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Soils and Rocks publishes papers in English in the broad fields of Geotechnical Engineering, Engineering Geology and Geo-environmental Engineering. The Journal is published in April, August and December. Subscription price is US$ 90.00 per year. The journal, with the name "Solos e Rochas", was first published in 1978 by the Graduate School of Engineering-Federal University of Rio de Janeiro (COPPE-UFRJ). In 1980 it became the official magazine of the Brazilian Association for Soil Mechanics and Geotechnical Engineering (ABMS), acquiring the national character that had been the intention of its founders. In 1986 it also became the official Journal of the Brazilian Association for Engineering Geology and the Environment (ABGE) and in 1999 became the Latin American Geotechnical Journal, following the support of Latin-American representatives gathered for the Pan-American Conference of Guadalajara (1996). In 2007 the journal acquired the status of an international journal under the name of Soils and Rocks, published by the Brazilian Association for Soil Mechanics and Geotechnical Engineering (ABMS), Brazilian Association for Engineering Geology and the Environment (ABGE) and Portuguese Society of Geotechnical Engineering (SPG).

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Letter from the ISRM President

As President of an international society - the International Society for Rock Mechanics - it is a great pleasure for me to wish every success to “Soils and Rocks” now that, with Volume 30, the journal has moved to the new international, all-English language format, *i.e.* from “Solos e Rochas” to “Soils and Rocks”. As readers of previous volumes will have been aware, for the first 29 years the articles in this Journal have been almost exclusively in Portuguese for readers in Brazil and Portugal. However, the language change to English will ensure that the Journal can now build up an international readership base across the world.

Reflecting this ambition, the Editorial Board now contains the names of many international authorities in the soil and rock subjects. Moreover, the articles in Volume 30, Number 1, are already representative of the high quality genre of papers which the Editors are targeting.

Based on my experience of editing an international rock mechanics journal, let me predict some of the future directions that the scientific papers in the now fully international “Soil and Rocks” journal will take:

- the number of submitted manuscripts will steadily increase as the years go by;
- the mean number of authors per paper will increase (a trend across many journals indicating the steady increase in collaborative work);
- there will be an increase in the number of manuscripts originating in Asia;
- there will be a shift to website submission and review operations; and
- special subject-thematic issues will become popular.

All these predicted developments will be beneficial to the journal.

Finally, I am really pleased to support an international journal with the title “Soils and Rocks”. Too often, in teaching and research there is strong emphasis on one specific geo-engineering aspect, when engineers building structures on or beneath the Earth’s surface have to deal with both soils and rocks, sometimes simultaneously. Needless to say, it is not then sufficient to be knowledgeable in one subject area and not the other. Through the wider subject coverage in this journal, the recipients will be encouraged to absorb information across the soil-rock spectrum. From my perspective, I for one am looking forward to learning more about soils!

Very best wishes to “Soils and Rocks” for a successful future.

John A Hudson *FREng*
Emeritus Professor, Imperial College, UK
President ISRM, 2007-2011

Articles

A Brief History of the Development of the Hoek-Brown Failure Criterion

Evert Hoek, Paul Marinos

Abstract. The Hoek-Brown failure criterion was developed in the late 1970s to provide input for the design of underground excavations. Bieniawski's RMR was originally used to link the criterion to engineering geology input from the field but a more specific classification system called the Geological Strength Index (GSI) was introduced in 1995. Both the Hoek Brown criterion and the GSI classification have evolved and continue to evolve to meet new applications and to deal with unusual conditions encountered by users.

Key words: Hoek-Brown failure criterion, Bieniawski's RMR, geological strength index, rock strength.

1. Introduction

The original Hoek-Brown failure criterion was developed during the preparation of the book *Underground Excavations in Rock* by E. Hoek and E.T. Brown, published in 1980. The criterion was required in order to provide input information for the design of underground excavations. Since no suitable methods for estimating rock mass strength appeared to be available at that time, the efforts were focussed on developing a dimensionless equation that could be scaled in relation to geological information. The original Hoek-Brown equation was neither new nor unique - an identical equation had been used for describing the failure of concrete as early as 1936.

The significant contribution that Hoek and Brown made was to link the equation to geological observations. It was recognised very early in the development of the criterion that it would have no practical value unless the parameters could be estimated from simple geological observations in the field. The idea of developing a 'classification' for this specific purpose was discussed but, since Bieniawski's RMR had been published in 1974 and had gained popularity with the rock mechanics community, it was decided to use this as the basic vehicle for geological input.

By 1995 it had become increasingly obvious that Bieniawski's RMR is difficult to apply to very poor quality rock masses and it was felt that a system based more heavily on fundamental geological observations and less on 'numbers' was needed. This resulted in the development of the Geological Strength Index, GSI, which continues to evolve as the principal vehicle for geological data input for the Hoek-Brown criterion.

2. Historical Development

1980: Hoek E. & Brown E.T. (1980) Underground Excavations in Rock. Institution of Mining and Metallurgy, London, 527 pp.

Hoek, E. & Brown, E.T. (1980) Empirical strength criterion for rock masses. J. Geotech. Engng Div., 106:GT9, p. 1013-1035.

The original criterion was conceived for use under the confined conditions surrounding underground excavations (see Table 1 - Appendix at the end of this paper). The data upon which some of the original relationships had been based came from tests on rock mass samples from the Bougainville open pit copper mine in Papua New Guinea. The rock mass here is very strong andesite (uniaxial compressive strength about 270 MPa) with numerous clean, rough, unfilled joints. One of the most important sets of data was from a series of triaxial tests carried out by Professor John Jaeger at the Australian National University in Canberra. These tests were on 150 mm diameter samples of heavily jointed andesite recovered by triple-tube diamond drilling from one of the exploration adits at Bougainville.

The original criterion, with its bias towards hard rock, was based upon the assumption that rock mass failure is controlled by translation and rotation of individual rock pieces, separated by numerous joint surfaces. Failure of the intact rock was assumed to play no significant role in the overall failure process and it was assumed that the joint pattern was 'chaotic' so that there are no preferred failure directions and the rock mass can be treated as isotropic.

1983: Hoek, E. (1983) Strength of jointed rock masses, 23rd. Rankine Lecture. Géotechnique 33:3, p. 187-223.

One of the issues that had been troublesome throughout the development of the criterion has been the relationship between Hoek-Brown criterion, with the non-linear parameters m and s, and the Mohr-Coulomb criterion, with the parameters c and ϕ. At that time, practically all software for soil and rock mechanics was written in terms of the Mohr-Coulomb criterion and it was necessary to define the relationship between m and s and c and ϕ in order to allow

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the criterion to be used for to provide input for this software.

An exact theoretical solution to this problem (for the original Hoek-Brown criterion) was developed by Dr John W. Bray at the Imperial College of Science and Technology and this solution was first published in the 1983 Rankine lecture. This publication also expanded on some of the concepts published by Hoek and Brown in 1980 and it represents the most comprehensive discussion on the original Hoek-Brown criterion.

1988: Hoek E & Brown E.T. (1988) The Hoek-Brown failure criterion - A 1988 update. Proc. 15th Canadian Rock Mech. Symp., Curran, J.H. (ed), University of Toronto, Toronto, pp. 31-38.

By 1988 the criterion was being widely used for a variety of rock engineering problems, including slope stability analyses. As pointed out earlier, the criterion was originally developed for the confined conditions surrounding underground excavations and it was recognised that it gave optimistic results for shallow failures in slopes. Consequently, in 1998, the idea of *undisturbed* and *disturbed* masses was introduced to provide a method for downgrading the properties for near surface rock masses.

This paper also defined a method of using Bieniawski's 1974 RMR classification for estimating the input parameters. In order to avoid double counting the effects of groundwater (an effective stress parameter in numerical analysis) and joint orientation (specific input for structural analysis), it was suggested that the rating for groundwater should always be set at 10 (completely dry) and the rating for joint orientation should always be set to zero (very favourable). Note that these ratings need to be adjusted in later versions of Bieniawski's RMR, for example, use 15 for ground water in the 1989 version.

1990: Hoek, E. (1990) Estimating Mohr-Coulomb friction and cohesion values from the Hoek-Brown failure criterion. Intnl. J. Rock Mech. & Mining Sci. & Geomechanics Abstracts, 12:3, p. 227-229.

This technical note addressed the on-going debate on the relationship between the Hoek-Brown and the Mohr-Coulomb criterion. Three different practical situations were described and it was demonstrated how Bray's solution could be applied in each case.

1992: Hoek, E.; Wood, D. & Shah, S. (1992) A modified Hoek-Brown criterion for jointed rock masses. Proc. Rock Characterization, Symp. Int. Soc. Rock Mech.: Eurock '92, Hudson, J. (ed), pp. 209-213.

The use of the Hoek-Brown criterion had now become widespread and, because of the lack of suitable alternatives, it was now being used on very poor quality rock masses. These rock masses differ significantly from the tightly interlocked hard rock mass model used in the development of the original criterion. In particular it was felt that the finite tensile strength predicted by the original Hoek-Brown criterion was too optimistic and that it needed to be revised. Based upon work carried out by Dr Sandip Shah for his Ph.D thesis at the University of Toronto, a modified criterion was proposed. This criterion contains a new parameter *a* that provides the means for changing the curvature of the failure envelope, particularly in the very low normal stress range. Basically, the modified Hoek-Brown criterion forces the failure envelope to produce zero tensile strength.

1994: Hoek, E. (1994) Strength of rock and rock masses. ISRM News Journal, 2:2, p. 4-16.

1995: Hoek, E.; Kaiser, P.K. & Bawden. W.F. (1995) Support of Underground Excavations in Hard Rock. Balkema, Rotterdam, 210 pp.

It soon became evident that the modified criterion was too conservative when used for better quality rock masses and a 'generalised' failure criterion was proposed in these two publications. This generalised criterion incorporated both the original and the modified criteria with a 'switch' at an RMR value of approximately 25. Hence, for excellent to fair quality rock masses, the original Hoek-Brown criterion is used while, for poor and extremely poor rock masses, the modified criterion (published in 1992) with zero tensile strength is used.

These publications (which are practically identical) also introduced the concept of the Geological Strength Index (GSI, see Appendix) as a replacement for Bieniawski's RMR. It had become increasingly obvious that Bieniawski's RMR is difficult to apply to very poor quality rock masses and also that the relationship between RMR and *m* and *s* is no longer linear in these very low ranges. It was also felt that a system based more heavily on fundamental geological observations and less on 'numbers' was needed.

The idea of *undisturbed* and *disturbed* rock masses was dropped and it was left to the user to decide which GSI value best described the various rock types exposed on a site. The original *disturbed* parameters were derived by simply reducing the strength by one row in the classification table. It was felt that this was too arbitrary and it was decided that it would be preferable to allow the user to decide what sort of disturbance is involved and to allow users to make their own judgement on how much to reduce the GSI value to account for the strength loss.

1997: Hoek, E. & Brown, E.T. (1997) Practical estimates or rock mass strength. Intnl. J. Rock Mech. & Mining Sci. & Geomechanics Abstracts, 34:8, p. 1165-1186.

This was the most comprehensive paper published to date and it incorporated all of the refinements described above. In addition, a new method for estimating the equivalent Mohr Coulomb cohesion and friction angle was introduced. In this method the Hoek-Brown criterion is used to generate a series of values relating axial strength to confining pressure (or shear strength to normal stress) and these are treated as the results of a hypothetical large scale in situ triaxial or shear test. A linear regression method is used to find the average slope and intercept and these are then transformed into a cohesive strength *c* and a friction angle ϕ.

The most important aspect of this curve fitting process is to decide upon the stress range over which the hypothetical *in situ* 'tests' should be carried out. This was determined experimentally by carrying out a large number of comparative theoretical studies in which the results of both surface and underground excavation stability analyses, using both the Hoek-Brown and Mohr-Coulomb parameters, were compared.

1998: Hoek, E.; Marinos, P. & Benissi, M. (1998) Applicability of the Geological Strength Index (GSI) classification for very weak and sheared rock masses. The case of the Athens Schist Formation. Bull. Engg. Geol. Env., 57:2, p. 151-160.

This paper extends the range of the Geological Strength Index (GSI) down to 5 to include extremely poor quality schistose rock masses such as the 'schist' encountered in the excavations for the Athens Metro and the graphitic phyllites encountered in some of the tunnels in Venezuela. This extension to GSI is based largely on the work of Paul Marinos and Maria Benissi on the Athens Metro. Note that there were now 2 GSI charts. The first of these, for better quality rock masses published in 1994 and the new chart for very poor quality rock masses published in this paper.

2000: Hoek, E. & Marinos, P. (2000) Predicting Tunnel Squeezing. Tunnels and Tunnelling International. Part 1, 32/11, p. 45-51, Part 2, 32/12, p. 33-36.

This paper introduced an important application of the Hoek-Brown criterion in the prediction of conditions for tunnel squeezing, utilising a critical strain concept proposed by Sakurai in 1983.

2000: Marinos, P. & Hoek, E. (2000) From the geological to the rock mass model: Driving the Egnatia Highway through difficult geological conditions, Northern Greece. Proc. 10th International Conference of Italian National Council of Geologists, Rome, pp. 325-334.

This paper puts more geology into the Hoek-Brown failure criterion than that which has been available previously. In particular, the properties of very weak rocks are addressed in detail for the first time. There is no change in the mathematical interpretation of the criterion in these papers.

2000: Hoek, E. & Karzulovic, A. (2000) Rock-mass properties for surface mines. Hustralid, W.A.; McCarter M.K. & van Zyl, D.J.A. (eds), Slope Stability in Surface Mining. Society for Mining, Metallurgical and Exploration, Littleton, pp. 59-70.

This paper repeats most of the material contained in Hoek and Brown, 1997, but adds a discussion on blast damage.

2000: Marinos, P & Hoek, E. (2000) GSI: A geologically friendly tool for rock mass strength estimation. Proc. International Conference on Geotechnical & Geological Engineering, GeoEng-2000, Melbourne, Technomic publ., pp. 1422-1442.

2001: Marinos, P. & Hoek, E. (2001) Estimating the geotechnical properties of heterogeneous rock masses such as flysch. Bulletin of the Engineering Geology & the Environment, v. 60, p. 85-92.

These papers do not add anything significant to the fundamental concepts of the Hoek-Brown criterion but they demonstrate how to choose appropriate ranges of GSI for different rock mass types. In particular, the 2001 paper on flysch discussed difficult weak and tectonically disturbed materials on the basis of the authors' experience in dealing with these rocks in major projects in northern Greece.

2002: Hoek, E.; Carranza-Torres, C. & Corkum, B. (2002) Hoek-Brown criterion - 2002 ed. Proc. NARMS-TAC Conference, Toronto, v. 1, pp. 267-273.

This paper represents a major re-examination of the entire Hoek-Brown criterion and includes new derivations of the relationships between *m, s, a* and *GSI*. A new parameter D is introduced to deal with blast damage. The relationships between the Mohr-Coulomb and the Hoek-Brown criteria are examined for slopes and for underground excavations and a set of equations linking the two are presented. The final relationships were derived by comparing hundreds of tunnel and slope stability analyses in which both the Hoek-Brown and the Mohr-Coulomb criteria were used and the best match was found by iteration. A Windows based program called *RocLab* was developed to include all of these new derivations and this program can be downloaded (free). A copy of the paper is included with the download.

2004: Chandler R.J.; De Freitas M.H. & P.G. Marinos (2004) Geotechnical characterisation of soils and rocks: A geological perspective. Key-

note paper in Advances in Geotechnical Engineering, The Skempton Conference, Thomas Telford, ICE, London, v. 1, pp. 67-102.

A brief contribution on the Geological Strength Index within a more general paper on engineering geology of soils and rock.

2005: Marinos, V.; Marinos, P. & Hoek, E. (2005) The geological Strength index: applications and limitations. Bull. Eng. Geol. Environ., 64, p. 55-65.

A discussion on the range of application and the limitations of GSI. General guidelines for the use of GSI are given.

2005: Hoek, E.; Marinos, P. & Marinos, V. (2005) Characterization and engineering properties of tectonically undisturbed but lithologically varied sedimentary rock masses, International Journal of Rock Mechanics and Mining Sciences, v. 42:2, p. 277-285.

A significant paper in which a new GSI chart for molassic rock masses is introduced. Molasse consists of a series of tectonically undisturbed sediments of sandstones, conglomerates, siltstones and marls, produced by the erosion of mountain ranges after the final phase of an orogeny. They behave as continuous rock masses when they are confined at depth and, even if lithologically heterogeneous, the bedding planes do not appear as clearly defined discontinuity surfaces. The paper discusses the difference between these rock masses and the flysch type rocks which have been severely disturbed by orogenic processes.

2006: Marinos, P.; Hoek, E. & Marinos, V. (2006) Variability of the engineering properties of rock masses quantified by the geological strength index: the case of ophiolites with special emphasis on tunnelling. Bull. Eng. Geol. Env., v. 65:2, p. 129-142.

The paper presents the geological model in which the ophiolitic complexes develop, their various petrographic types and their tectonic deformation, mainly due to overthrusts. The structure of the various rock masses include all types from massive strong to sheared weak, while the conditions of discontinuities are in most cases fair to poor or very poor due to the fact that they are affected by serpentinisation and shearing. Serpentinisation also reduces the initial intact rock strength. Associated pillow lavas, and tectonic mélanges are also characterised. A GSI chart for ophiolitic rock masses is presented.

2006: Hoek, E & Diederichs, M.S. (2006) Empirical estimation of rock mass modulus. International Journal of Rock Mechanics and Mining Sciences, v. 43, p. 203-215.

While not directly related to the Hoek-Brown failure criterion, the deformation modulus of a rock mass is an important input parameter in any analysis of rock mass behaviour that includes deformations. Field tests to determine this parameter directly are time consuming, expensive and the reliability of the results of these tests is sometimes questionable. Consequently, several authors have proposed empirical relationships for estimating the value of rock mass deformation modulus on the basis of classification schemes. These relationships are reviewed and their limitations are discussed. Based on data from a large number of in situ measurements from China and Taiwan, a new relationship between the deformation modulus and GSI is proposed. The properties of the intact rock as well as the effects of disturbance due to blast damage and/or stress relaxation are also included in this new relationship. The program RocLab has been updated (January 2007) to incorporate the method proposed by Hoek and Diederichs for estimating the rock mass deformation modulus.

3. Conclusions and Recommendations

The historical development of the Hoek-Brown failure criterion and the associated Geological Strength Index (GSI) has been presented. Evolution of both will continue in order to accommodate processes such as brittle spalling and anisotropy and to include a wider range of rock types. Great care is taken to retain the fundamental components of the system and to avoid changing "ratings" so that users need not go back to question or redo previous applications.

A fundamental assumption of the Hoek-Brown criterion is that the rock mass to which it is being applied is *homogeneous* and *isotropic*. It should *not be applied* to the analysis of structurally controlled failures in cases such as hard rock masses where the discontinuity spacing is similar to the size of the tunnel or slope being analysed and where the failure processes are clearly *anisotropic*.

The criterion also assumes that there is contact between intact rock pieces within the rock masses and it is these contacts that give rise to the highly non-linear characteristics of the criterion at low confining stresses. Where no such contact exists, for example when the components of the rock mass are predominantly soil or clay as in the case of fault gouges, the use of the Mohr-Coulomb criterion, with cohesion and friction parameters determined from laboratory tests, is more appropriate.

One of the greatest sources of error in applying the Hoek-Brown criterion is a misunderstanding of the contribution of the intact rock strength σ_{ci}, the role of which is almost equivalent to GSI in the evaluation of the rock mass properties. It is very common to see geologists confusing the intact strength with the rock mass strength and this results in significant under-estimates of the final rock mass strength. The authors encourage users to pay particular attention to the intact strength of the rock pieces that make up the rock mass. Measurement of the intact strength, using direct compression tests or point load tests where appropriate, should be considered.

Many engineers have requested that the GSI classification should be made more numerical so that in input parameters can be "measured" from core or rock exposures rather than estimated from geological observations. The authors and their colleagues have taken note of these request and work on providing quantitative methods for estimating GSI is ongoing, without however neglecting the basic geologic logic expressed by the GSI chart.

Many geotechnical software packages can now accommodate the Hoek-Brown criterion directly and, where this is the case, the exclusive use of the criterion is recommended. All of the necessary parameters can be calculated by means of the free program *RocLab* and this avoids the approximations and uncertainty associated with trying to determine equivalent Mohr-Coulomb parameters.

References

Hoek, E. & Brown, E.T. (1980) Underground Excavations in Rock. Institution of Mining and Metallurgy, London, 527 pp.

Hoek, E. & Brown, E.T. (1980) Empirical strength criterion for rock masses. J. Geotech. Engng Div., 106:GT9, p. 1013-1035.

Hoek, E. (1983) Strength of Jointed Rock Masses. 23rd Rankine Lecture. Géotechnique, v. 33:3, p. 187-223.

Hoek, E. & Brown, E.T. (1988) The Hoek-Brown failure criterion - A 1988 update. Proc. 15th Canadian Rock Mech. Symp, Curran, J.H. (ed) University of Toronto, Toronto, pp. 31-38.

Hoek, E. (1990) Estimating Mohr-Coulomb friction and cohesion values from the Hoek-Brown failure criterion. Intnl. J. Rock Mech. & Mining Sci. & Geomechanics Abstracts, v. 12:3, p. 227-229.

Hoek, E.; Wood, D. & Shah, S. (1992) A modified Hoek-Brown criterion for jointed rock masses. Proc. Rock Characterization Symp./Int. Soc. Rock Mech., Eurock '92, Hudson, J. (ed), British Geotechnical Society, London, pp. 209-213.

Hoek, E. (1994) Strength of rock and rock masses. ISRM News Journal, v. 2:2, p. 4-16.

Hoek, E.; Kaiser, P.K. & Bawden, W.F. (1995) Support of Underground Excavations in Hard Rock. Balkema, Rotterdam, 210 pp.

Hoek, E. & Brown, E.T. (1997) Practical estimates or rock mass strength. Intnl. J. Rock Mech. & Mining Sci. & Geomechanics, Abstract, v. 34:8, p. 1165-1186.

Hoek, E.; Marinos, P. & Benissi, M. (1998) Applicability of the Geological Strength Index (GSI) classification for very weak and sheared rock masses. The case of the Athens Schist Formation. Bull. Eng. Geol. Env. v. 57:2, p. 151-160.

Hoek, E. & Marinos, P. (2000) Predicting Tunnel Squeezing. Tunnels and Tunnelling International, Part 1, v. 32:11, p. 45-51, Part 2, v. 32:12, p. 33-36.

Marinos, P. & Hoek, E. (2000) From the geological to the rock mass model: Driving the Egnatia Highway through difficult geological conditions, Northern Greece. Proc. 10th International Conference of Italian National Council of Geologists, Rome, pp. 325-334.

Hoek, E. & Karzulovic, A. (2000) Rock-mass properties for surface mines. Hustralid, W.A.; McCarter, M.K. & van Zyl, D.J.A. (eds) Slope Stability in Surface Mining, Society for Mining, Metallurgical and Exploration, Littleton, pp. 59-70.

Marinos, P. & Hoek, E. (2000) GSI: A geologically friendly tool for rock mass strength estimation. Proc. International Conference on Geotechnical & Geological Engineering, GeoEng2000, , Melbourne, Technomic publ., p. 1422-1442.

Marinos, P. & Hoek, E. (2001) Estimating the geotechnical properties of heterogeneous rock masses such as flysch. Bulletin of the Engineering Geology & the Environment, v: 60:1, p. 85-92.

Hoek, E.; Carranza-Torres, C. & Corkum, B. (2002) Hoek-Brown criterion - 2002 edition. Proc. NARMS-TAC Conference, Toronto, v. 1, p. 267-273.

Chandler, R.J.; De Freitas, M.H. & Marinos, P.G. (2004) Geotechnical characterisation of soils and rocks: A geological perspective. Keynote paper in Advances in Geotechnical Engineering, The Skempton Conference, Thomas Telford, ICE, London v. 1, p. 67-102.

Marinos, V.; Marinos, P. & Hoek, E. (2005) The geological Strength index: Applications and limitations. Bull. Eng. Geol. Environ., v. 64:1, p. 55-65.

Hoek, E.; Marinos P. & Marinos V. (2005) Characterization and engineering properties of tectonically undisturbed but lithologically varied sedimentary rock masses. International Journal of Rock Mechanics and Mining Sciences, 42/2, p. 277-285.

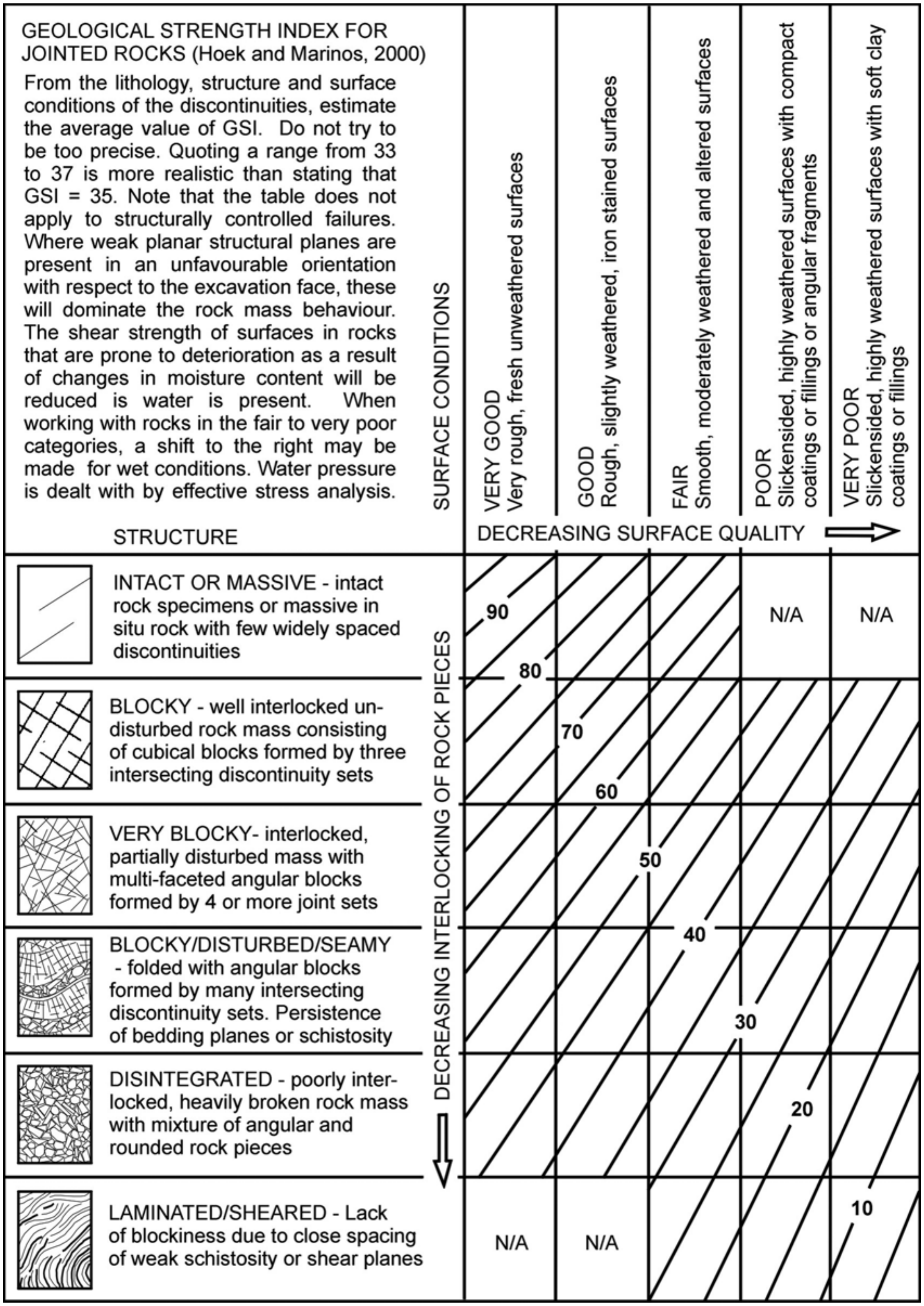
Marinos, P.; Hoek, E. & Marinos, V. (2006) Variability of the engineering properties of rock masses quantified by the geological strength index: The case of ophiolites with special emphasis on tunnelling. Bull. Eng. Geol. Env., v. 65:2, p. 129-142.

Hoek, E. & Diederichs, M.S. (85-92) Empirical estimation of rock mass modulus. International Journal of Rock Mechanics and Mining Sciences, v. 43:2, p. 203-215.

Table 1 - Appendix - Summary of equations

Publication	Coverage	Equations
Hoek & Brown (1980)	Original criterion for jointed hard rock masses tightly interbedded with no fines. Mohr envelope was obtained by statistical curve fitting to a number of (σ'_n, τ) pairs calculated by the method published by Balmer σ'_1, σ'_3 are major and minor effective principal stresses at failure, respectively σ_{ci} is the uniaxial compressive strength of the intact rock σ_t is the tensile strength of the rock mass *m* and *s* are material constants (*s* = 1 for intact rock) σ'_n, τ are effective normal and shear stresses, respectively.	$\sigma'_1 = \sigma'_3 + \sigma_{ci}\sqrt{m\sigma'_1/\sigma'_3 + s}$ $\sigma_t = \frac{\sigma_{ci}}{2}(m - \sqrt{m^2 + 4s})$ $\tau = A\sigma_{ci}\left((\sigma'_n - \sigma_t)/\sigma_{ci}\right)^B$ $\sigma'_n = \sigma'_3 + \left((\sigma'_1 - \sigma'_3)\big/\left(1 + \partial\sigma'_1/\partial\sigma'_3\right)\right)$ $\tau = \left(\sigma'_n - \sigma'_3\right)\sqrt{\partial\sigma'_1/\partial\sigma'_3}$ $\partial\sigma'_1/\partial\sigma'_3 = m\sigma_{ci}/2(\sigma'_1 - \sigma'_3)$
Hoek (1983)	Original criterion for jointed hard rock masses tightly interlocked with no fines with a discussion on anisotropic failure and an exact solution for the Mohr envelope by Dr J.W. Bray.	$\sigma'_1 = \sigma'_3 + \sigma_{ci}\sqrt{m\sigma'_3/\sigma'_{ci} + s}$ $\tau = (\cot\phi'_i - \cos\phi'_i)m\sigma_{ci}/8$ $\phi'_i = \arctan(1\big/\sqrt{4h\cos^2\theta - 1})$ $\theta = (90 + \arctan(1\big/\sqrt{h^3 - 1}))\big/3$ $h = 1 + (16(m\sigma'_n + s\sigma_{ci})\big/(3m^2\sigma_{ci}))$
Hoek & Brown (1988)	As for Hoek 1983 but with the addition of relationships between constants *m* and *s* and *a* modified form of *RMR* in which the Groundwater rating was assigned a fixed value of 10 and the Adjustment for Joint Orientation was set at 0. Also a distinction between *disturbed* and *undisturbed* rock masses was introduced together with means of estimating deformation modulus *E* (after Serafim and Pereira). Note that the ground water rating assigned a final value of 15 in the RMR 1989 version.	Disturbed rock masses: $m_b/m_i = \exp((RMR - 100)/14)$ $s = \exp((RMR - 100)/6)$ Undisturbed or interlocking rock masses: $m_b/m_i = \exp((RMR - 100)/28)$ $s = \exp((RMR - 100)/9)$ $E = 10^{((RMR-10)/40)}$ m_b, m_i are petrographic constants for broken and intact rock, respectively.
Hoek, Wood & Shah (1992)	Modified criterion to account for the fact the heavily jointed rock masses have zero tensile strength. Balmer's technique for calculating shear and normal stress pairs was utilised. Material parameter *a* is introduced.	$\sigma'_1 = \sigma'_3 + \sigma_{ci}(m_b\sigma'_3/\sigma_{ci})^a$ $\sigma'_n = \sigma'_3 + \left((\sigma'_1 - \sigma'_3)\big/\left(1 + \partial\sigma'_1/\partial\sigma'_3\right)\right)$ $\tau = \left(\sigma'_n - \sigma'_3\right)\sqrt{\partial\sigma'_1/\partial\sigma'_3}$ $\partial\sigma'_1/\partial\sigma'_3 = 1 + \alpha m_b^a(\sigma'_3/\sigma_{ci})^{(a-1)}$
Hoek (1994), Hoek, Kaiser & Bawden (1995)	Introduction of the Generalised Hoek-Brown criterion, incorporating both the original criterion for excellent to fair quality rock masses and the modified criterion for poor to very poor quality rock masses with increasing fines content. The Geological Strength Index *GSI* was introduced to overcome the deficiencies in Bieniawski's *RMR* for very poor quality rock masses. The distinction between disturbed and undisturbed rock masses was dropped on the basis that disturbance is generally induced by engineering activities and should be allowed for by downgrading the value of *GSI*.	$\sigma'_1 = \sigma'_3 + \sigma_{ci}(m\sigma'_3/\sigma_{ci} + s)^a$ for $GSI > 25$ $m_b/m_i = \exp((GSI - 100)/28)$ $s = \exp((GSI - 100)/9)$ $a = 0.5$ for $GSI < 25$ $s = 0$ $a = 0.65 - GSI/200$
Hoek, Carranza-Torres and Corkum (2002)	A new set of relationships between GSI, m_b, *s* and *a* is introduced to give a smoother transition between very poor quality rock masses (GSI 25) and stronger rocks. A disturbance factor D to account for stress relaxation and blast damage is also introduced. Equations for the calculation of Mohr Coulomb parameters *c* and ϕ are introduced for specific ranges of the confining stress σ'_{3max} for tunnels and slopes. All of these equations are incorporated into the Windows program RocLab that can be downloaded from the Internet site . A copy of the full paper is included with the download.	$\sigma'_1 = \sigma'_3 + \sigma_{ci}(m_b\sigma'_3/\sigma_{ci} + s)^a$ $m_b = m_i\exp(GSI - 100/28 - 14D)$ $s = \exp(RMR - 100/9 - 3D)$ $a = 1/2 + 1/6(e^{-GSI/15} - e^{-20/3})$ $E_m(GPa) = \left(1 - D/2\right)\sqrt{\sigma_{ci}/100}.10^{((GSI-10)/40)}$ $\phi = \sin^{-1}\left[\frac{6am_b(s + m_b\sigma'_{3n})^{a-1}}{2(1+a)(2+a) + 6am_b(s + m_b\sigma'_{3n})^{a-1}}\right]$ $c' = \frac{\sigma_{ci}[(1+2a)s + (1-a)m_b\sigma'_{3n}](s + m_b\sigma'_{3n})^{a-1}}{(1+a)(2+a)\sqrt{1 + (6am_b(s + m_b\sigma'_{3n})^{a-1})\big/(1+a)(2+a}}$ where, for tunnels $\frac{\sigma'_{3\max}}{\sigma'_{cm}} = 0.47\left(\frac{\sigma'_{cm}}{\gamma H}\right)^{-0,94}$ - *H* is the depth below surface

		for slopes $\frac{\sigma'_{3\max}}{\sigma'_{cm}} = 0.72\left(\frac{\sigma'_{cm}}{\gamma H}\right)^{-0,91}$ - H is the slope height γ is the unit weight of the rock mass.
Hoek and Diederichs (2006)	Based on an analysis of a data set from China and Taiwan, a new relationship between the rock mass deformation modulus E_{rm} and GSI is proposed. This is based on a sigmoid function and two forms of the relationship are presented. The simplified equation depends on GSI and D only and it should be used with caution, only when no information in the intact rock properties are available. The more comprehensive equation includes the intact rock modulus. When laboratory data for the modulus are not available a means of estimating this modulus from the intact rock strength σ_{ci} is given, based on a modulus reduction factor MR.	Sigmoid function: $y = c + \frac{a}{1 + e^{-((x - x_0)/b)}}$ Simplified Hoek and Diederichs equation: $E_{rm}(MPa) = 100\ 000\left(\frac{1 - D/2}{1 + e^{((75 + 25D - GSI)/11)}}\right)$ Hoek and Diederichs equation: $E_{rm} = E_i\left(0.02 + \frac{1 - D/2}{1 + e^{((60 + 15D - GSI)/11)}}\right)$ Estimated intact rock modulus: $E_i = MR\ \sigma_{ci}$.



Geological strength index chart.

Underground Mining with Backfills

R. Rankine, M. Pacheco, N. Sivakugan

The mining industry worldwide has typically not conducted the development of mines with the overall design objective of a safe, environmentally sound and aesthetically satisfactory post-operational mine-site. Mine waste has typically not been engineered to any large degree but has rather been disposed of in the easiest or most cost effective manner with little (if any) regards for the social and/or environmental consequences. The backfilling of mines is an integral part of the mining process and requires the same level of attention generally afforded to the more commonly recognised “profit-producing” parts of the operation. The change in perception of backfilling from an additional cost to mining operations to one of a pre-profit activity will aid the required advancement in technology required for backfills. Backfilling is required for the continuance and efficiency of mining operations. Additional benefits include: improved regional and local rock stability through the support provided by the backfill, reduced costs of building significant tailings disposal structures on the surface, and the reduced environmental impacts by the underground containment of waste material. All these focus the operation towards the overall design objective of a safe, environmentally sound and aesthetically satisfactory post-operational mine-site. With these objectives in mind, the purpose of this paper is to highlight the basic geotechnical issues regarding undergroung mining with backfills, following new developments by the Australian mining industry.

Key words: paste fill, stope, underground mining.

1. Introduction

Discovery of gold in Minas Gerais in 1693 made Brazil the leading gold producer then. A brief historical overview of the mining activities in Brazil was given by Machado & Figueiroa (2001). The mining sector, with more than 1400 active mining companies operating in the country, has an important role to play in the overall economy of Brazil. Brazil is the world’s leading producer of iron ore and Latin America’s leading producer of manganese, aluminium, ferroalloys, tin, gold and steel. The major minerals recovered from the Brazilian mines include bauxite, gold, iron ore, manganese, nickel, phosphates, platinum, tin and uranium. The states of Minas Gerais (40%), Pará (20%), São Paulo (10%), Bahia (8%) and Goiás (6%) represent 84% of the mining Gross Domestic Product (GDP) in the country. In 2000, the mineral-based industries produced US$50.5 billion, contributing to 8.5% of GDP. In 2004, Brazil opened Sossego mine, it is the largest and a world-class copper mine, in the state of Pará, owned and operated by Companhia Vale do Rio Doce (CVRD).

Australia, by any standards, is extremely well endowed with most minerals even though it has barely scratched the surface of its mineral resources. The nation holds the world’s largest known economic resources of bauxite, lead, zinc, silver, uranium, industrial diamonds and mineral sands. The need to ensure the longevity of the nation’s economic wealth through the proper and efficient mining operation of mines is then obvious.

South Africa is also particularly rich in mineral resources and is one of the leading raw material exporters in the world. The main mineral raw materials are gold, diamonds, platinum, chromium, vanadium, manganese, iron ore and coal. These goods make up about 60% of the entire export. With platinum, manganese, vanadium and chromium, South Africa is number one globally, as far as mineral resources as well as the actual mining and export volumes are concerned. The mining industry in Canada is strong also. Canada has over 200 producing metal, non-metal and coal mines over 3,000 stone quarries and gravel pits. Diamonds, oil sands and uranium are the main export commodities, with Canada producing over one-third of the worlds global output of Uranium. Canada also has significant mineral deposits of coal, iron ore, nickel, gold and copper. These mines and their economic output accounts for about four percent of Canadian GDP. The USA have abundant natural resources and are the world’s leading producer of beryllium, soda ash, molybdenum, phosphate rock and salt. California is the largest producer of non-fuel minerals of any US state, producing approximately 10% of the national mineral product. The concentration of the mining operations located along the western seaboard in Fig. 1 pays testament to the contribution of the Californian mineral reserves.

Since the introduction of favorable mining laws in Chile in the late 1970’s, it has become a very attractive minerals target for a number of large national and international mining companies including BHP Billiton, Anglo American, Rio Tinto, Placer Dome, Phelps Dodge, Falconbridge, Barrick Gold, Newmont etc. Chile is the undisputed capital of mining in Latin America and is the world’s largest cop-

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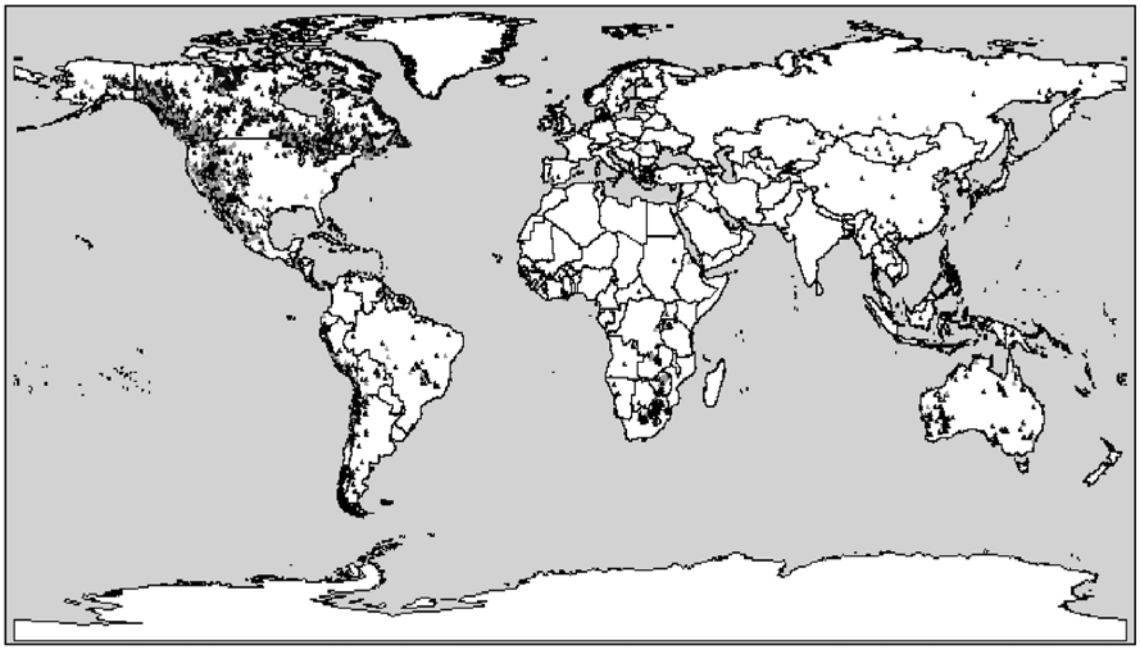

Figure 1 - World-wide mining and exploration activities (Infomining Inc., 2006).

per producer (approximately 20%) as well as exporting gold, silver, molybdenum, zinc, manganese and iron ore. Escondida is the world's largest copper mine and contributes 8% of the world's copper output alone! It is nestled in the cuprous porphyry ore bodies that occur prolifically in the high altitudes and harsh environments of the Andean Cordillera.

Experience gained from the failures of the Ok Tedi Mine (Papua New Guinea) (Kirsch 1996, 2002; Harper and Ravi Rajan 2002) and Marcopper Mine (Phillipines) (UNEP 1996, PDTS 2001), underlines the need to dispose of mine waste in a safe, stable and economically attractive manner. This has highlighted the requirement to be able to accurately predict backfill behaviour and performance. The empirical relationships and operator experience used in yesteryear needs to be replaced by the specific engineering of mine waste. Canada, United States, South Africa, China, Australia, Brazil and Chile are some of the countries that are at the forefront of mining and exploration activities, as shown in Fig. 1. In this paper, a brief description of the main types of backfill is presented, with emphasis on hydraulic fills and paste fills, the most popular backfills used world-wide.

2. Environmental and Safety Issues

The report of the Tribunal (Davies 1967) appointed to inquire into the Disaster at Aberfan (Wales) on October 21st, 1966 detailed the events leading up to and causes of the massive tailings slip from the Merthyr Vale Colliery onto the small mining village of Aberafan, killing 144 people, 116 of whom were school children. The Tribunal was scathing in its appraisal of the competency of those responsible for the stability of the colliery likening them to "moles being asked about the habits of birds". Since then the disposal of mine waste has had a great deal more attention paid to the design and placement of tailings, resulting in a highly engineered "designer waste" (Jones 2000).

Mining activities generally involve several social and economical issues. While millions of tons of soil and rock are removed from the earth's crust, to extract a very small fraction of useful minerals, the rest of the waste material needs to be disposed of. There are strict environmental guidelines stipulating that the mine site is left in good condition on the completion of the mining operation, with all the underground voids backfilled, all toxic material disposed in an environmentally friendly manner, the flora and fauna in the region be protected etc. Mines are required to allocate significant funds to carry out the mine site rehabilitation program, in bringing back the site to a condition similar to what was there before.

Upon extraction of minerals from the ore, there is very large amount of crushed rock, in the form of tailings that has to be disposed of. The most sensible thing to do is to send them back to where they came from. *i.e.*, to backfill the underground voids created in the mining process, using these tailings. With the specific gravity of the parent rock in the order of 2.8-4.0 and the dry unit weight of the backfill of about 15-20 kN/m^3, only about little more than 50% can be placed back into the underground voids. The rest of the tailings have to be sent to the tailings dams or disposed on the surface. Backfills are placed in stopes, which comprise the excavated volumetric unit holding approximately the shape of a rectangular prism, which is subsequently backfilled with some type of waste material. Therefore, any waste materials that are placed into the underground voids (stopes) in the mine are referred to as backfills or minefills. A schematic diagram of a mine stope with access drains is shown in Fig. 2. While serving as an effective means for the tailings disposal, the backfilling process improves the stability of the surroundings, facilitating the mining activities including excavation for ore removal in the nearby areas. Not taking adequate care in the tailing disposal in the underground mine stopes can result in catastrophic accidents that can include fatalities. Mine accidents are reported worldwide, and often these are due to the breach of barricades that prevent fills from flowing into the drives.

3. Main Types of Mine Backfills

In mining engineering, backfill refers to any waste material that is placed into the voids mined underground (stopes) for the purposes of either disposal or to perform some engineering function. Backfills that are used only to fill the voids created by mining need only to have sufficient strength to prevent any form of remobilisation through liquefaction, typically caused by dynamic loading. However, where backfills are used as engineering materials, sufficient strength is required to ensure stability during exposure during ore pillar mining in tall vertical faces or undercuts, particularly in the case of paste fills or other cemented fills. In addition to the excavated rock, other forms of backfill are commonly used such as surface placement of hydraulic fill in a tailings dam or discharge of paste fill from a reticulation pipe underground, as shown in Fig. 3.

Backfills can be divided into two broad categories, cemented or uncemented. Cemented backfills generally include a small dosage of pozzolanic binder such as cement, fly ash etc. to improve the strength. This includes cemented rock fills (CRF), cemented aggregate fills (CAF), cemented hydraulic fills (CHF) and paste fills (PF). The uncemented aggregate fills can be in the form of hydraulic fills (HF), rock fills (RF), sand fills (SF) and aggregate fills (AF). Uncemented backfills, as the name suggests, do not use any binding agents mixed in with the filling material. The mechanical behavior and performance of uncemented backfills can thus be studied using soil mechanics theories. Hydraulic fills are the most common uncemented backfills used world-wide. These are sandy silts or silty sands, with no clay fraction, classified in the Unified Soil Classification System (USCS) as ML or SM. The fine fraction is removed by a process known as desliming. Rock fills (RF) are produced by crushing rock to grain sizes of 25- 300 mm. Materials finer than 25 mm that has been rejected from RF production is described as Aggregate Fill (AF).

Cemented backfills incorporate the use of a small amount of binder material, normally Portland cement, or a blend of Portland cement with other pozzolans such as fly-ash, gypsum or blast furnace slag to the parent backfill material to produce a binding agent for the fill. Cement Hydraulic Fill (CHF) is the most common type of cemented backfill. CHF is produced by the addition of 3-5% cement to deslimed mill tailings, which have the grain size distribution very similar to those of hydraulic fills. CHF is the most similar form of backfill to paste fill with the most significant difference being the larger grain size distribution of CHF when compared to paste fill. Typically in CHF all tailings particles are less than 420 μm and have been deslimed (Bloss 1992), whereas paste fill utilizes the very fine frac-

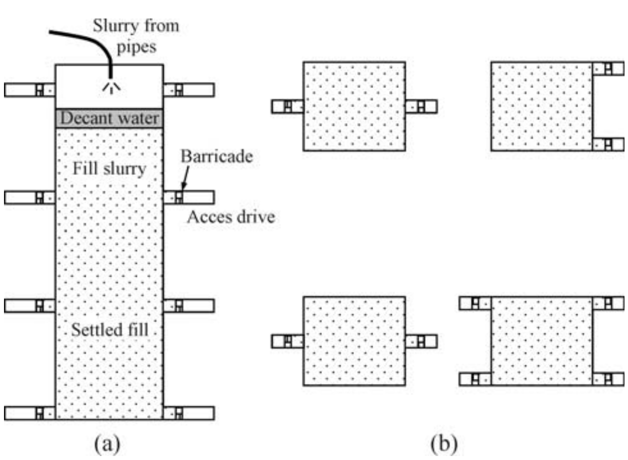

Figure 2 - Schematic diagram of a mine stope with access drains: (a) front view of a stope; (b) possible drain layouts in plan view.

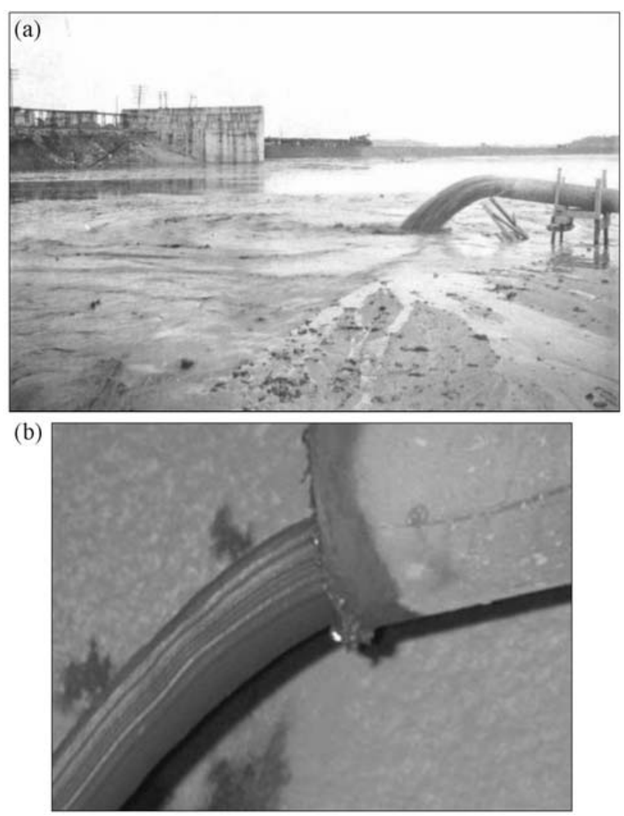

Figure 3 - Various forms of backfill: (a) surface placement of hydraulic fill in a tailings dam; (b) discharge of paste fill from a reticulation pipe underground.

tion of the tailings that provide large specific surface areas and absorb the excess water within the slurry. The grain size distribution of paste fills is significantly finer than CHF and contains a minimum of 15% of particles smaller than 20 μm.

Cemented rock fills (CRF) are prepared by transporting the rock fills to the stope and mixing with cemented hydraulic fills at the ratios of 1:1 to 3:1 (RF:CHF), by weight. The properties of CRFs vary significantly within the stope as the two fills segregate during placement. The ratio of RF:CHF at any location is the dominant factor of the fills behavior at that point (Bloss 1992). As with CRF, AF are mixed with CHF at a ratio of approximately 1:3 AF:CHF by weight. The resulting fill is termed Cemented Aggregate Fill (CAF). CAF typically suffer from segregation during placement, and thus properties at any location within a stope are again dependent on the ratio of AF:CHF at that point as in the case of CRF (Bloss 1992).

Paste fill is the newest form of mining backfill. It is produced from the full mill tailings and has a much finer grain size distribution than any other form of backfill. Typically it has a minimum of 15% of the material smaller than 20 μm, and the maximum grain size in paste fill is between 350-400 μm. Hydraulic fills and paste fills, the two most popular backfills used world-wide, are discussed in detail below.

4. Hydraulic Fills

Hydraulic fills are granular soils with no clay fraction. The fines are removed through the desliming process using hydrocyclones. Rankine *et al.* (2006) and Sivakugan *et al.* (2005) summarised the geotechnical characteristics of the Australian hydraulic fills, based on an extensive laboratory testing program carried out at James Cook University to study more than 25 different hydraulic fills representing all major mines in Australia. It was shown that the grain size distributions for all the fills fall into a narrow band, as shown in Fig. 4. The specific gravity of the grains can be in the range of 2.8-4.5 due to the presence of heavy metals. Since the tailings are fresh from the grinding process, the grains are often very sharp and angular, giving higher friction angles than those for natural soils. A scanning electron micrograph of a hydraulic fill sample is shown in Fig. 5, where the angularities of the grains can be seen. All hydraulic fills, settling only under self-weight, manage to settle to rather high relative densities of 50-80% and porosities of 37-49%, and to dry density (tf/m^3 or gf/cm^3) of 0.57 times the specific gravity or dry unit weight (kN/m^3) of 5.7 times the specific gravity, what is equivalent to dry unit weights in the range of 12 to 25 kN/m^3.

The hydraulic fills are initially transported to the stope in the form of slurry, through pipe lines and bore holes, at solid contents of 65-75%, corresponding to 33-54% water contents. The drives are blocked by a barricade wall, made of special porous concrete bricks, which

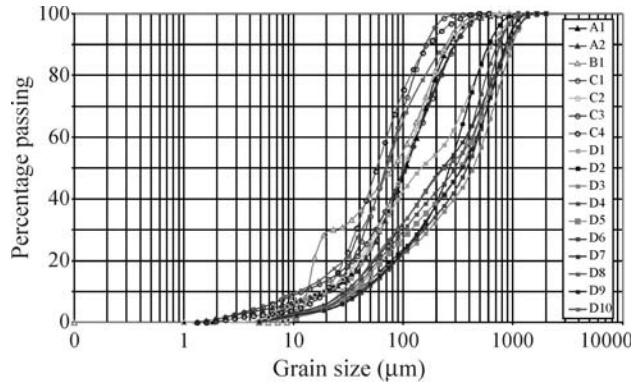

Figure 4 - Grain size distribution curves for various hydraulic fills from Australian mines, Rankine *et al.* (2006) and Sivakugan *et al.* (2005).

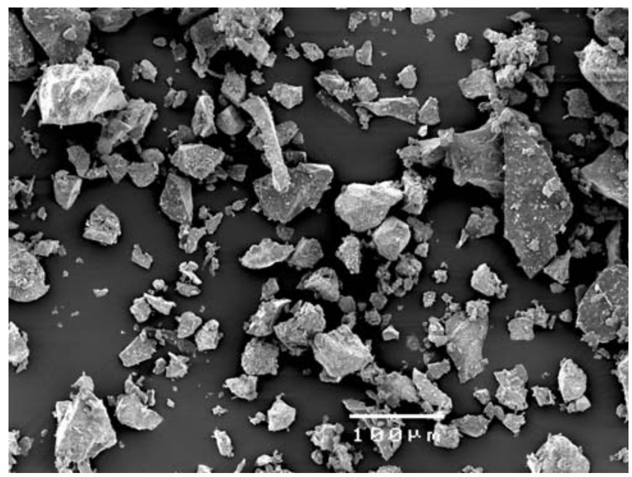

Figure 5 - Scanning electron micrograph of a hydraulic fill.

retains the settled fill and allows the water to drain through. Filling of the stope does not always occur continuously. For example, depending on the site constraints, the filling schedule can be 12 h fill and 12 h rest, continued till the stope is filled. The drainage starts as soon as the filling commences, and continues for weeks, well after the filling ends. At the time when the drainage seems to have finished, the hydraulic fills still have a residual water content, typically about 20-30%, and this residual water remains within the fill indefinitely.

4.1. Drainage issues

Drainage is the prime consideration in the design of hydraulic fill system for a stope. In the absence of good drainage, pore water pressure builds up and reduces the effective stresses within the stope, causing liquefaction, which is one of the prime causes of the barricade failures of hydraulic fill stopes. Any breach of barricades takes place when the hydraulic fill is wet and therefore, every attempt should be taken to get the water out as quickly as possible.

Several trials are generally carried out to arrive at a suitable grain size distribution that will give good drainage characteristics. Herget & de Korompay (1978) suggested the value of 100 mm/h as the minimum hydraulic conductivity required for the hydraulic fill in the mine to perform satisfactorily. Grice (1998) suggested that ensuring D_{10} value greater than 10 μm will ensure adequate drainage throughout the fill. Nevertheless, more than 25 different hydraulic fills, tested at James Cook University laboratory had hydraulic conductivity values in the range of 1-40 mm/h and all these stopes performed satisfactorily. All these fills had D_{10} values in the order of 10-40 μm, satisfying Grice (1998) recommendation. This shows that Herget & de Korompay's threshold value of 100 mm/h is too conservative. Rankine *et al.* (2004) measured the hydraulic conductivity of the barricade bricks using a unique permeameter, under one-dimensional flow conditions as in the barricade wall in the mine. The results showed that the hydraulic conductivity of the special porous brick is 2-3 orders of magnitude greater than that of the hydraulic fills. This enables the fill-barricade boundary to be modelled as free-draining in numerical modelling. The laboratory test procedures and the apparatuses are discussed by Sivakugan *et al.* (2006a).

Isaacs & Carter (1983) developed the first computer model, for two-dimensional stopes, to simulate the hydraulic filling schedule and to monitor the heights of water and tailings within the stope, the pore water pressures, and the discharge through various drains. Rankine *et al.* (2003) extended these to three dimensions and showed that, for the usual stope sizes, two dimension models can simulate satisfactorily the flow in the central portion of the stope. The flow nets of stopes with single and multiple sub-level drains, as obtained by numerical analyses, are shown in Fig. 6. It can be seen in Fig. 6 b that most of the flow takes place through the bottom drain, and this has been observed in the mines. Therefore, for practical purposes, the upper level drains can be neglected, and the stope can be analysed with only one drain at the bottom. Sivakugan *et al.* (2006b) proposed a simpler model with closed form solutions, based on the method of fragments (Harr, 1962), to estimate the maximum pore water pressure and discharge in a two-dimensional stope with a single drain at the bottom. Research is currently underway to extend this closed form solution to 3-dimensions.

5. Paste Fills

Paste fill falls into the broad category of thickened tailings, a concept which was introduced by Dr. Eli Robinsky in the mid 1970's while describing surface disposal of concentrated tailings using pipeline reticulation (Robinsky 1975, 1978). However the first true "paste" backfill was produced at the Bad Grund Mine in Germany in 1979. Acceptance of paste backfill, as a viable alternative to hydraulic slurry and rock fill, did not truly occur until the mid 1990's with the construction and successful operation of several paste backfill systems in Canada and Cannington Mine in Australia in the late 1990's.

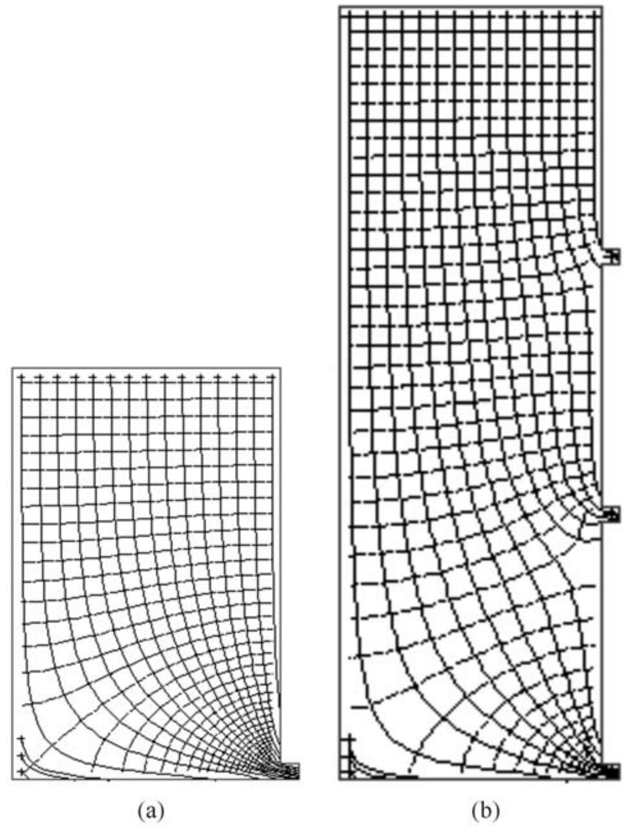

Figure 6 - Flow nets in 30 m wide stopes with one and three drains: (a) single drain; (b) multiple drains.

The definition of "paste backfill" has been one of great debate since its inception in the late 1980's. Primarily because a number of different industries were involved in the evolution of paste and the definition which is adopted by each industry reflects their respective needs and experiences. In an attempt to unify the various definitions of "paste", Fig. 7 was devised and is formally recognised by a number of industry experts and academics (Jewell *et al.* 2002). It should be noted that the boundaries

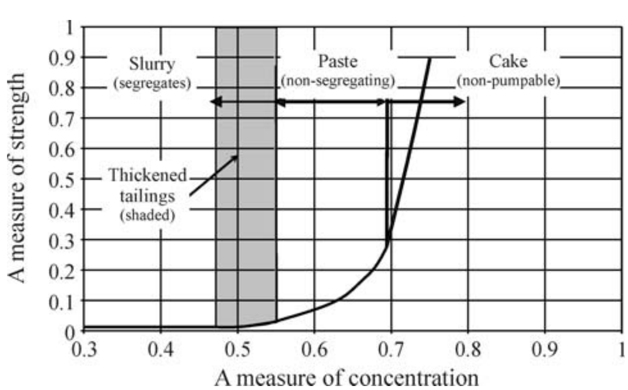

Figure 7 - Concept of thickened tailings continuum (Jewell *et al.*, 2002).

between the mediums are not defined at specific levels or concentrations. Rather they depend on a number of physical and material characteristics of the tailings materials. The shaded area represents the backfills that are commonly referred to as “thickened tailings”. Thickened tailings are a special case of slurry tailings and tend to show many similar characteristics to paste. The similarity of thickened tailings and paste, while moving, is the basis for thickened tailings being commonly confused with or wrongly identified as paste. The primary difference between thickened tailings and paste is that thickened tailings will segregate or settle out once a minimum velocity is reached. A more detailed description of the characteristics which define the difference between slurry, thickened tailings and paste is given in Table 1. When referring to Fig. 7 “thickened tailings” is typically referring to the shaded portion of the graph.

Paste fill rheology closely conforms to the Bingham plastic flow model (Rankine 2004), which is strongly non-Newtonian in its behaviour. Fig. 8 shows a number of fluid models and plots the change in shear stress that is experienced as a function of the shear rate. Fluids which exhibit

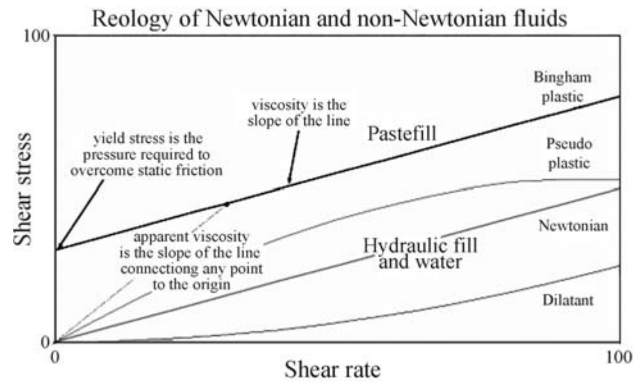

Figure 8 - Rheological curves for particulate fluids.

Newtonian flow characteristics have no “yield stress” to be overcome to initiate movement and have a constant viscosity. Viscosity is defined as the rate of rise of shear stress with the increase in shear rate. Water is an example of a Newtonian fluid. Fluids exhibiting non-Newtonian characteristics are the Power Law (pseudo plastic and dilatant) flow and Bingham plastic flow regimes. Pseudo-plastic fluids are characterised by the reduction of the viscosity with

Table 1 - Material properties for thickened tailings continuum (Jones 2000).

Material property	Slurry	Thickened tailings	Paste
Particle size	Coarse fraction only. No particles less than 20 µm. Segregation during transportation and or placement is dependent only on the coarse fraction	Some fines included (typically < 15%), Fines content tends to modify behaviour from slurry - *i.e.* rheological characteristics more similar to paste, however does segregate when bought to rest. Segregation during transportation and or placement is dependent only on the coarse fraction	Additional / most fines (typically 15%(min) > 20 µm
Pulp density	60%-72%	70%-78%	78%-82%
Flow regimes/ line velocities	Critical flow velocity. To maintain flow must have turbulent flow ($v > 2$ m/s). If $v < 2$ m/s settling occurs. Newtonian flow	Critical flow velocity. To maintain flow must have turbulent flow ($v > 2$ m/s). If $v < 2$ m/s partial settling occurs. Newtonian flow	No critical pipeline flow velocity. *i.e.* no settling in pipe. Laminar/ plug flow
Yield stress	No minimum yield stress	No minimum yield stress	Minimum yield stress
Preparation	Cyclone	Cyclone end elutriation	Filter/ centrifuge
Segregation in stope	Yes/high	Slight/partial	None
Drainage from stope	Yes	Partial/limited	None/insignificant
Final density	Low	Medium/high	High
Supernatant water	High	Some	None
Post placement shrinkage	High	Insignificant	Insignificant
Rehabilitation	Delayed	Immediate	Immediate
Permeability	Medium/low	Low	Very low
Application	Above ground	Above ground	Above ground and underground
Water consumption	High	Medium	Low
Reagent recovery	Low	Medium	High

an increased shear rate (shear thinning), whereas the viscosity for dilatant fluids increases with an increased shear rate (shear thickening).

Bingham plastics exhibit a significant shear stress that must be overcome before movement (shearing) commences. This value of shear stress is commonly referred to as "yield stress". Once shearing has commenced, the viscosity remains approximately linear. The key determinant of the rheological properties of paste fill is the yield stress and can be shown to be exponentially proportional to the solids density of the mix before the effect of cement is realized.

Pullum (2003) suggests that there are effectively two separate forms of paste: homogeneous and heterogeneous paste. The heterogeneous pastes satisfy the minimum rule of thumb of 15% finer than 20 μm. However, during transport, Pullum (2003) has shown stratification of paste during pipe flow with all paste fills with a maximum grain size of over 20 μm. Paste fills with a maximum grain size of under 20 μm tend to form homogenous paste fills during both transportation and deposition.

6. Stress Developement in Backfilled Stopes

The most important issues regarding the stability of backfilled stopes are the failure of the barricade walls and the self supporting ability of the fill mass when exposed.

6.1. Failure of the barricade walls

Catastrophic failure of a barricade results in the inrush of material into the mine workings, which is commonly referred to by miners as a "mud rush" (*i.e.* liquefaction). Also, the barricade can fail under static loading, when the paste lateral stress overcomes the barricade yielding strength. Prior to the curing of the cement, backfill materials have very little self supporting ability. Subsequently, during the backfilling process, an isotropic stress condition equal to the product of the fill's unit weight and gravity develops. As the cement cures, fibrous bonds form between the fill particles and the shear strength of the fill increases. The ability of the fill to sustain and transfer loads through shear stress increases until such time as the cemented fill is strong enough to support the self weight and any additional loads placed on top of the fill. At this point in time the vertical shear stresses acting in the stope walls reach the maximum value, as a result of arching. Accordingly, the vertical normal stresses at a point within the hydraulic fill stope can be substantially less than what is estimated as the product of the depth and unit weight. Therefore, a significant fraction of the fill weight is carried by the rock walls in the form of shear stresses. Similarly, arching causes the lateral stress at the barricade to reach its maximum value when the paste cures. To ensure structural integrity of the barricade, its ultimate strength must be greater than the applied lateral earth pressures at all times.

6.2. Self supporting ability of the fill mass when exposed

The backfill must have sufficient strength to prevent collapse when exposed during the mining sequence, as the surrounding ore is sequentially removed and the backfill is unsupported. Static failure is deemed to have occurred when the principal applied stress is greater than the unconfined compressive strength of the fill mix. This may be conservative for the confined fills within the center of the stope, but provides a reasonable approximation to the fill on the face of the exposure.

To evaluate the risk associated with either form of failures, it is necessary to develop a thorough understanding of the stress developments within the minefills. A closed form solution for estimating the average vertical normal stress ant any depth within a narrow stope containing a cohesionless soil was developed originally by Marston (1930), and later extended by Terzaghi (1943) to include cohesive soils. The general equation to determine the average vertical normal stress (σ_v) at a depth of h within a fill contained in a narrow stope is:

$$\sigma_v = \frac{(\gamma B - 2c)}{2K \tan\delta}\left[1 - \exp\left(\frac{2Kh\tan\delta}{B}\right)\right] \tag{1}$$

B in Equ. (1) is the stope width, γ is the unit weight of the fill, c is the cohesion at the fill-wall interface, δ is the friction angle between the fill and the wall, and K is the ratio of horizontal to vertical normal stress. The corresponding horizontal normal stress is given by:

$$\sigma_h = K\sigma_v\,. \tag{2}$$

Marston (1930) assumed K is the same as Rankine's active earth pressure coefficient, and δ ranging from 0.33 to 0.67 times ϕ, where ϕ is the friction angle of the fill. Terazghi assumed $\delta = \phi$, and $K = 1/(1+2\tan^2\phi)$, which gives a slightly higher value for K than the coefficient of earth pressure at rest K_0. Aubertin *et al.* (2003) suggested a wide range of values for K, from the active to passive earth pressure coefficients. Intuitively, neglecting the yielding of the rock walls, the coefficient of earth pressure at rest seems more plausible. Accordingly, Pirapakaran & Sivakugan (2006) compared the above values with numerical model predictions, and suggested that $\delta = 0.67\phi$ and $K = K_0$ in the above equations would match numerical predictions and measured results better. They also extended the above equations for rectangular and circular stopes, containing cohesionless fills.

Numerical models appear to be quite effective in predicting the stress developments within the minefills. The variation of the vertical normal stress with depth, along the vertical centre line, as obtained from numerical modeling, for a 30 m wide very long stope and a 30 m diameter circular stope of 150 m height is shown in Fig. 9. Pirapakaran & Sivakugan (2006) compared the vertical stress values

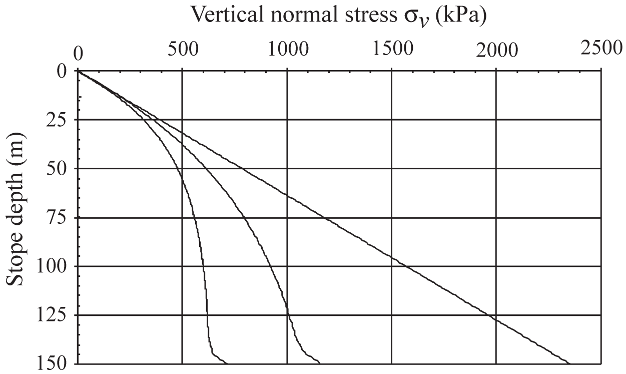

Figure 9 - Variation of vertical normal stresses along the centre line of narrow, circular and square stopes (Pirapakaran & Sivakugan, 2006).

within a square and circular stope of same dimensions and found them to be very close. They used numerical, analytical and laboratory model tests to demonstrate this point.

7. Summary and Conclusions

The use of backfills in underground mining has become a more critical issue in the modern era of the "resource boom". Increased commodity prices and production demands have increased mining rates and required filling rates to record levels. Backfill technology has more capital investment focused into it now than ever before. To ensure that backfilling carried out in underground mines is conducted in a technically proficient and economically attractive manner, a review of the current practice was deemed appropriate, in addition to a brief review of the mining industry in the wider global sense. In addition to this the overall environmental and safety issues in relation to backfilling were reviewed, highlighting the need to learn from historical events.

A variety of backfill types were reviewed and specific focus placed on the two most popular forms of backfill: hydraulic and paste fill. Both were reviewed and the critical issues of stability, drainage, and rheology identified for the hydraulic and paste fills respectively.

The vertical stress development in the fill masses was investigated in relation to the stability of backfill barricades and the static stability of backfilled stopes when exposed. The current analytical solutions were compared against numerical modelling techniques which were found to be an effective means of predicting vertical stress within a backfill stope.

References

Aubertin, M.; Li, L.; Arnoldi, S. & Simon, R. (2003) Interaction between backfills and rock mass in narrow stopes. Soil and Rock Mechanics America, pp. 1157-1164.

Bloss, M.L. (1992) Prediction of Cemented Rock Fill Stability - Design Procedures and Modelling Techniques. PhD Dissertation, The University of Queensland, Australia, 363 p.

Davies, H.E. (1967) Report of the tribunal appointed to inquire into the disaster at Aberfan on October 21st, 1966. London H.M.S.O., 151 p.

Grice, A.G. (1998) Underground mining with backfill. Proc. 2nd Annual Summit - Mine Tailings Disposal Systems, Brisbane, Australia, pp. 234-239.

Harper, K & Ravi Rajan, S. (2002) International environmental Justice: Building the natural assets of the world's poor. International Conference on Natural Assets, Tagaytay City, Philippines, pp. 1-31.

Harr, M.E. (1962) Ground Water and Seepage, McGraw-Hill, Inc, New York, 315 p.

Herget, G. & de Korompay, V. (1978) *In situ* drainage properties of hydraulic backfills. Proc. Mining with Backfills, Research and Innovations, CIM Special v. 19, pp. 117-123.

Infomine Inc., map.infomine, http://www.infomine.com/maps/, (May 11, 2006).

Isaacs, L.T. & Carter, J.P. (1983) Theoretical study of pore water pressures developed in hydraulic fill in mine stopes. Transactions of Institution of Mining and Metallurgy, Section A: Minerals Industry, v. 92, pp. A93-A102.

Jones, H. (2000) Designer Waste. Paste Technology 2000, An International Seminar on the Production and Disposal of Thickened/ Paste Tailings for Mine Backfill or on the Surface, Australian Center for Geomechanics, Section 1, pp. 1-11.

Jewell, R.J.; Fourie A.B. & Lord, E.R. (2002) Paste and Thickened Tailings - A Guide. Australian Center for Geomechanics, Perth, 152 p.

Kirsch, S. (1996) Cleaning up Ok Tedi: Settlement Favors Yonggom People. The Journal of the International Institute, v: 4:1, pp. 1-3.

Kirsch, S. (2002) Anthropology and advocacy: A case study of the campaign against the Ok Tedi Mine. Critique of Anthropology v. 22:5, pp. 175-200.

Marston, A. (1930) The theory of external loads on closed conduits in the light of latest experiments. A research report from Iowa Engineering Experiment Station, Ames, pp. 5-8.

Machado, I.F & Figueiroa, S.F. (2001) 500 years of mining in Brazil: A brief review. Resources Policy, v. 27:1, pp. 9-24.

PDTS - Placer Dome Technical Services (2001) Post Spill Impact Assessment n. 2 (PSIA-2). Marcopper Project, Marinduque Island, Philippines, 57 p.

Pirapakaran, K. & Sivakugan, N. (2006) Arching within hydraulic fill stopes. Geotechnical & Geological Engineering, In Press.

Pullum, L. (2003) Pipeline performance. International Seminar on Paste and Thickened Tailings, Australian

Center for Geomechanics, Melbourne, Section 8, pp. 1-13.

Rankine, K.J.; Rankine, K.S. & Sivakugan, N. (2003) Three dimensional drainage modelling of hydraulic fill mines. Proc. 12th Asian Regional Conference on Soil Mechanics and Geotechnical Engineering, World Scientific Publishing, Singapore, pp. 937-940.

Rankine, R.M. (2004) Geotechnical Characterisation & Stability Analysis of BHP Billiton's Cannington Mine Paste Fill. PhD Dissertation, James Cook Univeristy, Australia, 353 p.

Rankine, K.J.; Sivakugan, N. & Rankine, K.S. (2004) Laboratory tests for minefills and barricade bricks. Proc. 9th ANZ Conference on Geomechanics, Auckland, v. 1, pp. 218-224.

Rankine, K.J.; Sivakugan, N. & Cowling, R. (2006) Emplaced geotechnical characteristics of hydraulic fills in a number of Australian mines. Geotechnical and Geological Engineering, v: 24:1, pp. 1-14.

Robinsky, E.I. (1975) Thickened discharge - A new approach to tailings disposal. CIM Bulletin, v. 68, pp. 47-53.

Robinsky, E.I. (1978) Tailing disposal by the thickened discharge method for improved economy and environmental control. Tailings Disposal Today, Proc. 2nd International Tailing Symposium, Denver, v. 2, pp. 75-92.

Sivakugan, N.; Rankine, K.J. & Rankine, R.M. (2005) Ground improvements - Case histories. Indraratna, B. & Chu, J. (eds) Geotechnical Aspects of Hydraulic Filling of Underground Mine Stopes in Australia. Elsevier, Netherlands, chap. 18, pp. 515-540.

Sivakugan, N.; Rankine, K. & Rankine, R. (2006a) Permeability of hydraulic fills and barricade bricks. Geotechnical and Geological Engineering, v: 24:3, pp. 661-673.

Sivakugan, N.; Rankine, K. & Rankine, K (2006b) Study of drainage through hydraulic fill stopes using method of fragments. Geotechnical and Geological Engineering, v: 24:1, pp. 79-89.

Terzaghi, K. (1943) Theoretical Soil Mechanics, John Wiley & Sons, New York, 528 p.

UNEP - United Nations Environment Programme (1996). Final Report of the United Nations Expert Assessment Mission to Marinduque Island Philippines,UNEP, pp. 66-70.

Lateritic Soil and Bentonite Mixtures Assessment for Liner Usage Purpose

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Abstract. Among all the constructive elements in waste containment systems, base liner may be the one of greatest concern. In Brazil, the use of compacted lateritic soils for this purpose is widely accepted due to their wide occurrence. While they usually achieve compaction and strength requirements, sometimes they may need some improvement in terms of hydraulic conductivity and reactivity to contaminants. This paper describes a laboratory assessment of two mixtures of a lateritic soil and bentonite for liner usage purpose. Laboratory tests included geotechnical, mineralogical and physicochemical characterization, hydraulic conductivity determination in compacted samples and modified Atterberg Limits and grain-size distribution analysis. These two last tests were intended to evaluate the compatibility between the soil and different chemical solutions. The characterization tests showed that the clay content of the mixtures decreased and the plasticity and activity increased relative to the natural soil sample. The hydraulic conductivity of the compacted mixtures decreased two to four times when compared to the compacted natural soil. The results also have demonstrated that the addition of bentonite in the proportions used in the tests increased the compatibility of the natural soil.

Key words: hydraulic conductivity, compatibility, lateritic soil, liner.

1. Introduction

Liner systems play a major role in waste containment facilities such as sanitary and industrial landfills and waste ponds. They are intended to reduce infiltration and attenuate the effect of contaminant solutions by means of sorption, filtration, redox reactions and other processes. Compacted clayey liners (CCL) are widely accepted as one of the main components of these systems (Daniel & Koerner, 1995; Rowe *et al.*, 1995; Bouazza & Van Impe, 1998; Rowe, 2001), and are routinely applied along with geosynthetics and other materials.

Lateritic soils are widely used for liner construction in Brazil and other tropical countries. However, while they usually achieve compaction and strength requirements, sometimes they may not attend the required low hydraulic conductivity and high sorption parameters of CCL's (cation exchange capacity-CEC, for instance). Their peculiar mineral constitution, mainly quartz, kaolinite and Fe-Al oxides/hydroxides, is the major responsible for that.

In this way, because of its intense swelling and CEC properties, bentonite seems to be a good alternative, at least theoretically, as an addictive material to improve lateritic soils for liner usage purpose. The swelling property of bentonite when hydrated would seal the pore space between grains, reducing the hydraulic conductivity. Additionally, an increase in the CEC of the soil would be also achieved.

As pointed out by Schackelford (1994), besides the mechanical and the contaminant transport requirements, other important issue that must be addressed for choosing suitable CCL materials is the clay/waste interaction, which is called liner compatibility. A good compatibility means that no significant changes will take place when the liner material is exposed to waste solutions (organic or inorganic).

An extensive literature review has revealed that all the issues aforementioned have been individually addressed in many papers. For instance, Boscov *et al.* (1999a,b), Leite & Paraguassú (1999a,b), Leite & Paraguassú (2002) and Leite *et al.* (2003) researched on the contaminant transport through lateritic soils. Other papers such as Matos *et al.* (1999 and 2000), Fontes *et al.* (2000), Sarkar *et al.* (2000), Gomes *et al.* (2001), Goldberg *et al.* (2001), Saha *et al.* (2001 and 2002) and Grubb & Hemsi (2002) deal exclusively with heavy metal mobility in lateritic soils and pure oxides-hydroxides samples. In turn, the research by Bowders (1985), Bowders & Daniel (1987) and Foreman & Daniel (1984) investigate the interactions of different contaminant solutions with distinct soil samples and the geotechnical response to those interactions. Finally, the papers by Anderson & Hee (1995) and Osinubi & Nwaiwu (2002) deal with geotechnical issues of mixtures of lateritic soils and bentonite for liner usage purpose.

The aforementioned review has showed no investigations on hydraulic conductivity and compatibility issues together, and this lack is even accentuated when mixtures of lateritic soils and bentonite are included. Shackelford (1994) stressed the importance of the compatibility investigation for liner construction.

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This paper describes a combined investigation on the hydraulic conductivity and compatibility of a lateritic soil and its mixtures with two different proportions of bentonite (3 and 6%, dry weight basis). The hydraulic conductivity was assessed by percolating water in compacted samples using rigid wall permeameters and the compatibility was investigated using modified Atterbeg limits and grain size analysis, both tests using water and different chemical solutions.

2. Material and Methods

2.1. Sampling and sample preparation

Disturbed samples were extracted from a site near the city of Ouro Preto, state of Minas Gerais, Brazil. In place, it is a very clayey dark red oxisoil, which means that advanced laterization processes took place in its formation.

For the soil properties determination and compatibility tests, the samples were prepared according to the standard ABNT-NBR6457 (1986) and Nogueira (1995), which basically consists of air drying at room temperature, homogenization, sample reduction and sieving. These samples are called here as SN.

The bentonite sample (BK), in turn, is commercially available. According to the vendor, it came from the city of Boa Vista, Paraíba, Brazil. Santos (1989) and Rodrigues *et al.* (2004) mentioned that most of the bentonite from this city is treated with concentrated sodium chloride solutions in order to improve their swelling characteristics. More properties of this sample can be found in the item Results and Discussion - Sample Properties.

The proportions of soil and bentonite used in the mixtures (M3 and M6 samples) were chosen based on the values from Anderson & Hee (1995). They are as follows (dry weight basis):

• M3 Sample = 97% soil (SN sample) + 3% bentonite (BK sample)

• M6 Sample = 94% soil (SN sample) + 6% bentonite (BK sample)

The samples used in the hydraulic conductivity tests were dynamically compacted under Normal Proctor energy, at moisture contents from 2 to 4% wet of optimum. After compaction, cylindrical samples were molded with dimensions of 5 cm in diameter and 12 cm height.

2.2. Sample properties determination

The references used to determine the sample properties are presented in Table 1. Most of these properties were determined by conventional and standardized tests usually applied in geotechnical engineering. Some of them, however, are not routinely found in geotechnical laboratories or are more related to geology and pedology. These last tests deserve more details, as given in the paragraphs below.

Free swell tests, as given by Acar & Olivieri (1989), consists of pouring 10 cm^3 of dry soil into a 100 cm^3 graduated cylinder, which is after filled with destilled-deionized water. After a 24-hour period of rest, the free swelling index (S) can be obtained according to Eq. (1).

$$S(\%) = \frac{V_f}{V_i} \times 100 \qquad (1)$$

where V_i is the initial volume of the sediment column inside the graduated cylinder and V_f is its final volume after swelling.

The pH and electrical conductivity (EC) were determined based on Camargo *et al.* (1986). The pH was directly measured in a suspension of 1:2.5 (soil:solution ratio). EC was measured in the water extract obtained from a 1:1 (soil:solution) suspension.

The cation exchange capacity (CEC) and the specific surface (SS) can be estimated from the blue dye adsorption test (filter paper spot test). According to Pejon (1992), the methylene blue ($C_{16}H_{18}N_3SCl$-$3H_2O$) is a strong dye which adsorbs on the mineral surface via cation exchange. The amount of adsorbed blue dye is directly proportional to the CEC and SS of the soil sample when the adsorption saturation is spotted on a filter paper. This test was executed in the soil sample (2 to 4 g, dry weight) that passed through the 10 mm sieve.

The mineral composition of the fine fraction (passing in sieve 0,075 mm) of SN and BK samples was estimated by X-ray diffraction techniques. For that, oriented thin glass sections were elaborated by sedimentation. In turn, two of these thin sections were saturated in ethylene-glycol gas and in KCl solution. The X-ray conditions were: cupper

Table 1 - References used in sample characterization.

Test	Reference
Physical characterization	
- Grain-size analysis	ABNT-NBR7181(1984)
- Liquid limit - ω_L	ABNT-NBR6459 (1984)
- Plastic limit - ω_P	ABNT-NBR7180 (1984)
- Activity - A	Skempton (1953), cited in Michell (1993)
- Specific gravity of solids - G_s	ABNT-NBR6508 (1984)
- Compaction (normal proctor)	ABNT-NBR7182 (1986)
- Optimum moisture content - w_{ot}	
- Maximum dry unit weight - ρ_{dmax}	
- Free swelling test - S	Acar & Olivieri (1989)
Physicochemical characterization	
- pH determination (H_2O)	Camargo *et al.* (1986)
- pH determination (KCl)	Idem
- Electrical Conductivity	Idem
- Blue metilene adsorption test	Pejon (1992)
- Cation exchange capacity	
- Specific Surface	
Mineralogical evaluation	
- X-ray diffraction	Brown & Brindley (1984)

tube, velocity of 1 to 2° per second and angles from 2 to 70°.

2.3. Hydraulic conductivity tests

The hydraulic conductivities (*k*) of SN, M3 and M6 samples were estimated according to the standard ABNT - NBR14545 (2000). The BK sample was not tested because of operational reasons, since bentonite is well known to be difficult to percolate. Anyway, *k* values of less than 10^{-12} ms^{-1} were expected for this sample when compacted.

Figure 1 depicts the permeability cell used for the tests. The main details of this cell are as follows:

• Course grained sand and filter paper (high drainage) were used at the top and base of the soil sample as protection filters;

• Use of wet bentonite (BK sample) to seal the void between the compacted soil sample and the permeability cell wall so as to reduce the "wall effect";

• Use of paraffin at the top and base of the bentonite seal, surrounding the soil sample, in order to prevent the contact between the bentonite and the sand layer.

Test procedures basically involve soil sample saturation and percolation at variable head conditions.

The saturation process was performed by connecting the water tube to the base water port of the permeability cell (see Fig. 1). In this way water would percolate from the bottom to the top of soil sample, making it easier to eliminate the air bubbles trapped in the soil. The air vent (see Fig. 1) was used throughout the saturation time. Full saturation was considered when water popped up at the top water port. This process usually lasted between a month and half.

It is believed that no "wall effect" or other preferential flow way took place at the permeability cell because of the sealing capacity provided by the swelled bentonite and paraffin together (see Fig. 1). Anyway, the use of flexible wall permeameter is suggested in the future. The referred preferential ways are not expected in this type of test. Anderson & Hee (1995) performed several tests on a flexible wall permeameter under confining pressures of 10, 50 and 100 kPa and they found reasonable results.

After saturation, percolation and water level measures (falling head condittions) usually lasted for a week. The hydraulic gradients used in the tests are presented in Table 2, where it can be seen that the maximum gradient did not reach more than 1.84. These low gradient values were intended to prevent any sample disturbance and consequent leaks. A total of 7 hydraulic conductivity tests were performed, 3 for SN sample, 2 for M6 sample and 2 for M3 sample.

2.4. Compatibility tests

Schakelford (1994) mentioned two types of laboratory tests for estimating the compatibility between soil samples and different chemical solutions: (1) modified index properties and (2) percolation tests.

Type-1 test gives only a qualitative indication of possible changes on soil behavior when it is exposed to different chemical solutions. The compatibility is evaluated by comparing the results from conventional tests using water with the results from the tests in which other fluids are applied. This type of tests were applied by Bowders (1985), Bowders & Daniel (1987), Acar & Olivieri (1989) and others.

It is introduced here the so called Plastic Incompatibility Index (IC_p), defined in Eq. (2). This parameter directly indicates the incompatibility of the soil to different

Table 2 - Hydraulic gradients used in tests.

Sample	Minimum value	Maximum value	Average
SN	0.42	1.84	1.14
M3	0.20	0.63	0.39
M6	0.01	0.64	0.16

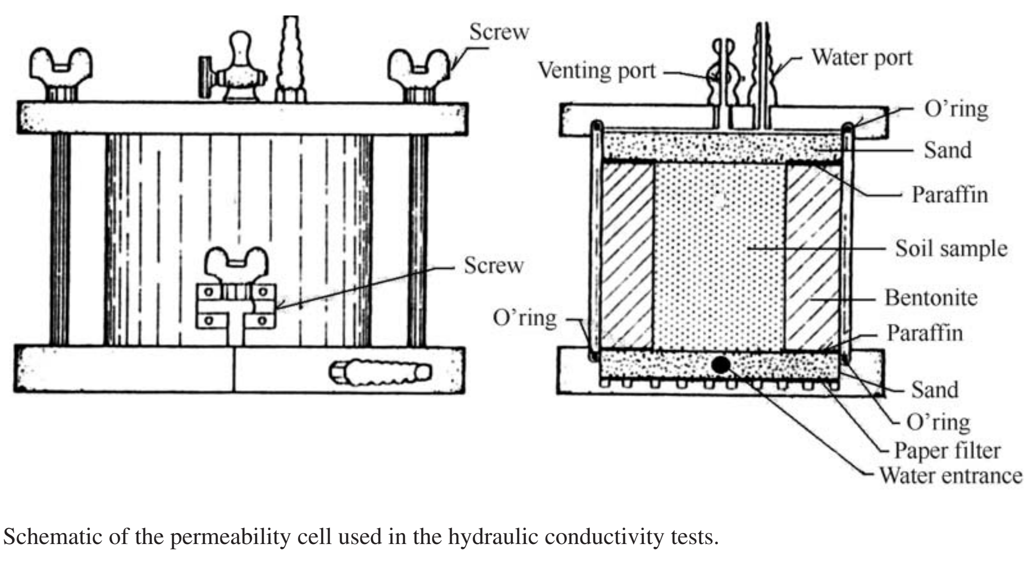

Figure 1 - Schematic of the permeability cell used in the hydraulic conductivity tests.

fluids in terms of plasticity. From Eq. (2) it can be inferred that the higher the IC_p value, the higher the incompatibility of the soil with the percolating fluid compared with water. Additionally, positive values of IC_p indicate a reduction in the plasticity of sample with the fluid in relation to water and vice-versa.

$$IC_p(\%) = \frac{I_{pw} - I_{pf}}{I_{pw}} \times 100 \quad (2)$$

where I_{pw} means the plasticity index with water and I_{pf} the plasticity index with the analyzed fluid.

Type-2 test directly measure the changes in hydraulic conductivity induced by percolating the chemical solutions through the soil. By comparing the *k* value obtained with percolation of water (k_w) and other fluids (k_f), the compatibility is directly evaluated using the relative hydraulic conductivity index (k_R), as can be seen in Eq. (3). This type of test was used by Fernandez & Quigley (1985), Foreman & Daniel (1986), Bowders & Daniel (1987), Fernandes & Quigley (1988), Acar & Olivieri (1989), Budhu *et al.* (1991) and others.

$$K_R = \frac{K_f}{K_w} \quad (3)$$

In the present study, type-1 tests were performed through modified Atterberg limits and grain-size distribution analysis.

The modified Atterberg limits, plastic limit (ω_p) and liquid limit (ω_L), were determined with water and six different solutions: Sodium Chloride-NaCl (1000 mgL^{-1}); Sodium Hydroxide-NaOH (pH = 9 and pH = 11); Nitric Acid-HNO_3 (pH = 3 and pH = 5) and pure Toluene. Exception made to Toluene, the same solutions were applied to modified grain-size distribution analysis.

These different chemical solutions have been chosen considering four specific and extreme environments: acidic, alkaline, salty and pure organic with low dielectric constant. These conditions would represent some of the most "aggressive" environments that liners could be exposed in waste containment facilities, exception made to radioactive and high temperature conditions.

Procedures for Atterberg Limits determination followed the standards ABNT-NBR6459 (1984) and ABNT-NBR7180 (1984). In modified grain-size distribution analysis the hydrometer test was performed in three ways: (1) using a solution of water+defloculant (Sodium Hexametaphosfate), a conventional procedure that is indicated in the standard ABNT-NBR 7181 (1984); (2) using only water and (3) using the chemical solutions aforementioned, except the Toluene. Procedure (2) was intended to better evaluate the effect of the defloculant solution, sodium hexametafosfate-45,7 gL^{-1} + sodium carbonate-pH 8 to 9 (ABNT-NBR7181, 1984). Procedure (3), in turn, intended to assess the effect of the chemical solutions in the grain-size fractions.

3. Results and Discussions

3.1. Sample properties

Table 3 presents the geotechnical properties of the samples. It must be emphasized that the grain-size distribution presented in this table was determined using method (1) aforementioned.

It can be seen from Table 3 that the clay fractions in M3 and M6 samples are smaller than in SN sample. This result was not expected since bentonite (BK samples) was mixed with M3 and M6 samples, which theoretically would lead to an increase in clay fraction of the sample. On the other hand, the silt and fine sand fractions in M3 and M6 samples are bigger than in the SN sample. These facts conduct to the conclusion that the clay provided by the bentonite addition aggregates the SN grains, leading to bigger silt and sand fractions.

As it was expected, ω_p, ω_L and activity of M3 and M6 samples considerably increased compared to the SN sample. This is due to the higher specific surface of bentonite, which hold more water and increased the plasticity of the SN sample. In terms of compaction, w_{ot} slightly increased for the M3 and M6 samples compared to the SN sample. On the other hand, the maximum dry unit weight decreased with bentonite addition.

It is interesting to observe that M3 and M6 samples are classified as CH (high plasticity inorganic clays), according to Unified Classification System. In turn, SN sample is CL (low plasticity inorganic clays), which also shows the effect of bentonite addition in SN sample.

Table 4 depicts the grain-size fractions obtained using procedure-(2) test, as described in Materials and Methods item.

From Tables 3 and 4 it can be seen that all the fractions but clay and silt (M6 sample) have increased and that the coarse sand fraction has experimented the biggest increase. This increase was expected since there was no deflocculant solution. Therefore it is concluded that this solution was effective in dispersing grains in hydrometer test.

Table 5 presents some of the physicochemical properties of the samples. It is observed that:(1) the SN sample was acidic in water with positive ΔpH. This was expected because of the high laterization of this sample; (2) the BK sample was quite alkaline in water and KCl and this fact increased the pH of M3 and M6 samples; (3) ΔpH was negative for M3, M6 and BK samples as was expected since bentonite have predominantly permanent and negative surface charge; (4) the EC of BK sample was quite high, which demonstrates the high salinity of this sample; (5) the differences between CEC and SS values between SN and M3-M6 samples are significant, which shows the expressive contribution of the BK sample in these mixtures.

Table 3 - Sample properties.

Property		Sample			
		SN	BK	M3	M6
Grain-size distribution[1]					
Clay (ϕ < 0,002 mm)	(%)	44	85	30	29
Silt (0,002 < ϕ < 0,06 mm)	(%)	26	11	34	39
Fine sand (0,06 < ϕ < 0,2 mm)	(%)	16	3	22	19
Median sand (0,2 < ϕ < 0,6 mm)	(%)	12	1	11	9
Coarse sand (0,6 < ϕ < 2 mm)	(%)	2	0	2	3
Gravel (ϕ > 2 mm)	(%)	0	0	1	1
Liquid limit - ω_L	(%)	32.30	494.12	55.20	58.80
Plastic limit - ω_p	(%)	20.05	93.84	26.15	26.69
Plastic index - I_p	(%)	12.25	400.28	29.05	32.11
Activity - A		0.28	4.71	0.97	1.11
Specific gravity of solids (G_s)		3.166	3.075	3.047	2.834
Optimum moisture content - w_{ot}	(%)	20.5	NR[2]	23.7	22.6
Maximum dry unit weight - ρ_{dmax}	(g/cm^3)	1.85	NR	1.68	1.62
Free swelling index - S	(%)	NR	162	NR	NR
Unified classification (SUCS)		CL	CH	CH	CH

[1]Grain-size scale of the Massachusetts Institute of Technology - MIT. [2]No results.

Table 4 - Grain-size fractions of the samples without using a deflocculant solution.

Sample	Fraction (%)					
	Clay	Silt	Fine sand	Medium sand	Coarse sand	Gravel
SN	10	38	26	18	6	2
M3	8	40	27	17	6	2
M6	11	35	25	19	8	2
BK	61	32	7	0	0	0

Concluding, X-ray diffraction results have showed the following composition for the SN sample: kaolinite (dominant), gibbsite, goetite and hematite. This composition is typical for lateritic soils. In turn, the BK sample showed predominantly smectite, some kaolinite, quartz and mica.

3.2. Hydraulic conductivity tests

The hydraulic conductivity tests for SN, M3 and M6 samples (Table 6) shows that k values ranged from the order of 10^{-9} to 10^{-10} ms^{-1}.

The effect of bentonite addition is evidenced by the reduction in the average k-value of M3 and M6 samples in respect to the SN sample. This reduction was more expressive for M6 sample in comparison to M3 sample. In fact, the reduction in the average k value of M3 sample compared to SN sample is not so significant, considering all the possible experimental variability.

3.3. Compatibility tests

The results of modified Atterberg limits are presented in Tables 7 and 8, including statistical parameters and the incompatibility index (IC_p) values (see Eq. (2)).

From Tables 7 and 8 it can be seen that the plasticity index coefficient of variation followed this order:

Table 5 - Physicochemical properties of the samples.

Property	Sample			
	SN	BK	M3	M6
pH (H_20) (1: 2.5 soil:solution ratio)	5.09	10.43	6.94	8.11
pH (KCl) (1: 2.5 soil:solution ratio)	5.39	9.40	5.66	3.23
ΔpH (pH KCl - pH H_2O)	+0.30	-1.03	-1.28	-4.88
Electrical conductivity - EC (mScm^{-1})	0.08	2.37	0.85	1.42
Cation exchance capacity - CEC (cmol kg^{-1})	0.85	53.13	3.3	7.06
Specific surface - SS (m^{2}g^{-1})	6.64	414.88	25.73	55.08

SN ≈ M3 > M6 > BK, which indicates greater homogeneity for M6 and BK samples. It is considered that more variability of data indicates more incompatibility of the samples, thus in this case M6 and BK samples are more compatible than SN and M3 in terms of plasticity.

Table 6 - Hydraulic conductivity results.

Test	*k* (ms^{-1})		
	SN sample	M3 sample	M6 sample
1	5.06×10^{-9}	1.48×10^{-9}	4.23×10^{-10}
2	1.30×10^{-9}	1.30×10^{-9}	7.49×10^{-10}
3	3.20×10^{-9}	—	—
Average	3.18×10^{-9}	1.39×10^{-9}	5.86×10^{-10}

The following observations can be deduced from the IC_p values exhibited on Tables 7 and 8: (1) the plasticity of all samples decreased with fluids other than water, exception made to the tests with M3 sample (HNO_3, pH5) and BK sample (NaCl, HNO_3, pH3; NaOH, pH9); (2) the biggest IC_p values were registered for samples SN (NaCl) and M3 (NaOH, pH9) and the smallest for samples BK (HNO_3, pH5) and M3 (NaOH, pH11); (3) an overall observation clearly indicates smaller IC_p values for M6 and BK samples, which leads to the conclusion that these samples are more compatible with the fluids being tested than SN and M3 samples.

It is worth to point out that when toluene was used samples did not offer any plasticity. Similar results where reported by Acar & Olivieri (1989), who observed a significant decrease in the plasticity of montmorilonitic clays

Table 7 - Modified Atterberg Limits results for SN and BK samples.

Fluid	Atterberg limits							
	SN sample				BK sample			
	ω_L (%)	ω_p (%)	I_p (%)	*ICp* (%)	ω_L (%)	ω_p (%)	I_p (%)	*ICp* (%)
Water	32.3	20.1	12.3	-	264.1	71.7	192.4	-
NaCl	30.5	25.8	4.7	61.6	270.2	65.0	205.2	-6.7
HNO_3 - pH 3	34.2	24.7	9.5	22.4	266.7	50.7	216.0	-12.3
HNO_3 - pH 5	34.2	26.6	7.5	38.5	247.2	56.0	191.2	0.6
NaOH - pH 9	33.9	27.1	6.8	44.5	273.7	64.1	209.6	-8.9
NaOH - pH 11	32.3	25.5	6.8	44.5	250.6	62.5	188.1	2.2
Toluene	NP	NP	NP	-	NP	NP	NP	-
Average	32.9	25.0	7.9	-	262.1	61.7	200.4	-
Standard deviation	1.34	2.3	2.4	-	9.8	6.7	10.4	-
Variation coefficient (%)	4.08	9.34	30.14	-	3.75	10.88	5.19	-

NP - Non plastic.

Table 8 - Modified Atterberg Limits results for M3 and M6 samples.

Fluid	Atterberg limits							
	M3 sample				M6 sample			
	ω_L (%)	ω_p (%)	I_p (%)	*ICp* (%)	ω_L (%)	ω_p (%)	I_p (%)	*ICp* (%)
Water	54.2	26.6	27.6	-	60.6	29.1	31.5	-
NaCl	46.1	30.1	16.0	42.0	59.7	35.7	24.0	23.8
HNO_3 - pH 3	48.8	24.3	24.5	11.3	51.2	23.6	27.6	12.3
HNO3 - pH 5	57.5	25.9	31.6	-14.6	50.0	28.0	22.0	30.1
NaOH - pH 9	40.3	29.0	11.3	59.1	55.9	29.6	26.3	16.5
NaOH - pH 11	56.7	29.6	27.1	1.8	51.4	22.5	28.9	8.3
Toluene	NP	NP	NP	-	NP	NP	NP	-
Average	50.6	27.6	23.0	-	54.8	28.1	26.7	-
Standard deviation	6.2	2.1	7.1	-	4.2	4.3	3.1	-
Variation coefficient (%)	12.17	7.67	30.74	-	7.68	15.45	11.65	-

NP - Non plastic.

when in contact with organic fluids of low dieletric constant relative to water. Toluene, in turn, presents a dieletric constant of 2.4.

Bar diagrams were made using the data from Tables 7 and 8 in order to identify possible trends, as shown in Fig. 2. It is observed that I_p decreases for high pH values in the case of SN sample (Fig. 2a). This trend was not observed for M3 and M6 samples, where ω_L, ω_p and I_p oscillate with the pH increasing (Figs. 2b and 2c). Figure 2d clearly shows that BK sample offers the smallest variation in Atterberg limits relative to the other samples. At the same time, a very small increase in ω_p with increasing pH is observed for this last sample.

It is expected that in the field scale the variation in the plasticity indicated on Fig. 2 can lead to modifications on the hydraulic and strength performance of the base liner under the influence of the disposed fluid. This variation is probably associated with flocculation and/or dispersion of the soil grains and peds.

Figures 3 to 6 present bar diagrams for the grain-size fractions (%) obtained using different solutions in the modified hydrometer tests.

It is clear from all these figures that the clay fraction significantly decreases in the absence of the deflocculant solution for all samples. At the same time, the silt and sand fractions increase, which indicates that the flocculated soil (peds) are included in these specific fractions.

The clay fraction was absent in SN, M3 and M6 samples when the NaCl salt was used. When comparing this re-

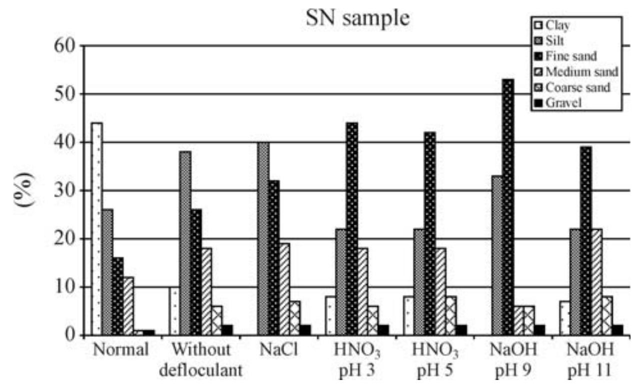

Figure 3 - Grain-size fractions of SN sample exposed to different fluids.

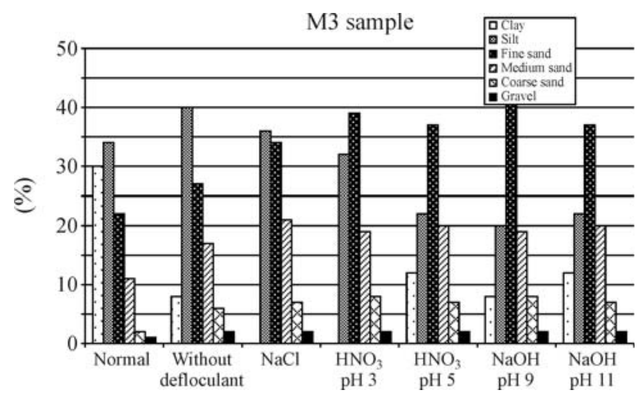

Figure 4 - Grain-size fractions of M3 sample exposed to different fluids.

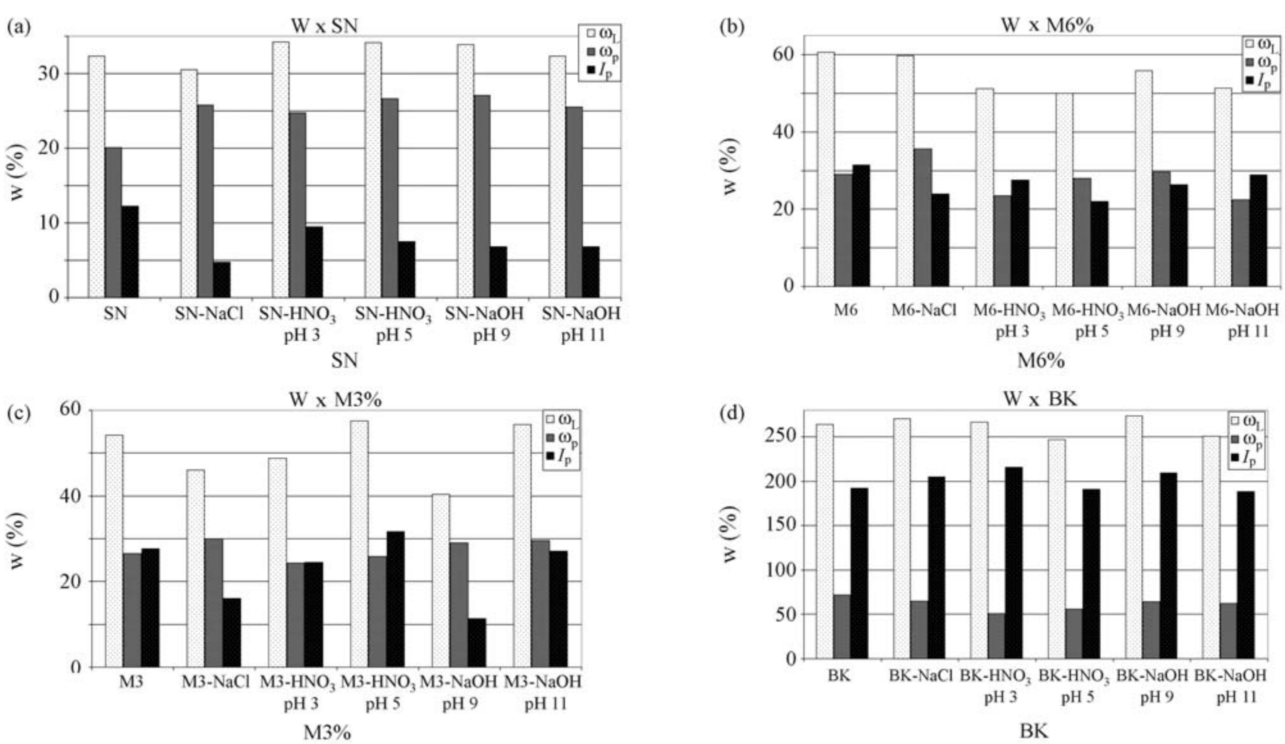

Figure 2 - Atterberg limits using different fluids: (a) SN sample; (b) M3 sample; (c) M6 sample and (d) BK sample.

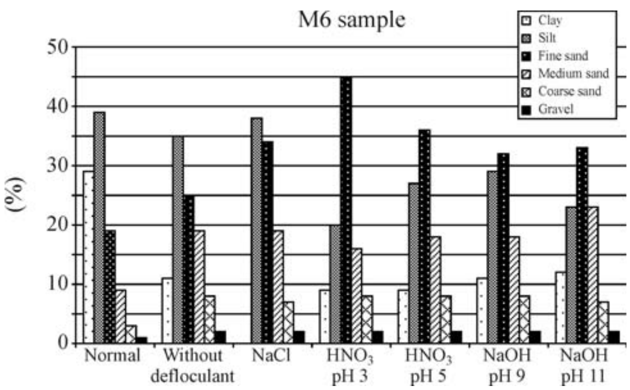

Figure 5 - Grain-size fractions of M6 sample exposed to different fluids.

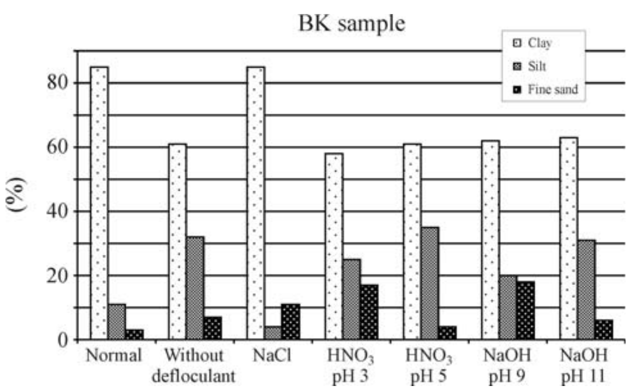

Figure 6 - Grain-size fractions of BK sample exposed to different fluids.

sult with the clay fraction obtained using a solution without deflocculant, it can be concluded that NaCl acted as a flocculant agent. In turn, the clay fraction of BK sample did not change with the use of NaCl, but the silt and sand fractions experimented some variation.

The evaluation of the grain-size distribution for different pH conditions reveals that no specific trends exist for SN, M3 and BK samples. On the other hand, for M6 sample the fine sand fraction decreases as the pH increases. It is worth to point out that for this sample the fine sand fraction achieved its maximum (45%) at pH = 3 compared to all samples and solutions.

Finally, no medium/coarse sand nor gravel were observed in BK sample for all the solutions being tested.

4. Conclusions

In a general way the following conclusions can be stated with respect to the changes in the characteristics of the natural soil sample (SN) when mixed with bentonite (M3 and M6 samples):

(1) The clay fraction of the mixtures decreased relative to the natural soil and this is probably due to the "cement" effect of the bentonite among the SN grains;

(2) As was expected, the presence of bentonite has led to a significant increase in the plasticity of the natural soil;

(3) As was expected, this increase in plasticity has led to a decrease in the maximum dry unit weight obtained under Proctor energy of compaction;

(4) The physical-chemical parameters such as pH, electrical conductivity, cation exchange capacity and specific surface experimented significant changes;

The hydraulic conductivity of the compacted mixtures decreased two to four times when compared to the compacted natural soil. A greater reduction was expected for these mixtures since the bentonite used in the tests is quite swelling.

In terms of compatibility, the results have demonstrated that the greatest changes in the plasticity and grain-size distribution due to the contact with different chemical solutions occurred for SN and M3 samples. These changes may lead to structural modifications in the compacted soil, which may affect its overall performance as liner. On the other hand, M6 and BK samples have shown to be more compatible than SN and M3 samples, which leads to the conclusion that the addition of bentonite in the proportions used in the tests improves the compatibility of the natural soil.

From a perspective of using these mixtures for base liner purposes, it is concluded that the addition of bentonite to lateritic soils brings more activity and compatibility, which may be good in terms of contaminant retention and interaction. Additionally, the influence of the bentonite on the reduction of the hydraulic conductivity was non-linear.

It must be pointed out that the results and conclusions presented here are preliminary and more research on this subject is needed. It is suggested more laboratory tests using different lateritic soils and their direct permeation with different chemical solutions and waste liquids using flexible wall permeameters. Likewise, the structural changes due to bentonite addition deserve more investigation.

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References

Acar, Y.B. & Olivieri, I. (1989) Pore fluid effects on the fabric and hydraulic conductivity of laboratory-compacted clay. Transportation Research Record, v. 1219, p. 144-159.

Anderson, S.A. & Hee, B.H. (1995) Hydraulic conductivity of compacted lateritic soil with bentonite admixture.

Environmental & Engineering Geoscience, v. 1:3, pp. 299-312.

ABNT (1984) Soil Grains Passing the Sieve 4,8 mm - Determination of Solids Density. NBR 6508, Rio de Janeiro, 8 p. (In Portuguese).

ABNT (1984) Soil - Determination of the Liquid Limit. NBR 6459, Rio de Janeiro, 6 p. (In Portuguese).

ABNT (1984) Soil - Determination of Plastic Limit. NBR 7180, Rio de Janeiro, 6 p. (In Portuguese).

ABNT (1984) Soil - Grain-size Distribution Analyses. NBR 7181, Rio de Janeiro, 13 p. (In Portuguese).

ABNT (1986) Soil - Compaction Test. NBR 7182, Rio de Janeiro, 10 p. (In Portuguese).

ABNT (1986) Soil Samples - Preparation for Characterization and Compaction tests. NBR 6457, Rio de Janeiro, 9 p. (In Portuguese).

ABNT (2000) Soil - Determination of the Permeability Coefficient of Clayey Soils Under Variable Charge. NBR 14545, Rio de Janeiro, 12 p. (In Portuguese).

Bouazza, A. & van Impe, W.F. (1998) Liner design for waste disposal sites. Environmental Geology, v. 35:1, p. 41-54.

Boscov, M.E.; Oliveira, E.; Ghilardi, M.P. & Silva, M.M. (1999a) Metal diffusion through a compacted lateritic clay. Proc. 4th Brazilian Congress of Environmental Geotechnics, ITA, São José dos Campos, pp. 323-330 (In Portuguese).

Boscov, M.E.; Pedreira, W.F.; Sarkis, J.E.S. & Huang, Y.C. (1999b) Uranium migration through a layer of compacted soil. Proc. 4th Brazilian Congress of Environmental Geotechnics, ITA, São José dos Campos, pp. 367-374 (In Portuguese).

Bowders, J.J. Jr. (1985) The Influence of Various Concentrations of Organic Liquids on the Hydraulic Conductivity of Compacted Clay. PhD Dissertation GT85-2, Geotechnical Engineering Center, Civil Engineering Department, University of Texas, 350 pp.

Bowders, J.J. Jr. & Daniel, D.E. (1987) Hydraulic conductivity of compacted clay to dilute organic chemicals. Journal of Geotechnical Engineering, v. 117:8. p. 1278-1280.

Brown, G. & Brindley, G.W. (1984) X-ray diffraction procedures for clay mineral identification. Brindley, G.W. & Brown, G. (eds) Crystal Structures of Clay Minerals and Their X-ray Identification. London Mineralogical Society, London, pp. 305-359.

Budhu, M.; Giese Jr., R.F.; Campbell, G. & Baumgrass, L. (1991) The permeability of soils with organic fluids. Canadian Geotechnical Journal, v. 28, pp. 140-147.

Camargo, O.A.; Moniz, A.C.; Jorge, J.A. & Valadares, J.M.A.S. (1986) Methods for Chemical, Mineralogical and Physical Analyses of Soils of the Instituto Agronômico de Campinas. IAC Technical Bulletim n. 106, Campinas, 95 p (In Portuguese).

Daniel, D.E. & Koerner, R.M. (1995) Waste Containment Systems: Guidance for Construction, Quality Assurance, and Quality Control of Liner and Cover Systems. ASCE Press, Reston, VA, 354 pp.

Fernandez, F. & Quigley, R.Q. (1985) Hydraulic conductivity of natural clays permeated with simple liquid hydrocarbons. Canadian Geotechnical Journal, v. 22, p. 205-214.

Fernandez, F. & Quigley, R.Q. (1988) Viscosity and dieletric constant controls on the hydraulic conductivty of clayey soils permeated with water-soluble organics. Canadian Geotechnical Journal, v. 25, p. 582-589.

Fontes, M.P.F.; Matos, A.T.; Costa, L.M. & Neves, J.C.L. (2000) Competitive adsorption of zinc, cadmium, copper, and lead in three highly-weathered Brazilian soils. Commun. Soil Sci. Plant. Anal., v. 31:17-18, p. 2939-2958.

Foreman, D.E. & Daniel, D.E. (1984) Effects of hydraulic gradient and method of testing on the hydraulic conductivity of compacted clay to water, methanol, and heptane. Proc. Tenth Annual Research Symposium on Land Disposal of Hazardous Waste, Cincinnati, EPA-600/9-84-007, pp. 138-144.

Foreman, D.E. & Daniel, D.E. (1986) Permeation of compacted clay with organic chemicals. Journal of Geotechnical Engineering., v. 112:7, p. 669-681.

Gomes, M.P.F.; Gomes, P.C.; Silva, A.G.; Mendonça, E.S. & Neto, A.R. (2001) Selectivity sequence and competitive adsorption of heavy metals by brazilian soils. Soil Science Society of America Journal, v. 65, p. 1115-1121.

Goldberg, S.; Lebron, I.; Suarez, D.L. & Hinedi, Z.R. (2001) Surface characterization of amorphous aluminum oxides. Soil Science Society of America Journal, v. 65, p. 78-86.

Grubb, D.G. & Hemsi, P.S. (2002) Phosphate uptake by a Brazilian soils, Iron-Rich™ and acidic fly ash. Proc. 4th International Congress on Environmental Geotechnics, Rio de Janeiro, v. 1, p. 47-52.

Leite, A.L. & Paraguassú, A.B. (1999a) Potassium diffusion/retention simulation in latosoil mixtures. Proc. 4th Brazilian Congress on Environmental Geotechnics. ITA, São José dos Campos, p. 315-321 (In Portuguese).

Leite, A.L. & Paraguassú, A.B. (1999b) K+ diffusion in compacted tropical soils: A new modelling approach. Proc. 9th Brazilian Congress on Engineering Geology, ABGE-USP-UNESP, São Pedro, p. 55 (CD ROM - In Portuguese).

Leite, A.L. & Paraguassú, A.B. (2002) Diffusion of inorganic chemicals in some compacted tropical soils. Proc. 4th Brazilian Congress on Environmental Geotechnics. ITA, São José dos Campos, pp. 39-46 (In Portuguese).

Leite, A.L.; Paraguassú, A.B. & Rowe, R.K. (2003) Sorption of inorganic chemicals by some tropical soils. Canadian Geotechnical Journal, v. 40:3, p. 619-642.

Matos, A.T.; Costa, L.M.; Fontes, M.P.F. & Martinez, M.A. (1999) Retardation factors and the dispersion/diffusion coefficients of Zn, Cd, Cu, and Pb in soils from Viçosa MG, Brazil. Transactions of the American Society of Agricultural Engineers, v. 42:4, p. 903-910.

Matos, A.T.; Fontes, M.P.F.; Fontes Costa, L.M. & Martinez, M.A. (2000) Mobility of heavy metals as related to soil chemical and mineralogical characteristics of Brazilian soils. Environmental Pollution, v. 111, p. 429-435.

Mitchell, J.K. (1993) Fundamentals of Soil Behavior, 2nd ed. John Wiley & Sons, New York, 437 pp.

Nogueira, J.B. (1995) Soil Mechanics - Laboratory Tests . EESC, São Carlos, 248 pp. (In Portuguese).

Osinubi, K.J. & Nwaiwu, C.M.O. (2002) Compacted lateritic soils as hydraulic barriers in waste containment systems. Proc. 4th International Congress on Environmental Geotechnics, Rio de Janeiro, pp. 225-230.

Pejon, O.J. (1992) Engineering Geological Mapping of the Piracicaba, SP (1:100000 scale): A Study of Methods, Characterization and Presentation of the Attributes. PhD Thesis, Engineering School of Sao Carlos, University of São Paulo, pp. 59-71 (In Portuguese).

Rodrigues, M.G.F.; Silva, M.P.L. & Silva, M.G.C. (2004) Characterization of bentonite clay for removing lead from synthetic effluent. Cerâmica, v. 50:315, p. 190-196 (In Portuguese).

Rowe, R.K.; Quigley, R.M. & Booker, J.R. (1995) Clayey Barrier Systems for Waste Disposal Facilities. E & FN SPON, London, 390 pp.

Rowe, R.K. (2001) Barrier systems. Rowe, R.K. (ed) Geotechnical and Geoenvironmental Engineering Handbook, chapter 25. Kluwer Academic Publishing, Norwell, pp. 739-788.

Saha, U.K.; Taniguchi, S. & Sakurai, K. (2001) Adsorption behavior of cadmium, zinc, and lead on hydroxyaluminum-and hydroxyaluminosilicate-montmorillonite complexes. Soil Science Society of America Journal, v. 65, p. 694-703.

Saha, U.K.; Taniguchi, S. & Sakurai, K. (2002) Simultaneous adsorption of cadmium, zinc, and lead on hy hydroxyaluminum-and hydroxyaluminosilicate-montmorillonite complexes. Soil Science Society of America Journal, v. 66, p. 117-128.

Sarkar, D.; Essington, M.E. & Misra, K.C. (2000) Adsorption of mercury(II) by kaolinite. Soil Science Society of America Journal, v. 64, p. 1968-1975.

Santos, P.S. (1989) Clay Science and Technology, 2nd ed., v. 1. Edgard Blücher, São Paulo, 499 pp. (In Portuguese).

Shackelford, C.D. (1994) Waste-soil interactions that alter hydraulic conductivity. Daniel, D.E. & Trautwein S.J. (eds) Hydraulic Conductivity and Waste Contaminant Transport in Soil. ASTM-STP 1142, Philadelphia, pp. 112-166.

Symbols

A: activity
BK: bentonite sample
CCL: compacted clay liner
EC: electrical conductivity
CEC: cation exchange capacity
G_s: specific gravity of solids
k: hydraulic conductivity
k_R relative hydraulic conductivity
k_f : hydraulic conductivity with a different fluid
k_w: hydraulic conductivity with water
ICp: incompatibility index
ω_L: liquid limit
I_P: plasticity index
I_{pw} : plasticity index using water
I_{pf}: plasticity index using a different fluid
ω_P: plastic limit
M3: SN sample with 3% of bentonite (dry weight basis)
M6: SN sample with 6% of bentonite (dry weight basis)
pH: potential hydrogen
S: free swelling index
SN: natural soil sample
SS: specific surface
w_{ot}: optimum moisture content
ρ_{dmax}: maximum dry unit weight
V_f : final volume of sediment column
V_i: initial volume of sediment column

Dynamic Plate Load Tests

Luciene S. de Moraes, José Carlos A. Cintra, Nelson Aoki

Abstract. Dynamic load test is often used for bearing capacity evaluation of driven pile foundation. It is also reported the successful use of this test in bored piles and caissons. This research is mainly concerned with the adaptation, performance and interpretation of the dynamic load test in circular steel plate aiming at the verification of the bearing capacity of shallow foundations. The dynamic increasing energy test (DIET) was used. The tests were performed at the USP/São Carlos experimental foundation site, which soil profile consists of superficial unsaturated, porous and collapsible soil, and for that reason matric suction's measurements were made, since it has strong influence on the bearing capacity. It could be verified that it is possible to make use of the dynamic load test to plates, associated with analysis methods based on the stress propagation in bars (wave equation), to infer the plate-soil system bearing capacity. Good adjustments were found from dynamic and static load tests performed in a plate tested in this research field. It was also verified that the plate penetration into the soil caused an important increase in the plate-soil system bearing capacity.

Key words: plate static load test, plate dynamic load test, collapsible soil.

1. Introduction

An alternative method of loading testing for shallow foundations is presented: the dynamic plate load test. The dynamic load test is often used in the determination of bearing capacity for driven pile foundation. The monitoring during pile driving is an important tool extensively used in current practice in Brazil and also abroad. This system provides a quick answer in the determination of mobilized resistance, during pile's installation, when the dynamic increasing energy test (DIET) approach is used (Aoki, 2000). For some years, the monitoring has also been extended to bored piles, to evaluate the static mobilized resistance in a faster and less expensive way when compared with static load tests. Researches have already proved the applicability of the technique also in caissons.

This research is mainly concerned with the adaptation, performance and interpretation of the dynamic load test in a 25 mm thickness circular steel plate with 0.80 m diameter. The objective of the research program was to verify the possible use of the proposed analysis based on the stress propagation in bars (wave equation), to infer the plate-soil system bearing capacity.

The results obtained from dynamic and static load tests performed at the Experimental Foundation Site of USP/ São Carlos are compared. The soil profile typically presents a very porous superficial layer, unsaturated and collapsible, due to the weathering action under typical tropical conditions, common in the region. For this reason, the tests were performed with matric suction monitoring by means of tensiometers. This monitoring is of great importance, as the matric suction strongly influences the bearing capacity (Costa, 1999).

In that porous soil, the load-settlement curves obtained from several plate, footing and caisson tests showed that almost all settlement results from plastic (irreversible) deformation and the soil presents strain hardening, not clearly indicating the failure condition. The penetration of the plate then causes a major increase in the bearing capacity, by increasing the stiffness of the soil underneath it.

2. Site Characterization

The plate load tests were carried out at the Experimental Foundation Site of the University of São Paulo, in the city of São Carlos, Brazil.

The city of São Carlos is located 800 m above sea level, on top of rocks from the São Bento Group, which are composed by sandstone from the Botucatu and Pirambóia formations and basalt from the Serra Geral formation (Bortolucci, 1983).

The Experimental Foundation Site is at the extreme south portion of the University campus, in an area with a geological-geotechnical profile considered representative of the center-western region of São Paulo state (Cintra *et al.*, 1991). Figure 1 shows the geological profile from part of the city of São Carlos and the location of the University campus.

The typical soil profile at the test site includes a superficial lateritic clayey sand layer (brown colluvium) and the soil is very porous, unsaturated and collapsible, with low bearing capacity parameters and N_{SPT} ranging from 1 to 5 blows. A 0.3 m thick layer of pebbles, located at a depth of approximately 6 m, separates the superficial layer from a residual soil layer, which is composed of reddish clayey sand (saprolite). Both layers are classified as clayey sand (SC) according to the Unified Classification System. A

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clayey silt layer with fragmented and altered basaltic rock is reached at a depth of 24 m. The water table varies seasonally from 7 to 10 m below ground surface. The area of the experimental site was geotechnically characterized by laboratory and *in situ* tests and Fig. 2 summarizes the results obtained.

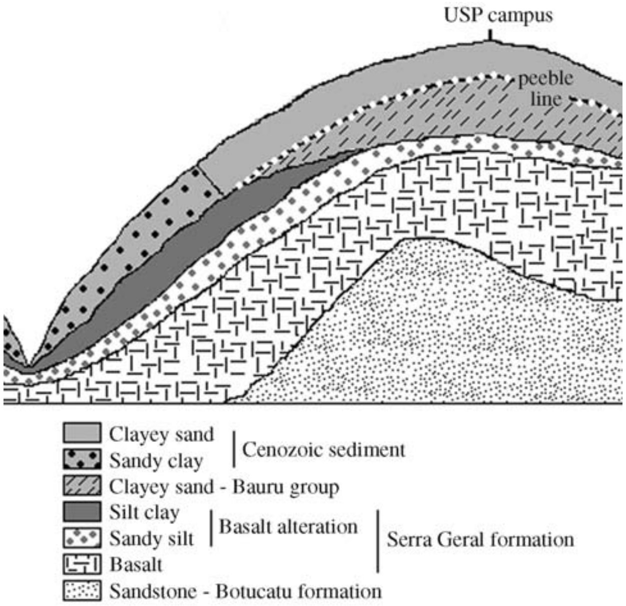

Figure 1 - São Carlos' shallow depth geology schematic section (Bortolucci, 1983).

Many researches have been previously carried out at the same site. These researches included the performance of dynamic load tests in caissons and static compression load tests in caissons, in pile groups and in many kinds of isolated piles with different geometries, including also uplift and horizontal static load tests in some types of piles and static load tests in footings and plates of several diameters and depths.

The results from plate, footing and caisson static load tests carried out at the site show that the load x settlement curves do not clearly indicate failure. The final portion of the curves presents a continuous, almost linear, resistance increase with settlement. A typical plate load test result is presented in Fig. 3. The test was performed at the foundation experimental site in an 80 cm steel plate.

Benvenutti (2001), testing caissons with successive static load tests in this same experimental site, showed that besides the non-definition of failure there is an increase of the caisson-soil system's bearing capacity, as the caisson penetrates the soil. Figure 4 presents the results of three static load tests performed sequentially in the same caisson. It can be seen that the final parts of the load x settlement curves constitute one almost straight line.

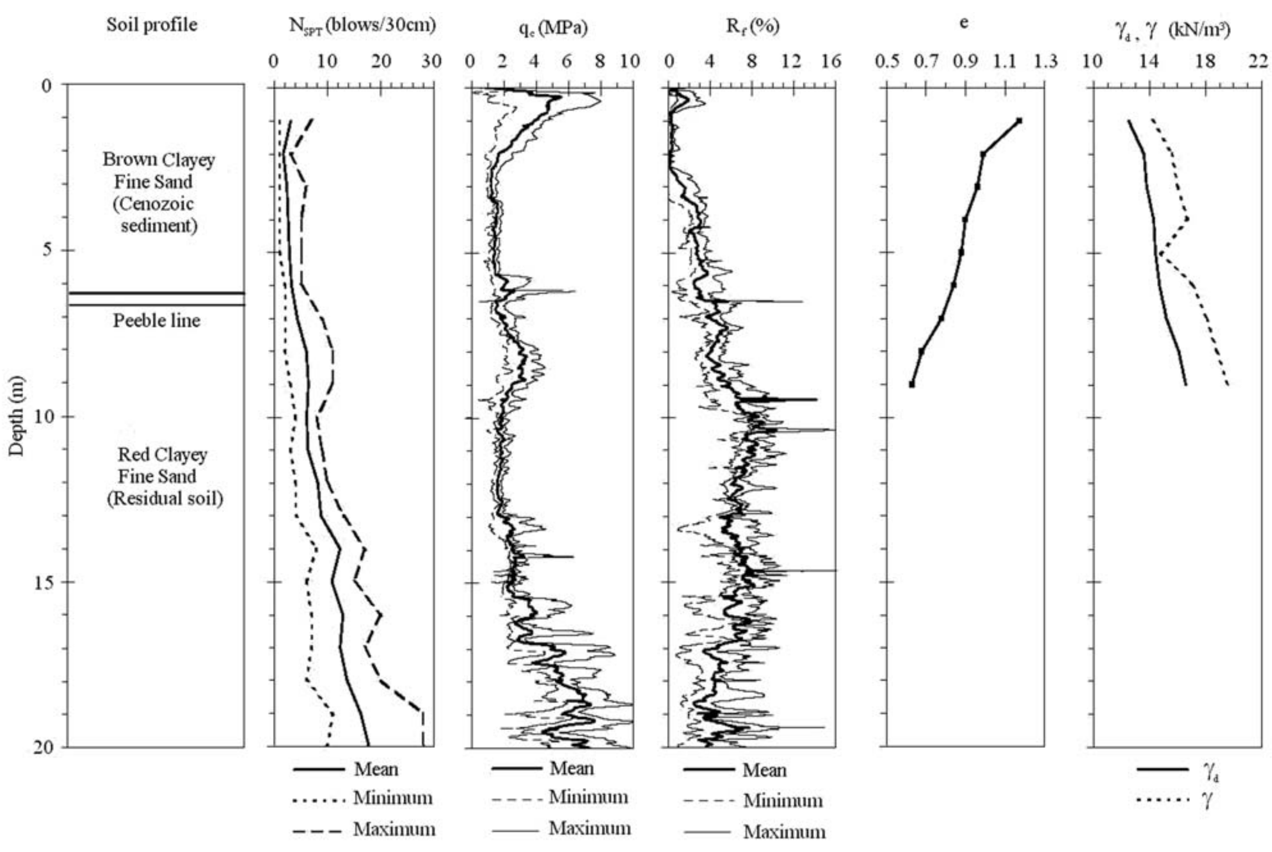

Figure 2 - *In situ* and laboratory tests results for the site's soil profile (Menegotto & Vianna, 2000).

In order to interpret the results with due account of the caissons penetration, Benvenutti (2001) considered that the final portion of each of the three successive curves almost align as a single straight line, disregarding subsequent unloading and reloading sequences up to reaching the maximum testing load from the previous test. A conventional failure criterion was chosen to determine the bearing capacity for some penetration levels. The bearing capacity was established as the load corresponding to a 25 mm settlement. Since the final portions of the curves constitute one straight line, it is possible to determine the bearing capacity for any desired penetration levels, which would correspond to an additional 25 mm settlement.

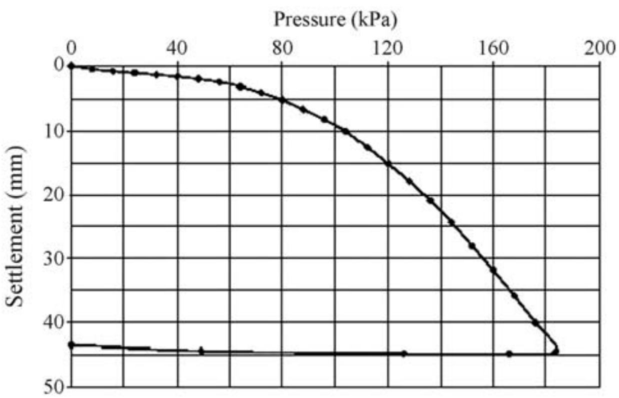

Figure 3 - Plate static load test at the experimental foundation site (Macacari, 2001).

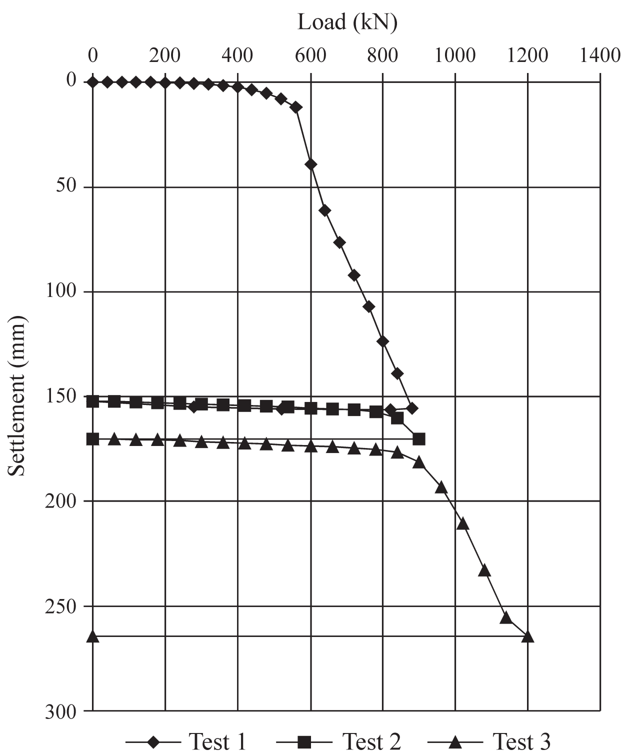

Figure 4 - Load x settlement curves for three tests performed on the same caisson (Benvenutti, 2001).

Besides, since the site presents a thick layer of unsaturated soil, the matric suction has to be taken into account. At the experimental site, an important influence of the matric suction on the bearing capacity has been observed (Costa, 1999). By means of static plate load tests in 1.5 m deep pits it has been verified that a small increase in the matric suction causes a major increase in the bearing capacity, as it can be seen in Fig. 5, where Ψ_m represents the mean soil matric suction measured by four tensiometers. The almost zero matric suction condition was obtained by flooding the pit for 24 h prior to the test.

3. Materials and Methods

Two dynamic load tests and four static load tests were performed on a 25 mm thickness circular steel rigid plate, 80 cm in diameter. The tests were performed in three pits (A, B and C) 0.90 to 1.00 m diameter and 1.5 m deep. Figure 6 presents the plan view of the site.

3.1. Dynamic load tests

Each increasing energy hammer blow was monitored with a scheme used in pre-cast pile tests, with the application of the PDA (Pile Driving Analyzer). The instrumentation consisted of two accelerometers, two strain transducers and a driving analyzer.

Additionally to the dynamic monitoring, settlement was measured using paper sheet and pencil. The measurements were performed by taping a sheet of paper in the tube connected to the plate as shown in Fig. 7.

3.2. Adaptation for dynamic plate load tests

The circular steel plate tested is 25 mm thick with 0.80 m diameter, resulting in a 0.50 m^2 soil contact area. To increase the stiffness, the plate is 25 mm thicker from its center up to 30 cm diameter (a 30 cm plate is welded to the center of the 80 cm plate, creating a projection that enhances the plate's stiffness).

The use of common dynamic test instrumentation, according to the Brazilian Standard NBR 13208/94 (ABNT

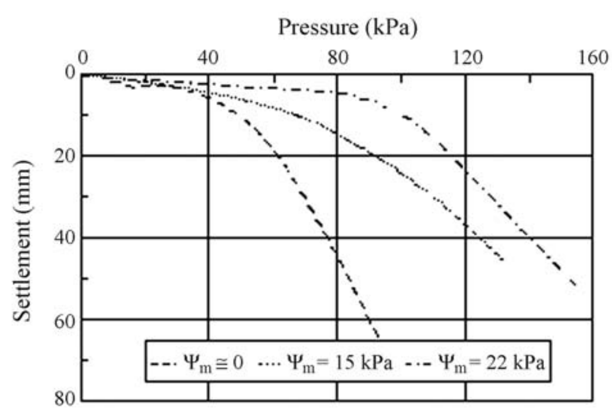

Figure 5 - Pressure *vs.* settlement static load test curves on 0.80 m diameter plate with different matric suction levels (Costa, 1999).

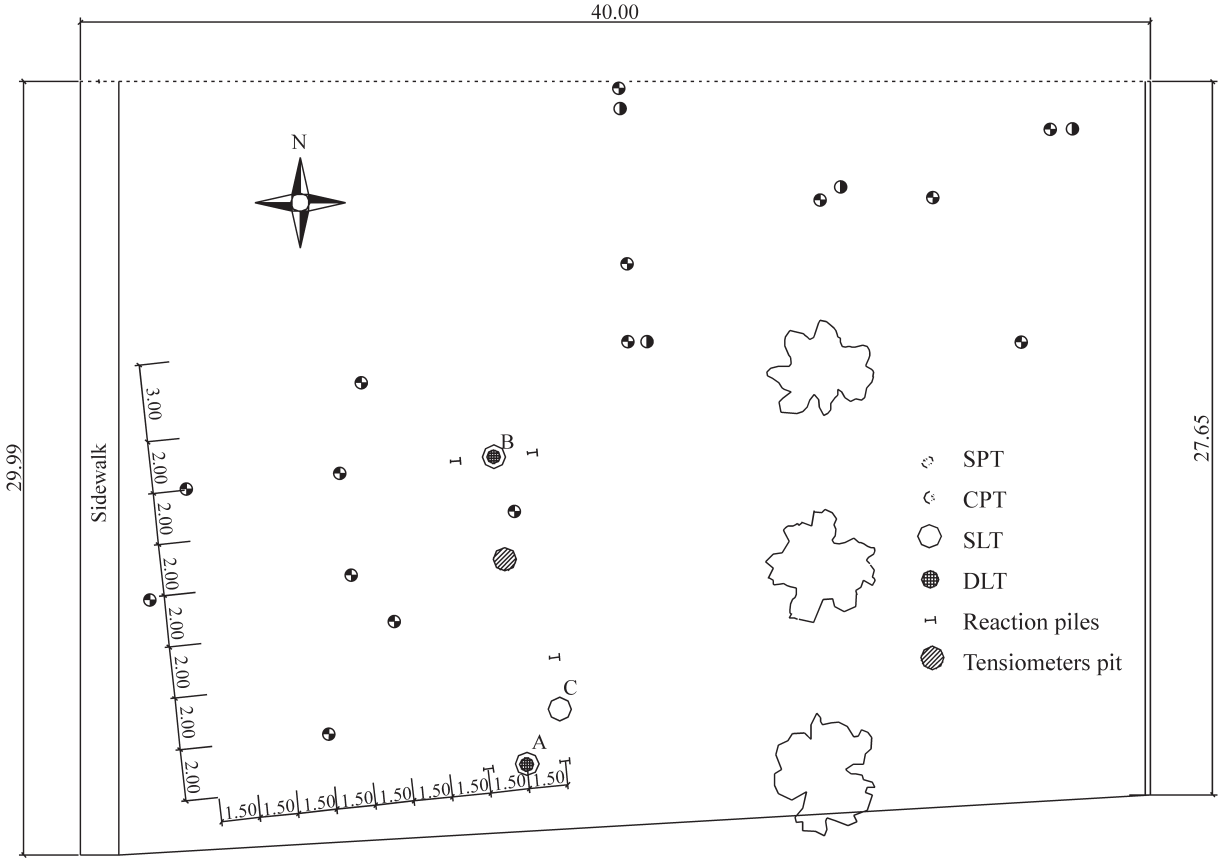

Figure 6 - Site plan view (dimensions in meters).

Figure 7 - Settlement measurements using paper sheet and pencil.

1994), was made possible by connecting the plate to a 7 m long steel tube, with external diameter of 0.22 m and 8 mm thick wall. The height of the tube was chosen based on numerical simulations of a stress wave running downwards and upwards a steel tube. It was simulated that the tube was 3, 4, 5 and 6 m long and the latter value provided good results. Thus, a 7 m long tube was then chosen, because the instrumentation can not be placed near the tube head, due to interference from the blows and possible damage of instruments. In addition, if the tube is not long enough the electric signals registered by the instrumentation are difficult to analyze, because of wave reflections. These numerical simulations were made using a wave equation application computer code developed by Aoki (1989).

A ring shaped frame was welded to the base of the tube described above. This frame was bolted to the plate as shown in Fig. 8.

A free fall 15 kN hammer mounted on a pile driver was used to perform the tests with increasing fall heights (2.5; 5.0; 7.5; 10.0; 12.5; 15.0; 20.0; 25.0 and 30.0 cm). Figure 9 shows the equipment being placed inside the pit (Fig. 9a) and the general setup of the dynamic testing assembly (Fig. 9b).

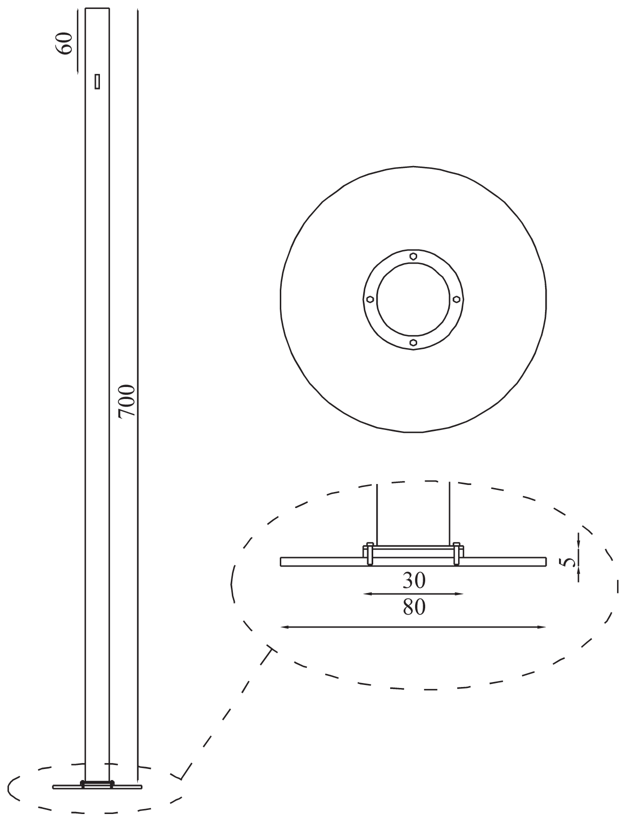

Figure 8 - Tube-plate arrangement used in the tests (dimensions in cm).

3.3. Static load tests

Two static load tests were performed after the dynamic tests at the same pits. The static load tests were performed with QML - Quick Maintained Load, according to Brazilian Standard NBR 12131/91 (ABNT, 1991), except for the time intervals that were altered to 15 min, according to Fellenius (1975) proposition. During each time interval the load was maintained and settlements readings were made at 0, 1, 2, 3, 6, 9, 12 and 15 min; unloading was made in two 15 min stages.

Costa (1999) performed other tests in nearby pits, excavated in the same condition as the ones described in this paper. One of them was a quick maintained load test with matric suction value close to the ones measured in the present work and therefore represents a similar situation. These results were used in the present paper for comparison purposes.

Figure 10 presents a schematic view of the static testing reaction assembly.

3.4. Matric suction measurements

During the time while the tests were performed, periodic matric suction measurements were made by means of tensiometers installed in a control pit. This pit was not used for testing and was excavated under the same condition and close to the other pits. Four tensiometers were installed

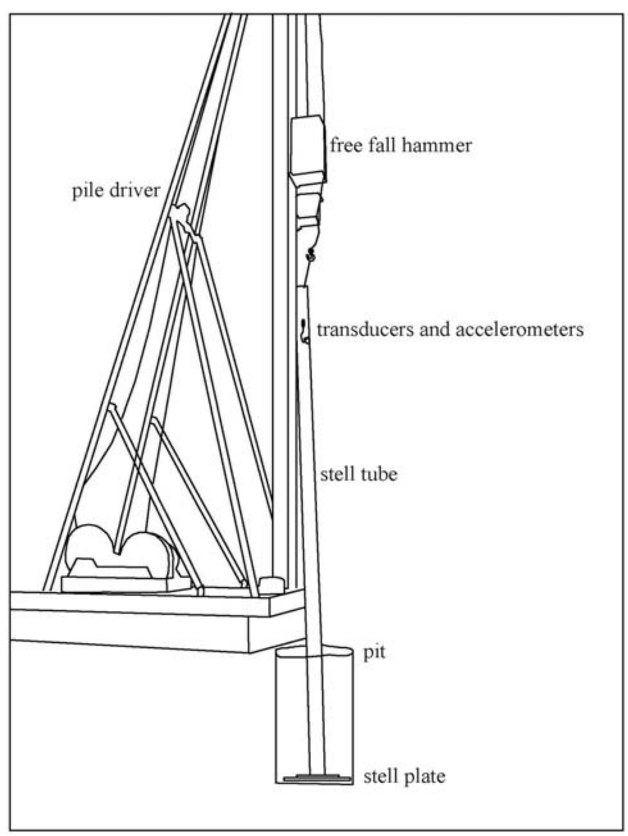

Figure 9 - (a) Plate-tube arrangement being placed inside the pit and (b) Test general assembly.

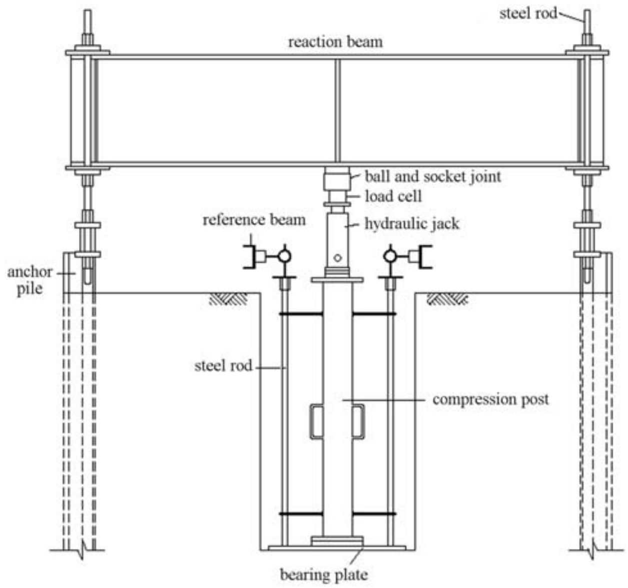

Figure 10 - General scheme of the static testing assembly.

0.20 m deep, connected to four vacuum meters. A portable vacuum pump was used to remove persistent air bubbles from the system. Figure 11 shows the positions of the tensiometers in the control pit.

The static and dynamic tests designations and brief descriptions are summarized in Table 1.

4. Results

The pressure x settlement curves obtained for the three static plate load tests are shown in Fig. 12. Figure 13 shows the results from the two dynamic plate load tests in terms of static reaction force obtained through CAPWAP® analysis and the maximum displacement measured by the (PDA). CAPWAP (Case Pile Wave Analysis Program) is a software based on the wave equation theory and signal matching procedure that predicts total bearing capacity of a pile or shaft, as well as resistance sharing between pile shaft and tip. The program takes as input the force and velocity data obtained from the PDA. Figure 14 presents the same results in terms of pressure x settlement. The static reaction forces were divided by the plate area and plotted against the maximum displacements measured by the PDA.

Table 1 - Testing program.

Test designation	Test type	Pit
SLT1	Static load test	C
SLT2	Static load test	A
SLT3	Static load test	B
DLT1	Dynamic load test	A
DLT2	Dynamic load test	B

Figure 11 - Tensiometers positions in the control pit.

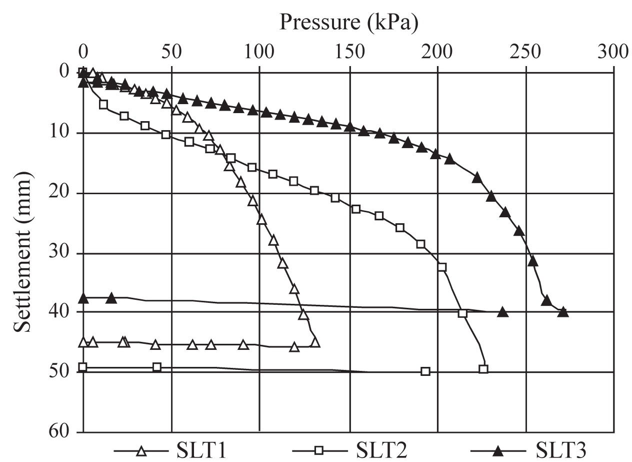

Figure 12 - Results of static load tests.

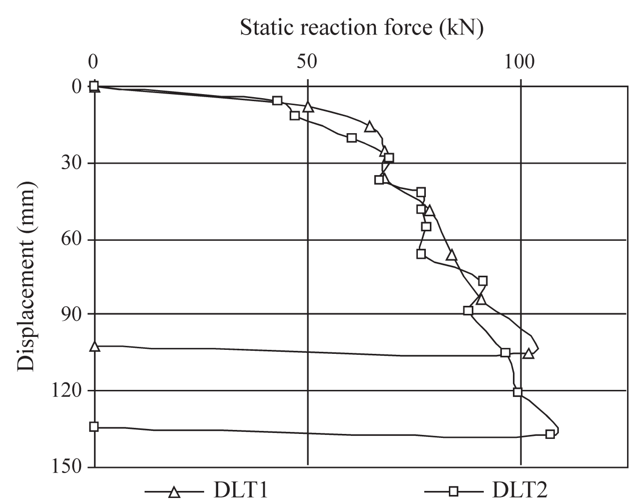

Figure 13 - Results of dynamic load tests.

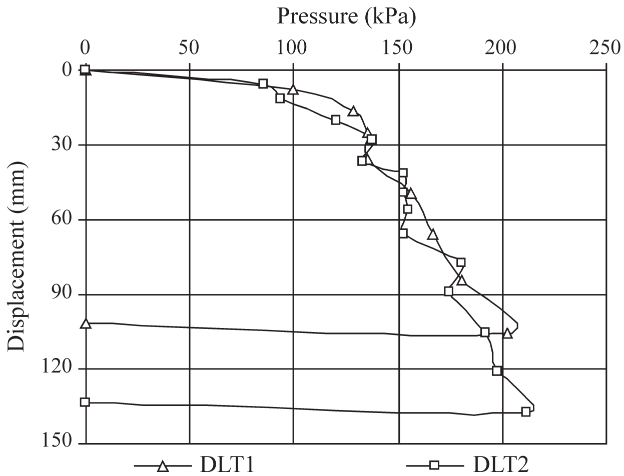

Figure 14 - Results of dynamic load tests.

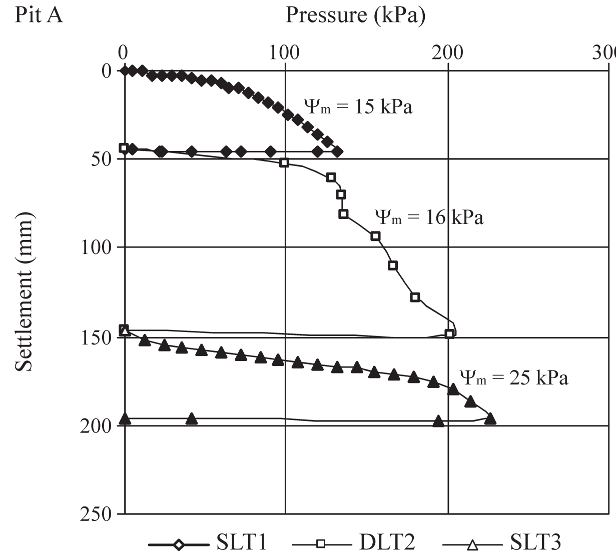

Figure 15 - Static, dynamic and static load tests pressure x settlement curves for tests in pit A.

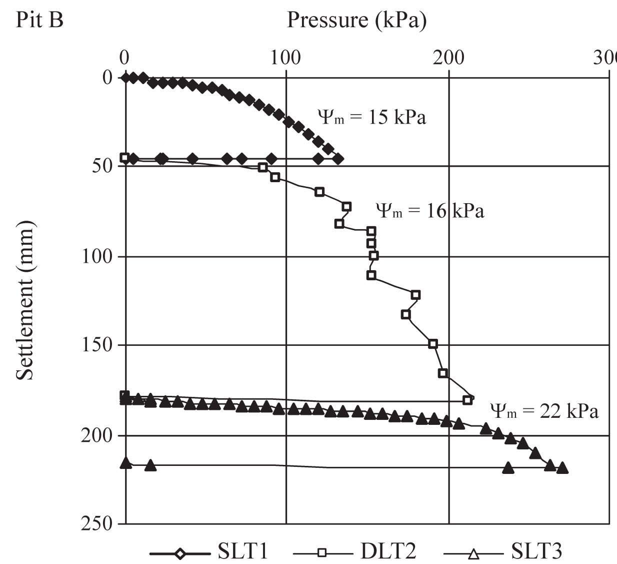

Figure 16 - Static, dynamic and static load tests pressure x settlement curves for tests in pit B.

5. Discussions

5.1. Comparison between static and dynamic plate load tests

Figures 15 and 16 show the pressure x settlement curves obtained from load tests performed in three pits: 1) Costa's (1999) static load test (SLT 1) in pit C; 2) Dynamic plate load tests (DLT 1 in pit A and DLT 2 in pit B) and 3) Static plate load tests performed after the dynamic tests (SLT 2 in pit A and SLT 3 in pit B). The three graphics show the accumulated settlement, according to the tests execution sequence. For the dynamic tests, the static reactive forces obtained through CAPWAP® analysis divided by the plate area are also shown in the figures. The settlement corresponds to the maximum displacement given by the PDA.

The three pressure x settlement curves of each Fig. (15 and 16) show an almost linear relationship between settlement and corresponding applied stress for the plastic strain phase, and almost horizontal unloading line. This kind of curve can also be observed in other load tests at the same location, performed in plates and caissons.

It can be seen in Figs. 15 and 16 that the final part of each pressure x settlement curve from the successive tests in the same pit consists of an almost straight line, as observed by Benvenutti (2001). Disregarding the subsequent unloading and reloading up to the maximum resistance reached at the previous test, a continuous consistent curve can be obtained with reasonable approximation. This continuous curve is formed by linking the three load test curves (static, dynamic and static again) performed on each pit, obtaining a single curve, that has typical shape, found in all plate load tests results at the experimental foundation site.

The dynamic load tests' curves fit reasonably well the two curves obtained from static loading tests. This consistent behavior observed in the static and dynamic tests indicates the viability of the execution the dynamic test with plates, similarly to those already in use for piles.

5.2. Dynamic load test validation through energy approach analysis

Another way found to validate the plate dynamic load test is to compare the energy given by the PDA with the one calculated from the test results. The energy given by the PDA is the total kinetic energy of the system. Figures 17 and 18 show the accumulated maximum applied energy given by the PDA against the calculated accumulated maximum energy, which is the area under the force x settlement curve (Fig. 13). According to Aoki & Cintra (1997), the calculated energy is the total kinetic energy of the system minus the work done and it is typically 70-90% of the total kinetic energy given by the PDA. The average ratio between calculated energy and PDA given energy was equal to 79% for pit A and 87% for pit B, which is consistent with the results reported by Aoki & Cintra (1997).

5.3. Influence of plate penetration on the bearing capacity

For the analyses of the influence of plate penetration on the plate-soil system bearing capacity, the single line formed by the three pressure x settlement curves of each pit was employed.

Since an open curve does not define failure because any stress increment causes soil stiffness to increase, it was necessary to use a conventional failure criterion in order to analyze the successive tests performed on the same plate.

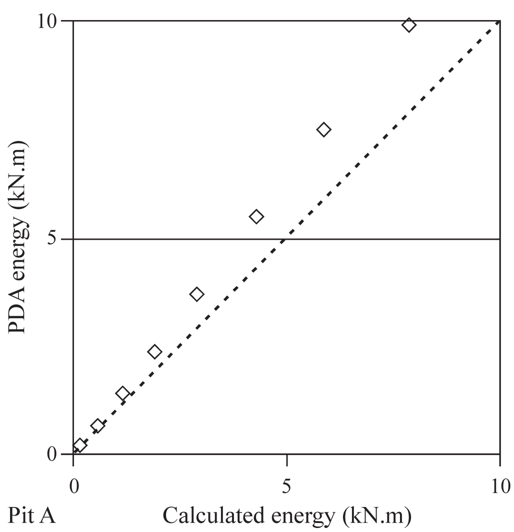

Figure 17 - Energy given by PDA against calculated energy (pit A).

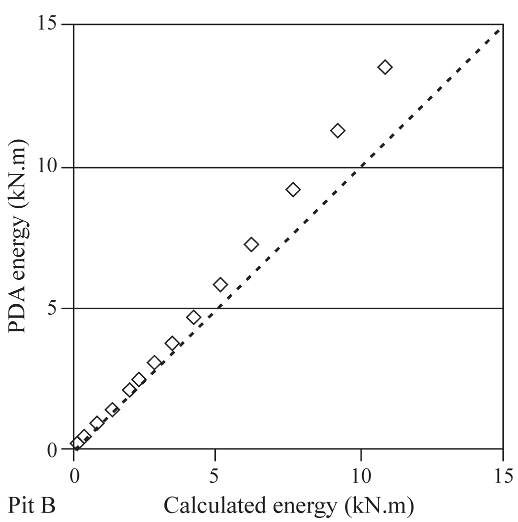

Figure 18 - Energy given by PDA against calculated energy (pit B).

The chosen criterion establishes that the bearing capacity corresponds to a 25 mm settlement. This is a total displacement limit criterion, where the bearing capacity is a function of a pre-determined displacement, based on the Boston construction code. The code establishes that the allowable stress is the smallest between two values: the stress corresponding to 10 mm settlement and the stress corresponding to 25 mm settlement divided by two. Teixeira & Godoy (1998) consider the value two as a safety factor, and therefore the bearing capacity is the stress corresponding to a 25 mm settlement itself. The value 10 mm would be an allowable settlement.

The solution used by Benvenutti (2001) to interpret successive static load tests in two caissons in the same experimental site was used to verify the bearing capacity increase with plate penetration. The failure criterion adopted assumed the bearing capacity as equal to the stress corresponding to an additional 25 mm settlement of the plate from its previous position.

The penetration depths chosen for the beginning of the loading phase were 0, 50, 100 e 150 mm. The value zero means no penetration prior to test. In other words the plate is at the surface in the beginning of the test. The bearing capacity values were considered as the pressures corresponding to 25, 75, 125 and 175 mm settlement, respectively.

Tables 2 and 3 present the bearing capacity values for each pit and the matric suction at the time the test was performed.

Comparing the stresses obtained for a 150 mm penetration with those for zero penetration, there were bearing capacity increases of 105% for pit A (Table 2) and 168% for pit B (Table 3). It can be seen in Fig. 19 that the bearing capacity increases linearly with the plate penetration .

Table 2 - Penetration and bearing capacity for pit A.

Penetration (mm)	Test	σ_r (kPa)
0	SLT1	102
50	DLT1	135
100	DLT1	185
150	SLT2	209

Note: σ_r = bearing capacity.

Table 3 - Penetration and bearing capacity for pit B.

Penetration (mm)	Test	σ_r (kPa)
0	SLT1	102
50	DLT2	153
100	DLT2	202
150	SLT3	273

Note: σ_r = bearing capacity.

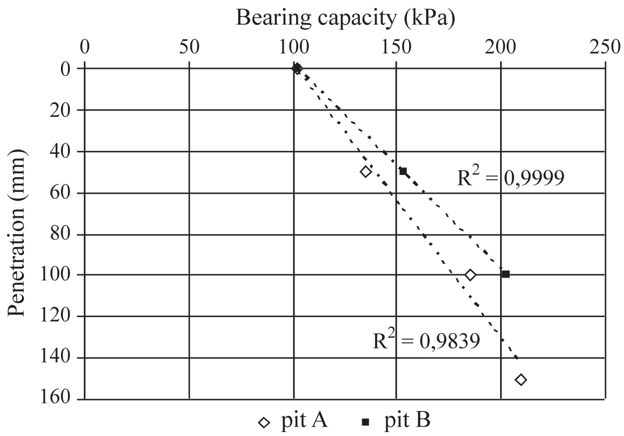

Figure 19 - Relationship between plate penetration and bearing capacity.

6. Conclusions

This paper presented the interpretation of the performance of dynamic load test on rigid circular 0.80 m diameter steel plate. A new dynamic load test approach was validated by the comparison between dynamic and static plate load tests and by dynamic load test analysis using the energy approach.

Because the tests were performed using the increasing energy approach, it was possible to obtain static pressure x displacement curves, similar to the pressure x settlement curves from static load tests.

Comparing the results from CAPWAP® analysis for blows with increasing energy with those obtained from static load tests, performed before and after the dynamic tests, an approximate continuous curve was observed, indicating the viability of the application of dynamic test to plates.

The average ratios between calculated energy from CAPWAP® analysis and the total kinetic energy given by the PDA were 0.79 for pit A and 0.87 for pit B, which are consistent with the results obtained by Aoki & Cintra (1997), validating the dynamic load tests with plates as well as the analysis and models used.

In the two pits described in the paper, four static load tests were performed (two per pit). Two dynamic load tests were performed during the time interval between static load tests. The three individual pressure x settlement curves from each pit were interpreted as a single curve, with reasonable coherence, not considering the unloading and subsequent reloading up to the maximum pressure from the previous loading stage. The final parts of the curves from the successive tests were nearly a straight line. It was observed that the bearing capacity increases with plate penetration. Without considering the penetration, the bearing capacity value obtained was 102 kPa. Considering a plate penetration of 150 mm, an average increase on bearing capacity of 137% was observed. So, another conclusion that could be detached from the studies is the penetration influence on the bearing capacity.

This research contributes to foundation engineering by validating the adaptation of the dynamic load test to a 0.80 m diameter steel plate. The dynamic test can be an alternative to the static load tests.

Acknowledgments

The authors thank the support from In Situ Geotecnia on the dynamic load tests and CNPq for the scholarship granted.

References

ABNT (1991) Piles - Static Load Test, NBR 12131, Rio de Janeiro, 4 p. (in Portuguese).

ABNT (1994) Piles - Dynamic Load Test, NBR 13208, Rio de Janeiro, 12 p. (in Portuguese).

Aoki, N. (1989) A new dynamic load test concept. Proc. XII International Conference on Soil Mechanics and Foundation Engineering. TC Pile Driving, Rio de Janeiro, Discussion Section 14, v. 1, pp. 1-4.

Aoki, N. & Cintra, J.C.A. (1997) New interpretations of the dynamic loading curves for driven piles base on the energy approach. Proc. International Symposium on Recent Developments in Soil and Pavement Mechanics, Rio de Janeiro, v. 1, pp. 467-472.

Aoki, N. (2000) Improving the reliability of pile bearing capacity prediction by the dynamic increasing energy test (DIET). Proc. VI International Conference on the Application of Stress-Wave Theory to Piles, São Paulo, v. 1, pp. 635-650.

Benvenutti, M. (2001) Driving Caissons to Improve its Bearing Capacity in Collapsible Soils. MSc. Thesis, Escola de Engenharia de São Carlos, Universidade de São Paulo, São Carlos, 115 p. (in Portuguese).

Bortolucci, A.A. (1983) Geological-geotechnical characterization of São Carlos' urban area. MSc. Thesis, Escola de Engenharia de São Carlos, Universidade de São Paulo, São Carlos, 67 p. (in Portuguese).

Cintra, J.C.A.; Carvalho, D.; Giachetti, H.L.; Bortolucci, A.A. & Albiero, J.H. (1991). Foundation experimental site in São Carlos. Proc. II Special Foundation Engineering Seminar, São Paulo, v. 1, pp. 96-105. (in Portuguese).

Costa, Y.D.J. (1999) Unsaturated soil behavior study through plate load tests. MSc. Thesis, Escola de Engenharia de São Carlos, Universidade de São Paulo, São Carlos, 131 p. (in Portuguese).

Fellenius, B.H. (1975) Test load of piles and new proof test procedure. Journal of Geotechnical Engineering Division, v. 101:GT9, pp. 855-869.

Macacari, M. (2001) Bearing capacity variation with matric suction and depth from plate load tests in collapsible soil. MSc. Thesis, Escola de Engenharia de São Carlos, Universidade de São Paulo, São Carlos, 96 p. (in Portuguese).

Menegotto, M.L. & Vianna, A.P.F. (2000) Tests performed at EESC-USP experimental foundation site. Internal communication, Escola de Engenharia de São Carlos, Universidade de São Paulo, São Carlos, 30 p. (in Portuguese).

Teixeira, A.H. & Godoy, N.S. (1998) Analysis, design and construction of shallow foundations. Hachich, W.; Falconi, F.; Saes, J.L.; Frota, R.; Carvalho, C. & Niyama, S. (eds) Foundations: Theory and Practice. 2 ed. PINI, São Paulo, chap. 7, p. 227-264. (in Portuguese).

Symbols

DLT: Dynamic load test
e: Void ratio
N_{SPT}: Standard penetration resistance
PDA: Pile driving analyzer
q_c: Cone tip resistance
R_f: Friction ratio
SLT: Static load test
γ: Specific weight
γ_d: Dry specific weight
Ψ_m: Matric suction
σ_r: bearing capacity

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Author index

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Proceedings (printed matter or CD-ROM): Jamiolkowski, M.; Ladd, C.C.; Germaine, J.T & Lancellotta, R. (1985) New developments in field and laboratory testing of soils. Proc. 11th Int. Conf. on Soil Mech. and Found. Engn., ISSMFE, San Francisco, v. 1, pp. 57-153.(specify if CD – ROM)

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Equations shall appear isolated in a single line of the text. Numbers identifying equations must be flush with the right margin. International

System (SI) units are to be used. The symbols used in the equations shall be listed in the List of Symbols. It is recommended that the used symbols be in accordance with Lexicon in 8 Languages, ISSMFE (1981) and the ISRM List of Symbols.

The text of the submitted manuscript (including figures, tables and references) intended to be published as an article paper or a case history should not contain more than 30 pages formatted according to the instructions mentioned above. Technical notes and discussions should have no more than 15 and 8 pages, respectively. Longer manuscripts may be exceptionally accepted if the authors provide proper explanation for the need of the required extra space in the cover letter.

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Discussions must be written in English. The first page of a discussion paper should contain:

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