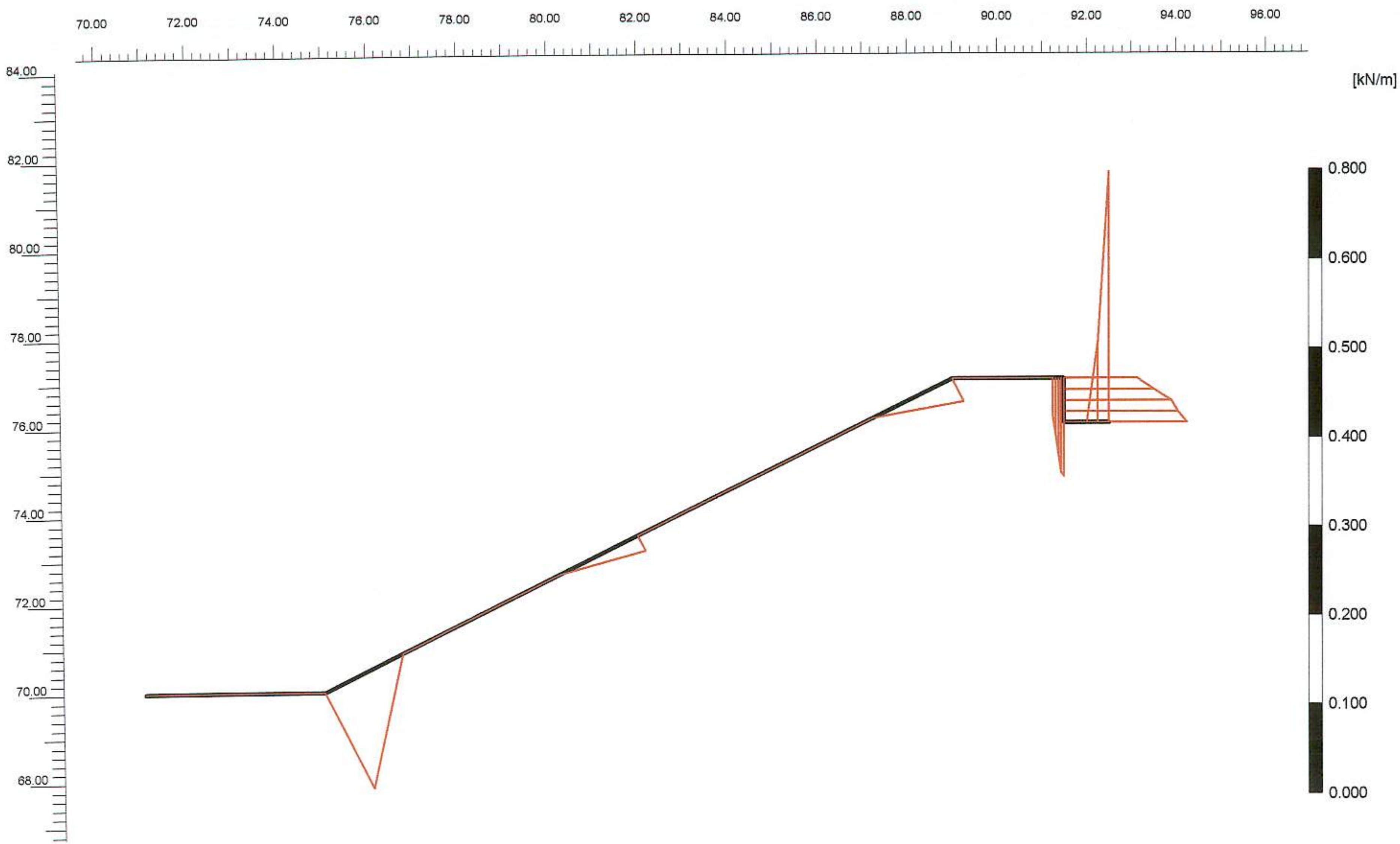
Axial forces
Extreme axial force 883.57*10^{-3} kN/m

Appendix SRA 3 - 7

PLAXIS Finite Element Code for Soil and Rock Analyses	*Project description* Axial Forces In Middle FML After Partial Waste Lift			
	Project name Pen-y-Bont	*Step* 90	*Date* 27/10/04	*User name* Encia Consulting Limited

Version 8.2.4.133

PLAXIS 8.0



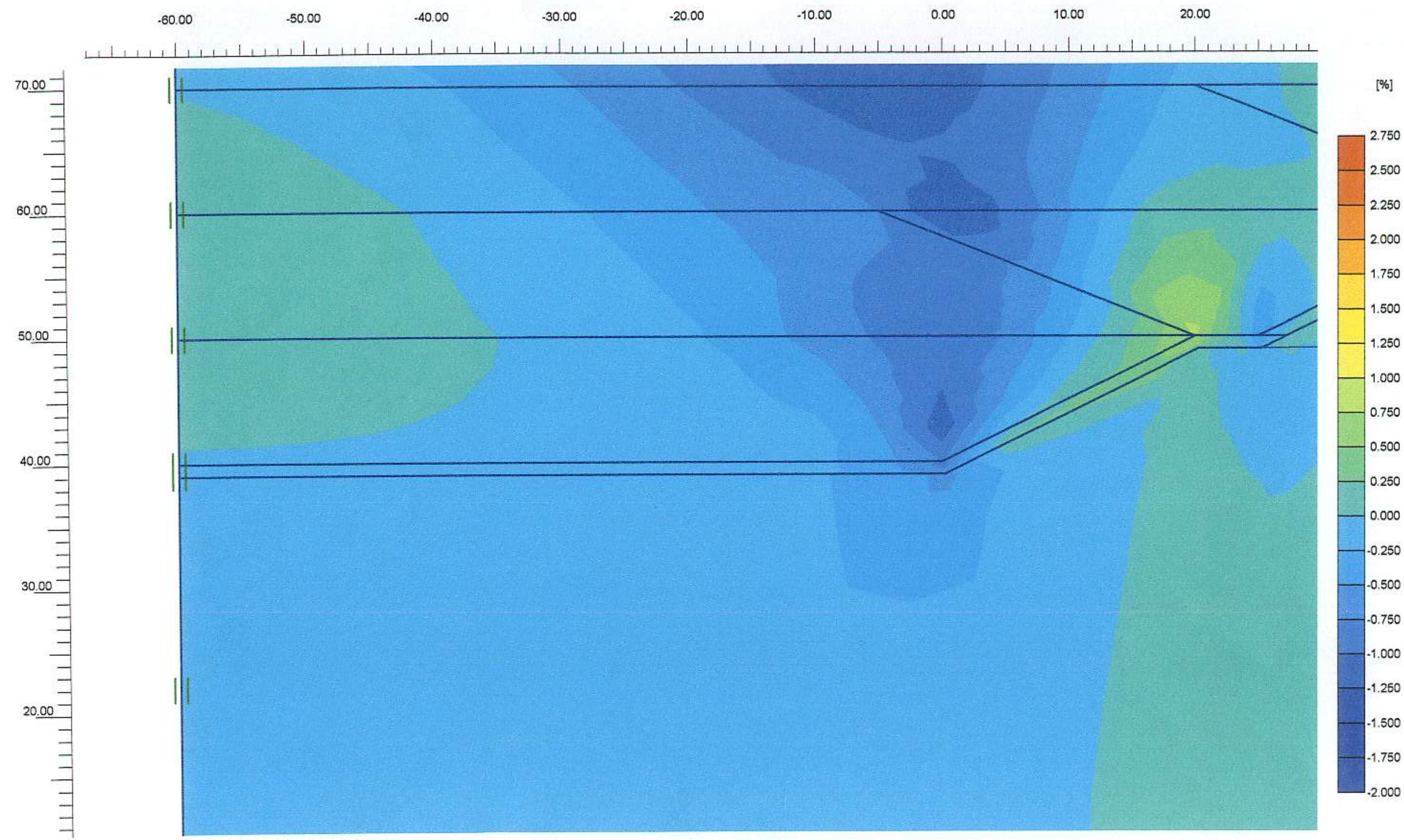
Axial forces
Extreme axial force 280.41*10^-3 kN/m

Appendix SRA 3-8

PLAXIS Finite Element Code for Soil and Rock Analyses	Project description Axial Forces In Upper FML After Partial Waste Lift		
	Project name Pen-y-Bont	Step 90	Date 27/10/04	User name Encia Consulting Limited

Version 8.2.4.133

PLAXIS V8



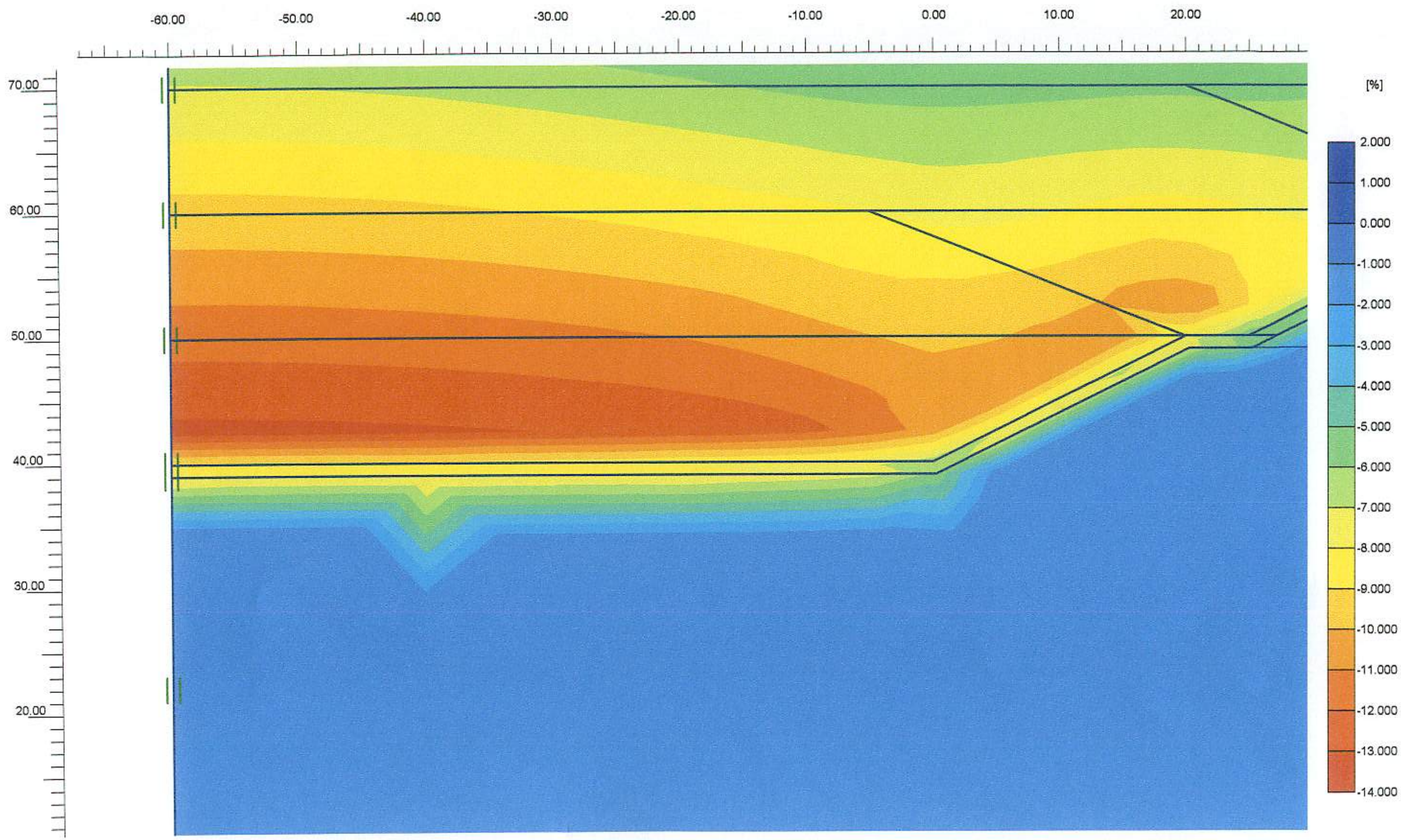
Horizontal strains (Eps-xx)
Extreme Eps-xx 2.52%

Appendix SRA 3 – 9

PLAXIS Finite Element Code for Soil and Rock Analyses	*Project description* Horizontal Strains In Basal Liner After Final Waste Lift			
	Project name Pen-y-Bont	*Step* 90	*Date* 27/10/04	*User name* Encia Consulting Limited

Version 8.2.4.133

PLAXIS V8



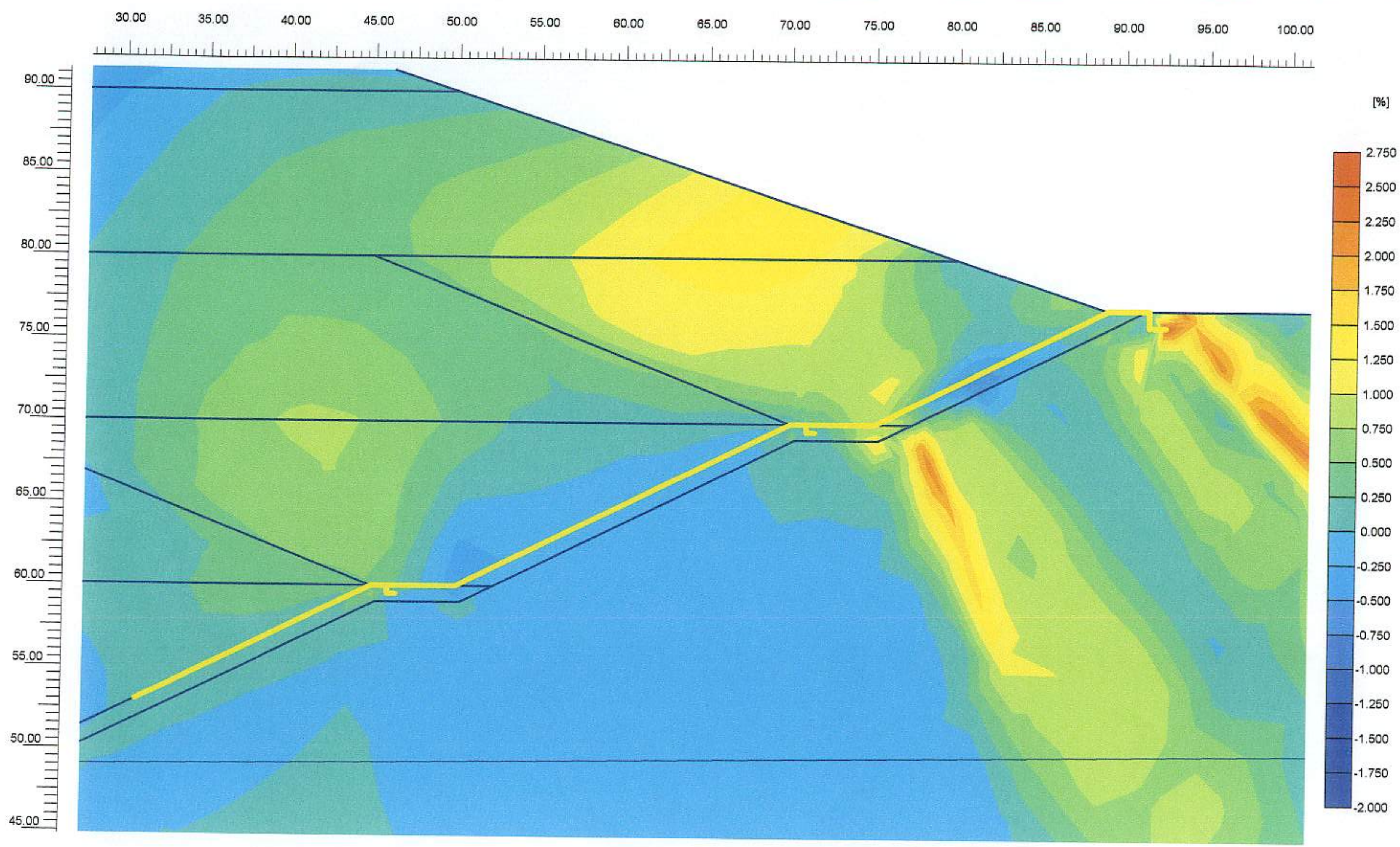
Vertical strains (Eps-yy)
Extreme Eps-yy -13.26%

Appendix SRA 3 - 10

PLAXIS Finite Element Code for Soil and Rock Analyses	Project description Vertical Strains In Basal Liner After Final Waste Lift			
	Project name Pen-y-Bont	Step 90	Date 27/10/04	User name Encia Consulting Limited

Version 8.2.4.133

PLAXIS V8



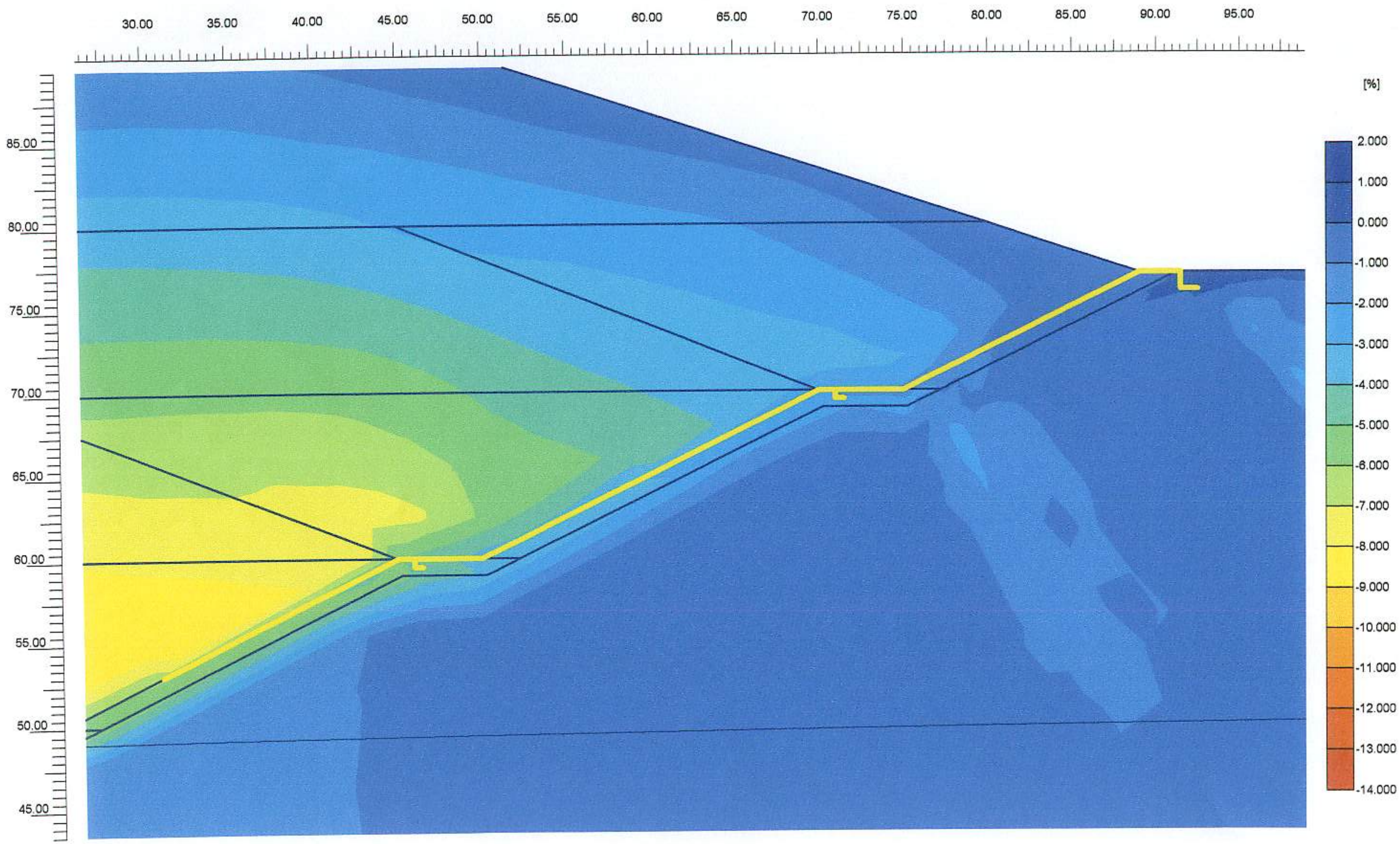
Horizontal strains (Eps-xx)
Extreme Eps-xx 2.52%

Appendix SRA 3-11

PLAXIS Finite Element Code for Soil and Rock Analyses	Project description: Horizontal Strains In Side Slope Lining System			
	Project name: Pen-y-Bont	Step: 90	Date: 27/10/04	User name: Encia Consulting Limited

Version 8.2.4.133

PLAXIS V8



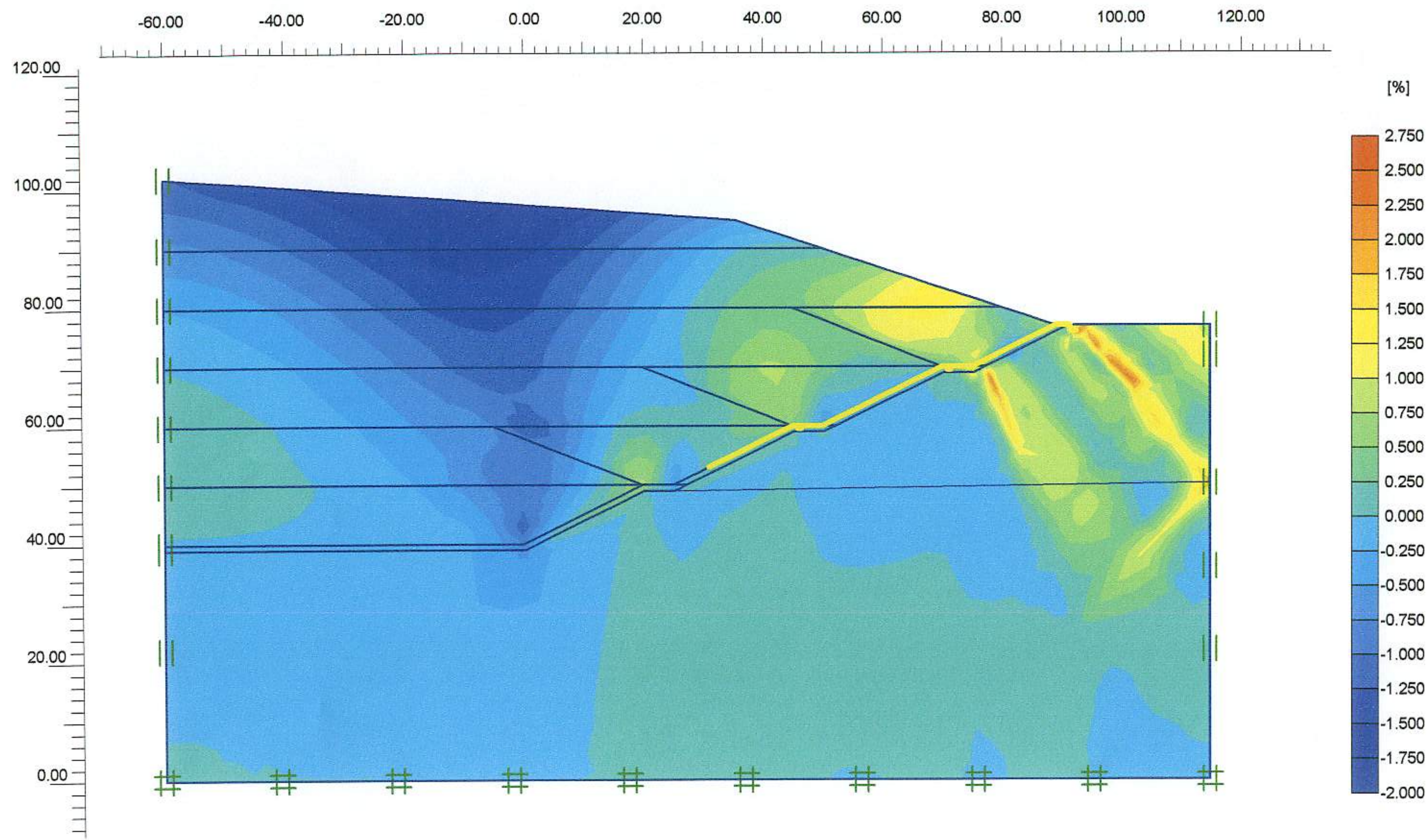
Vertical strains (Eps-yy)
Extreme Eps-yy -13.26%

Appendix SRA 3-12

PLAXIS Finite Element Code for Soil and Rock Analyses	Project description Vertical Strains In Side Slope Lining System			
	Project name Pen-y-Bont	Step 90	Date 27/10/04	User name Encia Consulting Limited

Version 8.2.4.133

PLAXIS V8



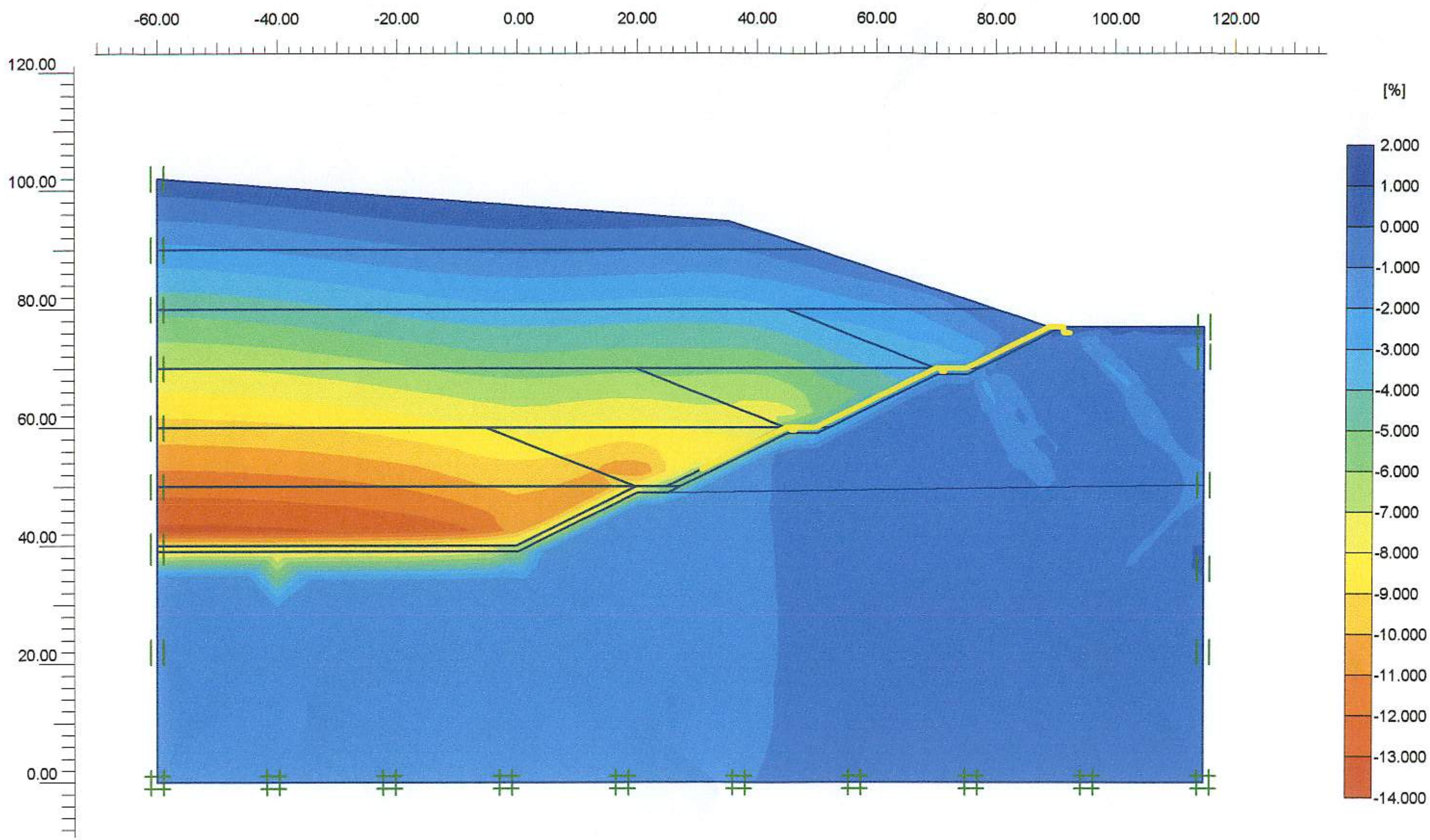
Horizontal strains (Eps-xx)
Extreme Eps-xx 2.52%

Appendix SRA 3-13

PLAXIS Finite Element Code for Soil and Rock Analyses	Project description Horizontal Strains After Final Waste Lift			
	Project name Pen-y-Bont	Step 90	Date 27/10/04	User name Encia Consulting Limited

Version 8.2.4.133

PLAXIS V8



Vertical strains (Eps-yy)
Extreme Eps-yy -13.26%

Appendix SRA 3 — 14

PLAXIS Finite Element Code for Soil and Rock Analyses	*Project description* Vertical Strains After Final Waste Lift			
	Project name Pen-y-Bont	*Step* 90	*Date* 27/10/04	*User name* Encia Consulting Limited

Version 8.2.4.133

PLAXIS 8.0

From Manufacturer's Stress Strain Relationship

29 kN/m ≏ 12% Strain at Yield (Uni-axial) for 2mm HDPE

0.849 kN/m ≏ 0.351 % < 3% Acceptable Limit for HDPE ✓✓ o.k.

Axial forces

Extreme axial force 849.77*10^{-3} kN/m

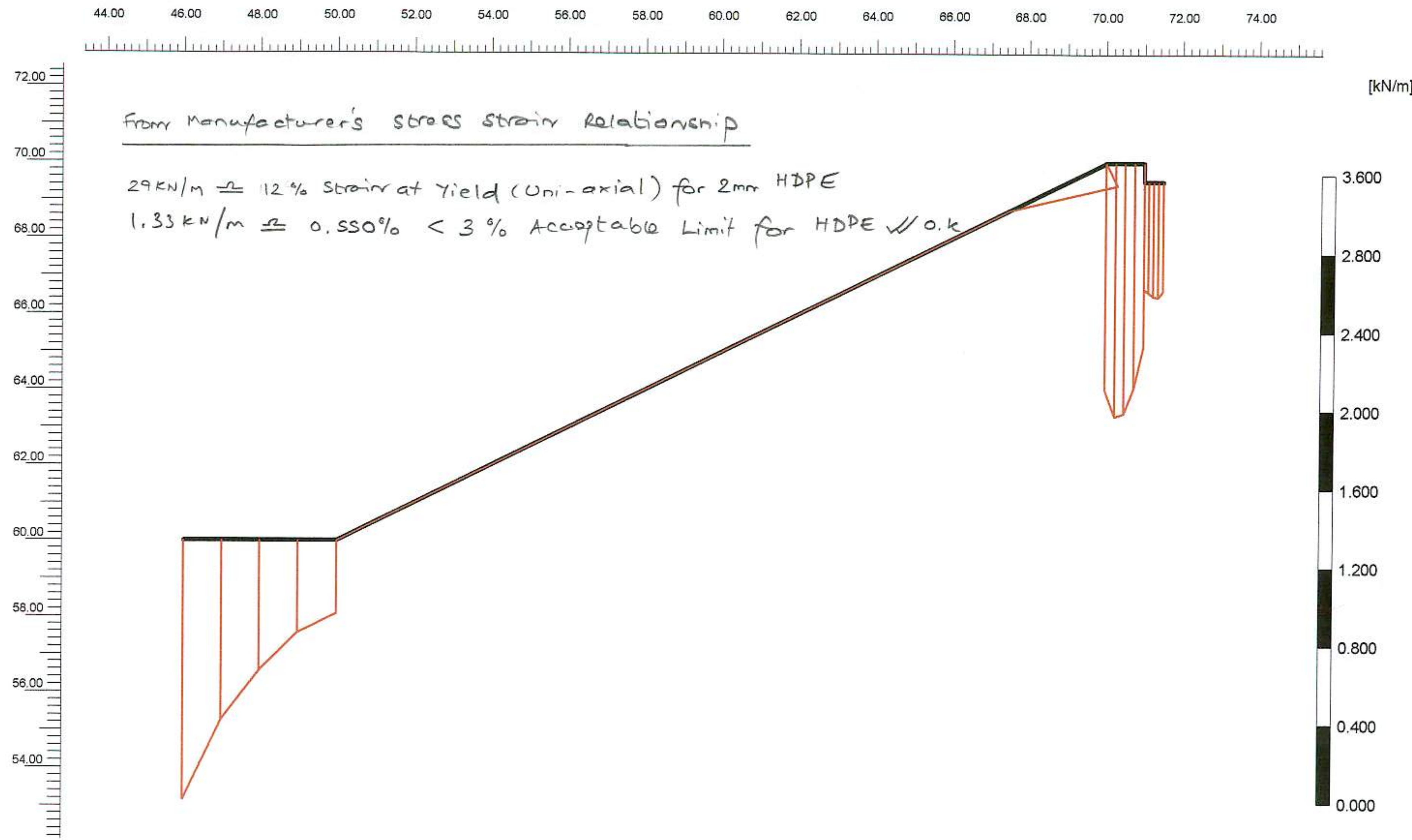
Interface friction Angle = 17.5° to 25°

Appendix SRA 3-15

PLAXIS Finite Element Code for Soil and Rock Analyses	Project description			
	Axial Forces In Upper FML After Final Waste Infill			
	Project name: Pen-y-Bont	Step: 86	Date: 27/10/04	User name: Encia Consulting Limited

Version 8.2.4.133

PLAXIS 8.0



Axial forces
Extreme axial force 1.33 kN/m

Appendix SRA3-16

PLAXIS Finite Element Code for Soil and Rock Analyses	Project description Axial Forces In Middle FML After Final Waste Infill		
	Project name Pen-y-Bont	Step 86	Date 27/10/04	User name Encia Consulting Limited

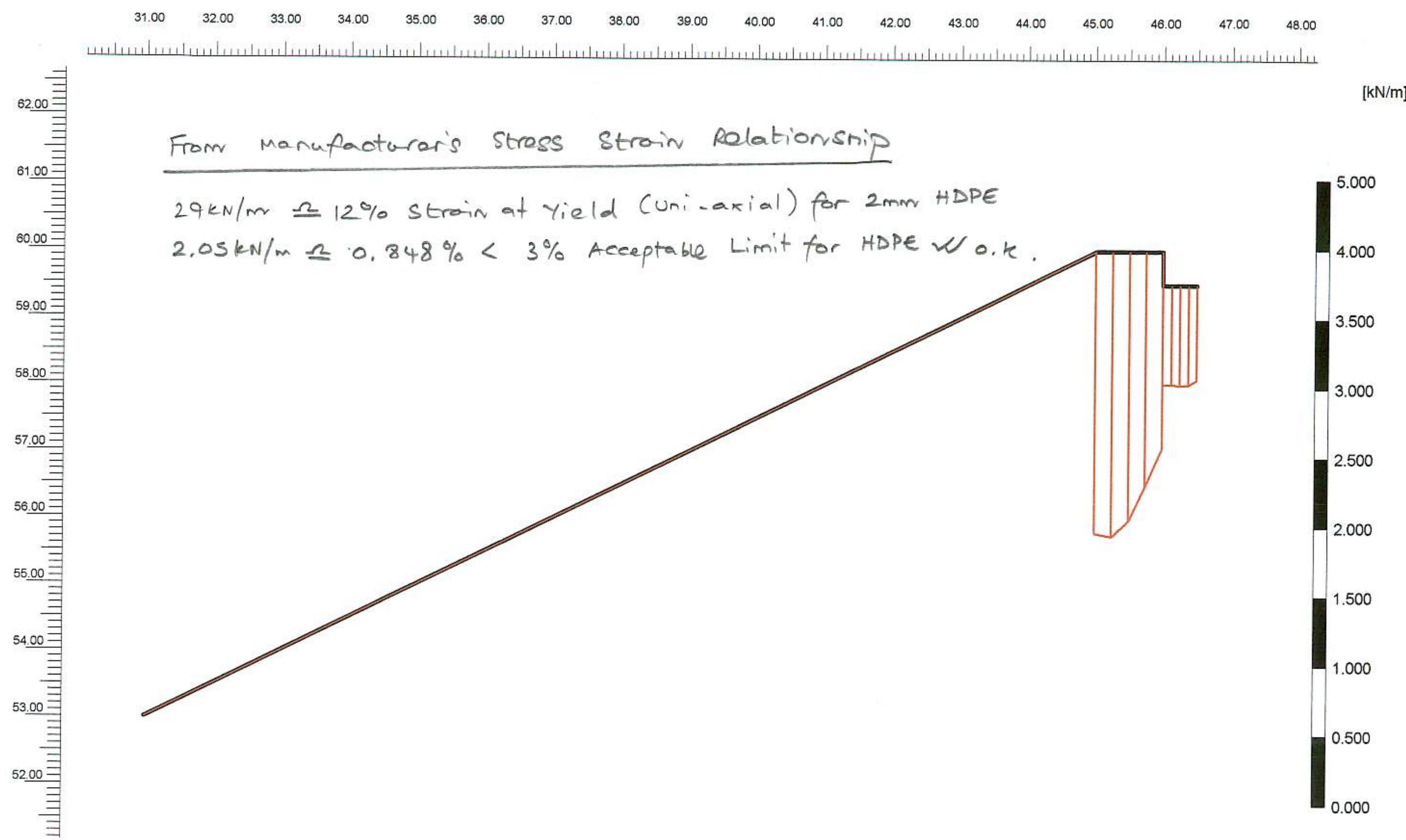
Version 8.2.4.133

PLAXIS 8.0

From manufacturer's Stress Strain Relationship

29 kN/m ≈ 12% strain at Yield (Uni-axial) for 2mm HDPE

2.05 kN/m ≈ 0.848% < 3% Acceptable Limit for HDPE ✓ o.k.



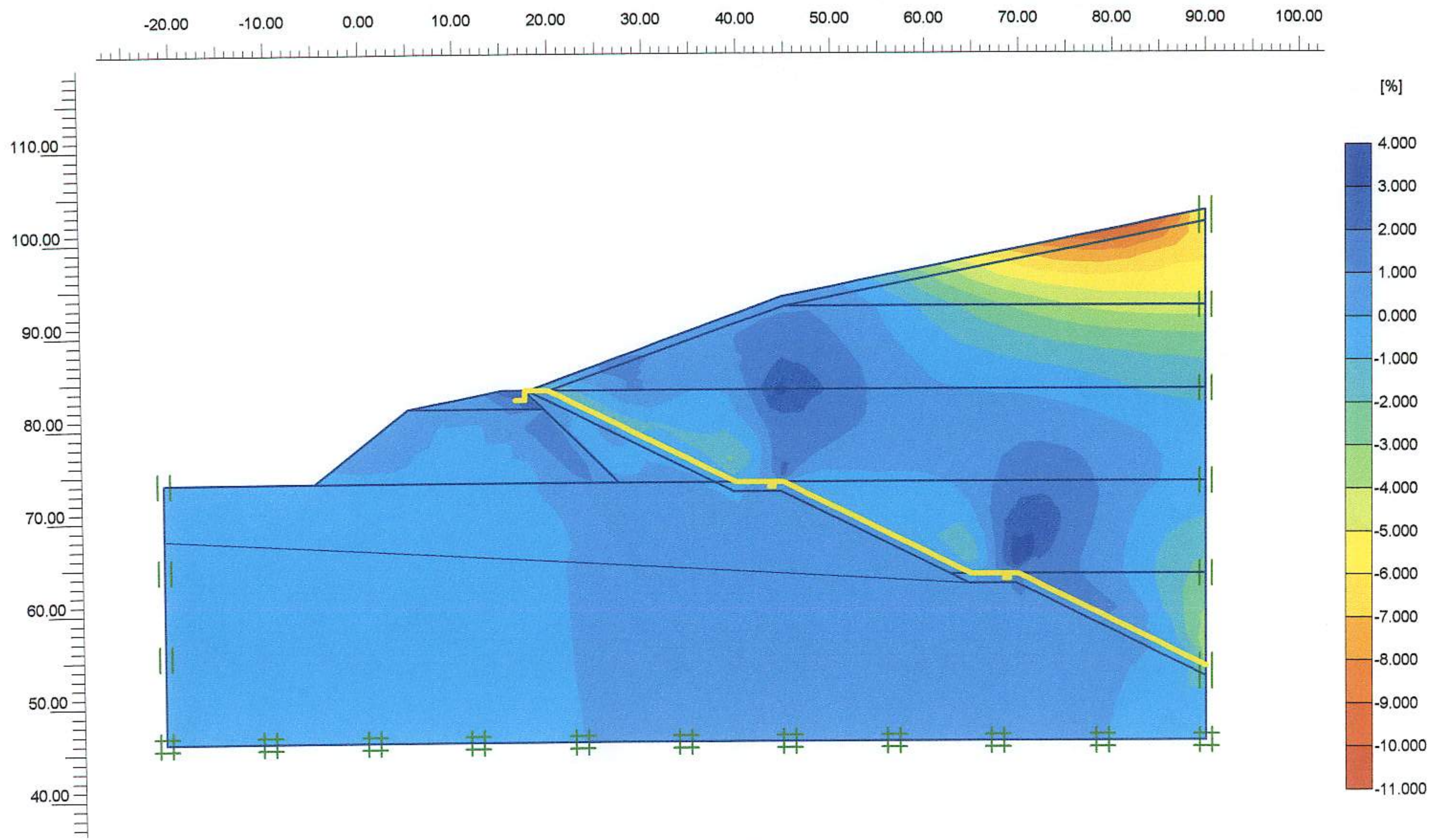
Axial forces
Extreme axial force 2.05 kN/m

Appendix SRA3-17

Project description			
Axial Forces In Lower FML After Final Waste Infill			
Project name	Step	Date	User name
Pen-y-Bont	86	27/10/04	Encia Consulting Limited

PLAXIS
Finite Element Code for Soil and Rock Analyses
Version 8.2.4.133

PLAXIS V8



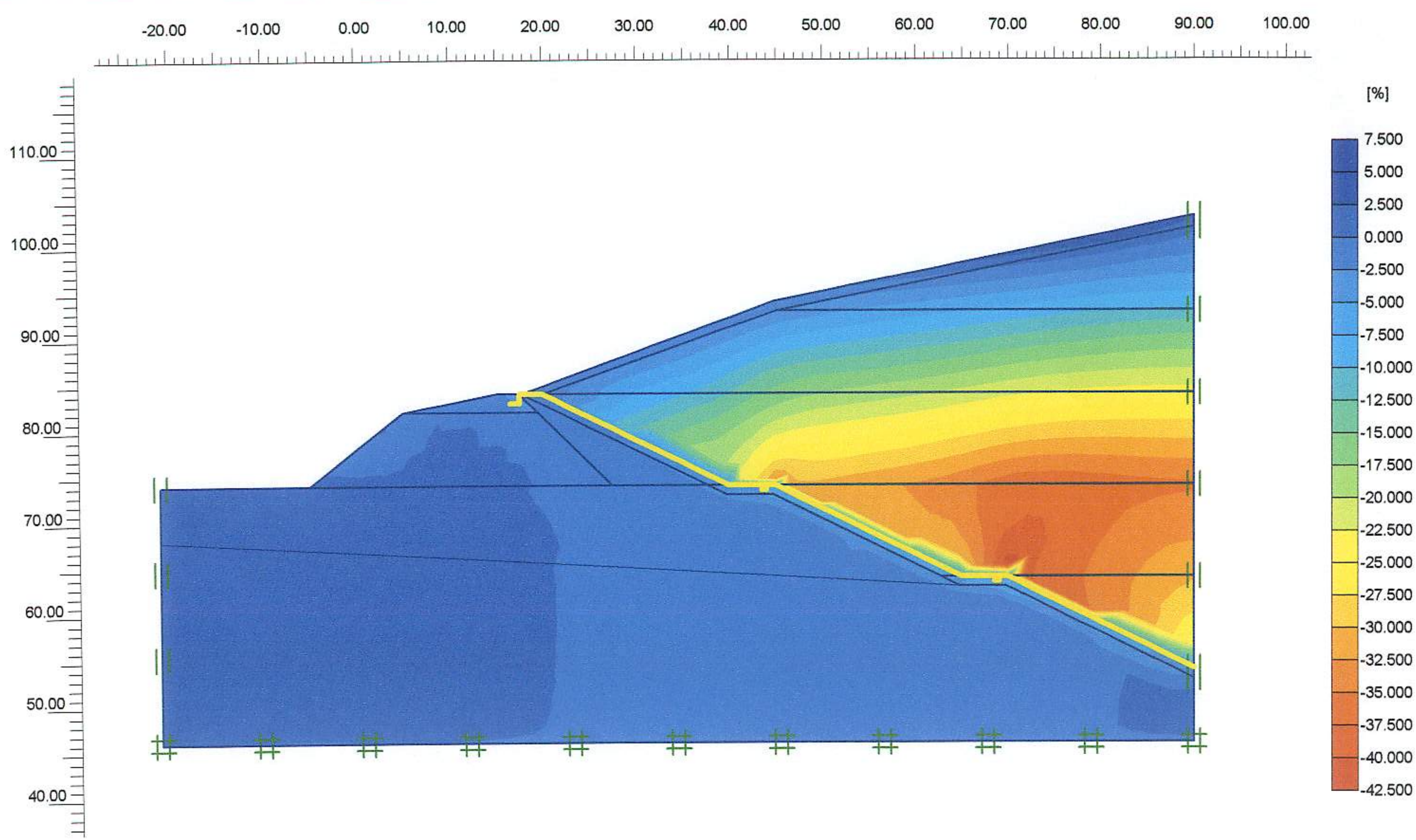
Horizontal strains (Eps-xx)
Extreme Eps-xx -10.39%

Appendix SRA 3 – 18

PLAXIS Finite Element Code for Soil and Rock Analyses	Project description: Horizontal Strains in South Eastern Side Lining System			
	Project name: Side Slope	Step: 35	Date: 27/10/04	User name: Encia Consulting Limited

Version 8.2.4.133

PLAXIS V8



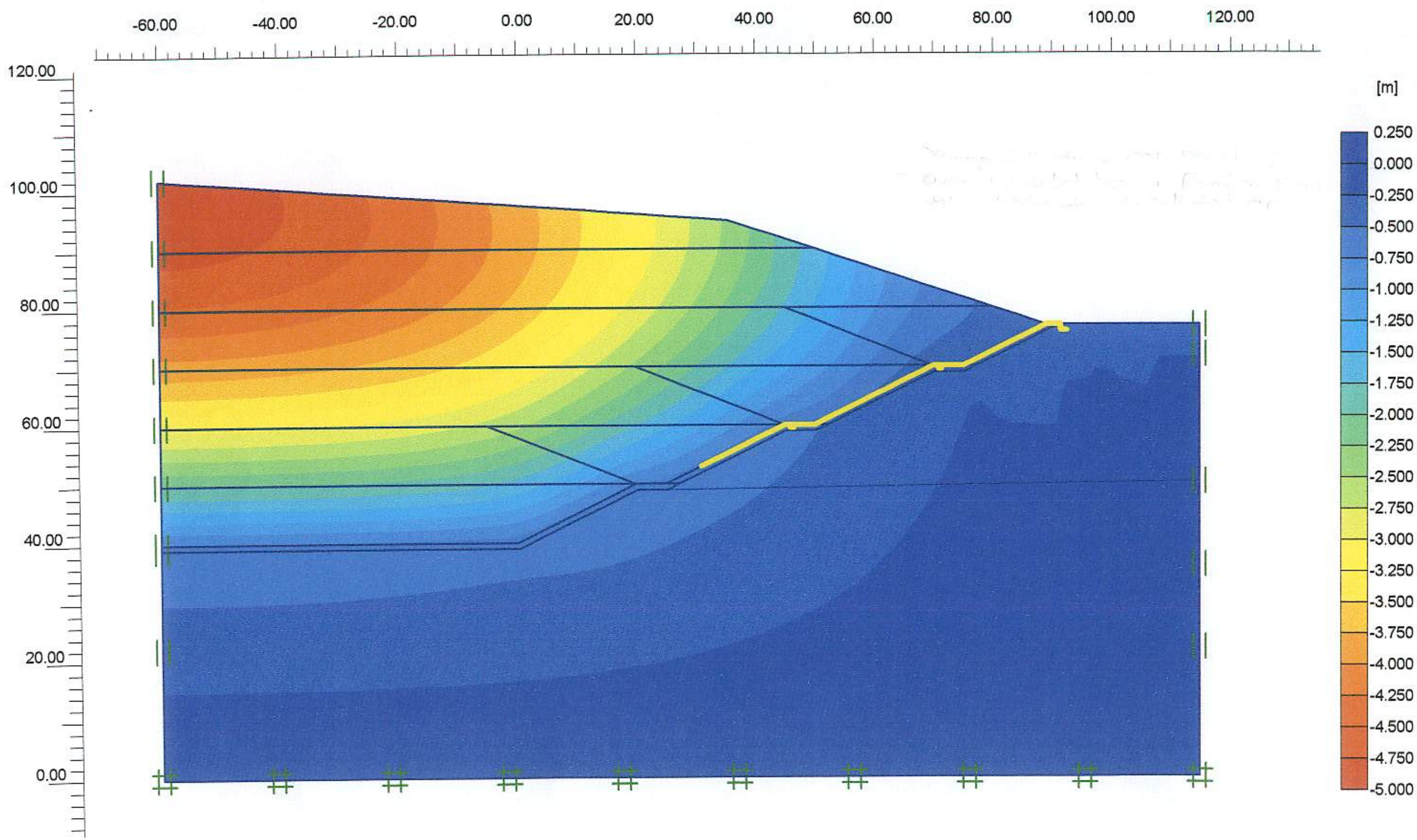
Vertical strains (Eps-yy)
Extreme Eps-yy -41.41%

Appendix SRA 3 – 19

PLAXIS Finite Element Code for Soil and Rock Analyses	*Project description* Vertical Shear Strains in South Eastern Side Lining System			
	Project name Side Slope	*Step* 35	*Date* 27/10/04	*User name* Encia Consulting Limited

Version 8.2.4.133

PLAXIS V8



Vertical displacements (Uy)
Extreme Uy -4.93 m

WR4446 / SRA

Appendix SRA3-20

PLAXIS Finite Element Code for Soil and Rock Analyses	*Project description* Vertical Displacements After Final Waste Lift			
	Project name Pen-y-Bont	*Step* 90	*Date* 27/10/04	*User name* Encia Consulting Limited

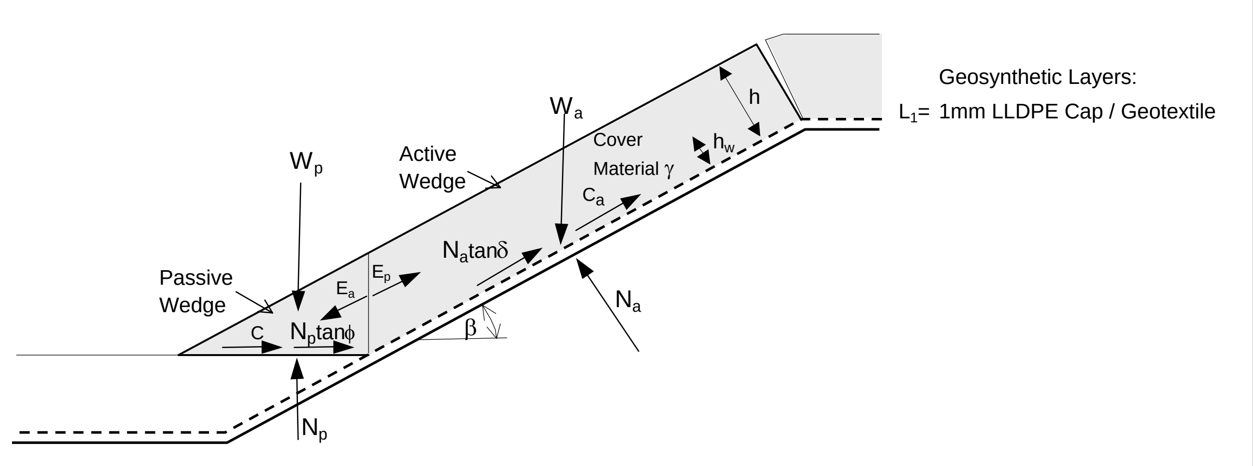
Version 8.2.4.133

APPENDIX SRA4

CAPPING SYSTEM STABILITY ANALYSES

SLOPE STABILITY CALCULATION

UNIFORM COVER THICKNESS, FINITE SLOPE METHOD Jones & Dixon



INPUT DATA TO SHADED BOXES

β	Slope angle(°)	19.65	1 in 2.8
h	Thickness of the cover material (m)	1.00	
γ	Unit weight of cover material (kN/m³)	18	
h_w	Thickness of saturated cover material (m)	0	
γ_s	Saturated unit weight of the cover material (kN/m³)	21	
δ_u	Interface friction angle at the upper interface of material L_1(°)	22	
α_u	Apparent adhesion at upper interface of material L_1(kPa)	0.5	
δ_1	Interface friction angle at the lower interface of material L_1(°)	22	
α_1	Apparent adhesion at lower interface of material L_1(kPa)	0.5	
c	Cohesion of cover soil (kPa)	0.5	
ϕ	Angle of internal friction of cover soil (°)	25	
L	Slope length (m)	45	

FACTOR OF SAFETY CALCULATION:

H	Slope height	15.13	
Wa	Total weight of active wedge	781.58	
Wp	Total weight of passive wedge	28.42	
Na	Effective force normal to the failure plane of the active wedge (kN/m)	736.07	
Un	Resultant of the pore pressures acting perpendicular to the slope (kN/m)	0.00	
Uh	Resultant of the pore pressures acting on the interwedge surfaces (kN/m)	0.00	
Uv	Resultant of the vertical pore pressures acting on the passive wedge (kN/m)	0.00	
a		247.52	
b		-357.21	
c		50.16	
FS	Factor of safety (Cover soil/L_1 interface)	**1.29**	
T_1	Tensile force in the geosynthetic material L_1(kN)	**-73.45**	no tension
a		247.52	
b		-357.21	
c		50.16	
FS	Factor of safety (L_1/Subgrade interface)	**1.29**	

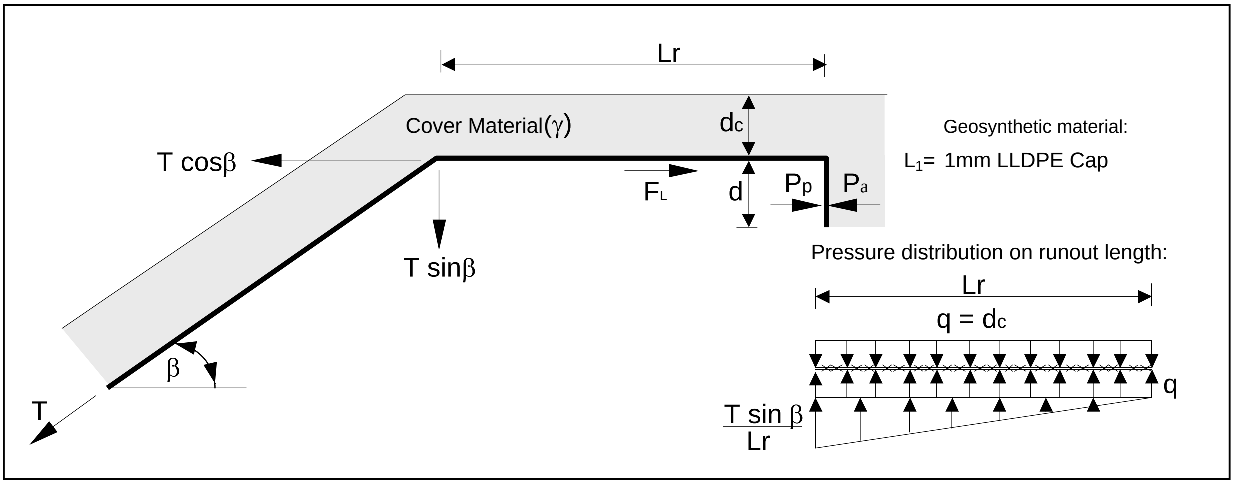
References:

Robert M Koerner “Designing with Geosynthetic” 1990 Fourth Edition

Jones & Dixon"The stability of geosynthetic landfill lining systems"

REQUIRED ANCHORAGE TRENCH

SLOPE WITH UNIFORM COVER THICKNESS, INFINITE SLOPE METHOD



β	Slope angle(°)	19.65
d_c	Thickness of the cover material (m)	1.00
γ	Unit weight of cover material (kN/m³)	18
δ_1	Interface friction angle between cover material and L_1 (°)	22
δ_2	Interface friction angle below the layer L_1 (°)	22
T_{ult}	Ultimate tensile strength of the geosynthetic material (kN/m)	10
CRF	Cumulative reduction factor for the geosynthetic material	1.5
T	Allowable tensile force in the geosynthetic material (kN/m)	6.67
L_r	Length of runout section (m)	**0.9**
d	Depth of anchor trench (m)	**0.9**

FACTOR OF SAFETY CALCULATION:

q	Cover material surcharge (kN/m)	18.00
F_L	Friction force below the layer L_1 (kN/m)	7.00
K_a	Coefficient of active earth pressure	0.45
P_a	Active earth pressure (kN/m)	10.69
K_p	Coefficient of passive earth pressure	2.20
P_p	Passive earth pressure (kN/m)	51.63

FACTOR OF SAFETY - ANCHORAGE TRENCH:

FS	**Factor of Safety against pullout of the layer L_1**	**7.64**

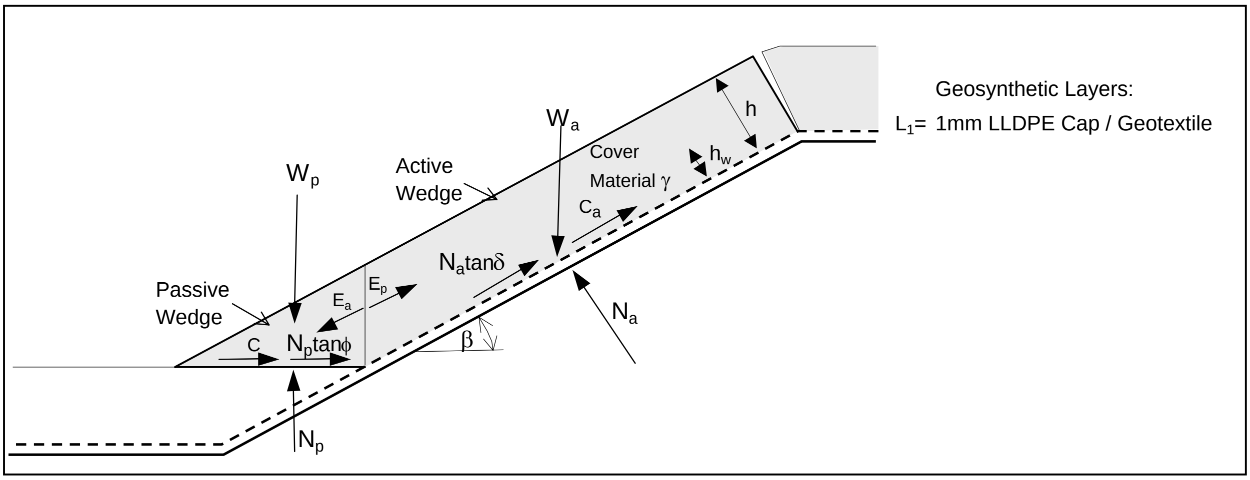
References:

Robert M Koerner “Designing with Geosynthetic” 1990 Fourth Edition

Jones & Dixon"The stability of geosynthetic landfill lining systems"

SLOPE STABILITY CALCULATION

UNIFORM COVER THICKNESS, FINITE SLOPE METHOD Jones & Dixon



INPUT DATA TO SHADED BOXES

β	Slope angle(°)	10.49	1 in 5.4
h	Thickness of the cover material (m)	1.00	
γ	Unit weight of cover material (kN/m³)	18	
h_w	Thickness of saturated cover material (m)	0.1	
γ_s	Saturated unit weight of the cover material (kN/m³)	21	
δ_u	Interface friction angle at the upper interface of material L_1(°)	22	
α_u	Apparent adhesion at upper interface of material L_1(kPa)	0.5	
δ_1	Interface friction angle at the lower interface of material L_1(°)	22	
α_1	Apparent adhesion at lower interface of material L_1(kPa)	0.5	
c	Cohesion of cover soil (kPa)	0.5	
ϕ	Angle of internal friction of cover soil (°)	25	
L	Slope length (m)	105	

FACTOR OF SAFETY CALCULATION:

H	Slope height	19.12	
Wa	Total weight of active wedge	1871.14	
Wp	Total weight of passive wedge	50.36	
Na	Effective force normal to the failure plane of the active wedge (kN/m)	1736.91	
Un	Resultant of the pore pressures acting perpendicular to the slope (kN/m)	102.97	
Uh	Resultant of the pore pressures acting on the interwedge surfaces (kN/m)	0.05	
Uv	Resultant of the vertical pore pressures acting on the passive wedge (kN/m)	0.27	
a		334.98	
b		-796.67	
c		64.03	
FS	Factor of safety (Cover soil/L_1 interface)	**2.29**	
T_1	Tensile force in the geosynthetic material L_1(kN)	**-460.37**	no tension
a		334.98	
b		-796.67	
c		64.03	
FS	Factor of safety (L_1/Subgrade interface)	**2.29**	

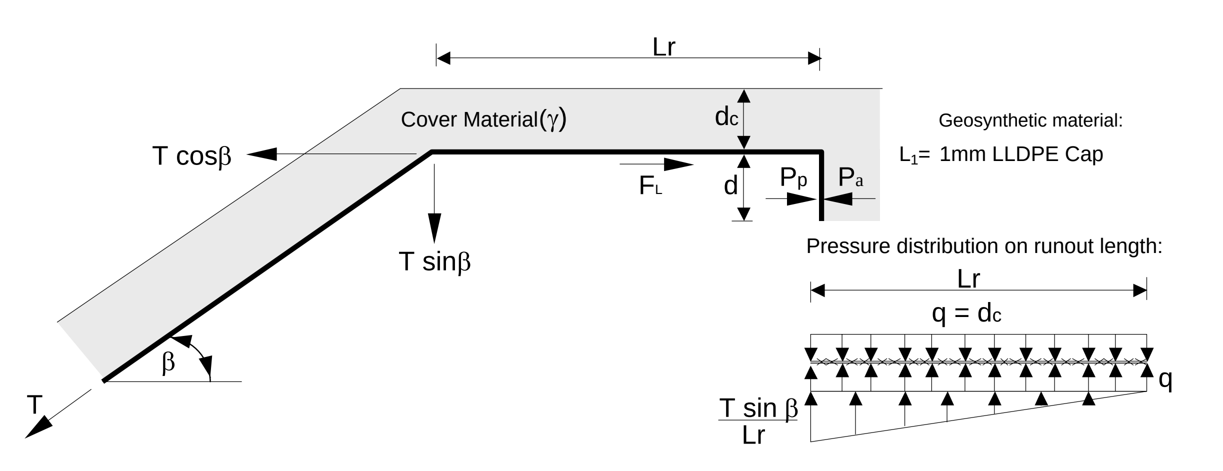
References:

Robert M Koerner “Designing with Geosynthetic” 1990 Fourth Edition

Jones & Dixon"The stability of geosynthetic landfill lining systems"

REQUIRED ANCHORAGE TRENCH

SLOPE WITH UNIFORM COVER THICKNESS, INFINITE SLOPE METHOD



β	Slope angle(°)	10.49
d_c	Thickness of the cover material (m)	1.00
γ	Unit weight of cover material (kN/m³)	18
δ_1	Interface friction angle between cover material and L_1(°)	22
δ_2	Interface friction angle below the layer L_1(°)	22
T_{ult}	Ultimate tensile strength of the geosynthetic material (kN/m)	10
CRF	Cumulative reduction factor for the geosynthetic material	1.5
T	Allowable tensile force in the geosynthetic material (kN/m)	6.67
L_r	Length of runout section (m)	**0.9**
d	Depth of anchor trench (m)	**0.9**

FACTOR OF SAFETY CALCULATION:

q	Cover material surcharge (kN/m)	18.00
F_L	Friction force below the layer L_1 (kN/m)	6.79
K_a	Coefficient of active earth pressure	0.45
P_a	Active earth pressure (kN/m)	10.69
K_p	Coefficient of passive earth pressure	2.20
P_p	Passive earth pressure (kN/m)	51.63

FACTOR OF SAFETY - ANCHORAGE TRENCH:

FS	**Factor of Safety against pullout of the layer L_1**	**7.28**

References:

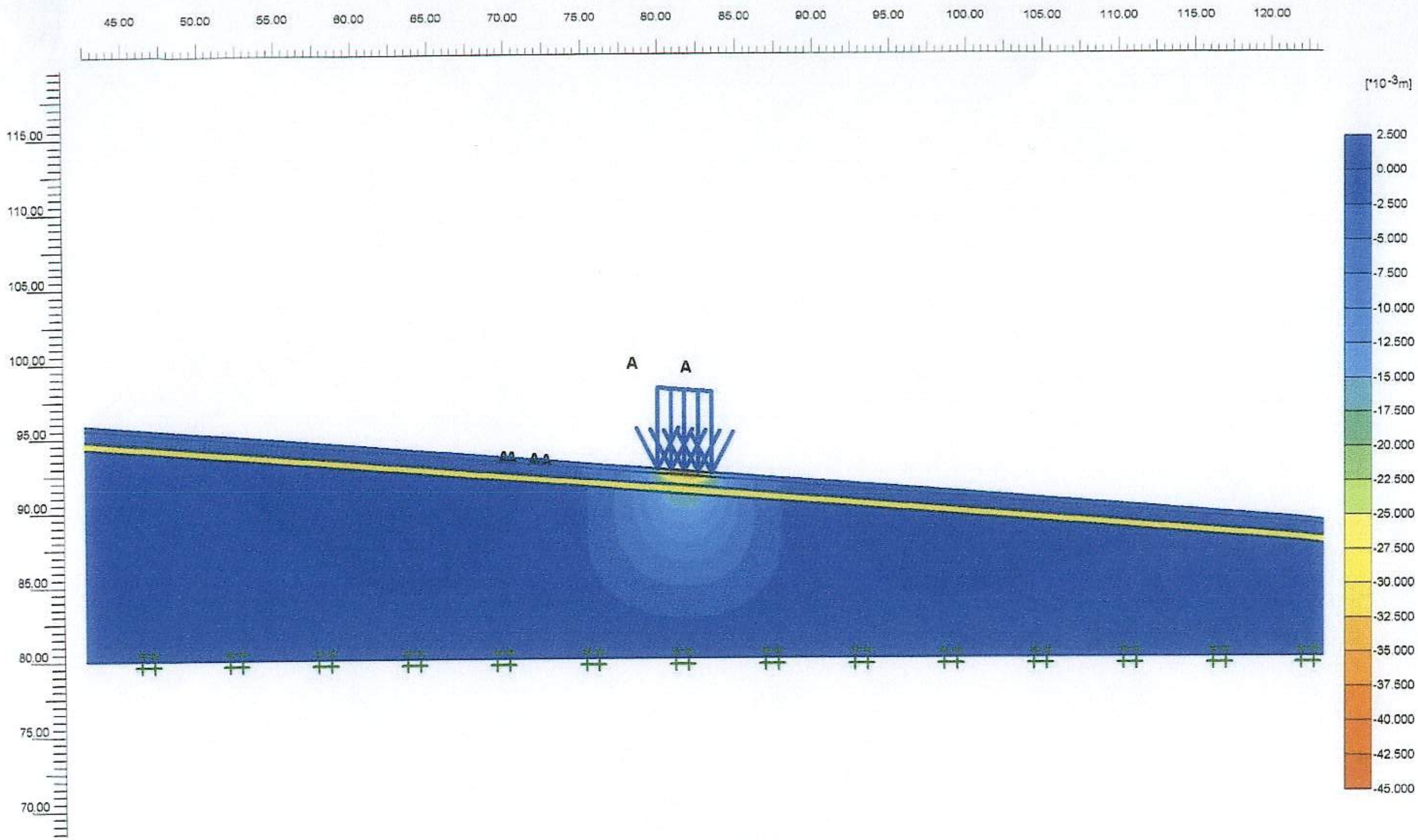
Robert M Koerner “Designing with Geosynthetic” 1990 Fourth Edition

Jones & Dixon"The stability of geosynthetic landfill lining systems"

APPENDIX SRA5

CAPPING SYSTEM INTEGRITY ANALYSES

PLAXIS V8



Vertical displacements (Uy)
Extreme Uy -43.79*10^-3 m

WR4446 / SRA

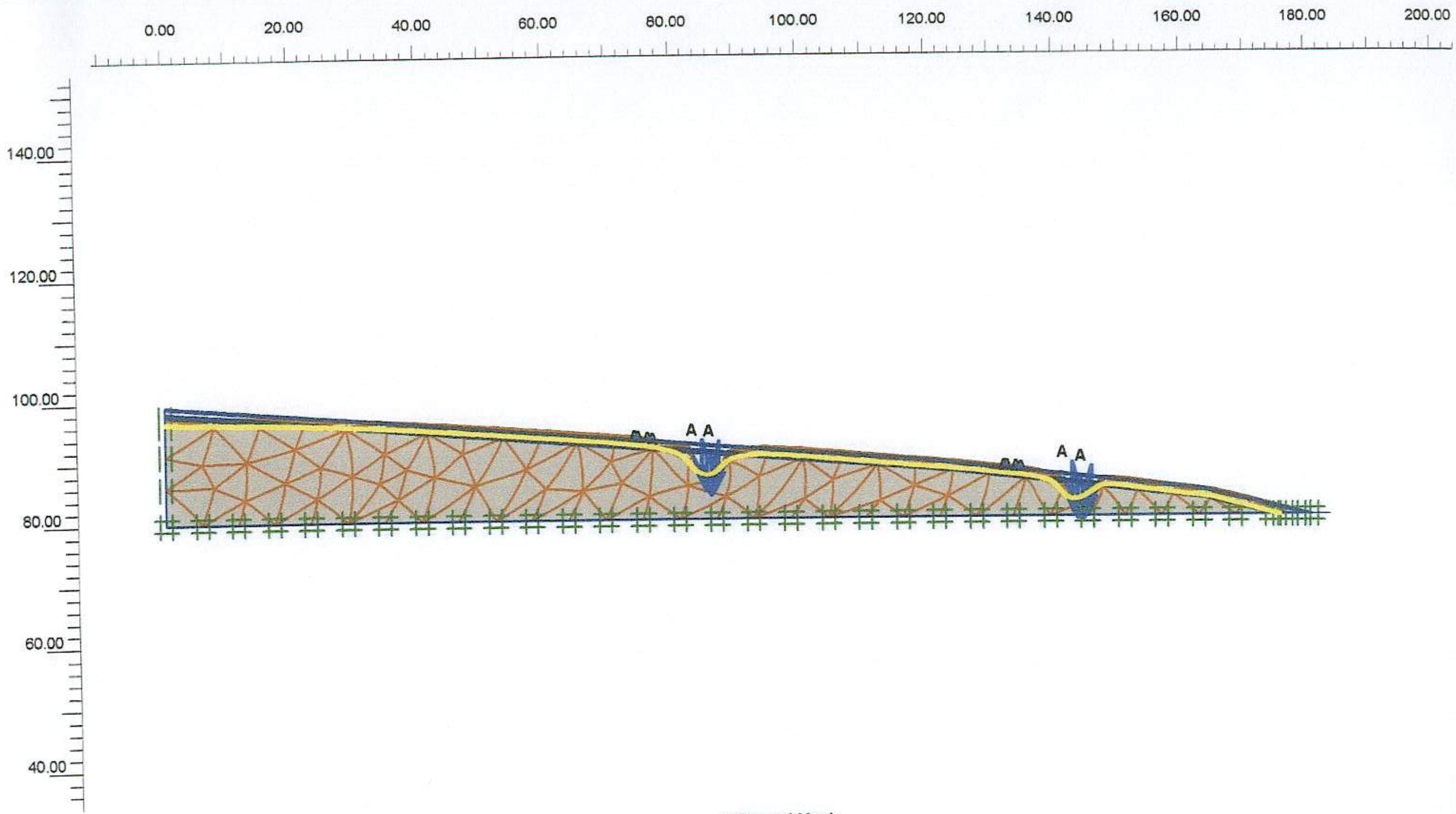
Appendix SRA5-1

PLAXIS
Finite Element Code for Soil and Rock Analyses

Project description			
Geomembrane Cap - D6 Bull Dozer / 360 Degree Tracked Excavator			
Project name: Pen-y-Bont PPC	Step: 15	Date: 31/08/04	User name: Encia Consulting Limited

Version 8.2.4.133

PLAXIS V8



Deformed Mesh
Extreme total displacement 49.99×10^{-3} m
(displacements scaled up 200.00 times)

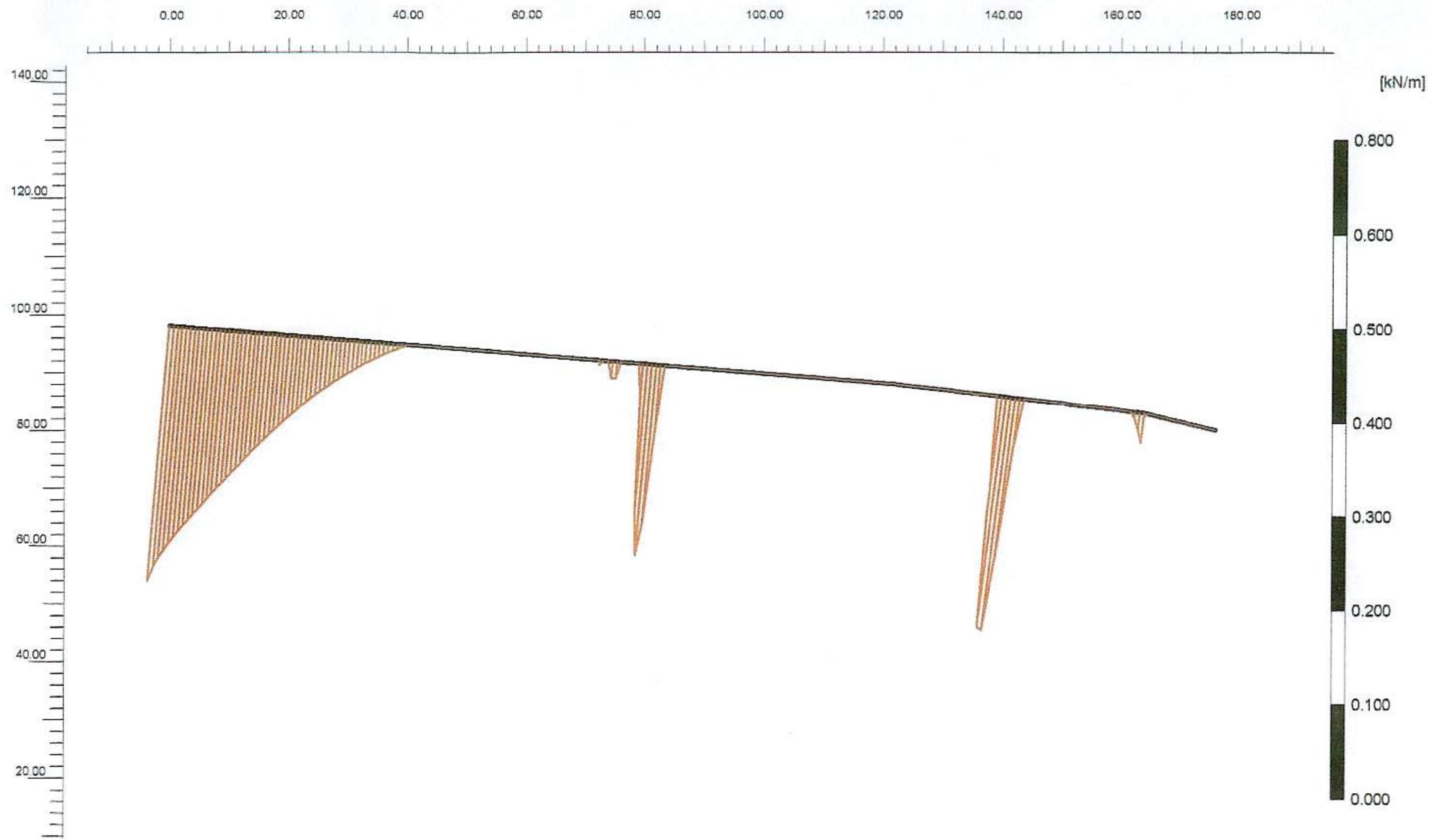
Appendix SRA S-2

WR4446/SRA

PLAXIS Finite Element Code for Soil and Rock Analyses	Project description: Geomembrane - D6 Bull Dozer / 360 Degree Tracked Excavator			
	Project name: Pen-y-Bont PPC	Step: 15	Date: 31/08/04	User name: Encia Consulting Limited

Version 8.2.4.133

PLAXIS 8.0



Axial forces
Extreme axial force 273.03×10^{-3} kN/m

WR446 / SRA

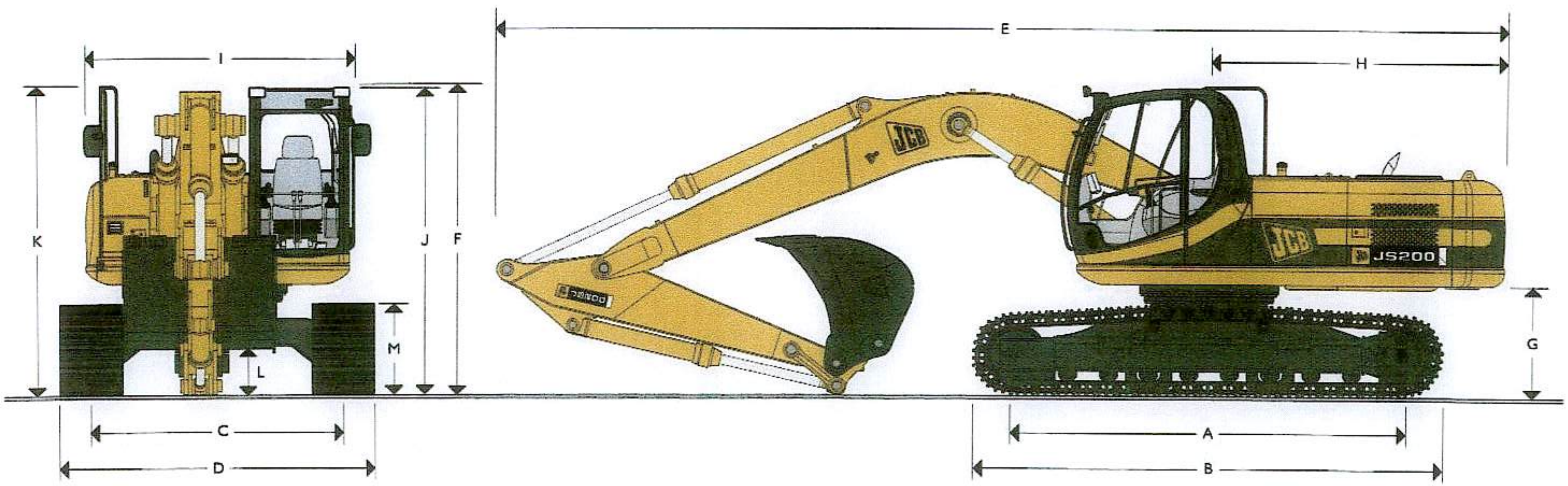
Appendix SRA5-3

Finite Element Code for Soil and Rock Analyses	Project description: Geomembrane Cap - D6 Bull Dozer / 360 Degree Tracked Excavator			
	Project name: Pen-y-Bont PPC	Step: 15	Date: 31/08/04	User name: Encia Consulting Limited

Version 8.2.4.133

JCB TRACKED EXCAVATOR | JS 200 NC/SC/LC

MAX. OPERATING WEIGHT 21190 kg
NETT ENGINE POWER 102 kW



STATIC DIMENSIONS

Dimensions in millimetres (ft-in)	NC	SC	LC
A Track length on ground	3370 (11-1)	3370 (11-1)	3660 (12-0)
B Undercarriage overall length	4170 (13-8)	4170 (13-8)	4460 (14-8)
C Track gauge	1990 (6-6)	22 0 (7-2)	2390 (7-10)
D Width over tracks (500mm trackshoes)	2490 (8-2)	2700 (8-10)	–
D Width over tracks (600mm trackshoes)	2590 (8-6)	2800 (9-2)	2990 (9-10)
D Width over tracks (700mm trackshoes)	2690 (8-10)	2900 (9-6)	3090 (10-2)

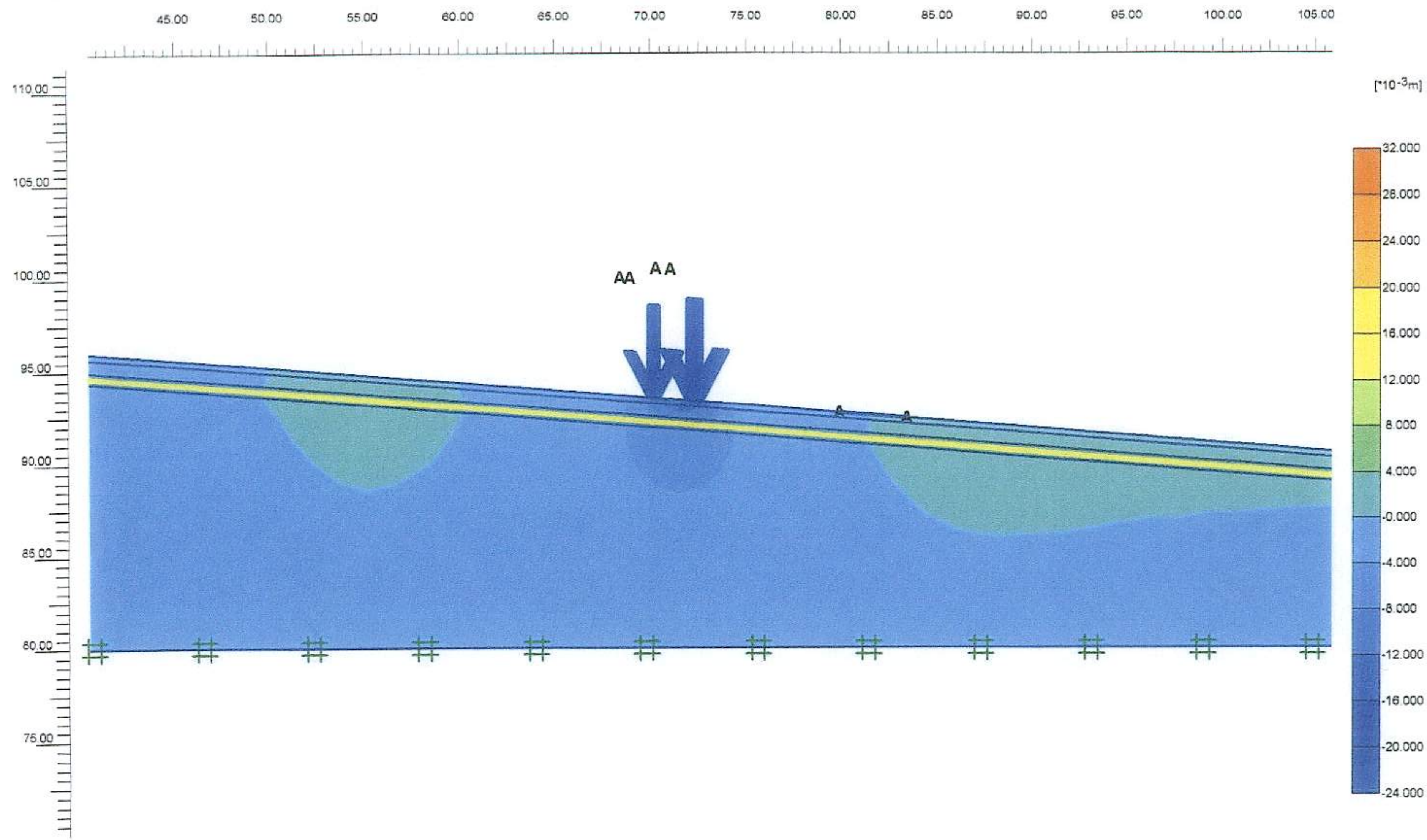
Dipper lengths	1.91m	2.40m	3.0m
E Transport length with Monoboom	9570 (31-5)	9560 (31-4)	9440 (31-0)
F Transport height with Monoboom	3055 (10-0)	3060 (10-0)	3025 (9-11)

Dimensions in millimetres (ft-in)	
G Counterweight clearance	1066 (3-6)
H Tailswing radius	2825 (9-3)

WR4446/SRA

Appendix SRA 5-4

PLAXIS V8



Vertical displacements (Uy)
Extreme Uy 31.28*10^{-3} m

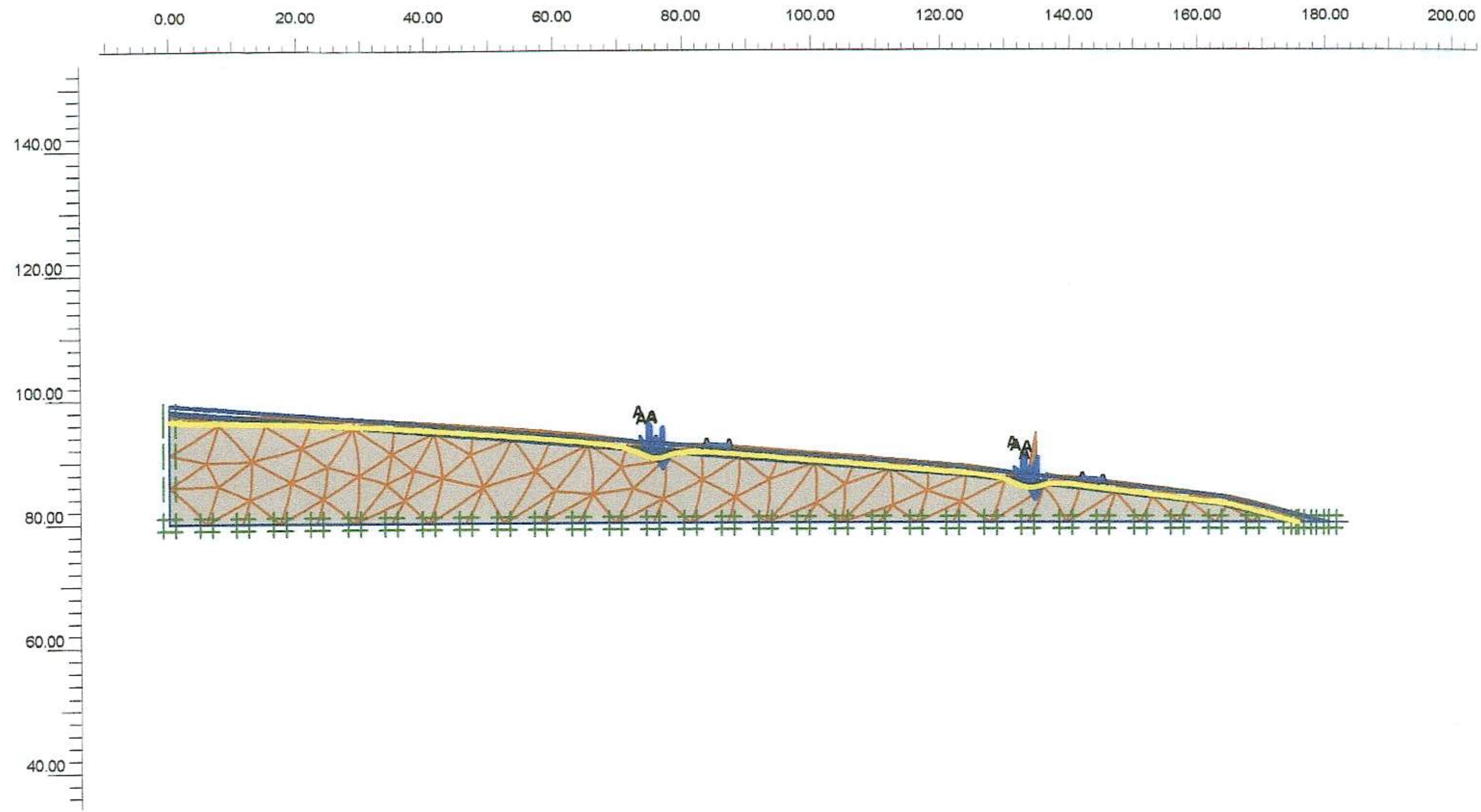
WR4446/SRA

Appendix SRA5-S

PLAXIS Finite Element Code for Soil and Rock Analyses	Project description Geomembrane Cap - JCB 3CX Backhoe Excavator			
	Project name Pen-y-Bont PPC	Step 11	Date 31/08/04	User name Encia Consulting Limited

Version 8.2.4.133

PLAXIS V8



Deformed Mesh
Extreme total displacement 37.90×10^{-3} m
(displacements scaled up 200.00 times)

WR4446/SRA

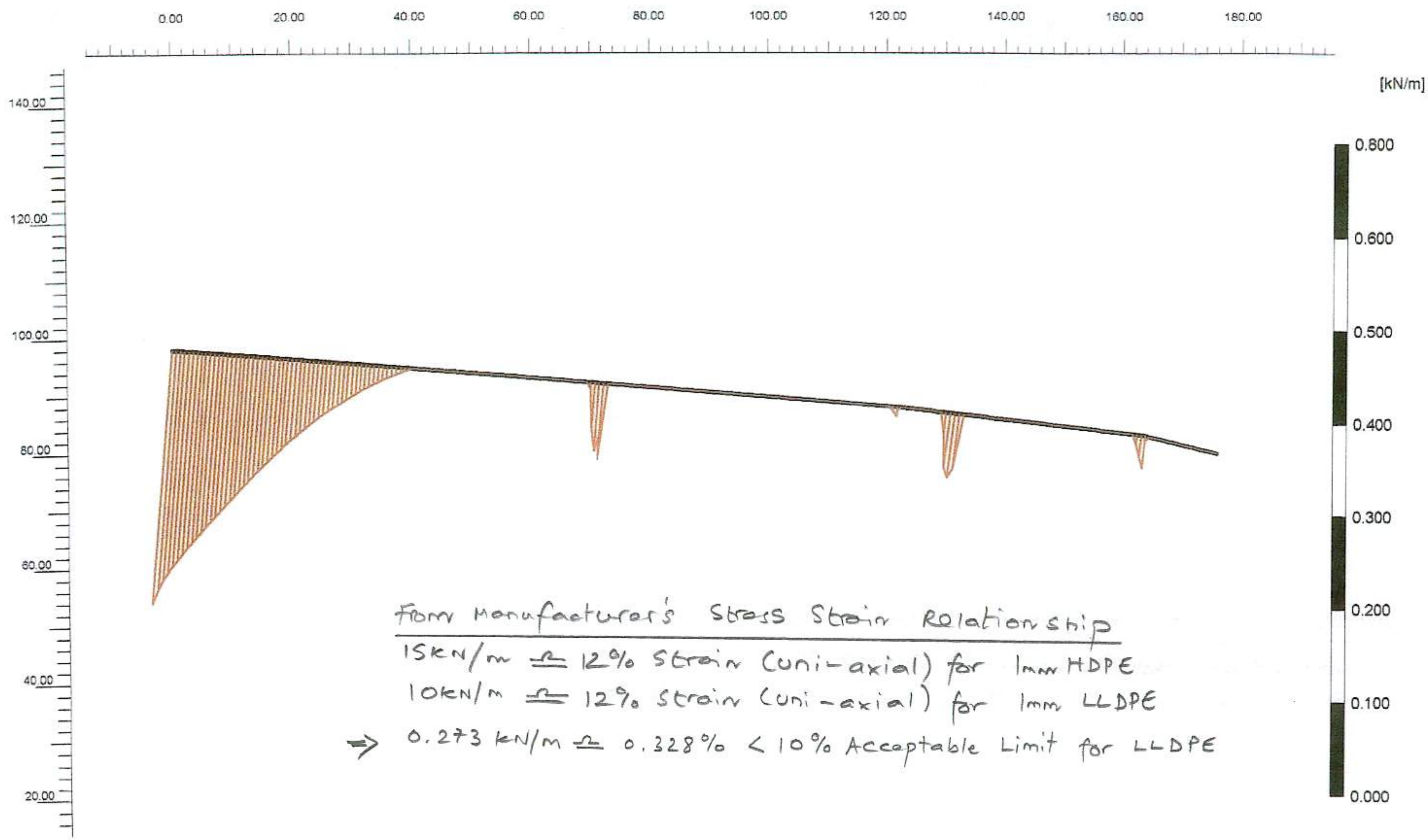
Appendix SRAS-6

PLAXIS
Finite Element Code for Soil and Rock Analyses

Project description			
Geomembrane Cap - JCB 3CX Backhoe Excavator			
Project name: Pen-y-Bont PPC	Step: 11	Date: 31/08/04	User name: Encia Consulting Limited

Version 8.2.4.133

PLAXIS 8.0



From manufacturer's Stress Strain Relationship

15kN/m ≏ 12% strain (uni-axial) for 1mm HDPE
10kN/m ≏ 12% strain (uni-axial) for 1mm LLDPE
⇒ 0.273 kN/m ≏ 0.328% < 10% Acceptable Limit for LLDPE

Axial forces
Extreme axial force 273.94*10^-3 kN/m

WR4446/SRA

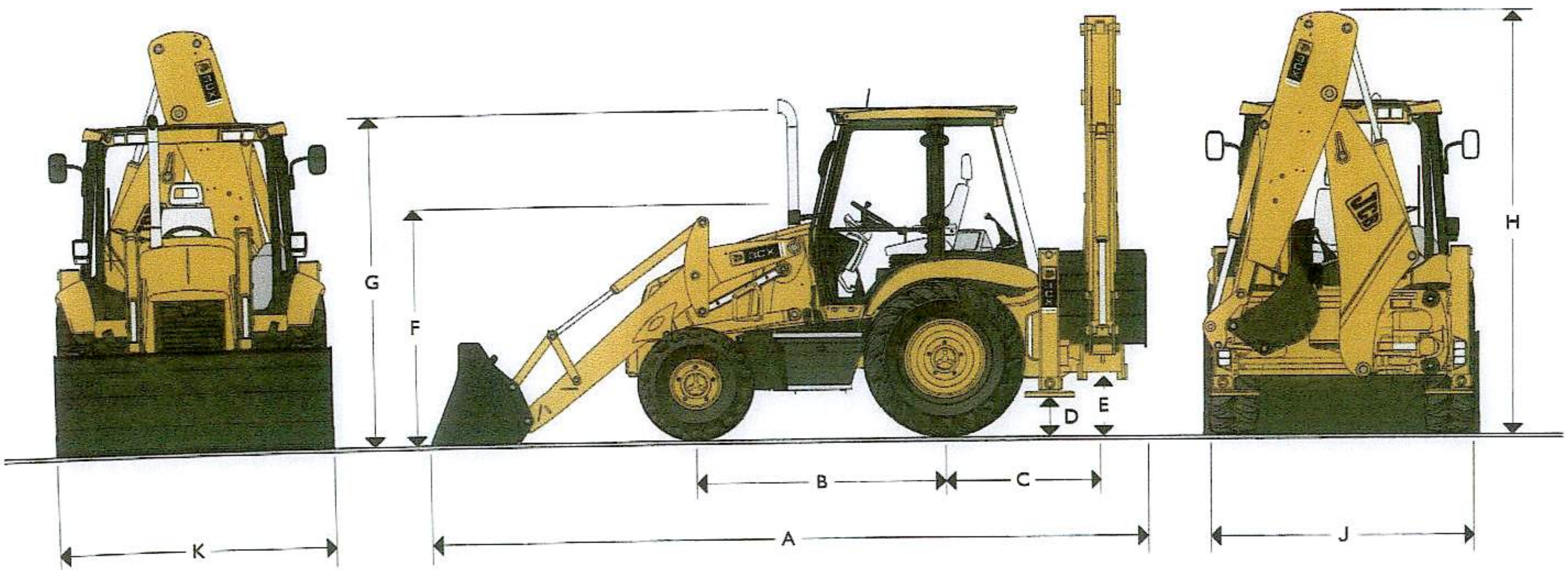
Appendix SRAS-7

PLAXIS Finite Element Code for Soil and Rock Analyses	Project description: Geomembrane Cap - JCB 3CX Backhoe Excavator			
	Project name: Pen-y-Bont PPC	Step: 11	Date: 31/08/04	User name: Encia Consulting Limited

Version 8.2.4.133

JCB BACKHOE LOADER | 3CX SITEMASTER

MAX. ENGINE POWER	61.5kW (83hp) or 68.5kW (92hp)
MAX. BACKHOE DIG DEPTH:	5.46 metres (17ft 11in)
MAX. LOADER CAPACITY:	$1.0m^3$ ($1.32yd^3$)



CB8002

STATIC DIMENSIONS

JCB Backhoe Loaders feature heavy duty one piece mainframe, componentised driveline and fully enclosed engine compartment. All fluid and component compartments are lockable using the ignition key.

Machine model	3CX Sitemaster
	m (ft-in)
A Total travel length	5.62 (18-5)
B Axle centreline distance	2.17 (7-1)
C Slew centre to rear axle centre distance	1.36 (4-6)

Machine model	3CX Sitemaster
	m (ft-in)
H Total travel clearance	3.61 (11-10)
J Rear frame width	2.36 (7-9)
K Shovel width	2.35 (7-8)

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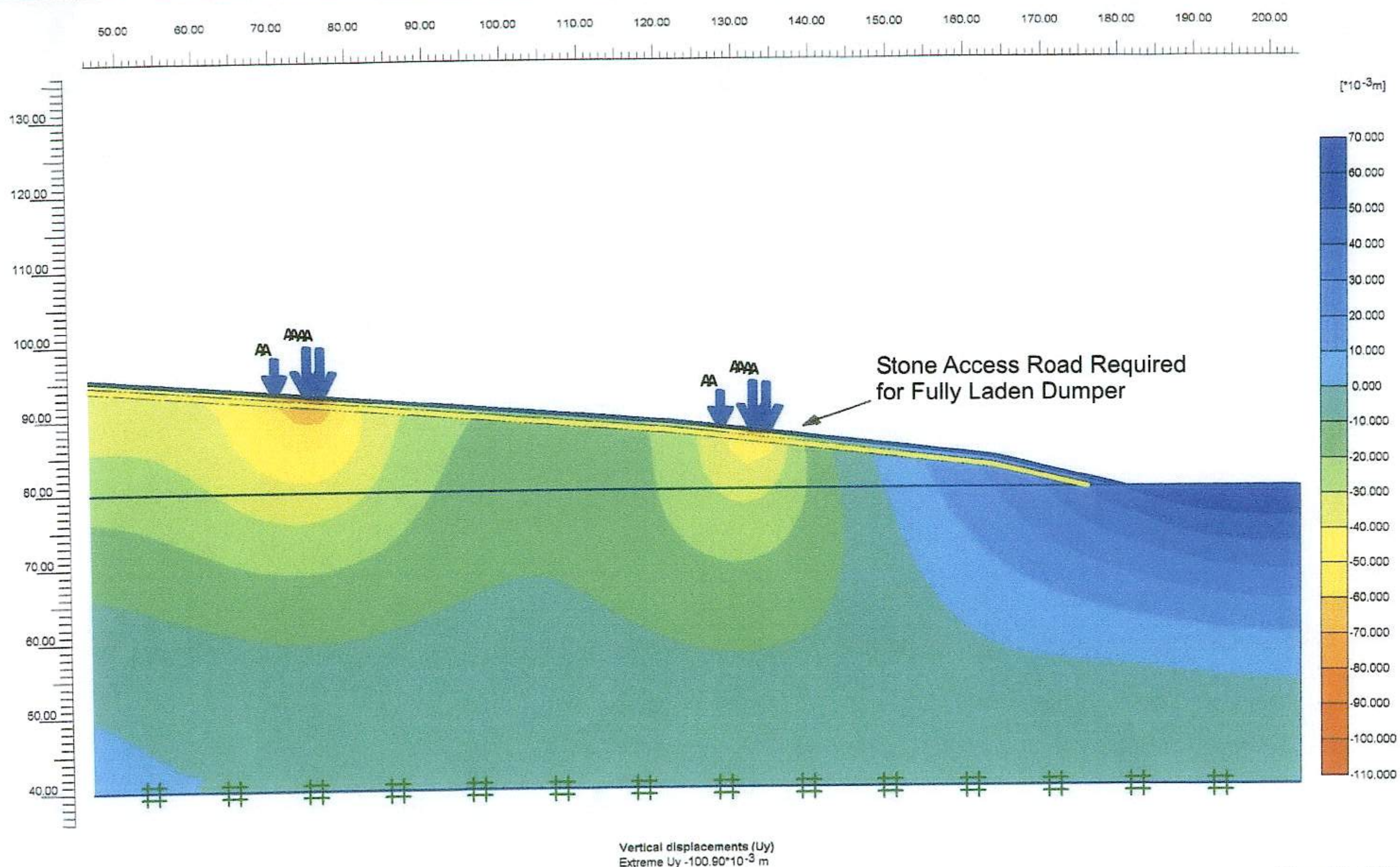
Appendix SRA5-8

PLAXIS V8



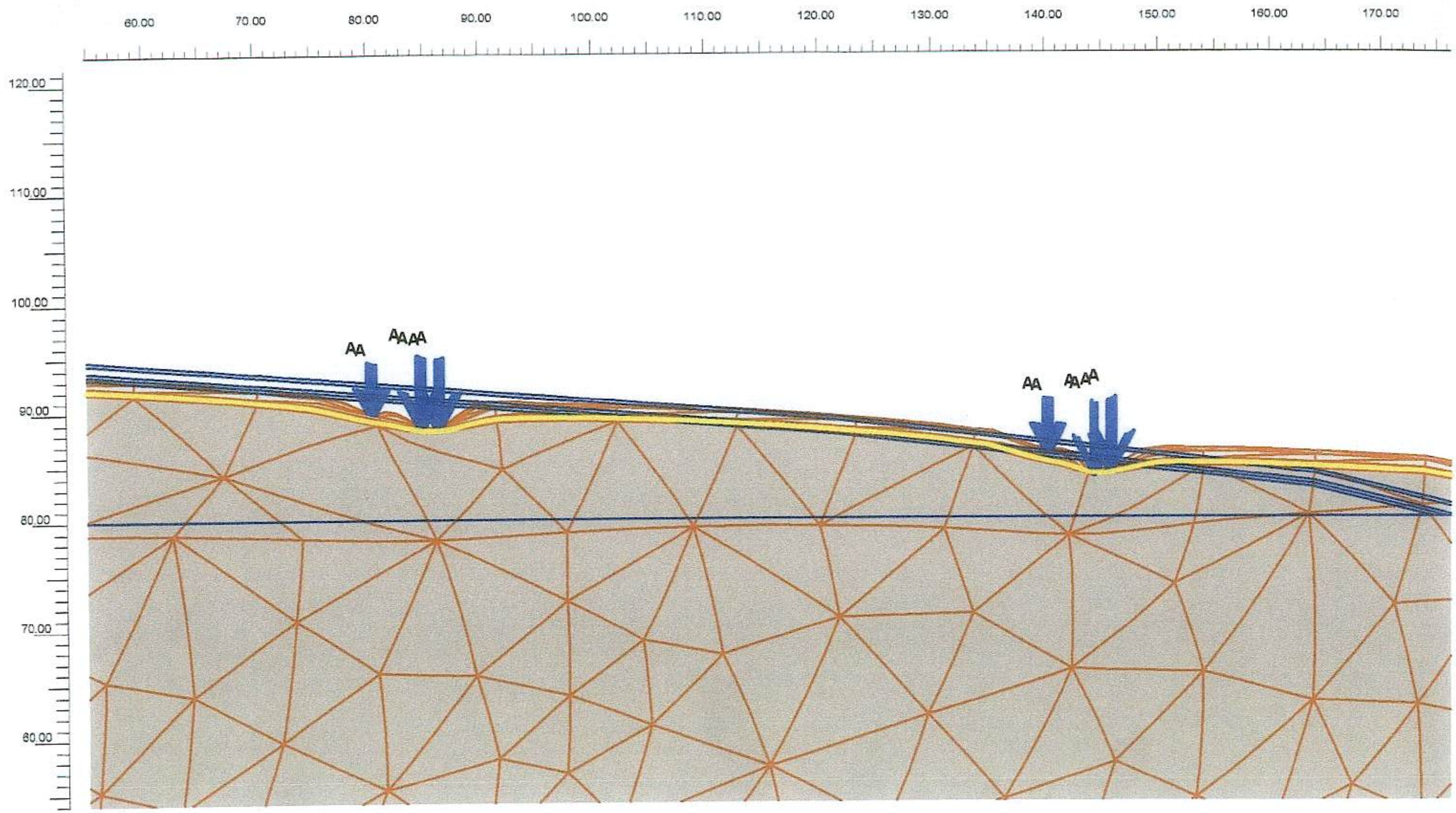
WR4446/SRA

Appendix SRA 5 - 9

PLAXIS Finite Element Code for Soil and Rock Analyses	Project description Geomembrane Cap With Haul Road - Fully Laden Dump Truck		
	Project name Pen-y-Bont PPC	Step 7	Date 31/08/04	User name Encia Consulting Limited

Version 8.2.4.133

PLAXIS V8



Deformed Mesh
Extreme total displacement 253.15×10^{-3} m
(displacements scaled up 50.00 times)

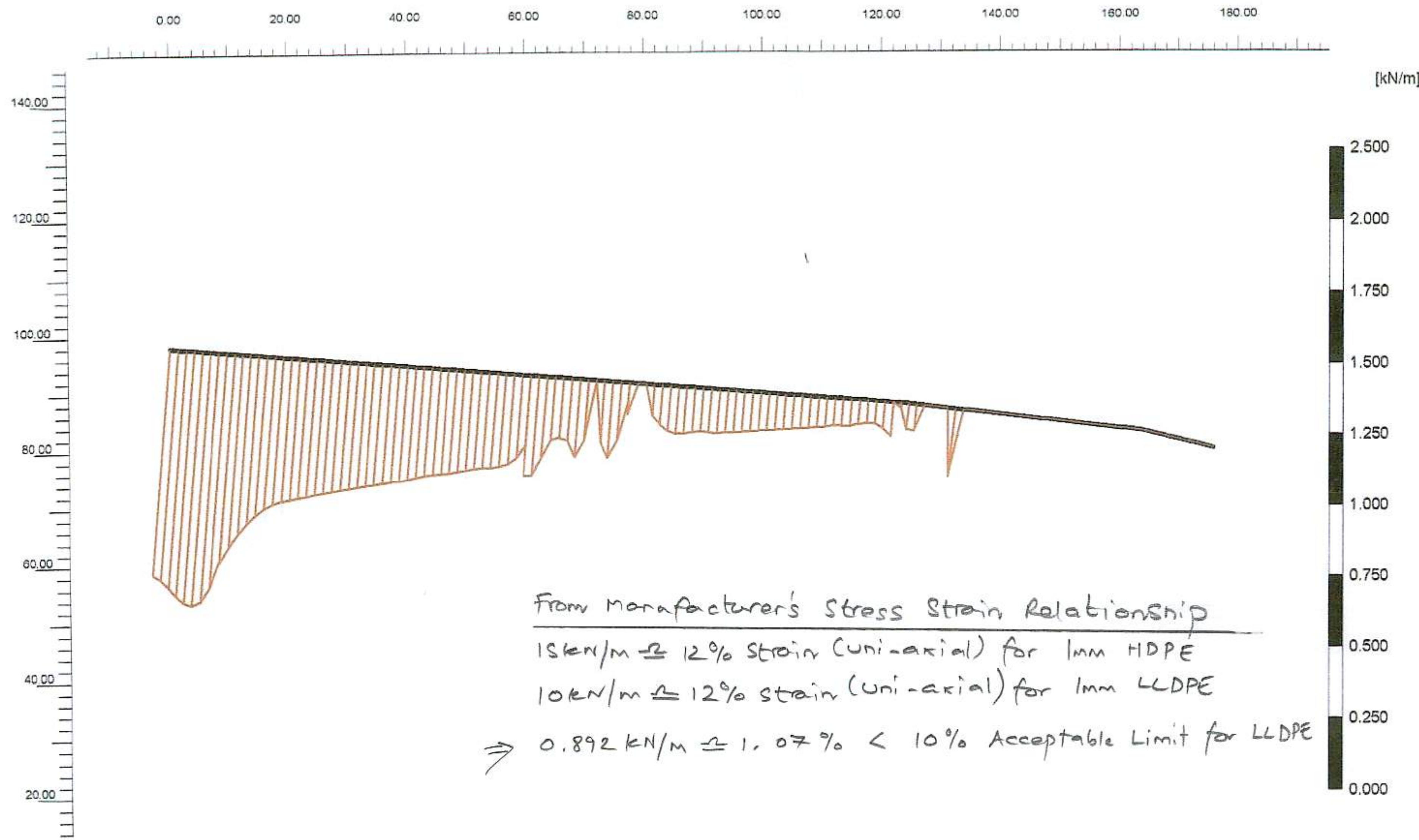
WR4446/SRA

Appendix SRAS-10

PLAXIS Finite Element Code for Soil and Rock Analyses	*Project description* Geomembrane Cap With Haul Road - Fully Laden Dump Truck			
	Project name Pen-y-Bont PPC	*Step* 7	*Date* 31/08/04	*User name* Encia Consulting Limited

Version 8.2.4.133

PLAXIS 8.0



From manufacturer's Stress Strain Relationship

15kN/m ≏ 12% Strain (uni-axial) for 1mm HDPE

10kN/m ≏ 12% strain (uni-axial) for 1mm LLDPE

⇒ 0.892 kN/m ≏ 1.07% < 10% Acceptable Limit for LLDPE

Axial forces
Extreme axial force 891.92*10^{-3} kN/m

WR4446/SRA

Appendix SRAS-11

PLAXIS
Finite Element Code for Soil and Rock Analyses

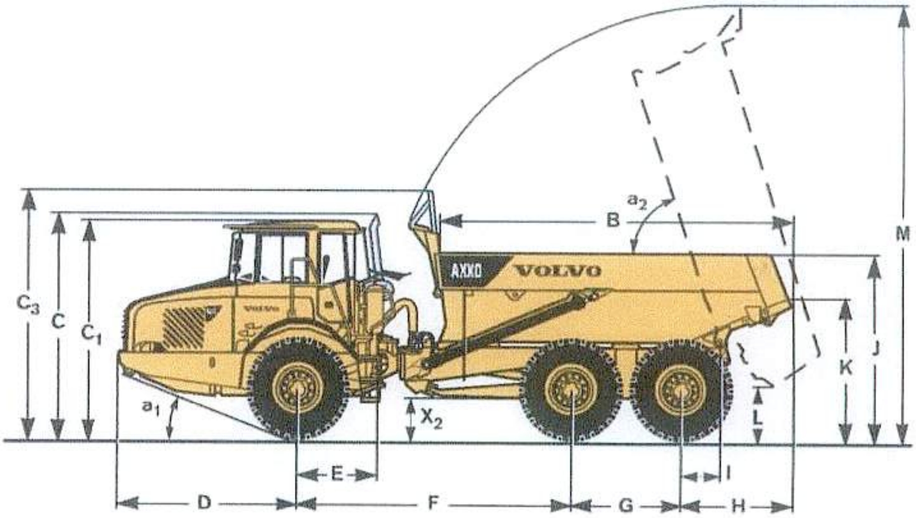
Project description			
Geomembrane Cap With Road - Fully Laden Dump Truck			
Project name: Pen-y-Bont PPC	Step: 7	Date: 31/08/04	User name: Encia Consulting Limited

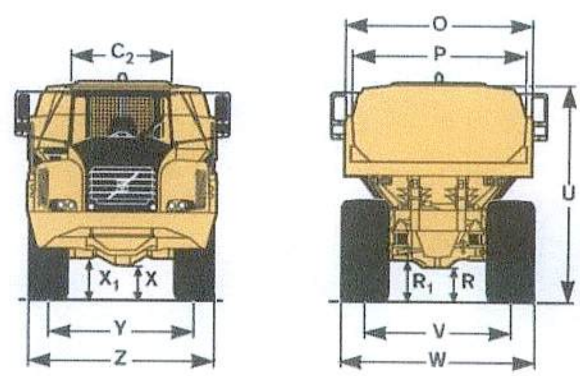
Version 8.2.4.133

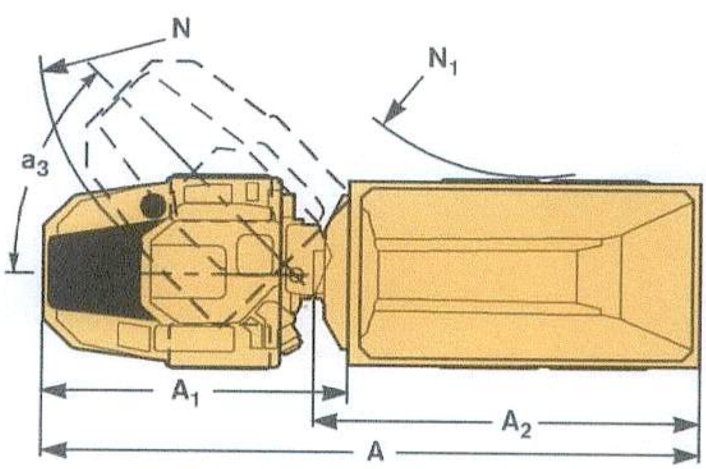
Specifications A25D – A30D

Pos	Metric (mm)		Imperial (Feet)	
	A25D	A30D	A25D	A30D
A	10 220	10 297	33'6"	33'9"
A_1	4 954	4 954	16'3"	16'3"
A_2	5 764	6 002	18'11"	19'8"
B	5 152	5 339	16'11"	17'6"
C	3 428	3 428	11'3"	11'3"
C_1	3 318	3 318	10'11"	10'11"
C_2	1 768	1 768	5'10"	5'10"
C_3	3 760	3 834	12'4"	12'7"
D	2 764	2 764	9'1"	9'1"
E	1 210	1 210	4'0"	4'0"
F	4 175	4 175	13'8"	13'8"
G	1 670	1 670	5'6"	5'6"
H	1 610	1 688	5'3"	5'6"
I	608	608	2'0"	2'0"
J	2 778	2 856	9'1"	9'4"
K	2 102	2 181	6'11"	7'2"
L	677	686	2'3"	2'3"
M	6 559	6 592	21'6"	21'8"
N	8 105	8 105	26'7"	26'7"
N_1	4 079	4 037	13'5"	13'3"
O	2 700	2 900	8'10"	9'6"
P	2 490	2 706	8'2"	8'11"
R	512	513	1'8"	1'8"
R_1	634	635	2'1"	2'1"
U	3 257	3 310	10'8"	10'10"
V	2 258	2 216	7'5"	7'3"
V*	-----	2 258	-----	7'5"
W	2 859	2 941	9'5"	9'8"
W*	-----	2 859	-----	9'5"
X	456	456	1'6"	1'6"
X_1	581	582	1'11"	1'11"
X_2	659	659	2'2"	2'2"
Y	2 258	2 216	7'5"	7'3"
Y*	-----	2 258	-----	7'5"
Z	2 859	2 941	9'5"	9'8"
Z*	-----	2 859	-----	9'5"
a_1	23,5°	23,5°	-----	-----
a_2	74°	70°	-----	-----
a_3	45°	45°	-----	-----

A25D: Unloaded machine with 23,5R25
A30D: Unloaded machine with 750/65R25
* A30D with optional 23,5R25 tires







WR4446/SRA

Appendix SRAS-12