

**SEXTA
CONFERENCIA RAÚL J. MARSAL**

**Non-linear Shear Strength for Rock, Rock Joints,
Rockfill and Interfaces**

NICK BARTON

2014

SOCIEDAD MEXICANA DE INGENIERÍA GEOTÉCNICA, A.C.

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NICK BARTON

XXVII Reunión Nacional de Ingeniería Geotécnica

**Puerto Vallarta Jalisco, México
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Non-linear Shear Strength for Rock, Rock Joints, Rockfill and Interfaces

Contenido

PREFACIO	ix
SEMLANZA DEL PROFESOR RAÚL J. MARSAL CÓRDOBA	xi
SEMLANZA DEL Dr. NICK BARTON	xvii
QUOTATION	xxi
A RESPONSE in 2004	xxiii
DEDICATION	xxv
1. INTRODUCTION	27
2. FROM TENSION FRACTURES TO ROCK JOINTS	29
3. SCALE EFFECTS FOR ROCK JOINTS	33
4. FROM ROCK JOINTS TO ROCKFILL	35
5. FROM ROCKFILL TO INTERFACES	43
6. DISCUSSION	47
7. CONCLUSIONS	49
REFERENCES	50
APPENDIX	52



PREFACIO

Esta ocasión la Conferencia Magistral Raúl J. Marsal, en su sexta edición, continúa una trayectoria ya arraigada en la ingeniería geotécnica mexicana. Fue instituida por la hoy extinta Sociedad Mexicana de Mecánica de Rocas (SMMR) como homenaje al profesor Marsal por sus notables contribuciones al desarrollo de grandes proyectos en México, como las presas Chicoasén, Peñitas, Caracol y Aguamilpa, entre otras, especialmente en el campo de la Mecánica de Rocas, la SMIG ha dado continuidad a este espacio técnico por la trascendencia de los temas que se pueden abordar en esta especialidad.

La primera Conferencia Marsal fue expuesta por el profesor Jesús Alberro en 1993. Tres conferencias más fueron impartidas durante las siguientes reuniones nacionales de mecánica de rocas, posteriormente, el acervo y actividades de la SMMR se han sumado a la SMIG, por lo que ya en nuestra reunión nacional del 2012 se incluyó en el programa la quinta edición.

Ahora en el 2014 el conferencista invitado es el Dr. Nick Barton, quien amablemente ha impartido ya conferencias y cursos recientemente dentro del programa de sesiones técnicas de la SMIG, por lo cual consideramos que es un reconocimiento a su valiosa contribución a la ingeniería geotécnica y es una garantía de que el contenido de este documento, será un referente de consulta para nuestros asociados y geotecnistas en general dentro y fuera de México.

La conferencia, titulada “Resistencia no lineal al corte de rocas, juntas, enrocamientos e interfaces”, permitirá a los especialistas en estabilidad de taludes, laderas y obras subterráneas, conocer y aplicar los aspectos teórico-prácticos de la misma.

A nombre de la SMIG, agradezco al doctor Barton su dedicación para sumar este material al acervo de la SMIG, así como a nuestro expresidente, Maestro Juan de Dios Alemán y al Maestro Valentín Castellanos, presidente del Comité de Mecánica de Rocas de la SMIG, por las gestiones que han hecho para la edición del presente volumen y por mantener activa la difusión del conocimiento en esta área.

David Yáñez Santillán
Presidente Mesa Directiva 2013-2014
Sociedad Mexicana de Ingeniería Geotécnica
Noviembre de 2014.
Puerto Vallarta, Jalisco, México

X

Sociedad Mexicana de Ingeniería Geotécnica A.C.
Dr. Nick Barton

Sexta Conferencia Raúl J. Marsal



SEMLANZA DEL PROFESOR RAÚL J. MARSAL CÓRDOBA (1995-1990)

Daniel Reséndiz Núñez



Profesor Raúl J. Marsal Córdoba

Raúl Jaime Marsal Córdoba nació en Buenos Aires en 1915 y murió en la Ciudad de México en 1990. Estudió Ingeniería civil en la Escuela de Ciencias Exactas, Físicas y Naturales de la Universidad de Buenos Aires, donde se graduó con honores. Inmediatamente después trabajó un par de años, primero como residente de obras en la construcción de puentes carreteros y luego en el diseño de sistemas de dotación de agua potable y alcantarillado.

Por su calidad como estudiante, su escuela lo becó para estudiar un posgrado en el Massachusetts Institute of Tech-nology (MIT) de los Estados Unidos. En septiembre de 1942 llegó al MIT, donde fue alumno distinguido de Donald W. Taylor y obtuvo el grado de maestro en Ciencias en 1944. En seguida, las estrechas relaciones académicas entre el MIT y la cercana Universidad de Harvard

propiciaron que Marsal decidiera obtener en ésta su doctorado, lo que para él tenía, entre otros atractivos, trabajar bajo la dirección de Arthur Casagrande, el más notable profesor que la mecánica de suelos ha tenido; tomar algún curso con el fundador de la disciplina, Karl Terzaghi, quien enseñaba en Harvard tras haberlo hecho en el MIT, y seguir disfrutando la estrecha relación que había establecido con Taylor. De inmediato tomó los cursos que le interesaban en Harvard y comenzó a desarrollar su tesis doctoral con base en el trabajo de campo y laboratorio que Casagrande había concebido para un proyecto del gobierno de los Estados Unidos: la construcción de un gran relleno que ganaría terreno al mar a fin de alojar en él lo que hoy es el aeropuerto Logan de Boston. Para sorpresa de Marsal, en el ambiente turbulento de la Segunda Guerra Mundial, su permanencia en Harvard se interrumpió súbitamente en 1945 a causa de complejidades de la relación entre Argentina, su país de origen, y los Estados Unidos.

A esas fechas Marsal ya había acumulado amplio conocimiento tanto de su nueva especialidad como del campo general de la ingeniería civil; sus profesores lo reconocían por su honorabilidad, apego al trabajo, perseverancia y habilidad experimental. Tales virtudes habían determinado que, pese a la brevedad de su estancia en Harvard, ya hubiese consolidado una relación profesional y humana intensa y permanente con Casagrande, a tal grado que éste se comunicara de inmediato con Nabor Carrillo, su discípulo dilecto, a fin de buscar acomodo profesional en México para Marsal, con la esperanza de que los problemas migratorios de éste en Estados Unidos se resolvieran pronto y pudiera volver a Harvard. Carrillo consiguió para Marsal una beca de la Comisión Impulsora y Coordinadora de la Investigación Científica (antecesora del Conacyt) para emprender el estudio del hundimiento de la Ciudad de México, lo que Casagrande le ponderó como una alternativa más que apetecible al proyecto de tesis doctoral iniciado en Logan. Además de ocuparse de tal estudio, Marsal asesoraba a la Comisión Nacional de Irrigación, donde se encontró con Fernando Hiriart, quien pronto se convirtió en muy cercano amigo con quien Marsal a partir de entonces realizó numerosas actividades profesionales.

Su fortuita llegada a México en 1945 fue un suceso trascendente para Marsal, pues aquí habría de llevar a cabo una labor admirable. Más importante aún sería su presencia para quienes después tuvimos la oportunidad de formarnos bajo su tutela, y en general para el país. La generosidad de Marsal, esa rara virtud, hizo de él un ingeniero y un científico fuera de lo común. Seguro de sus actos y convencido de que laboraba para los demás, siguió siempre una trayectoria de exigencias a sí mismo y, de modo indirecto y tácito, a quienes quisimos trabajar con él. Lo vi de lejos por primera vez a fines de 1957, cuando con motivo del sismo que ese año golpeó a la ciudad él impartió, junto con Fernando Hiriart y Emilio Rosenblueth, una conferencia sobre los estudios que al respecto estaba haciendo el recién fundado Instituto de Ingeniería. Era yo estudiante del tercer año de licenciatura en la entonces Escuela Nacional de Ingenieros de la UNAM; no entendí todo lo que ahí se dijo, pero quedé impresionado por los tres conferencistas, tan diferentes entre sí y cada uno con una grata personalidad. Tampoco imaginaba entonces el papel que después y durante

el resto de sus respectivas y fructíferas vidas tendrían los tres en mi formación y mi actividad profesional.

Dos años después, cuando ya estaba yo terminando mi licenciatura y pensando en qué haría después, Nabor Carrillo me dijo un día que fuera a ver en el Instituto al profesor Marsal; lo hice y cinco minutos después ya era yo su ayudante, sentado ante una mesa, con una pila de papeles con tablas y ecuaciones, más una calculadora mecánica, escuchando escuetas instrucciones de él sobre cálculos por realizar porque los necesitaba con urgencia para el capítulo XIII (Estudios teóricos sobre el hundimiento), último por terminar de su libro *El subsuelo de la Ciudad de México*, cuyo coautor era Marcos Mazari. Su publicación a fines de 1959 marca el fin de la primera gran contribución de Marsal a la ciencia y la ingeniería, casi 15 años después de haberla emprendido; con ella se clarificaron fuera de toda duda las causas y la evolución pasada y futura del hundimiento del Valle de México; fue un trabajo monumental de ensayos de laboratorio, instrumentación, observaciones de campo y estudios analíticos que luego permitió la zonificación geotécnica y sísmica del valle, un mejor reglamento de construcciones para la ciudad, bien fundados criterios de diseño para cimentaciones, obras subterráneas y estructuras diversas en el valle y, por extrapolación ante condiciones similares, en otros sitios del mundo.

La siguiente gran tarea que como investigador emprendió Marsal, ésta con visionario y generoso patrocinio de la Comisión Federal de Electricidad (CFE) conseguido por Hiriart, fue dilucidar aspectos hasta entonces no estudiados del diseño y construcción de grandes presas terreas. Su contacto con este tipo de obras databa de los años cincuenta, en los que fue consultor de la Comisión Nacional de Irrigación y otros organismos gubernamentales, pero su esfuerzo concentrado en el tema comenzó en 1960 y ocupó los siguientes 16 años de su vida. La cuestión era de interés en todo el mundo, y en México la CFE estaba a punto de emprender la construcción de una serie de esas obras. El primer proyecto sería el de El Infiernillo y luego vendrían La Villita, Malpaso, La Angostura, Chicoasén, etc. Ocupado en mi tesis de licenciatura y estudios de posgrado por indicación del propio Marsal, no pude sino ocasionalmente participar en los estudios de la presa El Infiernillo, pero poco después de concluido el doctorado que él mismo había prescrito para mí, me propuso colaborar a tiempo parcial en la asesoría técnica de la CFE. Él coordinaba desde ahí el trabajo de campo del gran proyecto para modernizar el diseño de este tipo de presas; ahí comenzó también mi interacción con Hiriart durante la planeación y el diseño de las tres primeras grandes presas del Grijalva. Después me propuso ser coautor del libro *Presas de tierra y enrocamiento*, que concluimos en 1976 con algunos capítulos aportados por distinguidos ingenieros mexicanos.

Antes de que Marsal emprendiera la investigación referida, el diseño de presas de enrocamiento era un arte esencialmente intuitivo: el pedraplén, componente principal de estas obras, se construía con normas simplemente empíricas, pues no había equipo ni técnicas para medir sus propiedades en función de su composición, granulometría y grado de compactación. Tampoco había registros suficientemente finos del comportamiento de estas obras, pues no existía la instrumentación

apropiada para registrarlo. Marsal no se detuvo ante el reto de ambos obstáculos: diseñó, probó y desarrolló equipo para ensayar masas de enrocamiento de varios metros cúbicos bajo esfuerzos equiparables a los que ocurren en la base de presas más altas que las hasta entonces construidas, e instrumentos robustos y confiables para medir esfuerzos y deformaciones en el interior de esas estructuras.

El primer resultado de este ambicioso programa fue un laboratorio peculiar, el primero del mundo capaz de medir las propiedades mecánicas de enrocamientos bajo condiciones de deformación triaxial, unidimensional y plana. Un resultado subsecuente fue el diseño más racional de las presas, primero en México e inmediatamente después en otras partes del mundo. En el aspecto científico, se descubrieron hechos sorprendentes del comportamiento de medios granulares bajo grandes fuerzas de contacto entre sus partículas y nació un nuevo campo de estudio. Estos resultados, más la instrumentación y el monitoreo sistemático de todas las nuevas presas que se fueron construyendo, crearon un acervo de nuevos conocimientos que mejoraron radicalmente la construcción de estas obras y abrieron la posibilidad de realizar de modo más seguro los mayores proyectos de este tipo que fueron siendo necesarios en cualquier país.



El profesor Marsal acompañado por sus discípulos Jesús Alberro, Daniel Reséndiz y Sergio Covarrubias durante el Séptimo Congreso Internacional de Mecánica de Suelos e Ingeniería de Cimentaciones, Moscú, 1973. En el extremo izquierdo, el ingeniero Roberto Graue.

La velocidad con que las contribuciones de Marsal penetraron en la práctica de la ingeniería en México y el mundo se debió desde luego al sólido, amplio y convincente soporte científico y técnico que las caracterizaba, pero también al carácter y la contextura moral de su autor, sin sombra de egoísmo ni reserva al aconsejar a otros cómo usarlas, así como a su disposición para colaborar en ello desinteresadamente con quien lo quisiera.

Cada una de las dos extensas contribuciones de Marsal aquí comentadas le exigió tres lustros de trabajo paciente y sin desmayo: 30 años en total. Si se revisa lo que en cada caso logró, uno encuentra que en los dos campos él generó e integró un nuevo sistema de conocimientos ampliamente aceptado, tan firme y coherente que parece haber existido desde siempre. Suele haber la pretensión de que lo moral es ajeno o irrelevante en el desarrollo de la ciencia y la ingeniería. La obra de Marsal tiene, por supuesto, un valor intrínseco independiente de su autor y de lo que se haga con ella, pero para que esa obra trascendiera, fuera reconocida a plenitud y aceptada con rapidez, fue determinante la transparencia, el altruismo y el desprendimiento generoso de su autor.

Fuente: Resendiz-Núñez, D. (2011) “Recordar a Marsal”, semblanza, *revista Geotecnia, órgano oficial de la Sociedad Mexicana de Ingeniería Geotécnica*, A.C., No. 220, 10-13

SEMLANZA DEL DR. NICK BARTON SEXTO CONFERENCISTA RAÚL J. MARSAL

Juan de Dios Alemán Velásquez



Dr. Nick Barton

Nicholas R. Barton, nació en Inglaterra en 1944, pero vivió en Wales entre 1957 y 1963, ahí estudió en Oxford; luego se trasladó a Londres donde realizó sus estudios universitarios en Ingeniería Civil entre 1963 y 1966 en el King's College y los de doctorado que culminó en 1971 en el Imperial College. Su trabajo de investigación fue en mecánica de rocas enfocado a la estabilidad de taludes en roca, el cual aportó las bases para sus posteriores estudios del comportamiento mecánico de discontinuidades en roca y su criterio de resistencia al corte pico.

Al comienzo de su carrera trabajó en el Norwegian Geotechnical Institute (NGI) en Oslo, entre 1971 y 1980 colaborando con el grupo de presas y rocas. En esta etapa de su vida desarrolló algunas de sus principales aportaciones a la Mecánica de Rocas: el sistema Q de clasificación de macizos rocosos para el diseño de revestimientos de túneles publicado en 1974, conocido posteriormente como NMT (Método Noruego de Túneles), y su criterio de resistencia al corte pico de discontinuidades en roca, desarrollado en 1973, modificado en 1977 con el reemplazo del ángulo de fricción básico por el ángulo de fricción residual, y mejorado en 1978 involucrando los conceptos de movilización y degradación de la rugosidad de juntas con el desplazamiento en el coeficiente de rugosidad de la junta, JRC.

Entre 1981 y 1984, Barton trabajó en Terra Tek (hoy Schlumberger) en la ciudad de Salt Lake, Utah, Estados Unidos, el último año como Gerente de la División de Geomecánica. Ese año también ocupó el cargo de profesor adjunto del Departamento de Minas de la Universidad de Utah. Luego, regresó al NGI como Director de la División de Presas, Rocas, Túneles, Deslizamientos y Yacimientos Petroleros, cargo que ocupó por cinco años, lo cual combinó con su estancia como profesor visitante en la Universidad de Luleå en Suecia. De sus trabajos de investigación en estos años de carrera se destaca el desarrollo de su criterio de resistencia al corte de enrocamientos (1981), su participación como co-desarrollador de la ley constitutiva para el modelamiento de discontinuidades en roca o modelo Barton-Bandis en 1982 y su posterior adición al Código UDEC-BB de Cundall en 1985.

Entre 1990 y 1999 pasó a ser Asesor Técnico de la División de Ingeniería de Rocas y Mecánica de Yacimientos de la NGI, y en 2000 su consultor internacional; también fue profesor visitante de la Universidad Politécnica de São Paulo (1997-2001). Durante este periodo Barton presentó dos aportaciones importantes a la mecánica de rocas: en el año 1995, mejora su sistema Q considerando la resistencia a la compresión simple de la roca en el parámetro Q_c , el cual correlaciona bien con las velocidades sísmicas y el módulo de deformabilidad; más tarde, en 1999, desarrolla el método QTBM para predecir el rendimiento de escudos sencillo y doble, y estimar el refuerzo y soportes necesarios en túneles excavados con tuneleadora en roca fracturada y fallada. En el año 2000 publica su libro “TBM tunnelling in jointed and faulted rock”, en el cual desarrolla el Método de Predicción QTBM, que hoy en día se utiliza extensivamente en proyectos de excavación de túneles con tuneleadora (TBM).

En 2000 formó su propia empresa de consultoría internacional en ingeniería de rocas, Barton & Associates, con base en las ciudades de Oslo y São Paulo. Su criterio de resistencia al cortante para la interfaz enrocamiento-roca de cimentación desarrollado en 2004, es de sus más recientes aportaciones.

Barton es autor y co-autor de 290 artículos publicados, y autor del libro ‘Rock Quality, Seismic Velocity, Attenuation and Anisotropy’ publicado en 2006, considerado como el mayor libro de texto sobre temas de Mecánica de Rocas.

En los últimos cuarenta años ha sido consultor internacional en 35 diferentes países, participado en proyectos relacionados con el diseño, el seguimiento y la revisión de numerosos túneles y cavernas, en análisis de subsidencia de depósito, en las mediciones de esfuerzos en roca, y la caracterización y disposición de residuos nucleares. Ha colaborado en proyectos de modelación, diseño e investigación en los EE.UU., Canadá, Suecia y el Reino Unido, y de caracterización y estimación de la resistencia al corte de macizos rocosos en proyectos de presas de arco en Irán, Turquía y China.

Nuestro conferencista Raúl J. Marsal ha sido merecedor de siete premios internacionales incluyendo dos premios de la sociedad profesional de los Estados Unidos y conferencias

conmemorativa de Portugal, Noruega y Croacia por su trabajo en el desarrollo de túneles de roca y comportamiento de macizos rocosos fracturados. En 2004 recibió un grado de *Doctor Honoris Causa* por la Universidad de Córdoba, Argentina y en 2011 fue ganador del sexto premio Müller de la Sociedad Internacional de Mecánica de Rocas, ISRM. Este premio, que se entrega cada cuatro años, honra la memoria de Profesor Leopold Müller, fundador de la ISRM, y se le otorgó al Dr. Nick Barton en reconocimiento a sus distinguidas contribuciones a la Mecánica de Rocas y a la Ingeniería de Rocas.

QUOTATION

Terzaghi (1920) quoted by Marsal (1973)

The fundamental error was introduced by Coulomb, who purposely ignored the fact that sand consists of individual grains, and who dealt with the sand as if it were a homogeneous mass with certain mechanical properties. Coulomb's idea.....developed into an obstacle against further progress as soon as its hypothetical character came to be forgotten by Coulomb's successors.

RESPONSE in 2014

The ‘Coulomb error’ is still present in rock mechanics. In fact most rock-related strength envelopes are non-linear, the exception being planar rock joints. Even rough-surfaced rock joints have no cohesion unless with actual steps preventing shearing. If a rock mass has a cohesive component due to intact ‘bridges’ (or such steps) it is not correct to add the cohesive and frictional components: they are mobilized at successively larger shear strain or displacement. Rock masses are also ‘particulate’ media, with blocks instead of sand grains or gravel or stones.

Use of linear Mohr-Coulomb is a tragic error pervading structural geology, petroleum engineering and a lot of rock mechanics. It gives many misleading results in relation to reality, including an exaggeration of petroleum reservoir production potential in the weaker rock types.

DEDICATION

Thirty five years ago, while preparing a paper summarizing some work on the shear strength of rockfill (Barton and Kjærnsli, 1981), the writer came across some of the work by the person we are now honouring. With degrees from MIT and Harvard and lectures from Terzaghi and Casagrande, and more importantly a distinguished career in famous institutes in Mexico City, it is very clear that Raul Marsal was a person whose work had a high status in the world of rockfill and embankment dam engineering.

One of his landmark papers was the following: Marsal, R.J. 'Mechanical Properties of Rockfill', Embankment-Dam Engineering, Casagrande Volume, R.C. Hirshfeld and S.J. Poulos, eds., John Wiley and Sons, Inc., New York, N.Y., 1973, pp.109-200. Note the large number of pages: this was a major chapter by Dr. Marsal, honouring Casagrande. I quote from part of his abstract:

The analysis and measurement is described of particle breakage, which is a function of the contact forces and the crushing strength of the particles. The devices developed to investigate the shear strength and stress-strain characteristics of rockfills and the techniques for preparation of specimens are described, as well as the measurement of shear strength of rockfill in the triaxial and plane-strain apparatus.

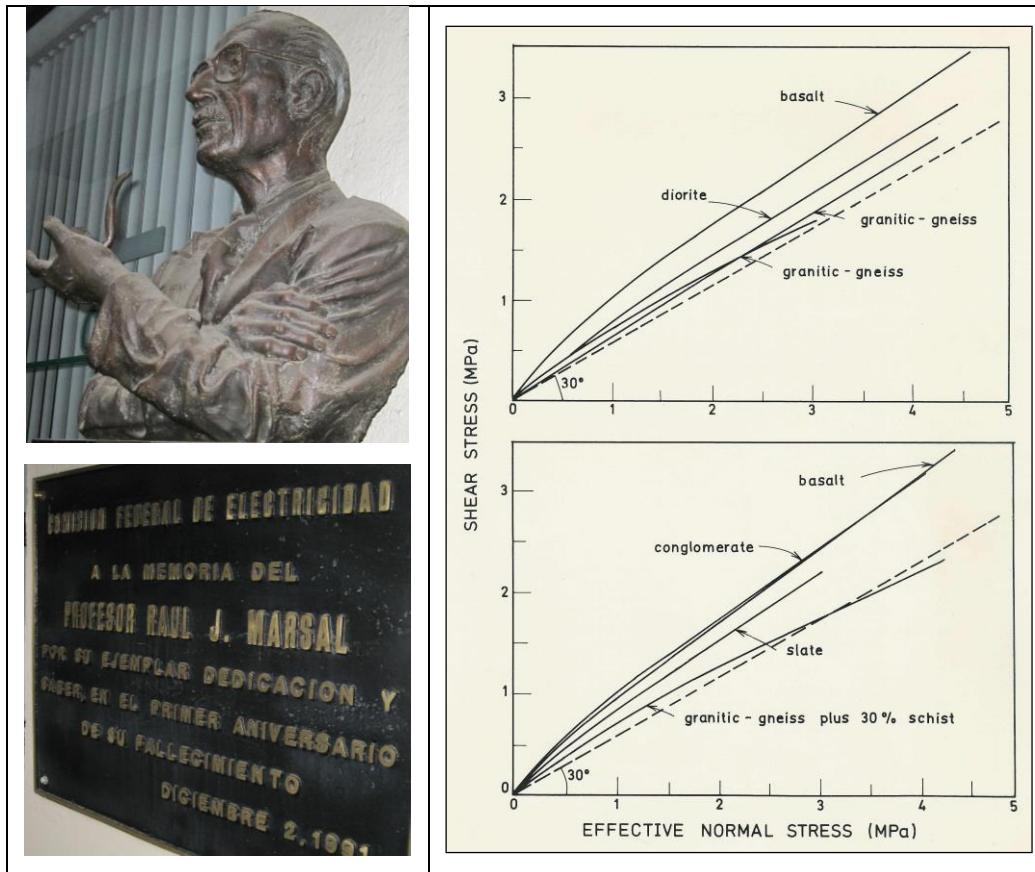


Figure 1 The significantly non-linear shear strength of various rockfills, as interpreted by Marsal, 1973. These simple curves were immediately a source of inspiration to the writer, because they looked just like the shear strength of rock joints (Barton, 1973). There are several reasons why this is so, and these will be detailed in this paper. It obviously has to do with the highly stressed points of contact in both cases.

Non-linear Shear Strength for Rock, Rock Joints, Rockfill and Interfaces

1. INTRODUCTION

Although intact rock is frequently represented by linear Mohr-Coulomb shear strength envelopes, the actual true behaviour, if taken over a wide range of confining stress, is extremely non-linear. Why is this important? Probably because the real stress across points of contact in both rockfill and rock joints, is approaching (or trying to exceed) the crushing strength, and if local confined stresses are equally high, strong non-linearity will be experienced. If we utilize the unconfined compression strength of the rockfill, or of the rock joint surfaces, suitably scaled-down due to size effects, we are part way towards a useful strength criterion. In fact, as we will see, we are ‘one third’ of the way, as the roughness (of particles and asperities) and a measure of the non-dilatant (residual) frictional strengths are also needed for what we will see are two very closely related strength criteria.

Figure 2 is a representation of the complete shear strength envelope for intact rock, as suggested by Barton, 1976 following a wide review of other researchers high-pressure triaxial data for intact rock, in particular the well-known and numerous studies of Byerlee and Mogi from the nineteen sixties. Both these researchers were concerned with the brittle-ductile transition. The present author suggested a simple ‘definition’ of the top (horizontal) part of the strength envelopes - in rock mechanics terminology called the ‘critical state’. The complete shear strength envelopes of rock, and how much they deviate from linear Mohr-Coulomb has recently been quantified in a new criterion which may become known as the Singh-Singh criterion (Singh et al. 2011).

The horizontal part of the shear strength envelopes for a large group of silicate and carbonaceous rocks, suggested the following simple relation:

$$\sigma_{1 \max} = 3 \sigma_{3 \text{ critical}} \quad (1)$$

It will be noted that the uniaxial (unconfined) circle (#2) and the critical confining pressure circle (#4) are drawn as nearly tangent to one another. This potential simplicity has recently been confirmed by Singh et al. (2011), who found that the majority of rocks exhibited this tendency, i.e. $\sigma_{3 \text{ critical}} \approx \sigma_c$. This actually implies that if we reach a confining pressure (over the small area/volume in contact) approximately equal to UCS or σ_c , a local critical state may potentially be reached if (less confined) crushing has not already occurred. The maximum local rock strength will likely

have been exceeded, both for the case of rock joint asperities and the contacting areas/volumes of rockfill particles/stones.

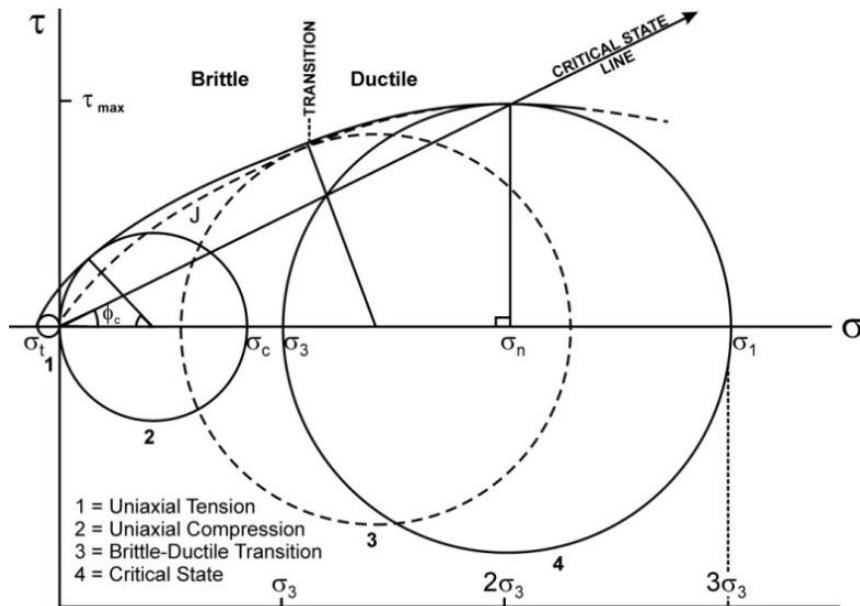


Figure 2 When tested over a wide range of confining stress, the non-linearity of the shear strength envelopes for rock is very marked. Note the closeness of Mohr circles #2 and #4 (Barton, 1976).

In Figure 3 we see the near ‘super-position’ of two envelopes (rock joints and rockfill). It should be noted that the top three envelopes, together with the fifth (lowest) envelope, symbolize the four components of the shear strength of rock masses. For the last 50 years, since the beginning of rock mechanics, this important *in situ* strength has been assumed to consist of the Mohr-Coulomb $\tau = c + \sigma_n \tan \phi'$ (or more recently as the non-linear complex-algebra GSI-based Hoek-Brown criterion). In reality, contrary to the M-C or H-B assumptions, the shear strength of rock masses consists of the degradation of the cohesion (the intact ‘bridges’) followed at larger strain by the mobilization of friction and roughness causing dilation. At still greater shear ‘strain’ any clay-filled discontinuities present may also be mobilized. The failure is therefore a progression of components mobilized at different levels of shear ‘strain’ (or displacement).

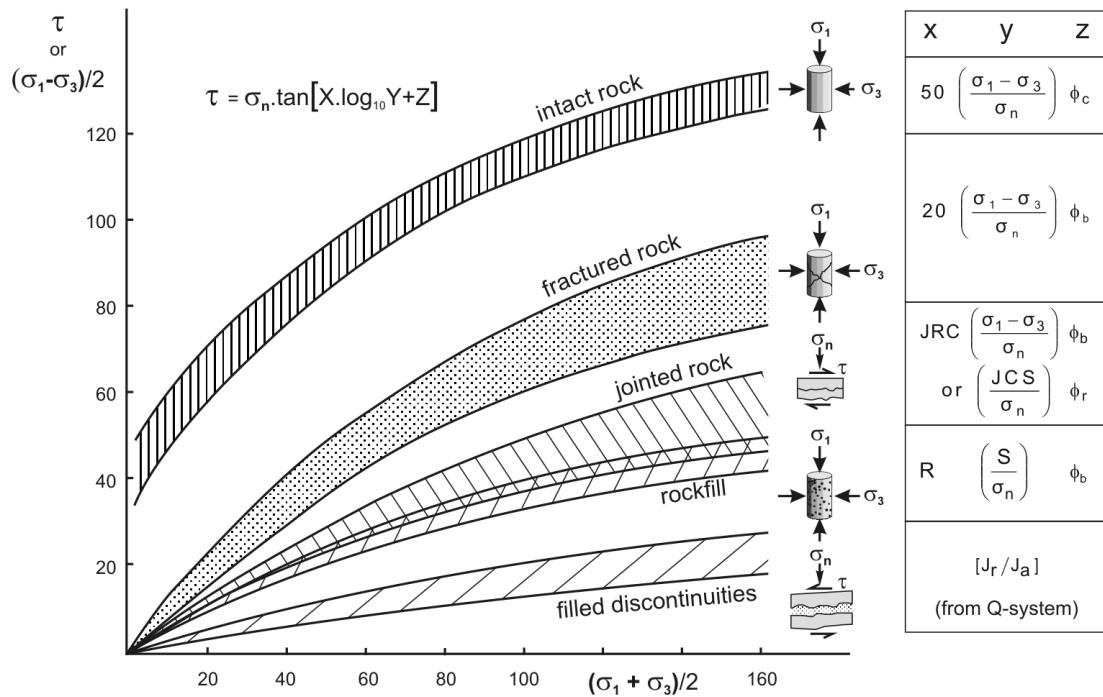


Figure 3 The basic non-linearity of rock, rock fractures, rock joints, and rockfill. Note the deliberate ‘super-position’ of the strength envelopes for the rock joints and rockfill. Barton (2006).

2. FROM TENSION FRACTURES TO ROCK JOINTS

A promising start to an understanding of the shear strength of rough rock surfaces with opposed asperities in contact was experienced by the writer in about 1968, while developing a method to create ‘2D’ models of variously jointed (fractured) rock masses. Figure 4 (top-left) shows shear-displaced roughness profiles of brittle tension fractures which have been sheared and dilated exactly as measured in direct shear tests of the same (2D profiled) samples. Very small areas of contact (and therefore high contact stresses) are seen as a result of the pre-peak and post-peak shearing. The direct shear tests produced strongly curved peak-strength envelopes, especially at very low normal stresses, as shown in the top-right diagram. (Such results were the forerunner of tilt tests developed 10 years later, illustrated in Figure 5). The two sets of experimental data with results expressed as a function of peak dilation angles (bottom-left) required a small rotation *to the dotted lines*, to produce the forerunner of the Barton (1973) non-linear peak strength criterion. In 1968 ‘JRC’ was 20 (very rough tension fractures), ‘JCS’ was UCS (due to no weathering), and ϕ_b was 30° (also due to no weathering).

The terms JRC (*joint roughness coefficient*) and JCS (*joint wall compression strength*) were first used in Barton (1973). In subsequent experimental work with fresh and partly weathered rock joints (130 samples), acquired from road cuttings in the Oslo area, Barton and Choubey (1977) showed

that ϕ_b should be replaced by ϕ_r which could be several degrees lower in cases with weathered joint surfaces. The writer developed the *tilt test* at this time, as a formal way to back-calculate the variable joint roughness JRC. Examples of this simple technique are shown in Figure 5. (Do three tilt-tests per sample plus roughness profiling).

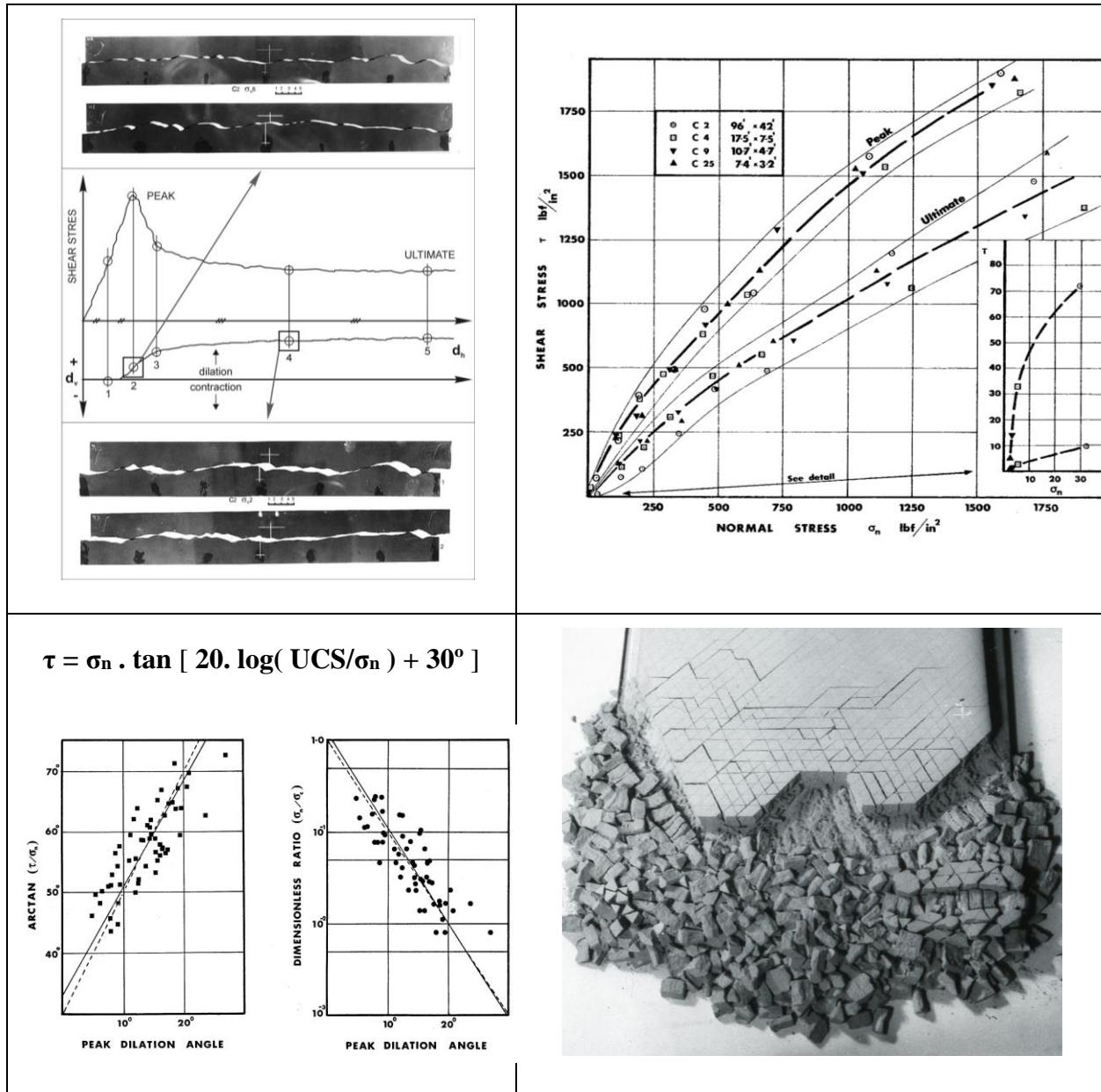


Figure 4 The origins of the non-linear shear strength criterion for rock joints (Barton, 1973) was the direct shear tests performed on 200 tension fractures in various weak, brittle model materials, which were designed to represent jointed rock masses in 2D, in fact a forerunner of the much more flexible UDEC of Cundall, who was a fellow Ph.D. colleague at Imperial College. Note that the prototype (up-scaled) stress levels (lbf/in^2) and sample sizes (feet) are given in the top-right diagram.

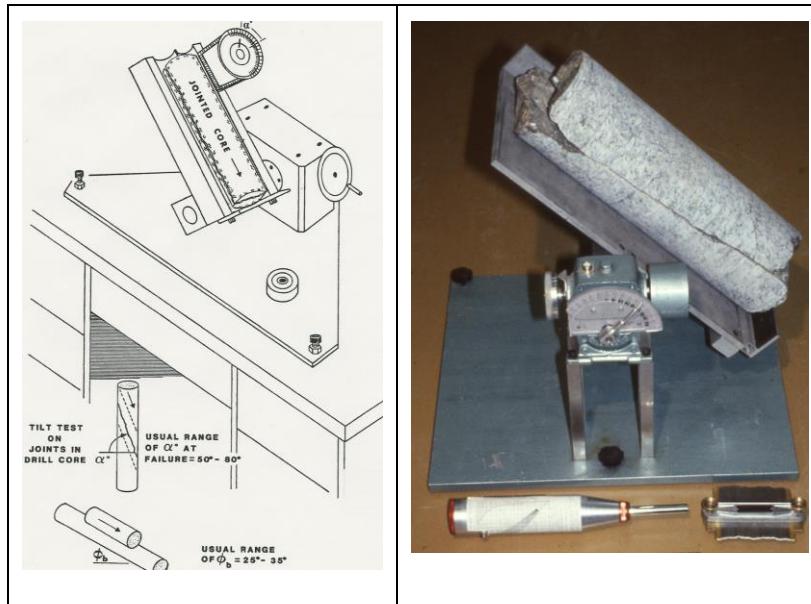


Figure 5 Tilt tests for back-calculating the joint roughness coefficient JRC. When sliding occurs (usually at tilt angles of 40° to 80°) the gravity loading ensures a uniform shear stress and a uniform normal stress of only about 0.001 MPa. The non-linear form of the strength criterion allows this JRC value to be used at three or four orders of magnitude higher normal stress, e.g. 1 to 10 MPa in rock mechanics design studies. Note the two-core-sticks test for ϕ_b (no polishing, no ridges, use sand-blasting if in doubt). Also shown is a Schmidt hammer and a roughness-profile gauge. The apparatus on the left was used when the writer was at TerraTek (now Schlumberger) and represents one of the simplest rock mechanics tests.

Figure 6 shows some of the joint samples tested (with increasing roughness from #1 to #10, and two sets of shear strength results. The three shear strength criteria (M-C, Patton, and Barton/Choubey) are compared. By chance the writer's career in rock mechanics started in 1966, just after the 1st ISRM congress in Lisbon, where Patton (1966) presented his bi-linear equation, which was developed from tests on interlocked 'saw-teeth'. It was an immediate goal to try to improve upon this ' $\phi + i$ ' criterion, since ' i ' was not sufficiently defined. In due course of time it was found that ' i ' was both *stress and scale* dependent. (The term $JRC_n \log_{10}JCS_n/\sigma_n$ is required).

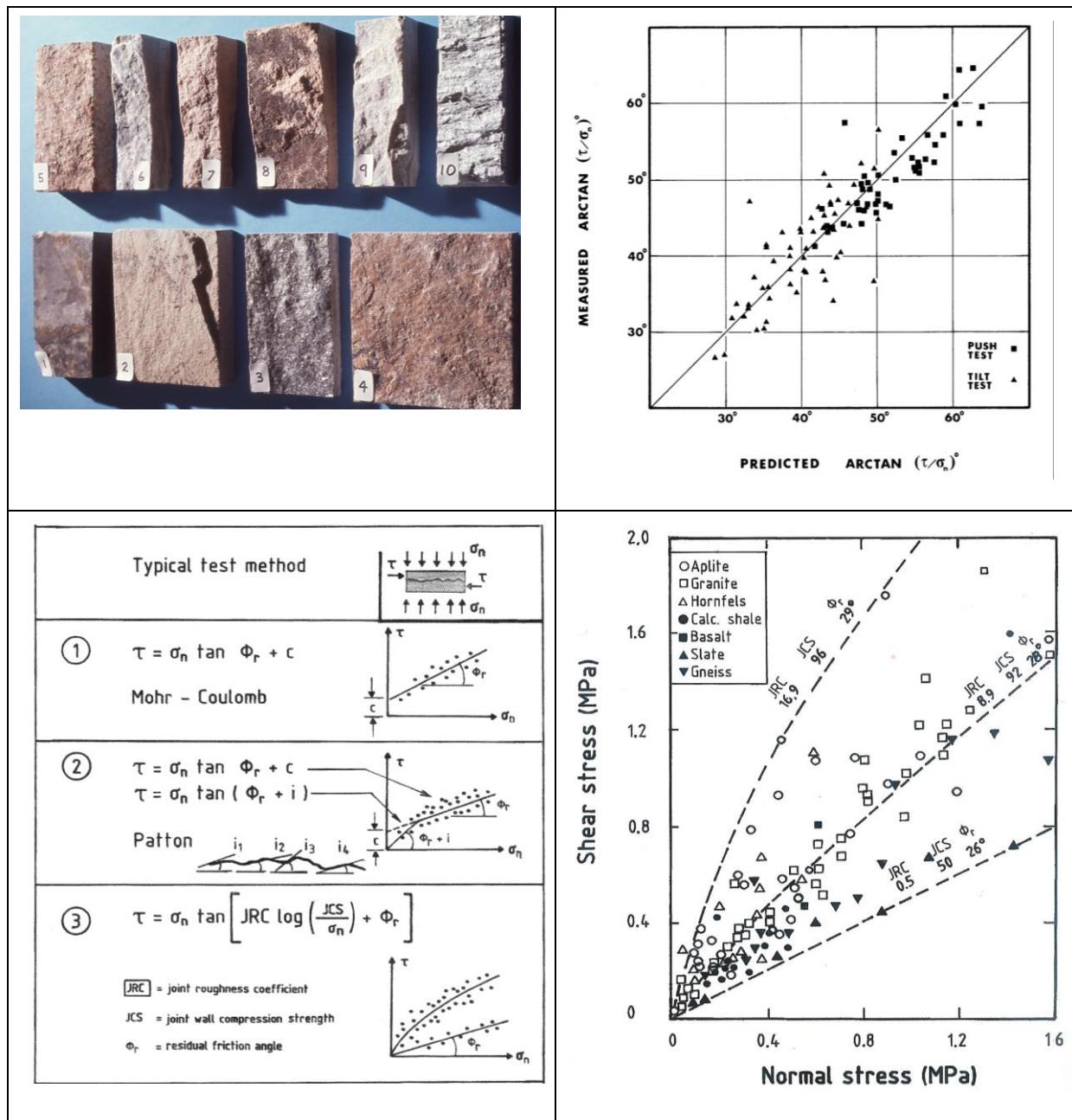


Figure 6 A brief summary of Barton and Choubey (1977). Examples of the 130 joint samples are shown top-left. (However, sample #6 was one of six artificial tension fractures). Top-right shows peak friction angles measured in DST compared with the prediction based on gravity-loaded tilt tests and push tests. Good correlation is shown in general, over a wide range of peak shear strengths. The results are also plotted as τ versus σ_n (bottom-right), and three strength envelopes are drawn (dashed-lines) using maximum strength (JRC = 16.9), mean values of JRC, JCS and ϕ_r , and minimum values. The non-linear criterion (#3) is compared with Mohr-Coulomb and with Patton, 1966 (bottom-left). Note that rock joints do not have cohesion unless steep steps are sheared through. But peak total friction angles (including friction, dilation and the asperity strength component) can be very high at low normal stress.

3. SCALE EFFECTS FOR ROCK JOINTS

A ‘final’ stage concerning the writer’s non-linear shear strength criterion was the correction of the input parameters JRC and JCS for scale. As we shall see later, a similar scale correction was also necessary for rockfill. The impetus for scale corrections was the result of a milestone series of direct shear tests on joint replicas of different size. By happy coincidence Bandis started his

Ph.D. studies just after the work of Barton and Choubey (1977), and made active use of JRC and JCS to record strong scale effects. Figures 7b and 8 summarise what Bandis found. As may be noted from Figure 7a, performing tilt tests on jointed blocks is also a scale-dependent process.

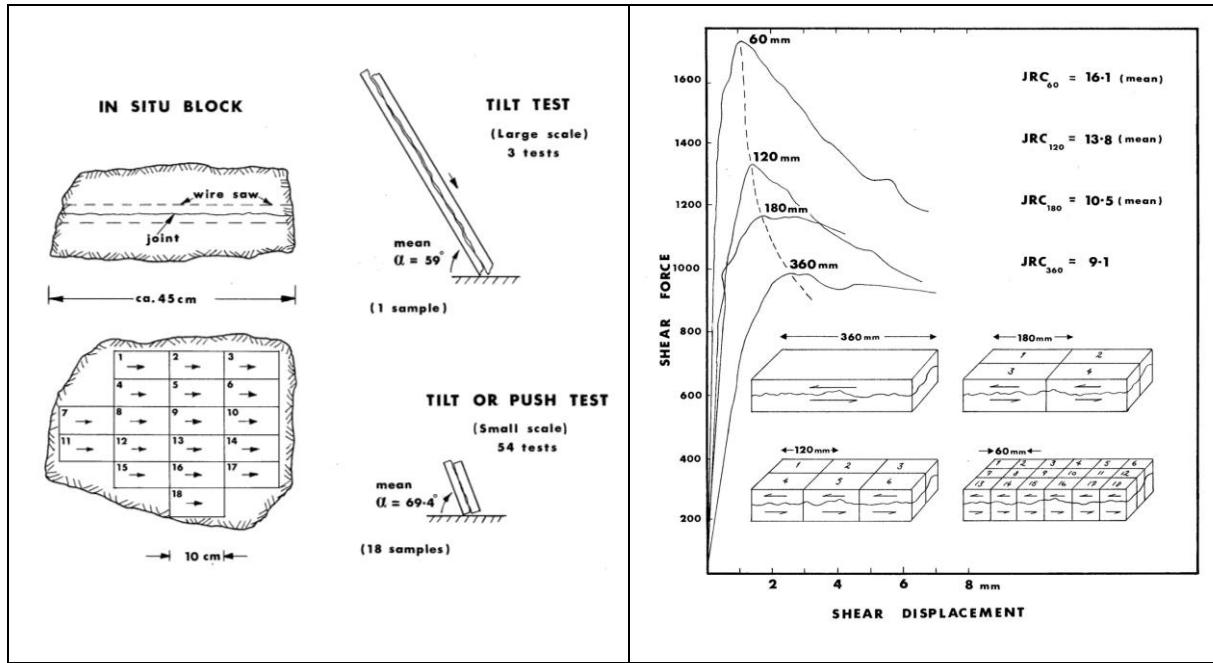


Figure 7 Left: Tilt tests performed on a joint in granite by Barton and Choubey (1977). Right: direct shear tests of joint replicas by Bandis (1980) performed during his Ph.D. studies, published in Bandis et al. (1981). Each of the above indicate that block-size dependent scale-corrections are needed for JRC.

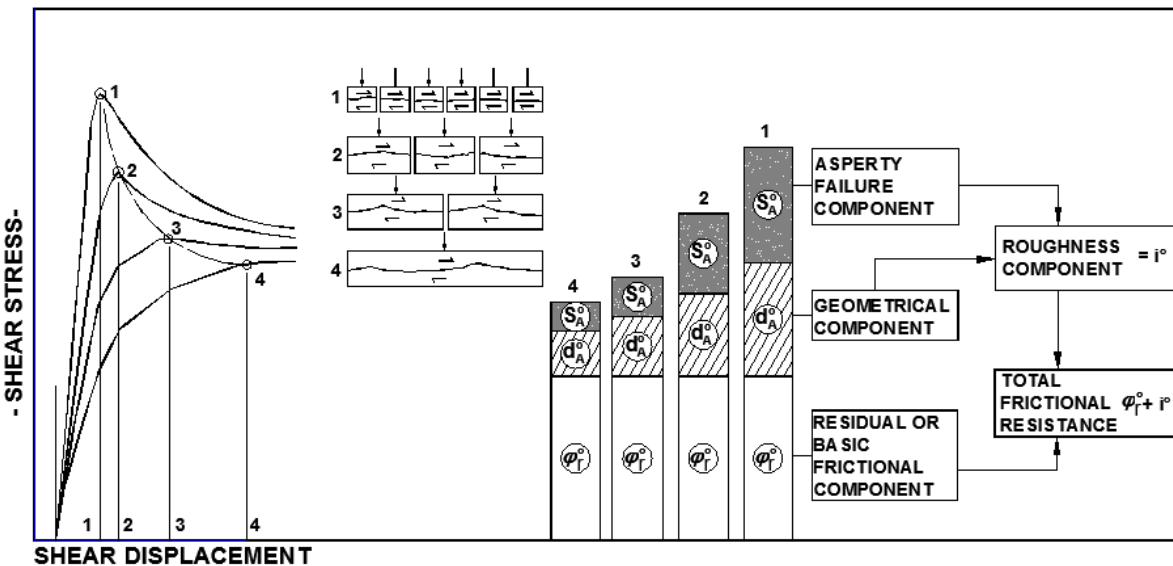


Figure 8 Two of the key angular components of the shear strength of rock joints (SA = asperity failure component, and d_n = peak dilation angle) were shown by Bandis et al. (1981) to be scale dependent. The parameters JRC and JCS are involved in both these parameters. This figure illustrates why it is not correct to subtract the dilation angle to obtain a ‘basic friction angle’, as proposed many times by Hencher, because peak strength includes an asperity strength component. This error can amount to 10° as Hencher proposes ‘basic friction angles’ as high as 40° for rock slope design, on to which roughness is added. The problem is that these so-called ‘basic friction angles’ are (unintentionally) scale-dependent, as clearly shown in this figure from Bandis. Scaling rules for JRC and JCS to obtain JRC_n and JCS_n are shown in Figure 9. They were published in Bandis et al. (1981).

The various stages of development of the shear strength criterion for rock joints, starting with artificial tension fractures, and ending with weathering and scale-corrections are listed below. As we shall see, the proposed empirical equation for the shear strength of rock fill, and for rockfill/rock-foundation interfaces, have their origin in the highly-stressed asperities shown in Figure 4a.

$$\tau = \sigma_n' \tan [20 \log_{10} (\text{UCS}/\sigma_n') + 30^\circ] \quad (\text{Barton}, 1971) \quad (2)$$

$$\tau = \sigma_n' \tan [\text{JRC} \log_{10} (\text{JCS}/\sigma_n') + \phi_b] \quad (\text{Barton}, 1973) \quad (3)$$

$$\tau = \sigma_n' \tan [\text{JRC} \log_{10} (\text{JCS}/\sigma_n') + \phi_r] \quad (\text{Barton and Choubey}, 1977) \quad (4)$$

$$\tau = \sigma_n' \tan [\text{JRC}_n \log_{10} (\text{JCS}_n/\sigma_n') + \phi_r] \quad (\text{Bandis et al.}, 1981) \quad (5)$$

The collection of simple index tests for obtaining input data for fractures or joints, using tests on core or on samples sawn or drilled from outcrops is shown in Figure 10. In the left-hand column, direct shear tests are also illustrated, which can be used to verify the results from the empirically-

developed index tests. Note that the Schmidt hammer should be used on clamped pieces of core to estimate the uniaxial compression strength UCS or σ_c . This is done on dry pieces of core (rebound R, use top 50% of results). For the joint wall strength JCS, this is done on saturated samples (rebound r, use top 50% of results). These samples also need to be clamped (e.g. to a heavy metal base). Three methods of joint-wall or fracture-wall roughness (JRC) can be used: tilt tests to measure tilt angle α (i.e. Figure 5), or a/L (amplitude/length) measurement, or roughness profile matching: the latter obviously more subjective.

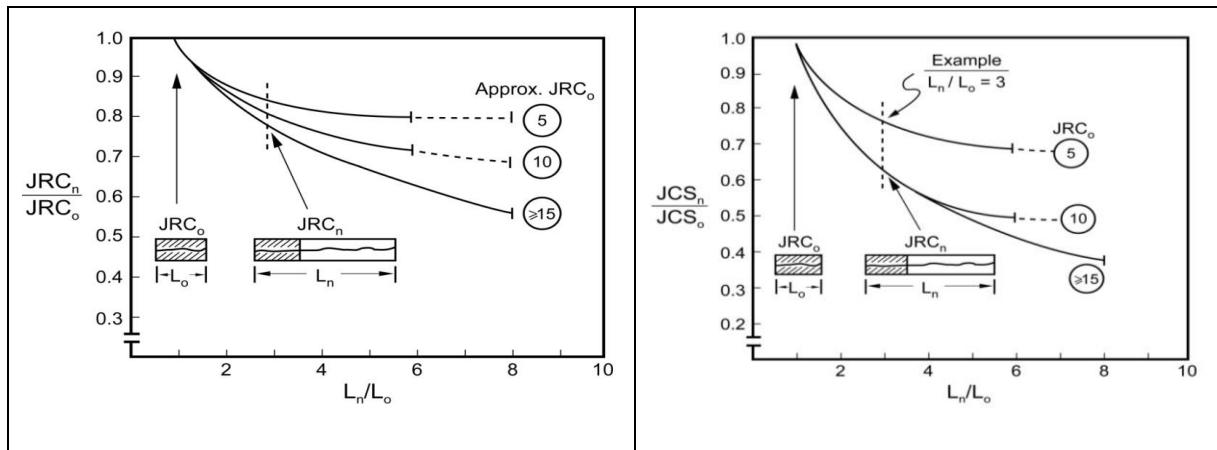


Figure 9 An empirically-based method to correct JRC and JCS for the effect of block size. Note that L_0 is the nominal lab-scale value, while L_n is the spacing of cross-joints defining the block-size in question.

4. FROM ROCK JOINTS TO ROCKFILL

As indicated in the Dedication to this article, the non-linear shear strength envelopes for rockfills (see Figure 1) presented by Marsal (1973), made a strong impression when these were first seen by the writer in about 1979. Non-linear strength envelopes were also the dominant theme for rock joints, in the author's work since about 1967: first with artificial tension fractures (in brittle model materials) then with rock joints (see Figure 11), with the latter contributions starting in Barton (1973). In fact it is interesting to reproduce one of the early diagrams when JRC and JCS were first suggested, since six of the curved envelopes (Figure 12) contain within their range, something similar to the shear strength of rockfill. It seems that non-linearity was a common but independent pre-occupation in Mexico (for rockfill) and in Norway (for rock joints) in the early 1970's.

Figure 12 shows that there are strong similarities between the shear strength of rockfill and that of rock joints. This is because they both have 'points in contact', i.e. highly stressed contacting

asperities or opposing stones. In fact these contacting points may be close to their crushing strength, such that similar shear strength equations to those used for rock joints can apply to rockfill, and indeed to rockfill/rock interfaces. This can be seen in the progression of equations.

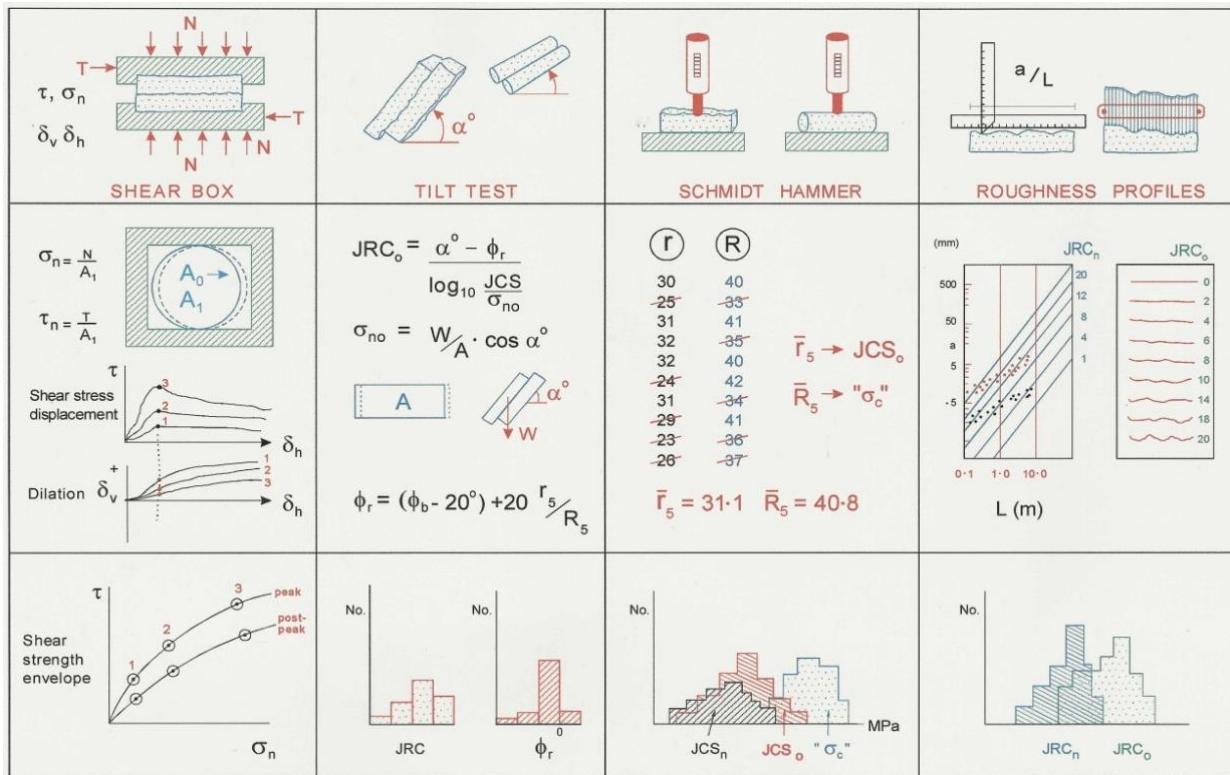


Figure 10 The index tests required to characterize rock joints, also used to obtain input for shear strength. The tests were introduced by Barton and Choubey (1977) and Barton (1981) proposed the a/L method for JRC The present format of tests was presented in Barton (1999).

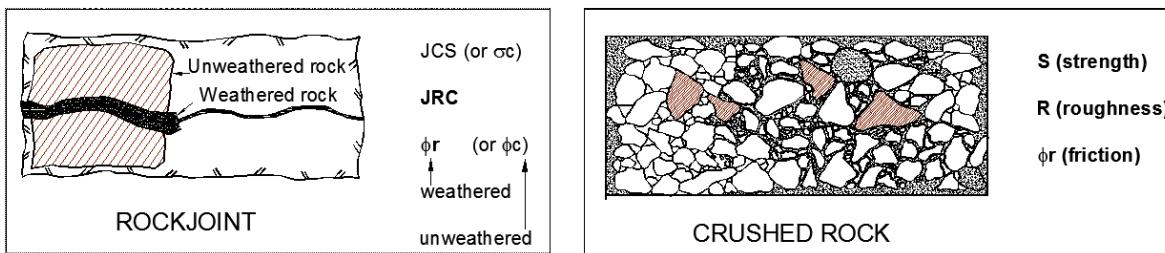


Figure 11 The empirically-derived index properties for rock joints (JRC and JCS) from Barton (1973) are the source of the equivalent parameters (R and S) for rockfill, used by Barton and Kjærnsli (1981).

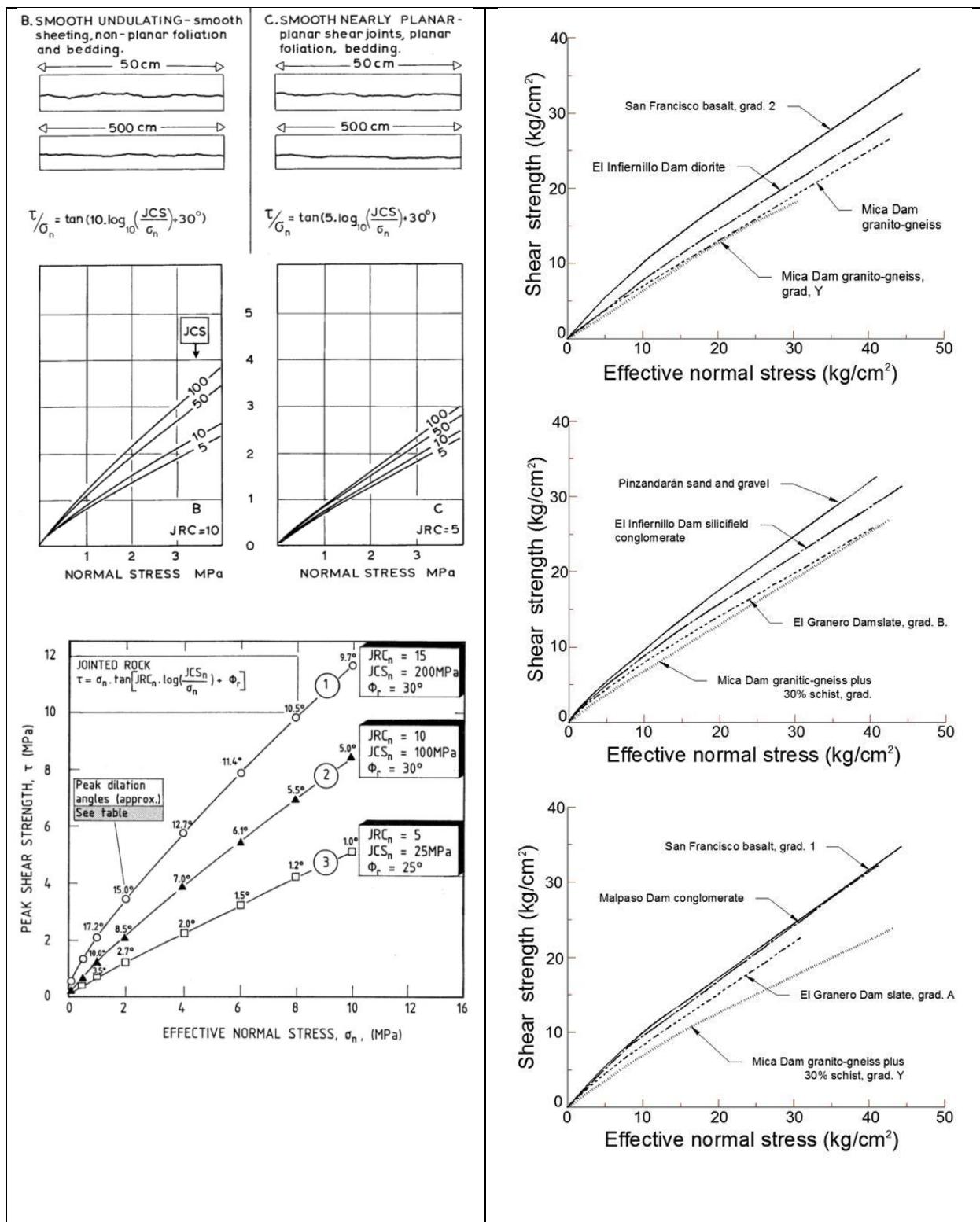


Figure 12 Top-left: part of the suggested method to estimate the peak shear strength envelopes for rock joints of different roughness, from Barton (1973). **Bottom-left:** the addition of estimated peak dilation angles is shown, for three strength envelopes, from Barton (1993). Envelopes #2 and #3 also resemble the rockfill strengths: **Right:** the complete set of Mohr envelopes for various granular materials given at the end of the Marsal (1973) 'Casagrande' book chapter.

The three equations to be utilized in the remainder of this paper have the shear strength of rock joints as a starting point. The first equation listed was equation 4:

$$(1) \tau/\sigma_n = \tan[JRC \cdot \log(JCS/\sigma_n) + \varphi_r] \dots \text{applies to rock joints}$$

$$(2) \tau/\sigma_n = \tan[R \cdot \log(S/\sigma_n) + \varphi_b] \dots \text{applies to rockfill.} \quad (6)$$

$$(3) \tau/\sigma_n = \tan[JRC \cdot \log(S/\sigma_n) + \varphi_r] \dots \text{might apply to interfaces (see later)} \quad (7)$$

The equation to be used for a *rockfill/rock-foundation interface* will depend on whether there is ‘JRC’ control, or ‘R’ control. This distinction: preferential sliding on the interface or shearing within the rockfill, is described and illustrated later.

Concerning the potentially very high friction angles at low stress levels, or close to the surface of a rockfill dam embankment (e.g. Figure 13), Marsal (1973) makes mention of a basalt rockfill having φ' of up to 70° under a normal effective stress of only 0.1 MPa. Although the ‘asperity strength’ component (Figure 8) is uncertain for rockfill, if it were of similar magnitude to the *peak dilation angle* ($d_{n peak}$), as in the case of rock joints, then the latter could be in the region of 20° at similarly low stress, compatible with the low stress predictions of peak dilation given in Figure 4.2 (bottom left) and in Table 1. Marsal (1973) considered that within a rockfill dam the friction angle of the materials within the range of confining stresses of interest was usually less than the $40\text{--}45^\circ$ previously assumed, due to the effect of the confining pressure and the non-linearity. This will be explored further.

Table 1 The complete set of estimated peak dilation angles, for the three envelopes in Figure 12 (bottom-left). Note the extremely high values of $d_{n peak}$ predicted when stress levels are low. This resembles rockfill.

Curve No.	Effective normal stress MPa							
	0.1	0.5	1.0	2.0	4.0	6.0	8.0	10.0
1	24.7	19.5	17.2	15.0	12.7	11.4	10.5	9.7
2	15.0	11.5	10.0	8.5	7.0	6.1	5.5	5.0
3	6.0	4.2	3.5	2.7	2.0	1.5	1.2	1.0

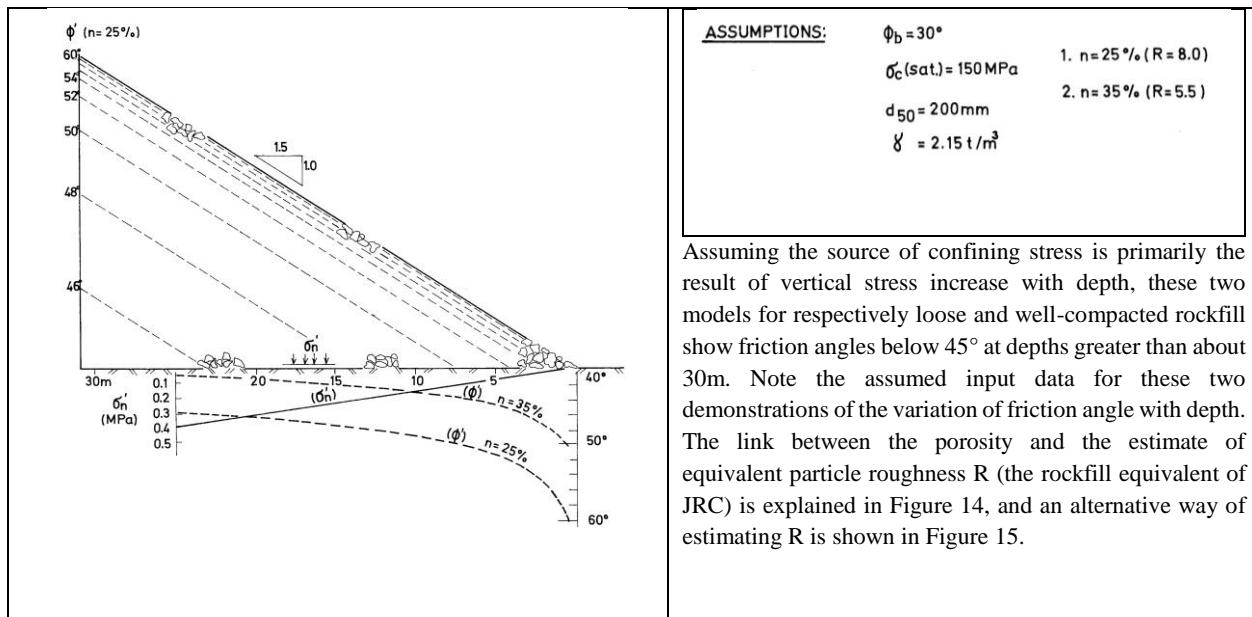


Figure 13 Two estimates of the stress-dependence of friction angles for prospective loose and dense rockfill, using (vertical) depth within a dam slope as a simplified model of confining stress increase.

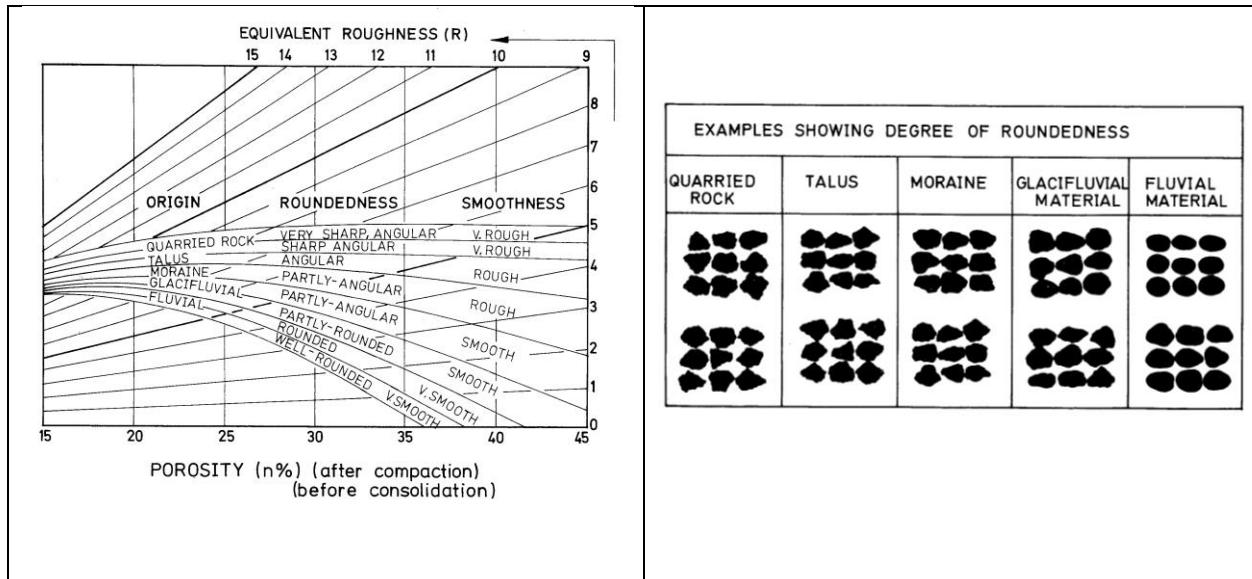


Figure 14 A simple empirical model from Barton and Kjærnsli (1981) for obtaining a preliminary estimate of equivalent roughness R for rockfill materials of different origin, roundedness and porosity. The workings of R (and S) as equivalent rock joint parameters JRC (and JCS) are demonstrated in parallel to the rockfill data assembled by Leps (1970), in Figure 15.

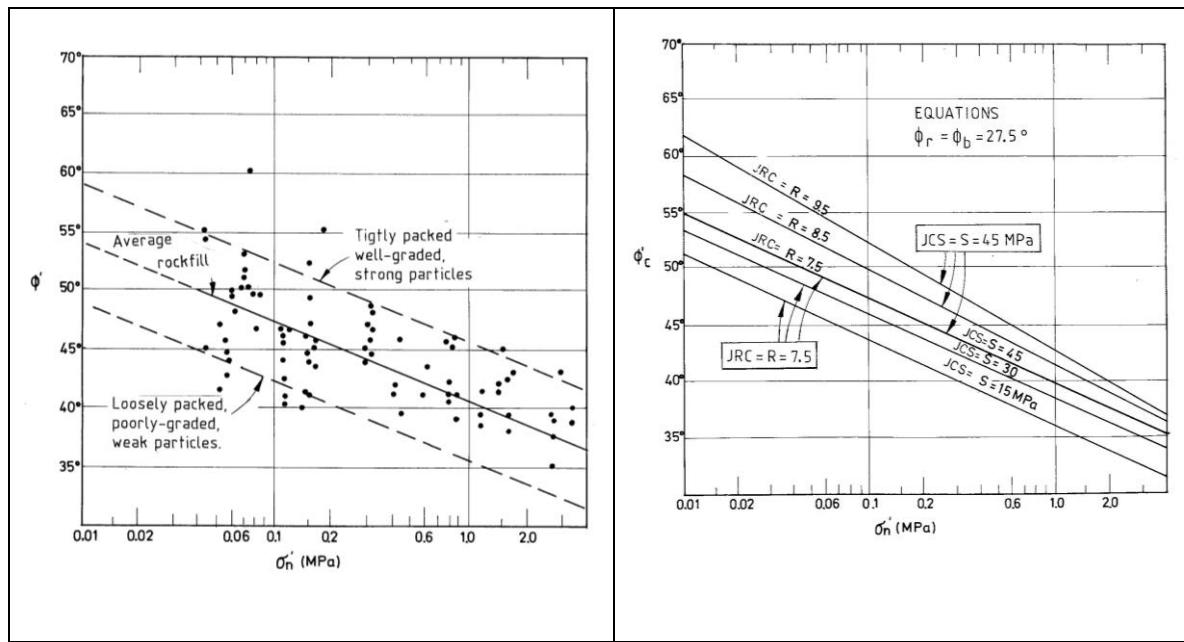


Figure 15 Left: Assembly of peak shear strength data for rockfills, from Leps, 1970. The logarithmic scale of effective normal stress is further confirmation of the similarity of rockfill and rock joints. **Right:** Comparative JRC or R, and JCS or S values used to generate similar gradients to Leps 1970 data for rockfill. R = 5 to 10, and S = 10 to 100 MPa appear to cover the range of strengths assembled by Leps.

The non-linearity implied by the rockfill data assembled by Leps (1970) shown in the left diagram of Figure 15 is provided by the linear-phi and log-stress format, which is exactly consistent with the experiences of the shear strength of rock joints shown in the diagram on the right side of Figure 15. Note the direct ‘substitution’ of R for JRC and S for JCS, in order to demonstrate similarity to the data for rockfill. A similar gradient to that drawn by Leps (the middle-of-the-data line) seems to be provided by R = 7.5 and S = 45 MPa.

Large-scale vacuum triaxial cylinders as seen in Figure 16, have a limit of confinement of < 100kPa. Somewhat smaller but higher pressure triaxial cells, as also used by Marsal (1973), and by many others during several decades, were also used in these studies, in order to reach the higher confining pressures. Of particular note besides the non-linear stress-dependent friction angles, is the steepness of their decline with increasing stress.

Simple inspection of one order of magnitude stress change, from 0.16 to 1.6 MPa, bringing ϕ' from 51° down to almost 39° suggests an equivalent R-value as much as 10.5, even steeper (rougher) than the high-lighted data shown in Figure 15. (If smaller particle gradings had been used at the higher stresses, the decline would be much reduced due to higher S-values (see Figure 17). Note from Figure 14 that if quarried rock could be so well graded (thereby filling its voids) and so well

compacted that a porosity of 20% was reached, then an R-value as high as 10 can be predicted (close to the 'Q' of QUARRIED ROCK in Figure 14). Beneath 500m of cover all is possible.

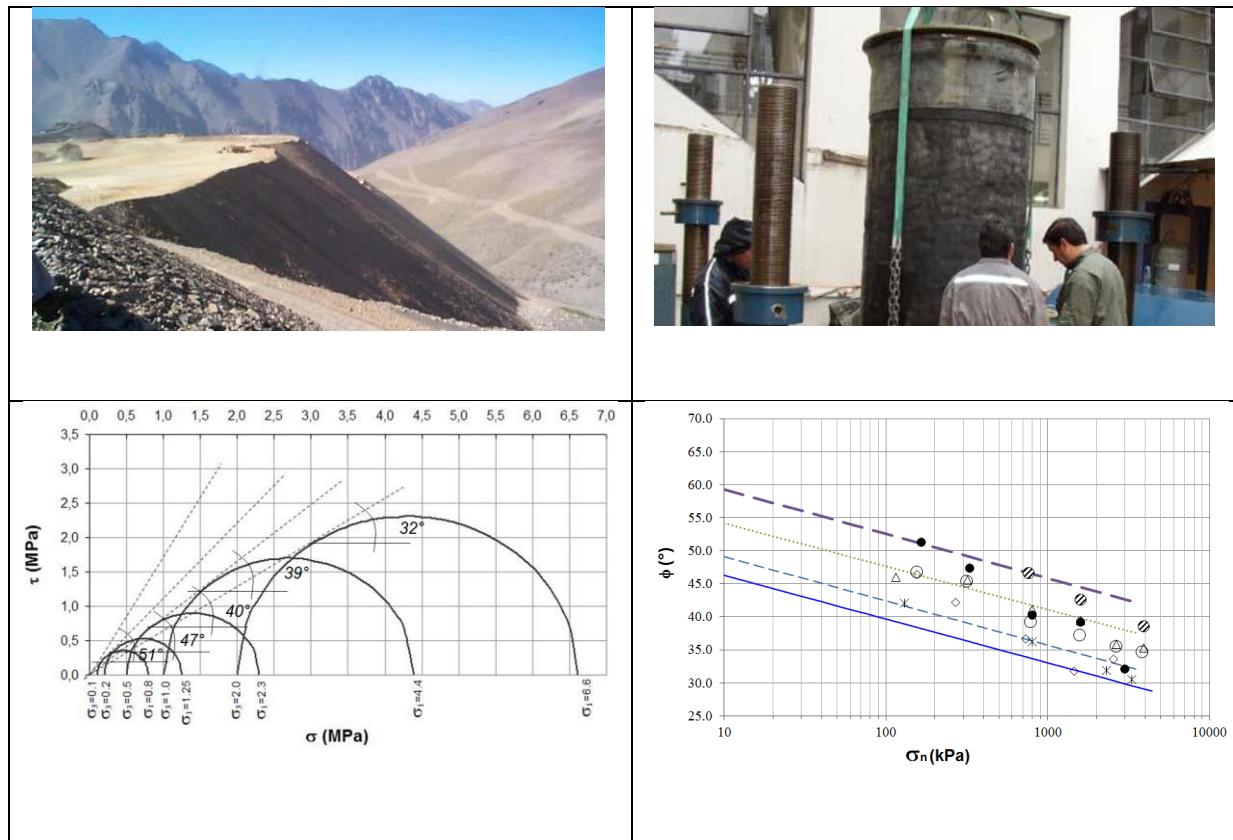


Figure 16 In Chile there are plans for mine-operations rock dumps with thicknesses far exceeding even the highest rockfill dams. A figure of 500 m has been seen. Consequently, some large-scale investigations of rockfill have been performed, one of which by SRK is reproduced with permission from Linero and Palma (2006). Note that Marsal (1973) obtained comparable ranges of $\varphi'_{\max} = 46$ to 53° , and $\varphi'_{\min} = 34$ to 39° from the lowest σ_3' of about 0.04 to 0.09 MPa up to the highest σ_3' of about 2.4 to 2.5 MPa. There were some exceptions to this for the case of tests on mixtures of schist and (partly weathered) granitic gneiss for the Mica dam, with minimum values at the highest stress down to 28, 29 and 30° .

The necessity to have a scale-effect adjustment for the uniaxial compressive strength of rockfill, following approximately the trend obtained in rock mechanics studies is clear. For simplicity and consistent reference, the strength of the d_{50} size is recommended. In the case of the UCS of rock the following equation suggests itself, based on an averaging of Hoek and Brown and Wagner versions with their slightly diverging exponents of 0.18 and 0.22: $\sigma_c = \sigma_{c50}(50/d_{50})^{0.2}$. The term σ_{c50} is the standard (50 mm diameter) size of UCS specimens. The diameter d_{50} is the stone size. The way that R (or JRC) causes a change of R° (or JRC°) (i.e. change of angular degrees) per

decade change of effective stress or equivalent overburden depth, is demonstrated in Table 2. It seems logical to assume that 20, and JRC (and JRC_n) seen earlier in equations 2 through 7 actually have units of degrees, as in the case of Patton's 'i'-value shown in Figure 6 (bottom-left diagram). The \log_{10} (strength/stress) adjustment factor determines how many decades (or fractions of a decade) the change of stress 'multiplier' of R (and JRC) will be.

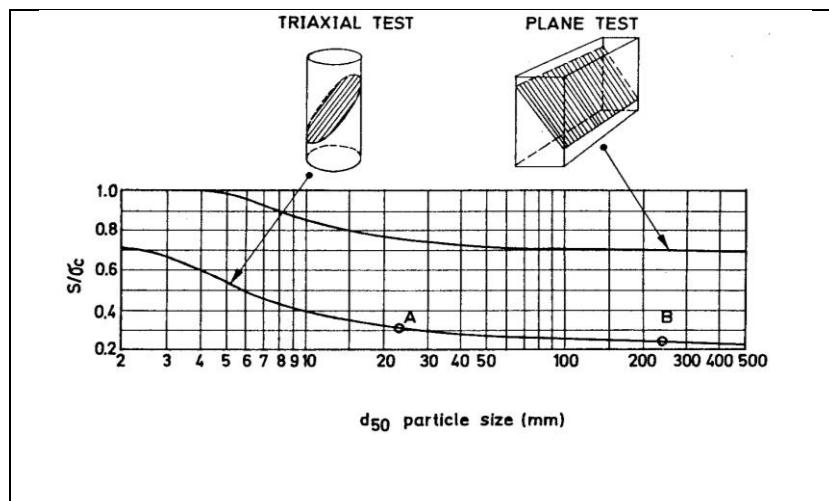


Figure 17 An empirically developed way to estimate the approximate scale effect on particle strength (S), based on the d_{50} particle size. Note the differentiation of triaxial test data and the more realistic in-dam plane test data, which results in a few degrees higher frictional strength, as demonstrated in Table 2.

Table 2 In the same manner as JRC for rock joints, a given R value such as 6 in this example, causes the predicted frictional strength to change by 6° for each decade change in effective normal stress, when using the proposed non-linear shear strength criterion. (This table was presented by Barton and Kjærnsli (1981). An exactly equivalent table was presented by Barton and Bandis (1990) when presenting the effect of e.g. 1000 times higher stress when progressing from a small-scale tilt test on a rock joint (Figure 5), in which $\sigma_n' \approx 0.001$ MPa at failure, to the much higher stresses (≈ 1 MPa) 40 m beneath a rock slope.

σ'_n , in MPa (1)	Equivalent overburden (2)	ϕ' plane (3)	ϕ' triaxial (4)
0.0001	5 mm	67°	65°
0.001	50 mm	61°	59°
0.01	500 mm	55°	53°
0.1	5 m	49°	47°
1.0	50 m	43°	41°
10.0	500 m	37°	35°

Note: 25.4 mm= 1 in; 1 MPa = 145 psi.

For example a large-scale tilt test (as in Figure 18 and 19) with a normal stress at failure of about 0.01 MPa, using a rockfill material having a scaled compressive strength of 10 MPa for the d_{50} particle size, involves $\log_{10}(10/0.01) = 3$ ($\times R$) for the case of the tilt test, while it will be $\log_{10}(10/1) = 1$ ($\times R$) inside the dam at e.g. 45 m depth. If we assumed $R = 7$, there would therefore be a 14° lower friction angle inside the dam than in the tilt test: for instance 48° in the tilt test, reducing to 34° inside the dam where the confining stress is assumed to be 1 MPa, or 100 times higher than in the tilt test.

Figure 18 shows the Barton and Kjærnsli (1981) suggestion for large scale tests on dams under construction. By chance NGI staff had arranged for such tests in Italy, just before the writer returned to NGI after four years in the USA in 1984. Due to pressing new administrative duties, the writer was not able to physically accompany the testing in Italy. The sequence of photographs later shown to the writer (some examples are given in Figure 19), suggest that the Italian contractor at the dam did not follow the recommendation of ‘standard compaction’ within the ‘next lift’ of the dam, followed by ‘excavation’ of the tilt-shear box. Possibly they found this to be too burdensome. So it appears that other, probably less efficient methods, may have been used to compact the rockfill before tilt testing. A bulldozer on a ramp is seen in one of the photographs. It appears that the tilt angles were mostly in the range of 45° to 50° .

A particular feature of these large-scale tests is that the gravity loading provides a much more uniform loading than the usual top-platten-and-end-loading in a standard shear box. The relative absence of any tendency to rotate when tilted is easy to see. (*In the case of some very rough rock joints, the top half of the tilt test sample needs to be made slender (low H/L ratio) in order to prevent toppling (rotating) instead of parallel sliding. There seems to be no such problem with the 1/5 ratio seen in Figure 19.*)

5. FROM ROCKFILL TO INTERFACES

Because some dam sites in glaciated mountainous countries like Norway, Switzerland, and Austria have insufficient foundation roughness to prevent preferential shearing along the rockfill/rock foundation interface, artificial ‘trenching’ is needed. Various scales of investigation of interface strength have been published, and these were synthesized by Barton (1980). The physical reality of a rockfill/rock interface is illustrated in Figure 20, and potentially *correct* input parameters for estimating the shear strength are suggested.

Figure 21 illustrates two rockfill dams with moraine cores, while they were under construction. Both appear to be on relatively smooth rock foundations, in particular the Norwegian example on

the left. Here it is uncertain if the foundation is rough enough to prevent the interface from being ‘the weakest link’.

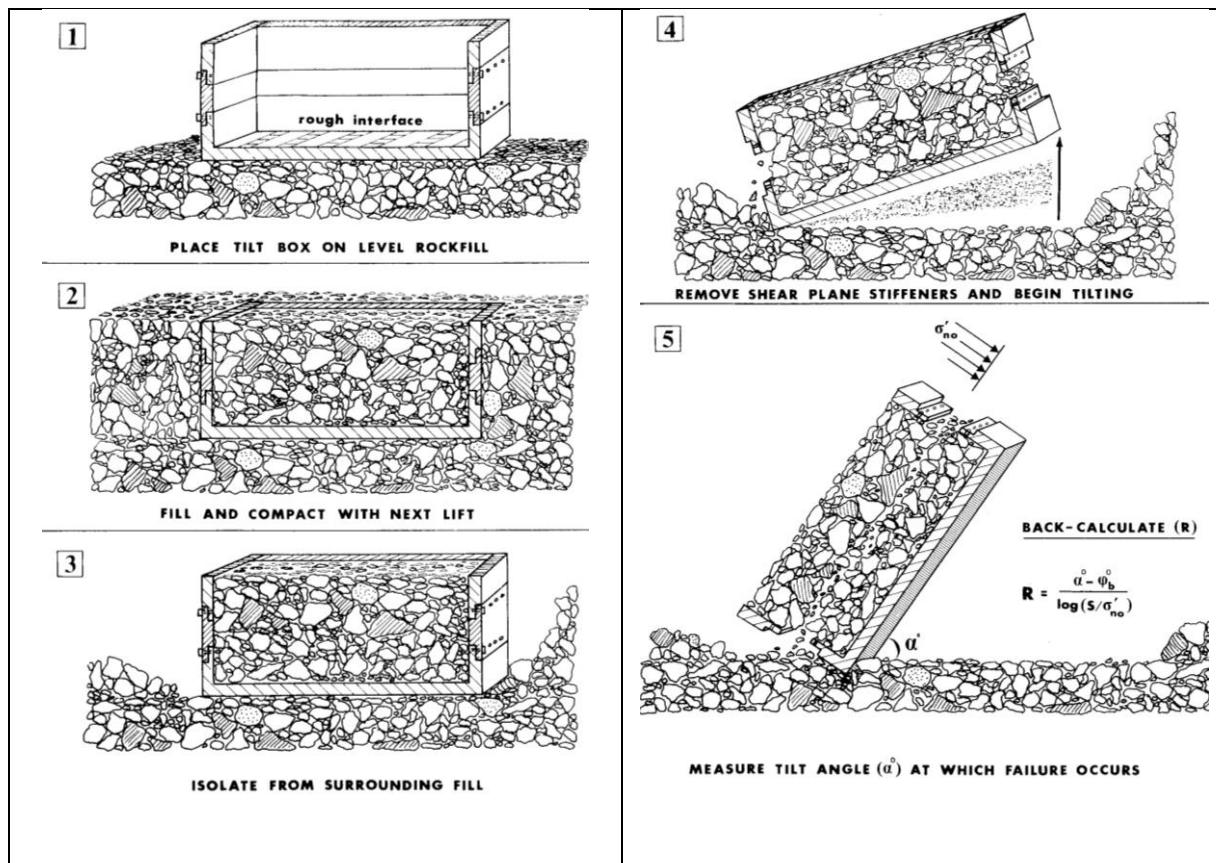


Figure 18 Barton and Kjærnsli (1981) suggested that full-scale tests could be made on rockfill, without needing to use parallel grading curves. The sequence of diagrams and accompanying texts by the writer were designed to be self-explanatory. Note that one ‘error’ discovered some years later when performing such tests at a dam site in Italy at 2 x 2 x 5 m scale (Figure 19) was that the ‘shear plane stiffener’ which is to be moved before the test, was too wide: material was lost from the outer parts of the shear plane during tilt testing, unless the gap was reduced to some 10 centimeters.



Figure 19 Tilt testing of ‘as-built’ rockfill, as suggested in Barton and Kjærnsli (1981), with performance of ten tests at a rockfill dam in Italy. The tilt-shear box measures 5 m × 2 m × 2 m. Three tests are shown, using pairs of photographs. The top-left pair shows the horizontal condition: a 10 to 15 cm gap is advised. The sheared ‘detail’ shows the displaced instrumentation (shear and dilation) in the post-peak (ultimate) condition. Note the relative absence of non-parallel shear (or ‘toppling’) due to uniform gravity loading.

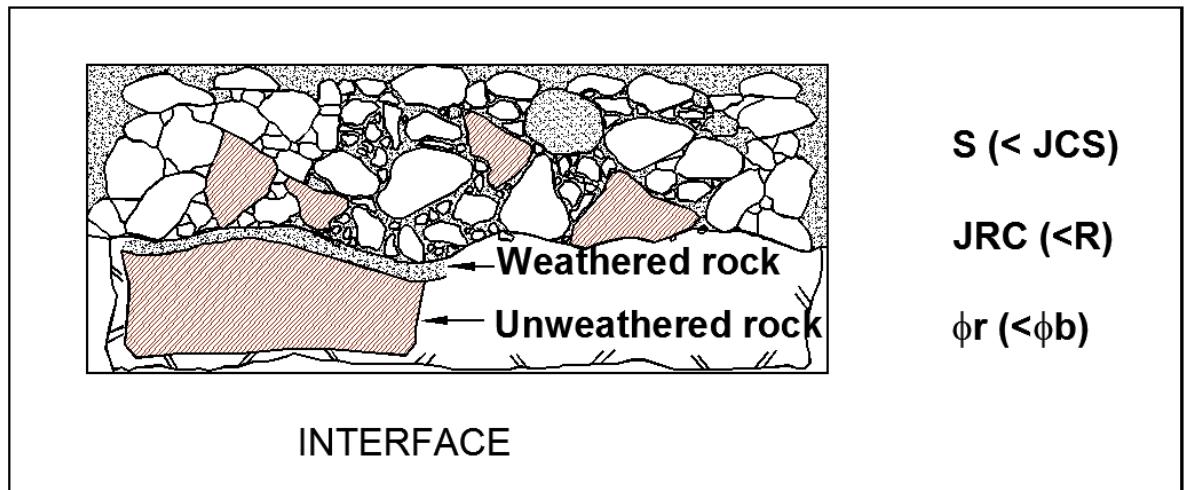


Figure 20 The rockfill/rock-foundation interface problem includes the likelihood of a lightly weathered surface (even in glaciated terrain), so it is appropriate to suggest ϕ_r (implying $\phi_r < \phi_b$ and $r < R$ when using a Schmidt hammer in Figure 10). The choice of JRC (meaning the interface is weakest) instead of R (meaning that the interface is strongest) provides the test of whether shear failure may occur preferentially through the rockfill (or not at all) because of a strongly ‘inter-locked’ foundation.



Figure 21 Left: The Svartevann Dam in Norway appears to have limited roughness in parts of its steep (U-shaped valley) glaciated rock foundation abutments. Right: As will be demonstrated in Figure 22, the (discarded) boulders seen in the foreground of the Brazilian dam (Camargo Correa photo) are obviously too large to interlock with the relatively smooth rock foundation.

Figure 22 illustrates examples of these two categories of shearing, in which the ‘weakest link’ determines the mode of sliding: whether the interface is smooth enough and the particles big enough to prevent good interlock (JRC-controlled), or the opposite R- controlled behaviour, with preferential failure within the rockfill, due to good inter-lock across the interface. Numerous cases reported in the literature were analysed in unpublished research performed by the author, and can be summarized by the data points plotted in Figure 22 (left). The a/L versus d_{50} ratios for the

Norwegian dam shown in Figure 21 are not known. However JRC-controlled interface behaviour seems likely in places. The (discarded) boulders in the foreground of the Brazilian dam (right) would give JRC-controlled behaviour. They are too big to interlock with the smooth rock surface.

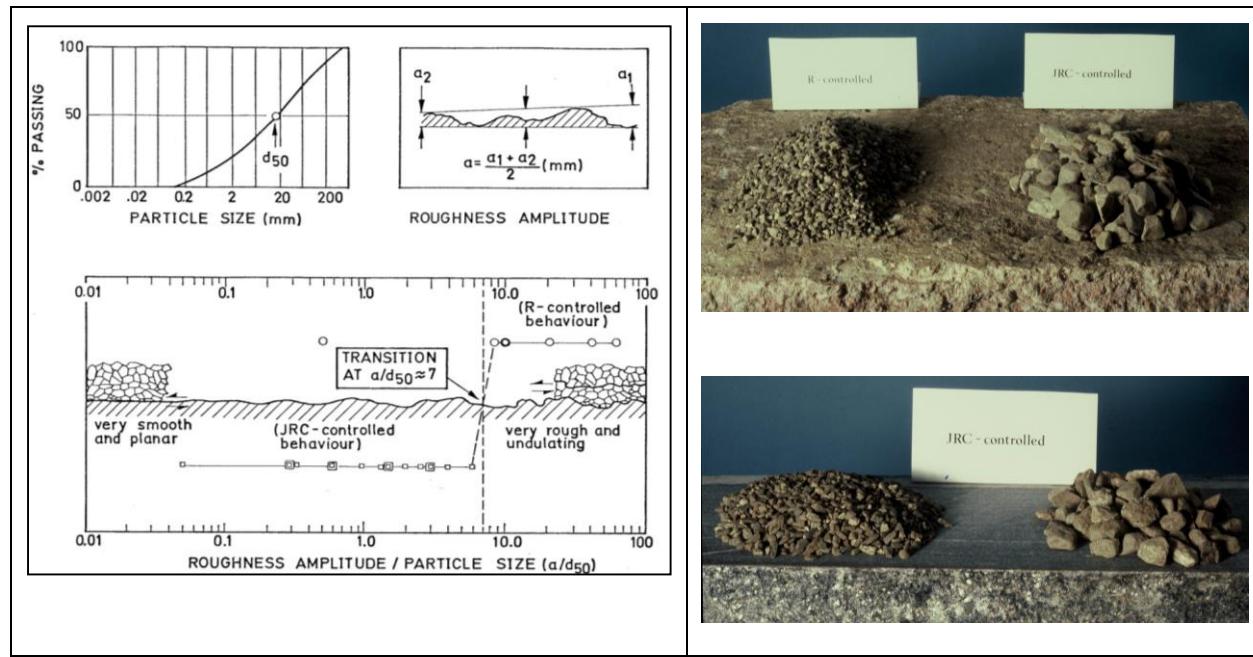


Figure 22 Left: The results of interface/rockfill and interface/sand (and gravel) direct shear tests can be separated by means of the ratio a/d_{50} , into R-controlled and JRC-controlled categories. The transition occurs at $a/d_{50} \approx 7$. **Right:** Four examples of a/d_{50} which demonstrate either preference for interface sliding or preference for internal shearing in the rockfill. This can be verified (at low stress) by tilt-testing.

6. DISCUSSION

This paper has seen an attempt to link the shear strengths of rockfill – with its many possible varieties, materials, porosities, stress levels – to the shear strength of rock and rock joints. Naturally, the key to such a linkage are the highly-stressed points of contacts. Of course in 1967 when the writer started experimenting with the shear strength of rough tension fractures, in an attempt to improve upon Mohr-Coulomb and Patton for describing rock joints, the *full significance* of the asperities in contact, as seen in Figure 4 (top-left) was not understood. Nevertheless, the first shear strength criterion involving the ratio of material compressive strength and effective normal stress was produced, and ten years later, even for the case of weathered joints, this ratio continued to be a central part of the strength criterion, in the form JCS/σ_n . For rockfill we have found that

S/σ_n , with both JCS and S corrected for block size (L_n) or particle size (d_{50}), has strong relevance to the understanding of peak shear strength.

In remarkably painstaking work by Marsal (1973), pre-occupation with particle contacts and particle contact forces is reflected by the drawing of particle contacts in Figure 23, and by an example of the statistics Raul Marsal produced, with observations of particle contacts numbering in their thousands.

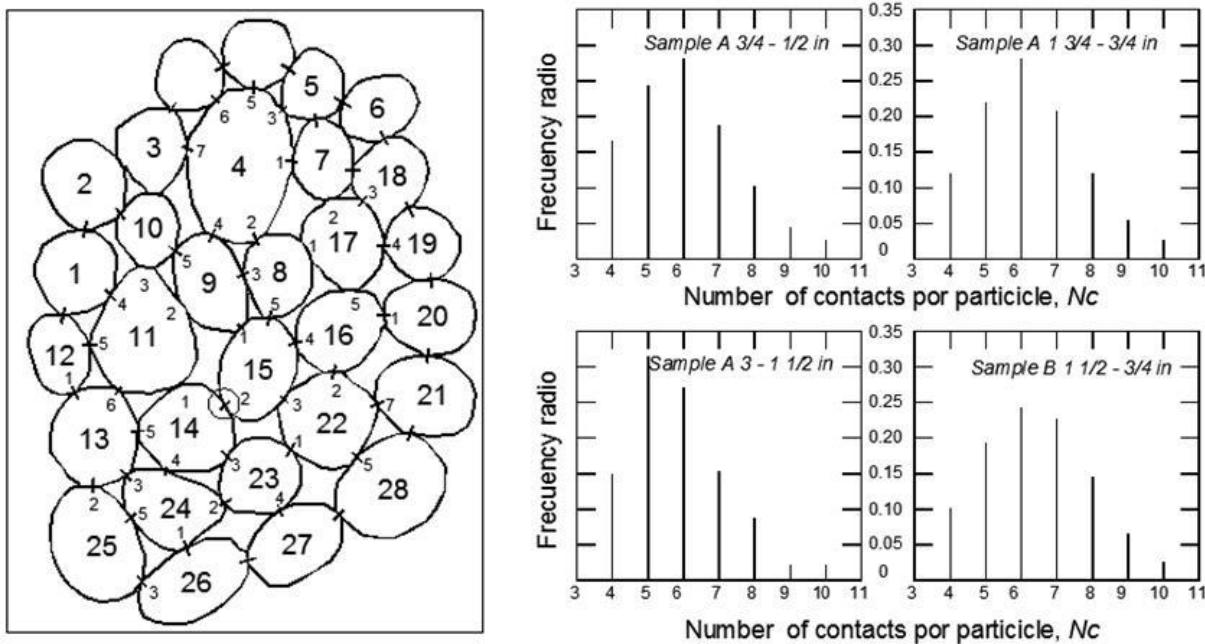


Figure 23 A brief example of the painstaking work performed by Marsal (1973) on the subject of particle contacts and their possible stress levels. The numbered contacts shown here are referenced to a horizontal plane, and the statistics on the right refer to a loose, uniformly graded gravel. In this case the number of contacts N_c ranged from 4 to 10 per particle, with means between 5.9 and 6.4. In the case of densely packed specimens, the range was 4 to 13, with means ranging from 7.1 to 8.3.

The method used by Marsal to record the (undisturbed) number of contacts was novel in the extreme. He flooded the rockfill or gravel samples from the bottom upwards with *paint*, allowed it to dry for 24 hours, and then began the careful work of ‘excavating’ up to several thousand grains counting the paint coated contacts as he progressed. This deserves our strong admiration.

Of course Marsal was characteristically modest in his conclusions about work remaining to be done. Presumably, due to capillary effects, the paint-covered contacts, while an accurate reflection of numbers, must have presented a remaining question about actual areas in contact and therefore actual stress levels. In work on rock joints, the writer has assumed that the small percentage of

asperities in contact must raise actual stress levels up to UCS or JCS levels. Whether this could hold for rockfill is less certain, as too much particle adjustment is possible.

7. CONCLUSIONS

1. Rock material, which inevitably controls the behaviour of rock joints and the particulate components of rockfill, plays an extra sensitive role in shear strength development, because the true contact areas transferring the shear and normal stresses are very small, and therefore the stress levels are correspondingly very high. So the crushing strength of the rock becomes important.
2. The asperities in contact on either side of a rock joint and the small contacting points around the rockfill particles appear to follow the same basic non-linear relation between shear strength and effective normal stress. Both materials dilate strongly at low stress, causing high apparent total friction angles. At high stress levels there is reduced dilation and asperity strength is mobilized due to crushing.
3. It is now known that when the local confining pressure reaches approximately the level of the uniaxial compressive strength, the rock in question will generally have reached its maximum possible strength, or ‘critical state’. This is important at these points in contact.
4. The strong non-linearity and lack of actual cohesion for both rock joints and rockfill give a special bonus. One can perform extremely low-stress gravity-loaded tilt tests in both cases, using the non-linear strength criteria to extrapolate results to many orders of magnitude higher stress.
5. Raúl Marsal played a strong indirect role in linking rockfill and rock joint behaviour because of his painstaking investigations of the particles-in-contact phenomena, and his presentation of triaxial test results in a format familiar in rock mechanics, namely τ versus σ_n' , as a result of analyzing the triaxial strengths in relation to the failure planes.

This paper has mostly addressed only the peak shear strengths of rock joints and rockfill. The mobilization of friction and dilation up to peak strength, followed by degradation of roughness towards ultimate and residual strength is also important if numerical modelling is to be performed. The appendix shows by means of three figures how this can be estimated for the case of rock joints.

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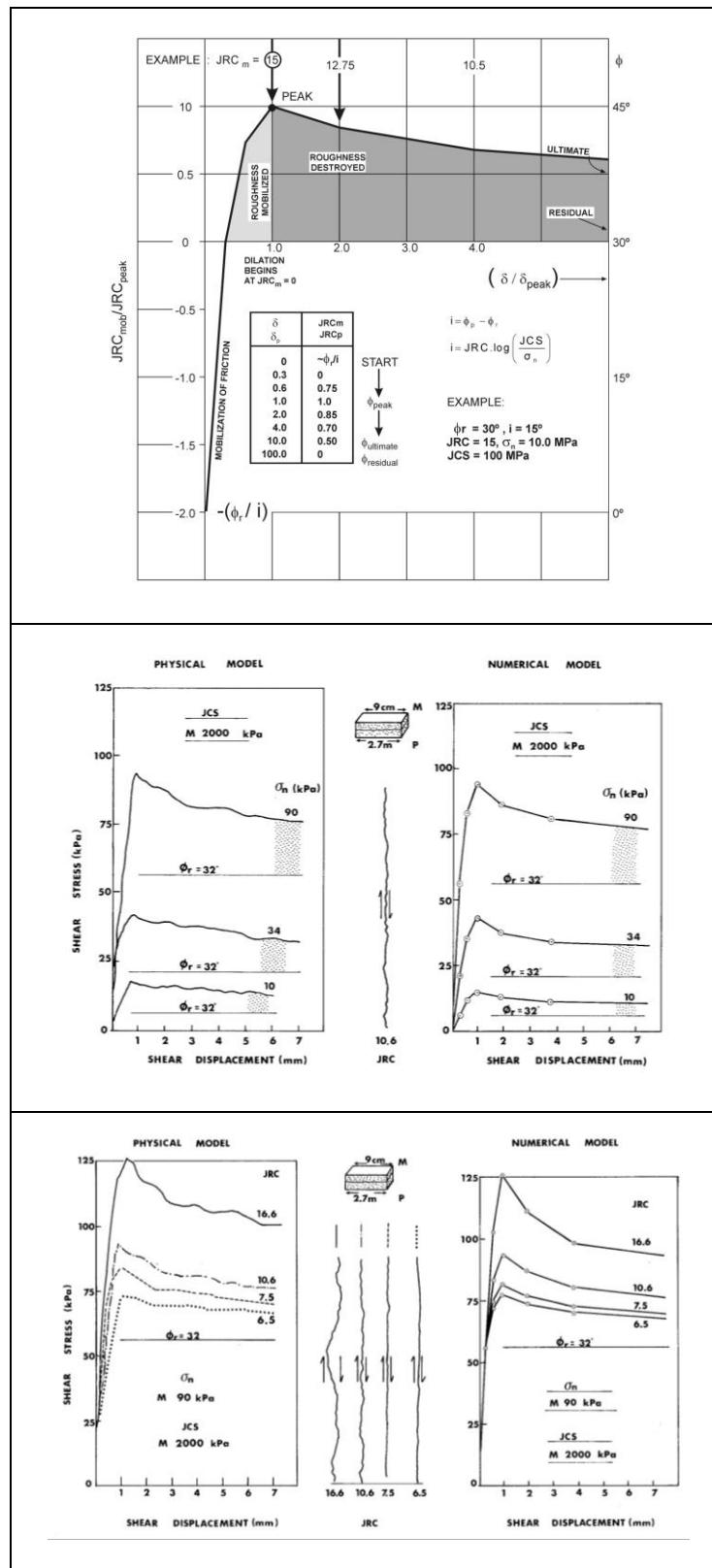
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APPENDIX 1



Peak strengths have been the subject of this paper. In case the shear strength-displacement is required, for instance as input to distinct element modelling such as UDEC or PFC, then the $JRC_{mobilized}$ concept can be used (for the case of rock joints). The matching of Bandis experiments with Barton predictions is shown. Barton (1982). There seems no reason why ' $R_{mobilized}$ ' could not be used for rockfill.