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PhD Thesis:

ESTUDIO DE LA ESTABILIDAD DE LOS TALUDES EN UNA MINA OPERATIVA EXCAVADA EN SUELO DURO/ROCA BLANDA

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DECLARATION

I declare that this thesis is my own work. It is being submitted for the Degree of Doctor of Philosophy at the Polytechnic University of Madrid (Universidad Politécnica de Madrid). It has not been submitted before for any other degree or examination at any other University.

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RESUMEN

A medida que los precios de los metales bajan en los mercados y las leyes minerales también comienzan a disminuir, existe la necesidad de que las compañías mineras optimicen sus operaciones y los diseños de las cortas, reduciendo las ratios de desmonte mediante aumentos en los ángulos de los taludes.

El requisito de estabilidad en estos taludes se puede acometer utilizando diferentes filosofías. En la mina de Las Cruces en el sur de España, el enfoque general para garantizar la estabilidad de taludes de la corta, sigue una política de cero daños.

Como una mina española moderna, Las Cruces está comprometida con el uso de las mejores tecnologías y prácticas disponibles a la fecha, con el objetivo de garantizar la producción de mineral en un ambiente seguro. Desde una perspectiva práctica, esto implica un afinado diseño de la corta un mapeo geotécnico continuo de los taludes excavados para garantizar su conformidad con el diseño, y el uso de unas tecnologías de vanguardia de auscultación geotécnica para complementar la vigilancia continua.

El propósito de esta tesis es planificar e identificar cuáles han sido, a lo largo de estos años, los procesos implementados para garantizar la estabilidad geotécnica y la seguridad en la mina de Las Cruces, desde las primeras actividades de la excavación del pre-stripping, así como las mejoras introducidas para la estabilidad de taludes de la corta a cielo abierto, destacando los siguientes elementos cruciales:

1. Caracterización geomecánica de los materiales: los parámetros resistentes y deformacionales utilizados en el análisis son fundamentales para cualquier diseño. Esta tesis investiga la investigación práctica de las características de los materiales a lo largo de la vida de la mina, desde su fase de proyecto hasta la fecha, haciendo hincapié en los métodos utilizados para la actualización periódica de esta información. Los métodos para determinar estas características fueron predominantemente:
 - Trabajos de investigación, durante la fase de los diseños de ingeniería, a partir fundamentalmente de sondeos geotécnicos específicos.
 - Ensayos de laboratorio sobre las muestras obtenidas en los sondeos y, sobre muestras talladas in situ directamente de los taludes de la corta durante el desarrollo de la excavación.
 - Observaciones visuales y las determinaciones de la competencia de las rocas directamente realizadas durante la cartografía geotécnica de los bancos de los taludes.
 - Correlaciones de varias litologías con tipos de rocas similares conocidos en otras minas y más específicamente en minas de la Faja Pirítica Ibérica.
2. Diseño optimizado de la corta: a lo largo del desarrollo de la excavación de la corta minera, se realizaron mejoras en el diseño inicial de los taludes de la mina, utilizando para ello los datos geomecánicos de investigaciones complementarias de campo, basada en sondeos y ensayos de laboratorio e in-situ. Se utilizaron las configuraciones finales de la corta para proporcionar unas geometrías que han sido analizadas, mediante:

Análisis de estado límite 2D, con el programa SLIDE de Rocscience:

El trabajo consistió en la optimización del diseño de la corta original y una revisión de ese diseño incorporando:

- Planos de estratificación mapeados y de otras estructuras geológicas.
- Segregación de las margas, en dos niveles, superior e inferior, en términos de parámetros de resistencia.
- Parámetros de resistencia mejorados a partir de muestras talladas in situ, menos disturbadas que las muestras de sondeo, obtenidas directamente de los taludes y del fondo de la corta.
- Variaciones en el paleozoico, incluidas las pizarras problemáticas del muro en el lado sur de la mineralización.

Análisis mediante diferencias finitas con el programa FLAC3D, de Itasca:

En este análisis la corta minera se dividió en dos mitades. Una del lado sur y otra del lado norte, al objeto de reducir los requisitos y por lo tanto los plazos de procesamiento informático de los datos, y con ello el tiempo de cálculo. Con respecto a las margas, se utilizó un modelo constitutivo de strain-softening con acoplamiento hidromecánico, capaz de modelar con eficacia las disipaciones de presión de poro como resultado de la descompresión y de la rotación de esfuerzos principales en la cara de los taludes, tras la excavación, con una reducción general de las tensiones desestabilizantes.

Análisis mediante diferencias finitas con el programa FLAC2D, de Itasca:

Se analizó la geometría de la corta más desfavorable que se ubica en lado norte de la corta. Este cálculo se efectuó, al igual que en el caso anterior, también con acoplamiento hidro-mecánico. Sin embargo, la restricción a dos dimensiones en el análisis, posibilitó un refinamiento de la malla de cálculo desde los 5 m x 5 m, empleados en los dos análisis 3D, hasta 0.5 m x 0.5 m. Esto permitió una mejor modelización de los elementos con respeto a la hidrología de la sección analizada, y sobre todo, la consideración de planos de estratificación reconocidos en los inclinómetros, con comportamiento residual. La sección de cálculo bidimensional, también permitió incorporar en el modelo, la escombrera norte en su configuración final para verificar tanto su estabilidad, como su efecto sobre los taludes de la corta.

Análisis mediante elementos finitos con el programa Phase2 de Rocscience:

En este caso, el análisis se efectuó como un método para verificar el análisis de FLAC 2D anterior, como chequeo adicional, formando parte de un análisis QAQC. Para ello se utilizó el programa de elementos finitos Phase2 para resolver el análisis tenso-deformacional. En este caso no se consideró el acoplamiento hidromecánico, al objeto de cuantificar la importancia de este factor como reducción de la presión de poros, para la optimización de los taludes de cortas a cielo abierto a gran escala excavadas en medios impermeables.

3. Optimización geotécnica de las voladuras

Se llevó a cabo una optimización de las voladuras en la corta. Este enfoque fue distinto al que habitualmente realiza un ingeniero especialista de voladuras, centrando la atención en las características geotécnicas del terreno a partir de los datos procedentes del mapeo de los taludes y de los ensayos de laboratorio, para caracterizar los litotipos.

A partir de estos datos de campo y laboratorio, se desarrolló una metodología para derivar los parámetros de resistencia y elasticidad dinámica del macizo rocoso. A partir de estos parámetros, se determinaron las características de la propagación de ondas de voladura y se compararon con los datos recogidos de pruebas de onda semilla de las voladuras de campo, con resultados favorables.

Las estimaciones de la profundidad del daño inducido por la vibración de la voladura en el macizo rocoso se realizaron utilizando los parámetros de propagación y la resistencia a la roca. Finalmente, se propuso un mayor desarrollo del uso del factor de daño de D del criterio de rotura de Hoek-Brown, y se realizó un análisis posterior en los taludes, previamente excavados para simular los modos de falla observados. La comparación con observaciones in situ fue favorable y la metodología se utilizó posteriormente para analizar y mejorar la voladura para la excavación de los taludes en las zonas más profundas de la mina, con resultados altamente efectivos y demostrables. La reconciliación de la observación de los efectos de las voladuras se llevó a cabo mediante la incorporación de la metodología de reconciliación de Stacey en el mapeo geotécnico de taludes excavados.

4. Monitoreo de la corta - Sistemas de mapeo y vigilancia.

El monitoreo intensivo de los taludes se lleva a cabo desde las primeras etapas de la excavación de la corta utilizando, para ello, inclinómetros, piezómetros y una estación topográfica total robotizada.

En esta tesis, el autor ha enfatizado la diferencia entre disponer de:

- Un diseño apropiado inicial y su correcta implementación que garantiza una alta probabilidad de estabilidad de los taludes durante los trabajos de excavación.
- Un sistema de monitoreo continuo de los taludes mediante instrumentación geotécnica, generalmente realizado por un ingeniero geotécnico in situ que asegura que cualquier inestabilidad potencial sea detectada, rastreada y comunicada al equipo operacional, de forma que se pueda mantener en todo momento las adecuadas condiciones de seguridad.

Esta diferenciación entre la estabilidad de los taludes (que constituye la primera defensa para garantizar operaciones mineras seguras) y el monitoreo de los taludes (que es la última defensa en la seguridad de la mina) se

considera fundamental para un buen rendimiento geotécnico en cualquier operación minera.

Esta tesis explora estos aspectos, implementando nuevas técnicas innovadoras en lugar de las técnicas estándar utilizadas en la minería histórica, proporcionando notables mejoras que podrían implementarse fácilmente en otras operaciones mineras en el futuro para garantizar que se implementen las mejores prácticas disponibles.

ABSTRACT

As metal prices drop in the commodity markets and mineral grades also begin to lessen, there is a need for mining companies to attempt to optimise their pit operations and designs, reducing stripping ratios via increases in pit slope angles.

The requirement for stability in these pit slopes can be accomplished using different philosophies. At Las Cruces mine in Southern Spain a zero harm policy defines and defends the general approach to slope stability.

As a modern Spanish mine, Las Cruces is committed to the use of best available technology and practices with the aim of guaranteeing the production of ore in a safe environment. From a practical perspective, this involves high quality pit design, on-going geotechnical mapping of exposed slopes to ensure conformity with design, and use of leading geotechnical surveillance technology to complement on-going monitoring.

The purpose of this thesis is to map out and identify the process of ensuring stability and geotechnical safety, at Las Cruces, since first stripping activities of open pit slope stability maintenance and improvements, highlighting the following crucial elements:

1. Material Characterisations – Fundamental to any design are the parameters utilised in analysis. This thesis investigates the practical investigation of material characteristics over the mine life to date and the methods utilised for the periodic update of this information. The methods of determining these characteristics were predominately:
 - Intrusive investigation work comprising bespoke geotechnical drillholes.
 - Laboratory testing on samples obtained from drillholes and bulk samples obtained from the pit slopes during excavation development.
 - Visual observations and direct rock competence determinations undertaken during slope cartography.
 - Correlations of various rock lithologies with known similar rock types in other mines and more specifically in mines of the pyritic belt.
2. Optimised Pit Design – Using the geo-mechanical data from the site investigation based on boreholes and laboratory and in-situ testing, improvements were made to the initial design of the slopes undertaken for the mine, Final pit shell configurations were utilised to provide a geometry for:

2D limit-state analysis, with Rocscience commercial programme SLIDE.

This consisted of the optimisation of the original pit design and a review of that design incorporating:

- Known bedding planes and other geological structures.
- Segregation of the upper and lower marls in terms of resistance parameters.
- Improved resistance parameters from less disturbed bulk samples obtained directly from the pit slopes and floor.
- Variations in the Palaeozoic, including the challenging footwall shales adjacent to the south side of mineral ore body.

FLAC3D finite difference three-dimensional phased analysis.

The analysis was divided into a south side and a north side to reduce processing requirements and time frames. With respect to the marls, an important hydro-mechanical coupling process was fully incorporated which was able to effectively model pore pressure dissipations as a result of principle stress rotation near the excavation slopes and an overall reduction in stress.

FLAC2D finite difference two-dimensional phased analysis.

A specific worst case geometry was analysed on the north side of the pit, again taking into account a coupling of the hydrological and mechanical elements. However the two dimensional nature of the analysis enabled a refinement of the mesh from 5m x 5m down to 0.5m x 0.5m. This allowed an improved modelling of the hydrological elements of the analysed section. The section also enabled the modelling of the mines adjacent north dump in its final configuration to check both its local stability state and also the effect on the pit.

Phase 2D finite element two-dimensional phased analyses.

This was undertaken as a method of checking the FLAC2D analysis for QAQC purposes. As such a completely unrelated stress-strain deformation analysis programme was utilised, PHASE2D. In addition the hydro-mechanical coupling method was not implemented to provide emphasis on the importance of coupling these interdependent factors for mining pit slope optimisation in large open pits.

3. Geotechnical Blast Optimisation – Optimisation of the blast activities was undertaken. This approach was distinct to that of the mine blast engineer with attention being focused upon laboratory data and in pit geotechnical mapping in order to better determine the characteristics of each lithotype present in the mining pit.

A methodology was developed to derive, from this laboratory and field data, rock mass dynamic elasticity and strengths parameters. From these parameters, corresponding blast propagation parameters were also determined and cross-checked against in field blast signature testing, with favourable results.

Estimations of the blast induced damage depth into the rock mass were made utilising the propagation and rock resistance parameters. Finally further development of the use of the Hoek-Brown damage factor was proposed and a back analysis was undertaken on previously blasted slopes in order to emulate observed failure modes. A comparison with onsite observations was favourable and the methodology was subsequently utilised to analyse and improve blasting for slope construction in the deeper areas of the mine, with highly effective demonstrable results. Observation reconciliation of the effects of this blasting was undertaken via the incorporation of the Stacey blast reconciliation methodology in geotechnical mapping of formed slopes.

4. Pit monitoring – Mapping and vigilance systems.

Intensive monitoring of the slopes is conducted from the very first stages of pre-stripping using inclinometers, piezometers and a robotic topographical total station.

In the mine, the author has emphasised the difference between:

- An appropriate on-paper design and its correct implementation which provides a high probability of slope stability during excavation works.
- On-going slope and instrumentation monitoring, usually undertaken by an onsite geotechnical engineer which ensures that any potential instabilities are tracked and highlighted to the operational team such that safety can be maintained.

This distinction between slope stability (the first defence for safe mining operations) and slope monitoring (the last defence) is considered fundamental for good mining geotechnical performance. This thesis explores these aspects, implementing innovating new techniques over those used in historic mine practices and provides improvements that could with ease be implemented in other mining operations in the future to ensure that best available practices are in place.

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1 INTRODUCTION

1.1 GENERAL CONTEXT

Las Cruces mineral exploitation is centred around an open pit, which has been advanced in various phases of a ten-year period; the first phase being an initial stripping to access the mineral, a second phase of pit widening to arrive at the slope design limits in the west, followed by 4 additional phases towards the East chasing out the mineral mass. Currently the mine is developing the sixth phase towards the East. The final dimensions of the mining pit will be, in its largest configuration, 1,500 metres long, 900 metres wide and 245 metres deep.

Las Cruces mine consists of the exploitation and development of a Volcanogenic Massive Sulphide deposit situated within the Iberian Pyrite Belt (IPB). The IPB represents a zone of historic mining development with the presence of many polymetallic mineral ore bodies.

The Las Cruces primary massive sulphide is essentially similar to other VMS deposits within the IPB. However, unlike other VMS deposits within the region, the gossan and supergene mineralisation at Las Cruces is undisturbed by historical mining activity or erosion, being extremely well preserved under approximately 150 metres of Tertiary deposits.

From the initial design stage, slope stability was a foremost concern, not only from the perspective of good mining practices with aspirations to develop a modern mine with good safety practices, but also due to uncertainties associated with the geotechnical behaviour of these Tertiary marls, with the mine being the deepest excavation in the world in these materials. These uncertainties, and the occurrence of the mining tailings breach at an adjacent mine in Aznalcollar in the late 90's determined that the geotechnical aspects at Las Cruces mine were well considered from the outset of mining development.

The general slope angle in the marls is 28°, stepped in benches of 10 metres height and 60° inclination. However in some areas there exist benches of 20 metres height and others of 10 metres height with 50° inclination angles.

1.2 OBJECTIVES

The principle objective of this thesis is to study the methods utilised to ensure geotechnical safety and stability at Las Cruces. These methods comprise principally, good initial design and on-going design improvements, instrumentation and in field vigilance, and geotechnical input on on-going mining operations.

This thesis records the works done by the author over the last eight years to advance certain design principles, improve vigilance techniques and develop new improved analysis techniques for operational activities within the mine such as overburden

stripping (free excavate verses blast assisted) and evaluation of blasting limits close to sensitive or final slope faces.

A number of concepts are visited consisting in:

- Improved rock type and behavioural characteristics.
- Slope instability observations and remediation methods.
- Design concepts and on-going year-by-year improvements.

Here new knowledge is gained on the effects of hydro-mechanical coupling, the merits of various analysis types, this new knowledge and the in depth analysis undertaken enabled the mine to extract significant quantities of mineral un contemplated in the original mine plan.

- Operative blasting geotechnical optimisation.

This thesis highlights new work in the optimisation of mine blasting, from a geotechnical perspective applying a first principles evaluation of blast related rock parameters derived from laboratory testing and geotechnical mapping, calibrated with in-situ signature blast.

The damage factor D utilised in the Hoek-Brown constitutive model was modified progressively within the slope in such a manner as to reduce geotechnical parameters to emulate the effects of blast damage. The results of these analyses in various lithotypes within the pit were compared to instabilities observed in the mine with good correspondence between observation and the results of final stability analysis.

Most importantly, the conclusions of this in depth analysis were successfully applied to an area of poor quality shales bordering high-grade mineral enabling the mine to safely extract the mineral with stable slopes.

Finally, improvements to the Holmberg and Persson blast propagation formula were undertaken which effectively eliminates the distinction between near and far field analysis, allow explicit calculation of wave propagation at any distance provided a blasthole stemming length and distance from the blasthole is known.

- Vigilance systems utilised at the mine (Visual, Instrumental and Topographical).

The thesis provides in depth analysis of the use of new techniques in mining:

Visual: Additional work was undertaken incorporating into the geotechnical cartography a more objectified blast reconciliation (the Stacey blast reconciliation system) to provide feedback information on blast performance to the mining team.

Instrumentation: Implementation of, knowledge gained from and improvements made to inclinometers with the installation of real time monitoring sensors and “web based” data recollection.

Topographical: The year on year improvements in topographical monitoring, from initial marker point GPS monitoring, to the installation of prisms and robotic total stations, to the final implemented use of direct slope laser scanning implemented for mineral extraction in difficult or confined access areas of the mining pit.

These last eight years have enabled the author to observe several geotechnical failure variants comprising, toppling, block failure, planar failure and traditional circular failures. ((1) Goodman. et al.) Observations of these events and their management are recorded within this thesis as well as the advancements in knowledge gained from these experiences.

At the very basis of this thesis is the concept of formalising the above developments, led by the author at an operative open pit mine in potentially problematic soils and rock. The ideal progression would be that such improvements could be, as required, incorporated at earlier stages rather than later stages in the next generation of pits or large excavations, or at very least undertaken in a programmed or planned manner taking into account these lessons learnt at a previous geotechnically successful large open pit excavation.

2 ANTECEDENTS

2.1 FORMATION OF LAS CRUCES VMS

Volcanogenic massive sulphides are deposited as submarine hot spring exhalative metal sediment, typically in tectonic areas of active submarine volcanism, including rift spreading centres and island arc subduction zones. In these geologic environments the volcanic rocks may be interlayered to a variable degree with contemporaneous volcanic sediments. Massive sulphide bodies are deposited as metallic sulphide sediments, with a variable component of volcanic sediment and silica, in conformable layers blanketing the area surrounding the exhalative centre. Massive sulphide layers consist predominantly of pyrite or pyrrhotite, with metal sulphides such as chalcopyrite, sphalerite, and galena. Subsequent tectonics can cause uplift, folding, and faulting of the original massive sulphide deposit.

The massive sulphide at Las Cruces was subjected to tectonically induced forces following deposition in the Variscan Orogeny which resulted in regional uplift, rock mass deformation and faulting. An erosion process commenced up until the Miocene Epoch resulting in atmospheric exposure of the massive sulphide and the formation of a gold and silver containing gossan zone. Copper was viably transported to the lower depths, via a process of precipitation replacing the un-oxidized primary massive sulphide at depth, forming a secondary enrichment of the sulphide deposit.

During the Miocene age the ore body was buried under 150 metres of Miocene age sediments. The Iberian Peninsula is largely underlain by a Hercynian belt of approximately 750 km in length, extending in a NW-SE direction. The Hercynian belt is formed of discrete zones or terranes, placed during the Pan-African/Cadomian and Hercynian Orogenies. According to (2) Leistel et al., (1998). these zones can be segregated into:

- Cantabrian Zone,
- West Asturian-Leonese Zone,
- Galicia Tras-os-Montes Zone,
- Central Iberian Zone,
- Badajoz-Cordoba Shear Zone,
- Ossa-Morena Zone,
- Pulo do Lobo,
- Southern Portuguese Zone
- Porto Tomar Shear Zone.

The Badajoz-Cordoba Shear Zone formed during the Pan-African and Hercynian Orogenies (3) (Quesada, 1991). The CIZ and accreted OMZ underwent a passive margin type evolution in the northern margin of Gondwana until the onset of the Hercynian orogeny in early to mid-Devonian (Leistel et al., 1998). The Pulo do Lobo Zone is a complex ophiolite sequence formed as a result of the subduction of oceanic lithosphere at the outer margin and underneath the OMZ (Leistel et al., 1998).

Collision between the Ossa-Morena Zone and the Southern Portuguese Zone coincided with bimodal magmatism, hydrothermal circulation and ore deposition (Leistel et al., 1998; (4) Oliveira, 1990 and (5) Quesada et al., 1991), and it is these

units that represent today's Iberian Pyrite Belt as shows **Figure 2.1.a**. During the Hercynian plate convergence an extensional tectonic environmental setting developed, which resulted in deformation and accretion of the South Portuguese terrane to the Iberian Palaeozoic continental block (Leistel et al., 1998).

Figure 2.1.a.- The Iberian Pyrite Belt (from Quesada 1991).

The map shows the locations of the main Volcanogenic Massive Sulphide (VMS) deposits, including the Las Cruces deposit, positioned toward the southeast of the region under the post Palaeozoic cover. (Modified from Quesada, 1991)

Photo 2.1.b shows a typical crystallised mineralisation of Calcosine at Las Cruces.

Photo 2.1.b.- Photo Crystallised mineralisation at Las Cruces.

2.2 CHARACTERISATION OF THE MIOCENE MARLS

The Guadalquivir “Blue marls” formation was deposited in the Guadalquivir basin in stable marine waters during the middle to upper Miocene without any significant eustatic changes (6) (Perconig 1962). These sediments were over consolidated as a result of greater depths of overlying sediment in the past. It is a fine grained clay sized material with massive and compact texture varying in strength with depth from firm to extremely stiff. It is blue and grey coloured and brown orange coloured in weathered zones.

These marls have been thoroughly studied in the past from a geo-mechanical point of view ((7) Ayala, 1978; (8) Tsige, 1999 or (9) Oteo, 2000).

In particular the marls at Las Cruces site were subjected to a detailed characterisation. The result of this work was synthesized in (10) Galera et. al., 2009 a.

2.2.1 CHEMICAL COMPOSITION

The first types of tests carried out during laboratory testing scheduling were related to petrology and mineralogy of the marls. Mineralogical composition was determined using X-ray diffraction method. For this purpose 44 RX Diffractions and 154 carbonate content were undertaken. These relationships correlated with depth are shown in **Figure 2.2.1.a**:

Figure 2.2.1.a.- Mineral composition with depth.

The main clay mineral is illite, from 20 to 40%. With depth Kaolinite decreases to negligible levels around 100 metres while conversely the content of smectite and Calcite increases notably after 100 m depth. The carbonate content shows a high scatter near the surface due to weathering becoming more consistent with depth ranging between 15 and 30 % down to 100 m. At that depth the carbonate content increases to an average value of 30%. It is likely that this carbonate content provides some form of cementation leading to the requirement of the mining department to utilise explosive to excavate the marl.

These relationships correlated with depth are shown in **Figure 2.2.1.b.**

Figure 2.2.1.b.- Content of Carbonates with depth

2.2.2 PETRO-PHYSICAL PARAMETERS

The microfabric of the marls and its engineering geological significance was presented by (11) Tsige et al (1995). **Photo 2.2.2.a** shows a microscopic photography of a sample undertaken in the mine.

The following lab tests and determinations were undertaken to determine the petro-physical parameters of the marl soil unit:

- 352 dry density.
- 115 specific weight of solid particles.
- 359 humidity.
- 221 Plasticity index.

Photo 2.2.2.a.- Micrographs undertaken at the mine showing an abundance of clay sized particles and coccoliths.

Their results can be seen in Galera et al (2009 a).

Moisture content is lower than the humidity of the plastic limit, corresponding to very stiff highly consolidated soils. A summary of the parameters with depth is shown in **Table 2.2.2.I:**

SECTION	DEPTH (m)	SPECIFIC WEIGHT OF SOLID PARTICLES (t/m ³)	DRY DENSITY (t/m ³)	MOISTURE CONTENT (%)	PLASTICITY INDEX (%)
MET-1	0-10	2,714	1,415	30,3	34,3
MET-2	10-23	2,714	1,459	28,5	30,2
MET-3	23-31	2,714	1,496	27,1	30,8
LEVEL-1	31-80	2,714	1,528	25,5	38,1
LEVEL-2	80-110	2,714	1,585	24,2	39,1
LEVEL-3	110-115	2,714	1,579	24,2	38,5
LEVEL-4	115-130	2,714	1,620	23,1	37,0
SANDY MARLS	130-140	2,714	1,622	25,7	35,3

**Table 2.2.2.I.- Summary table of petro-physical parameters with depth.
Galera et al (2009 a)**

Casagrande plasticity index indicates that the marls are classified as highly plastic clay/silt, as shown in **Figure 2.2.2.b**.

Figure 2.2.2.b.- Plasticity Chart. Galera et al (2009 a).

2.2.3 GEOTECHNICAL PROPERTIES

The marls constitute a more challenging unit from a geotechnical point of view as they have low strength and poor deformational parameters. This has strongly influenced the original pit geometry, dictating an overall low slope angle of 28° from the surface down to 150 m depth.

From the initial and subsequent investigations for the pit's geotechnical design the marls can be described as highly plastic (plastic and liquid limits of 35% and 65% respectively). They are of relatively low density (1.67 g/cm³ dry density) and comprise a high proportion of clay sized platy particles (98% passing a 75µm sieve).

The mine's miocenic marl unit is characterised by a very low permeability (with k values of 10⁻⁹ to 10⁻¹⁰ m/s). This means that without considering other drainage mechanisms, such as hydro-mechanical coupling, negligible pore pressure drop will be predicted post excavation in the modelling process. Between the marls and the ore there is the sandy regional aquifer 'Niebla-Posadas' which at the mine is protected near the mine by a system of drainage of water and reinjection at an appropriate distance from the mine.

Table 2.2.3.I shows the main geotechnical values of each horizon of the detailed stratigraphy of the Tertiary sequence that overlies the ore, as well as the change in UCS with depth.

Description	Horizon	Depth (m)	σ_{ci} (kp/cm ²)	m	s	c (kp/cm ²)	Φ (°)	γ_p (t/m ³)	γ (t/m ³)	ω (%)	PI (%)
Highly weathered marls	MET-1	0-10	3.5	2	1	1.10	22	2.714	1.415	30.3	34.3
Moderately weathered marls	MET-2	10-23	3.8	4	1	1.50	21	2.714	1.459	28.5	30.2
Slightly weathered marls	MET-3	23-31	3.8	4	0.07	1.50	21	2.714	1.496	27.1	30.8
Unweathered marls (low strength)	LEVEL-1	31-80	4.0	6	0.05	2.1	20	2.714	1.528	25.5	38.1
Unweathered marls (medium strength)	LEVEL-2	80-110	4.0	6	0.05	2.7	18	2.714	1.585	24.2	39.1
Unweathered marls (high strength)	LEVEL-4	115-130	6.0	6	0.01	2.8	18	2.714	1.579	24.2	38.5

Table 2.2.3.I.- Geo-mechanical values for each geotechnical horizon – Tertiary.

2.3 GEO-MECHANICAL CHARACTERISATION OF PALAEOZOIC HOST ROCK AND MINERALISATION

The mineralisation is hosted by rocks typical of the Palaeozoic within the Iberian Pyrite Belt. For the purposes of geotechnical evaluation, the following lithologies were established: gossan, tuffs, slates, and sulphides. Geologically, these lithologies can be further sub-categorized with respect to their geochemistry.

Group	Intact rock					Joints					
	ρ (kN/m ³)	Elastic		Plastic		Strike		Peak		Residual	
		Ei (MPa)	ν	σ_{ci} (MPa)	mi	Dip (°)	Dir (°)	Jc (kPa)	J ϕ (°)	Jc (kPa)	J ϕ (°)
GOSSAN	28.90	20 000	0.25	19.3	7.23	-	-	-	-	-	-
TUFFS	23.00	8 200	0.25	16.0	8.24	-	-	-	-	-	-
SHALES	24.00	22 500	0.20	4.4	4.78	70	004	130	22	0	22
SULPHIDES	44.00	80 000	0.20	115.0	12.85	-	-	-	-	-	-

Table 2.3.I.- Strength and deformability values for each lithology.

As can be discerned from the table, the only lithology with apparent continuous formative structures are the shales. This sequence is very well known locally, and holds the self-descriptive name in the pyrite belt as “soapy shales” giving an excellent indication of the joint-to-joint resistive qualities. The other Palaeozoic rock comprise relatively competent volcanic with few notable continuous faults.

Nonetheless, two major faults do run through the ore body, denominated:

- F1 - fault 1, tensional, bifurcating the ore body west to east.
- F2 - fault 2, compressional, represents the contact with the over-thrust hanging wall to the ore body.

A number of minor complementary faults were interpreted (initially 7 in total), predominately introduced to explain the presence of localised enrichment of the ore body. As of the time of publication of this thesis, most of these faults have been difficult to discern visually in the field with one of those faults being reunited with fault F1 in the phase 4 pit area.

2.4 MINING LEGISLATION

The determination of what constitutes a reasonable safety factor in mining is set forth in the I.T.C instructions section ITC 07.1.03. (12) (*Desarrollo de las labores*). The majority of what is considered as constituting good design has its foundations set out in this document specifically:

La altura máxima del frente de trabajo será de 20 metros. En casos especiales, la Autoridad minera podrá aprobar alturas superiores, que nunca excederán de los 30 metros, siempre que se realice un estudio geotécnico en el que al tener en cuenta las fuerzas resistentes y desestabilizadoras que actúan en el talud, resulte de la relación de ambas un coeficiente de seguridad de 1,2 o de 1,1 en el caso de que se haya considerado también el riesgo sísmico.

Paraphrasing, these guidelines therefore permit pit benches up to 20 metres with factors of safety of 1.2, or 1.1 under seismic risk consideration.

From an external perspective, the wisdom of extrapolating these safety factors for global pit or dump instability (the usual practice in Spain) could be questioned. However these safety factors are backed by years of successful implementation in Spain, do generally tie in well with international standards and are reasonable when considering the usually temporary nature of earthwork operations in mining when compared to those of civil engineering projects, where safety factors as high as 1.5 may be considered for similar earthworks with exponentially beneficial effects on subsequent stability. Later in this thesis, an analysis is undertaken on one particular slope remediation to determine the effect of lower safety factors on probabilities of further slope instabilities.

2.5 MINING OPERATIONS

At Las Cruces mine, a pre-stripping operation is undertaken removing nearly 150 metres of Miocene marls to access the considerable deposits of secondary enriched poly-metallic mineral at those depths.

This stripping operation can be divided into two methods, free dig, that is to say excavation without the requirement for blasting, and blast assisted excavation, with ANFO blasting utilised to loosen and fracture the marls in preparation for excavation using large scale diggers. The characteristics of the marl indicate that avoiding the blasting step may have benefits on the overall slope stability. It is noted, however, that as the marl dries it tends to crack and break up into an unconsolidated mass with little strength and that it is rather this drying effect on the marls that appears to be a prevalent factor for the observable small scale instabilities on the open-pit bench faces.

Las Cruces began operations as predominately a summer season mine, with a ramp up in production beginning in May through to December. January through to April were dedicated to housekeeping, fine tuning and supplying mine to plant blended stockpiles and repair of damage induced by the heavy rainfalls on the susceptible marls of the mining pit.

In 2011 operational modes were modified to extend the mineral extraction season, with the incorporation of stone to the principal haul road. This modification provided rapid access to the hard rock mineralised zones following rains, but also presented new difficulties in vigilance of the marl pit slope walls directly after those rains. Nonetheless, with a number of additional safety procedures implemented this modification has been proved effective and safe, enabling the mining division to more easily meet the ramp up requirements of the mineral processing plant.

3 METHODOLOGY FOR NEW PIT. CHARACTERISATION WORKS

3.1 PREVIOUS GEOTECHNICAL CAMPAIGNS AND SAMPLING

Geotechnical investigations were undertaken at the mine since the early geological campaigns and initially comprised eight geotechnical drill-holes undertaken to further detail the geo-mechanical and geochemical characteristics, predominately of the marls. At that early time in the mines development (nearly twenty years ago, whilst the mine was still a green field site), the majority of focus was placed on the mineralisation with less attention given to the geotechnical aspects of either the ore body or the surrounding host rocks which were initially considered “hard” rock.

Those early campaigns, from the late 1990s through to around 2005 were distinguished by a large amount of geotechnical testing on the overlying marls, with relatively little testing undertaken on the Palaeozoic. This provides an indication of where the concerns lay with respect to slope stability, with actual strength test on the Palaeozoic comprising a rather misleading single hammer blow estimation of over 150MPa.

The lack of representation of this single hammer blow test was clearly highlighted during exposure of the Palaeozoic host rock in 2010, revealing highly altered (or alterable) footwall shale on the south side of the mine’s phase 2 slopes.

3.2 NEW PIT CHARACTERISATION WORK

The exposure of these shales, combined with a higher than expected difference between the geological block model and recovered mineral, provoked the instigation of the first of a series of infill drilling campaigns (and associated analyses described later in this text). It was during the first of these campaigns that the geotechnical division at the mine implemented a robust series of geotechnical testings on the geological drillholes as well as undertaking a number of dedicated geotechnical drillholes for additional testing including downhole surveys to better determine rock structure orientations. ((13). Zemanek,J et al).

In 2012 a summary of this work was compiled in an internal report denominated “Pit Optimisation for CLC” which included subsequent updates on the previous limit state slope stability analysis. In the **Table 3.2.1** the dedicated geotechnical drillholes are highlighted. These were additional holes drilled during the geology department’s infill drilling campaign specifically to obtain an improved understanding of the pit geotechnical regime and the geotechnical characterisation of the pit’s various encountered lithological units.

Also shown in **Figure 3.2.a** is an example of one of the downhole geophysical inspections undertaken (acoustic televiewer).

NOMBRE	UBICACIÓN	INCLINACIÓN	INICIO	FIN	DIÁMETRO	PROF. ESTIMADA (m)	PROF. REAL (m)	COORDENADAS UTM		
								x	y	z
SGT-1	Fondo de corta	60/227	16/04/2012	18/04/2012	PQ	75	75	756995.43	4154615.02	-139.31
SGT-2	Fondo de corta	60/227	18/04/2012	20/04/2012	PQ	75	59,05	756998.16	4154564.27	-140.57
SGT-3	Fondo de corta	Vertical	30/04/2012	03/05/2012	HQ	40	55,15	757148.18	4154531.61	-145.85
SGT-4	Fondo de corta	Vertical	26/04/2012	30/04/2012	HQ	70	75.5	757008.89	4154735.48	-140.23
SGT-5	Fondo de corta	Vertical	23/04/2012	26/04/2012	HQ	65	70.1	757045.72	4154750.27	-139.97
SGT-6	Fondo de corta	Vertical	20/05/2012	22/05/2012	HQ	40	40.1	757042.88	4154541.22	-145.00
SGT-7	Fondo de corta	Vertical	17/05/2012	18/05/2012	HQ	30	45	757101.73	4154536.27	-145.00
SGT-8	Fase 3 en margas	Vertical	09/05/2012	14/05/2012	HQ	150	132	757242.29	4154962.47	-44.88
SGT-9	Fondo de corta	Vertical	04/05/2012	08/05/2012	HQ	50	72.75	757044.91	4154645.59	-145.00
SGT-10	Fase 3 en margas	Vertical	11/05/2012	16/05/2012	PQ	150	141	757493.79	4154631.65	-38.91
IN-34	Fase 3 en margas	20/270	-	-	HQ	-	108.55	757437.00	4154548.00	-35.00
IN-84	Fase 3 en margas	20/270	-	-	HQ	-	110	757359.00	4154518.00	-35.00
IN-14	Perímetro corta sector Noreste	Vertical	22/05/2012	26/05/2012	HQ	200	150	757517.90	4155302.56	29.00

Table 3.2.I.- Dedicated Geotechnical Drillholes undertaken during the 2012 geological infill drilling campaign.

Figure 3.2.a.- Geo-physical work undertaken in one drillhole (SGT-3) to determine shale bedding and plane orientations.

3.2.1 MARL LABORATORY TESTS

The laboratory tests carried out initially were done at stress levels that were much lower than the insitu stress state, and the direct shear tests were carried out at a high strain rate. On site all of the test data was directly reviewed and data assessed in detail to develop what was believed to be an appropriate conservative interpretation of the strengths from the available test data. In general, these interpreted strengths are lower than those derived from a statistical approach. A potential strength check was also developed to confirm the undrained strength assessment based on the over consolidation ratio (OCR), which also was calibrated to tri-axial testing and direct shear data.

Lower Strength Bedding Layers and Higher Cross Bedding Strengths

The marls were deposited in a marine environment which resulted in layers of varying plasticity and strength. Typically, the lower strength bedding layers occur every 6 or 7 metres and can be observed in the open pit and in the inclinometer readings. Movements typically occur along the lower strength bedding layer(s) and this anisotropy needs to be incorporated into the stability analyses. The cross bedding strength, however is an average of each of the layers and, as a result is higher than the bedding strength.

Stress-Strain Testing

The marl is a strain softening material and the friction angles can reduce from values of 20° to a residual friction angle of around 12° with as little as a few centimetres of movement. This is an important consideration in selection of the design factor of safety as the factor is variable along the failure plane due to the variations in the stress state.

Pore Pressures Testing

The marls have a very low permeability ((14). Luque. J.A) of around (10^{-10} m/s), which results in two response conditions:

When loaded, the pore pressures increase, ((15) Skempton et al.) with a response parameter Bbar in the order of 0.6 to 0.8 of the change in stress (load). This increases the pore pressure and reduces the stability for the loading conditions.

When unloaded, as in the open pit, the pore pressures decrease due the load reduction and negative pore pressures develop, which increases the stability. Pore pressure response in the fills is less than in the insitu marls due to the increased presence of voids post excavation. In comparison, compacted marls in the marl dumps usually have a pore pressure response of 0.3.

The value of permeability was derived both from laboratory testing and also from 12 large scaled permeability tests monitoring the water head in 55 piezometers during long term pump tests. The sensors were located in the marls, aquifer and fractured top zones of the Palaeozoic.

3.2.2 PALAEOZOIC ROCK PARAMETERS

Resistance Parameter Testing

On the Palaeozoic, resistance testing was undertaken in order to obtain a significant data set for the geotechnical resistance parameters.

Thorough laboratory testing was undertaken during the 2012 campaign comprising:

- 46no. Apparent density determination.
- 21no. Uniaxial Compression Strength Determination.
- 15no. Compression Strength with Band Determination.
- 23no. 19 Tri-axial Tests.
- 7no. Shear box test on discontinuities.
- 6no. Slake durability tests (SDT).
- 1no. Additional dry density determination.

The **Table 3.2.2.I** show a summary of all results from the 2012 testing regime. Average parameters for all samples irrespective of lithology is included. As detailed later in this study, subsequently determination of parameters for each lithological unit was undertaken.

ENSAYOS DE LABORATORIO																					
Prospección	Muestra	Profundidad (m)			Litología	H %	γ_{mo} (g/cm ³)	γ_d (g/cm ³)	UCS roca				Brasileño σ_1 (MPa)	Tenacidad σ_1/σ_c	Triaxial roca		Corte en discontinuidades			Stake Durability Index	
		Inicial	Final	Media (m)					σ_{ci} (MPa)	EI (MPa)	v	E/σ_{ci}			σ_1 (MPa)	σ_3 (MPa)	C (MPa)	Fricción (°)	JRC	Id ₁	Id ₂
IN-34	M-1	80,29	80,74	80,5	Gossan		2,04		8,40				1,21	0,14							
	M-2	83,41	84,3	83,9	Gossan		2,18		9,40	2739,20	0,22	291,40			21,80	2,00					
	M-3	94,61	95,05	94,8	Tobas		2,63		2,20				1,24	0,56							
	M-4	79,67	79,99	79,8	Gossan		2,13		14,50				2,34	0,16							
	M-5	86,09	86,45	86,3	Gossan		2,13		9,70	2966,30	0,13	307,67									
	M-6	192,43	192,89	192,7	Gossan		2,11		6,40				1,02	0,16	34,50	5,00					
IN-84	M-1	76,68	77,16	76,9	Gossan		2,19		9,80				1,70	0,17							
	M-2	78,85	79,19	79,0	Gossan		2,13								37,10	5,00					
	M-3	82,05	82,51	82,3	Gossan		2,20		9,80	1488,50	0,37	151,89									
	M-4	83,9	84,31	84,1	Gossan		2,25								22,40	3,00					
SGT-1	M-1	43,33	43,7	43,5	Sulfuros		4,46		69,90				9,15								
	M-3	67	67,4	67,2	Piz. Duras		2,16		6,10				0,89	0,15							
	M-4	68,68	69,25	69,0	Piz. Blandas		2,50		5,00								0,14	30,81	(6-8; 12-14; 12-14)		
	M-4 BIS	68,3	68,68	68,5	Piz. Blandas								1,15								
SGT-2	M-1	17,47	18,25	17,9	Sulfuros		4,58		32,80	102026,10	0,19	3110,55			43,50	5,00					
	M-3	42,2	42,48	42,3	Piz. Duras		2,64		79,80	81372,90	0,14	1019,71									
	M-4	57,8	58,05	57,9	Piz. Blandas		2,39		2,70	10146,40	0,17	3757,93									
	M-5	50,81	51,5	51,2	Piz. Blandas		2,14		1,80	26349,20	0,28	15749,56			5,80	1,00					
	M-6	55,25	55,45	55,4	Piz. Blandas												0,15	17,39	(6-8; 8-10; 14-16)		
SGT-3	M-1	20,3	20,72	20,5	Tobas	4,44	2,47	2,37	0,96	148,10		154,59	0,22		7,40	2,00					
	M-2	23,22	23,65	23,4	Tobas		2,88		7,00	1743,90	0,29	249,13			25,90	5,00					
SGT-4	M-1	71,3	71,63	71,5	Piz. Blandas		2,45		1,90				1,19	0,63						67,65	48,62
	M-2	73,48	73,86	73,7	Piz. Blandas		2,58		3,30				0,46	0,14							
	M-3	67,41	67,68	67,5	Piz. Blandas										25,20	3,00				93,99	89,02
	M-4	71,9	72,15	72,0	Piz. Blandas		2,25										0,18	44,48	(8-10; 8-10; 8-10)		
SGT-5	M-1	63	63,55	63,3	Piz. Blandas		2,38		4,00				0,98	0,25	15,10	2,00					
	M-2	68,7	68,98	68,8	Piz. Blandas												0,10	32,37	6-8		
	M-3	69,85	70,1	70,0	Piz. Blandas		2,53		14,00	85129,11	0,16	6080,65									
SGT-6	M-1	8,6	9,01	8,8	Sulfuros		4,54		34,40				9,69	0,28							
	M-2	9,54	9,91	9,7	Sulfuros		4,69								37,60	1,00					
	M-3	14,5	15,05	14,8	Sulfuros		4,70		35,90	22036,70	0,20	613,84									
	M-4	24,66	24,87	24,8	Piz. Duras												0,60	32,87	(8-10; 10-12)		
	M-5	27,71	28,18	27,9	Piz. Duras		2,36		19,00	45491,10	0,19	2394,27			12,50	1,00					
	M-6	32,04	32,54	32,3	Piz. Blandas		2,17		2,50												
	M-7	37,2	37,77	37,5	Piz. Blandas		2,52								8,40	2,00				55,31	30,29
SGT-7	M-1	9,49	10	9,7	Sulfuros		4,10		111,10	108813,40	0,19	979,42			126,60	2,00					

ENSAYOS DE LABORATORIO																					
Prospección	Muestra	Profundidad (m)			Litología	H %	Y _{so} (g/cm³)	Y _d (g/cm³)	UCS roca				Brasileño σ _t (MPa)	Tenacidad σ _t /σ _c	Triaxial roca		Corte en discontinuidades			Slake Durability Index	
		Inicial	Final	Media (m)					σ _{ci} (MPa)	EI (MPa)	v	E/σ _{ci}			σ ₁ (MPa)	σ ₃ (MPa)	C (MPa)	Fricción (°)	JRC	Id ₁	Id ₂
	M-2	8,18	8,63	8,4	Sulfuros		4,42		89,30				18,78	0,21							
	M-3	13,8	14,22	14,0	Sulfuros		4,16								176,70	3,00					
	M-4	26,9	27,43	27,2	Piz. Blandas							0,60									
	M-5	35,95	36,23	36,1	Piz. Blandas																
	M-6	38,1	38,6	38,4	Piz. Duras		2,55		3,50			1,03	0,29								
	M-7	42,37	42,74	42,6	Piz. Duras		2,58								16,10	3,00					
SGT-8	M-1	71,8	72,2	72,0	Piz. Duras												0,03	43,42	4-6		
	M-2	76,7	77,35	77,0	Piz. Duras																
	M-2 BIS	75,5	75,93	75,7	Piz. Duras		2,69							26,30	5,00						
	M-3	96,3	96,68	96,5	Riolitas		2,91		14,80			10,71	0,72							99,36	99,06
	M-4	108,5	108,9	108,7	Riolitas		2,76							57,20	5,00					98,80	98,28
	M-6	119,03	119,4	119,2	Riolitas		2,80		63,30			8,10	0,13								
	M-8	74,85	75,04	74,9	Piz. Duras											0,13	36,92	8-10			
	M-9	77,35	77,7	77,5	Piz. Duras																
SGT-9	M-9 BIS	78,08	78,47	78,3	Piz. Duras							2,05									
	M-1	53,03	53,41	53,2	Sulfuros		4,45		66,80			16,19	0,24								
	M-2 BIS	29,55	29,88	29,7	Sulfuros																
SGT-10	M-2 BIS	35,2	35,48	35,3	Sulfuros		4,62		52,10				10,43	0,20							
	M-1	74,01	74,5	74,3	Gossan		2,19		12,30				2,65	0,22							
	M-2	82,56	82,9	82,7	Tobas		2,33		2,10				0,41	0,20						77,65	63,36
	M-3	97,35	97,72	97,5	Tobas		2,35		3,30	6212,70	0,20	1882,64									
	M-4	106,8	106,17	106,5	Tobas		2,26								20,40	3,00					
ESTADÍSTICAS	M-5	117,5	117,93	117,7	Tobas		2,71		20,60	24448,30	0,14	1186,81									
	NÚMERO DE MUESTRAS					1	46	1	36	15	14	15	23	18	19	19	7	7	7	6	6
	VALOR MÁXIMO					4,44	4,70	2,37	111,10	108813,40	0,37	15749,56	18,78	0,72	176,70	5,00	0,60	44,48	0,00	99,36	99,06
	VALOR MÍNIMO					4,44	2,04	2,37	0,96	148,10	0,13	151,89	0,22	0,13	5,80	1,00	0,03	17,39	0,00	55,31	30,29
	MEDIANA					4,44	2,51	2,37		22036,70	0,19	1019,71	1,24	0,21	25,20	3,00	0,14	32,87		85,82	76,19
	MEDIA					4,44	2,85	2,37	23,07	34875,46	0,21	2528,68	4,44	0,27	37,92	3,05	0,19	34,04		82,13	71,44
	DESVIACIÓN ESTANDAR						0,89		28,98	39635,33	0,07	4020,10	5,47	0,18	42,99	1,51	0,19	9,10		18,25	28,54
COEFICIENTE DE VARIACIÓN (%)						31,32		125,61	113,65	32,68	158,98	123,04	66,15	113,37	49,41	99,24	26,74		22,22	39,95	

Table 3.2.2.I.- Paleozoic Rock Test.

4 REFINING THE PIT DESIGN

As mentioned in section 3.2 above, the exposure of shales on the south wall of the phase 2 pit led to concerns regarding the possibility of future exposure of similar rock types and their possible effects on pit stability. By this point, the pit development had allowed exposure of many of the anticipated rock types in slope, allowing a specific unification of the rock characteristics with their real in-slope performance.

At around the same time the mining leadership had decided to investigate the robustness of the geological/geochemical block model and tabled the question as to whether it would be advantageous to include additional geotechnical drilling and testing in the same geological infill drilling campaign. Thus, an opportunity arose to do so; to improve further geotechnical knowledge and enclose this information in a new pit optimisation representing the compilation of all knowledge gained to date as well as data from new drillholes undertaken between 2011 and 2012.

A detailed report of this work was compiled in a project report denominated "Pit Optimisation for CLC". A well-described history of all the works carried out for the pit design refining can be found at (16) Cooper et al (2015). To follow is an exhaustive examination of the analytical works undertaken as well as the obtained results.

4.1 ANALYSIS AND SELECTION OF GEOTECHNICAL PARAMETERS

In order to obtain representative geotechnical parameters for the marls and underlying Paleozoic, an exhaustive recompilation of historic information was processed comprising work independently exercised by areas of hydro-geology, geology and the onsite geotechnical division.

4.1.1 INITIAL PARAMETER ANALYSIS AND SELECTION

The materials were characterised geo-mechanically, first separating out the soils and the rocks, given that they would subsequently require distinct analysis methodologies.

For the characterisation of the soils (marls), the data collection included the information from different retrospective analysis undertaken, in accordance with concession conditions, at depths of 60, 90, 120 and 150 metres depth as well as the onsite experience acquired during the first 6 years of excavations.

For the characterisation of the different rock based lithotypes, use was made of the entire laboratory testing undertaken during the 2012 campaign referred to in the previous section.

4.1.2 GEOTECHNICAL CHARACTERIZATION OF THE TERTIARY MARL UNIT

As discussed in the earlier, there was a significant amount of laboratory and in-situ test information compiled and available. Additionally observations during the

development of the pit stripping and blasting activities were utilised to improve significantly the marl unit characterization.

Sub-segregation of the Marl Unit.

Although the marl unit initially appears homogenous visually, a detailed analysis permits the differentiation of determined levels. These levels are confirmed by operational details such as the switch to pre-blasting at around 70 to 80 metres depth.

Visually the marls seem to be homogeneous; nonetheless, detailed analysis allows the distinction with depth sections of different behaviours in consideration of geo-mechanical characteristics. The following stratification can certainly be discerned:

- **WEATHERED MARLS:** at surface highly altered by weathering and designated MET. Initially two levels were distinguished but finally three sections were established during the stripping operation:
 - MET-1, highly weathered. Brownish coloured. It is 10 metres thick.
 - MET-2, down to 23 metres depth. The marl is heavily weathered and presents vertical desiccation fractures, spaced around each metre.
 - MET-3, down to 31 m depth. The marl is moderately weathered. The spacing of desiccation discontinuities is around 12 metres. The strength parameters at this level are similar to MET-2 level.
- **FRESH MARLS:** After 31 metres depth there are no visible signs of weathering with the marls now showing typical grey bluish colour. The following four levels can be distinguished:
 - LEVEL1, “very weak marl”, from 31 to 80 metres depth. There are no desiccation fractures but several sub-vertical joint sets can be observed as well as horizontal bedding planes spaced at 5m intervals.
 - LEVEL2, “weak marl” from 80 to 110 metres. Its strength is characteristic of a weak rock type 0 to 0-1, showing fragile failures.
 - LEVEL3, “weak marl”, these presents the same characteristics of level 1 with a thickness ranging from 5 to 10 metres. Laterally it disappears between levels 2 and 4 showing strength and deformability properties similar to level above.
 - LEVEL4, “strong marl”, these have a strength characteristic of the weak rock type 0-1. There is an appreciable increase of the strength and stiffness of the marl.
 - SANDY MARL, just above the sands of the “Niebla-Posadas” aquifer there is a massive layer of sandy marls with an approximate thickness of 5 metres.
 - SANDS, partially cemented with thickness ranging from 0 to 15 metres.

The profile is shown more concisely in the **Figure 4.1.2.a**.

Figure 4.1.2.a.- Sequence defined in the marls.

4.1.2.1 MARL STRENGTH DETERMINATION WITH DEPTH

Reviewing the resistance in uniaxial compressive testing with depth in marls, certain aspects can be highlighted. Firstly, follow up testing, extracting samples from the excavation during stripping revealed strength values higher than those obtained via drillholes. This was attributed to the disturbance factor present even in samples obtained and denominated “undisturbed”. In reality, in a relatively brittle material such as the marls, true undisturbed sampling is difficult and well-preserved bulk samples appeared to prove to be the easiest form to achieve something relatively unaffected by the mining stripping decompression and stripping activities.

It was precisely the use of these less disturbed marls strength values that enabled improvements in design angles to the pit referred to later in this thesis.

In the **Figure 4.1.2.1.a** the UCS value is contrasted with depth and a number of observations can be made.

Figure 4.1.2.1.a.- Uniaxial Compressive strength with depth.

A summary of each sub-segregation level of marls is given in the **Table 4.1.2.1.I** and subsequent descriptions below:

LEVEL	P (kN/m ³)	c (kPa)	ϕ (°)
MET-1	14,0	110	22
MET-2	19,7	150	21
MET-3	19,5	150	21
LEVEL -1	19,2	210	20
LEVEL -2	19,2	270	18
LEVEL -3	19,2	270	18
LEVEL -4	20,2	280	18
Sandy Marls	20,1	85	31
Sands	20,3	20	37
Paleozoic	21,2	500	35

Tabla 4.1.2.1.I.- Summary of each sub-segregation level of marls.

Observations:

- MET 1 to 3: From 0 metres to 31 metres, an increase in resistance with depth, although there is a significant dispersal in the strength results.
- LEVEL 1: From 31 metres to 80 metres depth the correlation of increased strength with depth is less notable, perhaps suggestive of an increased over-consolidation ratio. Strength results continued to be variable between samples.
- LEVEL 2 to 4: From 80 metres to 140 metres there is a clear increase of uniaxial compressive strength with depth.
- LEVEL 3: Between 115 metres to 130 metres there is a considerable dispersion of values with depth, and the data suggests behaviour consistent with that of a weak rock.

It is also worth highlighting a relation between uniaxial compressive strength and the content of carbonates. It is likely that this effect is due to minor cementing effects of these carbonates.

4.1.2.2 STRAIN SOFTENING BEHAVIOUR

The Guadalquivir marls are well known to exhibit strain-softening behaviour. The various peak and residual resistance parameters were separated and determined in stress-strain test for the various sub-segregated marl units as shown in **Figure 4.1.2.2.a**.

Figure 4.1.2.2.a.- Peak to residual behaviour with strain for each marl sub-segregation.

SECTION	DEPTH (m)	σ_{CI} (kp/cm ²)	m	s	c (kp/cm ²)	Φ (°)	R^2 ($\sigma_1 - \sigma_3$)	R^2 ($\sigma_N - \tau$)	N° UCS	N° TRIAXIAL C T	N° SHEAR T
MET-1	0-10	3,5	2	1	1,10	22	0,91	0,80	4	0	9
MET-2	10-23	3,8	4	1	1,50	21	0,21	0,73	7	5	18
MET-3	23-31	3,8	4	0,07	1,50	21	0,49	0,51	11	11	6
LEVEL-1	31-80	4,0	6	0,05	2,1	20	0,61	0,65	73	83	126
LEVEL-2	80-110	4,0	6	0,05	2,7	18	0,81	0,74	0	23	129
LEVEL-4	115-130	6,0	6	0,01	2,8	18	-	0,87	0	0	48

Tabla 4.1.2.2.I.- Summary of cohesion and friction angle resistance parameters.

The marls are notable for their high peak to residual resistance ratio. This means from a practical perspective that the marls exhibit a brittle behaviour and from a mine operations perspective, vigilance for excessive distortion, is a key factor to maintain pit slope stability.

4.1.2.3 EUROCODE 7 PARAMETER ANALYSIS FOR THE MARLS

For the selection of parameter values, statistical analysis was undertaken to obtain mean design values. In addition, a comparative analysis of the use of Eurocodes was undertaken. This was conducted for purely academic purposes on the marls overall mean values (without sub-segregation or specific reference to depth).

Eurocodes design philosophy permits a different perspective to the traditional mining and civils approach to slope stability. As such it was considered sensible to undertake a comparative analysis of the more sensitive and less competent marl parameters. As previously discussed mining law requirements (ITC's) determine specific global safety factors should be utilised in pit design (1.2 static and 1.1 with seismic considerations). In contrast, the Eurocode partial factor design philosophy allows safety factors to be placed on material characteristics and destabilising forces in line with the level of confidence attained following site investigation and are therefore case specific.

A comparative specific analysis was therefore undertaken to determine values appropriate for design work as shown the **Table 4.1.2.3.I.**

TRADITIONAL METHODS	COHESION (KPa)	ϕ (°)
Linear regression	46,87	27,61
Linear regression confidence 95 % about stress tangential	$X_{k,sup} = 53,93$ $X_{k,inf} = 39,24$	27,61
95 % confidence interval about estimators β_0 y β_1	$X_{k,sup} = 65,7$ $X_{k,inf} = 28,43$	$X_{k,sup} = 31,15$ $X_{k,inf} = 23,82$
Linear regression with a 95 % confidence about cohesion	$X_{k,sup} = 48,23$ $X_{k,inf} = 43,70$	$X_{k,sup} = 31,07$ $X_{k,inf} = 23,35$

Tabla 4.1.2.3.I.- Traditional statistical analysis to determine appropriate overall marl internal friction angles.

EUROCODE 7	COHESION (KPa)	ϕ (°)
Mean value of the sample parameters	44,40	27,16
Normal distribution 95 % confidence	$X_{k,sup} = 53,33$ $X_{k,inf} = 37,47$	$X_{k,sup} = 35,47$ $X_{k,inf} = 23,67$
Log-normal distribution 95 % confidence	$X_{k,sup} = 47,41$ $X_{k,inf} = 32,49$	$X_{k,sup} = 29,16$ $X_{k,inf} = 22,70$

Tabla 4.1.2.3.II.- Characteristic values obtained utilising the Eurocode 7 approach.

Comparison showed that there was virtually no significant difference in utilising either of the two techniques (16a) (Morán, 2015). A review of the results indicated that around a 4% decrease in cohesion being offset by resistance gains due to a slight (2%) increase in the internal friction angle resistance parameter.

It is notable that best fit lines on overall means indicate a high friction angle but lower cohesion. This may be due to subjectivity during the analysis of what exactly constitutes the best fit, or the elimination of depth consideration causing substantial scatter in the data patterns.

Initial considerations were that the Eurocode methodology could be transferred from a civil engineering environment to improve the statistical methodology utilised in mining, with the original intent being to continue towards implementation of these derived values in specific slope analysis to highlight this improvement. However upon review of the minor differences highlighted between the two techniques, and in the context of predominately structurally dominate stability characteristics; it was considered that further work was unwarranted in the area, at least at Las Cruces mine. Nonetheless, in mines where datasets are perhaps less abundant, the author believes it could be still represent a useful tool to enable determination of the effects of database size on geotechnical parameter variations, allowing an objective manner of providing justification for additional drilling for improved geotechnical characterisation.

4.1.3 GEOTECHNICAL CHARACTERIZATION OF THE HARD ROCK (PALAEOZOIC)

Within the great heterogeneity of the Palaeozoic rocks that are excavated in the pit, a grouping was carried out in six lithotypes: Gossan, Tobas, Soft Slates, Hard Slates, Riolites and Sulphides. These different groupings with the Palaeozoic can be clearly observed in the **Table 4.1.3.I.:**

Unit	Image	Description
Sandstone Aquifer		Includes arenites, calc-arenites, and conglomerates
Gossan		Bedrock that has been altered (silicified) and brecciated
Shale		'Weak' or 'strong' shales and slates, light to dark grey, moderately fractured to highly fractured
Sulphide		Sulphides include galena, chalcopyrite, sphalerite, covellite and pyrite
Tuff/ Rhyolite		'Tuffaceous' rhyolite is distinguished from other textures.

Table 4.1.3.I.- Rock groups distinguished in the Paleozoic

Within these large groups exist lithological variations and/or mixtures of several, nevertheless, the groupings have been realised according to geotechnical behaviour.

In each of the lithotypes sub-segregation of less competent altered (grade II) and an altered or associated level of fractures (grade III-IV) was differentiated.

These lithotypes constitute a conventional rocky mass, to which the usual methodology in these cases was applied, which consisted in estimating the intact rock properties of the lithotypes present, for the minimum secondary parameters and establishing the parameters characteristic of each lithotype.

Strength Properties of the Intact Rock

The first step for the mechanical characterization of a rock mass is the establishment of intact rock properties for each of the differentiated lithotypes. For this purpose, a series of laboratory tests were carried out to determine the mechanical and deformational properties of the various existing lithotypes.

Resistant parameters are determined from the laboratory tests performed, using single or uniaxial compression tests, indirect (Brazilian) tensile tests ((17) Krishnayya A.V.G 1974), and tri-axial compression tests.

From the laboratory tests performed, results can be represented in the plane $\sigma_3 - \sigma_1$, to obtain a point for each test. The different criteria of breakage determine a functional relationship between these points. The constitutive models applied in this were those of Mohr-Coulomb and Hoek-Brown, the latter in its last revision, published in the year 2002 (18) (Hoek et al, 2002).

The Mohr-Coulomb criterion assumes a linear relationship between σ_3 y σ_1

$$\sigma_1 = \lambda \cdot \sigma_3 + \sigma_c$$

Being the σ_c the uniaxial compression strength of the lithotype and λ a parameter related to the angle of friction according to the equation:

$$\lambda = \frac{1 + \text{sen}\phi}{1 - \text{sen}\phi} = \text{tg}^2\left(45 + \frac{\phi}{2}\right)$$

From σ_c and σ_3 the friction angle (Φ) and the cohesion of the lithotype (C) can be obtained by the expressions:

$$\phi_s = 2 \cdot \arctg \sqrt{\lambda_s} - 90$$

$$C = \sigma_c \frac{1 - \text{sen}\phi}{2 \cdot \cos\phi}$$

The Hoek-Brown criterion assumes a non-linear relationship between σ_1 y σ_3 in the form of:

$$\sigma_1' = \sigma_3' + \sigma_{ci} \cdot \left(m \cdot \frac{\sigma_3'}{\sigma_{ci}} + s \right)^a$$

Where m is a parameter that depends on the angle of friction, and s is a parameter that relates the resistance to simple compression of the intact rock (σ_{ci}) with the resistance to simple compression of the rock mass (σ_{cm}) by the expression.

$$\sigma_c^m = \sigma_{ci} \cdot s^a$$

The parameter "a" is a constant of the rock mass, given by:

$$a = \frac{1}{2} + \frac{1}{6} \cdot \left(e^{\frac{-GSI}{15}} - e^{\frac{-20}{3}} \right)$$

In the case of laboratory tests, s is equal to unity since it is intact rock, and if m is called m_i for that of intact rock the equation takes the form:

$$\sigma_1' = \sigma_3' + \sigma_{ci} \cdot \left(m_i \cdot \frac{\sigma_3'}{\sigma_{ci}} + 1 \right)^a$$

While a tends to be equal to 0,5.

Substituting the following variables:

$$x = \sigma_3'$$

$$y = (\sigma_1' - \sigma_3')^{1/a}$$

The following relationship is obtained for the intact rock:

$$y = m_i \cdot x \cdot \sigma_{ci}^{(1/a-1)} + \sigma_{ci}^{1/a}$$

Assuming a fixed σ_{ci} value, determined in this case by laboratory tests, the value of the parameter m_i can be obtained, which will be used later to evaluate the properties of the rock mass.

A linear regression of the values σ_1' and σ_3' obtained from the laboratory tests was performed to determine the values of the parameters of the plastification criterion, both the Mohr-Coulomb criterion and the Hoek-Brown criterion. Thus, for the Mohr-Coulomb criterion, the straight line of the above formula is adjusted, and for that of Hoek-Brown, in case of intact rock, it is expressed in the form previously mentioned. In general, these adjustments are very sensitive to the value of σ_{ci} and, normally, the value of σ_{ci} obtained in the laboratory should be similar to those adjusted for the two criteria of failure.

And, for the Hoek and Brown criteria:

$$m_e = \frac{1}{n} \sum \left[\frac{(\sigma_{1i}' - \sigma_{3i}')^{1/a} - \sigma_{ci}^{1/a}}{\left(\sigma_{3i}' \cdot \sigma_{ci}^{\left(\frac{1}{a} - 1 \right)} \right)} \right]$$

Where the subscript "e" indicates the adjusted parameters and the subscript "i" each of the points corresponding to an essay, where n is the total of points $\sigma_1' - \sigma_3'$ considered by lithotype.

The correlation coefficients are obtained from the known expression:

$$r^2 = \frac{\sum (Y_{est} - Y_{med})^2}{\sum (Y_{est} - Y_{med})^2}$$

Deformational parameters of the intact rock

The relative modulus, or relation between the modulus of elasticity E_i and the resistance to simple compression σ_{ci} , varies according to the lithology. The **Figures 4.1.3.a, b and c** show this relation for igneous, metamorphic and sedimentary rocks.

Figure 4.1.3.a.- Classification of intact rocks for igneous rocks (19)(modified from Deere and Miller, 1966)

Figure 4.1.3.b.- Classification of intact rocks for metamorphic rocks (modified by Deere and Miller, 1966)

Figure 4.1.3.c.- Classification of intact rocks for sedimentary rocks (modified by Deere and Miller, 1966)

More recently (20) Hoek and Drederich (2006) published the values for the relative modele MR, between E_i and σ_{ci} , based on the previous work of Deere and Miller (1966) and (21) Palmstrom and Singh (2001), as shown the **Table 4.1.3.II**.

Rock type	Class	Group	Texture			
			Coarse	Medium	Fine	Very fine
Sedimentary	Clastic		Conglomerates 300-400 Breccias 230-350	Sandstones 200-350	Siltstones 350-400 Greywackes 350	Claystones 200-300 Shales 150-250 ^a Marls 150-200
	Non-clastic	Carbonates	Crystalline limestones 400-600	Sparitic limestones 600-800	Micritic Limestones 800-1000	Dolomites 350-500
		Evaporites		Gypsum (350) ^b	Anhydrite (350) ^b	
		Organic				Chalk 1000+
Metamorphic	Non-foliated		Marble 700-1000	Hornfels 400-700 Metasandstone 200-300	Quartzites 300-450	
	Slightly foliated		Migmatite 350-400	Amphibolites 400-500	Gneiss 300-750 ^a	
	Foliated ^a			Schists 250-1100 ^a	Phyllites/Mica Schist 300-800 ^a	Slates 400-600 ^a
Igneous	Plutonic	Light	Granite ^c 300-550 Granodiorite ^c 400-450	Diorite ^c 300-350		
		Dark	Gabbro 400-500 Norite 350-400	Dolerite 300-400		
	Hypabyssal			Porphyries (400) ^b	Diabase 300-350	Peridotite 250-300
	Volcanic	Lava		Rhyolite 300-500 Andesite 300-500	Dacite 350-450 Basalt 250-450	
		Pyroclastic	Agglomerate 400-600	Volcanic breccia (500) ^b	Tuff 200-400	

^aHighly anisotropic rocks: the value of MR will be significantly different if normal strain and/or loading occurs parallel (high MR) or perpendicular (low MR) to a weakness plane. Uniaxial test loading direction should be equivalent to field application.

^bNo data available, estimated on the basis of geological logic.

^cFelsic Granitoids: coarse grained or altered (high MR), fined grained (low MR).

Table 4.1.3.II.- Values for relative module (MR) E_i / σ_{ci} (Deere 1966, Palmstrom and Singh 2001).

Best fit analysis (least squares) was undertaken to obtain required geotechnical parameters for the various lithologies. These are shown intact rock parameters are shown in **Figures 4.1.3.d, e, f, g, h, i**:

Gossan

PARÁMETROS DE LA ROCA INTACTA - GOSSAN					
γ_s (t/m ³)	σ_{ci} (MPa)	E_i (MPa)	Coefficiente de Poisson (ν)	σ_{ti} (MPa)	m_i
2,89	13,92	20.000	0,25	1,78	7,23

Figure 4.1.3.d.- Analysis done for gossan.

Volcanic Tuffs

PARÁMETROS DE LA ROCA INTACTA - TOBAS					
γ_s (t/m ³)	σ_{ci} (MPa)	E_i (MPa)	Coefficiente de Poisson (ν)	σ_{ci} (MPa)	mi
2,30	8,0	8.200	0,25	0,7	8,24

Figure 4.1.3.e.- Analysis done volcanic tuffs.

Riolites

PARÁMETROS DE LA ROCA INTACTA - RIOLITAS					
γ_s (t/m ³)	σ_{ci} (MPa)	E_i (MPa)	Coefficiente de Poisson (ν)	σ_{ci} (MPa)	mi
2,6	42,8	150.000	0,25	8,1	20

Figure 4.1.3.f.- Analysis done volcanic tuffs.f

Weak shales

PARÁMETROS DE LA ROCA INTACTA - PIZARRAS BLANDAS					
γ_s (t/m ³)	σ_{ci} (MPa)	E_i (MPa)	Coefficiente de Poisson (ν)	σ_{ci} (MPa)	mi
2,4	4,4	22.500	0,2	0,88	4,775

Figure 4.1.3.g.- Analysis done weak shales

Strong shales

PARÁMETROS DE LA ROCA INTACTA – PIZARRAS DURAS					
γ_s (t/m ³)	σ_{ci} (MPa)	E_i (MPa)	Coefficiente de Poisson (ν)	σ_{ti} (MPa)	m_i
2,7	12,05	64.000	0,2	2,05	6,218

Figure 4.1.3.h.- Analysis done Strong shales

Massive and semi-massive sulphides

PARÁMETROS DE LA ROCA INTACTA – SULFUROS					
γ_s (t/m ³)	σ_{ci} (MPa)	E_i (MPa)	Coefficiente de Poisson (ν)	σ_{ti} (MPa)	m_i
4,4	115	80.000	0,2	12,85	8,3

Figure 4.1.3.i.- Analysis done Strong shales

Strength Properties of the Rock Mass

In order to convert laboratory geotechnical parameters into usable rock mass parameters, observations of Rock Mass Rating (22) (Bieniawski, 1985) for the various rock types was undertaken. The rating system consists of utilising field observations to determine the competency of a rock using the following points based classification:

- Very Good Rock (RMR between 81 and 100).
- Good Rock (RMR between 61 and 80).
- Moderate Rock (RMR between 41 and 60).
- Poor quality Rock (RMR between 21 and 40).
- Very poor quality Rock (RMR less than 20).

To determine the assignable points the system shown in **Table 4.1.3.III** is utilised.

RMR (1)	STRENGTH OF INTACT ROCK MATERIAL								
VALUE (MPa)	> 240	240-175	175-125	125-100	100-75	75-50	50-30	30-10	< 10
RATING	15	14	12	10	8	6	4	2	1

RMR (2+3)	RQD AND SPACING OF DISCONTINUITIES																			
DISCONTINUITIES PER m	0	0,5	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
RATING	40	37	34	31	29	28	27	26	25	23	22	22	21	20	19	18	17	16	16	15
DISCONTINUITIES PER m	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38
RATING	15	14	14	12	12	11	11	10	10	10	9	9	9	33	8	8	8	37	7	7
DISCONTINUITIES PER m	39	40	41	42	43	44	45	46	47	48	49	50								
RATING	7	7	7	7	5	5	5	4	4	4	4	3								

RMR (4)	CONDITIONS OF DISCONTINUITIES				
PERSISTENCE	< 1 m	1-3 m	3-10 m	10-20 m	> 20 m
	6	4	2	1	0
SEPARATION	0	< 1 mm	0,1-1 mm	1-5 mm	> 5 mm
	6	5	4	1	0
ROUGHNESS	Very rough	Rough	Slightly rough	Smooth	Slicker sided
	6	5	3	1	0
INFILLING	None	Hard filling		Soft filling	
		< 5 mm	> 5 mm	< 5 mm	> 5 mm
	6	4	2	2	0
WEATHERING	Unweathered	Slightly weathered	Moderated weathered	Highly weathered	Discomposed
	6	5	3	1	0

RMR (5)	GROUNDWATER				
STATE	Completely dry	Damp	Wet	Dripping	Flowing
FLOW (l/min • m)	0	< 1	1-2,5	2,5-12,5	> 12,5
RATIO (u/σ ₁)	0	< 0,1	0,1-0,2	0,2-0,5	> 0,5
RATING	15	10	7	4	0

Table 4.1.3.III.- Definition of the RMR (89).

With the RMR value obtained in the field and assuming RMR = GSI, it is possible to reduce the properties obtained for the intact rock, using the Hoek-Brown breakage criterion expressed in the equation:

$$\sigma_1' = \sigma_3' + \sigma_{ci} \cdot \left(m \cdot \frac{\sigma_3'}{\sigma_{ci}} + s \right)^a$$

Where m and s are two parameters function of the lithology of the rock (m_i) and of the state of the rock mass characterized by the quoted GSI. This dependence is given by the expressions of (23) Hoek et al (2005), based on those of (24) (Priest and Brown (1983)):

$$m = m_i \cdot e^{\left(\frac{GSI-100}{28-14D} \right)}$$

$$s = e^{\left(\frac{GSI-100}{9-3D} \right)}$$

$$a = \frac{1}{2} + \frac{1}{6} \cdot \left(e^{-GSI/15} - e^{-20/3} \right)$$

Being m_i a parameter dependent of the lithology of the rock. Its value can be taken from the bibliography, or determined, in each case by means of laboratory tests. D is a factor that depends on the degree of de-confining produced in the rock by the excavation and relaxation of tensions. It varies between 0 for rocks unaffected by the excavation and 1 for those that have been very affected. The uniaxial compression strength of the rock mass can be obtained by the following expression:

$$\sigma_c = \sigma_{ci} \cdot s^a$$

Once obtained, m and s can be estimated the values of cohesion and friction of the rocky mass, since there is a ratio for a given level of confining pressure. The enveloping intrinsic resistance curve of the ultimate tensile states defined by the Hoek and Brown criteria can be replaced by a line defined by the angle of friction and cohesion, for which different methods have been proposed.

In general, and based on experience, it has been found that some of these methods obtain excessively high values for the cohesion and friction parameters. Therefore, in this case, it is proposed to follow the adjustment for a given interval of the confining pressure σ_3 , by means of a Mohr-Coulomb line secant to the breaking curve of Hoek-Brown, which complies with the following expressions:

$$\lambda_m = 1 - \frac{\sigma_c \cdot \sqrt{s}}{\sigma_3} + \sqrt{\frac{\sigma_c}{\sigma_3} \cdot m + \left(\frac{\sigma_c}{\sigma_3} \right)^2 \cdot s}$$

$$\phi_m = 2 \cdot \arctg \sqrt{\lambda_m} - 90$$

$$C_m = \frac{\sqrt{s} \cdot \sigma_c \cdot (1 - \sin \phi_m)}{2 \cdot \cos \phi_m}$$

" m " refers to the rocky mass and σ_3 is taken equal to the litostatic load of the terrain.

Deformational parameters

The modulus of elasticity of the rock mass has been estimated for many years, empirically from numerous correlations and the two most recent have been proposed by (25) Galera et al, (2005) and Hoek and Diederichs (2005). The correlation of Bieniawski-Galera (1995) relates the module at the level of mass with the index RMR and the module of Elasticity at intact rock level as follows:

$$E_m = E_i \cdot e^{((RMR-100)/36)}$$

The correlation of Hoek-Diederichs obtains the modulus of elasticity of the massif from the parameter D , the module a level of intact rock and the parameter GSI (assimilable to the RMR index in this case) as follows:

$$E_m = E_i \cdot \left(0,02 + \frac{1 - \frac{D}{2}}{1 + e^{\left(\frac{(75 + 25D - GS)}{11} \right)}} \right)$$

RMR Index Determination

Utilising statistical and observational analysis of the various lithological groupings, determination of the Rock Mass Rating (RMR) was undertaken, as shown the **Figures 4.1.3.j, k and l**:

Gossan

Figure 4.1.3.j.- Rock Mass Rating distributed as a percentage.

Volcanic Tuffs and Riolites

(Due to the similar geotechnical characteristics of the riolites, conservatively the same values were associated to them as the volcanic tuffs).

Figure 4.1.3.k.- Rock Mass Rating distributed as a percentage.

Shales

Figure 4.1.3.I.- Rock Mass Rating distributed as a percentage.

In general the weaker and stronger shales parameters were analysed conjunctively. In this way the following ranges of RMR_b representative of the shales are considered:

- Healthy shales: $RMR_b > 75$
- Slightly fractured shales: $RMR_b = 50-75$
- Altered and fractured shales: $RMR_b = 30-50$
- Fractures and fracture zones on shales: $RMR_b = <30$

Table 4.1.3.III shows the results of the strength and deformational parameters considered for the Paleozoic rocks.

4.2 IMPROVEMENTS IN GEOTECHNICAL CHARACTERISATION FOR DESIGN

4.2.1 INITIAL CONCEPTS

The selection of refined parameters for the various lithologies of the pit undertaken in accordance with the methodology highlighted above. The overriding objective of this work was to provide calculation of safety factors for various geometrical considerations (Single Bench, Multiple Bench and Global) to ensure that pit stability was preserved and in conformance with the previously mentioned ITC regulatory requirements.

Prior to opening the pit, work had been undertaken for the mine's original feasibility study. However this was based on the original limited information from a few dedicated boreholes (six in total) and it was considered that this would require updating during operations commencement. A worthwhile notation is that at the early mine permitting stage, concern was predominately associated with the behaviour of the marl geological unit rather than the underlying hard rock. That first limit state analysis is shown in **Figure 4.2.1.a**:

Figure 4.2.1.a.- Example of limit state analysis undertaken for the original design.

A number of observations can be made of this initial analysis:

- A design angle of 28° was established as operationally stable within the marls.
- With limited information the Palaeozoic was considered a hard rock and to be reasonably stable with a global angle of 45° .
- Pore pressures were estimated from initial hydrogeological information supplied by CLC following drainage well installation (no hydrogeological mechanical coupling). The limit state programme provided an interpolation between known inputted values.
- The non-seismic (static) situation is shown to conform to ITC minimum requirements of a 1.2 safety factor.
- In order to construct a pseudo-dynamic model, horizontal acceleration was applied in analysis. A sensitivity analysis was also undertaken on the vector of this acceleration to ensure validity of its useage. A calculated value of $0.073g$ was obtained via implementation of seismic-resistance guidelines NCSE-02.
- The seismic situation was also shown to conform to minimum ITC requirements of FS of 1.1.
- The analysis worked well during the commencement of the mining operations and marl stripping activities with the exception of a few local instability mechanisms described below.

The objective from 2007 onwards was to refine the original relatively simple limit state analysis taking into account observations of geological structures, bulk samples taken during the excavation, and also instabilities observed during operations. Improvements in understanding of the pit behaviour was undertaken by the author from 2010 to the present date and comprised the following:

4.2.2 PROGRESSIVE OPTIMISATIONS

2010 to 2011

Incorporation of pit cartography and soil structural observations (weaker bedding planes and joint sets) into existing limit state analysis for various worst case configurations of the pit.

2012 to 2013

Pit Optimisation, improved lithological domaining in 10 metres benches cuts, segregation of upper and lower marls with reference to resistance parameters.

2014

FLAC 3D analysis of all phases of the pit development through to 2022 incorporating 2012 lithological domaining, previous segregation of marls, bedding planes (ubiquitous joint constitutive modelling) and hydromechanical coupling.

2015

FLAC 2D analysis of worst pit configuration - northern slopes with desfavorable bedding plane orientation (3° dip towards south), hydromechanical coupling, very fine 0.5 m meshing.

2016

Phase2 7.0 check on FLAC2D analysis without consideration of hydromechanical coupling, 1.0m meshing. The object being to provide a secondary check on FLAC results and review the real likely effect of the hydromechanical coupling on safety factors.

This section describes in detail each of the analysis process, as applied to the Las Cruces mining pit.

4.2.3 MINING EXCAVATION DEVELOPMENT

To understand the motives for refinement of the original geotechnical design, the advancement of mine stripping and exploitation should be clearly explained. Las Cruces Project commenced from a green field site, previously dedicated to agriculture and without previous mining activities due to the effect of many metres of marls overlying the mineral deposit.

The mine's general layout can be seen in the **Photos 4.2.3.a to 4.2.3.n**:

Photo 4.2.3.a.- General view of the CLC mine.

The pit itself is one of the world's largest excavations in soft rock. This provoked concerns with respect to applicability of soil mechanics and constitutive models to pit stability. To illustrate the scale of these stripping activities the year on year progress of the mine development has been recorded via aerial photography shown below:

Photo 4.2.3.b.- 2006: Green field site - day one: mining plant can be observed in the distance.

Photo 4.2.3.c.- 2007 - Initial stripping activities, note well preserved nature of the fresh slopes.

Photo 4.2.3.d.- 2008 - Push for phase 1 (internal cone shape directed towards highest grade ore).

Photo 4.2.3.e.- 2009 – Exposure of the phase 1 Palaeozoic (Gossan and Enriched Mineral).

Photo 4.2.3.f.- 2010 - (Mine operations stopped due to 2009 authorisation issues), note severe superficial erosion of marls following 2009/2010 heavy winter rains.

Photo 4.2.3.g.- 2011 - Expansion of phase 1 cone into phase 2 cone and commencement of phase 3 stripping towards east.

Photo 4.2.3.h.- 2012 – Phase 3 area substantially deepen, some initial preparatory stripping of phase 4.

Photo 4.2.3.i.- 2013- Phase 3 mineral exposed. Phase 4 pushback fully underway.

Photo 4.2.3.j.- 2014 – Phase 4 mineral exposed on the south side of the pit.

Photo 4.2.3.k.- 2015 - Phase 4 full exposed, phase 5 deepening significantly.

Photo 4.2.3.l.- 2016 - Viewing from old phase 1 area, phase 5 stripping continues, phase 1/2 marl backfilling has commenced.

Photo 4.2.3.m.- 2017 – View from east showing exposure of mineral in phase 5, exploitation of mineral in phase 4 and marl backfill operation underway in phase 1/2.

The final pit configuration as planned to finish in mid-2020 is shown in **Photo 4.2.3.n:**

Photo 4.2.3.n.- Final Planned Pit Configuration.

4.2.4 GEOLOGICAL STRUCTURAL OBSERVATIONS

During marl stripping activities geotechnical mapping of the marls was undertaken in preparation for future design refinements. **Figures 4.2.4.a** and **4.2.4.b** shows a typical sheet with a bench map of the pre-stripping stage and second one for the Paleozoic.

Figure 4.2.4.a.- Geo-mechanical mapping of a marl bench.

Figure 4.2.4.b.- Geomechanical mapping of a Paleozoic bench.

Due to this mapping many continuous and discontinuous geological structures were discerned. These observations quickly dispelled any initial design pre-concepts of a heterogeneous Miocene marl with many apparent structures and visually identifiable bedding planes.

The cartography of the marls (commencing in 2007) clearly highlighted structures and bedding planes. Improvements to the cartography were implemented in 2015 to incorporate blast performance observations and reconciliation.

Bedding planes were observed as the excavations developed. The bedding planes appear numerous and apparent at approximately 5 metre vertical intervals ranging from 3 to 7 m (26) (Cooper et al. 2011) with an average dip of 3° to the South-Southwest. Below in the left photo is the general aspect of the marls in which the different grey tones clearly highlight various bedding plane locations.

Photos 4.2.4.c, 4.2.4.d and 4.2.4.e show displacement in those bedding planes following excavation. Displacement was usually rapid (occurring and slowing to null over a 1 week period) and could total 25 mm on strongly defined bedding planes.

Photo 4.2.4.c

Photo 4.2.4.d

Photo 4.2.4.e.- Displacement and strong discontinuity along principal bedding planes.

These bedding planes were carefully monitored, measuring the displacement between both sides of the plane. Systematically it was established that the displacement ranges between a few mm to cm, an average being around 10 mm. The response in the inclinometers clearly showed the same phenomenon and temporal and spatial correlation with these slope faces displacements was achieved.

Figure 4.2.4.f shows a typical response of two pit inclinometers where the behaviour of the bedding planes, can clearly be observed.

Figure 4.2.4.f.- Bedding planes shown in two open pit inclinometers.

From the observations a number of conclusions could be drawn:

- The bedding planes are present with an exceptional vertical regularity every 3 to 7 metres with an average value of 5 metres. The displacement at each bedding plane registered is about 5 mm but in some cases reached up to 25 mm and created some initial alarm amongst the mining team.
- Several correlations between the displacements measured in the inclinometers and in the slopes, have been carried out. It was concluded that the displacement on the surface slopes in which the bedding planes outcropped, is always higher than inside the rock mass at the inclinometers which is logical considering the highest principal stress rotation occurs at the slope face.

The total displacements measured at the topographical prisms located at the perimeter of the pit were systematically higher than the ones registered at the head of the inclinometers. Consequently, the total displacement measured at the benches and at the head of the pit was a combination between the displacements at the bedding planes, an instantaneous and elastic deformation in the marl body, as a result of alleviation of overburden, and the deformations that take place at the marl “bridges” between bedding planes.

These structures were observed at relatively shallow levels and subsequently associated with localised toppling and block failures observed at those depths. The principal joint set was defined striking 80° towards the north-north east. In many cases complementary joints were observed at 90° to the joints sets dip direction with a similar dip as shown in **Photo 4.2.4.g**.

Photo 4.2.4.g.- Sub-vertical tectonic induced structures observed in the upper marls.

These structures on large scales, in combination with strongly defined bedding planes formed geometrical blocks which on some occasions caused significant block failures requiring subsequent slope remediation. The author considers that these joints sets are likely to be recent, tectonic in nature and possibly similar to those structures interpreted during the retrospective analysis undertaken of the Los Frailes tailings instability (27) (Alonso, E. E. & Gens, A. 2006). At Los Frailes the failure manifested a clear preference for deformations towards the north-east leading to the suggestion of a dis-favourable loss of lateral cohesion along that direction.

The **Figure 4.2.4.h** shows exactly that deformation mode at Los Frailes and the photo above may provide an example of the type of structures responsible. This is considered an area where further works could be undertaken in the Guadalquivir marls.

Figure 4.2.4.h.- Preferential movement observed at Los Frailes Dam.

The combination of these observations led to the return to a concept of separation into lower and upper marls, prevalent as a consideration in early design due to known similarities to the Oxford clays but discounted due to lack of sufficient evidence obtained in the first geotechnical investigation drill-holes. However, during mine stripping this aligned very well with observations of sharp increases in marl resistance at depth defined by the required use of pre-split blasting to enable efficient excavation. This transition occurred at around 75 metres depth and appeared to coincide well with the observation of a strong bedding plane at this depth, both visually and in the pit inclinometers.

The **Figure 4.2.4.i** shows the correlation process for the movements in inclinometer in a section crossing the pit towards the mines adjacent south dump.

Figure 4.2.4.i.- Displacement shown in various pit inclinometer, subsequently utilised to improve definition of marl sub-categorisation with depth.

To follow, a brief description of the main observable instabilities is provided.

4.2.5 OBSERVED GEOTECHNICAL INSTABILITIES

4.2.5.1 TOPPLING IN THE PIT

The first type of instability corresponds to toppling failures due to desiccation joints. These joints are almost vertical and often they are related to failures after heavy rains as shown in **Photo 4.2.5.1.a**.

Photo 4.2.5.1.a.- Toppling in 2010 following heavy rains.

In 2013, instability formed on the newly formed phase 4 slopes, where sub-vertical joint sets combined with a weakened bedding plane and water ingress to produce a considerable block failure mechanism, approximately 3 berms depth 30 metres height and 150 metres width.

4.2.5.2 NORTH AREA PHASE 4 BLOCK FAILURE AND ANALYSIS

Photos 4.2.5.2.a and 4.2.5.2.b show two views of this failure.

Photo 4.2.5.2.a.- Failure at the North Area in Phase 4.

Photo 4.2.5.2.b.- Detail of the failure at the North Area in Phase 4.

Subsequent analysis was undertaken of the nature of the failure both with block failure modes and also with a limit state analysis, with focused slip planes along the known failed geometry. The factors of safety were then utilised to predict the likelihood of this area providing future problems to mining operations.

For each scenario, potential consequences and factor of safety were evaluated, as well as an attempt to determine the probability of future occurrence. A relatively objective form of analysis of probability was undertaken using real case data (28) (Silva & Lamb, 2008) relating Safety Factor, quality of construction, and probability of occurrence as shown in **Figure 4.2.5.2.c.**

Photo 4.2.5.2.c.- Failure Probability Verse Safety Factor, (Silva & Lamb, 2008)

The graph is very useful as it represents the sum knowledge of many well renowned geotechnical engineers regarding design vs. performance, enabling an onsite engineer to provide non-geotechnical engineers a good understanding of the risks associated with lower safety factors at design stage or poorly executing the design itself. For the purposes of evaluation and conservatism, Las Cruces is placed as class III (Class II would likely be considered for an operation 100% supervised during construction, such as a dam construction).

The results of the analysis on the northern slopes block failure are shown in **Figures 4.2.5.2.d, e, f, g and h.**

Figure 4.2.5.2.d.- 1 Reactivation movement of the infill material.

Figure 4.2.5.2.e.- 2 Slip failure including some of the natural marls incorporating observed tension cracks behind existing electrical post.

Figure 4.2.5.2.f.- 3 Slip Failure including more natural marls and incorporating known bedding plane with residual friction angle.

Figure 4.2.5.2.g.- 4 Failure plane just behind the Ramira dump, beyond which we have the Molino policing zone.

Figure 4.2.5.2.h.- 5 Failure to the new Molino stream.

The use of Silva & Lamb's probability / safety factor correlations was useful in providing an indication of the severity of the problem and the possibility of affection to other areas. The analysis concluded the very low probability of the topping instability of the north slopes creating an issue for the nearest sensitive infrastructure, in this case a diverted stream to the east.

4.2.5.3 FOOTWALL SHALES

The concern over the poor behaviour of the footwall shales in late 2010 was sufficiently high that it warranted special attention in terms of the implementation of:

- More detailed regime of slope mapping.
- Specific kinematic instability analysis.
- Modifications to the pit geometrical configuration.
- Improvements to the slope superficial monitoring.
- Slope sustainment implementation (Rock bolting and meshing).
- Additional pit geotechnical investigation and characterisation.

Photo 4.2.5.3.a below shows the first exposure of the shales at the south slope.

Photo 4.2.5.3.a.- First exposure of problematic footwall shales.

The orientation of the joint sets corresponded dis-favourably at this location with the pit slope strike and dip, provoking toppling and planar failure mechanisms.

The failures in the footwall shales prompted detailed mapping and structural analysis to determine the exact failure mechanisms involved as well as probable locations where this poor material could be encountered in the futures as shown in **Figures 4.2.5.3.b** and **4.2.5.3.c**.

[illegible]

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Early analysis shown below was undertaken by the author to determine with depth the extents of the shales. Reviewing the raw data, it became clear that, closer to the mineralisation, the shales were altered and were more strongly metamorphic and by consequence of high competence.

On this basis a proposal was made and accepted to temporarily relocate the passing haul road as shown in **Figure 4.2.5.3.d**.

Figure 4.2.5.3.d.- Initial analysis of likely lithological to be encountered at depth.

This initial lithological domaining resulted in a translation of the mine's haul road from that shown in blue, to that shown in red. This resulted in stable slopes for the remainder of the pit phase 2 excavations with virtually no mineral loss. This is an example of the positive effective of a close working relation between the on-site geotechnical and mining operations team.

Once the initial problem was resolved, a programme of geotechnical, lithological and rock mass rating characterisation was implemented for the overall mine running from 2011 to 2012 and resulting in the pit optimisation analysis in 2012. Examples of the plans and sections undertaken are shown in **Figures 4.2.5.3.e** and **4.2.5.3.f**.

Figure 4.2.5.3.e.- Section highlighting the footwall shales in the south.

Figure 4.2.5.3.f.- Plan view at 150 metres depth.

Also, as shown in **Photo 4.2.5.3.g**, some minor reinforcement measures, rock bolting and wire mesh, were locally applied.

Photo 4.2.5.3.g.- Slope support locally implemented at the footwall shales.

In addition to the slope support, additional recommendations were made to distance the adjacent haul road from these slopes into more competent material and also modify the timing for the extract of mineral. The recommendations were implemented with great success and all mineral was extracted in a specific project undertaken in 2014. Additionally, geotechnical optimisation of the blasting in the area was implemented and this aspect will be highlighted later in this thesis.

4.2.5.4 PIT OPTIMISATION PROJECT IN 2012

From 2010 to 2012 the geological department were presented with a number of issues during re-conciliation of the geological block model with their operational grade control. These issues were predominately associated with insufficient drill-holes for interpolation purposes and information on the geological mineralogical structures which influenced the mineral enrichment. As such more drill-holes were planned in a series of drilling campaigns to fill these gaps in knowledge.

The author was asked if improvements in definition of the geotechnical aspects of the ore body was warranted. To this objective, a series of dedicated geotechnical drillholes for the purposes of increasing knowledge predominately of the Palaeozoic rock and further define the extents of poor quality rock such as the shales encountered in 2010/2011 in the phase 2 areas.

The methodology for the characterisation of the rocks during this campaign was defined in section 3 of this thesis. From a design perspective though, considerable works were undertaken on the conceptual models.

Specifically, and following review of the aforementioned evidence of observable bedding planes, increasing geotechnical strength parameters with depth (evidenced both in the laboratory and in mine blasting operations) The following premises were held true for the advancement of a new optimised pit limit state analysis:

1. Many bedding planes are observable at the mine, however the majority of these are not considered structurally relevant, with the exception of:
 - a. A bedding plane at -15 m AOD (approximately 45 metres depth depending on location).
 - b. A second bedding plane at -55 m AOD (approximately 75 metres depth).
2. In particular the second bedding plane has considerable thickness (200 mm to 300 mm) with crushed marls evident. This indicates long-term movement has occurred towards the south with likely disfavorable residual rather than peak strength parameters likely to be available along this bedding plane.
3. There was a significant increase in resistance parameters associated with the transition from “upper” marls to “lower” marls at around 75 metres depth. This can be demonstrated with the **Figure 4.2.5.4.a** which was subsequently refined during the optimisation.

Figure 4.2.5.4.a.- Sharp strength increases with analyses in 2012.

4. The Palaeozoic could no longer be considered as a uniform “hard rock” but would require specific attention in design considering a segregation of the rock type in:
- Massive Sulphides
 - Semi Massive Sulphides
 - Gossan
 - Volcanic Tuffs and Rhyolites
 - Shales

An objective of the pit optimisation was to utilise the segregation of upper and lower marls resistance parameters to review the possibility of increasing permissible slope angle in this material. At the same time, it was acknowledged that, logically, angles in the shales should decrease and slope angles in more competent rock could increase in value.

4.2.5.4.1 ANALYSIS OF GLOBAL STABILITY OF SLOPES IN MARLS

In the case of the marl package, the following analyses were carried out:

- Analysis of the general slope for inclinations of 24°, 26°, 28°, 30° and 32°.
- Analysis of the double slope in marl for slopes of 29°, 30° and 31°. In this analysis the inclination of the slope was increased over an average elevation of -45 meters above sea level, coinciding with LEVEL 1, from which it is necessary to use the blasting in the excavation of the slopes due to the considerable increase in competence of the marls.
- Analysis of the inter-ramp slopes. For this analysis the profile of the highest existing slope in marls was utilised as the most unfavourable case study.

The final optimised geometry is shown in the **Figures 4.2.5.4.1.a** and **4.2.5.4.1.b**, below, demonstrating direct compliance with the ITC regulations under worst case un-drained conditions and with both seismic and static considerations.

Figure 4.2.5.4.1.a.- Lower slopes – 31°, Upper Slopes 28°, Static and un-drained.

Figure 4.2.5.4.1.b.- Lower slopes – 31°, Upper Slopes 28°, Seismic and undrained.

Marl Slopes Sensitivity Analysis

As indicated above, the following analyses were carried out in the marl package for:

- Global slope angles of 24°, 26°, 28°, 30° and 32°.
- Lower level (below LEVEL-1) slope angles of 29°, 30° and 31°, maintaining a 28° in the upper slope.
- Slope inter-ramp from the analysis of the existing worst case slope configuration.

The results obtained in the analyses of the general slope and slope with double slope are shown in the **Figures 4.2.5.4.1.c** and **4.2.5.4.1.d**:

Figure 4.2.5.4.1.c.- Global slope angle sensitivity analysis in the marls in 2012.

Figure 4.2.5.4.1.d.- Lower marl slope angle sensitivity analysis in the marls in 2012.

From which it follows that:

- For the drained general slope, the minimum safety factors with an angle greater than 30° are exceeded.
- For the non-drained general slope, the requirements of ITC SM.07.1.03 are met for an angle of 29.5 °.
- For the double slope drained the safety factors comply for lower slope angle greater than 31°.
- For double slope not drained, the safety factors comply with a slope angle of less than 31 °.

In the analysis of the existing worst case interramp slope configuration in the excavation, the safety factors obtained exceed the minimum values required in all cases analysed, as shown at **Table 4.2.5.4.1.I:**

Safety Factor	General slope angle (28°)	Interramp slopes (31°)
Drained static	1.636	1.796
Drained dynamic	1.411	1.558
Undrained static	1.336	1.636
Undrained dynamic	1.137	1.423

Table 4.2.5.4.1.I.- Safety factors obtained in 2012.

For the optimised design of the slopes in marls, a slope divided in two zones was therefore proposed, whose limit is approximately the dimension -45 msnm, with general angle of 29.5° which is achieved by an angle of 30° or 31° in the lower zone (lower marls) and maintaining the overall angle of the current design of 28 ° in the upper zone.

This proposed design was supported by the experience gained during the first 6 years of excavation in which it was observed that the marls showed a more competent behaviour in the lower levels of the pit. In addition, it was felt with confidence that a slope with a general slope at 28° would be relatively conservative since there had been no significant global or large multi bench stability issues during this period.

For this reason, it was considered that we should maintain an angle of 28° in the less competent upper marls and force the slope slightly in the lower marls until a general slope angle around 29.5° was obtained.

As for the interramp slope, although the safety factors obtained were very high, a local angle limit of 34° was established to ensure localised failure could not develop.

4.2.5.4.2 ANALYSIS OF STABILITY OF SLOPES IN PALEOZOIC

Based on the lithotypes present in the excavation, an optimized design of the general slopes was proposed for each of them. To undertake this analysis, the following simplifications were required.

An average GSI (interchangeable with rock mass rating, RMR, for the purposes of this study) value of around 40 was taken in all cases. This simplification has been based on the existing RMR data from all the surveys of the previous campaigns and from the drillings drilled in the current campaign. Based on this information and given that the RMR allocation at survey scale is not completely realistic, it was adjusted to arrive at a more representative value of the slopes excavated in the cut with respect to observations.

Approximate slopes were defined for highly fractured areas and / or fault zones that may be encountered during excavation.

The optimized design angles proposed for each lithotype are for slope under drained conditions in all cases. This implies that in case of water in the pit, the slopes must be drained via other methods such as horizontal drainage systems.

Although the different types of shales have been characterised separately, during the excavation, shales can be found with behaviour and average properties between both, being difficult to differentiate. For operational reasons, optimised design angles were defined for those shales in which the orientation of the excavation slope with reference to the families of present discontinuities has also been taken into account.

The tuffs and rhyolites were also treated together, given the reduced representation of rhyolites in the pit.

Since the data are limited, empirical abacuses were created into which the results of the stability analyses performed were represented and the values of the safety factor were correlated. These abacuses were useful for establishing the preliminary slope design to be adopted in each case.

Gossan

The optimized overall slope design angle proposed for this rocky lithotype was 45°, although it could be increased up to 48°. In fractured areas or fault areas an angle of 40° could be adopted.

Tuffs

The optimized overall slope design angle proposed for this rock lithotype was 45°, although it could be increased up to 48°. In fractured areas or fault areas an angle of 40° could be adopted.

Rhyolites

The optimized overall slope design angle proposed for this rock lithotype was 45°, although it could be increased up to 48°. In fractured areas or fault areas an angle of 40° could be adopted.

The optimized overall slope design angle proposed for this rock lithotype was 32° for the South Slope and East Slope and 35° for the North Slope and West Slope. In fractured zones or fault areas, an angle of less than 28° could be adopted.

For the individual bench slope, it was not recommended to exceed 60° to avoid mismatches of stratification and / or schistosity that could trigger localised failures subsequently and potentially affecting the upper benches.

Shales

The optimized overall slope design angle proposed for this rock lithotype was 32° for the South Slope and East Slope and 35° for the North Slope and West Slope. In fractured zones or fault zones, an angle of less than 30° could be adopted.

For the individual bench slope, it was not recommended to exceed 60° to avoid mismatches of stratification and / or schistosity that could trigger localised failures subsequently and potentially affecting the upper benches.

Sulphides

The optimized overall slope design angle proposed for this rock lithotype was 50°. In fractured zones or fault areas a 45° angle could be adopted.

The results of these analyses have been represented in typical height-angle slope graphs, showing the ISO safety factor curves. These graphs are shown in the **Figures 4.2.5.4.2.a to 4.2.5.4.2.e**.

Figure 4.2.5.4.2.a.- Goossan: Height of slopes verses angles of slopes showing safety factor with RMR 0 (left) and 40 (right)

Figure 4.2.5.4.2.b.- Tuffs: Height of slopes verses angles of slopes showing safety factor with RMR 0 (left) and 40 (right)

Figure 4.2.5.4.2.c.- Rhyolites: Height of slopes versus angles of slopes showing safety factor with RMR 0 (left) and 40 (right)

Figure 4.2.5.4.2.d.- Weak shales: Height of slopes versus angles of slopes showing safety factor with RMR 0 (left) and 40 (right)

Figure 4.2.5.4.2.e.- Stronger shales: Height of slopes verses angles of slopes showing safety factor with RMR 0 (left) and 40 (right)

4.2.5.4.3 SUMMARY OF 2012 PIT OPTIMISATION RESULTS

The **Table 4.2.5.4.3.I** below summarises the optimized design angles in each case:

Lithotype	General slope angle	Fractured zones
Gossan	45-48	40
Tuffs and Rhyolites	45-48	40
Shales	32 south and east pit slopes 35 north and west pit slopes	<28
Sulphides	50	45

Table 4.2.5.4.3.I.- Recommended slope design angles for each lithology.

4.3 FURTHER IMPROVEMENTS TO THE PIT DESIGN: 3D FINITE DIFFERENCE MODELLING

The exposure of the footwalls on the south side of the pit in the pits south westerly aspect in 2012 resulted in a temporary loss of mineral, whilst haul road positions were modified to ensure stability. However, the prevalent thought in the geotechnical division was that this mineral could and should be recovered. Around the same time, on the north side of the pit, slight modifications to the crest in the form of haul road upfilling and the fill in the adjacent north dump raised new question as to stability effect within the pit. These questions needed answering and, in order to ensure stability was maintained during these operations, a new analysis was undertaken

utilising finite difference modelling software. Specifically for the above motives, and to reduce processing requirements the mining pit was effectively split into a northern aspect analysis and a southern aspect analysis.

Some fundamental concepts were incorporated into the analysis was as follows:

1. A tight **5m x 5m** mesh pattern would be used in the proximity of the pit increasing to **10m x 10m** at distance from the pit. This would effectively allow estimation of stress induced strain due to pit decompression, whilst keeping mesh dimensions of a manageable size.
2. The use of hydro-mechanical coupling to evaluate the effects of the pit decompression produced as a result of rotation of the marl masses principle stresses. In principle, this pit decompression should result in a reduction in pore pressure. The drop in pore pressure would result in increases in safety factors, allowing slope optimisation.
3. The analysis would incorporate both existing geometry and any proposed worst case situations, including slopes steepening to recover high grade ore.
4. The analysis would utilise the same coordinate systems as the mines visualisation software (VULCAN) such that any subsequent wireframes or geological categorisation could be easily incorporated into mining operations.
5. The effect of short-term negative pore pressures close to the faces would be evaluated and the longer-term loss of cohesion would also be accounted for. This would represent, in principle, a worst-case situation.

The results of this 3D modelling were summarised at (29) Cooper et al. 2014, and a general view of the geology incorporated in the model is shown in the **Figure 4.3.a**:

Figure 4.3.a.- Complex lithological groupings modelled in FLAC3D.

4.3.1 DESCRIPTION OF THE FLAC3D MODELLING PROCESS

For the three-dimensional numerical modification, the FLAC3D program was used, which is based on an explicit calculation in Finite Differences. The Finite Differences Method (Finite Differences Method, FDM) has a number of advantages with respect to the Finite Element Method (Finite Element Method, FEM) in the study of nonlinear problems (Large deformations, constituent models with post-rupture, etc. usual in geotechnics) in which the rigidity matrix Finite Element is not invertible.

The Finite Differences Method, being an explicit formulation that is solved by pseudo-temporal iterations, allows tracking of the evolution of the problem in large deformations, being able to analysis even up until the collapse failure point.

The program used in numerical modelling is the FLAC3D (Version 5.0) program from Itasca Consulting Group Inc., it allows simulation of the behaviour of three-dimensional structures formed by soil, rock or other materials that experience a plastic flow when they reach their resistance limit. To accurately model this plastic flow, FLAC3D uses a mixed discretisation technique. ((30) Itasca Consulting Group, et al 2006)

The domain is discretized in polyhedral elements (usually 6 or 4 faces), forming a three-dimensional mesh which is adjusted by the user to the geometry of the problem. The primitive meshes included in the program (tetrahedron type, pyramid, parallelepiped, cylinder, ring, etc.) allowing the generation of complex meshes.

The FLAC3D built-in graphic system allows the generation of high-resolution images of the mesh and all variables involved in the calculation. This representation can be made during the creation of the model and also during calculation steps.

The FISH internal programming language allows the user to extend the program's capabilities, creating new variables and functions, define complex load schemes, assist in the generation of meshes, etc. In the documentation the program includes an extensive library of FISH functions that serve both for direct use and to familiarise the user with the language capabilities.

The main features of the program are the simulation of continuous media with large deformations with the possibility of including:

- Discontinuity (interfaces).
- Explicit solution scheme that allows a numerically stable solution of physically unstable problems.
- Extensive library of constituent models.
- Random distribution of any property; Evolution and mechanical properties with stress-path.
- Easy imposition of general boundary conditions.
- Structural elements type, cables, beams, piles, plates and membranes, with a coupling to the generalized mesh.
- Analysis of water flow in transient regime with hydro-mechanical coupling.
- Dynamic analysis module to simulate the seismic response of a model in the time domain.
- Viscous module that allows the simulation of systems with time-dependent behavioural materials.

- Thermal module that allows the transient analysis of the heat flow as well as the thermo-mechanical coupling.

4.3.2 CALCULATION OF THE SAFETY FACTOR

The safety factor of a geotechnical problem can be calculated using a numerical model (either finite element or of finite differences) progressively reducing the resistance to cutting of the ground until the safety factor defined by the ratio between the original shear strength and resistance of the area where the failure occurs.

This technique, referred to as "reduction of shear strength", has a number of advantages over the usual limit state method calculation on which most programs are based. Among these, the most important is that of critical sliding surface which occurs automatically during the process of resistance reduction, and its shape (Flat, polygonal, circular, logarithmic spiral, etc.) which occurs naturally without it being necessary to introduce it when starting the calculation process. That is to say, the programme finds its own weakest areas and determines safety factors in those areas. In addition, it should be noted that the application of the threshold equilibrium in 3D problems is very limited in practice.

The failure resistance reduction technique has been limited in the past by the high computational cost (in calculation time), a difficulty that is now exceeded by the progress in the calculation speed of the personal computers.

The following subsections describe a set of advantages provided by numerical modelling, together with the technique of reduction of resistance along the failure plane, in the analyses of slope stability.

4.3.3 REDUCTION OF SHEAR STRENGTH FOR THE CALCULATION OF THE SAFETY FACTOR

Stability analysis using numerical models is based on the calculation of the safety factor using the methodology of reduction of soil shear strength. (31) Dawson, E.M. (1998) describes this methodology in detail and compares the results thus obtained with those calculated by the quotient between the shear strength along the breaking surface and the shear stress along the shear stress.

$$FS_{medio} = \frac{\int_S (c + \sigma_n \tan \phi) ds}{\int_S \tau ds}$$

In addition, (32) Varona, P. and Ferrer, M. (1998) have verified the methodology with a program of calculation in finite differences (FLAC) and finite elements (ZSOIL) respectively, checking the safety factors for cases of circular and planar failure.

These two references are selected to show that the cut resistance reduction is a valid methodology to calculate the safety factor of a slope or embankment, being applicable to more complex geometries represented in three-dimensional models with different lithologies ((33) Galera et al., 2003).

In addition, another important aspect is the uniqueness of the safety factor calculated by reducing failure resistance: according to Dawson E.M (1998), for an elastic material-perfectly plastic with a rule of flow, neither the elastic constants of the material, nor the initial stress state, nor stress-path have an effect on the loads at the time of collapse. The safety factor in these conditions (intrinsically assumed by the methods of calculation by means of slices) is unique and independent of these factors.

For practical purposes, in order to calculate FS by means of a numerical model a process of reduction of resistance is implemented, simultaneously reducing the cohesion and the tangent of the friction. The displacement of certain significant points of the model and the passage of a stable situation (with decreasing velocities and finite displacements) to an unstable situation (constant or increasing speeds and increasing displacements). In the present study all nodes of the mesh are included as significant points. The ratio by which the resistance parameters are divided to arrive at said equilibrium limit defines the safety factor.

4.3.4 ADVANTAGES OF SLOPE STABILITY ANALYSIS USING NUMERICAL MODELLING

As described in previous sections, the main advantage of calculating safety factors through reduction is that the sliding surface is automatically derived from the reduction of resistance and it is not necessary to introduce the shape of the same in advance by the programme user. Stability analysis using numerical modelling provides the following possibilities for analysis:

- Geometries and surfaces of complex breaks (especially in 3D)
- Advanced constitutive models.
- Realistic simulation of the constructive sequence
- Realistic simulation of holds.
- Hydro-mechanical coupling.

4.3.5 THE LAS CRUCES CALCULATION MODEL

A three-dimensional model was therefore developed with FLAC3D 5.0 in which the distribution of land and pit geometry was simulated. The purpose of this model was predominately to analyse the stability of the projected operating sequence for the future Phases 3 to 6 in the area of the pits northerly and, separately, southerly aspects.

The model simulates the excavation by means of an annual sequence, in which the effect of the coupling is incorporated comprising:

- Instantaneous mechanical equilibrium (non-drained calculation) after excavation.
- Flow with readjustment of the effective stresses in intervals of 1 month duration.

The safety factor of 2014 was utilised to provide a verification situation for calibration purposes. Following this, the hydro-mechanical balance of each excavation phase, 2015, 2017, 2019 and 2022 was evaluated.

Since this model includes hydro-mechanical coupling and flow, it should be noted that interstitial pressures remain constant during the soil resistance reduction process that is performed when calculating the safety factors.

4.3.6 HYDRO-MECHANICAL CONCEPT IN FLAC3D ANALYSIS

The most simple theoretical example of hydro-mechanical coupling in soils is the theory of effective stress ((34) Terzaghi et al). This theory was developed initially to explain the consolidation of soils over time and the dissipation of pore fluid pressures by flow. The vertical effective stress (σ') is equal to total stress, or the applied load (σ_n), minus the pressure in the pores or joints (μ); where:

$$\sigma' = \sigma_n - \mu$$

This is the fundamental equation for describing hydro-mechanical coupling. The concept of effective stress lies at the core of the understanding of hydro-mechanical coupling. In describing the consolidation response the two key parameters are Specific Storage (S_s), which is the volume of water released per unit drop in head, and hydraulic conductivity (K). In soil mechanics it is usual to describe consolidation in terms of the Coefficient of Consolidation, c_v . In groundwater hydrology, hydraulic conductivity and storage are also key parameters, although the terms normally used are Transmissivity (T) and Storage Coefficient (S). Transmissivity is equal to the hydraulic conductivity (K) times the saturated thickness. However overall these two different approaches to describing the response of a material to changes in water are the same, where:

$$c_v = \frac{K}{S_s} = \frac{T}{S}$$

Types of hydromechanical coupling

(35) Rutqvist and Stephansson (2003) and (36) Wang (2000) recognize two types of hydromechanical coupling; direct and indirect. Direct coupling occurs through deformation and pore fluid interactions. Specifically, solid to fluid coupling, when a change in the stress causes a change in the pore fluid pressure and as such is considered as a direct coupling process.

In general, the coupling associated with our mining pit involved the concept of a significant fall in pore pressure associated with dilatant behaviour of the slopes, a rotation and reduction of pore pressures near those slopes and an improvement in safety factors associated with that pore pressure dissipation.

Figure 4.3.6.a shows a diagrammatic representation of hydro-mechanical coupling.

Figure 4.3.6.a.- Diagrammatic representation of direct hydro-mechanical coupling.

An important aspect of the Las Cruces pit analysis was the very low permeability of the marls. This means that the coupling effect is likely to be near instantaneous and long lasting (since balancing flow into the system post excavation will take a significant period of time, way beyond the mine life).

The value of permeability was derived from 12 large scaled permeability tests monitoring the water head in 55 piezometers during long term pump tests. The sensors were located at the marls, aquifer and fractured top Palaeozoic materials. The value of permeability obtained for the marls ranges from 10^{-9} to 10^{-11} m/s, upto 1.5×10^{-8} m/s for the sands of the Niebla-Posadas aquifer and upto 2.3×10^{-8} m/s for the top of the Palaeozoic materials.

Some of the piezometers installed at the pit were dissipated rapidly the pore pressure as can be observed in **Figure 4.3.6.b.**

Figure 4.3.6.b.- Examples of the rapid pore pressure dissipation during the pit excavation.

During the instantaneous mechanical response, the pore pressure drops according to Skempton's B coefficient:

$$B = \frac{\Delta pp}{\Delta \sigma} = \frac{\Delta pp}{\Delta pp + \Delta \sigma'} = \frac{K_w/n}{K_w/n + K}$$

where:

Δp_p =pore pressure change.

$\Delta \sigma$ = change in mean total stress.

$\Delta \sigma'$ =change in effective stress.

K_w =water bulk modulus (2GPa for pure water).

n =porosity.

K =soil bulk modulus.

The poro-elastic response of a soft soil where $K \ll K_w/n$ is such that most of the change in total stress during unloading will be translated in a drop in pore pressure, as shown in **Figure 4.3.6.c**.

Figure 4.3.6.c.- Theoretical representation of hydromechanical coupling.

During the flow calculation in between excavations, there is a slight increase of pore pressure but in this particular problem it is very small due to the low permeability of the blue marls.

4.3.7 DEVELOPMENT OF GEOMETRY OF MINE PIT

To determine the dimensions of the new model and to incorporate the geometry and phases of pit and the adjacent dumps, monthly topographic surveys were utilised for each year corresponding to the month of December from the year 2006 to the year 2013. For the following years an estimation of the future developments of both the dump and the pit were incorporated. For the pit, the same temporal sequence of excavation as the one used in the model of the 2013 was utilised, incorporating development of the north dump.

Based on the data provided, the generated model covered a length of 2,550 m in the E-W direction (coordinate XUTM from 754,950 to 757,500) and 2000 m in the N-S direction (YUTM coordinate from 4,154,400 to 4,156,400). With reference to the contours, the upper part of the model corresponded to the surface area of the terrain, at + 35 m approx. Contours were incorporated down to a maximum depth of -425 m. The limits of the model were located at a sufficient distance to ensure that the boundary conditions assigned to these limits had no influence on the results.

In the elaboration of the meshing, elements of 10x10x10m were used in the surroundings of the excavation slopes, increasing with distance from the slopes. To have more definition in the geometry of the dumps, the elements representing were allowed to be of 5 m height. The final condition of the north dumps is shown in **Figure 4.3.7.a**:

Figure 4.3.7.a.- Final condition of north dump and proximity to pit (in green).

A cross section with the pit is shown in the **Figure 4.3.7.b** below:

Figure 4.3.7.b.- Pit Cross section.

In the **Figures 4.3.7.c, d, and e** the full development of the mine northern aspects is shown analysed year by year:

Figure 4.3.7.c.- Northern aspect, years 2006 -2011.

Figure 4.3.7.d.- Northern aspect, years 2012 -2017.

In the **Figures 4.3.7.e to 4.3.7.j** is shown the full development of the mine southerly aspects analysed year by year (colour contours show safety factors) between 2014 and 2022.

Figure 4.3.7.e.- Northern aspect, years 2017 -2022.

Figure 4.3.7.f.- Factor of safety contours with development in 2014.

Figure 4.3.7.g.- Factor of safety contours with development in 2015.

Figure 4.3.7.h.- Factor of safety contours with development in 2016.

Figure 4.3.7.i.- Factor of safety contours with development in 2017.

Figure 4.3.7.j.- Factor of safety contours with development in 2019.

Figure 4.3.7.k.- Factor of safety contours with development in 2022.

4.3.8 INSTRUMENTATION (CALIBRATIONS AND PREDICTIONS)

Existing piezometers and inclinometers were utilised to calibrate the analysis such that further confidence could be given to the analysis results going forward to 2022. Below examples of these comparisons are shown in the **Figures 4.3.8.a** and **4.3.8.b** below.

Specifically, one of many piezometers combinations (PP06-87m and PP06-120) on the south side of the pit, as well as an inclinometer (IN10) measuring deformations towards the pit due to the excavation operations.

Figure 4.3.8.a.- Piezometers used for the calibration of the 3D model.

Figure 4.3.8.b.- Inclinometers used for the calibration of the 3D model.

There are interesting features of the two examples that were apparent in the many of the calibration iterations. Specifically:

- The majority of decompression movement in the estimation predicated only significant movement at lower levels and at intervals of around 20metres. The discrecionalised movement is likely associated with the blocky nature of the analysis due to mesh size. The predication of higher displacement at depth is coherent with constitutive model theory with additional plastic deformation components becoming highly significant at these depths.
- In the real values, movement was focused on observed weak bedding planes located, in this area of the mine, at 55m depth and 95metres depth. As expected, stress vectors tended to group around weak areas, de-stressing significantly other areas and allowing those areas to experience lesser stress-associated decompression.

- At lower depths virtually no decompression movement was observed, consistent with observations of significantly higher resistance parameters at these depths.
- Predictions of piezo-metric levels were reasonable and within the analysis expected levels of precision. In general, predications were inferior to the real values but within around 20% during each year-to-year comparison.
- Pore pressure drops occur instantaneously in most rock masses during the excavation of a pit, but a high permeability will quickly cancel this beneficial effect. More work is needed to define consolidation times in large pits, but preliminary analysis indicate that permeabilities lower than 10^{-8} m/s are required in order to maintain the pore pressure drops during the active life of an average pit.

4.3.9 PREDICATIONS IN THE FOOTWALL SHALES

For the south side of the pit, a key question was the predicted values of stability, since local instability had occurred on these slopes with further questions arising as to whether further development towards large instability was likely. The analysis indeed predicted well the observed instability, but also indicated that further development of the instability was unlikely.

In the **Figure 4.3.9.a** below the predicted instability of 20 metres can be seen.

Figure 4.3.9.a.- Predicted instability at the footwall shales.

Conclusions for the analysis on the south side shales were:

- The minimum safety factor of the marl slopes in marls was 1.35 in 2013 at the time of analysis, reducing to 1.20 in successive phases.
- At the base of the southern slope an instability was indeed predicted of about 20m height in the shales with a GSI = 40 at the base of the failure. This instability was predicted to occur in 2012 (actually 2011) and in the analysis deteriorated slowly but progressively until the end of Phase 6, fitting relatively well with the on-site observations.

- When considering the water table at high elevation, and introducing the additional tasks of the South Phase 3, it was observed that, due to the rise in the water table level in the marls, global safety factors decrease slightly.
- When considering the marl backfilling operation in the bottom of pit the instability of 2012 disappears increasing in safety factor to 1.25.
- When increasing the value of the GSI, considering the average value of the range instead of the minimum as in the previous cases, it is obtained that:
- Instabilities predicted in isolated bench of the pit are eliminated.
- The slopes in marls maintain their value of FS, although the duration at minimum is reduced.
- The slopes in Palaeozoic located on the South wall increases in FS, obtaining a value higher than 1.4, compared to minimum FS = 1.25.

Most importantly the drainage and reinjection system (SDR) used to manage water from the underlying aquifer does not appear to significantly affect safety factors in a hydro-mechanical coupled model, once marl stripping commences. The system takes the water from the aquifer (in hydrological continuity with the overlying marls) adjacent to the pit walls and re-injects at a distance away from the mining pit. Predominately implemented for environmental reasons, originally the mines technical division believed that this process would also aid stability of the mining slopes. The down drainage hole appears to indicate a drop of around 80metres; however, discussions with hydrogeological experts indicate that this is likely to be a highly localised effect due to the relatively high variations in permeability of the aquifer along it's sedimentary plane. The FLAC analysis in affect ratifies the observations in the geotechnical vibrating wire piezometers of pore pressures drops of upto 900kPa, positioned at some distance from both the pit and those drainage holes. Thus, although unlikely to be detrimental to slope stability, any real quantifiable contribution to slope safety factors can be questioned. In the author's opinion, the benefits of hydro-mechanical coupling in large open pits are likely to far exceed the beneficial effects of pumping.

In the **Figure 4.3.9.b** below the practical evidence of coupling is clearly demonstrated.

Figure 4.3.9.b.- Pit piezometers showing the effect of the drainage system and the excavation.

Moreover, the probability of a development of negative pore pressures (suction) directly adjacent to the slope will likely significantly improve the localised stability of the marls for the un-drained time period. In a mine with such impermeable soils, this un-drained period is likely to exceed the mine life by at least an order of magnitude (+100 years for a two-bench drainage path). More work is needed on this aspect; however, the hydro-mechanical coupling concept is likely to be doubly beneficial for large open pit mine stability.

4.3.10 INCORPORATION OF INFORMATION INTO VULCAN

Upon request by the operational mine team, the author incorporated the lithology, rock mass ratings (RMR) and associated Q into a block model for use in the mines visualisation programme VULCAN. Predominately this provided important information on slope conditions when various pit configurations are checked. It therefore allows for significant potential optimisation of the pit dimensions and can help reduce mining operative costs and improve geotechnical stability in the mine. In the **Figure 4.3.10.a** below is a screenshot of wireframes extrapolated from the block model undertaken by the author for each lithology:

Figure 4.3.10.a.- Screenshot of the Vulcan block model.

4.4 FLAC2D NORTHERN SLOPE ANALYSIS

The 2014 FLAC3D analysis of the northerly aspect of the indicated sufficiently high safety factors for the pit and associated dumps. However after completing the 3D study, consideration of a reduced mesh was thought appropriate, especially after a review of the predicted geometry of the complex phreatic surfaces which indicated jumps associated with the relatively large block sizes (5m x 5m) increasing to 10m x 10m at distance from the pit. These block sizes were considered insufficiently tight to simulate reliable certain phenomena such as that of the piezometric levels within the aquifer. This highlighted some limitations of this analysis technique when applied to large-scale operations such as mines that can be affected locally by even relatively small changes in water levels.

This lead further to a consideration that a FLAC2D control check on the more disfavorable geometry of the northern slopes should be undertaken with an extremely fine mesh (0.5m x 0.5m) in order to completely remove any possibility of large mesh errors. The advantage of utilising the FLAC2D finite difference programme was the manageable size of the block model. However this fine mesh was obtained at the sacrifice of any three dimensional beneficial effects (such as containing hoop stresses) in safety factor, potentially created by the concave nature of the pit at this position.

The following analysis therefore utilised the same stress-strain advantages of the FLAC3D modelling code, but focused only one section of the northerly aspect taking into account the hydro-mechanical aspects of the pit, representing a worst case profile according to the final configuration of the approved exploitation project. The profile analysis is shown in the **Figure 4.4.a** below:

Figure 4.4.a.- Cross section of the Northern slope analysed.

In the analysis, water flow was not considered, but instead pore pressures were allocated following calibration of historical data and accounting for the different construction phases of the pit. Data was provided by piezometers within the pit. In this way the mechanical response of the model can be observed considering the interstitial pressure without the realization of a pure coupled model. In spite of restricting the flow of water within the analysis, there are variations in the level of pressures resulting from the mechanical deformations of the system (volume changes without water flow which generate variations in pore pressures). It is precisely this coupling that allows a thorough investigation and close approximation of the true safety factors for the slopes, using a more realistic model of the physical coupled interactions.

The soil groups utilised in the analysis were exactly as described in the previous sections. These groups can be seen in the **Figure 4.4.b** in a section showing the maximum excavated configuration of the analysed mining development:

Figure 4.4.b.- Lithological boundaries utilised in FLAC2D analysis.

4.4.1 CALCULATION STAGES

For an adequate calibration of the numerical model, a series of phases were followed:

- Phase 1.- Initial settlement with generation of hydrostatic pressure.
- Phase 2.- Excavation to 160m depth of exploitation, setting of pore pressure corresponding to this construction phase and inclusion of the adjacent north dump development with calculation of the safety factor.
- Phase 3.- Excavation up to final levels.
- Phase 4.- Assignment of pore pressures corresponding to this construction phase and inclusion of the filling with calculation of final safety factor.

4.4.2 PORE PRESSURE GENERATION

The initial map of pore pressure were generated via the aforementioned calibration, the use of point values of pore pressure from in situ instrumentation (predominately vibrating wire piezometer measured on a weekly basis) and an ordinary kriging interpolation by the FLAC2D programme to determine all interval values. The final assign values are shown in the section shown at the **Figure 4.4.2.a**:

Figure 4.4.2.a.- Pore pressure generation.

4.4.3 CONSTITUTIVE MODELS EMPLOYED

4.4.3.1 MOHR-COULOMB MODEL

For the Mohr-Coulomb constitutive model, the FLAC program requires the following elastoplastic properties for each one of the materials:

- Volumetric module ("Bulk")
- Shear module
- Cohesion
- Friction angle
- Density
- The volumetric module ("K") and the shear modulus ("G") can be obtained from the elastic parameters
- Young ("E") and the Poisson coefficient ("v"), applying the following expressions

$$K = \frac{E}{3(1-2\nu)}$$

$$G = \frac{E}{2(1+\nu)}$$

The **Table 4.4.3.1.I** shows a summary of the geotechnical parameters used at the different levels and are involved in the calculation model:

SECCIÓN TIPO	E^m (MPa)	ρ_{ap} (t/m ³)	ν	c (MPa)	K (MPa)	G (MPa)	Φ (°)
Espaldón compactado adosado a la Berma Barrera	13,28	1,49	0,3	0,020	11,0	5,11	15,4
Escombrera Norte	45	1,89	0,3	0,062	37,5	17,31	25,5
MET-1	28	1,89	0,33	0,110	27,45	10,53	22
MET-2	60	1,97	0,33	0,150	58,82	22,56	21
MET-3	70	1,95	0,33	0,150	68,63	26,32	21
NIVEL-1	106,5	1,92	0,33	0,210	104,41	40,04	20
NIVEL-2	150	1,92	0,33	0,270	147,06	56,39	18
NIVEL-3	150	1,92	0,33	0,270	147,06	56,39	18
NIVEL-4	225	20,2	0,33	0,300	220,59	84,59	20
MARGAS-ARENOSAS	300	2,01	0,33	0,175	294,12	112,78	33
ARENAS	300	2,0	0,3	0,020	250,0	115,38	37
SUSTRATO PALEOZOICO	5000	2,1	0,25	0,5	3333,3	2000	35

Table 4.4.3.1.I.- Geotechnical parameters used in the calculation.

4.4.3.2 UBIQUITOUS-JOINT MODEL

Due to the presence of the tertiary marls with marked horizontal stratification planes, it was decided to use the constitutive model "Ubiquitous-joint" to simulate the influence of these planes of weakness as potential fault planes in the profile analysed. As mentioned previously, the displacements that are registered in the pit are a combination of the displacements that enhance the stratification planes together with the elastic deformation of the marl matrix, a product of the relaxation of tensions in the excavation of the pit.

The properties used in these stratification planes are those shown in the **Table 4.4.3.2.I**:

PLANO DE ESTRATIFICACION	c_j (MPa)	Φ_j (°)	σ_{tens} (MPa)
PLANOS	0.1	22	0.247

Table 4.4.3.2.I.- Marl Ubiquitous-Joint details.

In the FLAC implementation, the presence of planes of weakness within the Mohr-Coulomb model is taken into account, to allow the possibility of fluency either in the solid part between planes of weakness or failure through the planes themselves. In FLAC, the failure is analysed without considering the planes of weakness, as is done in the Mohr-Coulomb model and the updated tensional state is analysed in the stratification planes (37) (Itasca, 2016).

The criterion of failure in the planes of weakness is shown in the **Figure 4.4.3.2.a**:

Figure 4.4.3.2.a.- Ubiquitous model used for the joints.

The local failure envelope is defined from point A to B by the Mohr-Coulomb criterion:

$$fs = \tau - \sigma'_{22} \operatorname{tg} \phi_j + c_j$$

And from B to C by the criterion of tensional failure.

$$fs = \sigma_j^t - \sigma'_{22}$$

Where:

ϕ_j : Friction angle: Cohesion of the plane of weakness, σ_j^t : Tensile strength

Defining the tensile strength as:

$$\sigma_{j,max}^t = \frac{c_j}{\operatorname{tg} \phi_j}$$

4.4.4 RESULTS

Below are the results for the most important construction phases, together with the calculation of the safety factor in the final phase.

- Mine Phase 2: Initial construction of the North Dump.
- Mine Phase 4: Construction of the Barrier Berm and final pit depth.

4.4.4.1 MINE STAGE 2 ANALYSIS - CONSTRUCTION PHASE OF THE NORTH DUMP

During this construction phase and subsequent to the excavation of the first level of the pit (160 m depth), a numerical simulation of the construction of the north dump was undertaken:

The **Figure 4.4.4.1.a** shows maximum deformations (scale magnified with maximum indicated deformations of up to 400 mm):

Figure 4.4.4.1.a.- Maximum deformation at stage 2.

The **Figure 4.4.4.1.b** shows areas where yield was predicted (note predominate yielding along the areas where ubiquitous joints were used on the sedimentary planes):

Figure 4.4.4.1.b.- Yield predicted at stage 2.

The **Figure 4.4.4.1.c** shows areas predicted pore pressure values:

Figure 4.4.4.1.c.- Stage 2 pore pressure distributions.

4.4.4.2 MINE STAGE 4: CONSTRUCTION OF THE BARRIER BERM AND FINAL PIT DEPTH

During this construction phase and after excavation of the last part of the pit in this zone and also the construction of the northern dumps barrier berm was incorporated, with its respective initialization of pore pressures.

The **Figure 4.4.4.2.a** shows maximum deformations (scale magnified showed final maximum indicated deformations have stabilised at around 450 mm):

Figure 4.4.4.2.a.- Maximum deformations in phase 4.

The **Figure 4.4.4.2.b** shows the yield prediction (note the previous yield areas now beginning to join along the sedimentary planes and along where the more typical slip plane could be anticipated):

Figure 4.4.4.2.b.- Yield prediction in stage 4.

The **Figure 4.4.4.2.c** shows areas predicted pore pressure values in the final pit configuration for this area:

Figure 4.4.4.2.c.- Stage 4 pore pressure distributions.

4.4.5 CALCULATION OF SAFETY FACTOR

After calculating the safety factor in static condition, a value of 1.20 is obtained. The **Figure 4.4.5.a** shows the potential failure areas associated with plastification indicators.

Figure 4.4.5.a.- Calculated safety factors.

4.4.6 CONCLUSIONS FROM FLAC2D ANALYSIS OF THE NORTHERN SLOPES

From these results, the following conclusions were obtained:

- A) Influence of the North dump on the NW slope stability of the mining pit.
 - The displacements obtained in the numerical model, coincide significantly with those that were measured before 2013 (when the berm barrier dump was executed), in the control instrumentation of the pit.
 - All model planes of stratification present relevant values of shear deformation increments, coincident with the inclinometer displacement registers.
 - The mechanical response of the model induces volumetric changes in the elements that generate small variations in pore pressure. The model records non-active plastifications (generated during the process of calculation) in the areas where higher values of the pore pressure and active in the foot of the global slope are assigned.
- B) Influence of the completion of execution of the compacted shoulder to the berm barrier in the stability of the slope NW with the pit in its final deepest configuration:
 - During this construction phase, the horizontal displacements in the pit increase slightly compared to the previous phase to around 450 mm.

- Increases in shear deformation increase with respect to the previous construction phase, with significant increases in the contact between tailings, but without generating active plastifications in the model.
- The plastification indicators are maintained with respect to the previous phase.
- The minimum safety factor obtained in the pit slope is 1.20, complying with the requirements of ITC 07.3.01. As anticipated, the potential fault surface suggests a connection between the generated in the slope of the pit with the generated in the dump although this does not result in slope instability.

4.5 PHASE2 NORTHERN SLOPE REANALYSIS

In 2016 the previous FLAC2D profile was revisited by the author. Specifically, the precise geometry of the pit previously utilised in the FLAC2D analysis was imported into the phase 2 two dimensional analysis programme for additional analysis.

This enabled effectively the reduction of variables between the two analysis techniques such that the effects of two factors could be considered. Specifically:

- The review of the efficacy of a finite difference technique (FLAC2D) compared to a finite element technique (PHASE2).
- Triangle mesh as opposed to the FLAC2D rectangular mesh.
- A comparison between similar but distinct stress-strain modelling techniques, one able to implement coupling of pore pressure values with the deformation of the discretionalised elements, the second without this advanced technique.
- 3 stages comprising:
 - a. First phase establishing initial conditions
 - b. Second phase evaluating the full pit excavation.
 - c. Third phase determining the effects of the construction of the adjacent north pit to its full geotechnical extents.

4.5.1 CONSTRUCTION STAGES CONSIDERED

Apart from the initial stage, two construction stages were considered and comprised:

- The excavation of the mining phase 2 area.
- The construction of the northern dump into its final configuration.

The stage constructions are shown in the **Figure 4.5.1.a, 4.1.5.b** and **4.1.5.c**:

Figure 4.5.1.a.- Initial conditions showing mesh and initial phreatic conditions (blue line 1).

Figure 4.5.1.b.- Second stage, excavation of the pit with phreatic level 2.

Figure 4.5.1.c.- Last stage comprising the development of the north dump to its final configuration.

4.5.2 RESULTS

The strength factor used to locally determine the safety factor is shown at the Figures 4.5.2.a and 4.5.2.b for each stage.

Figure 4.5.2.a.- Strength factor for the Stage 2.

Figure 4.5.2.b.- Strength factor for the final Stage.

4.5.3 PHASE 2D ANALYSIS OBSERVATIONS AND CONCLUSIONS

- When comparing these results to those of both limit state analysis and those of the FLAC2D analysis, a number of observations can be made from which conclusions can be drawn:
- The bedding planes were provided with both peak (identical to un-structured adjacent marls) and residual resistance parameters. The above results shown that sufficient lateral strains are provoked during excavation to transit the bedding plans to residual behaviour. That residual behaviour appears to be well represented by the phase2D finite element analysis method. Safety factors are lowest along the near horizontal (3° towards the south-southwest)

bedding planes, indicating that instability, if to occur, would be a block failure geometry.

- Safety factors, without the use of the hydro-mechanical coupling concepts would be low along the one of the mine's deeper known bedding planes. The safety factor without H/C coupling is an average of $SF=1.07$ compared to 1.20 achieved via the FLAC2D finite difference technique. This is a 12% improvement in safety factor. The use of hydro-mechanical coupling enabled the mine to confirm the 28° angle for the pit represented an optimal operational angle.

5 BLAST OPTIMISATION, A GEOTECHNICAL PERSPECTIVE

Following the exposure of the aforementioned poor-quality footwall shales in the mining pit, concern was expressed by the geotechnical team with respect to use of generic blast designs when the pits various slopes comprised clearly highly variable materials. With inappropriate blasting in some areas, the mining team asked for a geotechnical perspective and recommendations with respect to trim blasting (or blasting to form the perimeter geometry) in order to reduce localised instabilities in poorer quality rocks.

The approach for a geotechnical engineer towards blast design obviously is distinct to that of a mine blast engineer. In mining blast design, focus tends to be on insitu testing characterisation of signature waves, blast wave transmission and wave velocity. These characteristics are used with standard vibration calculation formulas, from which semi-empirical or empirical rules are utilised to calculate what should be a reasonable distance from the slope. This is then combined with blasting experience and observations during previous blasts to make timing and blast configuration adjustments to ensure damage does not significantly affect the slope. In the words of one blast consultant visiting the mine, "observation is key to all blast design, ultimately, if your post-blast slope shows a 1metre over-dig, the logical process is step-back the next blast by the same amount" Thus the observation approach is shown to be key factor amongst many variables that a blast consultant needs to consider.

The geotechnical approach taken by the author at the mine, and considered innovative from a design perspective took another form, as described below:

5.1 LITHOLOGICAL CHARACTERISATION VIA MAPPING, INVESTIGATION AND LABORATORY DATA

Work had been undertaken on this aspect over many years at the mine by the author and enough information was available to provide thorough geotechnical characterisation of the various lithologies defined operationally by the mining engineers as:

- Gossan
- Rhyolites

- Massive Sulphides
- Semi-Massive Sulphides
- Shales

These were practically the same as the geotechnical teams definitions with the sub-characterisation of the mines shales into separate weak/strong categories.

The parameters initially utilised in the blast design were as provided previously in this thesis and comprised in the following **Table 5.1.I.**

Gossan:

PARÁMETROS DE LA ROCA INTACTA - GOSSAN					
γ_s (t/m ³)	σ_c (MPa)	E_c (MPa)	Coefficiente de Poisson (ν)	σ_t (MPa)	mi
2.89	13.92	20.000	0.25	1.78	7.23

Riolitas

PARÁMETROS DE LA ROCA INTACTA - RIOLITAS					
γ_s (t/m ³)	σ_c (MPa)	E_c (MPa)	Coefficiente de Poisson (ν)	σ_t (MPa)	mi
2.6	42.8	150.000	0.25	8.1	20

Weak shales

PARÁMETROS DE LA ROCA INTACTA - PIZARRAS BLANDAS					
γ_s (t/m ³)	σ_c (MPa)	E_c (MPa)	Coefficiente de Poisson (ν)	σ_t (MPa)	mi
2.4	4.4	22.500	0.2	0.88	4.775

Stronger shales

PARÁMETROS DE LA ROCA INTACTA - PIZARRAS DURAS					
γ_s (t/m ³)	σ_c (MPa)	E_c (MPa)	Coefficiente de Poisson (ν)	σ_t (MPa)	mi
2.7	12.05	64.000	0.2	2.05	6.218

Massive sulphides

Table 5.1.I.- Rockmass parameters used the blast designs.

In the **Figure 5.1.a** an example of the zone identification is shown where the existence of higher strength shales in the pit was detected.

Figure 5.1.a.- Example of zone identification.

5.2 CHARACTERISTICS FOR THE INITIAL BLAST DESIGN

The optimisation process not only comprised a review of hole explosive loadings, but all other aspects, such as

- Burden
- Spacing
- Stemming
- Hole angles and depth
- Desired bench heights
- Over drilling (the sub-drilling usually undertaken to enable post excavation floor levelling)
- Free face availability and cleanliness
- Final timing

Initially the optimisation of the aspects was checked by the author with JKBench, a specialised blasting tool which precisely simulates the blast and can indicate potential energy foci during the blast.

In the **Table 5.2.I** below is an example of the initial design information provided by the mining team and also an example of the blast simulation to ensure the blast was optimised from an energy distribution perspective.

Design Data	Presplit Row	Batter Row	Buffer Row	Production Row	
Hole Diameter (mm):	152	152	152	152	
Hole Depth (m):	10.0	10.0	10.0	10.0	
Hole Angle (deg):					
Burden (m):	3.5	4.0	4.0	1.7	
Spacing (m):	2.5	4.5	4.5	5.0	
Airdeck (m):					
Subdrill or Standoff (m):					
Stemming (m):	8.7	8.2	7.1	7.1	
Total Charge Weight (kg):	25	35	56	56	Average
Pow der Factor (kg/tonne):	0.05	0.06	0.09	0.19	0.08
Energy Factor (kjoule/tonne):	154	169	270	590	235
Comments:					

Table 5.2.I.- Initial design information provided by mining team.

In **Figure 5.2.a** the different zones of the bench blast are shown.

Figure 5.2.a.- Issue of localised overbreak and induced instabilities whilst undertaking Mine Pit Contour blasting.

The blast was simulated in order to make an estimation of the energy at the blast and to ensure a correct distribution of it. **Figure 5.2.b** shows the result of this simulation.

Figure 5.2.b.- Blast design simulation undertaken to ensure correct energy distribution.

5.3 GEOTECHNICAL BLAST OPTIMISATION METHOD

For the geotechnical optimisation a different approach was taken to that of the blast engineer, first looking at the soil parameters and how blast waves were likely to interactive with the slopes.

The author compiled the following methodology for analysing the effect of blasts on the slopes, the combination of which represents a new method for reviewing blasting in pits.

Firstly, in order to reduce the variable factors to something manageable, a number of idealised assumptions need to be made and recorded. Thus, assumptions were:

- The slope runs directly parallel to the blast free face.
- There would be sufficient attention placed on timing and downhole blast delays to ensure minimisation of resonance during the blast (i.e. the effective burden is assumed to represent the real burden).
- There would be a non-accumulation of blast energy during the blast.
- The blast zone would be unsaturated.

The analysis methodology consisted of the following elements:

- Obtain a static insitu elastic modulus via the following empirical relations from (38) Hoek et al. (1995).

$$E_m = \sqrt{\frac{\sigma_{ci}}{100}} \cdot 10^{\left(\frac{GSI-10}{40}\right)}$$

Since, for each lithology, compressive strength and GSI (Rock Mass Rating) is known, the insitu, effective elastic modulus can be calculated.

- Convert the static on-mass elastic modulus into a dynamic modulus (for subsequent usage in vibration breakage calculations).

This in reality was a bottleneck in the process. Few works have been undertaken correlating a static modulus into dynamic modulus. However (39) Galera et al. (2001) and Galera y Rodríguez (2004), to undertake laboratory works relating the static and dynamic shear moduli of rock samples. The modulus obtained by flexible dilatometer tests and the dynamic modulus obtained with a full wave sonic logging. A correlation can be seen in **Figure 5.3.a** and below and the general best-fit linear equation for this relation is also provided:

Figure 5.3.a.- Relation between static and dynamic moduli (Galera et. al 2004)

Where G_d represents the dynamic shear modulus and G_{stat} the static shear modulus.

$$G_d(\text{MPa}) = 10.95 \cdot G_{stat}(\text{MPa}) + 1449$$

The relation between elastic and shear modulus was given by:

$$G = \frac{E}{2(1 + \nu)}$$

Where ν is Poisson's ratio for the rock mass body.

The P Wave Modulus, M , is given by:

$$M = \rho V_p^2$$

And also by:

$$\frac{E(1-\nu)}{(1+\nu)(1-2\nu)}$$

Leading to the equivalency of these two terms and the ability to express propagation velocity for a wave through a rock mass.

Rock, being stronger in compression than in tension, will tend to break due to the tractional element of the propagation wave. The known formula, which can be easily derived from first principles is well known in blast mining and relates, elastic modulus (derived above), compressive resistance (from laboratory testing) and progressive wave velocity (calculated from elastic modulus and Poisson's ratio) through the material to a peak particle velocity (PPV) which will result in breakage.

$$\epsilon = \frac{PPV}{V_p} = \sigma_T/E$$

From we obtain:

$$PPV_{crit} = \frac{\sigma_T}{E} V_p$$

(40) Holmberg and Persson (H-P)'s Swedish approach to contour blasting (41) Holmberg and Persson (1978, 1979), (42) Holmberg (1982)) is then utilised to determine the dissipation of vibration (PPV) into the rock mass, via the following formula.

The H-P approach is based upon rock damage being related to peak particle velocity. The basic equation upon which the design curves are based is:

$$PPV = K Q^\alpha / R^\beta$$

Where:

K , α , β = constants, PPV = peak particle velocity, Q = charge weight, R = distance

Hence, a necessary part of the approach is to determine the required constants K , α , and β for the particular site. Normally for the purposes or ease and practicality α is set as $\beta/2$. Blast Engineers know this as *distance is worth twice charge weight* rule.

Thus knowing the characteristics of the rock for wave propagation, and the limiting peak particle velocity beyond which tensional cracking or breakage will be induced, a method has now found to obtain a safe distance, into the slope beyond which significant damage is unlikely, and before which significant damage occurs.

It is noteworthy that many blasting engineers use the following empirical criteria for assessing damage:

- Intense Fracturing= $4 \times \text{PPV}_{\text{crit}}$
- New Fractures = $1 \times \text{PPV}_{\text{crit}}$
- Worsening of Existing Fractures= $1/4 \times \text{PPV}_{\text{crit}}$

- c. From a geotechnical perspective, and for the purposes of the analysis, the point at which the critical peak particle velocity is exceeded was considered a boundary condition beyond which the damage factor D utilised in the (43) Hoek-Brown constitutive model can be consider 0. This is a reasonable assumption but would require perhaps further investigation in the future.

The damage factor D appears in the constitutive model formulas as a reductive element as follows:

$$m_b = m_i \exp\left(\frac{GSI - 100}{28 - 14D}\right)$$

$$s = \exp\left(\frac{GSI - 100}{9 - 3D}\right)$$

This was considered a practical method of applying the damage factor D system into real slope stability evaluations beyond the original general guidance provided by Bieniawski summarised as follows:

- Large production blast, confined and with little or no control $T = 2$ to $2.5 H$

- Production blast with no control but blasting to a free face $T = 1$ to $1.5 H$
- Production blast, confined but with some control, e.g. one or more buffer rows $T = 1$ to $1.2 H$
- Production blast with some control, e.g. one or more buffer rows, and blasting to a free face $T = 0.5$ to $1 H$
- Carefully controlled production blast with a free face $T = 0.3$ to $0.5 H$

The improvement assumption, and the development of a linear distribution of **D** value from **1** (at the slope surface) to **0** (the point was critical peak particle velocity into the slope is exceeded), allows slope stability analysis to be developed with relatively simple limit state software taking blast damage fully into account.

All of the aforementioned semi-empirical and empirical relations can be summed and combined. Assuming a Poisson's ratio of 0.3 for the intact rock ((44), Based on Gercek, H., 2007), the final form simplifies somewhat. Thereby the following equation can be derived for estimating the approximate point within a rock face based on charge weight, young's modulus, tensile strength and wave progression parameters derived from signature wave testings:

$$R = Q^{\frac{1}{2}} \beta \sqrt{\frac{K(11E_S + 3770)}{\sigma_T \sqrt{\frac{15E_S + 5085}{\rho}}}}$$

Where:

E_S = Static Young's Modulus

σ_T = Tensile Strength (MPa), usually around 10% of the compressive strength

Q = Charge weight of blasthole

R = Distance to Damage Factor = 0 point within rock, measured from nearest blasthole adjacent to the face.

K, β = Wave propagation parameters.

This process was duly tested on site, and importantly was capable of modelling extremely well the problems that had been observed in the mine associated with the mines footwall shales, leading the author to implement a series of viable recommendations to the mining department.

All this methodology was implemented in an MS Excel file as shown in **Figure 5.3.b**.

Pit Contouring Blast design

Initial Assumptions:

- Full slope parallel free face available
- Sufficient downhill and surface delays to prevent harmonic action (i.e. effective burden is real burden)
- Non accumulative behaviour between blast energy release
- Use of Prowall 3.0 for vibration calculations
- Spreadsheet to be utilised to calculate point in rock mass where tensile strength will be exceeded causing at least some damage and the limiting boundary in which Hoek-Brown Disturbance Factor can be considered =0
- Subsequent analysis with limit state software utilising de limitation of zones with increasing disturbances towards slope face
- Non saturated rock zone

Rock Properties

P Wave Modulus

$$\lambda f = \rho V_p^2$$

and $M = \frac{E(1-\nu)}{(1+\nu)(1-2\nu)}$

Therefore

$$\rho V_p^2 = \frac{E(1-\nu)}{(1+\nu)(1-2\nu)}$$

PARAMETROS DE LA ROCA INTACTA - PIZARRAS BLANDAS

γ (t/m ³)	σ_c (MPa)	E (MPa)	Coefficiente de Poisson (ν)	σ_u (MPa)	m_i
2.4	4.4	22,500	0.2	0.88	4.775

Em = $\sqrt{\frac{\sigma_c}{100}} \times 10^{\left(\frac{200-10}{40}\right)}$ for $\sigma_c < 100$ MPa [Hoek and Brown, 1998]

Estat = 2797.218155 Mpa
Estat = 1185.307364 Mpa
Gdyna = 14211.30783 Mpa
Gdyna = 34207.13879 Mpa

UCS = 4.4 Mpa
Ecalculation = 2.79721815 Gpa

OS = 55

Velocity of propagation of P wave = 1236.595277 metres per second

Vibration study data: MAXIM 2012

Material	Velocity [m/s]	Velocity [m/s]
Air depending on temperature	310	360
Weather soil horizon	100	500
Gravel, dry sand	100	600
Loam	300	900
Wet sand	200	1800
Clay	1200	2500
Water (depending on temperature)	1430	1500
Sandstone friable	1500	2500
Sandstone dense	1800	4000
Chalk	1800	3500
Limestone	2500	6000
Marl	2000	3500
Gypsum	4500	4500
Ice	3100	4200
Granite	4000	5700
Metamorphosed rock	1500	6000

Pro Wall Simulation

Rock Damage Particle Velocity Prediction (mm/s)

Tensile Strength (MPa): 0
Young's Modulus (GPa): 58
Longitudinal Velocity (mps): 1.256
Calculated Critical Velocity (mm/s): 10

Extension of Existing Cracks (mm/s): 2 10
Slope of New Cracks (mm/s): 10 38
Crushed Rockmass Factor: 4
Crushed Rockmass (mm/s): 38

Chart Variable Initial Charge Weight (kg): 25
Charge Step Increase (kg): 5

Initial Distance (m): 10
Distance Step Increase (m): 4

Site Factors

	K	α	β
1865	0.82	1.64	

Typical Site Factors

	K	α	β
Massive Granite	700	0.7	1.5
Jointed Granite	190	0.86	1.72
Andesite	200	0.9	1.8
Strong Sandstone	400	0.78	1.56
Strong Shale	175	1.25	2.5

	Charge Weight (kg)									
	25	30	35	40	45	50	55	60	65	70
10	98	114	129	144	159	173	187	201	215	228
14	56	66	74	83	91	100	108	116	124	131
18	37	43	49	55	61	66	71	77	82	87
22	27	31	35	40	44	48	51	55	59	63
26	20	24	27	30	33	36	39	42	45	48
30	16	19	21	24	26	29	31	33	35	38
34	13	15	17	19	21	23	25	27	29	31
38	11	13	14	16	18	19	21	23	24	26
42	9	11	12	14	15	16	18	19	20	22
46	8	9	11	12	13	14	15	16	18	19
50	7	8	9	10	11	12	13	14	15	16
54	6	7	8	9	10	11	12	13	14	14
58	5	6	7	8	9	10	10	11	12	13
62	5	6	6	7	8	9	9	10	11	11

Figure 5.3.b.- Example of calculation sheet utilised to determine the point at which D factor is considered to be 0 (in red, based on explosive charge weight).

The **Figure 5.3.c** shows an example of modelling of the mines problematic shales of the practical application of the damage factor with near perfect predications as follows:

- Stability (FOS greater than 1.0) for initial benches of up to 20metres
- Beyond 20metres instability was predicted
- Global stability unaffected (problem remains localised over 4 benches)
- Free dig rather than blasting should be utilised in the weaker shale areas to maintain stable slopes. In reality the material was inherently weak enough to be excavated by this method without inducing excessive operative delays

Figure 5.3.c.- Stability analysis considering the effect of blasting in the footwall shales.

An interesting aspect of the system was the ability of the process to effectively predict propagation velocities in shales and massive sulphides. Around the same time as this optimisation process in 2014, the author undertook signature wave testing in the pit on these two rock types with the calculated P wave velocity in shales calculated as 1718m/s versus an insitu measured value of 1758 m/s. The massive sulphides were calculated as 2,540m/s versus an insitu measured value of 2510m/s. Given the number of assumptions and empirical relations utilised during the process this is considered this to be a highly acceptable estimation. **Figures 5.3.d** and **5.3.e** show these two aspects.

Figure 5.3.d.- Layout of a signature wave test undertaken on the pits massive sulphides to determine wave propagation characteristics of the rock.

Figure 5.3.e.- Example of vibration attenuation graph with best fit line to obtain propagation wave characteristics of rock. (Massive Sulphide)

Figure 5.3.f shows an example of the slope recommendations provided to the mining operation department, considering all the stability factors, including blasting.

Geometria Talud

Blast Pattern

Figure 5.3.f.- Example of final slope recommendations provided to the mining engineers to ensure geotechnical stability.

Figures 5.3.g, 5.3.h, 5.3.i and 5.3.j show the design justifications as they were presented to the mining operations department, in order to optimise blasting in the gossan materials, mineral, volcanic tuff and weak shales.

Figure 5.3.g.- Gossan blast design given to the mining operation department.

Figure 5.3.h.- Mineral blast design

Figure 5.3.i.- Volcanic tuff blast design

Figure 5.3.j.- Weak shale blast design indicate near wall blasting should not take place

5.4 FINAL CONSIDERATIONS TO OPTIMISE BLASTING

From the process undertaken it is clear that a geotechnically orientated engineer will have a rock/soils based perspective on the effects of explosives. In some areas of the pit, the process resulted in recommendations for blast-free excavations to avoid damage. In other areas where soil lithological variation was high, trust was placed in the experience of the explosives head engineer following explanations of ideal charges in each lithology, to determine the down-hole explosives charge on a hole by hole basis following a review of the blast hole arisings.

To show the practical effect of this optimisation, **Photograph 5.4.a** below shows the weak footwall shales in the pit. It shows the aforementioned shales of the phase 1/2 area with generic blast design and little optimisation (late 2010) at higher levels. At the lower levels the same shales are observed, which, following the geotechnical evaluation of location and characteristics, were subject to an optimisation process. The results of this process are visually very clear with the easy preservation of over three benches up to 35 metres without issues, compared to the original height achievement of only 15 metres until slope failure occurred. This provides an excellent example of the results of applied geotechnical approach to blasting. It is the author's belief that the mode of analysis that resulted in such improvement represents a new approach to blast optimisation.

Photograph 5.4.a.- Results of geotechnical optimisation of blasting on the mines footwall shales. The optimisation in this zone helped recover 140million euros of additional mineral.

5.5 IMPROVEMENTS TO THE HOLMBERG NEAR FIELD EQUATION

Holmberg & Persson (H-P) in 1979 undertook contour blasting with a method that is known popularly as the Swedish technique for contour blasting. The system consists of the use of the following formula for easy derivation of wave propagation characteristic for near field blasting.

$$PPV = K Q^{\alpha} / R^{\beta}$$

In 2002 a discrepancy was discovered in the derivation of this formula, specifically during the integration process for charge weight over the blast hole length. Mining engineers continue to use this formula practically since, effectively, the formula represents a power function where the parameters can be defined pragmatically on site via signature wave testing, effectively providing a reasonable approximation.

Whilst considering this error, the author considered the possibility of effectively eliminating the distinction between near field and far field, obtaining a unified formula which simply defines the effect of a blast, at any distance from a blasthole. Such a formula would clearly need to take into account the charge length rather than as a point (far field) or an infinite length (near field). Consideration was given to the analogy of an electrical fields induced by a current flowing through a cable.

The derivation of such a mathematical formula is shown in the **Figure 5.5.a**.

Figure 5.5.a.- Theoretical description of a pulse wave at a point, distanced r from a charged column.

For the purposes of analysis, the propagation wave only (y component) will be considered. In this y direction, the peak particle velocity (PPV) contribution from the above single element would be:

$$\Delta P_v = \frac{Kdq(\cos\theta)}{r^2}$$

This is based on pure inverse square law based on surface area expansion with distance from source, $A = 4\pi r^2$, dq represents a charge energy.

$$\cos\theta = \frac{y}{r}$$

and

$$dq = A dx$$

Where dx is the charge length and A is the explosive charge coefficient,

Therefore:

$$\Delta P_v = \frac{Kdq y}{r^3}$$

The integration of this term provides:

$$P_v = \oint_{-x/2}^{x/2} \frac{KAy dx}{(x^2 + y^2)^{3/2}} KAy dx$$

Where $r^2 = x^2 + y^2$

$$Pv = 2KAy \oint_0^{x/2} \frac{dx}{y^2(x^2 + y^2)^{3/2}} KAydx$$

This is an integral form that has an explicit solution

$$Pv = \frac{KA}{y} \frac{x}{\sqrt{y^2 + (\frac{x}{2})^2}}$$

The author believes, provided that the underlying acceptance of an inverse square law drop of charge energy (reasonable energy in = energy out), the above formula can be utilised for both near and far field, since both distance y and charge stem length x is accounted for in the equation. Empirical proofing of this can be undertaken by investigating how the formula behaves in the extreme far field case $y \gg x$ and the extreme near field case $x \gg y$ as follows:

Far field Case $y \gg x$, $x \neq 0$:

$$Pv \approx \frac{KAx}{y^2}$$

Thus in the far field case the peak particle velocity should be explainable via a combination of two linear coefficients K (distance drop off constant), A (explosive characteristic constant) and the square of the distance.

Near field case $x \gg y \neq 0$:

$$Pv \approx \frac{2KA}{y}$$

Interestingly, as one approaches the drillhole and the charge stem, the expression transits from an inverse square power function to an inverse linear function, without in principle the requirement for any input stem length (since in effect at the close point to the blast the quadratic root relation would tend to unity). This suggests that at points between near and far field (where the majority of blast optimisation should be considered) an intermediate power function such as the original Holmberg-Persson equation is a reasonable approximation (most signature blast analyses determine the β factor to fall between 1 and 2). However it is the author's opinion that, with the advancements recently made in blast simulation programmes, this approximation may not be required and the exact distance from blast to the point of interest with respect to vibrations, can be directly incorporated into the summation of each blast column such that calculations of contributions from each blast hole can be near explicit in form.

6 GEOTECHNICAL MONITORING SYSTEMS

The main part this thesis focuses on designs and their improvements over the years. For the mine design is a corner stone of ground control safety. However from an operational perspective, once confidence in design is achieved, focus should be placed on ensuring the slopes behave as anticipated and that personnel, working in the mining pit, are protected, via early warning, from pit slope instabilities.

Since the very first stages of the pre-stripping excavation of the pit an intensive monitoring of the slopes was performed. The instrumentation implemented was based on two types of measurements: deformations (using inclinometers and topographical surveys) and pore pressure (using wire-vibrating piezometers). **Figure 6.a** shows the general plan of the pit, were all the monitoring systems used can be observed.

6.1 DISPLACEMENT VELOCITY ALERT LEVELS

The established permitted deformation velocities in either inclinometers or surface monitoring (topographical markers or prisms) and associated actions set forth in the mines vigilance plan are shown in **Table 6.1.I.** ((45) Zavodni, Z.M., Broadbent, C.D et al.,)

<i>Level</i>	<i>Displacement velocity</i>	<i>Likely cause</i>	<i>Action</i>
Attention	1mm / day	Long term decompression from pore-pressure dissipation	Visual inspection, increase local inclinometer measurement frequency.
Alert	3mm / day	Short term decompression from rapid principle stress rotation (localized pit stripping) / potential instability.	As above plus recheck design with local geometrics. Consider likely progress pattern and the necessity of cessation of activities in zone.
Emergency	10mm / day	Instability.	As above plus cessation of activities in zone.

Table 6.1.I.- Monitoring Alert Levels.

6.2 IN-GROUND MONITORING

Over the mining facility (including both pit and dumps) a matrix of piezometers and inclinometers have been installed. The number of instruments has increased proportionally with the mine development since the early stages to the current contingency of 72 piezometers and over 50 inclinometers, of which 15 are associated with the mining pit.

Figure 6.a.- Plan view of the mine showing conventional inclinometers (light blue), piezometers (blue), topographical markers (red) and dynamic inclinometers orange.

6.2.1 PIEZOMETERS

Piezometric levels are monitored to ensure levels do not exceed those where the slope stability safety factors are considered to be adversely affected. The **Figure 6.2.1.a** below shows an example of the on-going monitoring of the pore-pressure around the pit.

Figure 6.2.1.a.- Piezometers used in the pit for monitoring pore pressure at the marls.

The information is also utilized to monitor effects of shutdown in sectors of the SDR system for maintenance purposes. Finally, the development of pore pressures due to increasing mining activities for over 10 years as described in section 3.0 provides invaluable information to check and calibrate the estimated pore pressures generated by the hydro-coupling process utilized in the mine's aforementioned FLAC3D and FLAC2D analysis.

Notable in these piezometers is that dissipation response is near immediate for those influenced by and in the proximity of the mining pit excavation.

The register of piezometers indicate only minor influences on the piezometric levels due to the use of the system of drainage and reinjection (SDR), with drops in levels of around 10metres, probably due to the vertical discontinuities in the marls stratification. In contrast, the change in levels as a result of the de-compressional dilation of the pit was observed to be between 50m to 60m representing a considerable drop in levels, thereby confirming the real value of hydro-mechanical coupling in pit wall safety.

6.2.2 INCLINOMETERS

As well as the installation of nearly 15 conventional inclinometers, in key areas of the pit, more technically advanced real time inclinometers were installed. These systems enable continuous information to be gathered as well as displacement exceedance alerts to be sent directly to the geotechnical engineers' mobile telephone (a functional tool also available for the topographical prism system). The **Figures 6.2.2.a, 6.2.2.b** and **6.2.2.c** below show the cumulative displacements detected in one of the mines real time inclinometer.

Figure 6.2.2.a.- Conventional Inclinometer monitoring at Las Cruces Pit.

Figure 6.2.2.b.- Dynamic Real Time Pit Inclinometer profile.

Figure 6.2.2.c.- Sensor deformation tracking over a month.

In 2015 the scope of the system was widened to include 4 additional dynamic real time inclinometers providing information and protection along the two longer (north and south) aspects of the mining pit, where decompression effects were likely to be most influential.

These real time inclinometers are more costly than conventional system but provide much more detailed information with respect to the evolution of movement and reaction of the substrata in the exact moments of a mining activity in that particular area, for example deepening of an adjacent mineral excavation. Given the extra costs involved, the most optimal form of implementation is to begin with a localised analysis of the slope in question and the commencement of monitoring with conventional inclinometer systems initially to understand where instability is most likely to occur. Once this is firmly established, the zones with higher risk of instability can then be categorically targeted with real time sensors. This has been the installation philosophy utilised at CLC.

These systems, traditionally utilised in smaller civil engineering projects for depths of upto 20 or 30 metres, were modified on a joint project with the supplier to extend down to 120 metres depth. The technology in this current did not exist for implementation and the project was undertaken as a possible loss leader, (i.e. accepting the possibility of non-success). As such it is likely that Las Cruces is unique in the aspect of the innovative usage real time inclinometers to provide vigilances on slopes.

6.3 TOPOGRAPHICAL MONITORING WITH A ROBOTIC TOTAL STATION

Monitoring of the slope surface movement was achieved via the installation of a GEOMOSS topographical system. **Photo 6.3.a** shows the aspect of the robotic monitoring station TS50 installed at CLC.

Photo 6.3.a.- Robotic Monitoring station TS50 installed at Las Cruces.

An early version of the system was installed during the first stages of pre-stripping enabling the movements to be catalogued and compared with determined velocity alert levels. These levels have been already shown in the table above while **Figure 6.3.b** shows a typical example of the topographical prism survey in the perimeter of the pit as well as inside it, upon the operational ramp. The system is capable of sending alerts if individual or groups of inclinometer manifest movements above established levels.

Over the years, the system has been overhauled to increase reliability and measurement frequency (as high as one reading every 20 minutes of well over 100 prisms).

Figure 6.3.b.- Topographical prisms around and inside the pit.

6.3.1 DIRECT SLOPE SCANNING

In 2014 two, further advanced, systems were purchased, a scan from one of which is showed in the **Photo 6.3.1.a**. The first of these was placed on the south side of phase 3 to reduce distances to prisms in newer phases and thereby increase precision. The most recently installed system the (Leica MS50) represented a significant jump in the technology, enabling direct laser measurement of the slope surfaces (instead of installed prisms). This system, near analogous to a more expensive radar system, was installed within the Palaeozoic to monitor slopes during an extraction of unanticipated high-grade mineral in contact with problematic shales. The purchase of the system was easily justified, enabling the operations team to guarantee personnel safety whilst extracting substantial additional mineral.

Photo 6.3.1.a.- Direct slope scanning implemented in 2014 to extract high grade mineral from the footwall shale area.

6.4 VISUAL MONITORING AND CARTOGRAPHY

Since the very first excavations in the mine were undertaken, geotechnical mapping has tracked the mining progress and development, registering the conditions of the slopes in order to predetermine the possibility of instability, rock falls and also to provide information on deformations such that pre-remedial actions can be taken or modifications can be implemented to ensure stability in the lower benches if required.

The observational information collected and demonstrated in the previous sections prove unequivocally the importance of registering slope structures and behaviour for subsequent analysis. In 2015, during the blast optimisation process, the need for pit blast reconciliation was highlighted. Logically this register for reconciliation would be best undertaken independently by the geotechnical division, since mapping had to take place anyway.

It was decided to implement Stacey's reconciliation system on site, incorporating the method into the on-going geotechnical cartography.

6.4.1 BLAST RECONCILIATION METHODOLOGY

The method fundamentally comprises in the combined evaluation of two independent components: Face Excavation Condition and Blast Damage Rating:

Figure 6.4.1.a and **6.4.1.b** below are direct extracts from the compendium: Guideline to Open Pit Design 2009 (46) Stacey & Read. 2008)

FACE EXCAVATION CONDITION	
GOOD (1)	- Achieved design easily, clean excavation, good final face condition, teeth marks barrels exposed (where applic.)
FAIR (2)	- Achieved design but with difficulty, irregular teeth marks, few barrels, rough face condition (some hazards exist)
POOR (3)	- Did not achieve design, poor face condition (some hazards remain), difficulty excavating
BAD (4)	- Did not achieve design, very difficult excavation, very poor face condition, residual high-risk hazards associated with face

Figure 6.4.1.a.- Face excavation condition extract from Stacey and Read (2008).

Figure 6.4.1.b.- Blast Damage Rating (Stacey and Read, 2008).

Conformity with design is checked simply by comparing the achieved geometry against the design geometry. An example is given in the **Figure 6.4.1.c** below. At Las Cruces this comparison in geometry was achieved via the use of precise 3D photography to achieve an exact geometry representation which could subsequently be imported in AutoCAD and the mines visual design programme VULCAN to obtain a comparative profile equal to the example below.

Figure 6.4.1.c.- Example of a comparison between the design and the real geometry of a given bench.

Finally these two factors are combined to achieve a rating. This is then mapped onto the graph shown at the **Figure 6.4.1.d** to give the mining team an indication of the blast performance such that remediation work can be undertaken or due consideration can be taken on future benches.

Figure 6.4.1.d.- Evaluation of the blast performance (Stacey and Read, 2008)

6.4.2 IMPLEMENTATION AT LAS CRUCES

The previously described methodology has been implemented at Las Cruces Pit. **Figure 6.4.2.a** shows a full page example of geotechnical mapping undertaken in 2015 shows the implementation of this reconciliation method at the mine.

Figure 6.4.2.a.- Example of a blast performance evaluation at Las Cruces pit.

7 THESIS CONCLUSIONS

Las Cruces mine holds a unique identity with respect to its complex geological sequencing and the widely varying geotechnical characteristics of those sequences. It is one of the deepest mines in soft rocks in the world with the current pit configuration taking it to near 250 metres depth. This thesis represents a compendium of the engineering improvements undertaken at Las Cruces mine over an 8-year period to ensure continued stability of the pit. This timeline effectively took the mine from its early first phase development, through to its current configuration with the final phase under development.

Improvements over the years in rock mass characterisation, mapping, design, operative techniques and vigilance monitoring have been registered. A number of innovative techniques and improvements are also highlighted.

Overall and for each element following the conclusions that can be reasonably drawn are described as follows:

7.1 ROCK MASS CHARACTERISATION

From the original eight geotechnical boreholes utilised for initial marl classification, vast improvements were developed in a pit angle optimisation process. From a material characterisation perspective, this project developed, from virtually no significant information on Palaeozoic host rock variability to full characterisation of the underlying anticipated material over the entire project development in 10 metre depth steps (corresponding with bench geometry) and 6 section profiles, providing information to a maximum depth of -245 metres depth.

Downhole structural mapping was undertaken to obtain further information on structural orientation and the presents of faults with depth.

7.2 INSTABILITIES AND REMEDIATION

Over these years, a number of bench or inter-bench level instabilities have been observed comprising, topping, sliding and block failure as well as more typical rotational instabilities. Bolting and mesh was successfully used in order to regain operational control of some of these areas. A draped meshing technique was also used, not to support, but instead to control rock and large block movements at the mines lower levels, thereby in some area accepting the lack of stability, but maintaining safety via an in depth control of the soil movements.

Innovative techniques were used in one case to determine the probability of future large instabilities, specifically applying the system of Silva and Lambe (1998) which relates safety factor and construction quality to a potential yearly failure probability, based on a number of worldwide cases, and ratified by many geotechnical engineers. This system aided the non-geotechnically trained mining directive to better understand associated risks and agree appropriate remediation activities.

7.3 DESIGN ASPECTS

The on-going improvement in geotechnical design techniques utilised at the mine resulted in a number of significant advances, comprising:

- Incorporation of observed structures (principally dis-favourable orientated bedding planes) into the existing limit state analysis.
- Refined limit state design incorporating the concept of upper and much stronger lower marl units into the analysis, shaving 3° off the optimal design angles for the lower marls.
- Use of FLAC3D to determine three-dimensional effects and the incorporation of the concept of hydro-mechanical coupling mechanism. This process highlighted the importance of the effect of H/C mechanisms on pore pressure dissipation and hence slope safety factors, compared to the less effects of the mines system of drainage and reinjection (SDR).
- Use of the FLAC2D process to analyse a least favourable geometry with an ultra-fine mesh to improve modelling precision whilst maintaining the coupling of pore-pressures and soil mass bulk moduli.
- Final check on the FLAC2D utilising a finite element programme PHASE2 (2D) without hydro-mechanical coupling, demonstrating the fundamental requirement of the coupling concept to better model large open pits.

7.4 GEOTECHNICAL BLAST OPTIMISATION

A geotechnical perspective to blast optimisation was developed, combining a series of semi-empirical relations to obtain wave transmissibility characteristics of the rock with limit state analysis and modification of material parameters based on the Bieniewski damage factor.

A summary equation was also obtained for the estimation of distance from the slope's nearest adjacent blasthole into the rock to the point at which the D factor can reasonably be considered null. This takes into account young's modulus and tensile strength of the rock, as well as the rock's signature wave propagation characteristics.

Finally, improvements to the Holmberg-Persson (H-P) wave propagation theory were proposed enabling the unification of near and far field behaviour.

These systems were implemented at the mine to significantly improve bench to bench preservation post blast and post excavation.

7.5 VIGILANCE SYSTEMS

A number of innovative systems for vigilance were implemented at the mine comprising:

Dynamic real time inclinometers, which can provide real time cloud, based information on pit stability and advise the geotechnical engineer of excessive movement or deformation velocity.

Topographical robot station monitoring systems, were developed to direct slope scanning system and utilised for intense vigilance during the last mineral extraction of the mine's phase 2 area, allowing the successful recovery of over 150 million euros worth of additional mineral.

This was a considerable achievement given that area in question was particularly susceptible to instabilities due to the presence of the mine's footwall shales.

3D photography for use in geotechnical cartography. This enabled improved bench blasting reconciliation and also safety operations, eliminating the requirement for engineers to approach potentially unstable slopes or areas subject to rock falls during evaluation.

7.6 GENERAL OBSERVATIONS

There is no panacea for maintaining stable slopes in an operating mine such as Las Cruces. Any activity can involve potential risks and, as such, the mine operates to reduce the risk to as low as reasonably achievable, combined with a global safety objective of zero harm. To achieve this objective, over the years, more robust and refined designs have been implemented to determine where the predominate risks in the mine lie, and to what extent. With the identification of instability susceptible slopes, critical monitoring systems were installed to manage the risk appropriately. In some circumstances, ore extraction has been justifiable only by increased local analysis, instrumentation, and monitoring.

It is not possible, either economically or technically, to implement all of these measures at the commencement of mining operations. Any pit design and monitoring plan needs to develop along with the mine's evolution, with increasing refinement in preparation for the mine's most critical periods (usually, but not always, when the mine is at its deepest configuration) and finally winding down, changing from new investment to on-going maintenance later in the mine life, when preparing for closure and restoration.

A fundamental objective of this thesis was to formalise the above developments, led by the author at an operative open pit mine in potentially problematic soils and rocks. Progression of these concepts can further be developed by the incorporation of good geotechnically orientated blast design and reconciliation, pit optimisation and implementation of cutting edge vigilance systems from the outset of a large pit excavation. Further to this, more advantage could be achieved by implementation, in a programmed manner, of these systems and methodologies following the successful incorporation of such improvements at initial mine development planning stage.

In the author's opinion, detailed consideration should be given to the effects of blasting at initial design stage. The concepts expounded in this thesis allow a rigorous treatment of blast damage. In general, the global geotechnical community in mining is now beginning to recognise formally the higher frequency and importance of planar or structurally dominated failure mechanisms in mining rather than traditional rotational failure mechanisms. The use of the blast damage parameter and its affectation of geotechnical resistance parameters should be recognised as an exceptionally useful tool in this context.

Vigilance will always remain as the last, but one of the most important defences with respect to the large pit stability (good design being the first). Automated topographical monitoring has improved dramatically over the last ten years and these systems should therefore be incorporated directly from the onset of the excavation. In large-scale excavations, the regional pre-excavation stress condition of the rock and the subsequent rotation of principal stresses during excavation should be of paramount concern for the geotechnical design engineer. Laboratory testing and an excellent register of displacements values provides invaluable insight into the peak-residual situation of a rock mass for subsequent analysis and determination of the

stress strain condition of a rock mass adjacent to excavated slopes, fundamental for superior design optimisations.

8 FUTURE INVESTIGATIONS

Future development should be undertaken on the following aspects:

- Further investigation of characteristic information on the PQ shale group, consolidating information from neighbouring mines to improve the database set. These are present in many IPB developed mineralisations in the hanging wall and, more dis-favourably, in the mines' footwall.
- Full development of total real time monitoring arrays for future mining projects, the viability of which has now been well established at Las Cruces mine. Future mining companies should be able to incorporate these innovative systems as standard given the knowledge gained through initial usage in a flagship mine such as Las Cruces.
- Implementation of the improvements of the Holmberg-Persson wave propagation equation as an option with blast simulation software. This will enable improved predication of wave propagation into slopes and predictions/avoidance of damage via modifications at design simulation stage.
- Improvements in the understanding and evidencing of the effect of large open pit stripping activities on overall principal stress reduction and rotation and the resulting effects on both pore pressure dissipation. Determination should be made of the as yet un-evidenced but logically anticipated development of negative pore pressure generation in low permeability marls close to the excavation slope faces.

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10 APPENDIX

The following is an extract of the mine's High Consequence Protocol (HCP), compiled and implemented at Las Cruces Mine by the Author.

PG-SYS- 020	CONTROL DEL TERRENO	Rev. 04 (21/10/2013)
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Process owner	Signature	Date
Stephen Cooper		

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Document Revision History				
Nº of revision	Date	Page number	Paragraph number	References of changes made
00	27/12/2010			Versión Inicial
01	01/04/2012	14/15		Incorporación de medidas de estación total en procedimiento
02	10/06/2012			Cambios de referencia de acuerdo con el nuevo sistema de Responsabilidad Corporativa
03	22/04/2013			Revisión de Geotécnico Responsable: a. mejorar la definición de trabajos que requieren permiso b. incluir plano de zonas de riesgos
04	21/10/2013			Cambio de formato

10.1 OBJETIVO

El motivo de este procedimiento es para asegurarnos que los trabajos que se llevan a cabo en nuestras instalaciones mineras están implementados en un ámbito seguro, en referencia a la estabilidad del terreno. El objetivo es el de construir una serie de requerimientos de control del terreno, para salvaguardar al personal y maquinaria contra desprendimientos del terreno, que puedan causar heridos o muertos.

10.2 ALCANCE

Este procedimiento contempla a todo el personal (personal de CLC y contratistas) que trabajen en las instalaciones mineras de Cobre Las Cruces, S.A. siendo aplicable a todo el personal cuyo trabajo puede desarrollarse en diversas zonas que pueden ser susceptibles de desprendimientos del terreno. Esto incluye la provisión de:

- a. El diseño general, construcción, y mantenimiento de cualquier talud y pistas (en corta y escombreras).
- b. Los trabajos que tienen dependencia en la capacidad del suelo para soportar cargas (por ejemplo, vehículos o equipos).
- c. El terreno necesita ser estable a los trabajos que se estén realizando sobre él.
- d. Zanjas y excavaciones
- e. Construcción, mantenimiento y limpieza de balsas.
- f. Construcción civil en general, incluidas infraestructuras.

10.3 PROCEDIMIENTO PARA LA ESTABILIDAD GENERAL

El Control del terreno en la mina depende de dos conceptos, el primero es para asegurar la estabilidad general de la corta y de las escombreras, el segundo pretende el control del terreno necesario para dar apoyo a trabajos locales.

El principal motivo para garantizar la estabilidad global de la mina se logra a través de la adhesión (durante la implementación) de una serie de recomendaciones del diseño acordado y autorizado por las autoridades.

Plan de vigilancia geotécnico:

El plan de vigilancia geotécnico da un método para evaluar la estabilidad global de la corta minera y las escombreras, basado en evaluaciones geotécnicas y rendimientos históricos. El plan de vigilancia geotécnico establece la forma de medidas usadas para la auscultación de la corta y las escombreras. Esto se logra con auscultación continua de hitos, lecturas regulares (normalmente semanal) de la instrumentación e inspecciones visuales regularmente, por parte del ingeniero geotécnico responsable.

Proyectos constructivos:

Para cada etapa de la operación de rellenos se lleva a cabo un proyecto constructivo, realizado por consultoras de geotecnia independientes y aprobados (externamente por la Junta de Andalucía e internamente por el departamento de minas).

Para mantener una adecuada estabilidad global para la mina, el director de mina, el ingeniero geotécnico y el director general necesitan trabajar juntos para asegurar esto:

Las operaciones de corte y relleno se llevarán a cabo de acuerdo con los proyectos constructivos y el estudio de explotación (aprobado por la Junta).

Cualquier cambio o modificación de los ángulos de inclinación recomendados para los taludes, geometría o métodos de compactación deberán ser consultados a el ingeniero geotécnico y el director de minas para futuras consideraciones

Señales de inestabilidad geotécnica visuales (grietas, deslizamientos,...) o instrumental (topográfica, inclinométrica, piezométrica,...) serán estudiadas por el ingeniero geotécnico responsable.

El personal de mina recibirá una formación básica para reconocer situaciones problemáticas y actuar apropiadamente.

Medidas adicionales:

Además de los informes de los consultores, los planes de construcción y requisitos de la Junta, se llevara a cabo un proceso formal de inspección visual:

Inspección visual por el Área de Geotecnia

La mina contempla una superficie superior a los 25km² aproximadamente, y por lo tanto, debido a los limitados recursos las inspecciones visuales se llevarán a cabo como mínimo una vez al mes. Conocidas zonas problemáticas o críticas (la corta, acopios y tailing) se les dará un tratamiento como zonas prioritarias. Cualquier área problemática será fotografiada y se realizará una cartografía geológica con los problemas detectados, reportándolos al director facultativo del departamento de mina y el director general, si procede. Se llevará a cabo una cartografía de las margas para destacar las zonas de peligro y las áreas que requieren permiso explícito del ingeniero geotécnico responsable antes de la entrada del personal y el plan de trabajo. Con el avance de la corta en el mineral, se llevará a cabo cartografía geológica en intervalos mensuales, manteniendo un plan actualizado de los condiciones de la corta minera.

Turno de Inspecciones por el Supervisor de Mina

Se llevarán a cabo inspecciones diarias de las rampas de acceso a la corta y además del estado general de los taludes tanto en marga como en roca. Cualquier cambio notable a la condición de las pistas o de apariencia adversas deben notificarse al departamento de geotecnia. Las obras que se localicen en las zonas afectadas se detendrán hasta que las condiciones sean evaluadas, y si se requieren remediarlas. Se necesita utilizar un registro para las observaciones realizadas.

Fallas típicas y efectos observables:

ZONA DE LA MINA	TIPO DE SUELO	ROTURA TIPICA	EFFECTOS OBSERVABLES	TIPO DE AUSCULTACIÓN
CORTA	Margas con alteración	Deslizamientos de bloques/ rotura circular Desmoronamiento de suelo inducido por el agua	Grietas en cabecera, movimiento progresivo en el pie del talud.	Inspecciones visuales, medidas en hitos e instrumentación.
	Margas sin alteración	Bloques/deslizamiento translacional / rotura circular Desmoronamiento de suelo inducido por el agua	Grietas en cabecera, movimiento progresivo en el pie del talud.	Inspecciones visuales, medidas en hitos e instrumentación.
	Pizarras con alteración	Deslizamiento de bloques inducidos por el agua	Movimiento progresivo en el pie del talud.	Inspecciones visuales
	Pizarras sin alteración con juntas	Vuelcos	Apertura de juntas en el pie del talud.	Inspecciones visuales
	Pizarras sin alteración	Rotura en cuña	Apertura de las juntas en cabecera o las pendientes	Inspecciones visuales
	Rocas volcánicas con textura vesicular	Desmoronamiento	Caída del suelo/rocas hasta el pie del talud	Inspecciones visuales
	Gossan meteorizado	Rotura en cuña	Apertura de las juntas en cabecera o las pendientes	Inspecciones visuales
	Mineral en zona de cementación (con fracturas frecuentes y muy claras)	Rotura en cuña /deslizamientos de bloques	Apertura de las juntas en cabecera o las pendientes	Inspecciones visuales
	Mineral en zona de cementación (Mineral competente)	Rotura en cuña /deslizamientos de bloques	Apertura de las juntas en cabecera o las pendientes	Inspecciones visuales
Escombreras				
	Margas removidas	Deslizamientos de bloques/ rotura circular Desmoronamiento de suelo inducido por el agua	Grietas en cabecera, movimiento progresivo en el pie del talud.	Inspecciones visuales, medidas en hitos e instrumentación.
	Tailings	Desmoronamiento y caídas	Grietas en cabecera, movimiento progresivo en el pie del talud.	Inspecciones visuales
	Mineral	Caída de bloques	Grietas en cabecera, movimiento progresivo en el pie del talud.	Inspecciones visuales
	Arenas del acuífero	Desmoronamiento y caídas de bloques	Caída del suelo/rocas hasta la pie del taludes	Inspecciones visuales

POTENCIAL MECANISMOS DE FALLAS EN ROCA

WEDGE FAILURE

TOPPLING

PLANER FAILURE

POTENCIAL MECANISMOS DE FALLAS EN SUELO

Translational slip

Circular slip

Non-circular slip

Compound slip

10.4 PROCEDIMIENTO DE TRABAJO ASOCIADO AL CONTROL DEL TERRENO

Como se estipula en el punto 2 (alcance), todas las obras que dependan directamente del estado del suelo debe ser considerado en estos procedimientos. Antes de emprender cualquier obra en las instalaciones mineras de CLC, hay que cumplir con un requisito mínimo:

(Requisitos mínimos)

Todas las áreas de trabajo (Zona de Planta y Mina): Ejemplo - carga admisible del suelo para elevación de cargas o movimientos de vehículos, la colocación de la maquinaria, tránsito de personal.

Acción - Evaluación de la zona de trabajo por el supervisor del mismo, para determinar si hay zonas que puedan representar un riesgo geotécnico para el personal, equipo o producción. La base de conocimientos para esta evaluación se efectuará con el proceso de formación que se describe en la sección 7.0. Ha de tenerse en cuenta que si la formación básica no se ha proporcionado al supervisor, esta evaluación de la zona de trabajo se deberá pasar a un supervisor secundario de la mina o ingeniero responsable y la formación del supervisor de trabajo deben llevarse a cabo lo antes posible.

Conocimiento de zonas de riesgo geotécnico: Ejemplos: (trabajo en las excavaciones, trabajo en los bancos, trabajo en las bermas):

Acción - Un permiso de trabajo geotécnico para la zona en concreto, debe llevarse a cabo por el ingeniero responsable, asegurando que la zona de trabajo en cuestión tiene una inspección visual. En la expedición del permiso de trabajo los riesgos también deben explicarse a los supervisores de trabajo, y medidas de mitigación a tener en cuenta. Esta información debería ser explicada por el supervisor de trabajo directamente a los trabajadores en forma de una pre reunión de trabajo.

Proyectos: Cualquier obra civil, construcción de infraestructuras, balsas, presas, zanjas o excavaciones.

Acción - Todas las obras de este tipo, que requieren control del terreno, de cualquier manera deberá ser discutida, revisados y aprobados por el ingeniero geotécnico responsable, de la mina. Ningún trabajo se llevará a cabo sin esta aprobación. La solución a los problemas geotécnicos surgidos durante la ejecución de las obras serán examinados por el ingeniero geotécnico responsable antes de su realización.

Ejemplos de operaciones en las que se debe considerar la estabilidad del terreno local:

1. La competencia de los sub-estratos debajo de los estabilizadores de una grúa para sostener la carga durante una operación de elevación
2. La competencia de sub-estratos por debajo de los vehículos en los alrededores de las zonas de obras.
3. Personal que trabaja en las proximidades de las bermas donde una inestabilidad local podría ocurrir.
4. El personal que trabaja en las proximidades de las excavaciones.
5. Personal que entra en los bancos de roca de la corta después de las voladuras.

Documentación clave para utilizar.

Con los tipos de trabajos que ya se han tratado, al menos uno de esta serie de documentación adicional es requerido, en concreto:

1. Permiso de Trabajo.
2. Análisis Preventivo de Trabajo.
3. Orden de Trabajo.
4. Evaluación de riesgos geotécnicos.

De esta manera antes que cualquier operación se lleve a cabo, es necesario cumplir dos condiciones. La primera es comprobar que las condiciones generales de estabilidad global se mantienen. El segundo es considerar la situación local y las tareas a realizar. Prácticamente para satisfacer esta segunda condición, uno o más de los siguientes documentos deben ser utilizados.

Permiso Especial de Trabajo: Trabajo en Borde de Bancos

Los permisos de trabajo son necesarios para las actividades específicas que identifique el gerente de operaciones en la mina, (Permisos de Trabajo). Ningún trabajo debe llevarse a cabo en la zona minera sin su permiso expreso. Este permiso de trabajo será compilado entre el administrador de las operaciones mineras y el ingeniero geotécnico responsable. En el permiso de trabajo se verán reflejados las medidas y detalles que se deben tomar para mitigar los riesgos geotécnicos derivados de las obras a ejecutar. Una vez el permiso se haya completado estas medidas se tratarán directamente con los trabajadores afectados y los trabajadores firmarán la hoja de información para asegurarse de que han recibido y comprendido la información.

El permiso de trabajo es un requisito para cualquier obra donde sea necesario el acceso a:

1. Bermas - En cualquier caso.
2. Borde de taludes – Depende de la geometría, pero se requerirá permiso cuando el trabajo se desarrolle a un mínimo de 5 metros del borde del talud o en la zona entre el borde del talud y el punto de corte de una línea de 45 grados desde el pie del talud.
3. El pie de un talud - Depende de la geometría, pero se requerirá permiso cuando el trabajo se desarrolle a un mínimo de 5 metros del pie del talud o en la zona entre el pie del talud y el punto de corte de una línea de 45 grados desde borde del talud.

Las operaciones de control de leyes, preparando voladuras y la toma de medidas topográficas cerca de bordes, en acuerdo con la definición anterior, requieren de un permiso específico para sus trabajos.

1. Formación del HCP de control de terreno para supervisores de estas personas, dando información expresamente sobre posibles zonas de instabilidades geotecnicas y como estas instabilidades pueden reconocerse en campo.
2. Una chala de seguridad durante la revisión, asegurándose que las medidas preventivas exigidas en el permiso de trabajo quedan totalmente entendidas.

En general:

Análisis preventivo del trabajo:

En el análisis preventivo del trabajo se provee un método para identificar los riesgos asociados a un trabajo, para evaluar los riesgos que están presentes y qué medidas se necesitan incorporar al análisis preventivo del trabajo y subsecuentemente para el trabajo. El análisis preventivo del trabajo debe evaluar todos los riesgos (incluidos los geotécnicos) asociados con el trabajo a desarrollar.

Orden de trabajo

La orden de trabajo provee información a contratistas o personal afectados por riesgos que se localicen en la misma área geográfica. La orden de trabajo puede ser requerida por el supervisor del trabajo y podría ser incorporada información adicional dentro análisis preventivo del trabajo.

Evaluación de riesgos geotécnicos (Adjunto)

La evaluación de riesgos geotécnicos se basa en el sistema de conocimiento del impacto de INMET y elementos aplicables deben ser incorporados al análisis preventivo del trabajo y el permiso de trabajo. Una copia de esta evaluación estará adjunta a este documento. Cuando se lleve a cabo una evaluación de riesgos geotécnicos en una zona potencialmente peligrosa se necesita como mínimo una inspección visual de esta zona por parte de un ingeniero geotécnico responsable, idealmente conjuntamente con el supervisor del trabajo, para describir el tipo de trabajo y los requerimientos geotécnicos. El ingeniero geotécnico debe considerar los riesgos asociados con los trabajos, proveyendo medidas de control o mitigación y evaluar los riesgos residuales. Las medidas de control y los riesgos residuales se necesitan describir directamente al supervisor del trabajo, se necesita llegar a un acuerdo sobre las medidas implementadas para el control y finalmente se obtendrá una conformidad del acuerdo. Los riesgos deberán ser recordados dentro del análisis preventivo del trabajo.

10.5 PERSONAL RESPONSIBLE

- TRABAJADORES AFECTADOS:

Cualquier trabajador, contratista, o visita de la mina.

- a. Que accede a la corta o escombreras
- b. Que utilice vehículos o maquinas
- c. Que trabaje cerca de bermas
- d. Que trabaje cerca de excavación o zanjas
- e. Que trabaje cerca de los bancos de suelo o roca.

10.6 MÉTODOS DE FORMACIÓN

- INGENIERO GEOTÉCNICO RESPONSABLE:

Es el responsable de asegurar la estabilidad general de la corta minera y las escombreras por razones de seguridad, y que lleva a cabo estas obras en acuerdo con las recomendaciones de los consultores, los requisitos de la Junta y de INMET. Cualquier análisis preventivo del trabajo que identifique riesgos geotécnicos debe ser como mínimo evaluado y aprobado del ingeniero geotécnico responsable. El permiso de trabajo incorporará medidas de control y mitigación que deben ser aprobadas del ingeniero geotécnico responsable. Para trabajos en zonas donde se necesite control del terreno como uno riesgo primario el ingeniero debe visitarlas y compilar el análisis preventivo de trabajo directamente.

- SUPERVISOR DE MINA:

El supervisor de mina debe verificar que se lleve a cabo el trabajo de acuerdo con los requisitos del procedimiento y cualquier otro procedimiento que puede ser aplicable (por ejemplo trabajos con altura, trabajos en Espacio Confinado). El supervisor de mina tiene derecho de parar el trabajo en cualquier momento. El supervisor de mina es la principal persona que tiene responsabilidad sobre el trabajo y para asegurarse que cualquier deficiencia en seguridad se corrige durante la ejecución del trabajo.

- SUPERVISOR DEL TRABAJO:

El supervisor del trabajo se asegurará que la ejecución del trabajo se lleve a cabo de acuerdo con todos los procedimientos. Está Incluido en esto la responsabilidad en la ejecución de cualquier medida correctiva dada por el supervisor de mina.

Conocimientos geotécnicos deben ser proporcionados a varias áreas:

Departamentos de Mina y Agua

Mejora de conocimientos de conceptos geotécnicos en el departamento a ingenieros de minas (dado que sus conocimientos son básicos deben ampliarlos debido a que su trabajo directo está íntimamente relacionado con la estabilidad del terreno). Esto puede ser transmitido por métodos distintos:

- Discusiones informales
- Reuniones informales
- Reuniones formales
- Boletines de correo electrónico
- Memorándums internos

El contenido de tales reuniones puede ser variable y puede dirigirse a otros temas no relacionados con la geotecnia. Una comunicación típica se puede dar por el acontecimiento de un accidente o incidente reciente por ejemplo si hay un riesgo donde es necesario hacer medidas de control.

Toda persona que acceda a la mina (contratistas, consultores, personal en general o visitantes)

Proveer de una formación básica en geotecnia para todo el nuevo personal que accede a la mina como parte del curso básico de seguridad. El objetivo no es dar conocimiento de

especialista o avanzado, sino información básica con respeto a los peligros asociados con la corta y las escombreras, las excavaciones y los trabajos próximos a los taludes. Con las charlas de cinco minutos se trasladarán los conocimientos siendo este un vehículo de transmisión para las personas que ya están trabajando en la mina.

10.7 REVISIÓN DE PROCESOS

Una revisión de este procedimiento debería ser llevada a cabo al menos una vez cada año. Además las revisiones pueden realizarse esporádicamente o siguiendo observaciones in situ que den un motivo para la modificación o incorporación a esta documentación (o su misma documentación asociada). El personal que puede realizar una modificación son:

- Director General
- Director Facultativo
- Director de Mina
- Ingeniero Geotécnico responsable
- Jefe de Seguridad

Cualquier cambio debe ser notificado verbalmente al resto del personal y llegando a un acuerdo. El Director general tiene el poder de desautorizar cualquier cambio. Cualquier revisión debe ser registrada en la página de control de cambios del documento con la fecha de tales cambios.

10.8 PLAN DE EMERGENCIA

Evaluación de la estabilidad

Con respecto a la vigilancia general de la corta y las escombreras CLC trabaja de acuerdo con un Plan de Vigilancia que se usa para evaluar el riesgo potencial de un desprendimiento del terreno, como resultado del desplazamiento o cambios que muestren la instrumentación instalada:

- **Estado de Atención:** el rango de velocidades de las deformaciones es superior a 1 mm/día y se producen desviaciones de presión de poro de entre el 10 al 15% con respecto al previsto.
- **Estado de Alerta:** el rango de velocidades de las deformaciones es superior a 3 mm/día y se producen desviaciones de entre el 15 al 25% para la presión de poro estimada.
- **Estado de Alarma:** el rango de velocidades de las deformaciones es superior a 1 cm/día y se producen desviaciones del 25% para la presión de poro estimada en el modelo predictivo. En el caso de activarse este estado de alarma se requerirán acciones inmediatas de estabilización.

Para cada uno de estos estados, hay un protocolo de actuación:

- **Nivel 1 (atención):**
 - a. Incremento de la frecuencia de las lecturas, retrocediéndose un paso en la periodicidad de medida de la instrumentación en la zona donde se ha producido el incremento en la velocidad de deformación o de la presión de poro. (Esto es si se está midiendo mensualmente se pasará a medir semanalmente y así sucesivamente)
 - b. Inspección visual de detalle de la zona donde se ha generado la atención al objeto de detectar grietas, signos de movimiento, etc.
 - c. Comprobación con el Departamento de Mina de cuáles han sido las actividades desarrolladas en el área de influencia de la zona donde se ha detectado la atención.
- **Nivel 2 (alerta):**
 - a. Incremento de la frecuencia de lecturas, de acuerdo a los criterios señalados en el estado de atención con una periodicidad máxima de dos medidas por semana.
 - b. Inspección visual, e acuerdo a los mismos criterios ya descritos.
 - c. Comprobación con el Departamento de Mina de cuáles han sido las actividades desarrolladas en el área de influencia de la zona donde se ha detectado la atención.
 - d. Evaluación de la situación a partir de la magnitud de la presión de poro y de la velocidad de evolución de la deformación de acuerdo con el modelo predictivo.
 - e. Revisión de los procesos de cálculo.
- **Nivel 3 (alarma o emergencia):**
 - a. Análisis de la situación, basada en las cinco actuaciones descritas para el estado de alerta. En este caso la periodicidad de las medidas y de la inspección visual será diaria.
 - b. Colocación de Instrumentación complementaria si es preciso.
 - c. Realimentación de los modelos de cálculo empleados para la evaluación de la estabilidad de la estructura y del modelo predictivo de las deformaciones y presiones de poro.
 - d. Paralización, si procede, de la actividad en el área donde se ha detectado la alarma.
 - e. Introducción, si procede, de medidas correctoras o de refuerzo.

Estación Total.

La mina tiene implementado un sistema de monitoreo dinámico que comprende un sistema de estación total que controla hasta 70 prismas alrededor de la corta. Este sistema proporciona información muy útil acerca del movimiento en los taludes. Los movimientos excesivos en los taludes después de una excavación deben estar asociados a planos profundos subyacentes que pueden implicar un problema de seguridad para los trabajadores en las proximidades a dichos taludes o debajo de los mismos. Así la estación total proporciona, con sus 20 medidas al día, información durante las 24 horas, que es muy útil tanto para el ingeniero geotécnico como para el supervisor de mina de guardia. Además este sistema está dotado de una alerta vía teléfono móvil, que mantiene informado al supervisor de mina de guardia y al ingeniero geotécnico en cualquier momento.

Posibles variaciones de la temperatura que puedan darse a lo largo del día, pueden dar lugar a falsos niveles de alerta. Por este motivo, la estación total, tiene un sistema que toma una media de las medidas en periodos de 15 días.

Cuando la diferencia entre dos medidas individuales es de 40 mm o superior para un mismo prisma, es enviado un mensaje de forma automática, a los teléfonos del ingeniero geotécnico y el supervisor de mina de guardia.

Funcionamiento del sistema de estación total:

El sistema es comprobado diariamente por el equipo de geotecnia para asegurar que no hay ningún movimiento que exceda de los niveles de alerta.

Si un mensaje de alerta es recibido por el supervisor de guardia y el ingeniero geotécnico, estas son las actuaciones a tomar:

1. Comprobación del mensaje por parte del supervisor y el ingeniero geotécnico.
2. Comunicación entre el supervisor y el ingeniero geotécnico.
3. Evaluación de la gravedad del riesgo, llevada a cabo por el ingeniero geotécnico.
4. Si el ingeniero geotécnico considera que el riesgo es suficiente como para que los trabajadores se encuentren en peligro, el supervisor de mina debe seguir el plan de emergencia estándar de la mina, atendiendo a las modificaciones estipuladas en este procedimiento.

Precauciones Adicionales

La conformidad debe estar relacionada con el Plan General de Emergencia. Sin embargo, en situaciones correspondientes a un desprendimiento de tierras y donde haya gente que necesita ser rescatada, el plan general de emergencia debería ser implementado con medidas adicionales. Como mínimo las siguientes etapas se deben de llevar a cabo antes y durante una situación de rescate:

- Las empresas locales deben estar incluidas en un acuerdo con CLC, poniendo a nuestra disposición los servicios apropiados que se necesite en cada momento en una situación dada de desprendimiento del terreno. Esto puede ser:
 - Consultores Geotécnicos,
 - Servicios de investigación (perforación, catas),
 - Grúas,
 - Cabestrantes,
 - Especialistas en trabajos verticales.
- Se llevará a cabo una evaluación geotécnica de las aéreas colindantes al desprendimiento del terreno, para garantizar la estabilidad y la seguridad del resto de los trabajadores.
- Se llevará a cabo una evaluación o comprensión de los requisitos del terreno adyacente al desprendimiento para asegurar que las medidas de Socorro elegidas pueden realizarse con seguridad desde el punto de vista geotécnico. Por ejemplo, si se utiliza una grúa el terreno donde se soporta esta debe de garantizar su correcto asentamiento para realizar las labores.
- Los resultados de tales evaluaciones deberían ser compartidos inmediatamente con los servicios de rescate, los supervisores y personal implicado.

10.9 DOCUMENTACIÓN RELEVANTE

- Diagrama de procedimiento de control del terreno.
- Permiso de trabajo (Permiso de trabajo en bordes de bancos).
- Permiso de trabajo (Permiso de trabajo para zanjas y excavaciones).
- •Análisis Preventivo del trabajo.
- Ejemplo de Check list para inspección visual.
- Ejemplo de Check list para inspecciones de turnos.
- Resumen documentación geotecnia.
- Planificación de inspecciones visuales.
- Plano de zonas de más alto riesgo en la mina.
- Plano general de zonas de revisiones y frecuencia (modificable según requisitos).

10.10ABREVIATURAS

- OT: Orden de Trabajo
- PTBB: Permiso de trabajo en Borde de Banco

10.11 GLOSARIO DE TÉRMINOS

- Geotécnica
- Ground control
- Mecánicos del suelo/roca
- Geología

10.12 CONTROL DEL TERRENO (GUÍA DE BOLSILLO)

La finalidad del control del terreno es asegurar la estabilidad general en corta, escombreras y viales, zanjas y excavaciones, construcción, mantenimiento y limpieza de balsas y construcción civil, incluidas infraestructuras.

Nunca se debe comenzar a trabajar sin haber chequeado la zona de trabajo (zanja, talud, etc.).

No se realizarán trabajos a menos de 2 m de bordes o más de 1.8 m de profundidad sin usar el arnés de seguridad.

Nunca sentarme de espalda al talud o sentarme en él.

Limitar la presencia de vehículos y cargas al borde de excavaciones, zanjas, taludes.

El permiso de zanja y excavaciones y el permiso de trabajos al borde de banco, deberá ser revisado por el departamento de geotécnica de CLC.

Sistemas de control geotécnico

- Inspecciones visuales.
- Medidas de desplazamientos horizontales en el interior del terreno mediante inclinómetros.
- Medida de desplazamientos horizontales en superficie mediante topografía.

Avisar en caso de:

- Grietas o desplazamientos en el perímetro de corta y escombreras
- Grietas o desplazamientos en los taludes.
- Grietas o desplazamientos en el fondo de corta.

10.13 ZANJAS Y EXCAVACIONES (GUÍA DE BOLSILLO)

Permiso de excavación

Se necesita permiso de realización de zanjas y excavaciones (fuera de corta) para aquellas cuya profundidad sea igual o mayor a 1 m. (salvo que las excavaciones se realicen en zonas ya construidas o con conocimiento de existencia de algún tipo de conducción enterrada, en cuyo caso será necesario a partir de 0.2 m).

Personal autorizado

Todo el personal que trabaje en el interior de las zanjas debe conocer los riesgos a los que está sometido a través de un APT.

Inspección diaria

Las zanjas y excavaciones con acceso de personal deben ser inspeccionadas a diario. Se accederá a las excavaciones por medio de escaleras o rampas habilitadas.

Condiciones de trabajo

Cuando se acceda mediante una escalera, estará anclada en el borde superior de la zanja y apoyada sobre una superficie sólida de reparto de cargas.

El entorno de cualquier zanja se balizará y señalizará adecuadamente, mediante tochos clavados y unidos con malla naranja, situada a una distancia mínima de 2 metros del borde, salvo zonas habituales de paso, en las que se colocará barandilla con tramo intermedio o valla peatonal y rodapié. Dispondrán de al menos una puerta de acceso. El encargado de la excavación velará por que se mantenga el balizamiento de la misma.

Distancias de seguridad

No se realizarán trabajos a menos de 2 m del borde de las zanjas o en excavaciones de más de 1.8 m de profundidad sin usar el arnés de seguridad.

Cuando en los trabajos de excavación se empleen máquinas, camiones, etc. que supongan una sobrecarga en los bordes, así como la existencia de tráfico rodado que transmita vibraciones que puedan dar lugar a desprendimientos de tierras en los taludes, se adoptarán las medidas oportunas de refuerzo de entibaciones y balizamiento y señalización de las diferentes zonas (en taludes con $\alpha \geq 60^\circ$, deberá cumplir $d \geq h/2$ o $d \geq h$ en terrenos arenosos, siendo h la profundidad y α el ángulo de inclinación).

Atmosfera interior

Siempre que exista riesgo de formación de atmósferas peligrosas en el interior de una excavación, sea por los productos o equipos utilizados, por emanaciones del propio terreno o de las instalaciones descubiertas, o por proximidad de equipos o instalaciones, la excavación será considerada como espacio confinado.

Se evitará la introducción de productos peligrosos en el interior de las excavaciones.

No se trabajará a distintas alturas en la misma vertical.

Si al excavar surgiera cualquier anomalía no prevista, se paralizarán los trabajos y se comunicará de inmediato a la supervisión técnica.

Cuando la excavación requiera entibado, se montará siempre de arriba abajo, y su desmontaje se realizará en sentido contrario.

Material extraído

Los productos de la excavación que no hayan de retirarse de inmediato, así como los materiales que hayan de acopiarse, se apilarán a la distancia suficiente del borde de la excavación para que no supongan una sobrecarga que pueda dar lugar a desprendimientos o corrimientos de tierras en los taludes, al menos un metro.

Iluminación

Si los trabajos requieren iluminación se efectuará mediante torretas aisladas con toma a tierra, en las que se instalarán proyectores de intemperie, alimentados a través de un cuadro eléctrico general de obra.

Humedad

En las zanjas que tengan agua/humedad, los trabajos que requieren iluminación interior se efectuarán a 24 V. Los portátiles estarán provistos de rejilla protectora y de carcasa-mango aislados eléctricamente.

En régimen de lluvias y encharcamientos de las zanjas es imprescindible la revisión minuciosa y detallada de las zanjas y herramientas eléctricas, antes de reanudar los trabajos. Se efectuará el achique inmediato de las aguas que afloran en el interior de las zanjas para evitar que se altere la estabilidad de los taludes.