TR3055 S

Stellingen

behorende bij het proefschrift van Peter Nicolaas Willem Verhoef: *Wear of rock cutting tools: implications for the site investigation of rock dredging projects*
(Slijtage van gesteentesnijdende werktuigen: implicaties voor het grondonderzoek voor baggerprojecten in gesteente), te verdedigen op 12 december 1997, Technische Universiteit Delft

1. In plaats van de PIANC classificatie voor te baggeren grond en gesteente, zal een uitgebreide gids voor het uitvoeren van grondonderzoek nodig zijn, met een aanpassing van de paragrafen over het onderzoek van gesteenten en het vaststellen van de noodzaak van het presenteren van een drie-dimensionaal model van de te baggeren ondergrond.

   dit proefschrift

2. De kwaliteitsindex voor gesteente massa, RQD (Rock Quality Designation), is verouderd en mag niet meer gebruikt worden als enige descriptor van de scheurdichtheid in gesteente.

   dit proefschrift

3. Petrografisch onderzoek is onontbeerlijk om te kunnen vaststellen of een gesteente duurzaam zal zijn bij toepassing als constructiemateriaal.

   Verhoef & Van de Wall 1997: "Application of petrography in durability assessment of rock construction materials." In: P. Bobrowsky (ed.) Aggregate resources: a global perspective. Balkema, Rotterdam. p.307-330.

4. De methyleen blauw proef is een ideale proef om de aanwezigheid van kleine hoeveelheden zwellende kleimineralen in een gesteente op te sporen. Voor dit doel is de proef geschikter dan de standaard röntgen diffractie proef.

   Jones 1964: "New fast accurate test measures bentonite in drilling mud." Oil Gas Journal, June: 76-78.

   Stapel & Verhoef 1989: "The use of the methylene blue adsorption test in assessing the quality of basaltic tuff rock aggregate." Engineering Geology 26: 233-246.

5. Veldonderzoek is onmisbaar in de geologie, drie-dimensionale presentatie van de gegevens met behulp van computersystemen zal een onmisbaar hulpmiddel worden.

6. De poging tot synthese van struktureel geologisch werk in verschillende gebieden in de Pyreneeën door Capellà i Solà (1997) wordt ontsierd door de aanname dat "hoofdfoliaties" in verschillende gebieden met elkaar gecorreleerd zouden kunnen worden. Er moet rekening mee worden gehouden dat deformatiesequenties niet synchroon hoeven te zijn en dat "hoofdfoliaties" voornamelijk een gevolg zijn van de vervormingsintensiteit van een deformatie gebeurtenis.

   Capellà i Solà 1997: "Strain analysis in the axial zone of the Variscan basement of the Pyrenees." Geologie en Mijnbouw 75: 361-371.

   Verhoef, Vissers & Zwart 1984: "A new interpretation of the structural and metamorphic history of the western Aston Massif (Central Pyrenees, France)." Geologie en Mijnbouw 63: 399-410.

7. Zeker op een Technische Universiteit is het af te raden te streven naar het doen van zogenaamd fundamenteel onderzoek. Toegepast onderzoek is minstens zo uitdagend en geeft net zoveel kans op nieuwe wetenschappelijke vondsten.

8. De overeenkomst tussen wetenschap en godsdienst is de flinke dosis geloof die nodig is voor het beoefenen van beide.

Theses

belonging to the PhD thesis of Peter Nicolaas Willem Verhoef: *Wear of rock cutting tools: implications for the site investigation of rock dredging projects*
to be defended on December 12 1997, Delft University of Technology

1. The PIANC classification of soils and rocks to be dredged needs to be replaced by a comprehensive guide describing the investigation of rock and prescribing the presentation of a three dimensional model of the subsurface to be dredged.
this thesis

2. The Rock Quality Designation (RQD) generally used to describe the quality of rock mass, is dated and cannot be used as the only measure of the density of fractures in rock.
this thesis

3. Petrographic examination is essential for the assessment of the durability of rock intended for construction material.
Verhoef & Van de Wall 1997: "Application of petrography in durability assessment of rock construction materials." In: P. Bobrowsky (ed.) Aggregate resources: a global perspective. Balkema, Rotterdam. p.307-330.

4. The methylene blue adsorption method is a simple test to indicate the presence of small quantities of swelling clay minerals in rock. For this purpose the test is more reliable than the standard x-ray diffraction test.
Jones 1964: "New fast accurate test measures bentonite in drilling mud." Oil Gas Journal, June: 76-78.

Stapel & Verhoef 1989: "The use of the methylene blue adsorption test in assessing the quality of basaltic tuff rock aggregate." Engineering Geology 26: 233-246.

5. Field studies are indispensable in geology and computer assisted three-dimensional presentation of the data will become an essential tool.

6. The attempt of Capellà i Solà (1997) to synthesize structural geologic work carried out in several areas of the Central Pyrenees is hampered by the assumption that "main foliations" could be correlated with each other. One should realize that "main foliations" are primarily the result of the strain intensity of a deformation event and that deformation sequences in different areas need not be synchronous.
Capellà i Solà 1997: "Strain analysis in the axial zone of the Variscan basement of the Pyrenees." Geologie en Mijnbouw 75: 361-371.

Verhoef, Vissers & Zwart 1984: "A new interpretation of the structural and metamorphic history of the western Aston Massif (Central Pyrenees, France)." Geologie en Mijnbouw 63: 399-410.

7. Certainly at a Technical University one should not attempt to perform fundamental research. Applied research is challenging as well and gives ample chances on new scientific findings.

8. The similarity of science and religion exists in the high measure of faith needed to practice both.

WEAR OF ROCK CUTTING TOOLS: IMPLICATIONS FOR THE SITE INVESTIGATION OF ROCK DREDGING PROJECTS

SLIJTAGE VAN GESTEENTESNIJDENDE WERKTUIGEN, IMPLICATIES VOOR HET GRONDONDERZOEK VOOR BAGGERPROJECTEN IN GESTEENTE

PROEFSCHRIFT

ter verkrijging van de graad van doctor aan de Technische Universiteit Delft,
op gezag van de Rector Magnificus Prof. dr. ir. J. Blaauwendraad,
in het openbaar te verdedigen ten overstaan van een commissie,
door het College van Dekanen aangewezen,
op vrijdag 12 december 1997 te 10.30 uur

door

Peter Nicolaas Willem VERHOEF

doctorandus in de wiskunde en natuurwetenschappen (geologie)
geboren te Hengelo (O)

Dit proefschrift is goedgekeurd door de promotoren:
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Dit proefschrift is opgedragen aan mijn grootvader en mijn vader, die bij mij de liefde voor de wetenschap hebben opgewekt:

Ir. Nicolaas Verhoef (1895-1974), mijnbouwkundig ingenieur (Delft 1924)
Ir. Willem Verhoef (1927-1977), werktuigbouwkundig ingenieur (Delft 1952)

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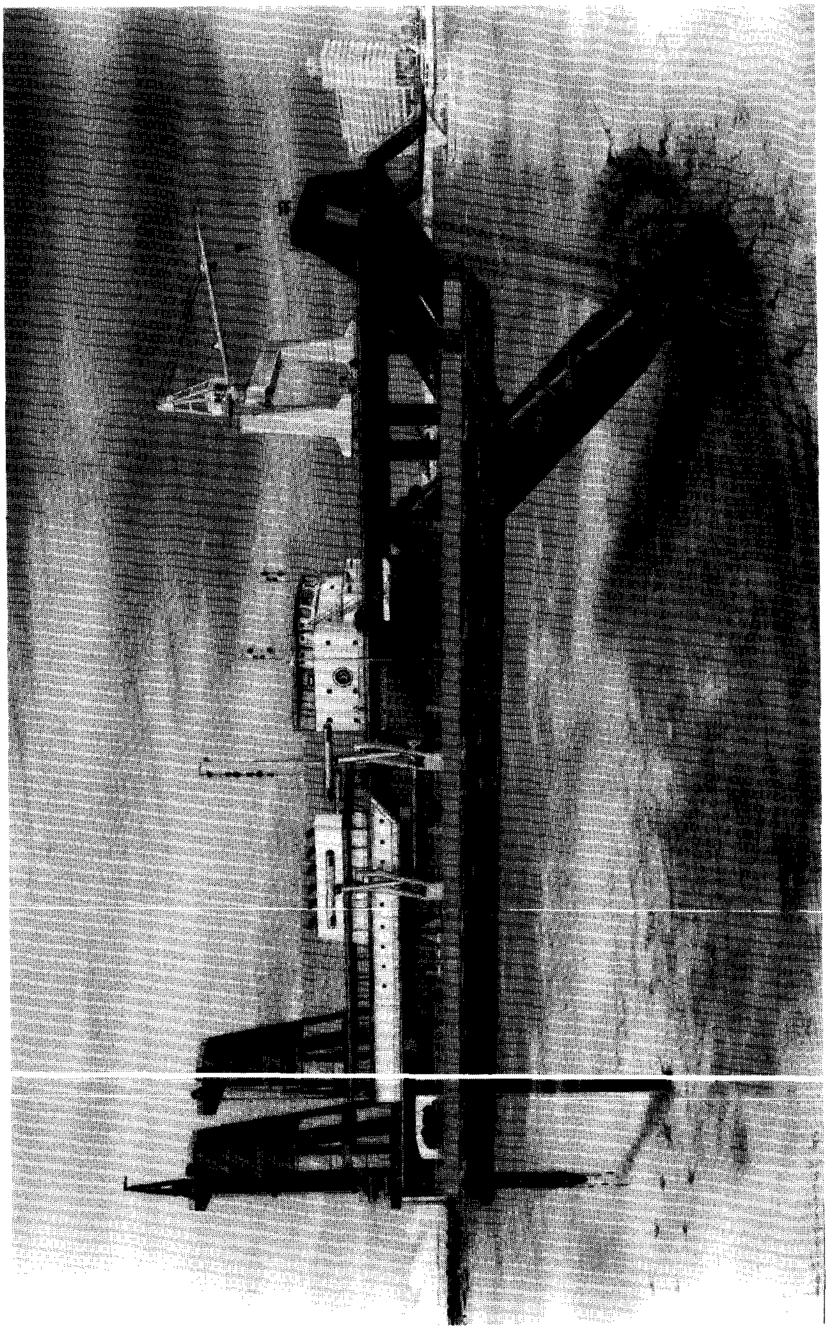
Voorwoord

Dit proefschrift is het resultaat van een studie die de afgelopen 10 jaar heeft plaatsgevonden. De opzet en het theoretische kader van het onderzoek naar slijtage van beitels in gesteente was mijn verantwoordelijkheid. Naast mijn eigen onderzoek zijn veel laboratoriumproeven en later ook veldstudies verricht door studenten in het kader van hun afstudeerwerk. De resultaten van dit werk worden besproken en geinterpreteerd in hoofdstuk 11, 17, 18.3 en 18.5. Jan Reinout Deketh heeft een belangrijke bijdrage geleverd aan het onderzoek. Hij kwam met het voorstel om mogelijke veranderingen van slijtageprocessen te bestuderen, tijdens het binnendringen van een beitel in gesteente. Het resultaat van dit werk (samengevat in hoofdstuk 12) is vastgelegd in zijn proefschrift, dat in 1995 is verschenen. Het was de bedoeling dat het werk dat nu voor u ligt tegelijkertijd gereed zou zijn. Ruim twee jaar later is het zover. Een voordeel van het latere tijdstip van verschijnen is dat de resultaten van veldopnamen van de prestaties van gesteente-sleuvengravers besproken konden worden in dit proefschrift. Er zijn velen die hebben bijgedragen aan het welslagen van dit project, ik dank allen voor hun inzet en ondersteuning.

Delft, september 1997
Peter N.W. Verhoef



De Mashhour is met 22.795 kW vermogen de krachtigste snijkopzuiger van dit moment. Bouwjaar 1996, scheepswerf IHC Holland.
The 22.795 kW Mashhour is the most powerful cutter suction dredger at this moment. It was built in 1996 by IHC Holland.

Slijtage van gesteentesnijdende werktuigen: implicaties voor het grondonderzoek voor baggerprojecten in gesteente

Baggeren van rots kan alleen als de technische eigenschappen van het gesteente dit toelaten. Zo zijn alleen de zwakke tot matig sterke gesteentesoorten (druksterkte kleiner dan 50 MPa) baggerbaar. En dan nog alleen wanneer een zware snijkopzuiger wordt ingezet. Sterker gesteente kan ook gebaggerd worden, als de gesteentemassa sterk doorsneden is met scheuren (natuurlijke scheuren: diaklazen, of kunstmatige: na fragmenteren met behulp van explosieven). Ondanks het feit dat informatie over sterkte van het gesteentemateriaal en de mate van gescheurdheid van de gesteentemassa vaak bekend is voor besloten wordt tot de inzet van een snijkopzuiger, komt men vaak voor onaangename verrassingen te staan. De moeilijkheden treden op bij de snijkop, waar excessieve slijtage van de tanden een grote invloed heeft op de teruggang in de weekproductie. Daarmee zijn zeer hoge geldbedragen gemoeid. De reden voor deze studie is dan ook de onbetrouwbaarheid van de schatting van het benodigde aantal snijtanden. Dit onderzoek is uitgevoerd om opheldering te geven over de gesteente-technische en geologische factoren die een rol spelen bij het mechanisch uitgraven van gesteente en in het bijzonder de factoren die van invloed zijn op slijtage van de snijtanden.

Het probleem is bestudeerd vanuit het vakgebied van de ingenieursgeologie, dat zich bezig houdt met terreinonderzoek voor civieltechnische werken. Een aantal aspecten kwamen naar voren, na bestudering van baggerprojecten waarbij zich excessieve tandslijtage had voorgedaan:

1. Veel projecten worden uitgevoerd aan de hand van incomplete en gebrekkige informatie over de geologie en de aard van het gesteente. Vitale informatie ontbreekt regelmatig, zoals bijvoorbeeld het kwartsgehalte van de gesteenten; het is algemeen bekend dat kwarts harder is dan staal en dus abrasief voor staal is.

2. Er is behoefte aan een verduidelijking van de factoren die een rol spelen bij het mechanisch uitgraven van gesteentemassa's.

3. Het is niet duidelijk welke slijtage processen optreden aan de tanden. Er treedt niet alleen hoge slijtage op in kwartsrijke gesteenten, maar soms ook in kalkstenen zonder kwarts (calciet is beduidend zachter dan staal en dus niet abrasief voor staal). Er is behoefte aan een simpele proef waarmee de abrasiviteit van een gesteente vastgesteld kan worden.

Dit proefschrift geeft een overzicht van de resultaten van het onderzoek, dat de afgelopen tien jaar onder leiding van de auteur is uitgevoerd.

Allereerst wordt in deel A (hoofdstuk 2-6) een overzicht gegeven van de problemen met slijtage die op kunnen treden bij het baggeren in rots met behulp van snijkopzuigers. De probleemstelling wordt beschreven in hoofdstuk 2. In hoofdstuk 3 wordt de werkwijze van een snijkopzuiger beschreven, alsmede de manier waarop de snijkop het gesteente snijdt. Bij de opzet van het onderzoek is begonnen met een fundamentele studie van het slijtage probleem met behulp van methoden uit de tribologie (kennis van slijtage- en smering processen). Een algemeen model van slijtage processen is beschreven in hoofdstuk 4. Hoofdstuk 5 beschrijft een geval uit de praktijk. Het betreft de excessieve slijtage die optrad bij een baggerwerk in Port Hedland, Australië in 1985. Conclusie van dit praktijkvoorbeeld was dat terreinonderzoek voor baggerwerken moet leiden tot een op de geologie gebaseerd ruimtelijk model, waarbij de mineralogische samenstelling van de verschillende gesteentetypen eveneens vastgesteld zou moeten worden. Tevens werd duidelijk dat aandacht gegeven moet worden aan de recente geologische geschiedenis van een kustgebied, in verband met de sterke zeespiegelveranderingen van meer dan 100 meter. Studie van de landschapsvormen en de geologie op het land, langs de kust van Port Hedland, heeft bij het vervolgonderzoek geholpen een duidelijk beeld te vormen van de gesteentemassa die onder de waterspiegel gebaggerd werd. Deze beide conclusies gelden niet alleen voor het Port Hedland project. De ervaring leert dat veel baggerprojecten worden aangenomen met summiere of gebrekkige geologische informatie. Bovendien bleek dat de vaak gehanteerde classificatie van grond en gesteente voor baggerwerken van de PIANC (Permanent International Association of Navigation Congresses) onvoldoende is voor de beoordeling van het snij- en slijtage proces wat optreedt bij het baggeren van rots.

In het tweede deel van het proefschrift, deel B (hoofdstuk 7-14), wordt bestudeerd welke eigenschappen van gesteente van invloed zijn op het mechanisch snijden en op de slijtage van snijtanden. Hoofdstuk 8 gaat in op het belang van de aanwezigheid van natuurlijke scheuren (*discontinuïteiten*) in de gesteentemassa. Het ruimtelijke patroon van deze discontinuïteiten deelt gesteente vaak op in een patroon van gestapelde blokken van intact gesteente. De grootte en vorm van deze blokken zijn van belang voor het mechanische uitgravingsproces. De potentiële snedediepte van een mechanische snijmachine kan vergeleken worden met de afstand van discontinuïteitsvlakken in een gesteente om vast te stellen of het uitgraven vergemakkelijkt zal worden. Indien de discontinuïteiten een zodanige oriëntatie en ruimtelijke structuur hebben dat te verwachten is dat een machine steenblokken los zal maken langs de discontinuïteiten (een proces dat *rippen* genoemd wordt), dan zal naar verwachting de sterkte van het gesteentemateriaal nauwelijks tot geen rol spelen bij het uitgravingsproces. In andere gevallen zal de machine (de snijkop en de tanden) geheel of gedeeltelijk in massief (intact) gesteente snijden.

In hoofdstuk 9 wordt de huidige kennis over het snijden van intact gesteente beschreven. Deze kennis is voornamelijk gebaseerd op proeven uitgevoerd in het laboratorium. De eenvoudige snijtheoriën gebruiken de druksterkte en/of de treksterkte om snijkrachten te voorspellen. Tegenwoordig is algemeen aanvaard dat tijdens het snijden van gesteente een complexe spanningssituatie rond de beitel aanwezig is, waarbij aan de tip van de beitel zeer hoge compressieve spanningen leiden tot de vorming van een vergruizingszone. Vanuit de vergruizingszone ontstaan schuifbreukjes, die naar het gesteente-oppervlak overgaan in trekscheuren. Dit brosse verspaningsproces leidt tot de vorming van gesteenteschilfers. Voor een meer complete beschrijving van het bezwijkproces van brosse gesteenten zou daarom een beschrijving van de complete bezwijkkromme in de spanningsruimte nodig zijn. Voor het doel van dit onderzoek is getracht om met eenvoudige gesteentemechanische proeven een benadering te geven van de complete bezwijkkromme. In het onderzoek is de Hoek-Brown bezwijkkromme hiervoor gebruikt (Appendix D), die het brosse deel van het bezwijken beschrijft. Bij de complete bezwijkkromme speelt de bros-taai overgang een rol. De bros-taai overgang is gerelateerd aan de verhouding tussen druk- en treksterkte. Bij de hoge spanningstoestanden in het taaie gebied vergruist het gesteente. Uit de praktijk van het mechanisch snijden is bekend dat gesteenten die taai zijn tijdens het snijden (dat wil zeggen dat er een grote vergruizingszone rond de beitel is en geen vorming van schilfers) extreme verhitting van het beitel oppervlak kunnen veroorzaken, waardoor adhesieve slijtage kan optreden (verweking van het beitel oppervlak). Of dit type van slijtage zal optreden zal afhangen van de sterkte van het gesteente in verhouding tot het vermogen en het ontwerp van de snijmachine.

Het snijproces in intact gesteente kan beschreven worden in termen van snijkrachten, specifieke energie, en slijtsnelheid van de beitels. De bestudeerde snij-experimenten laten zien dat in het onderzochte gebied de specifieke snij-energie afneemt met toenemende snijdiepte. De uitbreekhoek van de gesteenteschilfers bepaald de afstand die beitels van elkaar moeten hebben voor een goede interactie tijdens het snijden. De invloed van snijsnelheid, temperatuurontwikkeling in de beitel, het bot worden van de beitel en het snijden onder water wordt eveneens besproken in hoofdstuk 9.

Wat betreft de slijtage is het contrast in hardheid tussen de beitel en het gesteente een belangrijke parameter. Methoden om de hardheid van mineralen en gesteenten te bepalen worden besproken in hoofdstuk 10. Hoofdstuk 11-13 behandelen experimenteel werk naar de abrasiviteit van gesteenten. Deze experimenten zijn uitgevoerd om een inzicht te krijgen in de slijtageprocessen die optreden tussen de beitel en het gesteente. De eerste experimenten zijn uitgevoerd met behulp van een speciale draaibank waarop een pen-op-schijf opstelling was gemonteerd. In plaats van het gebruikelijke doel van de pen-op-schijf proef, het vergelijken van de slijtage gevoeligheid van verschillende soorten staal op een abrasieve schijf, werden nu schijven van verschillende soorten gesteente beproefd tegen een standaard pen. Er zijn kunstmatige gesteenten gebruikt, waar gesteenteparameters zoals kwartsgehalte, korrelgrootte van kwarts, treksterkte van het gesteente, soort abrasief (andere mineralen dan kwarts) zijn gevarieerd. De pen-op-schijf proef simuleert twee-lichamige slijtage (gesteente tegen staal) en er waren plannen om ook drie-lichamige slijtageproeven (gesteentegruis tussen gesteente en staal) op te zetten. Het bleek echter dat de interpretatie van de pen-op-schijf proef resultaten niet eenvoudig was. Dit had voornamelijk te maken met de variabiliteit van gesteente eigenschappen. Proefresultaten van verschillende soorten gesteenten konden niet direct met elkaar vergeleken worden. De proeven gaven wel inzicht in de slijtage mechanismen die optraden en illustreerden goed het fundamentele principe uit de tribologie, dat slijtage een systeem gebonden proces is.

Kritische beschouwing van de gangbare proeven die gebruikt worden om de abrasiviteit van gesteente vast te stellen, leidde tot dezelfde conclusie: standaard proeven kunnen hoogstens van nut zijn om een indruk te geven van abrasiviteit. Er zijn teveel factoren die de uitkomst van de proef bepalen. Vaak treedt bij verschillende gesteentetypen een ander slijtagemechanisme op tijdens de proef. Bovendien verschillen de proef omstandigheden teveel van de wijze van snijden die optreedt bij snijmachines, laat staan snijkopzuigers.

Deketh (1995) bestudeerde de processen die optreden bij het binnendringen van een beitel in het gesteente. Het bleek dat de meeste slijtage optrad bij het eerste contact van de beitel met het gesteente (hoge temperatuur adhesieve- en twee-lichamen abrasieve slijtage). De slijtage snelheid nam af wanneer de beitel voldoende diepte had gekregen en het gesteente ging snijden (drie-lichamen abrasieve slijtage).

Uit het experimentele onderzoek is naar voren gekomen welke gesteente-eigenschappen invloed hebben op de slijtage van beitels bij abrasieve slijtage. Dat zijn de sterkte parameters (druk- en treksterkte), het gehalte aan abrasieve mineralen (mineralen harder dan het beitelmateriaal tijdens het snijden) en de korrelgrootte van de abrasieve mineralen. De klassieke slijtage factor (Schimazek's factor F), het product van treksterkte, korrelgrootte en hardheid van mineralen (uitgedrukt ten opzichte van de hardheid van kwarts), blijkt bij de meeste laboratorium proeven redelijk te correleren met abrasieve slijtage. Wat betreft het vastleggen van de potentiële abrasiviteit van gesteente kan als conclusie worden getrokken dat de combinatie van een sterkteproef op het gesteente en een petrografisch onderzoek van het gesteente (waaruit gegevens over hardheid en korrelgrootte volgen) daartoe het meest geschikt is.

Uit deel A en deel B van het onderzoek is gebleken dat de volgende gegevens van de gesteentemassa van groot belang zijn om vast te stellen of machinaal uitgraven (c.q. baggeren) technisch en economisch mogelijk is:

1. De dichtheid en het geometrisch patroon van scheuren (discontinuïteiten). Dit patroon zal bepalen in welke mate het uitgraven door rippen, snijden of door een combinatie van beide zal geschieden.

2. De druk- en treksterkte van het intacte gesteentemateriaal. Beide zijn nodig in verband met een schatting van de mate van brosheid of taaiheid.

3. De mineralogische samenstelling en de korrelgrootte van de mineralen in het gesteente (petrografisch onderzoek). Samen met de sterkte van het gesteente kunnen gevolgtrekkingen gemaakt worden over de abrasiviteit. Daartoe zijn slijtproeven minder geschikt.

Verder is vast komen te staan dat het belangrijk is dat voldoende penetratie van de snijtand of beitel plaatsvindt tijdens het machinaal snijden. De productie is dan het beste (optimale specifieke snij-energie), maar ook het slijtage proces zal relatief gunstig zijn (drie-lichamige abrasie). Tijdens de indringfase van de beitel (die bij snijkoppen cyclisch plaatsvindt) treedt de meeste slijtage op. Bij een taai gesteente is de mogelijkheid van hoge temperatuur adhesieve slijtage van de beitels aanwezig.

In deel C (hoofdstuk 15-19) wordt bekeken of de bovengenoemde punten bevestigd worden door de praktijk. Het was mogelijk om de gegevens van het tandenverbruik en de productie van de snijkopzuiger *Kunara* te bestuderen aan de hand van het Sydney Harbour tunnel project. Ook konden gegevens van het tandverbruik van tunnelsnijmachines in hetzelfde gesteente, Hawkesbury Sandstone, bestudeerd worden (hoofdstuk 16). Er is getracht een schatting te maken van de slijtagefactor F van de gesteente-eenheden waarin uitgegraven is. Bovendien waren er gegevens van laboratorium snij- en slijtageproeven. De gegevens ondersteunen de these dat de *F*-factor (of beter: de combinatie van sterkte, mineralogie en korrelgrootte van het gesteente) een goede indruk geeft van de mate van abrasieve slijtage van de tanden.

Sinds 1993 konden regelmatig opnamen gemaakt worden van de prestaties van een bepaald type sleuvengravers (Vermeer T-850 rock cutting trencher). Deze opnamen dienden ter verificatie van de modelvorming gebaseerd op het literatuur- en experimentele onderzoek (hoofdstuk 17). Het effect van de discontinuïteiten is duidelijk naar voren gekomen: er zijn duidelijke gebieden aan te wijzen waar rippen optreedt. Bovendien treedt in die gebieden nauwelijks slijtage op aan de tanden. Bij toenemende blokgrootte is er een overgang naar snijden in het intacte, massieve gesteente. Daar kon inderdaad waargenomen worden dat bij lage penetratiediepte excessieve adhesieve slijtage optrad. Bij snijden was in het algemeen de slijtage graad lager. Beitelconsumptie hing voornamelijk af van breuk, veroorzaakt door sterk gesteente. Bij het compileren van de gegevens uit de verschillende projecten, kwam echter duidelijk naar voren dat meerdere factoren een rol spelen. Zo gedraagt de kalksteen groep zich duidelijk verschillend van de groep silicaatgesteenten. Ook was het moeilijk de invloed van de taaiheid van het gesteente op slijtage vast te leggen. Met de klassieke statistische correlatietechnieken waren nauwelijks relaties te leggen. Daarom zijn de gegevens verwerkt met behulp van regels opgesteld met behulp van vage logica (fuzzy logic; hoofdstuk 18.5). Deze techniek lijkt geschikt om de prestatie van graafmachines in gesteente vast te leggen. Een fuzzy model gebaseerd op gegevens van 16 projecten kan de productie en beitelconsumptie voorspellen, waarbij rekening gehouden wordt met blokgrootte en de sterkte van het gesteente. Ook beitelslijtage is op deze manier gemodelleerd.

In hoofdstuk 18 wordt verder een overzicht gegeven van de methoden die gebruikt zijn voor de voorspelling van de prestaties van graafmachines. Aandacht is besteed aan tunnelboormachines, bulldozers met rippers en snijkopzuigers. Uit het onderzoek dat tot nu toe gedaan is aan tunnelboormachines en ripper bulldozers blijkt dat grote hoeveelheden gegevens nodig zijn om een redelijke analyse van voortgangssnelheden of productie te kunnen maken. De gesteentemassa classificatiesystemen die gebruikelijk zijn in de ingenieursgeologie kunnen op zijn hoogst een indruk geven van de mogelijke uitgravingstechniek, zeker niet van productie hoeveelheden. Analyse van de gegevens van het Ra's Laffan (Qatar) project, waar de prestaties van de snijkopzuiger *Leonardo da Vinci* in kalksteen zijn bestudeerd, leidde tot dezelfde conclusie. Ook voor deze toepassing lijkt fuzzy-logic modelleren een goede mogelijkheid, zeker bij gebrek aan gegevens.

In deel D (hoofdstuk 20-24) wordt tenslotte een overzicht gegeven van de implicaties voor het terreinonderzoek voor nieuwe projecten waar gesteente gebaggerd moet worden. Van belang is ten eerste het verkrijgen van een goed drie-dimensionaal model van de ondergrond. Er is een goed geologisch model nodig van de verschillende grond- en gesteente-eenheden ter plaatse. Vervolgens wordt een indeling gemaakt in verschillende *ingenieursgeologische eenheden.* Deze eenheden worden gekozen aan de hand van zowel de geologische (lagen) opbouw als een karakteristiek bereik van geotechnische eigenschappen. Belangrijke informatie zoals laagdikte, discontinuiteit dichtheid en -oriëntatie (blokgrootte en -vorm), materiaal sterkte en -taaiheid, abrasiviteit (bepaald door sterkte en petrografie), bepaalt de keuze van de eenheden. De eenheden vormen onderdeel van het *geotechnisch model,* dat tegenwoordig in 3D-GIS systemen ingevoerd kan worden. Op de onderscheiden ingenieursgeologische eenheden kunnen berekeningen uitgevoerd worden, om een idee te krijgen van te verwachten productie of beitel- of tandverbruik (hoofdstuk 21). In hoofdstuk 22 worden relevante aspecten besproken die te maken hebben met het verzamelen van de technische gegevens over de materiaaleigenschappen van het gesteente en de discontinuïteiten. In hoofdstuk 23 wordt een samenvattende conclusie van het werk gegeven, aan de hand van twee stroomdiagrammen (zie Figuur 23.1). Bovendien worden aanbevelingen gedaan hoe tijdens het baggerproces de productie- en slijtage gegevens gemonitored kunnen worden. Van groot belang is het opbouwen van gegevensbanken, waar de prestaties van de snijkopzuigers vergeleken worden met de in dit werk naar voren gekomen gegevens over het gesteente. Het is niet ondenkbaar dat in de nabije toekomst het drie-dimensionale geotechnische model, met ingebouwde fuzzy expert modellen die productie en slijtage beschrijven, direct aan boord van de snijkopzuiger vergeleken kan worden met gegevens over specifieke snij-energie. Daarmee komt dan direct de gezochte informatie vrij over het tribologisch systeem, die nodig is om veel betrouwbaarder voorspellingen te kunnen maken over productie en slijtage bij verdere voortgang van het project.

Delft, september 1997
Peter N.W. Verhoef

CURRICULUM VITAE

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Scholing:

1966-1971 Eindhovens Protestants Lyceum (HBS-B)
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1978-1982. Wetenschappelijk assistent aan de Rijksuniversiteit van Leiden. Samen met de Faculteit Geologie overgeplaatst naar het Instituut voor Aardwetenschappen, Rijksuniversiteit Utrecht (1980). Betrokken bij onderzoek naar de structurele geologie van ZW-Finland, de Italiaanse Alpen en de Pyreneeën (structuuranalyse en microscopische en electronenmicroscopische studies van gedeformeerde metamorfe gesteenten). Wetenschappelijk secretaris van een internationaal symposium over opslag van hoogradioactief afval in zoutkoepels (1979) en betrokken bij de voorbereiding van een onderzoekprogramma op dit gebied.

1982-heden. Universitair docent aan de Technische Universiteit Delft, Faculteit Mijnbouwkunde en Petroleumwinning (sinds 1997 Subfaculteit Technische Aardwetenschappen). Doet onderzoek en advieswerk en geeft les op het gebied van ingenieursgeologie, gesteentemechanica, geologische constructiematerialen, terreinonderzoek en ingenieursgeologische veldwerktechnieken. De afgelopen jaren onder andere projectleider geweest van de ingenieursgeologische haalbaarheidsstudie voor een OPAC (Ondergrondse Pomp Accumulatie Centrale) in Nederland (1984-1987), was betrokken bij studies naar aardverschuivingen en modderstromen in Italië, Colombia en Oostenrijk, de stabiliteit van ondergrondse holruimten in de kalksteenmijnen van Zuid-Limburg, terreinonderzoek voor een irrigatie project in de Ogaden, Ethiopië (dam fundatie op oplosbaar gesteente, 1986). Advieswerk op het gebied van excessieve slijtage van baggerwerken (o.a. Port Laem Chabang, Thailand, 1989). Van 1989-1997 onderzoeker bij, en projectleider van, het door de Stichting Technische Wetenschappen (STW) gesponsorde project "Abrasive wear of cutting tools by rock" en het project "Geomaterialen", ondersteund door de Dienst Weg- en Waterbouw (DWW) van Rijkswaterstaat.

SUMMARY

Wear of rock cutting tools: Implication for the site investigation of rock dredging projects

Huge volumes of rock are commonly being dredged by large suction cutter dredgers. This is only possible if permitted by the technical properties of the rocks involved. Only weak to moderately strong rocks (unconfined compressive strength lower than 50 MPa) can be dredged today. Stronger rocks may be dredgeable as well, if the rock mass is transected by fractures. Such fractures may be naturally occurring in the rock (joints, faulting, shear zones) or artificially induced by drilling and blasting the rock. Despite the fact that information on the strength of the rock and the degree of fracturing is known before a dredging project starts, unpleasant surprises regularly occur. Especially at the cutterhead problems with excessive wear of the cutter teeth lead to dramatic reductions in production. Very high financial losses are experienced in such cases. The reason for this study is therefore the unreliability of the estimation of the amount of cutter teeth needed for a project. The present research has been carried out to clarify the rock engineering and geological factors involved in the mechanical excavation of rocks, with special emphasis on the factors that influence wear of the cutting teeth.

The problem has been studied from the perspective of Engineering Geology, a discipline that deals with the site investigation for civil engineering projects. After examination of a number of dredging projects at which excessive wear took place, several points were identified:

1. Many projects are carried out using incomplete and poor information on the geology and the nature of the rocks. Vital information is regularly lacking, such as the quartz content of the rocks. It is well-known that quartz is harder than dredging tooth steel, and hence is abrasive to steel.

2. Clarification of the factors that are involved in the mechanical excavation of rock is needed.

3. It is not clear which wear processes occur at the teeth. High wear rates occur not only in quartz-rich rocks, but sometimes also in limestones without quartz. Calcite, the rock-forming mineral of limestone, is softer than steel and hence not expected to be abrasive for steel. The dredging contractors expressed the wish to develop a simple test to determine abrasiveness.

This thesis gives the results of the research that was carried out over the past ten years under the direction of the author.

In Part A (Chapters 2-6) an overview is given of the wear problems that may occur while dredging rock with cutter suction dredgers. The problem is defined in Chapter 2. In Chapter 3 cutter suction dredging is described and how the cutterhead cuts into rock. The research commenced with a fundamental description of the wear processes using methods from tribology (science of friction, wear and lubrication; Chapter 4). Chapter 5 describes a case from practice, the excessive wear that occurred at the dredging work in Port Hedland, Australia in 1985. Conclusion of this case study was that site investigation for dredging works should result in a three-dimensional model of the subsurface, based on the geology, and that the mineralogical composition of the rocks involved also should be established. This case study also made clear that attention should be given to the recent geological history of the coastal area, with regard to the strong fluctuations of sea-level of more than 100 metres. A study of the morphology of the surface and the geology on-land, along the coast of Port Hedland, has helped in the investigations that followed to make a clear picture of the rock mass that was dredged below the water surface. These two conclusions are not only valid for the Port Hedland case. Experience teaches that many dredging projects taken on are based on poor geological information. On its own the commonly used PIANC (Permanent International Association of Navigation Congresses) *classification for soils and rocks to be dredged* is insufficient for the assessment of the cutting- and wear process that occurs during the dredging of rock.

The second part of the thesis, part B (Chapter 7-14), deals with the rock properties influencing the mechanical cutting of rock and the wear of cutting teeth. Chapter 8 discusses the importance of the presence of natural fractures (*discontinuities*) in the rock mass. The spatial distribution of these discontinuities often dissects the rock into a stacked pattern of blocks. The size and shape of these blocks influences the mechanical excavation process. The potential cutting depth of a mechanical excavation machine can be compared with the separation of the discontinuity planes in a rock mass, to establish whether the excavation will be facilitated. If the orientation and spatial distribution of the discontinuities is such that the machine can release the rock blocks along the discontinuities (a process called *ripping*), the intact rock material strength is not expected to influence the excavation process much. In other cases the machine (cutterhead and teeth) will cut partially or completely cut into intact rock.

In Chapter 9 the current knowledge on the cutting of intact rock is described. This knowledge is mainly based on experiments carried out in the laboratory. The simple rock cutting theories use the unconfined compressive- and/or tensile strength to predict cutting forces. It is generally accepted that during the cutting of rock a complex stress distribution is present around the chisel. At the tip of the chisel very high compressive stresses lead to the formation of a crushed zone. From the crushed zone shear fractures are generated which, towards the free rock surface, become tensile fractures. This process describes the formation of rock chips during brittle cutting. To describe the process fully, a definition of the complete failure envelope of the rock in stress space would be needed. For the purpose of this research attempts have been made to estimate the failure envelope using simple rock tests. The Hoek-Brown failure criterion has been used, which describes the brittle part of the failure envelope. The brittle-ductile transition is important in the definition of the complete failure envelope (Appendix D). This transition is related to the ratio of compressive to tensile strength. At high stress states in the ductile field the rock is crushed. From practice it is known that rocks which are ductile during cutting, which implies a large zone of crushed rock around the chisel tip with no formation of rock chips, can cause extreme heating of the chisel surface. Weakening of a thin veneer of steel on the chisel surface may result in adhesive wear. Whether this type of wear will occur or not depends on the strength of the rock relative to the power and the design of the cutting machine.

Cutting of intact rock can be described in terms of cutting forces, specific energy and wear rate of the cutting tools. The cutting experiments that have been studied show that, in the investigated range, specific energy decreases with increasing cutting depth. The sideward outbreak angle of the rock chips determines the spacing of cutting teeth for optimum interaction during cutting. The influence of cutting speed, temperature development in the chisel, the blunting of chisels and the cutting under water is discussed in Chapter 9.

Concerning wear, the hardness contrast between tool steel and the rock is an important parameter. Methods to determine the hardness of minerals and rock are discussed in Chapter 10. Chapters 11-13 treat experimental work on abrasiveness of rock. Experiments have been carried out to understand the wear processes that take place between chisel and rock. The first experiments were carried out using a special lathe mounted with a pin-on-disc test arrangement. Pin-on-disc tests are used in tribology to compare the wear sensitivity of different types of tool materials. In this case the test was used to compare different types of rock discs against a standard tool pin. Artificial rock types were tested, in which rock parameters like quartz content, quartz grain size, tensile strength of the rock, type of abrasive (other minerals than quartz) were varied. The pin-on-disc test simulates two body wear (rock against steel) and there were plans to perform also three-body wear tests (crushed rock powder between steel and rock). However, it turned out that the interpretation of the pin-on-disc test was not simple, mainly due to the variability of the rock properties. Test results of different rock types could not be compared, because wear processes seemed dependent on rock type. But the experiments gave insight into the wear mechanisms operating and illustrated the fundamental principle from tribology, that wear is a system dependent process.

Critical examination of common tests that are used to determine abrasiveness of rock led to the conclusion that standard tests can only be of use to give an impression of abrasiveness. Too many factors are involved that determine the outcome of a test. Often in different rock types another wear mechanism occurs during the test. More importantly, test conditions generally differ too much from the wear conditions present during cutting with cutting machines, and certainly rock cutting suction dredgers.

Deketh (1995) studied the processes that occur during the penetration of a chisel into rock. It was found that most wear occurred at the first contact of the chisel with the rock (high temperature adhesive wear and two-body abrasive wear). The wear rate decreased when the chisel had sufficient cutting depth and started to cut the rock (three-body abrasive wear).

From the experimental work the rock parameters influencing the wear of chisels during abrasive wear were derived. These were the strength parameters (compressive- and tensile strength), the amount of abrasive minerals (minerals harder than the tool material during cutting) and the grain size of abrasive minerals. The wear factor first described by Schimazek and Knatz (1970) appears to correlate

reasonably well with abrasive wear in most laboratory tests. This *F-value* is the product of tensile strength, grain size and hardness of minerals (where hardness is expressed relative to the hardness of quartz). It can be concluded that the determination of the potential abrasiveness of rock can best be done by a combination of a strength test and a petrographic examination of the rock (from which information on hardness and grain size follow).

From Part A and Part B of the thesis it follows that the following information about the rock mass is needed to examine the technical and economic prospects for mechanical excavation (including dredging):

1. The density and the geometrical pattern of fractures (discontinuities). This pattern will determine the degree in which ripping, cutting or a combination of both will occur.

2. The compressive- and the tensile strength of the intact rock material. Both are needed to estimate the degree of brittleness or ductility.

3. Data on the mineralogical composition and the grain size of the minerals in the rock (petrographic examination). From this, together with the strength of the rock, conclusions may be drawn on the abrasiveness. Wear tests are less suitable for this purpose.

Furthermore, during mechanical cutting, it is important that sufficient cutting depth of tooth or chisel is achieved. Not only production will be at its best (optimal specific energy), but also the wear process will be relatively favourable (three-body abrasive wear). During the penetration phase of the chisel, which occurs cyclically using cutterheads, most wear occurs. While cutting a ductile rock, the possibility of high temperature adhesive wear of the cutting tools is present.

Part C (Chapter 15-19) examines whether or not the above four points are confirmed by practice. It has been possible to study the data on production and tool consumption of the cutter suction dredger *Kunara* dredging a trench for the Sydney Harbour tunnel. Also the data on the performance of roadheader tunnelling machines in the same rock, Hawkesbury Sandstone, could be studied. Data of laboratory rock cutting and abrasion tests were available as well. The information supports the proposition that the *F-value*, or better the combination of strength, mineralogy and grain size, gives a good indication of the degree of abrasive wear of the rock cutting tools.

Since 1993 regular surveys could be made of the performance of Vermeer T-850 rock cutting trenchers (Chapter 17). These observations served to verify the ideas based on the literature and experimental investigation. The effect of discontinuities was clearly illustrated. Rock masses with a block size below a threshold value were clearly excavated by ripping. Hardly any wear occurs at the cutting bits in these cases. With increasing block size a transition occurs towards cutting into the intact, massive, rock. In that case it could be observed that adhesive wear of the bits occurred at shallow penetration depth. During cutting in general the wear rate was relatively low. Tool consumption mainly depended on breakage, caused by strong rock. While studying the data coming from different projects, however, it appeared that many factors played a role. The limestone rock group clearly behaves differently from the silicate rock types. It was also very difficult to verify the influence of rock ductility on wear. Hardly any relationships could be established (using the classical statistical correlation techniques).

Chapter 18 gives an overview of the techniques that are used to predict the performance of mechanical excavation machines. Tunnelboring machines, bulldozer rippers and cutter suction dredgers are discussed. From the published research on tunnelboring machines and rippers it is clear that large amounts of data are needed to make a reasonable analysis of advance rates or production. The rock mass classification systems that are common in Engineering Geology can only give an indication of the excavation technique to be used, not of production rates. Analysis of the data from the Ra's Lafan project, where the performance of the cutter suction dredger *Leonardo da Vinci* in limestone has been examined, led to the same conclusion. To interpret the data from the trencher project, fuzzy logic modelling was done. Results of 16 projects were used to model production and tool consumption. Also bit wear was modelled this way. The results are promising and it seems that this technique is suitable to model mechanical rock excavation, including rock dredging.

In Part D (Chapter 20-24) the implications for site investigation for rock dredging contracts are discussed. Firstly it is important to obtain a good three-dimensional model of the subsurface. Therefore a good geological model is needed of the different soil and rock units present at the site. Secondly a division into *engineering geological units* is made. These units are based on the geological (layering) framework and the range of geotechnical properties of the rock and soil types present. Important information like layer thickness, discontinuity density and geometry (block shape and block size), material strength and ductility, abrasiveness (determined by strength and petrography), decides the choice of the units. These units are part of the *geotechnical model*, which nowadays can be fed into 3D-GIS systems. Calculations can be performed using the properties of the engineering geological units, to obtain an impression of the expected production and tool consumption (Chapter 21). Chapter 22 discusses relevant aspects of the collection of geotechnical data on discontinuities and intact rock properties.

In Chapter 23 an overall conclusion of the work is given, using two flow diagrams (Figure 23.1). Recommendations are given to monitor, during dredging, the production and wear data concurrent with data on the rocks being dredged. It is of the utmost importance to build up data bases, so that the performance of cutter suction dredgers can be compared with the rock properties shown to be relevant in this research. Probably in the near future the three-dimensional geotechnical model, with built-in fuzzy expert models that describe production and wear, can be compared directly on board of the vessel with data on specific energy. In this way the information needed on the tribological wear system is generated and can be used to make much more reliable predictions about production and tool consumption for the remainder of the dredging contract.

Delft, September 1997
Peter N.W. Verhoef

CHAPTER 1

Introduction

Rock dredging is a new development. In the sixties rock cutter suction dredgers were used in the United States and somewhat later the Dutch dredging contractors also started with rock cutting dredging. Rock is quite a different material from the soils the dredging contractors are used to deal with. Dredging was very much a craft that has to be learned by experience and this also appears to be the way one learns rock dredging nowadays. The rock cutter dredgers can handle fairly difficult ground and apart from rock they dredge also weathered rock and difficult soils.

In the past decade unpleasant surprises have occurred frequently when dredging rock. Sometimes misjudgment can have very serious consequences. In the early eighties a large Dutch dredging contractor experienced excessive wear of dredging equipment at Port Hedland, Australia. Financial losses of over 100 million Australian dollars were claimed, and the contractor stated that the site investigation reports gave no warning of the difficult ground conditions leading to high abrasive wear of cutter teeth. Examination of the site conditions by a group of experts lead to the conclusion that indeed ground conditions were not described adequately in the geotechnical documents available to the contractor. This claim was settled for Aus $ 39 million; the largest settlement ever paid in Australia for a geotechnical project up to date (Coffey & Partners 1990 pers. comm.).

Dredging contractors are highly dependent on the site investigation reports available to them when they are assessing a project at the tendering stage. Normally not much time is available. In this short period important decisions have to be made about the type of dredging equipment to be used, the expected production rate and the amount of replacement needed for equipment liable to wear, such as pumps, pipes and the cutting teeth and cutters of the dredgers.

Clearly the type of ground to be dredged has a major influence on the decisions made. Any misjudgment from the production estimators may have important consequences, in the short term on the bidding during the tendering stage, but in the long term on the dredging operation itself, once a contract has been acquired.

Complications may arise when the information on the ground conditions is such that the contractor has not obtained a correct impression of the type of soils and rocks present. Even site investigation reports that look exhaustive and contain all the parameters that the contractor normally uses for his estimates can be misleading.

The Port Hedland case was one of the first in which the Engineering Geology

Section of the Faculty of Applied Earth Sciences (formerly Mining & Petroleum Engineering) of Delft University of Technology was involved, as Professor David Price was asked to give his expert opinion. Another case, somewhat later in time, was at Port Laem Chabang (Thailand), where granitic soils were dredged. Also here excessive wear was experienced. After examination of the ground conditions, it was also concluded that the soils were not described adequately.

Apparently practice in site investigation for dredging projects is such that major mistakes can occur frequently. This situation was the impetus to start this study. The problem of setting up an adequate site investigation programme for a particular project is the central theme of Engineering Geology. Site investigation is needed to define the geological factors of influence on a dredging project and to assemble geotechnical data. The geotechnical data are used to determine the type of dredging equipment to be applied and to estimate excavation production rates.

Since 1986 the Section of Engineering Geology has been studying the problem of wear of dredging cutting tools. It was proposed at a CEDA (Central Dredging Association) meeting in Delft in May 1986 to approach the problem using common engineering geological methods (Verhoef 1986). The alternative to solutions based on mechanical engineering appealed to the Dutch dredging community. A result of this meeting was, among others, that Dutch dredging contractors regularly started to send rock specimens to Delft for petrographic examination. The Section of Engineering Geology became involved in a working group with the task of designing a useful abrasiveness test for rock. This working group was funded by the CSB (Combinatie Speurwerk Baggertechniek; Dredging Research Association) and the development of the test, a cutting test on a shaper, was carried out by the Section of Soil Movement of the Faculty of Mechanical and Maritime Engineering (Van der Sman 1988, Davids & Adrichem 1990, Miedema 1990, Bisschop 1991). A proposal was made for a research project to STW (Stichting Technische Wetenschappen; Technology Foundation). The project, formulated and led by the writer, was granted to the Section of Engineering Geology for a period of four years, starting in January 1989. In 1993 a new grant for an additional four years was given to the project.

1.1 SITE INVESTIGATION FOR ROCK DREDGING

An interesting paper by Stone (1991) presents a summary of the art of site investigation for dredging works by someone with more than 30 years of practical experience. Stone stresses that: "the most frequent criticism of any dredging project that has been executed over the last 30 - 40 years, is that there is *insufficient* soil information. The second major criticism is that there is no general format that enables easy comparison of work by the different sampling companies or soil investigation contractors." According to Stone the essential requirement of the site investigation, to properly price a dredging contract, should be to assess the *dredgeability* of the rock or soil material. Stone summarizes the most important properties needed for his purpose: "In the most simple form, this requires the measurement of the in-situ soil strength, plus a grading or sieve curve. If the material is not homogeneous, or difficult to dredge, additional information is needed, regarding hardness, abrasiveness, angularity (side slope stability), the thickness of layers or the amount of separation of the strata, direction and trend of the interface of the material etc, but the *primary* interest is the soil strength and sieve curve." For rocks, Stone mentions a list of strength and deformation tests that could be executed.

The present work has to be seen against this background. The impulse to start this study came from the excessive wear problems that sometimes faced the contractors. Part of the problem stems also from insufficient site investigation and lack of understanding or coherence in site investigation reports.

1.2 OUTLINE OF THE RESEARCH PROJECT

The central purpose of the research project was to try to achieve an improvement in the prediction of rock cutting tool wear. During the preparation of the research programme two main areas that needed clarifying were identified. These were:

1. Examination of the rock cutting and tool wear process from the perspective of the rock properties. The main emphasis in the past had been mainly from a mechanical engineering perspective, with emphasis on the improvement of the tools.
2. The need for a thorough geological component in the site investigation for dredging projects, to be carried out by a team including an engineering geologist and a professional engineer conversant with the details of dredging practice.

Both aspects were to be addressed in the research. The first would be supported by laboratory studies, literature research and observations of rock cutting operations, the second by field studies of actual dredging projects.

1.2.1 *Laboratory and desk studies*

The main emphasis of the research was intended to deal with rock cutting tool consumption and assessment of cutting tool wear. The following subjects to be addressed in the research were defined:

- the characteristics of rock cutting equipment (rock cutting dredgers, tunnel boring machines, road headers, rippers, trenchers)
- the mechanics of rock cutting (wedge penetration)
- the local heating of tool and rock during mechanical excavation
- other factors (such as water pressure, cavitation, heat) of relevance to the cutting process
- the influence of rock mass structure (discontinuities) on the cutting process
- rock mechanical factors (intact rock failure)
- the brittle-ductile transition of rock failure and its influence on rock cutting and tool wear
- the process of abrasive wear
- the abrasive capacity of rock (influence of mineralogy, microscopic structure, abrasion process, abrasion tests, definition of "hardness")
- the microstructure of rock in relation to abrasive capacity
- rock microstructure classification

- the microscopic failure mechanisms of the different classes of rock; the nature of abrasion
- the implications for interpretation of abrasion index tests

In Engineering Geology practice it is customary to use simple field or laboratory tests to get an indication of engineering properties of soils or rocks. For example, to determine the abrasiveness of rock, the Cerchar scratch test is used, or the Schimazek *F*-value determined. The experimental part of the study was aimed at testing the validity of this approach. It was decided to use artificial rock, consisting of mortar-mineral mixtures, to be able to test the influence of various rock properties, such as strength, brittleness, hard mineral content, grain size of hard minerals, angularity of grains and hardness of minerals. The theoretical background of wear processes from the science of tribology was consulted.

Experiments were carried out by students of the Section of Engineering Geology of the Faculty of Mining and Petroleum Engineering as part of their MSc thesis. Cutting experiments were performed by Reinking (1989) and Bisschop (1991). Pin-on-disc tests were carried out by Van den Bold and Vermeer (Verhoef et al. 1990) and Deketh (1991).

In 1990 a research visit was made by the author to Sydney (Prof. Frank Roxborough, School of Mines, University of New South Wales) which proved very useful. Westham Dredging Company provided information on the rock cutting done for the Sydney Harbour Tunnel. Borehole data and test results of the site investigation reports were compared with the actual tool consumption taking place. Also cutting and abrasion tests were performed, using steel and tungsten carbide test chisels (Verhoef 1993).

The first two years of the project were used for the construction of a pin-on-disc test rig. It was decided to use a special type of lathe, where the feed is coupled to the rotation velocity, to ensure constant velocity under the pin or chisel. A variety of test set-ups can be built for this lathe (pin-on-disc, cutting-, abrasion set-ups or mini-disc cutter). An old lathe (1960) once used by Technical University Eindhoven was obtained for this purpose. The pin-on-disc test has been examined and used on a range of artificial and natural rocks (Deketh 1991). The results were interesting and useful for the development of theory, but the test proved not ideal for practical purposes. One major problem was the effect of surface roughness of the test surface, another the inability to test surfaces of strong rock. A major point was that rock cutting is a displacement-controlled process, whereas the pin-on-disc test is load controlled. It was concluded that it would be better to have a test where the development of cutting forces would be a function of the rock material, instead of being imposed on the rock specimen. Therefore a new test has been developed by Deketh, the scraping test. This test has proven to be of great value, and is described in detail in the work of Deketh (1995).

1.2.2 *Field studies*

The integration of dredging field studies with the laboratory work proved a difficulty. Although some promising statements were made, dredging contractors did not allow observations to be made during rock cutting dredging operations. The work done during consulting assignments was not allowed to be published. Several aspects play a role, which are shortly outlined here.

- Since the wear of cutting tools (or the tool consumption rate) is very difficult to predict (as is the production rate), such data is regarded highly confidential. Once a contractor has experience on a site, he keeps the data in his files, to be used for future projects in the same area (it is not uncommon that different projects are carried out over the years at the same harbour site).
- Despite the fact that a poor description of the geological situation can be detrimental to the contractor, he can use this as the basis for a claim based on "unforeseen geological conditions". Experts are used to give a scientific basis for the claim. However, when prior to a court verdict an agreement is made between client and contractor, the contractor does not want to have his relationship with the client spoiled by publications showing the poor quality of site investigation reports approved by the client, again with future works in mind.

The first argument is a valid one. The results of this study show that it is impossible at this stage to make accurate predictions on tool consumption and production.

The second aspect shows that the contractor has conflicting interests. Firstly he wants to accomplish a work in an appropriate way, according to high technical standards. To do this it is necessary to perform high quality site investigations. In fact he would prefer to do such investigations himself, because poor work performed by the clients' engineering consultants always can be used for claims. The better the site investigation reports made by the engineering consultants working for the client, the better the contractor can prepare his work, but the less grounds for claims he has when his predictions prove wrong.

The reason why contractors do not reveal the methods they use to analyze geological data for production and tool consumption estimates is therefore based on competitive arguments. There is ample evidence, however, that this secrecy has kept the average standard of site investigations for rock dredging works on a questionable level. Data that do give information on abrasiveness of rock, like mineralogical composition, is often not gathered during site investigations. In the end there will always be someone, either the contractor, the client, or the tax payer, who has to pay for project failures.

The drawback of this situation for this research project is obvious. In this work it has not been possible to present results of site investigation work relating to rock dredging projects, other than the analysis of the Port Hedland project (Chapter 5) and the Sydney Harbour project (Chapter 16). General points that have drawn attention will be discussed, however.

In 1993, Vermeer International Manufacturing Company (P. Sturm, Regional Export Office, Goes) showed interest in a study of the relationship between rock properties and production and tool consumption of their (on-land) rock cutting trenchers. Since the trenchers are capable of making excavations of several meters deep in even strong rock, these provide an excellent opportunity to compare closely the changes in wear and production to the trench geology. On-land trenches, of course, give better feed back than under-water excavations by rock cutting suction dredgers can ever do. It is not really practical to study the geology under water at the cutter head; normally the rocks excavated can only be monitored at the spray pipe, remote from the excavation site.

Monitoring of trenching projects has been taken place under the supervision of the author. A summary of the results from this work is given in Chapter 17.

It is believed that the theory and laboratory experiments developed in the course of this project relate to rock cutting in general, not specifically to rock cutting dredging. Especially the experimental work (Deketh 1995) is equally, or even better, related to "dry" excavations such as performed by tunnel boring machines, road headers and rock trenchers.

1.3 PRESENTATION OF RESULTS

The results of the research carried out is presented in two books. The present volume treats the implications of the insights gained on wear processes for site investigations involving rock dredging. The volume written by Deketh (1995) treats the experiments performed that have given insight into the basic wear mechanisms operating during rock cutting. The work on the performance of rock cutting trenchers, which is also a validation study for the concepts developed during this project, is still under way. The results are partly published here (Chapter 17).

1.4 ORGANISATION OF THIS WORK

The present work presents an overview of the knowledge gained during the execution of the project. It is divided into four parts.

Part A: *Problems of wear in rock dredging* gives the outline of the subject of this thesis. The wear problems encountered in rock cutting dredging using suction cutter dredgers are discussed. The basic principles of suction cutter dredgers are given in Chapter 3. Wear mechanisms and the principles used to study wear are discussed in Chapter 4, which introduces the tribological approach followed in the research. The types of wear that are commonly encountered in dredging are also treated in this chapter. Wear of the cutting tools (pick points) on the cutterheads and wear in the pipe lines and pumps. In order to appreciate the influence of the geology, especially the way the geology is described in site investigation reports, in Chapter 5 the case of excessive wear problems encountered at Port Hedland, Australia, is dealt with.

Part B: *Rock properties influencing cutting and wear* contains the available information on rock cutting and tool wear that can be used in practice. Chapter 8 addresses the influence of the fractured nature of the rock mass on the cutting process. Fracture (or *discontinuity)* density and geometry determines largely whether a rock cutting machine will excavate by *ripping* (loosening of rock blocks, whose shape and size are determined by the discontinuity geometry) or *cutting* (the pick points cut into the rock material). Chapter 9 treats the current rock cutting theories and explores which rock mechanical index tests relate to the cuttability of rock. The relation of rock cutting mechanisms with the wear mechanisms treated in Chapter 4 is discussed in this Chapter. The *mode of cutting*, either brittle or ductile, is important in this respect. This mode is related to the size of the crushed zone that forms near the tip of the cutting tool. The nature of the crushed rock material itself determines the wear processes that may operate. The following four chapters are dealing with the rock properties that relate to abrasive wear mechanisms. The concept of hardness of rocks and minerals is treated in Chapter 10. The hardness contrast between the abrasive (the rock surface or the crushed rock powder) and the tool (in dredging commonly hardened steel) during the cutting determines the intensity of wear. Tests that try to determine the abrasiveness of rock are treated in Chapter 11. The problem with wear tests is that the results normally only relate to the circumstances of the test itself. Correlation with practice is often fortuitous. The wear mode theory of Deketh (1995), summarized in Chapter 12, gives a tool to approach forecasting of wear, avoiding to rely on abrasion or cutting laboratory tests only. A combination of rock mechanics strength tests and petrographic examination of rocks is regarded as the best approach at present (Chapter 13).

Part C: *Application of theory to practice* examines the current methods that are used to assess excavation performance and tool wear. Since the information from real rock dredging projects is meagre, attention is given to the methods that are used in rock tunnelling, rock ripping and rock trenching. Chapter 16 is a study where the performance of roadheader tunnelling machines and a cutting suction dredger excavating in Hawkesbury sandstone is compared with site investigation data and laboratory cutting and abrasion tests. Chapter 17 the results of field observations of rock cutting trenchers are presented. It is shown that both rock mass properties (discontinuity density) and rock material properties (strength, mineral content and microstructure) influence the production and tool consumption. These can be understood using the knowledge gained from the experimental and theoretical study presented in Part B. In order to set out in what direction the performance predictions of rock cutter dredgers can be improved, the methods currently in use for tunnel boring machines and bulldozer rippers are given in Chapter 18. In this Chapter also the approach adopted for the rock cutting trenchers of our project, the application of fuzzy logic, is presented.

Part D: *Site investigation for rock dredging contracts* presents the methods to arrive at a geotechnical model of the subsurface to be used for a rock dredging project. The site investigation methods that relate to cutting performance and wear are emphasized. In Chapter 21 an outline is given of the requirements of a useful geotechnical model and the site investigation methods needed to arrive at such a model. In Chapter 22 the methods used to characterize a rock mass for a rock dredging project are discussed. It is shown that with help of relatively simple tests an impression on the expected cutting and abrasion behaviour can be obtained. The way the tests are to be interpreted is a matter of concern. Current practice is mainly interested in estimates of unconfined compressive strength. In this work it is shown that at least data on both compressive and tensile strength are needed (Appendix C). It is also shown that more sophisticated testing techniques (like UCS testing on servo-controlled stiff testing frames and triaxial tests) are needed to make intelligent assessment of the likelihood of ductile cutting behaviour (Appendix D). In Chapter 22 aspects related to discontinuities, needed for rock mass characterisation are treated. The rock mass to be excavated should be well described in three dimensions, based on a sound geological model. The geotechnical information on the properties of the rock materials and the discontinuities in the rock mass is currently processed using simple classification systems, or by empirical correlation formulations. New developments make use of fuzzy logic and expert systems. Chapter 23 concludes this work with a presentation of the wear assessments to be made within site investigations for rock dredging.

1.5 ACKNOWLEDGEMENTS

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CHAPTER 2

The problem

Several types of dredgers are able to dredge rock. Dredgers may be classified according to their basic method of extraction, transportation and deposition (Bray 1997, De Heer 1989, Herbich 1992). Present day dredging equipment may be divided into two categories, *mechanic* and *hydraulic*. The first embraces vessels which scoop up the soil (mechanical dredgers), the second dredge by suction (hydraulic dredgers). Mechanical excavation of rock under water occurs using backhoe, dipper (power shovel), bucket and grab dredgers, although these excavate only the weakest rocks or rock fragmented earlier by blasting or breaking. A combination of mechanical cutting and hydraulic suction is performed by cutter suction dredgers, some of which were designed to be able to dredge large quantities of rock. Figure 2.1 shows common dredger types and Table 2.1 sums up typical characteristics.

Increasingly stronger rocks can be dredged with heavy duty cutter suction dredgers (Chapter 3). Nowadays rocks with an unconfined compressive material strength of up to 30-50 MPa (moderately strong rocks[1]) may be dredged directly, without pretreatment. Stronger rocks may be only dredged when they are fragmented. Fragmentation of rock, i.e. the breaking up of rocks into blocks of manageable size, can be present naturally due to joints or other natural discontinuities in the rock. Fragmentation can be induced either by drilling and blasting, ripping, pneumatic hammering, or dropping of spud piles on the rock.

The choice of dredger depends on a number of considerations, related to the type of project, the volume of rock to be excavated and the nature of the rock. Waves and wind conditions play an important role. Cutter suction dredgers, for example, cannot be used during heavy wave conditions, making them less suitable for work in open waters. Backhoe and bucket dredgers can handle rock as well, but the backhoe requires relatively shallow depth. Other aspects relate to the dimensions of the work and the required tolerances (De Koning, 1968). When large volumes of rock have to be excavated, nearly always a cutter suction dredger (CSD) is considered for the work.

[1] For a generally used strength scale see Appendix A, Table A6, p.274.

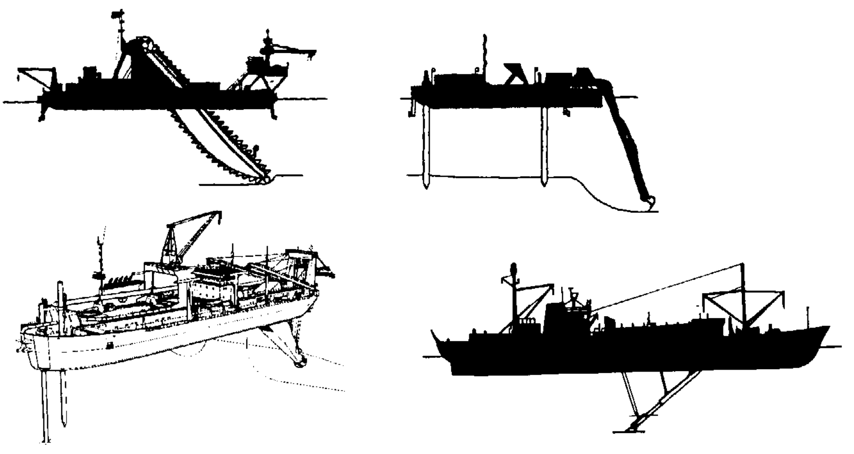

Figure 2.1 Common dredgers: bucket dredger; backhoe on pontoon; cutter suction dredger; trailer suction hopper (courtesy: IHC Holland).

2.1 WEAR IN ROCK DREDGING PROJECTS

The dredging industry has encountered specific problems related to the nature of the rock to be dredged. One of these is excessive wear, caused by (often unexpected) high abrasiveness of the dredged materials. In some cases severe financial losses have occurred. Abrasive wear is usually related to abrasive minerals present in the soil or rock, of which quartz is the most common. Abrasive wear is usually significant when the abrasive mineral is harder than the tool material. One scale of hardness that is commonly used for minerals is Mohs' Hardness (Chapter 10). Quartz has a Mohs Hardness of 7, which is higher than most steel types (Mohs Hardness 6). In one case, further elaborated upon in Chapter 5, substantial quantities of up to cm-size grains of quartz were present in a rock described as *calcarenite*, which strictly stands for a moderately weak limestone rock, consisting of sand-size carbonate grains (Mohs Hardness 3).[2] As a consequence of the presence of the quartz it was sometimes necessary to replace the cutting teeth (pick points) on the cutterhead after every 30 minutes of use (15 minutes replacement time; an idea of the costs involved can be derived from Petterson & Wijma 1997). Furthermore transport of the abrasive rock through the dredging system, complicated by the presence of clay in the dredged groundmass, forming quartz-armoured clay balls, eroded the pipes and the pump housing, which had to be replaced every two weeks.

This example illustrates a major cause of *unexpected* wear that occurs in practice

[2] The use of the PIANC classification of soils and rocks is advocated in the dredging world. This clasification follows the BS 5039:1981 standard (see Appendix A, Table A1, p.278). Calcarenite is defined as a rock of which at least 90% of the grains are carbonate (Appendix A, Table A2, p.264). No further information on the mineral composition can be obtained from this rock name.

Table 2.1 Dredgers capable of handling rock, either pre-treated or by direct cutting (after Bray, 1979).

Dredger type	Direct rock dredging	Dredging pre-treated rock	Remarks
Grab dredger	weak rocks, up to 20 MPa, using heavy bucket, but difficult	fragmentation 400 mm or less, rock has to be pre-treated by explosives or rock breakers	in very weak rock more efficient than backhoe
Bucket dredger	weak to moderately strong rock, but difficult	fragmentation 600 mm or less. Maximum size restricted by distance between ladder hoist wires	rock dredging only possible if buckets have cutting teeth adapted
Dipper dredger	weak rock, but difficult	only fragmented rock, fragmentation 800 mm or less.	dipper can pick up larger fragments than 800 mm, determined by its size and lifting power
Backhoe	weak rock to moderately strong, but difficult	fragmentation 300 mm or less	can pick up larger fragments
Trailing suction hopper dredger	weak rocks may be dredged	fragmentation 300 mm or less.	draghead may be equipped with ripper teeth
Cutter suction dredger	weak to moderately strong rock with increasing difficulty	fragmentation 300 mm or less	fragments need to pass cutter blades; consistent small fragments are favourable for optimum pipe line transport

as a consequence of *insufficient information* in the site investigation reports. Lack of recognition of the presence of such high quantities of quartz in a rock can have serious consequences as described in Chapter 5.

Rock excavation by dredging is relatively new and problems have occurred that can be ascribed to lack of understanding of rock engineering, for dredging is traditionally and mostly done in soils. In the worst case, site investigation reports for dredging works may lack the correct geological information, leading to the choice of a wrong type of dredger. Most commonly rock properties are lacking which are needed to estimate the amount of rock cutting teeth necessary for a project.

2.2 OUTLINE OF THE PROBLEM OF WEAR IN ROCK DREDGING

The immediate reason for this study was the lack of control on the prediction of the number of pick points needed for dredging projects of cutter suction dredgers in rock. Especially at the excavation front, where the cutterhead mounted with pick points is cutting into the rock, unexpected deviations of amounts of wear are recorded in practice (too high as well as too low estimates). Although wear occurs in other parts of the dredging process as well (in pumps, in pipelines, or the tracks of bulldozers on the reclamation site), the interest is mainly on the wear experienced at the cutting front. The focus of research into this subject had previously been very much concentrated on the mechanical engineering aspects and material properties of the pick points or cutting chisels. It was clear that the rock material itself played an important role as well. Often, when unexpected high wear rates were encountered, the reason was related to special geological circumstances or to particular features of the rock that had been overlooked.

After introducing basic features of cutter suction dredgers in Chapter 3, the wear processes that have been encountered are described in Chapter 4. Then a case history is given, in Chapter 5, of a dredging project where excessive wear took place. A definition of important subjects for further elaboration is given in Chapter 6.

CHAPTER 3

Cutter suction dredging

3.1 CHARACTERISTICS OF CUTTER SUCTION DREDGERS

The cutter suction dredger (CSD) is the most common type of dredger. The cutter suction dredger has two main components; the cutterhead and the dredge pump. The cutterhead is situated at the entrance of the suction pipe (typical diameter 300 - 900 mm) and is mounted on a supporting arm, the "ladder" (Figure 3.1). The cutterhead is commonly of a basket type, with spiral blades which are integral with the front hub and back wearing ring (Figure 3.2). The function of the cutterhead is to agitate softer materials or to cut harder materials, and ensure that these can be removed hydraulically through the suction pipe, placed axially in the ladder. For cutting rock or hard soils teeth are connected to the blades. The design of the cutterhead highly influences the efficiency of the cutting and suction operation. Cutters are usually operated at rotation velocities between 10 and 40 rpm. The rotary motor is commonly located on the ladder above the water. Location behind the cutter in a submersible drive unit is also possible. Two types of motors are used; electrical or hydraulic. Electric motors with engine characteristics can deal with sudden peak loads due to the increasing torque with decreasing cutter rotation speed, as shown in Figure 3.6a (p.22). They are therefore more suitable for use in drive units of rock cutters due to the variable load characteristic of the cutting process. Hydraulic motors can only deal with variable loading conditions by decreasing the average load of the engine. When overload occurs, safety valves are opened and the speed drops to zero. Such motors stop when the maximum torsional moment of the motor is exceeded (Figure 3.6b). Typical total installed power of rock dredgers is above 6000 kW, the largest have more than 20.000 kW[3], of which the dredging pumps have 3500 to 7500 kW and the cutter power is around 2000 to 4500 kW. Data of some cutter suction dredgers used for cutting rock are given in Table 3.1.

A rock cutter suction dredger excavates with a cutterhead of typically 3 m diameter armed with a number (about 50) of pick points. The reactive force is obtained from the spuds (Figure 3.1), the mass of the ladder and the side wires. The

[3] The international directory of dredgers, *Dredging + Port Construction*, August 1997.

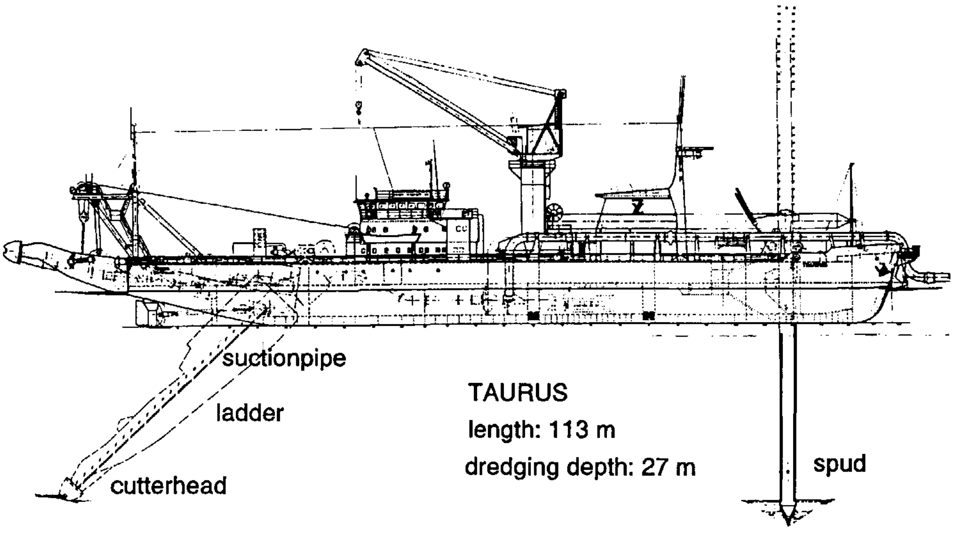

Figure 3.1 The cutter suction dredger Taurus (courtesy Boskalis).

nature of the excavation process itself is irregular and dynamic, because of the low stiffness of the excavation system of a cutter suction dredger. The cutting of the rock occurs in an impacting fashion and torsion forces, built up in the system, are suddenly released. The rock will therefore be cut with varying cutting velocities[4] (3-10 m/s; Prof. De Koning, pers. comm. 1990) and cutting forces (in the order of 40-80 kN per pick point). Peak forces may be six to seven times the nominal values.

The dredge pump is situated on the ladder or in the body of the dredger. Heavier dredgers may have pumps on the ladder and inside the vessel. The flow generated by the pump(s) picks up the cutted material and transports it via the pipes to the shore. The diameter of the suction pipe is normally somewhat larger than that of the discharge pipe. In the UK and the USA, cutter suction dredgers are rated corresponding to the diameter of their discharge pipes (Bray et al. 1997)[5]. Diameters range from 150 - 1100 mm.

The suction pipeline starts on the ladder, at the back-ring of the cutter. The ladder can be vertically hoisted or lowered. A flexible joint, consisting commonly of a reinforced rubber suction hose, connects the pipe on the ladder with the in-board pipeline and the pump mounted in the hull of the dredger. The discharge pipe runs from the pump, commonly over the deck, to the stern of the vessel. Here the pipeline is connected to a flexible floating pipeline, which transports the dredged material to the discharge area.

[4] Tangential velocity of pick point or tooth, see Chapter 3.3.

[5] In The Netherlands dredgers are classified according to total installed power. The diameter of the discharge pipes depends on this.

Table 3.1 Rock cutting dredgers, data off-shipyard.

	Oranje	Taurus	Leonardo da Vinci	Mashhour
year of construction	1978		1986	1996
shipyard	De Merwede	De Merwede	IHC	IHC
length overall (m)	132.3	112.6	129.2	140.3
length hull (m)	88.78	90.26	107	113.4
width (m)	19.50	19.00	22.4	22.4
moulded depth (m)	5.70 / 8.20		8.15	7.2
maximum draught (m)	5.67	4.60	5.13	4.95
suction pipe diameter (mm)	850		900	1000
discharge pipe diameter (mm)	890		900	850
maximum dredging depth (m)	31.50	27.00	30.0	35.0
minimum dredging depth (m)	5.00		5.00	8.0
anchorage system	spud carriage, christmas tree	idem	idem	idem
sailing speed (knots)	11.0		10.9	
total installed power (kW)	14.733	15.625	20.230	22.795
cutter output (kW)	3468	3680	4410	2400
suction pump output (kW)	2356	2210	2735	2400
discharge pump output (kW)	2 x 2868	2 x 3385	2 x 4485	2 x 5400
propulsion engines (kW)	2 x 2868	2 x 1840	2 x 2750	4 x 2650

To have a rough idea about the capability of these dredgers: according to Herbich (1992) a well designed 750 mm dredge with 3500 - 6000 kW on the pump and 1500 kW on the cutter, will pump 1500 - 5000 m^3/hour of soil material and 150 - 1500 m^3/hour of weak to moderately strong rock through a pipe line length of 4500 m.

3.2 CUTTER SUCTION DREDGING

3.2.1 *Operation of a cutter suction dredger (CSD)*

During operation the cutter suction dredger pivots around a spud pile (Figure 3.3A). The spud has the function of anchoring point, which provides a horizontal reaction force needed for the cutting. Two fore-side winches provide the tangential forces via sled cables (*side wires*) to the fore-side anchors. The fore-side winches are used for movement and control of the cutterhead. In hard material a considerable force is taken in the side wires. The largest rock cutter suction dredgers have a side pull of more than 1000 kN. Two stern spuds allow the dredger to advance in steps towards the dredging face. Modern dredgers have a movable spud carriage, which allows one spud to be horizontally moved along the axis of the dredging vessel, and an auxiliary spud pile, the *fixed* spud. Compared with dredgers with fixed spuds, dredgers with a spud carriage can make steps of all sizes, to be able to built up the required reaction forces while preventing the spuds moving into old spud holes. In hard rock material dredgers with fixed spuds can hardly be used, because of the possibility of

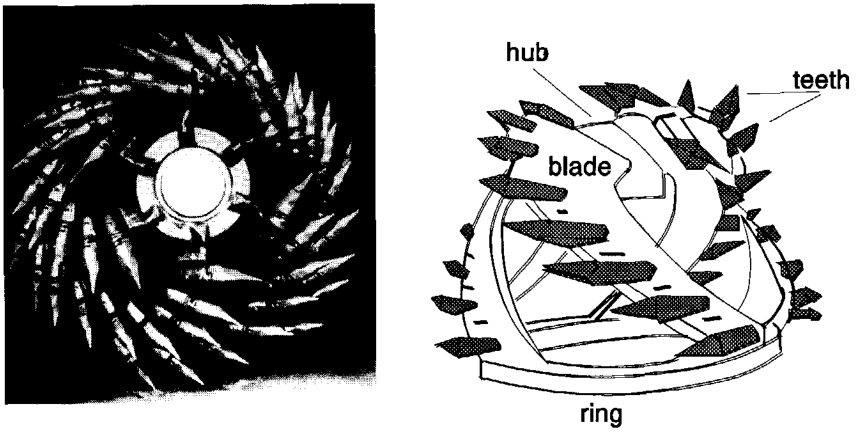

Figure 3.2 Cutterhead mounted with teeth (pick points).

pushing the spud back into the former position. The arrangement of spud carriage provides regular cutter tracks, without too much overlapping excavation during subsequent swings (Figure 3.3A). The swing width that can be made relates to the length of the vessel and the depth of the ladder. For larger vessels this may well exceed 100 m. In rock, the depth of cut (face, height of breach) is normally 0.5 - 1 times the diameter of the cutterhead (Figure 3.3B). In very hard rock the depth of cut may become less than 0.5 times the diameter. The maximum excavation depth is about 30 m for the largest cutter suction dredgers. Most CSD's have a maximum working depth of 20-25 m. To withstand the high dynamic vertical reaction forces, the mass of the ladder has to be sufficient. For large CSD's up to 1000 Mg.

When hard material is being dredged, the cutter is only used effectively during the swing in one direction, the *working-* or *undercutting swing*. When the cutterhead is swinging in the other direction (the *back-* or *overcutting swing*) the cutterhead has a tendency to roll across the material without cutting it. Whether rolling will occur or not is determined by the strength of the rock and the penetrating (thrust) force that the CSD is able to develop. The mass of the ladder is important in this respect, since it provides a vertical downward force of penetration. During rock cutting one therefore distinguishes the *working swing*, when the teeth are cutting upwards (Figure 3.4A), and the *back swing*, when the teeth are directed downwards (Figure 3.4B). During the back swing the cutterhead may bounce off the surface, if the rock is strong relative to the mass of the ladder, and not enough downward directed thrust force is available. In soft material this is not a problem (De Koning 1968).

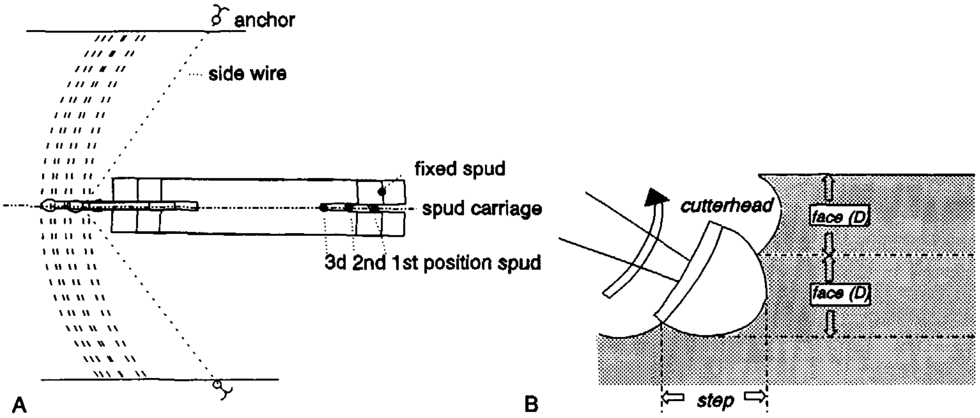

Figure 3.3 A. The cutter suction dredger rotates around a movable spud pile. A fixed spud is used when the vessel steps forward. B. Vertical excavation occurs in steps, the height is called face.

The *swing speed (haulage velocity, V_h)* varies normally from 5 m/min in strong rocks to a maximum of 20 m/min in weak rocks. Haulage velocity is related to the excavation production[6], Q, by:

$$Q = V_h \ x \ step \ length \ x \ face \ x \ time \ (m^3) \tag{3.1}$$

In order to obtain a good production, haulage velocity, step length and face (depth of cut) (Figure 3.3) are adapted to the rock conditions. For example, with a step of 1 m, a face of 1.5 m and a haulage velocity of 15 m/min, a production rate of 22.5 m^3/min = 1350 m^3/hour can be obtained at the cutterhead.

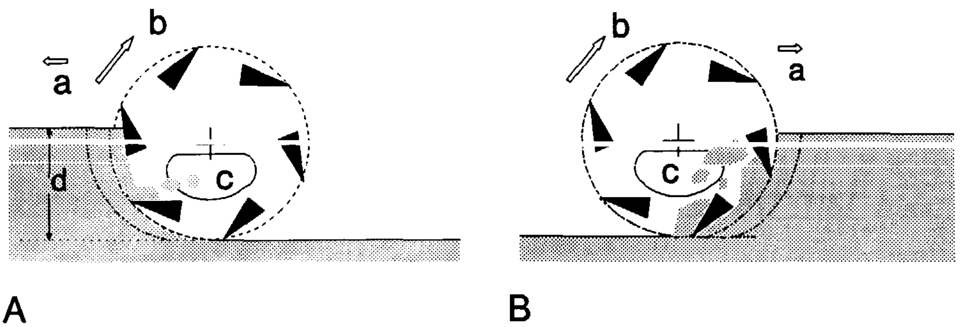

Figure 3.4 A. Undercutting operation (working swing). B. Overcutting (backswing, only in soft soils). a: direction of swing; b: direction of rotation; c: suction mouth; d: depth of cut (face).

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[6] The excavation production differs from the real production. Not all the material excavated will be taken up (there is spill), due to various reasons including pipe diameter and length and pumping capacity.

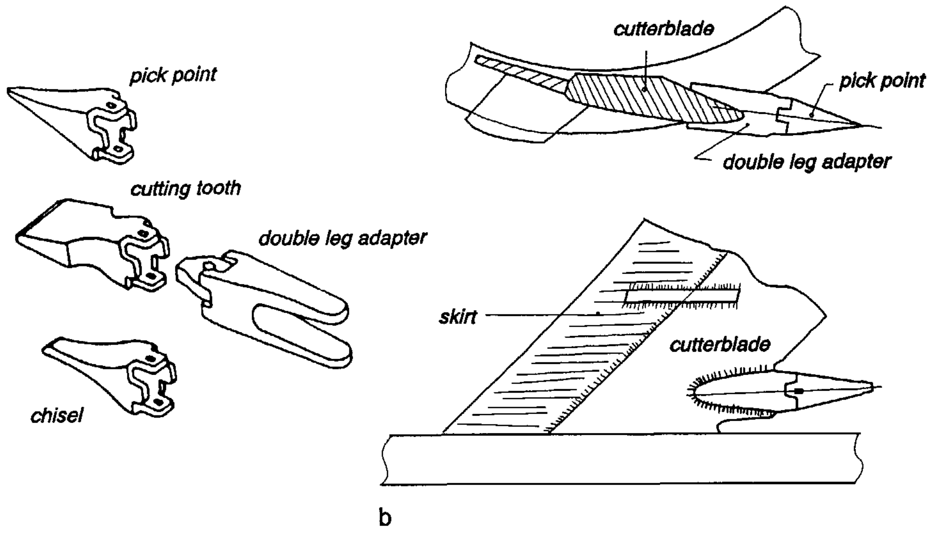

Figure 3.5 Examples of cutter teeth. a. pick point, cutting tooth, chisel and adapter (Esco). b. connection of a double leg adapter (courtesy W.J. Vlasblom).

The positioning of the cutter teeth on the cutterhead determines the performance greatly. Cutter teeth are fixed in adapters which are welded on the blades of the cutterhead (Figure 3.5). The angle at which the teeth are placed on the blades varies from the cutter ring to the hub, to assure an optimal cutting angle (see Figure 3.10 & 11b) at each position. The cutting angles (often about 65-75°) have been mainly based on practical experience. The spacing at which the teeth are placed on the blades is determined, apart from assuring chisel interaction (Chapter 9.3.2), by the maximum force that each teeth can handle (more than 1000 kN) and to ensure an even force distribution over the cutterhead. The teeth also have a function in protecting the adapters and blades from wear. Therefore they should not be widely spaced. Wear of the cutting teeth is noted by a decrease in production. When the dredgemaster notices the decrease in production, the ladder is hoisted and the cutterhead is inspected. Worn teeth are replaced one by one. In case also the adapters are worn or broken off, the complete cutterhead has to be replaced (replacement time about 40 minutes).

The cut material, being agitated by the cutting process, is taken up by the suction mouth, which normally is placed below the axis of the ladder. It often has an ellipsoidal shape, to optimize the inflowing stream of soil, rocks and water (the *slurry*).[7] The suction mouth and the cutter blades are often protected against wear by a layer of hard facing. Part of the excavated material, however, is not taken up by the dredger, the *spill*. During the back swing generally more spilling occurs than during the working swing.

Sometimes larger blocks of rock, being able to just enter the suction pipe, can cause blocking of the dredging pump. Such blockages may be prevented by keeping

[7] Plastic clays may cause clogging of the cutter. When weathered rock is cut this problem may be encountered. Sometimes waterjets are placed near the cutterhead to assist the loosening of cohesive materials.

the maximum space of the cutter blades smaller than the allowable width of the impeller of the pump.

Some rock types or soils cause a lot of wear on the pipes and pumps. Wear resistance of the pumphouse has been improved in recent designs by using double walls, whereby the inner casing is replaceable. This inner casing is constructed of highly wear resistant materials, like hardened steel, rubber or synthetics. The space between inside and outside housing is filled with water to compensate the inner pump pressure. After passing the pump the slurry is transported via pipes to the disposal area. Floating pipelines are regularly rotated to assure evenly distributed wear, before they have to be replaced, on-shore pipes are rotated if possible (see Chapter 4).[8]

The above described dredging method is versatile and very commonly used. Continuous and direct transport of the cut materials to the discharge or reclamation area is ensured.

3.2.2 *Soil and rock properties during transport and disposal*

The properties of the soils or rocks cut are again of major importance during the whole process just described. The volume occupied by the rock or soil *in situ* differs from the volume in the soil-water mixture during transport and later in the disposal area. The increase in volume in the disposal area is quantified by a *bulking factor*, which may be expressed by:

$$B = \frac{\gamma_{d,i}}{\gamma_{d,d}} \ (-) \tag{3.2}$$

where: B = bulking factor; $\gamma_{d,d}$ = dry volumetric weight in disposal area (kN/m^3); $\gamma_{d,i}$ = dry volumetric weight in situ (kN/m^3).

Bulking factors vary greatly for different types of rocks and soil, and for different methods of dredging. Factors of 1.3 - 2.0 are given for rock, where weak rocks have the lowest bulking factors. For soils factors varying from 1 to 1.4 are mentioned by Bray et al. (1997). During cutter suction dredging, however, the soil or rock material is mixed with water to form a low density slurry. Changes in specific volume during the excavation process are therefore very difficult to predict. They determine, however, some important features affecting the economy of the dredging operation:

- the compaction or consolidation of the material at the disposal site (and thus the bearing capacity of the soil).
- the payment is often related to the volume dredged and the volume placed on the disposal site.

A problem with clay-bearing soils is the possible formation of clay balls in the pipelines. These balls form during hydraulic transport from cohesive patches of clay. Clay balls highly damage the pipes if armoured with quartz grains of sand or gravel size.

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[8] The Oresund project is an example of a dredging project where high wear occurred due to the presence of flint (cryptocrystalline silica rock) in the limestones dredged. The monitoring system used for the wear of the pipes is described by De Kok et al. (1997).

Larger rock blocks tend to slide along the bottom of the pipelines, causing scouring of the pipe steel.

From this short review of the dredging process follows that geotechnical information of the material to be dredged is needed, since it influences the dredging process in all its stages. Information is needed on:

- geology (rock and soil types and their distribution)
- *in situ* volume and geotechnical properties of rock and soil types, including the presence of natural fractures (discontinuities) in rock.
- bulking factor
- properties during transport (density and viscosity slurry, mixing process, abrasiveness)
- properties during and after deposition (homogeneity, consolidation behaviour)

3.2.3 *Environmental impact during cutter suction dredging*

Environmental impact is of concern as well. The rotation of the cutterhead can produce a sediment cloud, not only when dredging in fine sands, silts or clays, but also when cutting rock types like weak limestones (calcarenites and calcilutites). According to Palermo & Hayes (1992) cutterhead dredges and hopper dredges without overflow generate much less suspended sediment than grab dredges or hopper dredges with overflow. They stress that reduction of suspension of sediment can be obtained by proper selection of cutter rotation speed, ladder swing speed, depth of cut, and hydraulic suction provided at the cutter.

3.3 BASIC MECHANICS OF ROCK CUTTING DREDGING BY CSD[9]

3.3.1 *Cutter power*

The mechanical description of the operation of a cutter suction dredge is extremely complicated. Miedema (1987) has written a simulation programme for the behaviour of a CSD, dredging sands in open waters. Miedema's model calculates the cutting forces developing on the blades of a cutterhead while cutting in saturated sands. For rock, such a model is being developed by Delft Hydraulics, but the results of this work up to now are not available. Many factors are to be considered when developing such models. The cutting forces that will develop depend on the rock failure mechanism operating. This will depend on the dynamic stress development in the rock, which is a function of the shape, size and mechanical properties of the cutting tool and the impact energy applied (Chapter 9). Deliac (1993) has developed general rock cutting models for rock cutting heads of tunnel boring machines and road headers. To get some idea of the potential power of a CSD, some basic statements can be made.

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[9] Information used for this Chapter was obtained from PBNA course notes (see footnote 10) and from course notes of the Training Institute for Dredging (TID: Cutters & Some aspects of cutter suction dredging, not dated; courtesy IHC).

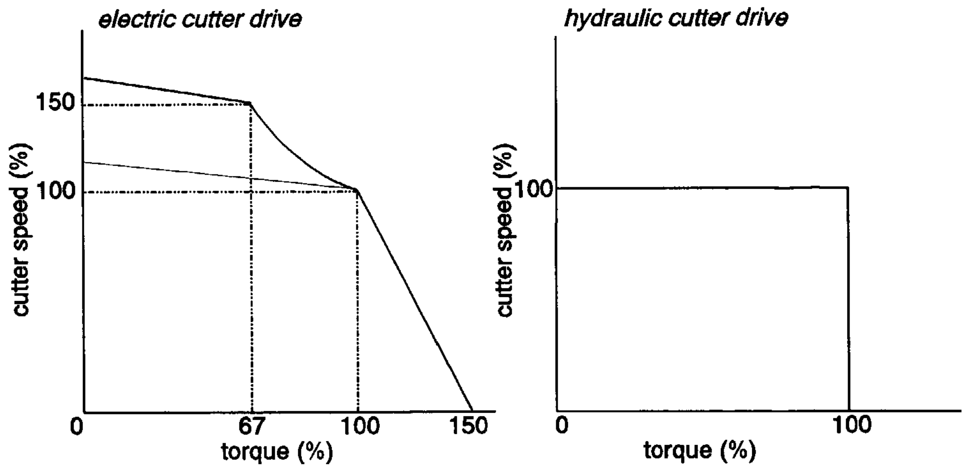

Figure 3.6 Characteristic relation of cutter speed with torque of electric and hydraulic cutter motors (courtesy W.J. Vlasblom).

According to Herbich (1992) the dimensions of the cutterhead vary, but have a relation with the dimension of the suction pipe, whereby the cutter diameter D_c is about 3 to 4 times the diameter of the suction pipe D_s. Normally the length of the cutter is 3/4 of the diameter: $S_c = 0.75D_c$. The velocity of the cutterhead varies from 10 to 40 rpm, with at 30 rpm often the optimum (nominal) torque. The required power may be approximately computed from:

$$P_c \approx \frac{F_c \pi D_c \omega}{60\eta} \ (Nm/s) \tag{3.3}$$

where: P_c is power capacity in W (Nm/s); F_c = cutting force at circumference of cutterhead (N); ω = speed in revolutions per minute (rpm); η = efficiency (-).

From this equation can be seen that, for a given power capacity, a decrease in diameter of the cutter allows a higher cutting force (torque = $F_c . \frac{1}{2}D_c$). The "efficiency" depends on a number of factors, such as properties of soil/rock, friction loss and wear. Electro-motors, which are generally used on heavy duty CSD's, deliver a constant speed (rotation velocity) below the nominal torsional moment (100%) of the motor. Above this value the rotation speed drops linearly to zero at 150% maximum torque. Hydraulic motors will stall in such a situation (Figure 3.6), which is one reason why electric drives are common on rock cutting suction dredgers.

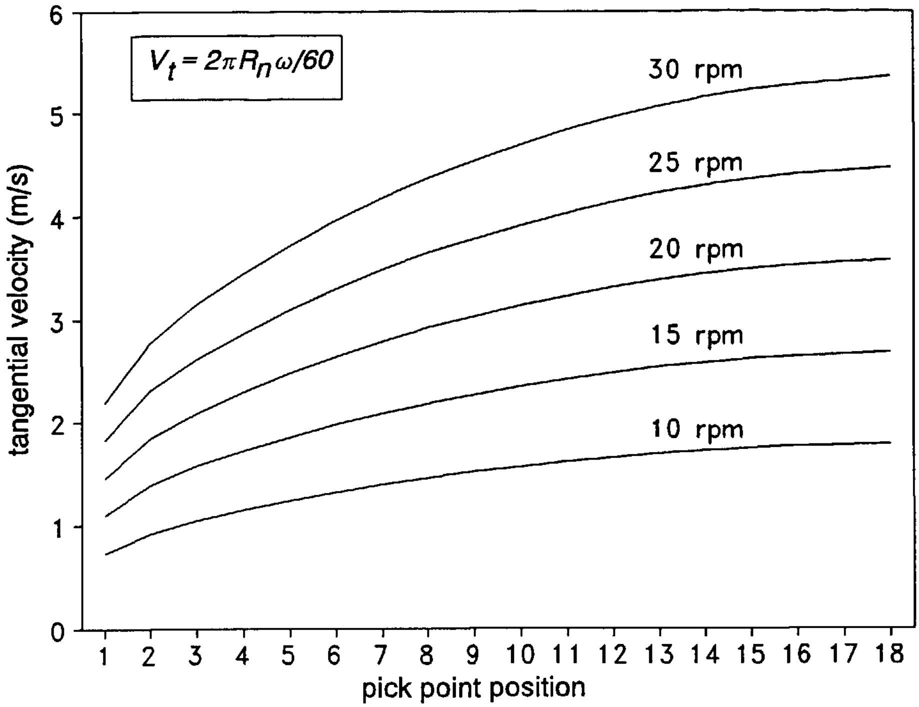

Figure 3.7 Velocity of pick points on cutterhead of 3.2 m diameter. Pick point position 1 is near the hub, position 18 near the back wearing ring of the cutterhead.

3.3.2 *Cutting by cutterhead*

The cutting teeth or pick points on the cutterhead take up different radial positions on the cutting blades and therefore each have different cutting velocities. Figure 3.7 shows an example of the variation in velocity with position of 18 pick points on a particular cutterhead.

The position of the teeth on the cutterhead is important. The spacing between the cuts made in the rock determines the economy of the excavation process. The teeth are placed in such a position that the grooves or zones of rock chips made by each tooth just overlap. In this way the excavation takes place in the most economical way (see chisel interaction, Chapter 9.3.2). The length of rock material cut per revolution of the cutterhead depends on the position of the pick point on the cutterhead and its orientation with respect to the rock cut and the height of the face (depending on position of the ladder, i.e. dredging depth; Figure 3.3B & 3.8).

The penetration d per pick point or tooth that can theoretically be made in the rock is an important parameter, since this influences the wear mechanisms operating (Chapter 12). The thickness of the slice of rock excavated depends on the cutterhead rotation rate (ω), the haulage velocity (swing speed) V_h and on the number of blades.

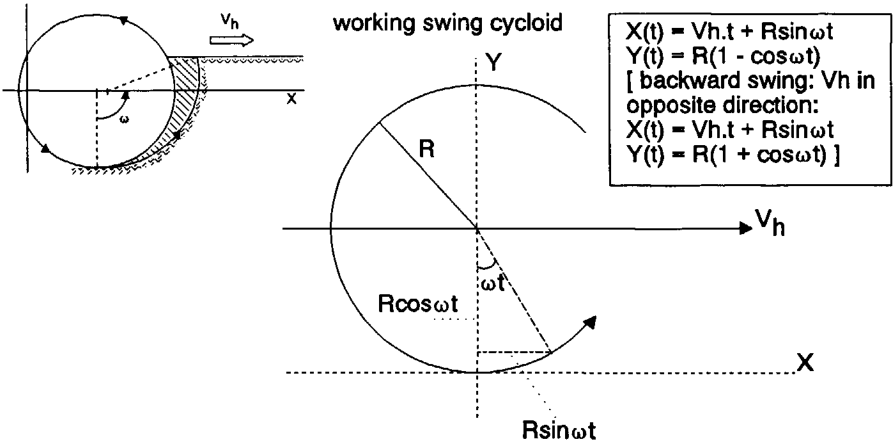

Figure 3.8 The path of each tooth follows a cycloidal track, the shape and length of which depend on radius of tooth and rotation and haulage velocity (see text).

This is illustrated by Figure 3.9.[10] Only the *working swing* (Figure 3.4) is regarded here. Figure 3.9 shows the evolution of a cutterhead with six cutting blades, mounted with teeth each on the same position R. Each rotation of the cutterhead the teeth are displaced by a distance a in horizontal direction. Each tooth therefore starts cutting a slice with a penetration depth d of zero, increasing to a maximum thickness a. The maximum thickness can be estimated from measurements of V_h and cutter rotation rate ω:

$$a \approx \frac{V_h}{\omega n_R} \ (m) \tag{3.4}$$

where n_R is the number of teeth with identical position R on the cutterhead. For example, if V_h is 15 m/min, ω 30 rpm, and there are 6 teeth on an identical position, than the maximum penetration depth for each tooth, per revolution of the cutterhead, is 0.08 m. If the teeth have a *staggered* position on the six blades, then there are three teeth with identical position ($n_R = 3$) and the maximum penetration is 0.17 m. Higher penetration depth is favourable, amongst others because of the higher cutting production per tooth, but factors such as chisel interaction and stability of the cutterhead can be negatively influenced by lowering n_R.

The soil or rock resists the cutting by a force F_r. The cutting tooth has to *penetrate* the rock with a penetration force F_n, normal to the cutting direction, and to cut the rock with a cutting force F_c, parallel to the cutting direction. Since the penetration force is, for a large part of the cutting action, about parallel to the swing direction, the haulage on the side anchors of the dredger contributes to this force. The cutting force is largely provided by the cutter motor of the cutterhead. However, mass and inertia of wires, ladder and ship contribute as well.

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[10] This figure comes from old -undated- course notes, probably written in the early 1970's: PBNA -Vereniging "Centrale Baggerbedrijf". Voortgezette opleiding uitvoering baggerwerken, deel 5. Snijkopzuiger, winzuiger, bakkenzuiger. These notes treat the basic principles of suction cutting dredging.

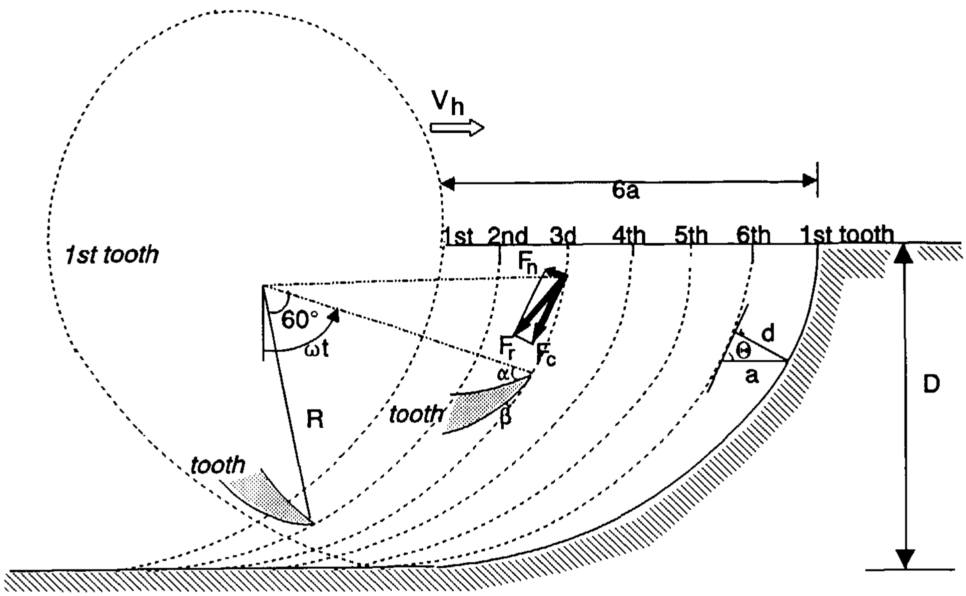

Figure 3.9 Cycloidal tracks of six teeth mounted on a cutterhead with six blades, each with identical distance (R) to the cutter axis. D = face (Figure 3.3). For explanation of other symbols see text.

The path of a cutting tooth or pick point is called *cycloid*. Depending on the position on the cutterhead each tooth describes its own cycloidal track. Both swing speed V_h (in the order of meters per minute) and rotation rate (10 - 40 rpm) determine the shape of the cycloidal tracks and thus the shape and thickness of the cut. With variation of these parameters, the actual rake angle α of the cutting teeth, according to their position on the cutterhead varies as well. Since rock properties influence swing speed and rpm, each rock type would need adaption of tooth position for optimal cutting (Giersch 1989). How the velocities are influencing the actual rake angle, is shown in Figure 3.10. The cutterhead as a whole delivers an average torque during cutting (Figure 3.6). The force experienced by each individual cutting tooth is varying. When it is cutting in the rock it varies with the cutting process. The force increases to overcome strength, decreases with chip forming in brittle rock and is zero when it is not in the face. The resistance offered by the rock determines the forces building up (Chapter 9.2). The rake angle and the clearance angle (Figure 3.10) also determine the resistance to cutting (TID)[11]: If the angle of clearance is less than 12°, both the resistance to cutting and the power required for cutting will increase strongly. The upper part of the cutting wedge wears strongly (Figure 3.11), resulting partly in the absence of any angle of clearance, which implies increased friction and penetration force. A large rake angle decreases resistance to cutting (Chapter 9.2.1). Both requirements lead to a thin knife tool, which is not robust enough for rock. Common rake angles are between 15° and 50°.

[11] See footnote 9.

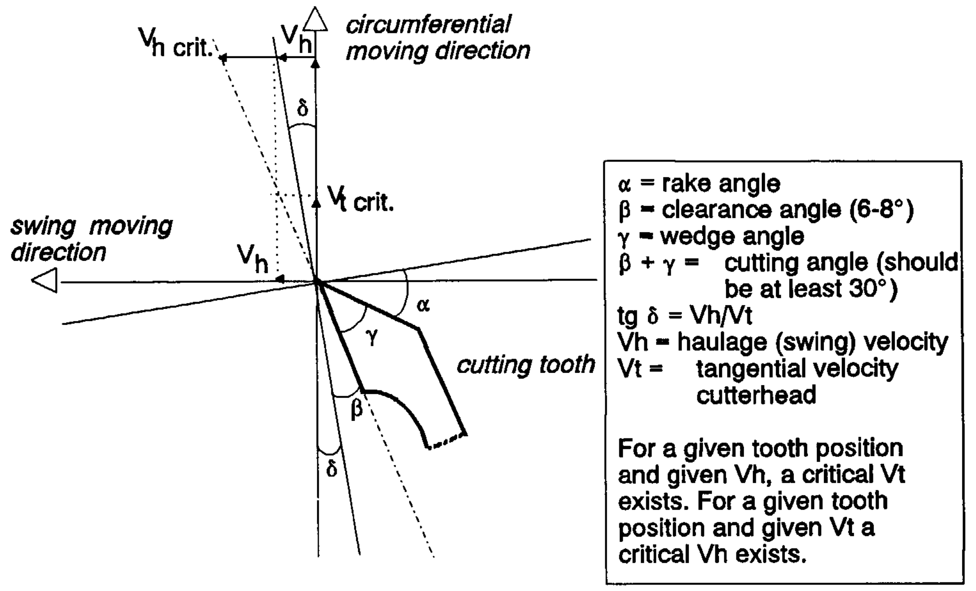

Figure 3.10 The cutting angle depends on cutterhead and haulage velocities. To maintain a correct angle, when V_h is increased, V_t should increase as well (Giersch 1989).

In dredging practice the dredge-masters' task is to optimize the excavation process of his particular job. He tries to make the process as efficient as possible. It is known that (PBNA)[12]:

1. The *rotation velocity of the cutterhead* and the *swing speed* (or haulage rate of the side wires) determine thickness of the rock slices cut. The amount of soil or rock cut is mainly determined by the haulage rate (Equation 3.1). By adjusting mainly the haulage velocity an optimum can be reached. This optimum is, besides the dredger power characteristics, dependent on the mechanical properties of the soil or rocks that are excavated. When the soil or rock properties change or vary, the dredgemaster adapts mainly haulage velocity to the new situation. The tangential velocity of a tooth on the cutterhead is described by: $V_t = \omega R$. The ratio between haulage velocity and tangential velocity, V_h/V_t, will be about 0.02-0.08 for weak rock. For normal sands and clays the haulage velocity can be higher, the ratio will be about 0.08-0.15 (TID).

2. The *design of the cutterhead and the teeth* influence the cutting process as well. The design of the teeth (their size and shape) and their position on the cutterhead are such that the cutting angle is as favourable as possible. This determines the shape of the cutting surface, which in its turn determines the amount of soil or rock cut (Figure 3.8 & 3.9). Due to the basket shape of the cutterhead, not all teeth are doing the same amount of work. The front teeth, although they have the shortest cutting path (Figure 3.8), are nearly always in contact with the rock. The teeth placed on the back sometimes hardly touch the rock. The basket shape is not really favourable to give optimal positions of cutting teeth.

A second important aspect of cutting tool design is that *friction* (and thus wear)

[12] See footnote 10.

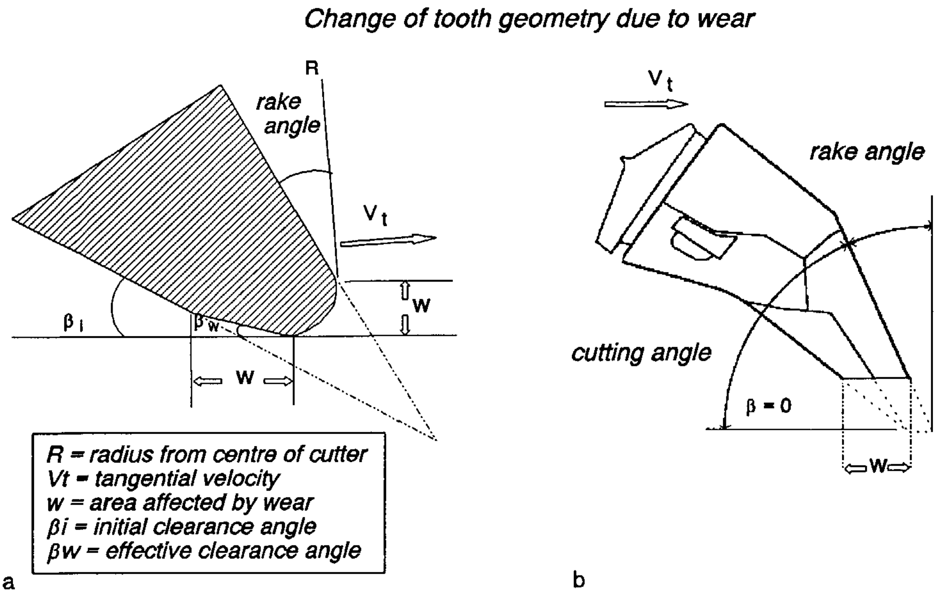

Figure 3.11 Due to wear the geometry of a tooth changes, resulting in a decrease of the clearance angle (a after TID; b modern tooth, VOSTA).

between soil-rock and the tool is to be prevented or minimized. That is why the *clearance angle* β is present. Wear of the tool causes a decrease in the clearance angle, which may even become negative (Figure 3.11a). This results in additional friction, by which the cutting forces will increase. The cutting tooth of Figure 3.11b ensures long wear length in the direction of cutting. The tool has no clearance angle in the cutting direction, but gives clearance along its sides (it is broad at front, becoming slender to its back). This design has shown improvement of stand time compared to its predecessors (Petterson & Wijma 1997).[13]

The strength of the cutting tool increases with the thickness of the cutting front. This requirement is difficult to combine with the ideal rake and clearance angles (see above).

Concluding, the *shape of the cutting surface* and the *production* depend on:
- the rotation velocity of the cutter
- the swing speed (haulage velocity)
- design cutting tool (shape and positions cutting teeth)
- the cutting properties of soil or rock

[13] The improved stand time is explained by the longer wear length of the tool in the cutting direction. Although available steel volume is important to have a long stand time during cutting, it is the available length of steel in the cutting direction which enhances the wear resistance of cutting teeth (K.G. Wijma, VOSTA, pers. comm. 1997).

3.3.3 *Specific energy*

The combination of the factors above have a dominant influence on the efficiency of the cutting process and the wear rate of the cutting tools. This efficiency is at an optimum, when at the given circumstances the lowest amount of work is done to excavate the soil or rock. A commonly used quantity to describe efficiency is the *specific energy*, which can be estimated on board of the dredger. It is the amount of energy consumption (or work done) per cubic meter of excavated material.

$$SPE = \frac{\sum (E_1 + E_2 + \ldots E_n)}{V} \quad (MJ/m^3) \tag{3.5}$$

where SPE is the specific energy, $\sum E$ refers to the total amount of energy used and lost during the process and V refers to the volume of excavated soil or rock. The SPE concept is commonly used in rock cutting and rock fragmentation literature. It is difficult to encompass the total mass - energy balance of a cutting process. Specific energy numbers quoted are always dependent on the machine used and the particular circumstances at which the measurements have been made (Chapter 9.3).

To estimate SPE of a dredger the volume of excavated material is needed. This could be determined from bathometric surveys, by seabed level and ladder-depth measurements, nowadays using the satellite global positioning system. The power used for cutter motor and pumps can be monitored on the vessel.

Concluding, it appears that most of the above mentioned characteristics are influenced by the soil and rock properties. These will determine the real cutting forces developing, the real penetration depths occurring and the real excavation production, for a particular dredger operating under prevailing site conditions. This is the reason why in this research emphasis is being placed on rock and the rock properties influencing the dredging process.

Regarding the rock properties, it is useful at this point to mention the influence of *discontinuities* in the rock. Many rocks are transected by fractures, which are generally called discontinuities in rock mechanics. The cutting forces must be influenced by these discontinuities so that, in certain rock types, blocks are dislodged and/or split but perhaps not cut. This aspect will be discussed in Chapter 8.

CHAPTER 4

Concepts of wear processes from tribology

In a cutter suction dredger important wear occurs in the following parts:

- The cutter teeth, adapters and blades on the cutter head
- The pump house and pump impeller
- The transport pipes

Table 4.1 gives a scheme of the predominant type of wear processes and mechanisms that are operating. The table further gives an indication of soil and rock properties that influence wear and tests that are used to measure them. In this Chapter a description and classification of these wear processes and mechanisms will be given, showing that the wear of tools during rock cutting is a result of processes very different from those occurring in pumps or pipes.

Table 4.1 Wear problems in cutter suction dredgers

Dredger component	Wear Process	Wear Mechanism	Important soil/rock property	Test
cutter teeth	sliding, impacting	two- and three body abrasion, adhesion	mineralogy, texture, strength, particle size/shape	petrography, strength test, cutting test
pump	erosion	abrasion, surface fatigue	mineralogy, particle size/shape	petrography, slurry erosion wear test
pipes	erosion	two body abrasion, corrosion	mineralogy, particle size/shape	petrography, slurry erosion wear test

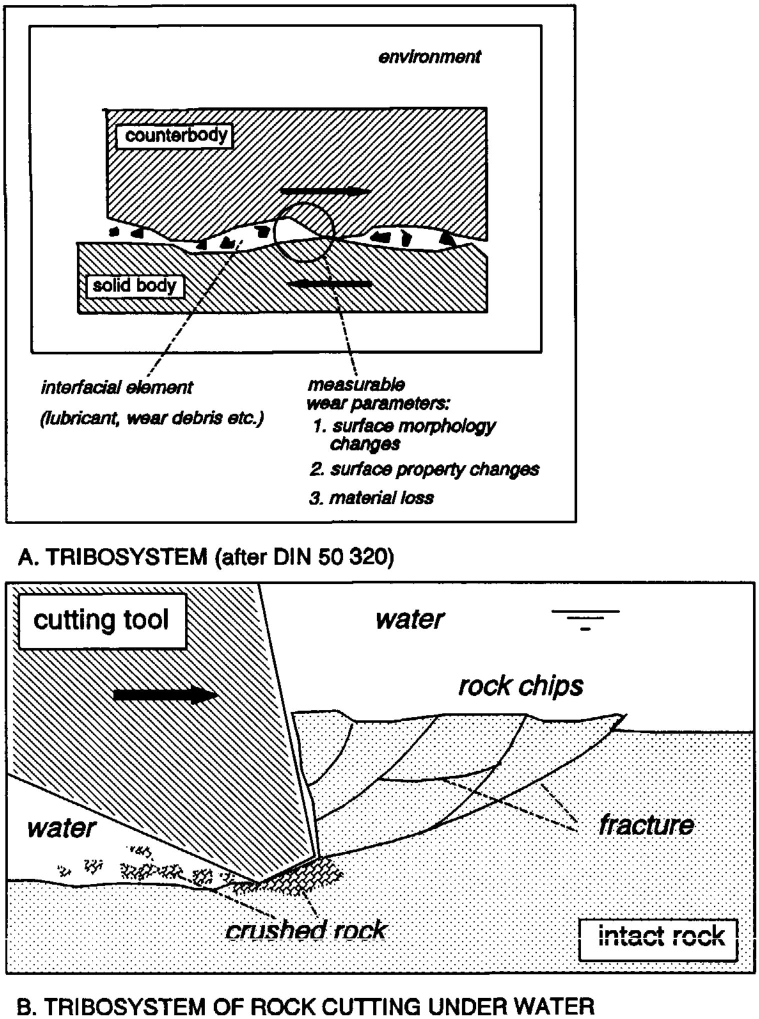

Figure 4.1 A. The elements of a tribological system. B. For rock cutting under water: solid body — cutting tool; counterbody — rock; interfacial element — rock gouge; environment — (sea)water.

4.1 BASIC CONCEPTS OF WEAR MECHANISMS

4.1.1 *Wear processes*

Wear is defined as the "progressive loss of material from the surface of a solid body due to mechanical action, i.e. the contact and relative motion against a solid, liquid or gaseous counterbody" (DIN 50320 1979, Zum Gahr 1987). Wear problems are difficult to handle. In the science of friction, lubrication and wear (tribology[14]) it is long known that wear is a system dependent process. To properly describe the *tribosystem*, to perform relevant tests and develop appropriate models to arrive at solutions to particular wear problems is typically the field of tribological engineering (Uetz 1986, Zum Gahr 1987 DIN 50320 1979, DIN 50321 1979).

Figure 4.1 gives an illustration of the factors which describe a tribological system. The wear problem at hand is described using the following terminology. The wearing part is defined as the *solid body*. The object causing the wear is described as the *counterbody*. In between these bodies an *interfacial medium* can be present, like wear debris or a lubricant. The whole system occurs in a certain *environment*, such as air or water. Figure 4.1B illustrates the system of a tool cutting rock under water. The steel cutting tool is the solid body under study, the rock forms the counterbody. In between tool and rock crushed rock *(gouge)* may be present. The cutting process occurs in (sea)water. To completely describe the process it is obvious that data are necessary, which describe the physical and chemical circumstances under which the process occurs.

To describe the *wear process*, which is defined as the type of action exercised on the *solid body* (tool which undergoes wear), use can be made of concepts laid down in DIN 50 320 (1979), a German standard. Included with this standard is a worksheet format which can be helpful in setting up the parameters necessary to describe a tribosystem. The elements of this worksheet are given in Figure 4.2.

A classification of wear processes is given in Figure 4.3. The classification is based on the type of motion that is occurring and the types of phases involved in the process (solid, liquid, gas). These actions are: *sliding, rolling, oscillation, impacting* and *erosion*. *Erosion* is the wear (removal of parts of a surface) caused by a flowing medium, like gas, water or air. The medium may be loaded with solid particles. The process is an open system and involves dynamic impact of the particles or fluids. Of course, a combination of actions, like sliding wear combined with impact motion is also possible (as for example in rock cutting with a cutter suction dredger).

The DIN 50 320 worksheet intends to assemble all information with regard to the collective energy input in the system (Figure 4.2). Therefore, data on loads imposed and their change in time, velocities occurring and their change in time, ambient temperature and change in temperature are important. The characteristics of the *solid body* and the *counterbody* (Figure 4.1 & 2) have to be described, as well as the properties of the *interfacial elements* and *environment*. These are characterized by their dimensions, chemical compositions, structure or aggregate configuration, physical properties like density, thermal expansion coefficient, thermal conductivity, volume, stress, surface properties, viscosity. Friction coefficients are determined and their change with time or displacement. Finally wear numbers are asked, often expressed in change of dimension or change of weight per time or displacement unit.

Applications of this concept are laid down in textbooks on tribology. For this work mainly the books by Uetz (1986) and Zum Gahr (1987) have been consulted.

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[14] Tribology is the science and technology of interacting surfaces in relative motion. The name is derived from the Greek word "tribos", meaning rubbing. Tribology embraces the scientific investigation of all types of friction, lubrication and wear and the technological application of this knowledge (Zum Gahr, 1987).

DATA TO DESCRIBE TRIBOSYSTEM (worksheet DIN 50 320)

I Technical function of the tribosystem

II Collective energy input (load, motion, temperature; including history in time)

Process: ☐ sliding, ☐ rolling, ☐ impacting, ☐ flowing, ☐ combinations:

Load on surface: F_N, record variation in time

Velocity: record the relative velocity along wearing surface

Temperature: record temperature history

III Structure of the tribosystem; s = F(A,P,R)

A: elements: (1) solid body, (2) counterbody, (3) interfacial elements, (4) environment

P: Describe properties of elements
for (1) and (2): sizes, chemical composition and fabric, physical properties, strength properties, surface hardness, surface roughness, physico-chemical surface properties
for (3) and (4): aggregate state, chemical composition and structure, physical properties, viscosity, other properties

R: Interaction of elements
Tribocontact surface, ratio of real contact area and total contact surface, state of friction, friction coefficient

IV Wear data on solid- and counter body. Measure total wear, wear rates: $\Delta l/\Delta t$; $\Delta l/\Delta s$ and record changes in time

Figure 4.2 Summary of the type of data that are suggested to be recorded for a proper description of a tribosystem (DIN 50 320, 1979).

The action leading to wear, the *wear process* (Figure 4.3), is commonly obvious. The *wear mechanism* taking place is deduced from the observations done. Zum Gahr (1987) explains that, although a large number of wear mechanisms are described in literature, these can all be brought back to four basic mechanisms. These four mechanisms are described as well in DIN 50 320: *adhesive wear*[15], *abrasion, surface fatigue* and *tribochemical reaction.*

Adhesive wear is the formation and breaking of interfacial adhesive bonds, for example cold welded junctions. Adhesive wear can occur when two surfaces slide

[15] In the context of wear, *adhesion* is used to mean adhesive wear. Literally adhesion is the attractive stress that may exist between two adjacent surfaces of different composition.

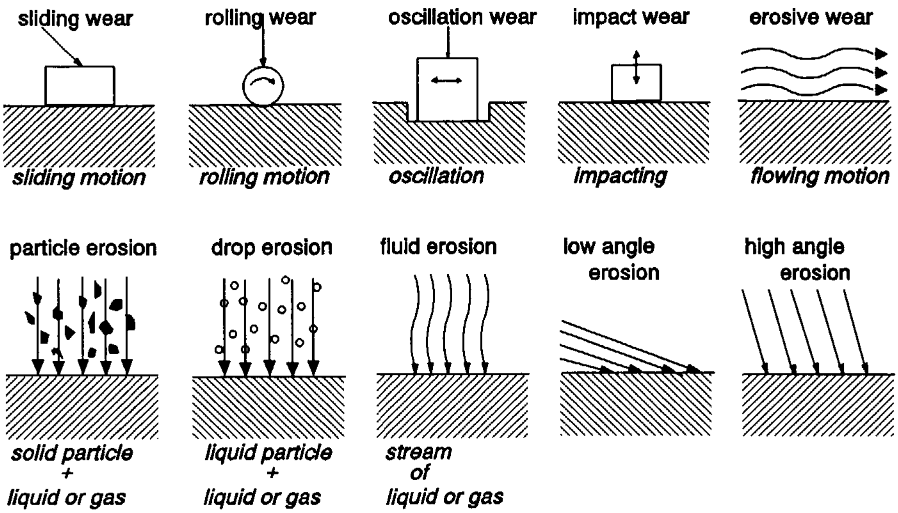

Figure 4.3 Classification of wear processes by the type of action on the surface of a solid body (wear mode). After Zum Gahr (1987).

along each other. High local pressure between contacting asperities results in plastic deformation, adhesion and the formation of local junctions. These junctions will shear off during continued sliding, thus leading to wear (Zum Gahr 1987).

Abrasion is the process causing removal or displacement of material ("wear") at a solid surface due to the presence of hard particles in between or embedded in one or both of two solid surfaces in relative sliding motion, or due to the presence of hard protuberances on one or both of the two moving surfaces (Zum Gahr 1987). If two surfaces are involved, or one surface against particles, the term *two body abrasive wear* is used. If particles are present between two surfaces, the abrasive mechanism is called *three body abrasive wear*. The contrast in hardness of the two solid surfaces is a fundamental factor in abrasive mechanisms. Although wear occurs when a softer abrasive is rubbing against a harder material (low level wear), the amount of wear increases dramatically when the abrasive is harder (Figure 4.4). *Hardness* is the resistance against indentation of a material (see Chapter 10).

Surface fatigue. Due to repeated dynamic loading of a surface, crack formation with resulting flaking may occur. Localized fatigue may occur on a microscopic scale due to repeated contact of asperities on the surfaces of solids in relative motion (Zum Gahr 1987).

Tribochemical reaction. During the rubbing contact of two surfaces a reaction with the environmental medium may occur, resulting in corrosive reactions (Zum Gahr 1987).

Figure 4.5 shows possible mechanisms that could operate in tribological wear processes considered relevant to cutter suction dredging. When the cutter teeth are cutting rock, impacting and sliding motions occur. From the science of tribology is known that certain mechanisms often happen under such circumstances (Figure 4.5). Impact loading often leads to *surface fatigue*. The sliding motion leads to a number of wear mechanisms, but *adhesion* and *tribochemical reaction* are the most common.

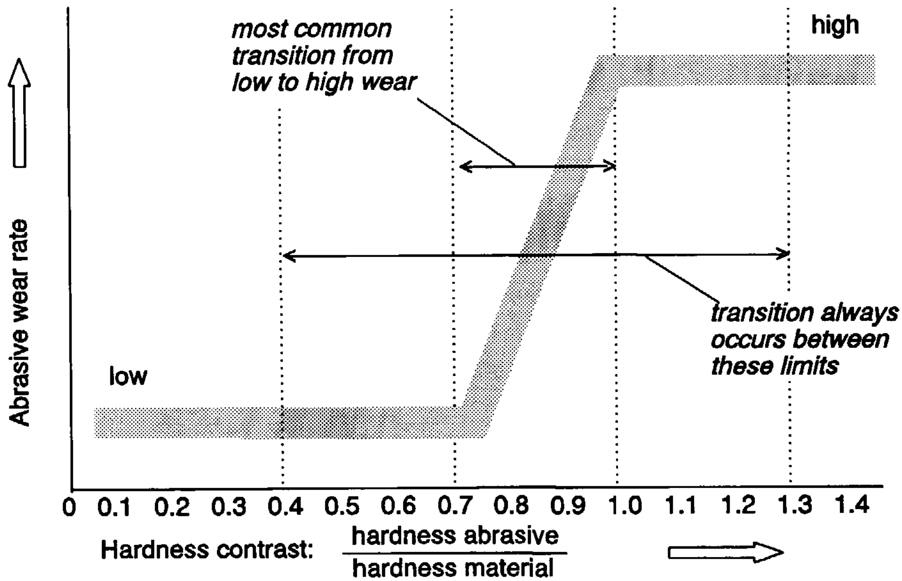

Figure 4.4 The amount of abrasive wear is a function of the hardness contrast between the abrasive minerals and the tool material.

When hard protuberances or particles are present, however, *two- or three body abrasion* are the most common wear mechanisms.

Deketh (1995) has performed cutting experiments in rock, to study the wear mechanisms operating during sliding motion. He found that *adhesion* and *abrasion* are the main mechanisms operating in rock cutting. In this case adhesion was used to describe the increase of wear caused by the development of high temperatures on the wearing surface, leading to plastic deformation and softening of a thin layer of steel at the wear flat. The combined effect of this weakening of the wearing surface and the continued abrasive action dramatically increases wear.

Also shown in Figure 4.5 are wear mechanisms operating during transport of fluids as in pumps and pipes. Abrasion and surface fatigue are the most common mechanisms found (Uetz 1986).

4.1.2 *Friction*

Apart from material loss (wear), other phenomena occur when two bodies are interacting in motion. Part of the energy is used to deform the bodies, either elastically or permanently (plastically). Part of the energy is transformed into heat during frictional contact. Both friction and wear are serious causes of energy and material dissipation (and are therefore of influence on the specific energy of the dredger system, Equation 3.5).

Since wear commonly is a result of moving contact of two or more bodies, *friction* is involved. Both friction and wear are not intrinsic material properties, but are characteristics of the engineering system. Friction is the resistance to motion arising from solids at the real area of contact of the surface.

PROCESS	MECHANISM				WEAR TYPE
	adhesion	*abrasion*	*surface fatigue*	*tribo-chem. reaction*	
impact motion					
solid - solid	□	□	■	□	*cracks, dimples (surface fatigue)*
solid - particle- solid	□	□	■	□	
sliding motion					
solid - solid	■	■ 2 body	□	□	*pits, hills, scaling, material transfer (adhesion); reaction products (trib.chem.reaction)*
solid - particle		■ 2 body		□	*parallel oriented scratches, grooves (abrasion)*
solid - particle		■ 2 body		□	
solid - particle- solid		■ 3 body		□	*irregular scratches, grooves, pits and hills (abrasion)*
flowing motion					
fluid with particles	□	■	■	□	*parallel or irregular scratches, grooves, pits and hills (abrasion)*
gas with particles	□	■	■	□	
fluid- or gas with particles	□	■	■	□	*cracks, dimples (surface fatigue)*
fluid- solid			■	□	*cavitation erosion: cracks, dimples (surface fatigue)*

Figure 4.5 Wear mechanisms thought to operate in cutting tools (impact and sliding motion) and pumps and pipes (flowing motion). Solid squares indicate dominant process. After DIN 50 320 (1979).

Especially when studying wear problems it is important to be aware of the effect of the "real" area of contact. Very often this real area of contact amounts to much less than the area over which the average stress is calculated (contact area values of less than 10% are common). Actual contact stresses may thus be much higher than the average stress.[16] This may lead to local elastic or plastic deformations, surface damage, local high temperatures (thermal damage), adhesive bonding across the contact surface (leading to *adhesive wear*), and other processes including chemical reactions[17] (Zum Gahr 1987).

In fact, the founders of the science of tribology, Bowden and Tabor (1950, 1964), used the notion of real contact area to develop the modern concept of friction (Scholz 1990). Bowden and Tabor assume that all real surfaces have roughness (Figure 4.6), which make contact at a few points, called bridges. The sum of all contact areas is the real contact area A_r. It is this contact surface that is really important during friction. Bowden and Tabor (1950) attempted to determine the real contact area across the metal surfaces they studied by measuring changes in electrical resistance. They assumed that with increasing normal load the bridges would increase in average size and number (with proportional decrease of electrical resistance). The small irregularities that hold the metal surfaces apart would flow under the applied load until their total cross section would be sufficient to support the load. Assuming that the plastic yield of the roughness asperities at a bridge contact would be approximately the same as that of the bulk metal, the real contact area A_r would be:

$$A_r = \frac{F_N}{p} \ (m^2) \tag{4.1}$$

where F_N is the normal load (N) and p is the penetration hardness (N/m^2), a measure of plastic yield strength as determined by an indentation test.[18] If the hardness of the two surfaces differ, the penetration hardness of the weaker metal would be chosen to calculate A_r.

In their first "adhesion theory", Bowden and Tabor (1950) assumed that due to the very high stress at the contact points, adhesion occurred there, welding the surfaces together to form junctions. In order to accommodate slip these junctions would have to be sheared through. To calculate the friction force the following was assumed:

- plastic yielding occurs at the contact points, until the contact area is just sufficient to support the normal load F_N.

$$F_N = pA_r \ (N) \tag{4.2}$$

- the friction force F_f necessary to achieve shearing through the welded asperities would be the sum of the shear strengths of the junctions (s_r) times the real surface area A_r:

[16] The German standard DIN 50 320 includes the recommendation to measure the *tribo-contact surface* A_r, in order to be able to calculate the nominal contact pressure $p_r = F_N/A_r$. The ratio of real to apparent area of contact may be as low as 10^4 (Zum Gahr 1987).

[17] Concerning chemical reactions: it is known that grinding of minerals may induce mineral transformations and chemical reactions; Juhász and Opoczky (1990) describe these *mechano-chemical* reactions.

[18] The hardness of a metal is the resistance to local deformation. Usually an indenter is pressed down with known load and the size of the resulting indentation is measured. Hardness is the load divided by the plastic deformation area (Bowden & Tabor 1964).

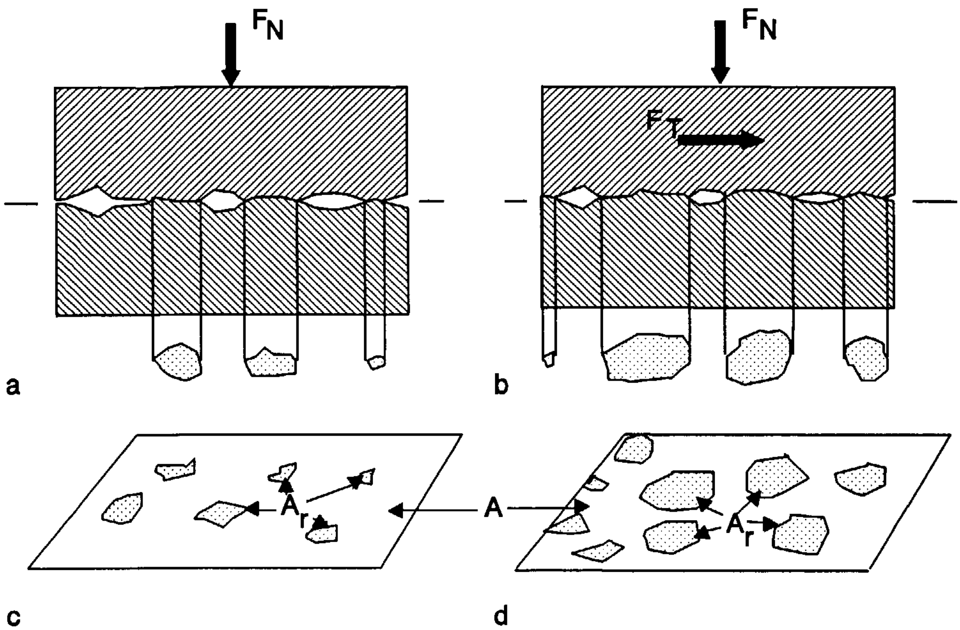

Figure 4.6 Real area of contact, A_r, during static loading (a,c) may differ from that during shear movement (b,d). a, b: cross sections; c, d: projections of contact area on the contact surface.

$$F_f = s_r A_r \ (N) \tag{4.3}$$

The coefficient of friction μ is the friction force divided by the normal force:

$$\mu = \frac{F_f}{F_N} = \frac{s_r}{p} \ (-) \tag{4.4}$$

Equation 4.2 and 4.3 incorporate the idea that the deformation of the asperities is a function of two different measures of strength of the material of the two surfaces in contact. [19] Scholz (1990) explains that, in first order, μ should be independent of material, temperature and sliding velocity, because both s_r and p are dependent on these parameters, but both to the same degree. And indeed predictions of the friction coefficient using shear- and compressive strength parameters often approach real values to a first order, exceptions can often be attributed to specific mechanisms (like lubrication) violating the assumptions made for Equations 4.1 - 4.4, the most important assumption being the proportional increase in A_r with F_N.

Bowden and Tabor's "adhesion theory" provided a model that for the first time explained the classical observations written down by Amontons in 1699 that friction is independent of the size of the apparent contact area A (Figure 4.6) and proportional to normal load. In other types of frictional resistance than adhesion, the real contact area will be important as well and often shear failures will also be

[19] If the two metal surfaces differ in strength, the shear strength and hardness of the weaker material are used (Bowden & Tabor 1950 p.99).

concentrated at the asperities of the contact surface. The model therefore has a more general application.

The reason that Equation 4.4 in most cases does not predict the correct value for μ has to do with the microscopic processes occurring near the asperities at the contact surface. For rocks, for example, it is clear that in many cases the assumption of plastic yield is incorrect, because of the brittle nature of rock. Then wear mechanisms play a role. If abrasive particles or hard protrusions exist along the surface, work will be done by microcutting, microploughing, microcracking or microfatigue (Figure 4.7). But, as Scholz (1990) emphasises, it is important to recognize that a constitutive law that describes μ is a combination of two constitutive laws that describe different processes: the contact of the surfaces, as in Equation 4.2, and the shearing of contact surfaces, as in Equation 4.3. Different mechanisms may be involved in these two processes and the interactions may be complex.[20] In general the friction coefficient may be approximated by models describing the shear and normal stress during movement along the actual contact surface:

$$\mu = \frac{[\textit{shear stress along movement surface}]}{[\textit{normal stress on movement surface}]} \tag{4.5}$$

Theoretical attempts to describe tribological processes often follow this approach (Landheer 1983, Zum Gahr 1987) and they show that friction and wear processes occurring at contact surfaces of sliding bodies are related phenomena. Equation 4.4 may, furthermore, be used to check in experimental work whether changes in wear mechanisms occur. A different friction coefficient indicates that different micro-processes are operating along the movement surface.

4.2 WEAR MECHANISMS IN DREDGER TEETH

Rock cutter suction dredgers have cutterheads with steel picks or chisels of a hard and tough quality. Normally so-called pick points (long or short) are used for cutting strong rock. Weaker rock may be cut by trapezoidal shaped steel teeth (Figure 3.5). Other rock excavation machinery, such as road headers or tunnel boring machines commonly use tungsten carbide drag bits. Attempts to improve the hardness of dredging teeth by tungsten carbide inserts have not been successful. The inserts usually break off. Carbide teeth are brittle, they would fail prematurely due to the high impact stresses occurring while cutting with a rock cutting dredger. In soft soils welding of tungsten carbide on the steel teeth, *hard facing*, has been used to improve wear resistance. From the perspective of tool wear the tendancy is to increase the *hardness* of the tools. The materials commonly used, such as hardened steel or tungsten carbide, tend to become more brittle with increasing hardness. They often display fracture damage when cutting stronger rocks. An optimum has to be sought

[20] There is an analogy with the relationship of shear strength and normal strength of a non-cohesive planar rock joint. At low confining pressure and temperature a linear ("Coulomb") failure relationship may exist, of which the friction coefficient is μ. If the joint is rough, or has rock bridges (cohesion), or at high pressures and temperatures, failure mechanisms are more complex; in general the *failure envelope* is curved.

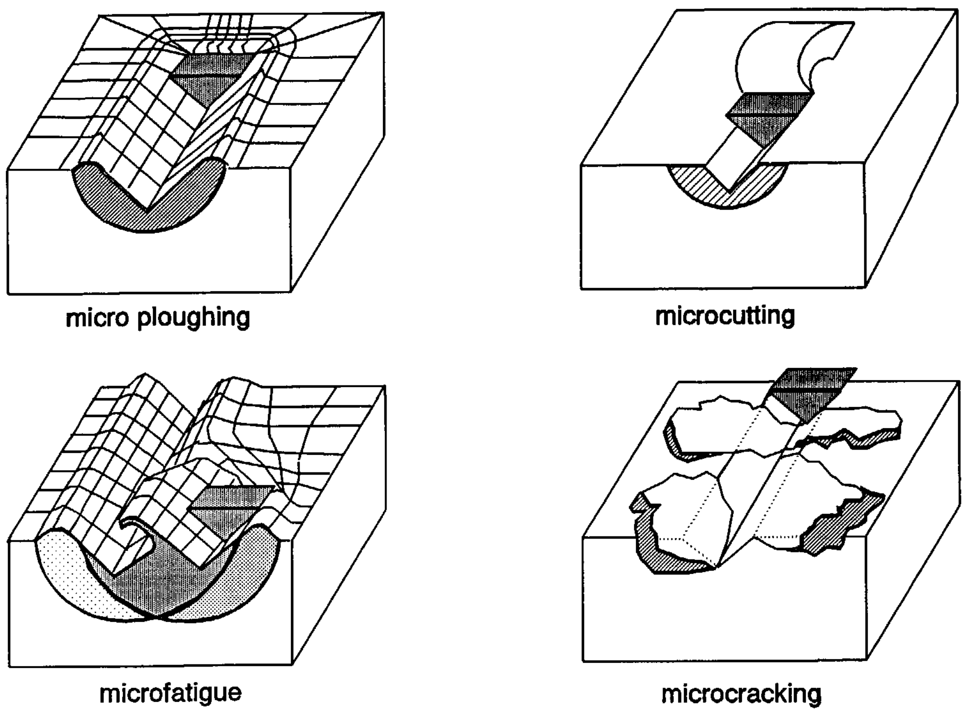

Figure 4.7 Four types of mechanisms operating on tool materials during abrasive wear (after Zum Gahr, 1987).

between conflicting requirements and the result is dependent on the type of soil or rock dredged.

From personal observation of worn dredging teeth at dredging projects and from accounts of dredging engineers, the following list of wear mechanisms can be made:

- grooving wear, parallel deep grooves (seen on teeth cutting in stiff, clayey sands, quartz-rich): high level two-body abrasion
- irregular deep scratches: high level three-body abrasion
- rubbing, polishing of surface, fine scratches: low level abrasion, surface fatigue
- plastic deformation of surface, sometimes accompanied by visual evidence of high temperatures (red-hot teeth): abrasion accompanied with stripping of plastically softened steel.
- development of crack patterns on wearing surface: surface fatigue (observed on teeth cutting in (non-abrasive) limestone rock.

The observations listed above are explained in terms of high-level- or low-level abrasive wear (Figure 4.4) and the wear mechanisms and wear types given in Figure 4.5. Excluded in the above list are the fracture of teeth, which occurs commonly when cutting too strong rock, and the breaking of teeth connections to the cutter arms, which are often also due to wear.

Most of the studies on wear of rock cutting tools deal with tungsten carbide picks (e.g. Osburn 1969, Phillips & Roxborough 1981, Speight & Fowell 1984, Hurt & MacAndrew, 1985, Clark 1987). No study of observations on worn dredger teeth is known, but work has been published on ripper tips (Muro 1985) and digger teeth (Mashloosh & Eyre 1985, Schweins 1986).

4.2.1 *Studies on the wear of digger teeth and ripper tips*

Mashloosh & Eyre (1985) studied the wear of digger teeth placed on a bucket drag line, working in a gravel pit, excavating under water. Such teeth are comparable to the teeth used for dredging. The digger teeth were made from low alloy carbon steel, heat treated to a Vickers hardness of 510 H_v (5000 MPa)[21], with high toughness[22]. The digger was excavating an aggregate of quartz gravel particles. Using Figure 4.5 a first guess of possible wear mechanisms may be made. Although some impact forces can occur using the drag line, the main motion operating will be sliding. The teeth will slide against an aggregate of particles. The surrounding medium is water. Two body abrasive wear and tribo-chemical reactions are common mechanisms under such circumstances. Mashloosh & Eyre (1985) made an extensive microscopic examination of the worn surface, using the scanning electronmicroscope to study a replica of the worn surface and a reflection microscope to study a polished section cut normal to the worn surface. The surface showed parallel grooves, produced by ploughing (Figure 4.7). The scanning electron microscope gave more detail on the ploughing mechanism and additional plastic deformation. The microscopic structures observed with the reflection microscope showed evidence for intense rubbing between the gravel particles and the teeth, leading to high contact temperatures at the surface. These high temperatures caused phase transformations in the steel up to a depth of 0.1 mm below the contact surface, with resulting changes in hardness (see Chapter 9.4). No indications of corrosion reactions were found, the water with pH of 7 apparently did not affect the wear. Abrasion was clearly the dominating mechanism, with besides ploughing and cutting a considerable amount of rubbing causing plastic deformation of the surface and frictional heating. In the particular case studied, the teeth lasted about 200 working hours, and their better performance against other steel types was attributed to their hardening ability during the wear process. However, Mashloosh & Eyre note that the same teeth would last only 20 hours in a pit where gravel was dug dry.

Muro (1985) describes extensive tests carried out on ripper tips coated with different metals or composites, to increase wear resistance. Usually ripper tips are made of wear resistant forging or cast steel with a Vickers hardness of 3900 - 4900 MPa, with high enough toughness to resist the large excavation forces. But, when strong rocks are being ripped, high temperatures develop leading to plastic softening of the steel. In addition, rocks with abundant quartz cause abrasive gouging wear. Muro studied 7 different types of metal; 1 untreated ripper tip, 4 coated with hard facing by thermal spraying and 2 welded faced tips. A laboratory abrasion test (pin-on-disc type; Chapter 11) was carried out, whereby the test specimens were tested against a granite rock surface. The tests had some relevance to the dredging problem

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[21] The units commonly used for Vickers hardness are kg/mm^2, 510 H_v corresponds to 5000 MPa. Quartz has a H_v of 1000 to 1200 (9800 - 11760 MPa; see Chapter 10).

[22] Toughness is defined as resistance to brittle failure at impact loading.

for they were carried out both dry and wet, wetting being done by jetting water into the contact between the steel test pins and the rock. It appeared that the amount of wear was greater in the dry state than in the wet state. The latter could be explained by the cooling effect of the water jetting. At higher contact pressures (5 - 25 MPa), temperatures from 550 - 1200 °C were measured at the steel contact in the dry tests, 1200 °C being the temperature at which the steel starts to melt. During the wet tests no temperature higher than 300 °C was recorded on untreated ripper tip steel. Also a field test was carried out, showing that both the untreated tip and the tips welded with a coating behaved well against granitic rock, but that the tips which were hard faced using thermal spraying showed increased wear, due to the softening of the hard, heat-treated material by the fusion process.

4.2.2 *Conclusions from tribological studies on cutting tool wear*

From the two studies on wear of steel cutting tools referred to, some impression of the complexity of the problem can be gained. It appears that the composition of the steel is important, whether or not water is present, and contact pressures have a large influence on wear, because high temperatures may develop. Much of this information comes from laboratory tests (both investigations involved pin-on-disc type tests). Only Muro pays much attention to the rock types being excavated (Muro 1985, Hata & Muro 1978). Examining the damage on the wear surfaces of the teeth, aided by microscopic techniques as was done by Mashloosh & Eyre (1985), can give a reasonable impression of the type of wear mechanisms operating. Both papers were written by specialists in tribology. Both mention abrasion and high temperature plastic deformation of the steel as the dominant mechanisms. Characteristic ally for studies in tribology, both performed tests on different tool materials in order to select the type best suited for the problem at hand. Apart from this, both studies showed that the *shape* of the cutting tool is important (self sharpening during wear is aimed at). Mashloosh & Eyre made the important observation that, when the circumstances changed (the type of gravel and dry instead of wet excavation) the amount of wear differed considerably.

When considering rock cutting as a tribological system (Figure 4.1), it is obvious that a large part of the problem of tool wear lies in the variable nature of the rock or soil *counterbody* and -perhaps to a lesser extent- the *environment* of (sea)water. Commonly rocks have a variable structure and composition, which makes rock cutting a type of tribo-system that is hard to study in the laboratory. In Chapter 11 it is shown that tests commonly used in tribology, like pin-on-disc, are probably less suitable to show the influence of *the rock type* on wear. Note that the tests used in tribology are mostly directed to the *solid body* -or tool- material. Very commonly for convenience sandpaper is used as a counterbody in pin-on-disc tests to test different types of steel; the rock or soil counterbody is then not even considered.

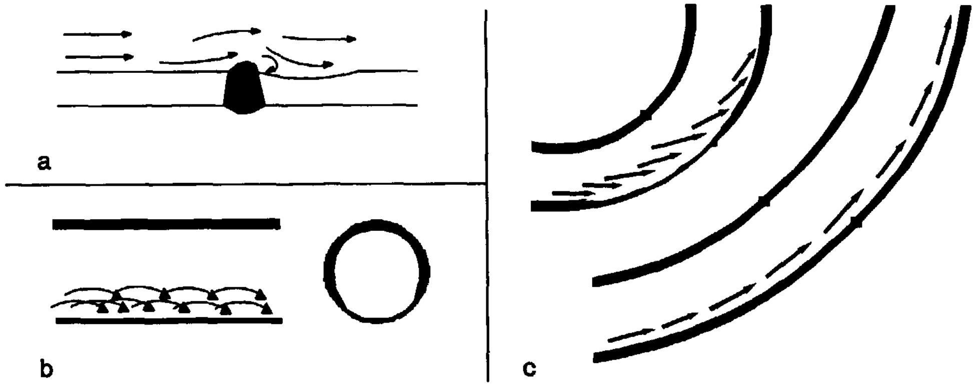

Figure 4.8 Erosive wear: a. abrasion downstream of weld b. thinning by sliding or bouncing particles c. at changes in direction, less wear at low curvature (after Wasp et al. 1977).

4.3 WEAR MECHANISMS IN PUMPS AND PIPELINES

Once the rock or soil has been excavated by the cutterhead, it is taken up by the suction mouth. The mixture of water and excavated material is generally referred to as *slurry*. About transport of slurries through pipeline much research work has been published and regularly conferences are being organised on hydraulic transport of solids in pipes.

The work by Wasp, Kenny & Gandhi (1977), treats the basic principles of slurry pipeline transportation, including corrosion and abrasion (Wasp et al. 1977). Slurry pipelines for mining and industrial purposes have often a design life of 20 to 50 years. They are commonly made of mild steel types. It is therefore important to assess the aggressiveness of the slurry. Corrosion and abrasion are the most important wear mechanisms. Internal corrosion is caused by the slurry water, often a redox reaction is set up between the steel of the pipe and the slurry water, where a low pH favours the reaction. The corrosion product is iron hydroxide (rust). If an abrasive is present in the slurry this assists in eroding the rusted parts, leaving fresh metal to be corroded. Corrosion can be controlled by neutralising the redox reactions at the pipe wall. This may be done by additives, causing a protective film on the pipe wall, or removing oxygen from the slurry. Also corrosion resistant linings of the inner pipe wall are known, which may be applied with the pipe already laid, new or old. If the slurry is not corrosive, only mechanical erosion processes operate.

Figures 4.3 and 4.5 indicate that during flow motion (erosion), abrasion (Figure 4.7) and surface fatigue (caused by impact of particles) are the main wear mechanisms operating, but both tribo-chemical reactions (corrosion) and adhesion occur as well. Figure 4.8 shows common effects of erosive wear. If the slurry contains coarse material of gravel size or larger, as will be often the case in rock dredging, sliding transport over the bottom of the pipe will be occurring, with consequently abrasive wear of the bottom part of the pipes.

In pipes less wear occurs along the walls than might be expected. This is due to lower transport velocities along the pipe walls and a tendency of rock particles to be

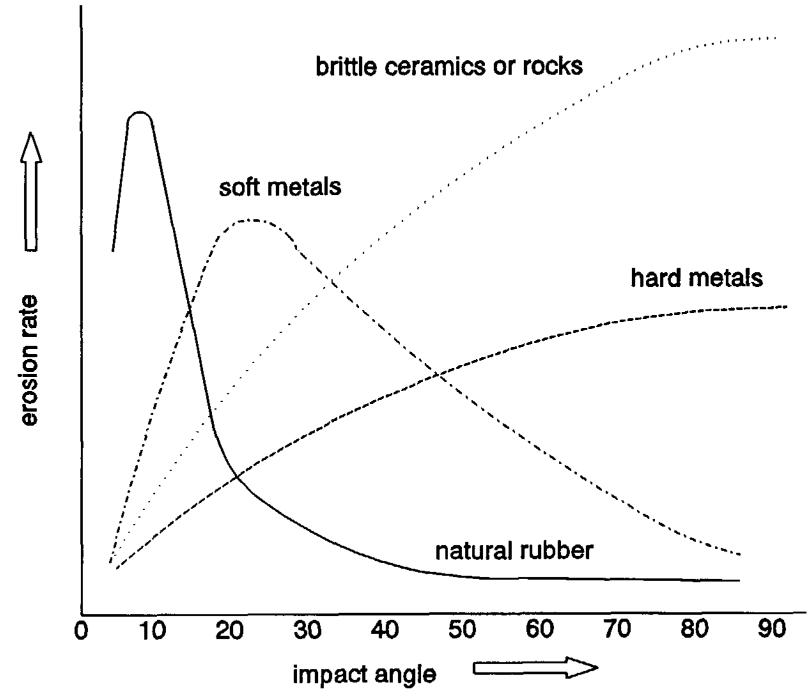

Figure 4.9 Rates of erosive wear of different materials depending on angle of impact (after Mens & de Gee 1986, Zum Gahr, 1987).

transported more centrally in the pipe. Most erosion occurs in bends and other irregularities in the pipe. Here changes in design can improve the situation (De Bree 1975).

4.3.1 *Slurry erosion test*

Especially for the slurries with finer grained solid particles, such as would be the case in dredging soils up to fine gravel size, a useful laboratory test exists. The *slurry erosion test* (Table 4.1) basically is a test container supporting a slurry in which test pieces of steel are rotated. Different types of steel may be tested and compared. The test container, appropriately called *Verschleißtopf* (wear pot) in German, comes in many varieties (Uetz, 1986). A suitable variety consists of two parallel discs on which four specimen holders are placed. Each disc is driven by a separate motor and placed in the container with the slurry. When the discs are rotated through the slurry, the specimens will be impacted by particles and are also subjected to interaction with the slurry liquid. Rotation speeds can be in the order of 4 - 30 m/s (Mens & de Gee 1986). The specimens can be oriented in such a way that the angle of impact of the abrasive particles can be varied (De Bree 1975, De Bree et al. 1980, 1982, Mens & de Gee 1986). The material best resisting wear due to the slurry may be found. The test was used to examine the effect of corrosion and erosion of coal slurries (Erlings et al. 1984) and the effect of erosion of seawater sand slurries (Mens & de Gee 1986), the latter being a dredging application. The

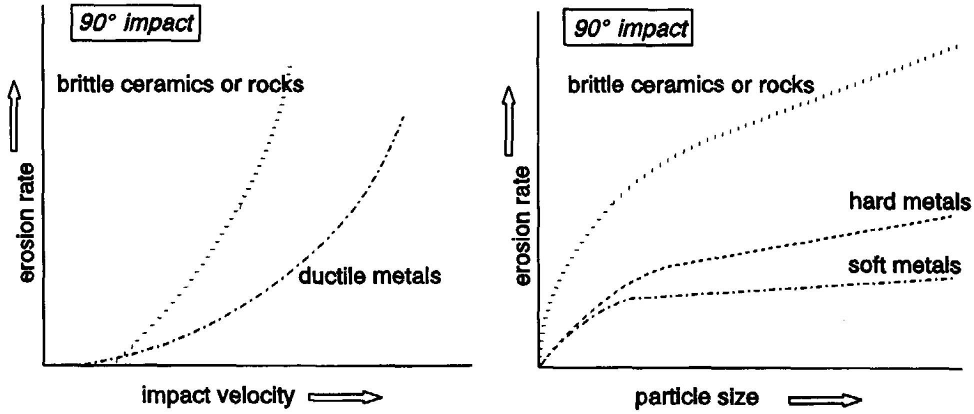

Figure 4.10 Erosion rate at 90° impact angle depending on particle size and velocity (after Zum Gahr 1987).

tests were done to compare the behaviour of different types of materials in a specific slurry type. Results common to such studies are shown in Figures 4.9 & 10.

4.3.2 *Parameters influencing wear in pumps and pipes*

Abrasive wear in a slurry pipeline is controlled by the size distribution of the abrasive particles, the slurry concentration and the velocity of flow. These properties are interdependent (Wasp et al. 1977). Grain size distribution and density determine the critical velocity necessary to be able to transport a slurry through a pipe of a certain diameter. Abrasive wear increases with the cube of slurry velocity, according to Wasp (1977) and confirmed by de Bree et al. (1982) and Mens & de Gee (1986), Figure 4.10. Mens & de Gee (1986) show that for soft steel this relationship also holds for other angles of attack than 90°. Since wear also increases with size, Figure 4.10, the effect of reducing the size of the particles is beneficial. The required transport velocity can be lowered, and the rate of abrasive wear will thereby be reduced.

The centrifugal pumps used for hydraulic dredging have usually a wear resistant metal lining. The pumps have generally higher impeller tip speeds and higher head capabilities than the pumps used for industrial slurry transport (Wasp et al. 1977). Resistance to wear should be higher than in pipes, mainly because of the higher impact velocities occurring in the pump. Apart from the mechanisms mentioned above, sometimes *cavitation erosion* may occur in pumps (Uetz 1986). This type of wear is the result of the forming and subsequent implosion of gas bubbles, due to local degassification in the fluid. The implosion of the bubbles causes cyclic stress impacts on the pump lining or impeller surface that may lead to crack formation. It can be seen as a type of surface fatigue wear mechanism and it it is more related to the composition of the slurry liquid than to the rock or soil component of the slurry.

Important geotechnical properties with regard to wear in pipelines and pumps are therefore clear. The shape, size and hardness of the particles will determine the abrasiveness. The hardness contrast between particles and lining (Figure 4.4)

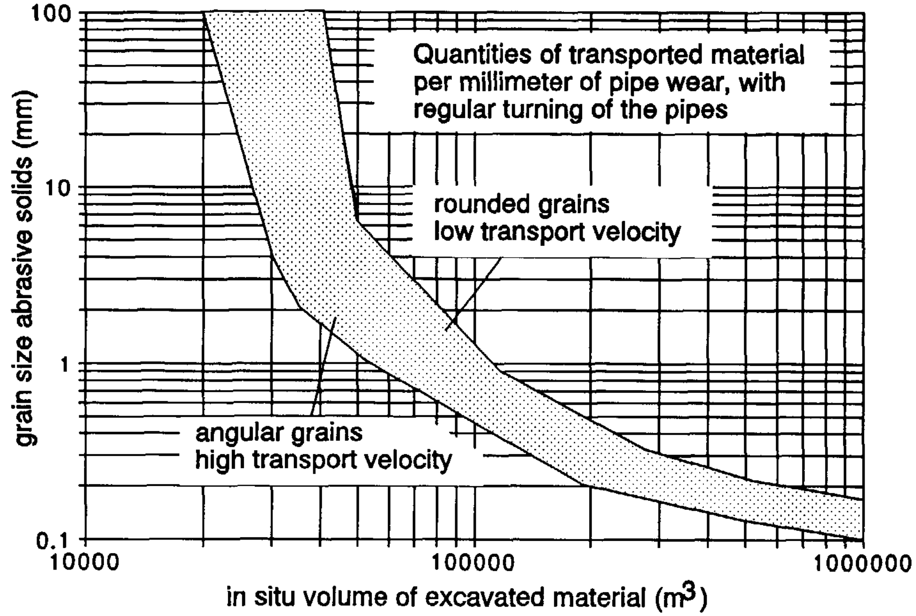

Figure 4.11 Indication of wear rate of dredging pipes, based on data of pipes which have been turned twice during their life (0°, 120°, 240°) (courtesy MTI-IHC Holland).

determines whether the wear rate will be on a high or low level. Also the strength of the particles is important; due to impact events size reduction may occur, with resulting decreasing rates of erosion. Presence of clays or fine silts may influence the viscosity of the slurry or cause the formation of clay balls.

The amount of wear is determined further by particle impact angle and to a high degree by solid particle concentration and impact velocity (De Bree et al. 1982, Mens & de Gee 1986), see also Figure 4.9 & 10. Wear can be controlled by controlling slurry composition (hardly an option that can be chosen while dredging), transport velocity, adapting shape and design of pumps and pipes and choosing special lining compositions (For example: rubber materials perform better under high angles of impact than steels, Figure 4.9).[23]

An indication of actual wear data of pipes is given in Figure 4.11.

4.4 THE TRIBOSYSTEM OF ROCK CUTTING DREDGING

The study of wear problems by defining the *system* in which the wear process operates can also be applied to the real scale: the dredger cutting through a rock mass.

The system of rock cutting dredging can be described by the mechanical and geometrical properties of the tool, the properties of the rock mass to be excavated

[23] Only when the material is of relatively small size and rounded; coarse angular material may damage the rubber lining by cutting.

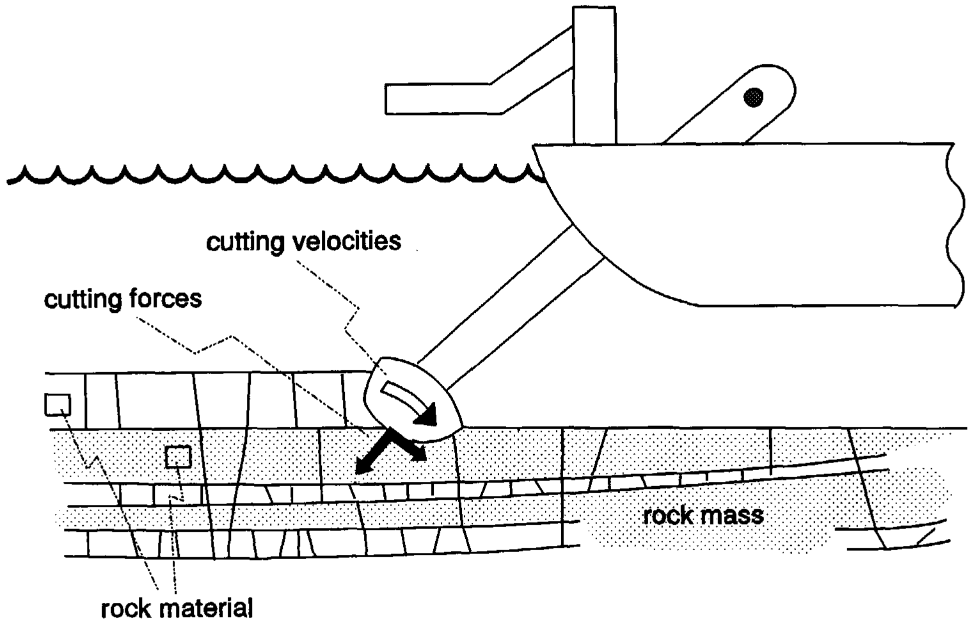

Figure 4.12 Factors affecting the system of a rock cutting dredger.

and the working conditions (Figure 4.12). The working conditions are described by for example the dynamic stresses involved. The environment involves climatic factors, presence of wave action, composition of the seawater. The complex interaction of these factors results in a particular wear process operating, which may be very dependent on site and dredger specific conditions.

Consulting Figure 4.2 the enormity of the task of properly determining the tribosystem on this scale becomes clear, but may be approached by scaling down the problem. The interaction between cutterhead and rock is dependent on variables, both internal and external, too numerous to be scaled down and simulated by simple laboratory tests. The system of a dredger cutting rock may be approached experimentally by the scheme shown in Figure 4.13. Simple laboratory tests like the pin-on-disc test (category VI in Figure 4.13) or the cutting test (category V) are remote from the actual process taking place when dredging rock. In tribology it is attempted to simulate as well as possible particular aspects of the process under study (using the same environmental conditions, simulate the proper temperatures and forces etc.). Tests in category VI or V are done to obtain an idea about relative performance of different types of cutting steel for example. The tools will undergo tests with increasingly complicated equipment and finally the prototypes are tested on the real machines.

A problem with this approach lies in the proper description of the *rock mass* (Figure 4.12). The rock mass may consist of different types of *rock materials*, may have *discontinuities* like rock joints (Chapter 8), and may be partially weathered. Sampling a rock mass to provide the laboratory with representative samples to perform tests in the categories V or VI (Figure 4.13) may lead to a prohibitive high number of tests to be performed. Most published literature on rock cutting or on

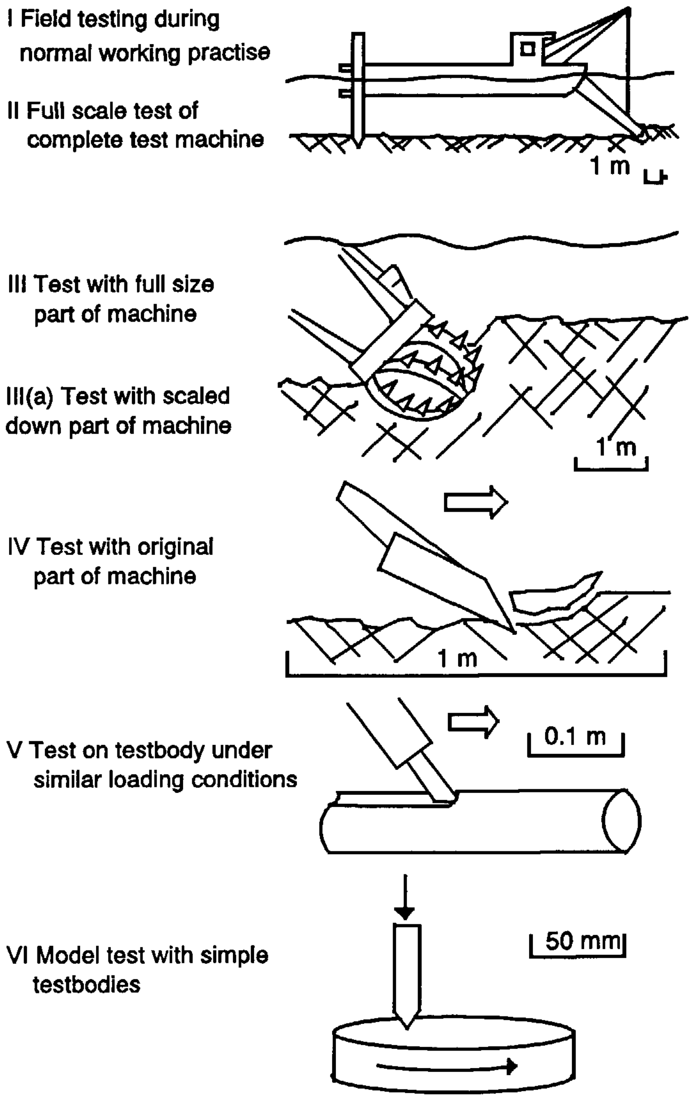

Figure 4.13 Tribological test categories for the rock dredging system. Explanation in the text.

wear testing involves studies where only one rock type, preferably homogenous, is tested.

Apparently, the best and most informative test would be with the dredger on site, but from the contractor's point of view this is may be far too late. A contractor would like to know *beforehand*, in the tendering stage, how much wear is to be expected, to be able to estimate tool consumption for a project.

Some might argue that wear must be determined in a prototype or one-to-one scale test, for each test done on another scale or in another environment could give false results, because it would not simulate all wear influencing factors. In practice, however, a prototype test or a 1:1 scale rock cutting test only takes place when dredging has started, after the site investigation and tendering stages. Often the 1:1 scale test turns out to be the real excavation process itself.

Another point to note concerns the location where a 1:1 scale test is carried out. In linear projects, like trench excavation or harbour channel deepening, prototype tests carried out on a local spot could give very misleading results if some variation of properties or environmental circumstances were present along the excavation route.

One solution could be that 1:1 scale tests are to be used as means of calibration of wear test data obtained at lower categories of wear testing. It is obvious that many tests could be performed for low cost on many samples on the laboratory scale. By calibrating the results of these data with a trial dredging operation on the real scale, contractors may be able to estimate to a much higher level of accuracy the tool replacement rate to be expected for a project. How this could be done is the subject of this work.

CHAPTER 5

Influence of site geology on wear problems

Many dredging contractors have gained experience with rock dredging in the Middle East area during the 1970's. Large dredging projects took place along the shores of the Gulf States and in the Suez canal. Especially in the Gulf area frequently a *caprock*[24] of strongly cemented rock was encountered within otherwise non- or slightly cemented sand deposits. This rock type required large rock cutting suction dredgers, which were specially built for this purpose (De Koning 1993). Many contractors became familiar with problems of rock cutting and high consumption of pick points. Somewhere in the area of Jubail *Pick Point Island* is located, which is said to be covered completely by used pick points. Much experimenting took place. Special designs of cutterheads were tried out, as were different types of cutting tools, like pick points with tungsten carbide inserts. The data gathered by the contractors during such projects were regarded as highly confidential, for competitive reasons (see Chapter 1.2.2). Considering the concept of the tribosystem of a cutting suction dredger, as described in Chapter 4.4. this is not surprising. Although accurate data on production rates and tool consumptions for these projects do exist, very often descriptions of the rock and soil materials on which these data apply are lacking. The reason is that monitoring of the material excavated is cumbersome on rock cutting suction dredgers. Samples should be taken at the disposal sites of the slurry, or underwater inspections should be performed. Another problem is that at that time, as even today, the importance of the exact nature of the rock or soil materials being dredged was not recognized.

During the author's research into wear problems of cutter suction dredging in rock, which started early 1986, close contact with dredging contractors, also in a special working group on tooth wear, hardly ever resulted in the disclosure of tool consumption rates for particular projects and if so it appeared that factual data on the rock involved was lacking or of poor character. The involvement of engineering geologists or geotechnical engineers familiar with geology in these projects was negligible. As mentioned in Chapter 1, this has to do with the unfamiliarity with

[24] Caprock (hard-pan, duricrust) is a layer of strongly cemented material of variable thickness (cm to m scale) occurring in otherwise unlithified sediments, often found close to a (paleo) ground surface. The cementing agent (calcareous, siliceous or ferruginous) is often precipitated from percolating groundwater.

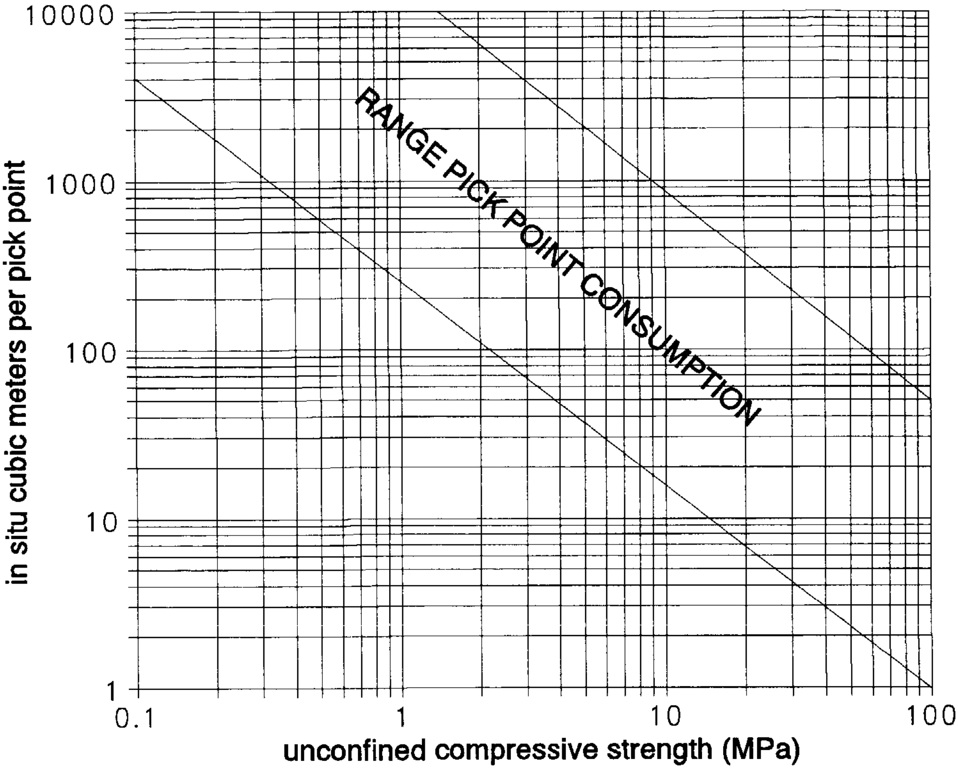

Figure 5.1 Diagram showing the range of pick point consumption per in-situ volume of excavated rock, based on limited data of dredging works with CSD's.

rock engineering. It must be stated, however, that in the field of rock engineering itself wear of rock cutting tools is still regarded as a difficult problem. Only in the 1980's were the first predictive models for rock tunnel boring machines published (see Chapter 18) and in these the determination of the abrasiveness of rock remained a difficulty. As will be illustrated with the following example, the type of rock data that were available often lacked a description of the *mineralogical composition* of the rocks. Emphasis was, understandably, on rock strength parameters. Contractors could see a relationship between dredgeability and rock strength and also correlations of wear with rock strength were found (Figure 5.1). Unconfined Compressive Strength of rocks and its variation in the different units to be dredged was regarded as a measure of importance, next to information on the general distribution of the rock and soil types. The abrasiveness of the mineral quartz on pipes, pumps and teeth was recognized, but not the necessity to identify this mineral within the different rock or soil types to be dredged.

The dredging works in the Middle East and Gulf areas had given the contractors a feel for rock dredging and considerable experience was built up, which was used to evaluate tenders for new projects.

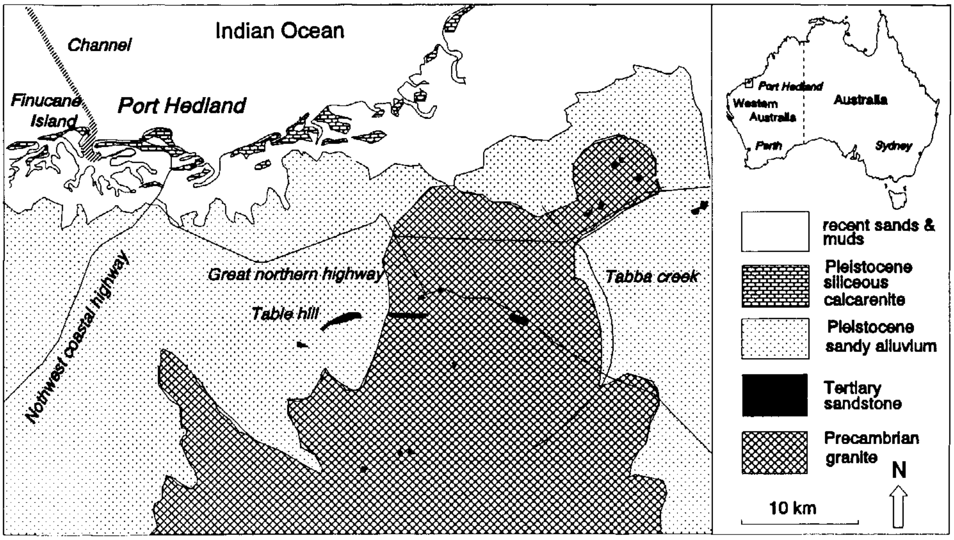

Figure 5.2 Geological sketch map of Port Hedland area. Part of the channel dredged is indicated by cross-hatching (after Coffey & Partners, Sydney).

5.1 PORT HEDLAND HARBOUR DREDGING PROJECT

In 1985 a dredging project was carried out at Port Hedland, Western Australia, which aroused concern. Extremely high wear rates were experienced, which were unexpected by the contractor involved. It turned out that the geotechnical information provided to the contractor missed vital information on the abrasiveness of the rocks. The project will be described emphasizing features important to the wear problem.[25]

The dredging works to be carried out included:

- deepening and extending the harbour approach channel for a distance of 21 kilometres
- deepening and widening of the inner harbour
- deepening and extending the berth pockets at the Mount Newman Mining Company's ore wharf
- widening of the existing channel on curves
- reclamation or removal of several areas of ore spillage and previously dredged spoil

The harbour and part of the channel are shown on the map of Figure 5.2. The amount of work involved the dredging of 12,400,000 m^3 of material.

[25] Site investigation reports of the Port Hedland project were available for study, with kind permission of Ballast Nedam Dredging, present owner of the dredging companies involved in Port Hedland in 1985.

Table 5.1 Classification of calcareous rocks as used in the original site investigation for Port Hedland dredging project.

Total Carbonate Content	GRAIN SIZE (mm) Fine grained	0.002	0.06 Medium	2 to Coarse gr. 60	Point Load Strength Is(50) kPa	Degree of cementation
100 – 50	CARBONATE MUD	CARBONATE SILT	CARBONATE SAND	CARBONATE GRAVEL	Friable (not possible to carry out test)	Soils description
50 – 0	CLAY	SILT	SAND	GRAVEL	100	Very weakly cemented (VWC)
100 – 50	CALCILUTITE	CALCISILTITE	CALCARENITE	CALCIRUDITE	300	Weakly cemented (WC)
50 – 0	calcareous CLAYSTONE	calcareous SILTSTONE	calcareous SANDSTONE	calcareous CONGLO-MERATE	1000	Moderately cemented (MC)
100 – 50	CRYSTALLINE LIMESTONE				3000	Strongly cemented (SC)
					10.000	(VSC)

5.1.1 *Site investigations for the dredging project*

The site investigation done by the consultant geotechnical engineer can be considered as typical for a dredging project. The attention was focused on the dredging site itself and emphasis was put on the gathering of geotechnical information. In 1980 a feasibility study for the project had been performed, involving 50 boreholes and testing of a small number of samples. The site investigation undertaken by the consultant engineer in 1984 involved:

- sinking 69 rotary cored boreholes to depths up to 18 m below datum, core size 100 mm, with SPT, Standard Penetration Testing, where appropriate.
- sinking 107 vibrocore boreholes (up to rock, with SPT tests)
- sinking 31 jet probes (up to rock level)
- undertaking laboratory tests on the samples recovered
- seismic reflection profiling
- seismic refraction profiling
- side scan sonar traverses
- a review of past investigation works, including the feasibility study for the project carried out in 1980
- some comments on experiences gained in past dredging works

During rotary boring the penetration rate, down force and rotary force on the drill bit were recorded, giving indirect information on characteristics of the materials. During vibrocoring also the rate of penetration, down force and vibratory frequencies and power were recorded. In the boreholes Standard Penetration Tests (SPT)[26] were

[26] A standard sample tube is hammered 300 mm into the soil section to be tested, at the bottom of the borehole, using a standard procedure. The number of blows, N, is a measure of penetration resistance of the soil and has some correlation with the shear strength of soils.

performed. Pocket penetrometer tests[27] were performed on some tube samples of clayey materials and on samples from the vibrocored boreholes. The boreholes of the 1980 feasibility study were relogged, in order to adapt the descriptions to the classification adopted in 1984. All soils were described using the PIANC classification (Appendix A). For the rocks, which are of calcareous and siliceous nature, an adapted version of the Clark & Walker (1977) classification system was used (Table 5.1). Table 5.2 gives the Clark & Walker classification for comparison. The essential features of the site investigation were reported in the form of drilling logs, laboratory test results and track plots of geophysical researches. A series of 85 cross sections and a longitudinal section down the centre line of the channel formed the geotechnical model (Table 5.3).

To have an idea of the extent of the work, including the boreholes of 1980, the borehole density was one borehole per 175 m length of channel (see Figure 5.4). Since the structure of the rocks was relatively simple (subhorizontal layering), this was regarded as normal for dredging projects (Verbeek 1984). The purpose of the geophysical work was to provide information on the subsurface between boreholes. This purpose was, however, not well met and the geophysical work was difficult to interpret. The combination of low borehole density with poor geophysics led to a relatively high uncertainty about the ground conditions. However, the geotechnical model was also based on an interpretation of the local geology.

5.1.2 *Interpretation of the regional geology in the site investigation report*

The regional geology was described by concentrating on the coastal area itself. The geology of the area was well known and several geological maps have been published. A regional geological map 1:250.000 (sheet SF50-4 Port Hedland-Bedout Island 1982, and a 1:50.000 urban geology map sheet 2657 (III) (Archer 1983). A sketch map based on these maps, such as Figure 5.2, was not given in the original site investigation report. The site investigation report mainly discusses the geology of the area of immediate interest, concentrating on the most recent geological history.

The framework of the geology of Port Hedland was described by the geotechnical consultant as "... made up of several parallel strongly cemented calcarenite beach rock and former dune ridges trending east north east. Inland of this skeleton to the south there is a large area of tidal creeks and estuaries, partially infilled with Mangrove mud. Red dunal sands of the Pindan Sand Association are encountered on the north side of the North West Coastal Highway, and the thickness of these superficial sands decreases southwards until Archaean granites appear, some 15 km to the south.

Port Hedland has always been a natural position for a port because the calcarenite ridges are gapped between the townsite and Finucane Island, and at the time that the sand ridges and beach sands were being lithified, the tidal influx kept a major channel open to the sea."

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[27] With the pocket penetrometer an estimate of the unconfined compressive strength of cohesive soils can be made. Can be used on small samples.

Table 5.2 The geotechnical classification for sedimentary calcareous rocks of Clark & Walker (1977).

Degree of INDURATION	Approximate Unconfined Compressive Strength (MPa)	ADDITIONAL DESCRIPTIVE TERMS BASED ON ORIGIN OF CONSTITUENT PARTICLES: NOT DISCERNIBLE		BIOCLASTIC (organic) OOLITE (inorganic)	SHELL (organic) CORAL (organic) ALGAL (organic) PISOLITES (inorganic)	TOTAL CARBONATE CONTENT % (constituent particles plus matrix)
		INCREASING GRAIN SIZE OF PARTICULATE DEPOSITS →				
		0.002	0.060	2	60 mm	
Non-indurated	Very soft to hard < 0.036 to > 0.3 MPa	CARBONATE MUD	CARBONATE SILT	CARBONATE SAND	CARBONATE GRAVEL	90
		clayey CARBONATE MUD	siliceous CARBONATE SILT (1)	siliceous CARBONATE SAND (1)	mixed carbonate and non-carbonate GRAVEL (2)	50
		calcareous CLAY	calcareous SILT (1)	calcareous silica SAND (1)		10
		CLAY	SILT	SAND	GRAVEL	
Slightly indurated	Hard to moderately weak 0.3 - 12.5 MPa	CALCILUTITE	CALCISILTITE	CALCARENITE	CALCIRUDITE (calc. congl. or breccia)	90
		clayey CALCILUTITE	siliceous CALCISILTITE	siliceous CALCARENITE	conglomeratic CALCIRUDITE (2)	50
		calcareous CLAYSTONE	calcareous SILTSTONE	calcareous SANDSTONE	calcareous CONGLOMERATE	10
		CLAYSTONE	SILTSTONE	SANDSTONE	CONGLOMERATE OR BRECCIA	
Moderately indurated	Moderately strong to strong 12.5 - 100 MPa	fine grained LIMESTONE		detrital LIMESTONE	CONGLOMERATE LIMESTONE	90
		fine grained argillaceous LIMESTONE	fine grained siliceous LIMESTONE	siliceous detrital LIMESTONE	conglomeratic LIMESTONE (2)	50
		calcareous CLAYSTONE	calcareous SILTSTONE	calcareous SANDSTONE	calcareous CONGLOMERATE	10
		CLAYSTONE	SILTSTONE	SANDSTONE	CONGLOMERATE OR BRECCIA	
Highly Ind.	Strong to extremely strong	CRYSTALLINE LIMESTONE OR MARBLE (tends towards uniformity of grainsize and loss of original texture)				50
		conventional metamorphic nomenclature applies in this section				

(1) Non-carbonate constituents are likely to be siliceous apart from local concentrations of minerals such as feldpar and mixed heavy minerals.
(2) In descriptions the rough proportions of carbonate and non-carbonate constituents should be quoted and details of both the particle minerals and matrix minerals should be included. (3) Calcareous is suggested as a general term to indicate the presence of unidentified carbonate. Where applicable, when mineral identification is possible, calcareous is referring to calcite or use alternative adjectives such as dolomitic, aragonitic, sideritic. (After Clark & Walker 1977)

The geological history was described as follows: "In late Pleistocene time when the sea level was not very different from the present one, erosion of the granitic hinterland to the south of Pt Hedland released large quantities of alluvial sands and kaolinitic type clays into local river systems, that discharged into the sea in deltaic deposits. This material was later reworked off an extensive sea beach, and calcarenite and capstone gravels and cobbles eroded from an exposed outcrop on the beach were added to the alluvial materials.

Sea level then fell, and a beach was formed in the vicinity of 16 and 17 km of the present channel and at right angles to it.[28] Sand dunes of shell grit and lime sand were blown inland from the shore forming primary and secondary dunes which were lithified in place from the solution and deposition activity of weak carbonic acid derived from CO_2 and dissolved in rain or sea water.

Sea level then fell and a new series of sand dunes were formed on the edge of the beach in the vicinity of 20.5 km. While these dunes were lithifying in place the old sandy clay landscape was being modified in river and terrace areas. Weathering and leaching processes became active with kaolin clay being changed into palygorskite.

A rise in sea level formed another sand beach in the vicinity of 5 to 7 km but no dunes were formed before the sea rose again this time to near its present day level. This sea beach became lithified.

Dune ridges were blown up on the beaches, and while the dunes were being lithified, tidal flows down Stingray and North West Creeks kept the gap between Pt Hedland and Finucane Island open, and a well formed tidal discharge stream was formed. During this period the leaching of the red sandy clays proceeded with enrichment and leaching processes continuing with variable palygorskite cementation.

Finally, sea level rose a few meters creating the flooded mangrove hinterland and estuarine muds were deposited in the areas clear of tidal flux. The two main upper calcarenite ridges resulting from the dune cementation ran from near Hunt Point, then down either side of the present main road to the vicinity of the Leslie Salt pans."

The emphasis was clearly on the distribution of the geological units to be encountered by the dredging work, although older rock formations, underlying the Quaternary deposits (mainly weathered sandstones) were not described in the geological overview.

5.1.3 *Geotechnical interpretation in the site investigation report*

The following tests were performed during the site investigation of 1984:

- On soils in the field: particle size distribution, in situ bulk density (wet & dry), moisture content, Atterberg plasticity index
- On soil samples in the laboratory: shear strength, unconfined compressive strength, carbonate content, organic content
- On rocks in the field: Point Load index (number of tests: 94), Protodyakonov number

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[28] The chainage of the channel has been described by km distance from the harbour to the north. A chainage of 17 km, implies 17 km along the channel axis to the northwest, Figure 5.2.

Table 5.3 Geotechnical model descriptions of material to be dredged at Port Hedland. (The data on strength properties have been added, see text).

CLASS	DESCRIPTION		Indicative Unconfined Compressive Strength (MPa)
A	UNCONSOLIDATED GEOLOGICAL SEDIMENTS		Not applicable
	Unit1	Clayey SILT, recent mud deposits	Not applicable
	Unit 2	Organic sandy, silty CLAY (Mangrove Mud)	Not applicable
	Unit 3	Carbonate MUD, grey to brown and cream	Not applicable
	Unit 5	Sandy GRAVEL -gravelly SAND - (old beaches)	Not applicable
B	CONSOLIDATED GEOLOGICAL SEDIMENTS		UCS: Range up to 10 MPa
	Unit 4	Carbonate SAND and GRAVEL (old channel deposits)	Not applicable
	Unit 7	Red clayey SAND to sandy CLAY with calcarenite gravel and cobbles	UCS: 0.17 ± 0.09 (10) PLS: 0.02 ± 0.01 (9)
	Unit 8	Gravelly sandy CLAY (calcrete clay)	UCS: 0.20 ± 0.10 (5) PLS: -
C	CEMENTED SEDIMENTS		UCS: Range up to 40 MPa
	Unit 6a	CALCARENITE, cemented and leached dune and beach deposits (Pt Hedland area)	UCS: 6.7 ± 2.4 (3) PLS: 0.55 ± 0.31 (20)
	Unit 6b	CALCARENITE cemented sands, beach ridges, reef deposits, moderately leached Outer Channel	UCS: 2.7 ± 0.9 (10) PLS: 0.21 ± 0.11 (50)
	Unit 6c	CALCARENITE variably cemented upper level band, middle channel	UCS: 0.6 (1) PLS: -
	Unit 6d	CALCARENITE SANDSTONE variably cemented lower band, middle channel	UCS: 0.7 ± 0.35 (6) PLS: 0.16 ± 0.17 (19)
	Unit 9	CALCIRUDITE CONGLOMERATE - SANDSTONE - SILTSTONE, variably cemented Unit 7	UCS: 0.6 ± 0.25 (3) PLS: 0.21 ± 0.22 (20)
	Unit 10	Calcareous and siliceous CONGLOMERATE Inner Harbour	UCS: 9.8 (1) PLS: 0.33 ± 0.24 (18)
	Unit 11	Leached CALCISILTITE capstone, Inner Harbour	no data

UCS: unconfined compressive strength. PLS: point load strength. Data are given as: average ± standard deviation (number of tests)

- On rocks in the laboratory: wet and dry density, bulk density (38 tests), unconfined compressive strength (41 tests), secant modulus of elasticity (41 tests), point load index (15 tests), sonic velocity (39 tests), mineralogy[29] (1 test)

The amount of samples tested is low. The geotechnical data was grouped into 11 units (Table 5.3). These were distinguished on the basis of the borehole information. The units were described in the report with remarks on where the units could be found in the dredged area. No geological connotation in the sense of stratigraphic position was given. The consultant engineer decided to subdivide the 11 geological units distinguished into three geotechnical classes (Table 5.3):

[29] One sample was examined on the nature of the cement (which was palygorskite clay).

A Unconsolidated geological sediments (implying non-lithified soils)
B Consolidated geological sediments (implying cemented soils/rocks)
C Cemented sediments (implying rocks)

As can be seen from Table 5.3 the division distinguishes soils (A) from weak to moderately weak (B) and moderately strong (C) rocks. Using the classification the consultant estimated the amount of materials of the A, B & C classes to be dredged.

5.1.4 *Tendering for the dredging project*

In the few weeks that were available for tendering, the competing contractors, on the basis of the site investigation report (and additional information provided by the consultant engineer on weather and water conditions, legal aspects etc.) had to prepare their bids. As can be deduced from the description given of the site investigation, the geotechnical model was not completely clear and the geophysical survey had not been successful in delineating the different units distinguished based on the borehole records. The contractors could, however, inspect the cores and the borehole records, which included also information on discontinuities, like RQD[30]. The bidding contractors, if not stating otherwise, accepted the report as containing sufficient information to prepare and execute their work. As stated earlier, the site investigation report was of a quality considered normal for dredging contracts. As usual the contract had the provision included in *clause 12*[31], stating that when physical conditions would occur that could not be foreseen by an experienced contractor, the extra costs would be paid by the employer, if the consulting engineer agrees with the claim of the contractor. This provision protects the contractor from surprises or gross errors in the interpretation of the ground conditions.

[30] Rock Quality Designation attempts to quantify the degree of natural fracturing of cored boreholes. It is the total length of intact cores with axial length > 100 mm divided by the total length of the section considered (%).

[31] Most contracts follow the Conditions of Contract (International) for works of Civil Engineering Construction, prepared by the FIDIC (Fédération Internationale des Ingénieurs-Conseils) and FIEC (Fédération Europeenne de la Construction) approved by many international construction associations. Sub-clause 1 of Clause 12 runs as follows: "If during the execution of the Site Operations the Contractor shall encounter physical conditions (other than weather conditions or conditions due to weather conditions) or artificial obstructions which conditions or obstructions he considers could not reasonably have been foreseen by an experienced contractor and the Contractor is of the opinion that additional cost will be incurred which would not have been incurred if the physical conditions or artificial obstructions had not been encountered he shall if he intends to make any claim for additional payment give notice to the Engineer pursuant to Clause 52(4) and shall specify in such notice the physical conditions and/or artificial obstructions encountered and with the notice if practicable or as soon as possible thereafter give details of the anticipated effects thereof the measures he is taking or proposing to take and the extend of the anticipated delay in or interference with the execution of the Investigation." (From the ICE -Institution of Civil Engineers, London- Conditions of Contract for Ground Investigation, Cottington & Akenhead 1984, p.97, 129). The remaining sub-clauses 3-4 provide for arrangements to be taken by the Engineer and the Contractor and, if the grounds for the claim are accepted, it states that the contractor is entitled to a reasonable compensation for the costs of overcoming the problems, and also the delay and disruption costs.

The winning contractor, after studying the information provided by the consultant engineer, decided to use the new cutter suction dredger *Castor* (11.691 Kw) to dredge an estimated 7,100,000 m^3 of more difficult material and the trailer hopper dredge Humber River to handle the remaining 5,300,000 m^3 softer material. In addition the Humber River would rehandle a further 3,900,000 m^3 of material dredged by the Castor. The planned working time for the dredging was 13 months.

The planning was probably largely based on the strength estimates of the different materials to be dredged (Table 5.3). The contractor had just completed a dredging project nearby, at Cape Lambert, using the Humber River in "coastal limestones" suggested by the consultant engineer to be similar to the cemented calcarenite shore deposits of Port Hedland. At Cape Lambert the rocks were even somewhat stronger (UCS up to 15 MPa), but could be dredged by the trailer hopper dredger Humber River.

5.2 EXCESSIVE WEAR AT PORT HEDLAND

Directly from the start of the dredging work extreme wear was experienced on the dredging equipment. Despite the fact that the rocks being dredged were not strong, the abrasiveness of the materials dredged affected both dredgers. Wear problems occurred everywhere and were not directly related to particular sites. Tooth wear on the Castor was nearly four times as high as expected, rates of pipe wear were 5 times as high and also pumps and impellers wore out exceptionally rapidly. Also on the Humber River wear was extremely high. Wear rates were 2 to 6 times greater than those it experienced at Cape Lambert. The pump casings, suction and discharge pipes of the trailer suction hopper were completely renewed twice during the Port Hedland work. It appeared that two factors contributed to the wear of the dredging equipment:

- the presence of quartz, particularly large angular crystals, in almost all materials being dredged
- the presence of stiff clays forming *clay balls*, armoured with quartz, during hydraulic transport

Due to the presence of the clay balls, the dredged material had to be pumped though the pipes at higher velocities, which aggravated the erosion rate (Figure 4.10). Another effect worsening pipelife was the decrease of production rate due to difficult cutting. Thereby the concentration of solids in the slurry would decrease, which is known to increase erosive wear per volume of solids dredged, Figure 5.3 (De Bree et al. 1982, Mens & de Gee 1986).

5.2.1 *Reappraisal of the engineering geological conditions at Port Hedland*

The contractor decided to have a reappraisal study performed on the geotechnical information contained in the site investigation report, which was carried out by another consulting geotechnical engineer and the advice of experts in engineering geology, geophysics and mining engineering was obtained. It was apparent that this was a typical case where *clause 12*, unforeseen physical ground conditions, would

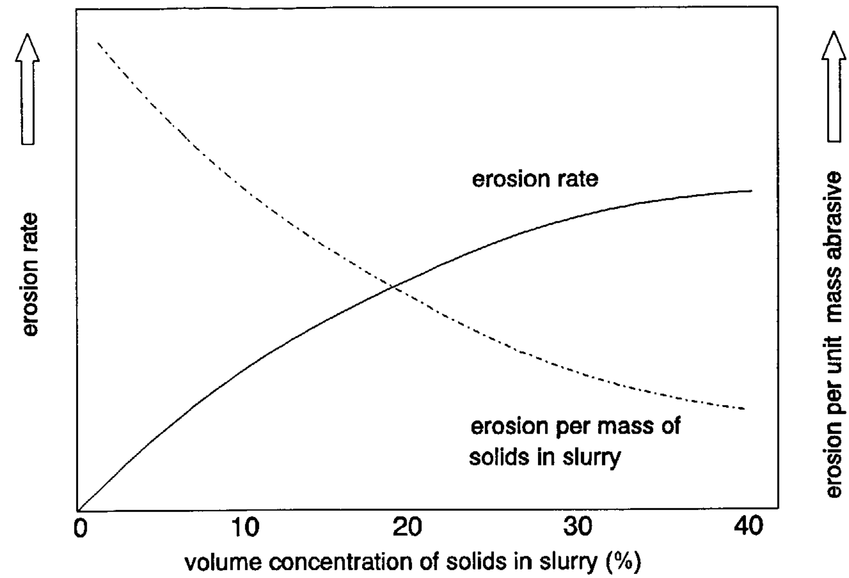

Figure 5.3 Erosion rate and erosion per mass of abrasive of pump housing or pipes change with concentration of the solids in the slurry (after Mens & de Gee 1986).

apply. To support the claim and to explain the cause of the extreme wear the following work was done:

- a study of the available data contained in site investigation report
- logging of the samples and cores from the 69 boreholes
- an additional petrographic study of core samples
- a site inspection involving outcrop studies in the surroundings of Port Hedland to obtain a picture of the regional geology, visiting reclamation areas to study materials dredged
- a geological analysis and development of a consistent model, using the results of an improved interpretation of the geophysical data

The picture that emerged from the reappraisal study was that in the earlier site investigation for this project the importance of quartz as an abrasive mineral was not recognized. The emphasis was on rock strength as being the decisive factor in rock dredging. The classification used to describe the rock mainly emphasized the strength of the rock, by focusing on the carbonate content, calcium carbonate being the cementing agent (Table 5.1). The way the geology was described in the descriptions accompanying the available 1:250.000 and 1:50.000 geological maps also did not emphasize the presence of quartz. The mineral name *Quartz* was hardly ever mentioned in these descriptions. The coastal limestones were often referred to as having a *sand* component, implying quartz from the granitic hinterland, but the descriptions were not explicit about this. In the site investigation report, when this sand component was discussed, the emphasis was unfortunately on grains of *lime* (calcium carbonate) being present.

The adaption of the established rock engineering classification for quartz-bearing weak limestones by Clark & Walker (1977) was unfortunate. In fact, during the investigations made to support the claim for the contractor, it was found that the presence of quartz, also in the coastal limestones, was obvious and readily recognizable in outcrops, hand specimens and rocks from the reclamation area. Had the classification of Clark & Walker (1977) been used in its unadapted form (Table 5.2), the presence of quartz would have come out in the name *siliceous calcarenites*. It was found from the petrographic study of 17 samples that 19-41 vol.% of quartz was present in rocks that had been described as *calcarenite*.

Another unfortunate point was that no attempt was made to place the site investigation results into a framework that could fit into the local geology. No attempt was made to ascribe the stratigraphic position of the units distinguished in Table 5.3.

5.2.2 *Development of a new geological model*

The reappraisal study, of course, was done knowing the two main causes of the problem: the high quartz content in nearly all the dredged materials, and the formation of quartz armoured clay balls. The clay was present in the weathered rocks that were dredged from *below* the (siliceous) calcarenite rocks.

The newly involved geotechnical consultant set out to do a regional geological survey, inspecting also outcrops of rocks on land. The rock cores were re-logged and the units were this time made on the basis of a stratigraphical model (Table 5.4). The broad division of Upper, Middle and Lower coincides with formations of rocks and soils known from the surrounding area. Cross sections showing the units based on this classification are straightforward and geologically justified (Figure 5.4). It is important to note that some of the rocks within a stratigraphic formation occur both as relatively fresh and weathered. In the latter case the rock is degraded to a soil. In the case of Tertiary sandstones this may be a *residual soil*[32]. As one of the weathering products of such rocks is clay, which can be very irregularly distributed within the residual soil and still is in contact with quartz grains, a potentially abrasive mixture is present. Especially near the present coast line much of the material dredged belongs to the Lower group of partially weathered siliceous rocks. These rocks could be studied in outcrops on land, for example at Table Hill and near Tabba Creek, where they are seen lying on top of the Precambrian (Archaean) granite, which underlies most of the region (Figure 5.2).

As described in Chapter 5.1, the engineering consultant described the regional geology emphasizing the changes of sea level in the recent past, concentrating on the position of coastal limestone (calcarenite). In the reappraisal study the engineering significance of the presence of the granite, although not being dredged, is made clear, Figure 5.2. Fresh granite contains quartz, feldspar and mica. When weathered feldspar and mica alter toward clay and ironhydroxydes. A residual soil of granite would consist of angular quartz crystals in a matrix of clay.

The sedimentary rocks in the area are composed of transported and deposited minerals which are the product of erosion of older rocks in the sediment catchment area. Sediments younger than the granite could therefore contain significant quantities of quartz, which is one of the minerals most resistant to weathering. If quartz is deposited near its source, it is generally angular and badly sorted (broad

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[32] A residual soil is a product of complete weathering of a rock occurring still at the position where it is formed. No transport has taken place.

Table 5.4 Regional stratigraphy of Port Hedland area compared with the grouping of the rock and soil materials found on site.

STRATIGRAPHY		MATERIAL GROUPINGS	
RECENT	Alluvial (land) and marine deposits	UPPER	Recent marine & alluvial sediments
		U1	Clays, silts & silty sands
UNCONFORMITY			
PLEISTOCENE	Coastal carbonate rocks	MIDDLE	Carbonate sediments
		M1	Siliceous calcarenite
		M2	Calcarenite gravel, sand & clay
UNCONFORMITY			
TERTIAIRY-CRETACEOUS	Clastic rocks formed from weathering products of quartz-rich older rocks	LOWER	Siliceous sediments
		L1	Sandy clay / clayey sand (residual sandstone)
		L2	Sandstone / calcareous sandstone
		L3	Quartz conglomerate
		L4	Sandstone breccia
UNCONFORMITY			
ARCHAEAN	Granite (the source of quartz in the overlying rocks)	Not seen on site	

spectrum of grain sizes). The Archaean granite delivers quartz and clay, which will both be present in sedimentary formations close to the source.

Nowadays, if one considers a dredging operation on the coasts of Australia, one of the first things that comes to mind are the sea-level fluctuations during the recent geological history. Sea level changes have been considerable in the Quaternary (past 2,000,000 years), as illustrated for the last 750,000 years by Figure 5.5A (Pillans 1987) and for the past 120,000 years by Figure 5.5B (Chappell 1987), which can be taken as representative for the Australian region as well. From about 18,000 BP[33] a rise in sea level has taken place, approaching present-day level at about 4000 BP. These sea level changes are associated to the past ice ages. Changes in sea level are a relative measure for also vertical movement of the land mass may have occurred. In this respect it is noted that the area around Port Hedland is seismically active. The site investigation report places the area in seismic risk zone 1, with recorded magnitudes of 5.0-5.9 occurring in the period 1959-1974. It is noted that the Quaternary sediments of the Port Hedland area show increasing depositional dip with age and clearly faulting and tilting has taken place since the Pleistocene. However, also in the case of Port Hedland, it is obvious that much of the near shore subsurface must have been dry land relatively recently. The dredging depth for the channel aimed at was around -14 m. A rough estimate would be that from 8000 BP to maybe 120,000 BP the elevation would have been above sea level, assuming the graph of Figure 5.5B also applies to the Port Hedland area. The most important point to be taken from this observation is that it is worth while to take into account the on-land geology near Port Hedland. Outcrops of rocks in the area give information directly applicable to the drowned land surface to be dredged. But also the morphology of the landscape, slope instability and distribution of weathering features could give

[33] years before present

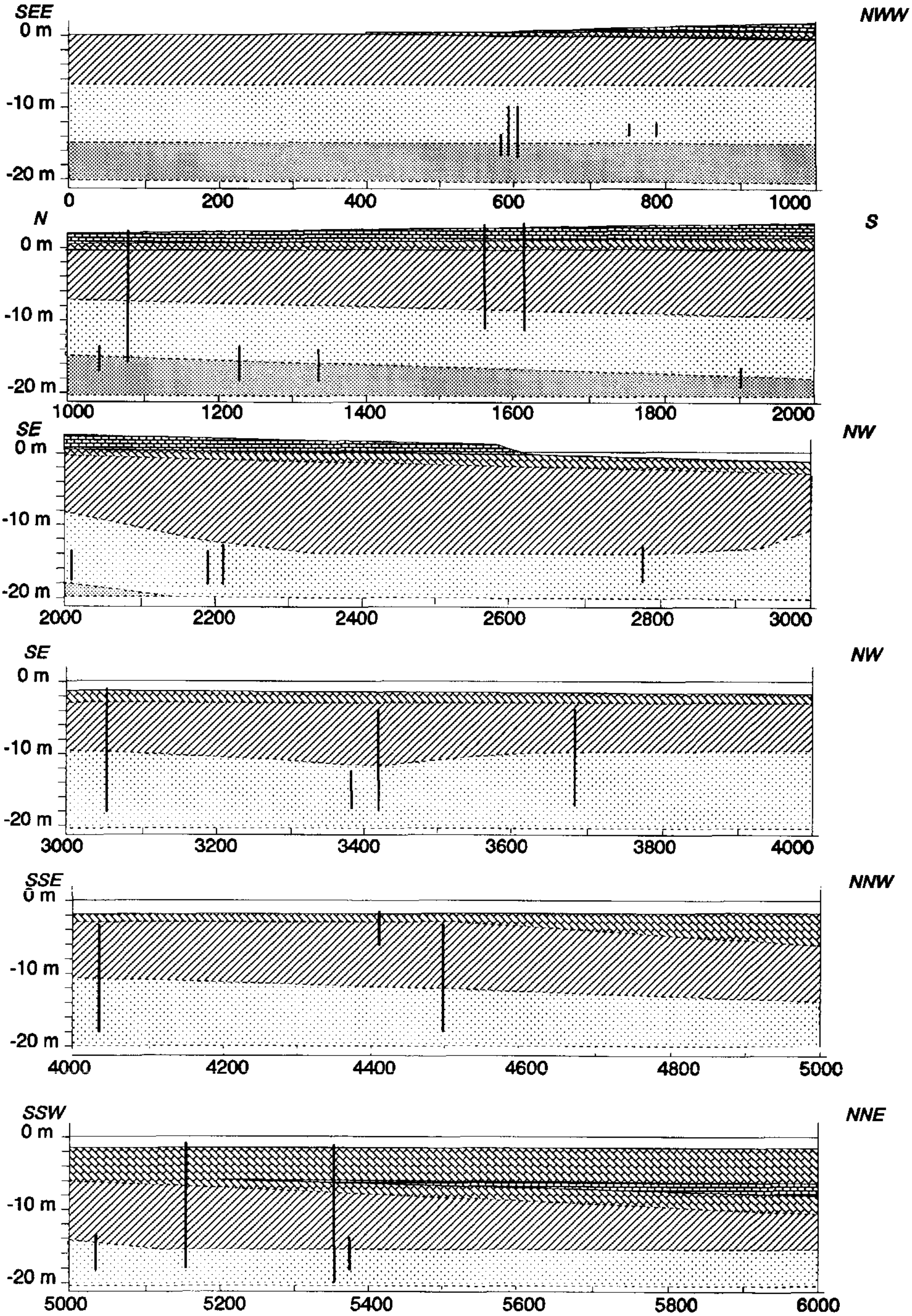

Figure 5.4 A. Interpretative cross section along channel axis Port Hedland, km 0 - 6. For position channel see Figure 5.2. Key to symbols in Figure 5.3B.

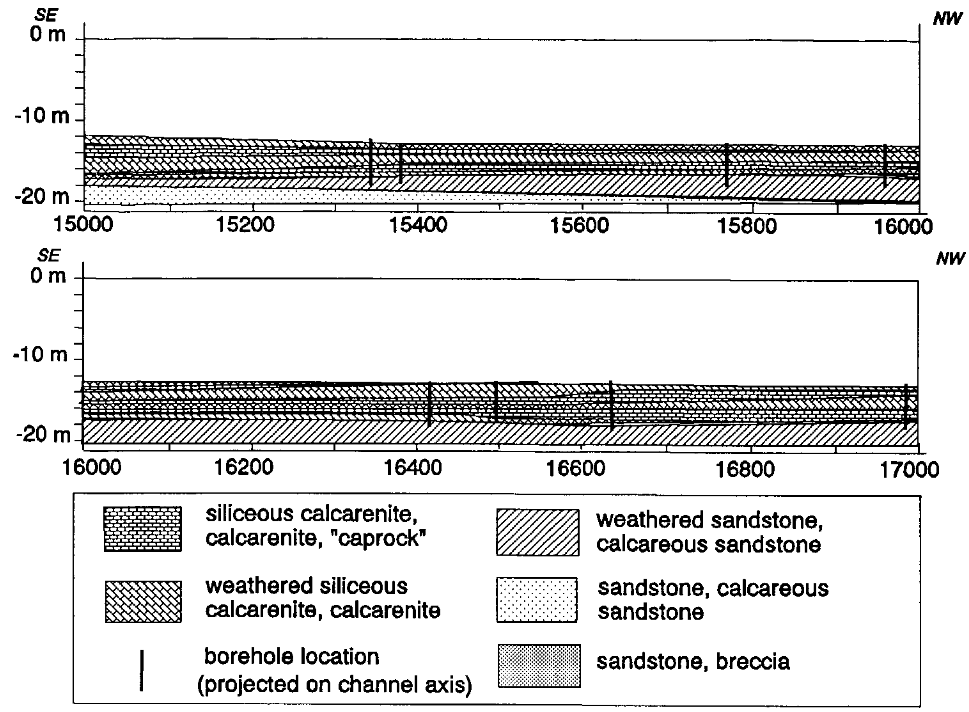

Figure 5.4 B. Interpretative cross section along channel axis Port Hedland, km 15 - 17 (after sections prepared by Coffey & Partners, Sydney, reappraisal study).

information on the drowned land area to be dredged off shore. The relatively recent Quaternary deposits (of Holocene or Pleistocene age), as described by the consultant engineer in Chapter 5.1 give information on the geological shoreline movements, and are related to the sea-level fluctuations given in Figures 5.5.

5.2.3 *Main conclusions of the reappraisal study at Port Hedland*

In many dredging projects the importance of studying the surrounding land geology for a dredging project is not recognized. Attention is usually focused on the off-shore drilling programme to determine as exactly as possible the type and quantity of materials to be dredged. The original site investigation report prepared for the dredging contract of Port Hedland is typical in this respect. No description of outcrops of relevant formations on land is given, although, because of the recent changes in land and sea levels a study of outcrops on land may aid assessment of geological and geomorphological conditions under water.

Apart from the ignorance of quartz as a problem, there was no recognition of the problems that dredging of residual, completely weathered sandstones could give. The presence of completely to highly weathered sandstones, having a high clay content, directly underlying the coastal limestones, aggravated matters considerably. Apart from the transport problems caused by the clay balls, these armoured themselves with angular quartz grains readily available in the slurry and contributed significantly

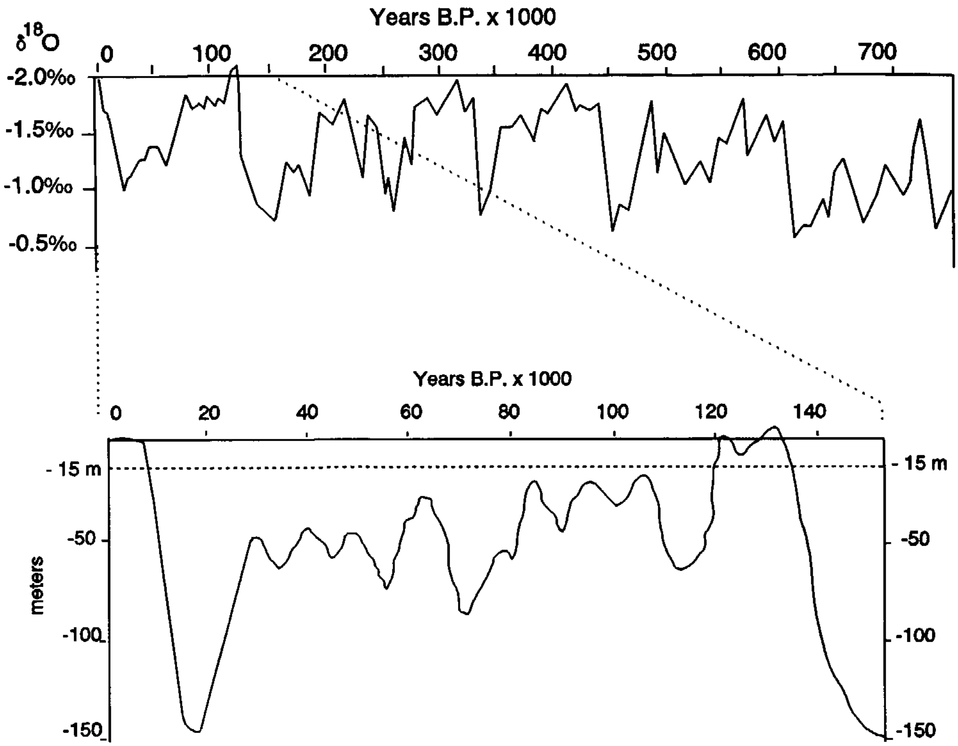

Figure 5.5 A. $\delta^{18}O$ isotope fluctuations indicate sealevel changes, data from Pacific core V28-238 (after Pillans 1987). B. Sea level changes from data of New Guinea coast (Chappell 1987).

to the wear. Probably similar wear problems must have occurred during the earlier dredging projects in the late 1960's and the 1970's. However, no information about extreme wear was ever reported in public. Possibly high wear rates were taken for granted, or considered normal by the dredgers involved. Dredging in rock was still newer at that time than it is today.

5.2.4 Other cases of extreme wear in dredging

Unfortunately the above case is not an exception. Commonly site investigation reports lack vital information on the mineral composition of the materials to be dredged and most of them are prepared without any proper geological background. It is still common to have reports where the presence of quartz is hardly ever mentioned, even in rocks with higher quartz contents than Port Hedland. This type of omission obviously leads to *clause 12* cases, where the contractor can claim some of his losses back. In the Port Hedland case a settlement was made between the contractor and employer.

CHAPTER 6

Conclusions

The following conclusions can be drawn from the survey of problems of wear in rock dredging in the preceding chapters:

1. Wear problems in rock dredging have commonly been approached from a mechanical engineering point of view, through the design of better performing excavation tools and the use of improved wear resistant materials. The science of tribology has made clear that wear processes in general are a result of the particular conditions that are operating. Intensive study of the tribological system, involving microscopic examination of the worn parts, may reveal the wear mechanisms that have operated on the tool. The tribological approach can be of help in the case of dredging, but is hampered by the varying nature of the rock masses being dredged. Even when the rock properties identified as having influence on wear (such as strength, mineral composition (hardness of the abrasive minerals), grain size and shape, rock mass structure) are properly described and measured in a site investigation, it remains problematic to predict the amount of wear on the dredging equipment. A trial excavation would always be needed.

2. Looking at the practice of rock dredging, it becomes clear that even basic information is sometimes lacking in site investigation reports. Often the most common abrasive mineral, quartz, has been ignored. This is probably the most important cause of unexpected wear problems. The resolution of these problems, of course, must be found in improved site investigation practice. For example, just complying with the *classification of soils and rocks to be dredged* of the PIANC (1984; see Appendix A), is not enough.

3. Site investigations should preferably be carried out with the involvement of engineering geologists, which are especially trained to translate geological information to the engineering project at hand. The engineering geologist studying dredging projects should consult geologists familiar with the geology of the site surroundings. The engineering geologist should be aware of the requirements of a dredging operation.

Table 6.1 lists the factors that influence the performance of a cutter suction dredger. The first are geological factors. The intact rock properties and rock mass properties that have influence on the cuttability or excavatability of the rock are mainly the rock mass structure and composition (Figure 4.12) and the material strength. The petrography (mineralogical composition) does not only influence the

Table 6.1 Factors influencing cutter suction dredger performance in rock.

	Factor	Influence on:
GEOLOGY & SUBMARINE GEOMORPHOLOGY	3D distribution of rock and soil units	choice of dredging method and equipment
INTACT ROCK PROPERTIES	Strength	Cuttability Tool wear
	Petrography (abrasive capacity)	Tool wear
	Petrography (clays & other secondary minerals)	Slurry content
ROCK MASS PROPERTIES	Discontinuities (Volumetric density; Orientation; Shear strength)	Cuttability, cut fragment size, production
	Degree of variation in rock (layering, weathering)	Cuttability Tool wear Slurry content
ENVIRONMENT	Water	Tool wear
	Cut slope stability	Production
	Weather; Wave action	Working conditions
CUTTERHEAD	Number of tools	
	Type of tool	Pick point, chisel Tool geometry Composition of steel
	Tool orientation	
CAPACITY	Haulage force & speed	
	Cutterhead power & speed	
	Rigidity of ladder & vessel	
OPERATIONAL CHARACTERISTICS		

cutting behaviour and potential abrasiveness, but also the composition and behaviour of the slurry, for example, when weathered rocks containing clay minerals are present. The geological factors are the subject for further study in this work; the rock properties influencing cutting and wear will be examined in part B (Chapter 7 - 14).

The remaining factors influencing the performance of a cutter suction dredger mentioned in Table 6.1, the environment of operation and the specifications of the CSD itself (cutterhead, capacity and operational characteristics) will not be addressed in this study.

CHAPTER 7

Introduction to part B: Rock properties influencing cutting and wear

This chapter introduces Part B: *Rock properties influencing cutting and wear.* The aim of this study is the improvement of the prediction of the tool consumption of cutter suction dredgers. In Part B the influence of the properties of the rocks on the cutting process are examined. Wear plays a role in this respect, since wearing of the tools causes a reduction in performance.

The rock cutting takes place in an often heterogeneous rock mass, which not only consists of intact rock materials, but also contains fractures (see Figure 4.12). Fractures are known to have a profound influence on excavation production. This effect is examined in Chapter 8. Depending on the fracture density and geometry in the rock mass and the specifications of the mechanical excavation machine, the tool will act either by loosening of rock blocks, or by cutting into intact rock, or a mixture of these processes will occur. Both production and tool consumption will be affected by the way the rock is excavated. The information available on mechanical excavation of rock mass is examined in more detail in Part C: Application of theory to practice, Chapter 17 & 18. Chapter 8 is mainly concerned with principles.

Most wear of the tools is expected to occur when the tool is cutting into intact rock. In Chapter 9 existing rock cutting theories are studied, to see what rock parameters are of importance in the cutting of intact rock. Rock cutting is, from the mechanical point of view, an extremely complicated process. It has been found that near the tip of a cutting tool very high confining pressures are present, which lead to compressive failure and the formation of a crushed zone. The extent of this crushed zone depends, among others, on the brittleness of the rock. Chip formation occurs by the generation of a shear crack from the crushed zone, that bifurcates into a tensile crack which grows towards the free surface. This complex process may only be described using numerical modelling, but important rock engineering parameters can be derived from the model. The more familiar cutting theories are discussed as well. These are based on simplifying assumptions on the cutting process and have limited value, but again illustrate the importance of rock mechanical parameters. Laboratory rock cutting tests have shown the influence of rock strength, geometry of the cutting tool and cutting velocity.

During rock cutting the development of high contact temperatures between tool and rock may lead to a sudden increase in wear of the tools. When examining the

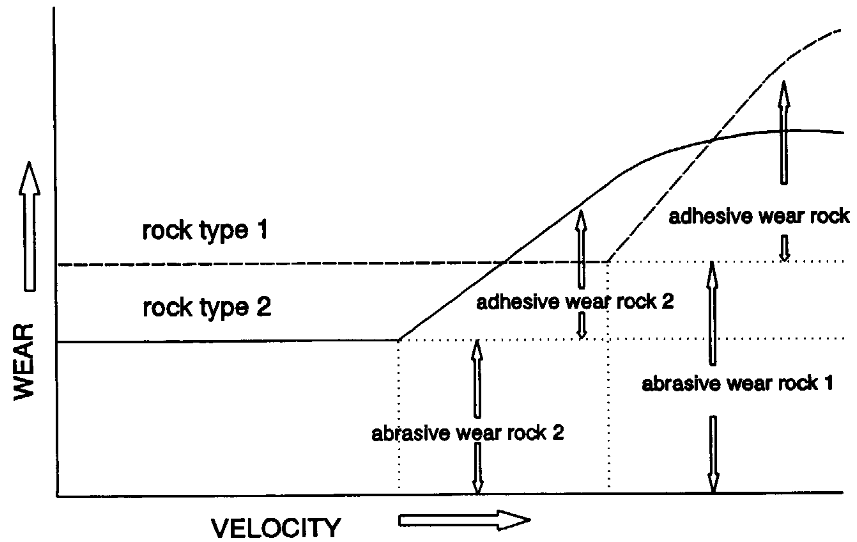

Figure 7.1 Increase of wear ascribed to thermal steel softening, illustrating that the transition to dominant adhesive wear depends on cutting velocity and rock type.

cutting tools, it can be shown that the plastic weakening has been caused by a temperature dependent phase transformation of the steel in a very thin zone. This type of wear has been termed *adhesive wear*. Deketh (1995) assumes that both abrasive and adhesive wear make up the greater part of the total wear of rock cutting tools. Figure 7.1 illustrates that adhesive wear is thought to contribute significantly to the wear at cutting velocities higher than a critical velocity. In the strict sense adhesion is adherence of tool metal against rock material. The wear mechanism is the rupture of adhered junctions. This mechanism in central in the theory of Bowden and Tabor (1950, 1964), see Chapter 4.1.2. Landheer (1983) points out that true adhesive wear is much less common than assumed by Bowden and Tabor. It is also not clear whether true adhesion is occurring when the wear increases above the critical velocity (Figure 7.1). The plastic softening of the steel is definitely the result of high temperatures, however, and the wear flat increases in size due to the rapid removal of this thin plastic layer. The process is mainly temperature controlled and not directly related to the hardness contrast between the tool and rock material.

Part B concentrates on the abrasive wear which is the result of the difference in hardness between the tool and the rock. The mechanisms of microploughing and microcutting of rock particles into the tool material are the basic operations of abrasive wear (Figure 4.7). Chapter 10 examines the hardness of minerals and rock and discusses methods used to determine hardness.

The custom of performing abrasion tests or cutting tests to determine abrasive capacity of rocks is partly due to the fact that, from petrographic and rock strength data alone, only qualitative assessments can be made on amounts of wear to be expected. The numbers derived from abrasion tests instead are supposed to be of more practical use, when production and tool consumption rates of cutting machines are involved. Common tests are discussed in Chapter 11. The work carried out in the context of this study has shown that single test results of standard type abrasion tests are insufficient to describe or predict wear of cutting tools. The *wear mode theory*, treated in Chapter 12, emphasizes that in rocks with abrasive minerals, such as sandstones, most of the wear takes place during the initial penetration of the tool into the rock. Wear is in the high wear mode, two-body abrasive grooving and adhesive wear are common. When the chisel is at depth and cutting, commonly three-body wear takes place at lower wear rates. The transition of the unfavourable to the more favourable wear modes takes place at a certain cutting depth, which depends on the rock properties strength, hardness of the constituent minerals and grain size of the minerals.

The laboratory abrasion tests discussed in Chapter 11 and 12 make clear that wear is a function of both rock strength properties and the mineral composition of the rock. The few correlation equations that exist to date which give information on abrasiveness of rock are treated in Chapter 13. In stead of relying on laboratory tests alone, it is concluded that a combination of rock strength tests and petrographic information should be the basis of an assessment of the potential tool wear that a rock may cause (Chapter 14).

CHAPTER 8

The influence of discontinuities in the rock mass

The larger rock excavating machines, like cutter suction dredgers, will excavate in a *rock mass* (Figure 4.12 & 8.1). In rock mechanics the distinction between *rock material* (German: *Gestein*, Dutch: *gesteente)* and *rock mass* (German: *Fels*, Dutch: *rots)* is made to emphasize an effect of scale that is quite common in rock. With *rock material,* the intact and coherent rock is meant. Larger volumes of rock usually contain fractures, which are termed *discontinuities* in rock mechanics. A discontinuity is a surface in a rock, such as a fracture, joint, fault, shear, weak bedding plane or contact that has zero or relatively low tensile strength. Due to the presence of fractures and other surfaces with low or negligible tensile strength, generally a larger volume of rock has lower strength and stiffness than a smaller volume of the same rock without such discontinuities (e.g. Hoek & Brown 1980, Brady & Brown 1985, Goodman 1989). The frequency of discontinuities in a rock mass can vary, depending on the rock type and its geological history. In some cases rocks can be extremely fragmented, even on hand specimen scale. Although sometimes done, it is not customary to speak of a rock mass in such a case; the *rock material* is then described as being fragmented. In most cases the laboratory samples for strength testing have a diameter of about 55 mm and a length of 110 mm, the size of a hand sample. When studying rock on this scale the term *material* is used. A distinction between *intact* and *broken* material can be made.

If more than one or two sets of discontinuities[34] are present, these tend to divide the rock mass into a system of blocks. The problem of excavation is then examined from the perspective of size and shape of these rock blocks. Figure 8.1 gives a diagram showing the interaction of the rock mass with a cutter suction dredger. Apart from the properties of the *intact* rock material, the properties of the rock mass are determined by fracture frequency and fracture orientation. If fracture spacing is large and the rock mass forms huge blocks, the strength of the mass is relatively high and one tends to speak of a *high quality* rock mass. If the frequency of fractures is high, normally the quality is considered low. Such rock masses commonly give problems in construction, like slope instability or tunnel face instability. From the

[34] A set is a series of parallel discontinuities within a rock mass.

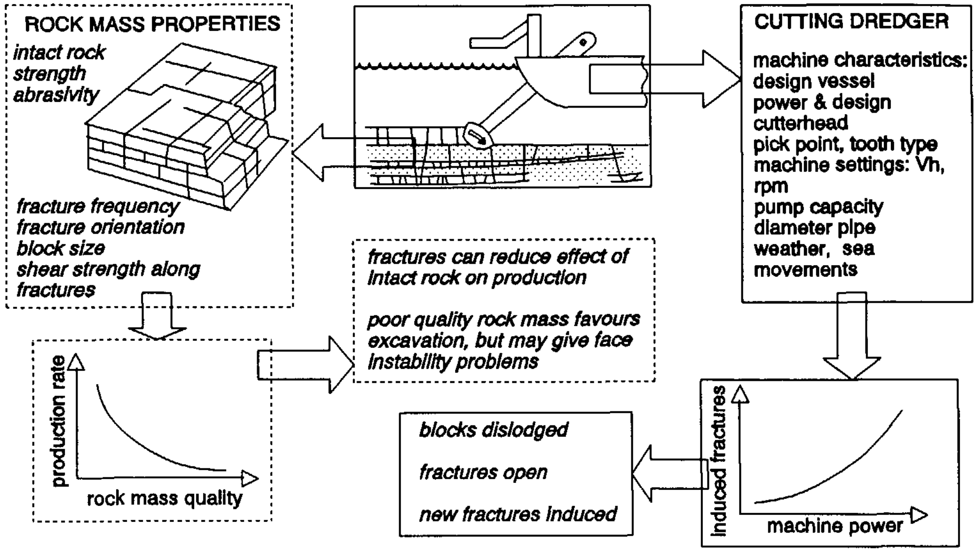

Figure 8.1 Interactions between rock mass properties and cutting performance for a rock cutting dredger (modified after Fowell & Johnson 1991).

point of view of excavation, the fragmentation of the rock by naturally present discontinuities favours excavation, provided the excavating machine or cutting tool is not threatened by the failing rock mass.

Rock mass engineering is well developed in engineering geology, for more information is referred to recent text books on the subject (e.g. Hoek & Brown 1980, Bell 1992).

With regard to rock cutting by machines, the main point of attention is whether the machine:

- excavates the rock by loosening and transporting the rock blocks bounded by the natural discontinuities (a process called *ripping* in this work)
- is cutting into the rock material (called *cutting* sensu stricto)
- excavates by a mixture of both ripping and cutting mechanisms. The properties of discontinuities play a decisive role in this respect.

8.1 TYPES OF DISCONTINUITIES

On the basis of their geological origin, the following types of discontinuities are distinguished:[35]

- layering or bedding (weak surfaces separating sedimentary bedding planes)
- joints (tensile fractures occurring systematically in a rock mass)
- shear zones (a zone of shear fractures, often locally transecting a rock mass)
- faults (irregular occurring fractures in rock).

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[35] Consult text books on geology or structural geology for examples of layering, faults, shear zones and joints. For example: Hobbs, Means & Williams (1976): An outline of structural geology (Wiley, New York).

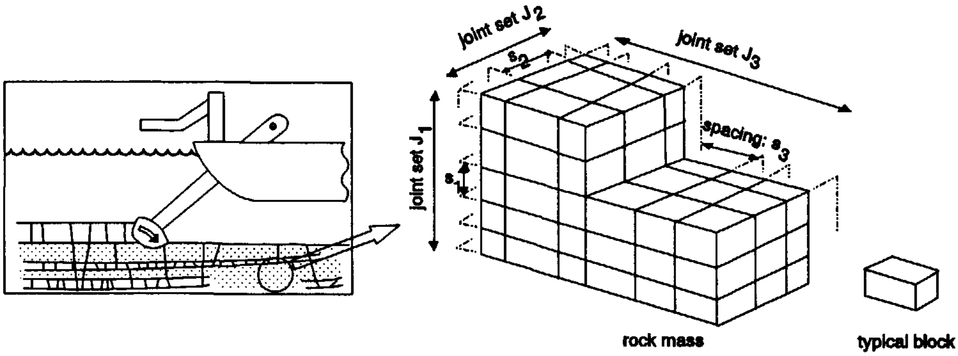

Figure 8.2 Illustration of a rock mass having a discontinuity system of three joint sets.

Discontinuity is the general term used for a significant break or fracture of negligible tensile strength in a rock. It makes no distinctions concerning the age or mode of origin of geometry of the feature (Priest 1993). We can distinguish *singular* discontinuities and *systematic* discontinuities. The systematic discontinuities can be described as one or more *sets* that describe a *system* or *network.* A set consists of a regular assemblage of discontinuities that have a common orientation (are more or less parallel). Examples of natural discontinuities that are commonly singular (are a local event) are faults and shear zones. Examples of discontinuities that occur in sets are layering or bedding discontinuities and joints. *Joints* are tensile fractures that are very common in rocks. They normally occur in a system of two or more sets. Figure 8.2 illustrates the concept of a rock mass with a system of three joint sets. In rock mechanics the term *joint* is very often used loosely synonymous with discontinuity, although joint is a precisely defined term in geology. Joints are the most common discontinuities in a rock mass.

8.2 EFFECT OF DISCONTINUITY PROPERTIES ON MACHINE EXCAVATION

8.2.1 *Discontinuity spacing*

If the spacing[36] of the discontinuities in a rock mass (Figure 8.2) is of the order of the size of the excavating tool (say a cutterhead) or less, their main effect is to reduce the cutting forces (Braybrooke 1988). This effect has been found in studies on the performance of tunnelling machines, like roadheaders or full face tunnel boring machines (TBM's). The influence of discontinuities on the excavation process is well illustrated by research done at the Norwegian Institute of Technology, Trondheim, Norway on the performance of TBM's cutting in strong rock (Movinkel & Johannessen 1986, Bruland et al. 1988). For example, for a particular 4.5 m diameter tunnel boring machine, drilling with specified thrust and rotation velocity on the cutterhead, when only the joint spacing is changed from 1.6 m to 0.2 m

[36] The *spacing* of a discontinuity set is the (average) distance between the individual discontinuities. *Frequency* is the inverse of spacing; the number of discontinuities per meter.

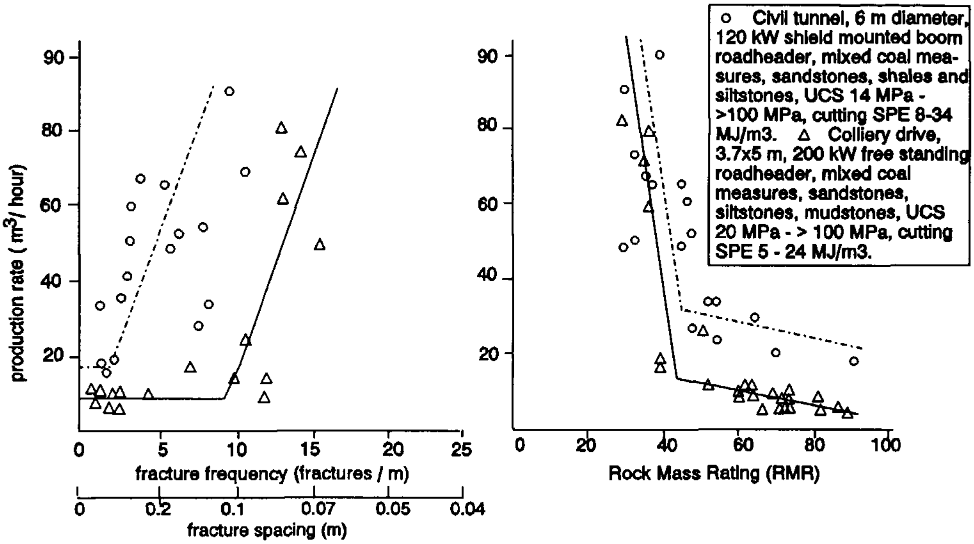

Figure 8.3 Production of two heavy duty roadheaders in comparable rock types, plotted against fracture frequency and RMR (data from Fowell & Johnson 1991).

(orientation is constant), the advance rate of the machine is expected to be twice as high in rocks with similar drillability (Movinkel & Johannessen 1986). It is thus important to find out what the critical spacing for fractures will be.

Some types of road headers (boom tunnelling machines or partial face tunnelling machines) also operate with a cutterhead. Fowell & Johnson (1991) mention that spacings of less than 0.3 m (i.e. frequency: > 3.3 fractures per meter) normally were required to make the excavation rates independent of intact rock properties for the cases they examined. Figure 8.3 is a plot made of the data presented by Fowell & Johnson (1991). Figure 8.3 gives data on the performance of two different heavy duty roadheaders, one operating on a civil tunnel project, the other working in a coal mine. A roadheader is equipped with a cutterhead on an arm, having therefore some similarity with a cutter suction dredger. Examining the left hand diagram of Figure 8.3, the difference in performance of the two roadheaders can be seen. It is suggested that the production rates linearly increase with fracture frequency, after a critical fracture spacing is reached. If spacings are wider, the excavation rates are low and mainly cutting in rock material takes place. The right hand diagram shows the production against the *Rock Mass Rating* (RMR). This rating system will be discussed in Chapter 8.3, but here suffices that the RMR is an attempt to rate the quality of the rock mass by combining information on the discontinuity properties and the intact rock properties. Interestingly, the RMR numbers indicate that below a value of about 45 production rates increase for both machines. In the RMR system this is on the boundary of fair to poor rock.

Gehring (1989, cited in Natau et al. 1991) states that spacing needs to be less than 0.15 m (frequency: > 6.7) to have effect for the road header tunnelling machines he has been studying.

Braybrooke (1988) discusses the results of a research project of the British National Coal Board carried out in the late 1970's on the cutting performance of road headers. There it was also found that in essentially homogeneous rocks, with

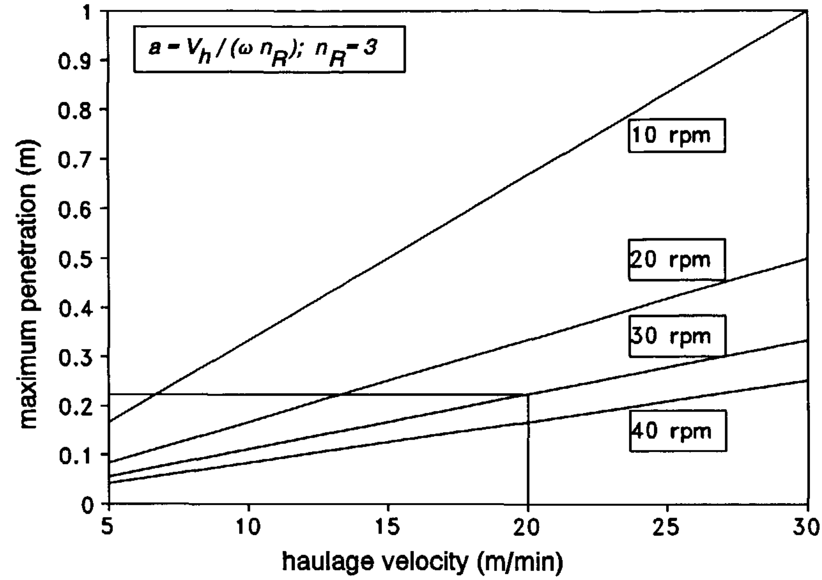

Figure 8.4 Maximum penetration for a range of haulage velocities and rotation velocities of cutting tools (Equation 3.4).

joints spaced greater than about 0.15 m, the excavation rate depended on the properties of the intact rock material (Aleman 1983).

Also from rock dredging practice the effect of discontinuities is known, but no data are available. A feel for the effect of discontinuity spacing can be obtained from the thickness of the cutting slices that are made in the rock mass by the cutting tool (i.e. the teeth of the cutterhead). Normally the cutter suction dredger is operated at around 30 rpm cutterhead rotation speed and in stronger rock a swing speed (haulage velocity) of 15 m/min (0.25 m/s) is common; in weak rock V_h may be 20 to 30 m/min (0.3-0.5 m/s). Using Equation 3.4, and assuming a staggered cutting tooth position with $n_R=3$, Figure 8.4 gives an indication of the maximum penetrations that are obtained for each cutting tooth (chisel or pick point). In this example, when the machine settings are 30 rpm and 20 m/min, discontinuities influence cutting when the spacing is less than 0.2 m. Figure 8.4 illustrates that in those cases where the rock has discontinuities with a wider spacing than the penetration depth, the cutting teeth will cut into intact rock.

The effect of discontinuity spacing probably operates on two scales, one related to the penetration depth of each cutting tooth and one on the scale of the size of the cutterhead. If the spacing is wider than the maximum penetration of the pick point, but smaller than the effective size of the cutterhead, excavation of the rock is assisted by easier loosening along the discontinuity surface, provided the orientation of the discontinuity surface is favourable. The pick points excavate for a major part in intact rock. If the spacing is wider than the size of the cutterhead, most pick points excavate in intact rock. If the spacing is smaller than the maximum penetration depth, the chance is high that the pick points are not cutting intact rock, but are loosening the fragmented rock mass. In such a case, even in abrasive rock, much less wear than would normally be expected occurs (Fowell & Johnson 1991).

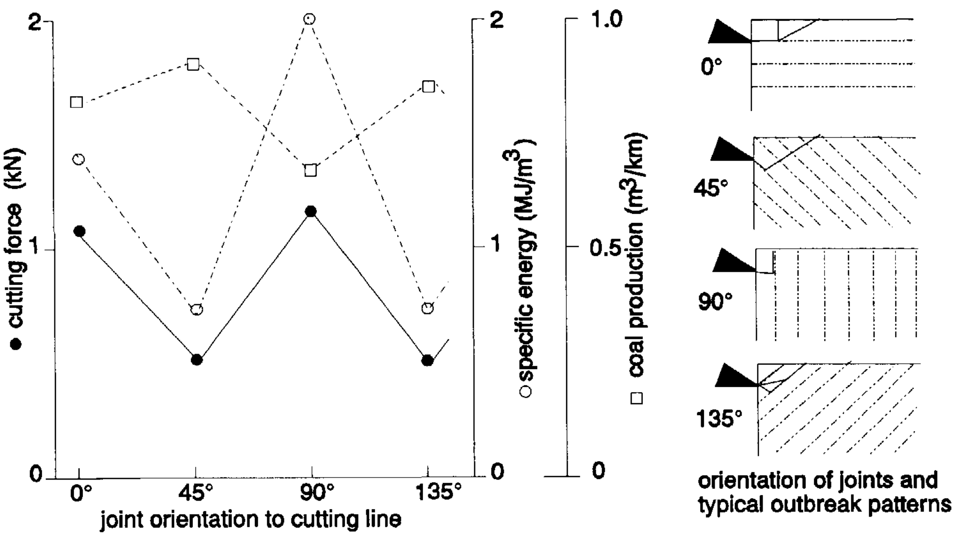

Figure 8.5 Effect of joint orientation on performance per pick point of coal mining machine (data from Roxborough & Sen 1986). Outbreak patterns after Evans & Pomeroy (1966).

Summarizing, the effect of the spacing of discontinuities in a rock mass can be evaluated as follows:

- when the spacing is larger than the size of the cutterhead, the cuttability of the rock is largely dependent on the properties of the intact rock material; *cutting* will be the main excavation mechanism.

- when the spacing is smaller than the size of the cutterhead, it should be examined whether the penetration depth of the individual cutting tooth (pick point) can reach the spacing of the discontinuities, to take full advantage and induce a *ripping* excavation mechanism.

Considerations like this can influence the choice of the type of dredger for a project.

8.2.2 *Discontinuity orientation*

The orientation of the discontinuities with respect to the cutting direction is known to have an important control on cutting performance. In coal mining much work on the effect of joints on the performance of mining machines has been done. Figure 8.5 illustrates the main features that are known. If narrow spaced joints are within the influence of a cutting tooth the outbreak pattern and consequently the production are clearly influenced.

Hoogerbrugge (1980) performed cutting experiments with a model wheel bottom cutter dredge in the testing tank of the Laboratory of Soil Movement (Mechanical & Maritime Engineering, Delft University). Four test blocks (1.5x1.0x0.25 m) with discontinuities spaced 0.07 m were prepared. The diameter of the model wheel cutter was 0.5 m, the width 0.17 m. The cutting depth of the wheel into the test blocks was varied between 0.10 and 0.15 m. The rotation rate of the wheel was 60 rpm and the haulage velocity varied between 0.6 and 1.5 m/min (0.01-0.025 m/s). No

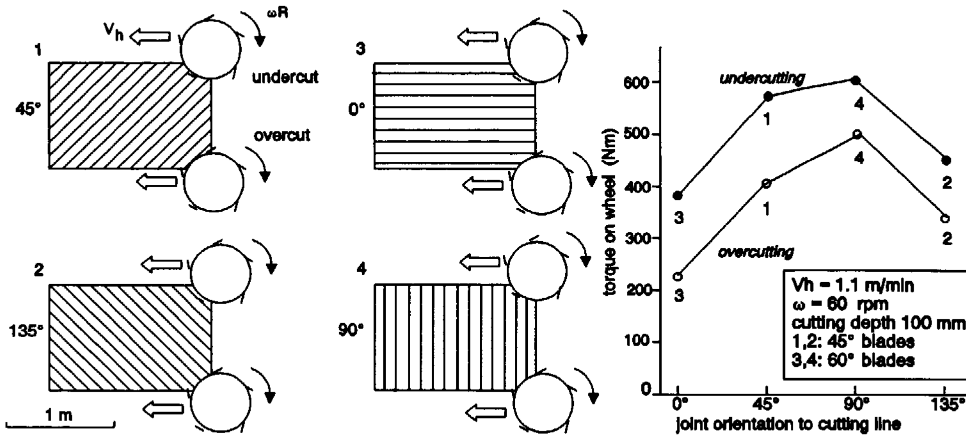

Figure 8.6 Results of cutting experiments by Hoogerbrugge (1980) on jointed mortar blocks (spacing 0.07 m) with a wheel cutter.

information on tooth position or number of teeth is given, but more than one tooth (judging from photo's $n_R=3$) occupies an identical position. The maximum penetration of each tooth from Equation 3.4 is then 0.025 m ($n_R=1$, $V_h=0.025$ m/s, 60 rpm). The experiments are therefore an example of the case where the spacing of the discontinuities is wider than the maximum penetration depth of the cutting teeth, but smaller than the size of the cutterhead. Hoogerbrugge's experiments suffered some drawbacks, due to wrong placement of the teeth in the first two experiments, with resulting excessive wear. Experiments 3 & 4 had to be performed with a cutter wheel with 60° blades in stead of 45°. This problem has effected the magnitudes of the forces measured. But the results given in Figure 8.6 do show an influence of joint orientation on cutting performance, with the 90° orientation being the least favourable. The experiments also demonstrate that the *overcutting mode*, where the teeth move initially at right angles to the free surface, is advantageous. As explained in Chapter 3.2.1, in stronger rock the cutterhead of a cutter suction dredger bounces off the surface of the rock during the back swing (overcutting mode), because not enough thrust force vertically downwards can be delivered. Therefore the main cutting production with a cutter suction dredger is delivered in the undercutting mode.

The results of the experiments on coal resemble those on wedge cutting experiments in anisotropic (transversely isotropic) intact rock (Pariseau 1970). The experiments on closely jointed coal, shown in Figure 8.5, have the lowest cutting forces in the 45° and 135° orientation, while in the experiments of Hoogerbrugge this was not found. One notable difference between the tests is that the spacing of the joints in Hoogerbrugge's experiments was larger than the penetration depth of the cutting teeth. The most important difference, however, is that the coal cutting experiments were linear, while the wheel cutting tests describe a cycloidal cutting path through the discontinuous rock (Figure 8.7). This makes the wheel cutting tests

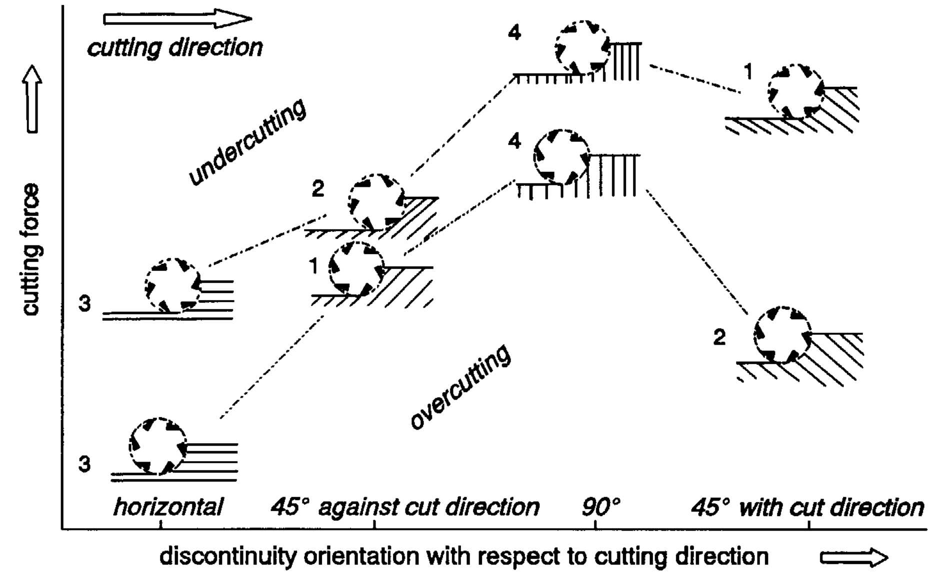

Figure 8.7 Tentative diagram on the effect of discontinuity orientation on the cutting forces of a wheel cutter. The numbers correspond to the experiments of Figure 8.6.

unique. In order to obtain a better understanding of the cycloidal cutting into jointed rock, more experimental work and modelling is needed.[37]

8.2.3 *Integral discontinuities*

Discontinuities are defined as surfaces in the rock mass with low or negligible tensile strength. In anisotropic rock (for example layered sedimentary rocks), the layering or lamination surfaces present might not have zero tensile strength, but such surfaces may well be significantly weaker than the surrounding rock. Price (1991) has termed such planes *integral discontinuities*. Although forming part of the integral rock material, during loading these surfaces might well develop into mechanical discontinuity surfaces. Therefore anisotropic intact rock should be tested for strength normal and parallel to the anisotropy. If weak integral discontinuities are present, their spacing can be compared with the probable penetration depth of the cutter teeth. Relatively small spacings will assist production (Chapter 8.2.1). Similar effects on production with respect to orientation as depicted in Figure 8.5 can be expected.

[37] Laboratory model tests on the ripping of jointed rock, like those of Hornung (1978), are commonly linear.

GEOMECHANICS CLASSIFICATION OF ROCK MASSES

1. Intact rock strength

Point Load Index (MPa)	UCS (MPa)	Rating
>10	>250	15
4-10	100-250	12
2-4	50-100	7
1-2	25-50	4
don't use	5-25	2
don't use	1-5	1
don't use	<1	0

2. Rock Quality Designation

RQD (%)	Rating
90-100	20
75-90	17
50-75	13
25-50	8
<25	3

3. Spacing of most influential joint set

Joint Spacing (m)	Rating
>2.0	20
0.6-2.0	15
0.2-0.6	10
0.06-0.2	8
<0.06	5

4. Joint condition

	Rating		Rating
Very rough surfaces of limited extent; hard wall rock	30	Smooth surfaces, or gouge filling 1-5 mm thick, or aperture of 1-5 mm Joints extend more than several meters	10
Slightly rough surfaces, aperture < 1mm; hard wall rock	25	Open joints filled with > 5mm gouge, or open more than 5mm, joints extend more than several meters	0
Slightly rough surfaces, aperture < 1mm; soft wall rock	20		

5. Groundwater situation

Inflow per 10m tunnel length (L/min)	OR Joint water pressure divided by major pricipal stress	OR General condition	Rating
none	0	completely dry	15
<10	0.0-0.1	damp	10
10-25	0.1-0.2	wet	7
25-125	0.2-0.5	dripping	4
>125	>0.5	flowing	0

A. UNADJUSTED RMR = SUM OF RATINGS 1 TO 5

Figure 8.8A Geomechanics classification of rock masses (Bieniawski 1979, 1989).

8.2.4 *Shear strength of discontinuities and rock mass classification*

The *shear strength* of the discontinuities is usually orders of magnitude lower than the shear strength of the intact rock. But the shear strength of discontinuities contributes to the cutting force as well, when excavating a jointed or fractured rock mass. Factors related to frictional resistance of joints are many, such as for example surface characteristics like roughness or infilling. Discontinuities may not be

6. Orientation of joint sets with respect to engineering work				
Assessment of influence of orientation on work	Rating tunnels	Rating Excavations	Rating Foundations	Rating Slopes
very favourable	0	-12	0	0
favourable	-2	-10	-2	-5
fair	-5	-5	-7	-25
unfavourable	-10	-2	-15	-50
very unfavourable	-12	0	-20	-60

B. ROCK MASS RATING RMR = (SUM OF 1-5) + (ADJUSTMENT 6)

GEOMECHANICS CLASSIFICATION			Estimate of Rock Mass	
Class	Description rock mass	RMR	cohesion (kPa)	friction angle (°)
I	very good rock	81-100	>400	>45
II	good rock	61-80	300-400	35-45
III	fair rock	41-60	200-300	25-35
IV	poor rock	21-40	100-200	15-25
V	very poor rock	0-20	<100	<15

Figure 8.8B Geomechanics classification, continued. Adjustment for excavation follows proposal by Fowell & Johnson (1991).

persistent, they may end in intact rock or have rock bridges increasing their cohesive resistance[38].

Because a rock mass generally has systems of pervading fractures, the problem has to be treated in a generalised, or statistical way. In mining and tunnelling, classification systems have been developed that try to incorporate those characteristics of the rock mass that determine strength. These include intact rock strength, discontinuity density and distribution and surface properties of the discontinuities, like roughness and width. Two systems are in general use nowadays, the Rock Mass Rating (RMR), developed by Bieniawski, and the Q system developed by Barton (Barton, Lien & Lunde 1974, Bieniawski 1979, 1989, Hoek & Brown 1980, Goodman 1989). These systems typically were designed to assess the stability of underground openings, but were later used for other purposes, such as estimation of foundation deformation and strength and assessment of stability of rock slopes. The RMR system is given in Figure 8.8. The use of these systems is now well established in engineering geology. They are of help when describing a rock mass, assuring that important rock features are not overlooked during site investigation. But, they also allow comparison of the rock mass under study with a large empirical data base of rock masses in which all types of engineering works have been executed and monitored. With very simple means a first guess of rock

[38] See Barton & Stephanson (1990) for information on rock discontinuity mechanics.

mass strength and deformability properties may be obtained (Bieniawski 1989, Hoek & Brown 1980).

The *Geomechanics Classification of Rock masses*, commonly known as the Rock Mass Rating System was first proposed by Bieniawski in 1974, but was revised in 1979. The rock mass is classified according to (Figure 8.8):

- the strength of the intact rock material (determined by UCS or PLS tests, see Chapter 21)
- the fracture density and the spacing of the prominent set of discontinuities (determined by RQD and average spacing, see Chapter 22)
- the surface characteristics of the discontinuities
- groundwater conditions

Ratings were given to these factors, based on a large data base of rock mass deformation and strength behaviour in mines and tunnels. The discontinuity orientation is judged with respect to the project of study and ratings can be adjusted accordingly. The reader is referred to Bieniawski (1989) for a description on how to apply the system.

For the purpose of excavation Fowell & Johnson (1991) adjusted their data with the ratings shown in Figure 8.8B. The result is presented in Figure 8.3, p.73. In this example, the RMR values show a more distinct relationship with production than the information on discontinuity spacing alone. There is a threshold value, below which for both machines the production increases significantly. Above this value production rates differ, but still show a distinct relationship with RMR. The difference in production rate at higher RMR can be explained by the difference in type of roadheader used and difference in the lithology of the rock mass (note that no information on rock lithology like quartz content and grain size is included). Apparently when RMR is lower due to high discontinuity density, both machines excavate with similar production rate and intact material properties are less important.

Bieniawski (1989) gives two other interesting examples of roadheader performance that show similar relation with RMR as depicted in Figure 8.3. In both cases significant increase in production occurs at or below an RMR of 40. One of these, from data of Sandbak (1985), is reproduced in Figure 8.9, since also cutting tool consumption is given. Tool consumption increases with RMR, with highest consumption corresponding to the lowest advance rates. Tool consumption, in this example, is apparently dependent on both the rock material and the rock discontinuity properties described by the RMR classification. Regarding tool consumption, it would be interesting to have information on the abrasiveness of the rocks. A disadvantage of classification systems is that information on rock type, which could give some indication of abrasiveness, gets lost.

These examples show that a rock mass classification system may be of help when trying to predict machine excavation performance. May be such systems could be adapted to machine excavation, by including for example information on abrasiveness. This will be further discussed in Chapter 18.

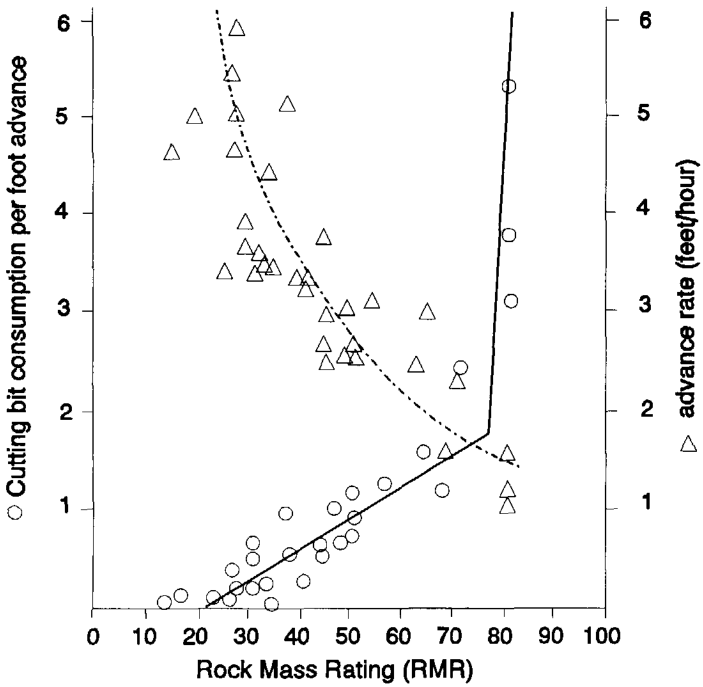

Figure 8.9 Roadheader (Dosco SL-120) performance data in variable rock conditions, San Manuel Copper mine, Arizona. Data from Sandbak (1985), after Bieniawski (1989).

8.2.5 *Summary of the influence of discontinuities*

Spacing and orientation of discontinuities influence the rock cutting process. Spacing should be regarded with respect to the penetration depth of a cutting machine (which is dependent on the thrust the machine is able to provide). If the spacing is near to, or smaller than, the cutting depth of the cutting tooth, the cutting process will be largely influenced by the discontinuity properties, like shear strength and orientation.

If the spacing is larger than the penetration depth of the cutting tooth, but smaller than the size of the cutterhead, discontinuities provide for an anisotropy in the rock. They facilitate the cutting process when the angle is lower than 90° with respect to the excavation direction. The most favourable orientation is when the haulage (direction of movement of the cutterhead) is parallel to the discontinuity orientation (Figure 8.6 & 8.7).

Rock mass classification may be of use when studying the excavatability of a rock mass. This topic will be further examined in Chapter 18.

In many cases penetration depth of the individual cutting teeth will be smaller than average discontinuity spacing. Cutting will be largely through intact rock, examined in the following Chapter 9.

CHAPTER 9

The cutting of intact rock material

Two questions may be asked in relation to the cutting of rock material:

1. What intact rock properties that relate to cutting should be assembled in the site investigation for a rock dredging project?
2. How can these properties be used to predict the production of a cutter suction dredger?

In this chapter a summary of knowledge on the cutting of intact rock is given, from which can be deduced what mechanical properties of intact rock relate to cutting. For the site investigation is important to know which rock mechanics tests should be performed and what laboratory cutting tests would give valuable information. Experimental work on rock cutting is mainly done on samples of intact rock, using linear cutting tests. Most of the present day knowledge on rock cuttability comes from these laboratory tests.

In Chapter 9.1 a phenomenological model of rock cutting is given, based on work carried out by Delft Hydraulics Laboratory. This model is based on rock cutting experiments on weak rocks, specially carried out in relation to rock cutting dredging. It illustrates the relation of rock cutting with rock failure theory and gives an indication of the type of rock mechanics tests that can provide information on cuttability.

Chapter 9.2 gives a discussion of the best known analytical cutting equations for drag cutting tools. Apart from a rock strength factor, these equations take into account changes in tool geometry (rake angle α). Since tool geometry is changing due to wear (see Figure 3.11), an indication of cutting stress changes due to wear is given, by decreasing rake angle α.

In Chapter 9.3 the results of linear cutting tests, examining the effect of cutting depth, spacing of cutting tools and the specific energy of cutting are discussed. The effects of cutting velocity and heat development (Chapter 9.4), cutting under water (Chapter 9.5) and wear of the tools (Chapter 9.6) are reviewed. In these chapters the approach to the problem of rock excavation by machines developed by Roxborough is followed, which is based on the results of numerous rock cutting tests in the laboratory (Roxborough 1973, Roxborough & Phillips 1981, Roxborough & Sen 1986). Results of tests carried out on rocks from the Sydney Harbour Tunnel project (described in Chapter 16) are given in Chapter 9.7. How the test results may be applied to practice is indicated in Chapter 9.8. Conclusions are given in Chapter 9.9.

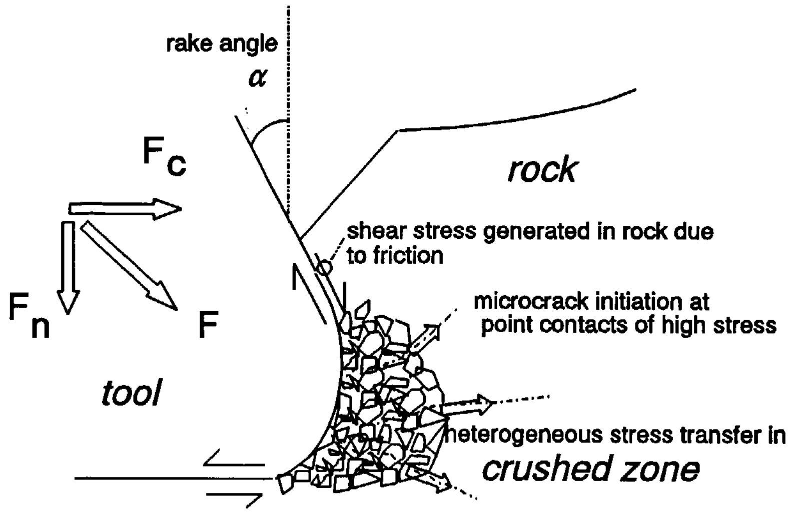

Figure 9.1 Due to friction between tool and rock compressive stresses are generated near the tool tip. F_n=normal force, F_c=cutting force (Fairhurst 1987).

9.1 A MODEL OF ROCK CUTTING[39]

Close observation of rock cutting experiments by high speed film techniques and other methods have shown that near the cutting tip a zone of crushed rock is nearly always present. The importance of the presence of this crushed zone for the chip forming cutting process has been emphasized by many researchers (see Mishnaevsky 1995 for a recent review). Probably up to 90% of the energy for cutting is spent in rock crushing near the cutting tip. Dynamic force measurements during cutting tests and cone penetration tests and analytical and numerical calculations have shown that confining stresses near the tool tip can be very high (Cools 1993, Van Kesteren 1995, Uittenbogaard et al., in prep.). The confining stress commonly exceeds the brittle-ductile transition of the rock, explaining why the rock fails in the cataclastic ductile mode. Depending on the type of rock, the crushed material consists of fragments of rock grains formed by fracturing along or through grain boundaries, which get compacted or ground down further by the tool.

The crushed zone plays an important role in the stress distribution near the cutting tip. Fairhurst (1987) studied the influence of the crushed zone ahead of the cutting tool on the cutting process. High confining stresses near the tool tip result from friction forces between tool and rock. Once a crushed zone is formed in the isostatic compression zone near the tool tip, forces are transmitted via particle to particle contact, forming load bearing ligaments. In this way the tool cutting force is applied

[39] Partly from: Verhoef, P.N.W., J.J. Ockeloen, W.G.M. van Kesteren 1996. The significance of rock ductility for mechanical rock excavation. in *M.Aubertin et al (eds.) Rock Mechanics, Tools and Techniques. Proceedings 2nd North American Rock Mechanics Conference, Montreal, June 1996:* 1, 709-716. Balkema Rotterdam.

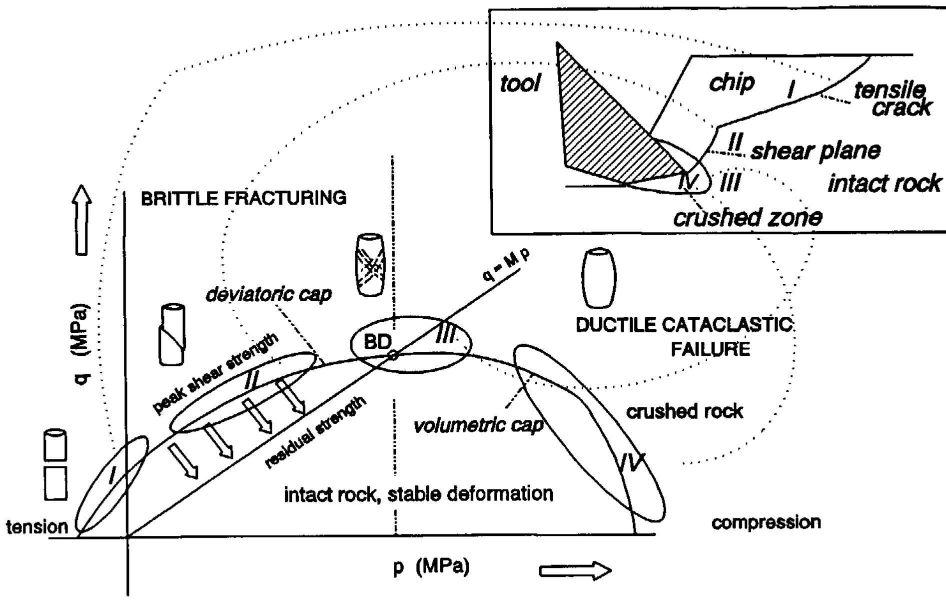

Figure 9.2 Failure during rock cutting involves the entire failure envelope, which is given in a p - q diagram, ($p=(\sigma_1+2\sigma_3)/3$; $q=\sigma_1-\sigma_3$).

to the intact rock as a series of discrete point loads. These point load contacts are important sources for microcrack initiation (Figure 9.1).

The results of a long term research project on rock cutting in the context of dredging (underwater rock cutting), carried out during the past 15 years by Delft Hydraulics Laboratory, have been recently released for publication (Uittenbogaard et al., in prep.). This work shows that the rock cutting process involves failure *along the complete failure envelope of intact rock*. Failure during cutting can best be described using critical state theory (Schofield and Wroth 1968, Farmer 1983), because during failure and formation of the crushed zone volume changes are important (both dilation and compression). Observations by Van Kesteren (1995) on the process of chip formation have been explained in terms of the critical state theory of rock failure (Figure 9.2). In the crushed zone (IV) the stress state is beyond the volumetric cap (*Roscoe surface*). At the boundary of the crushed zone (III) the stress state is on the volumetric cap, near the brittle-ductile transition stress (*BD*). When the cutting proceeds, the stress state remains the same, but the boundary of the crushed zone is moved into the intact rock, proportional to the tool displacement. Just outside the boundary of the crushed zone (II), shear failure occurs at a stress state on the deviatoric cap (*Horslev surface*). In the crushed zone, localised shear zones occur with a concentration near the chisel tip, these may match up with the shear planes outside the crushed zone. Further away in the intact rock, the stress state is below the brittle-ductile transition and isotropic stress decreases. The shear crack in this zone can bifurcate into a tensile crack (I). When after bifurcation unstable crack growth occurs, the chip will be formed.

This model of chip formation is supported by numerical modelling and experimental data (Uittenbogaard et al. in prep.). The effect of pore water pressures is also described, see chapter 9.5.1 (Van Kesteren et al. 1992, Van Kesteren 1995).

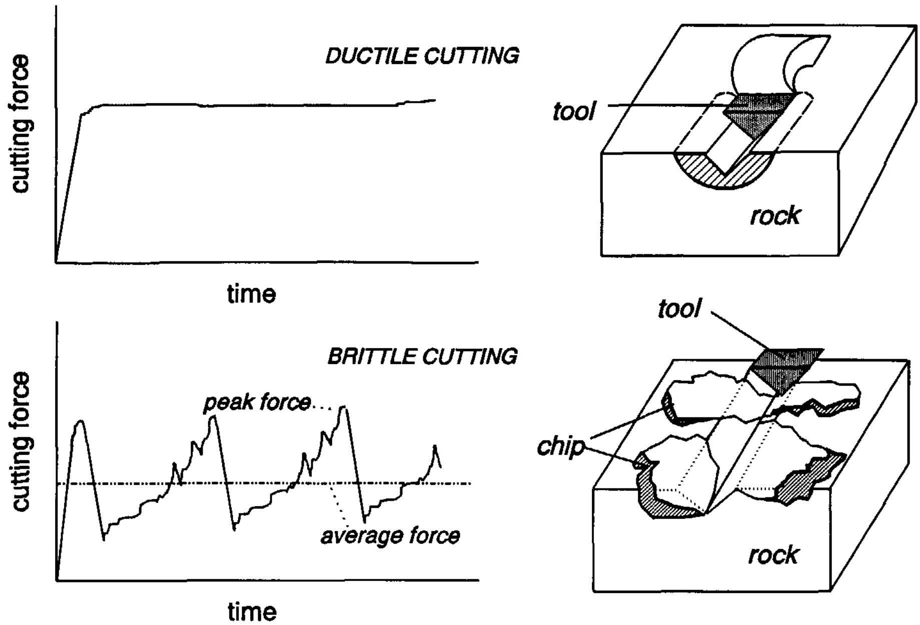

Figure 9.3 Brittle and ductile mode of rock cutting.

The model describes rock chip formation during the cutting of typical *brittle rocks*, i.e. rocks which fail in a brittle manner when tested at room temperature at unconfined conditions (Figure 9.3). The typical saw tooth pattern of curves of cutting force against time or displacement, is a result of the sequence of crushing, tensile chip failure, crushing. This behaviour relates to what is called *brittle failure* in rock mechanics. Some rocks, however, fail not by chip formation bounded by tensile fractures, but by ductile ploughing. The corresponding force-displacement diagram shows no drops in force, but a continuous and constant cutting force level. Such failure is termed *ductile* (Figure 9.3). In terms of the chip formation model, the difference between brittle and ductile cutting may be explained by the size of the crushed zone, which in the ductile case extends to the free rock surface.

The complete failure envelope is again presented in Figure 9.4, but now using the conventional τ - σ Mohr-Coulomb presentation. The closure of the friction envelope on the compression side of the diagram is normally not studied in engineering rock mechanics, but its relevance for the common engineering application of rock cutting is emphasized. The improved understanding of the failure of rock during cutting shows that ductile, transitional and brittle failure may take place simultaneously and the complete failure envelope is dealt with. To what extent each mechanism is operating is a function of rock type (rock properties) and tool type (geometry of chisel, machine settings). In this respect it is important to note that the design of the cutting tool can induce the failure mechanism. By adapting cutter design brittle failure can be promoted. However, with constant design, such as the common pick points used for cutter suction dredgers, each rock type will show its own specific cutting behaviour, depending on its brittleness (or ductility).

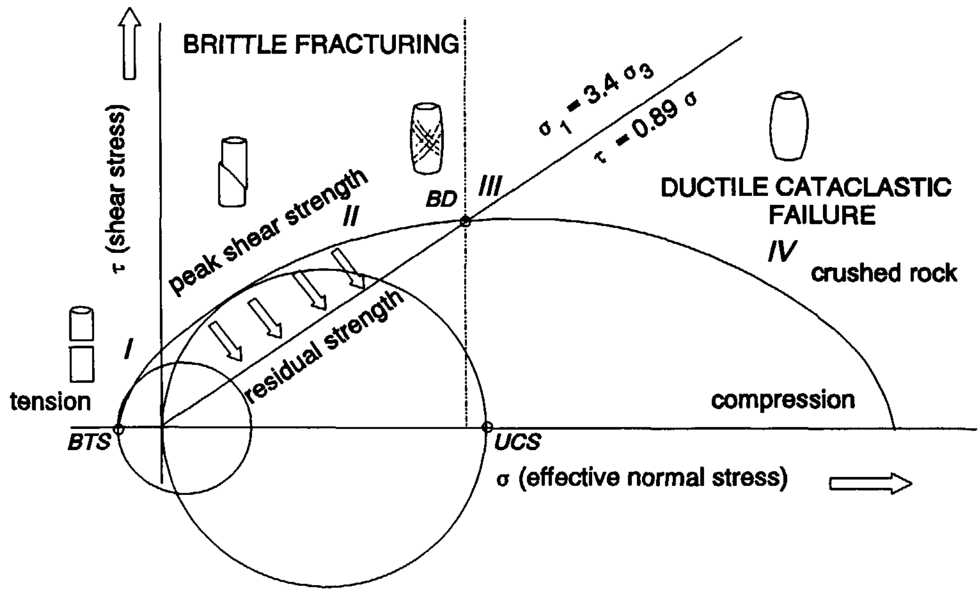

Figure 9.4 Complete failure envelope (Mohr-Coulomb presentation). Mogi's brittle-ductile transition line cuts the envelope in the BD transition point (see Appendix D).

9.1.1 *Rock mechanical aspects*

The most interesting implication of the failure model of rock cutting (Figure 9.2 & 9.4) is that the complete failure envelope for rock tested at room temperature is involved. A complete definition of this envelope would require extensive triaxial testing. Normally such tests are done at relatively static testing conditions. The effect of loading rate on failure is to increase the stress level of failure (see Chapter 9.4). An expansion of the failure envelope (Figure 9.2 & 9.4) into the failure field may result which merits further study.

Establishment of the failure envelope should ideally be done using triaxial tests, including measurement of volume change (and water pressure if relevant). In site investigations for projects that involve mechanical rock cutting, like dredging, large rock mechanical testing programs are cost prohibitive. The variability of natural rock requires much testing to be done. Therefore simpler methods that can give information on the shape and size of the failure envelope are of interest. Index tests that may indicate the extent and shape of the failure envelope of Figure 9.4 are the *UCS (unconfined compressive strength),* the *BTS (Brazilian tensile strength* and the *PLS (point load strength)* test. These tests help to define the shape of the failure envelope at the left site of the diagrams of Figure 9.2 & 9.4. With help of the empirical Hoek-Brown failure criterion, the brittle part of the failure envelope can be further defined (Hoek & Brown 1980, Hoek et al. 1995). Estimation of the brittle-ductile transition stress can be done using the relationship $\sigma_1 = 3.4\sigma_3$, based on the work of Mogi (1966), as an approximation of the brittle-ductile transition (Hoek and Brown 1980). In Appendix C & D the methods to determine strength of intact rock and the brittle-ductile transition are treated further.

Table 9.1 Ductility number (UCS/BTS), with a classification following Gehring (1987).

Ductility number (UCS/BTS)	Failure mode
< 9	Ductile
9 - 15	Average
> 15	Brittle

9.1.2 *Cutting of ductile rock*

In dredging, ductile cutting behaviour can be encountered in some types of cohesive (clayey) soils, or in weathered parts of a rock mass. In most cases forces are lower than when cutting rock, but if the material contains abrasive quartz, grooving abrasive sliding is known to give high amounts of wear, caused by the continuous contact of the abrasive grains with the cutting tool in two-body wear mode. Ductile cutting is therefore disadvantageous. By adaption of tool design it is possible to manipulate the cutting behaviour within certain limits. If possible brittle cutting behaviour is induced.

The relative *ductility* of the rock is a parameter often discussed when assessing the cuttability of rock by tunnelling machines such as roadheaders. Gehring (1987) has found that the ratio of unconfined compressive strength to unconfined tensile strength[40] is a measure for *ductility* and relates to production of roadheader tunnelling machines. The more brittle a rock, the larger the chips that are formed and the higher the production. *Ductility*, or its opposite *brittleness*, may be indicated by a *ductility number*, which is the ratio of unconfined compressive strength to unconfined tensile strength (Table 9.1). This notion of the importance of this ratio fits into the rock failure model already discussed. Figure 9.5 illustrates that for two hypothetical rocks with the same (unconfined) tensile strength (TS), but differing unconfined compressive strength (UCS), the failure envelope of the rock with lower ductility number is mainly in the ductile field, while that of the rock with higher ductility number occupies a large part of the brittle field.

The following account of rock cutting experiments, specially designed to investigate the influence of the wear flat of real scale cutter teeth (test category IV of Figure 4.13), illustrates the importance of the ductile crushing zone near the tool tip. Cools (1993) designed large-scale cutting experiments to measure the temperatures developing at the wear flat of a dredger cutting tooth during the cutting of rock. Two 2.5 m long rock blocks were tested in the Delft Hydraulics laboratory test rig, under water. One block of non-abrasive limestone (UCS 25.2 MPa, BTS 1.5 MPa) and a block abrasive sandstone (UCS 19.5 MPa, BTS 1.7 MPa). The surprising result of these experiments was that, while the sandstone did not cause measurable wear at the machined wear flat, the limestone did give appreciable wear. The extreme high normal forces that were measured during the limestone cutting indicated that the normal stress under the wear flat amounted to about 300 MPa, but for the sandstone this was considerably less, 130 MPa. Both values indicate that near

[40] The (unconfined) tensile strength is normally estimated by the Brazilian Tensile Strength test (BTS). Gehring (1987) also used this test method.

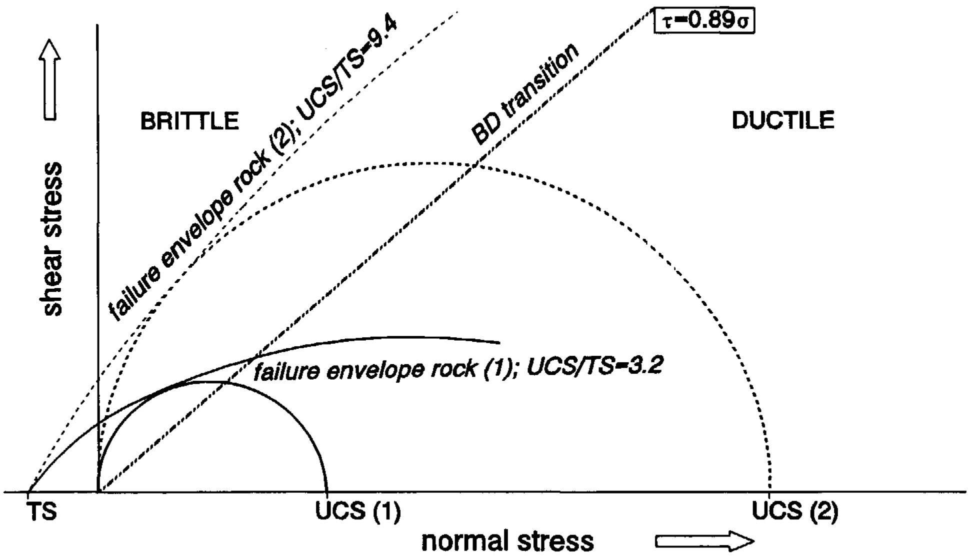

Figure 9.5 The ratio of UCS and UTS (ductility number) indicates the shape of the failure envelope and relates to the probability of ductile failure (compare with Figure 9.4).

the wear flat failure was in the ductile field; for the limestone significantly so. Triaxial test results indicated that the confining - maximal pressure set ($\sigma_{3,T}$, $\sigma_{1,T}$) at which the brittle - ductile transition of failure occurred was at 23 and 96 MPa for the limestone and at 41 and 119 MPa for the sandstone (Figure 9.6).[41]

The temperature at the wear flat of the limestone was approximately 800 °C. This temperature was high enough to cause softening of the dredger tooth steel in a fraction of a mm thin zone at the wear flat, leading to wear. The limestone itself had probably developed a large ductile crushing zone around the chisel (judging the high confining pressure under the wear flat, well above the brittle-ductile transition), which probably was a cause of the development of the high temperatures. The temperature at the wear flat of the sandstone was 550 °C, a temperature too low to cause thermal softening at the wear flat of the chisel. In Chapter 17.2, field observations on the performance of rock cutting trenchers are given. The calculated stresses under the bits, the temperature estimates on the bits and the BD transition of the rock types are determined to examine further the relevance of rock ductility on the cutting process.

9.1.3 *Implications of the rock cutting model*

During rock cutting near the cutting tool very high compressive stresses occur that lead to cataclastic[42] ductile failure. The complete failure envelope of intact rock

[41] The ductility number for the limestone in the above example is 16.8, for the sandstone 11.7; these numbers do not indicate ductile cutting behaviour using Gehring's system.

[42] Cataclastic refers to the microscopic failure mechanisms operating during failure. This involves processes such as microfracturing, grain crushing and grainboundary sliding, which

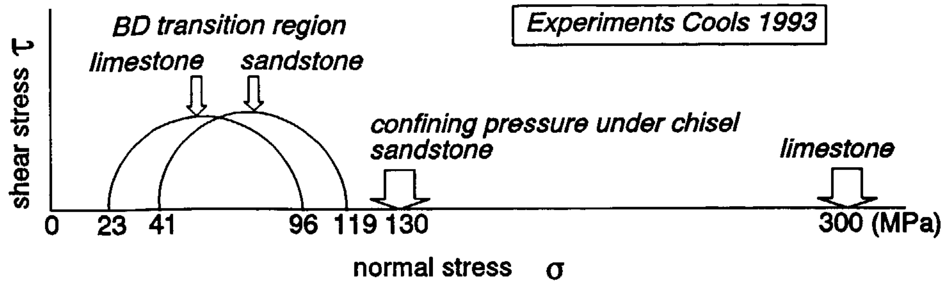

Figure 9.6 The brittle-ductile transition stress states and the calculated isotropic stress under the test chisels of the experiments of Cools (see text).

material (Figure 9.2 & 9.4) is addressed. The brittle-ductile transition stress is important in this respect, because it probably relates to the size of the crushed zone. A large zone will result into ductile cutting of the rock, a smaller zone into brittle cutting. The brittle-ductile transition stress (BD) gives a measure of the size of the failure envelope of the rock material. A small envelope points to the likeliness of ductile cutting, a large one to brittle cutting. Since brittle cutting is advantageous, the design of the cutting tool should be such that a small crushed zone is favoured.

With regard to the rock parameters that should be assembled during site investigation for rock dredging projects, these should give information on both unconfined tensile and compressive strength of the rock material (see Chapter 22 and Appendix C & D).

9.2 SIMPLE ROCK CUTTING MODELS

The types of cutting teeth that are used in dredging rock fall under the group of *drag tools*. These tools generally cut parallel to the free surface. In brittle rock normally *chips* are formed, by fractures growing in front of the cutting edge and breaking out sideways of the cutting tool (Figure 9.7).

Commonly used rock cutting theories are recently summarized by Roxborough & Sen (1986), Whittaker & Frith (1990) and Whittaker, Singh & Sun (1992).

9.2.1 *Evans' cutting model*

Much of the early theory on rock cutting with drag picks has been developed by Evans, studying coal cutting. The theory is largely based on observations of experiments. Evans (1962, 1965) considered that the breakage of coal is essentially tensile and occurs along a failure surface *ab*, which approximates a circular arc (Figure 9.8). Evans developed his model by studying the penetration of a wedge normal to a rock surface, as displayed in Figure 9.8 (Evans & Pomeroy 1966,

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are micro-brittle processes. At a macroscopic scale, however, the sample fails in a ductile mode.

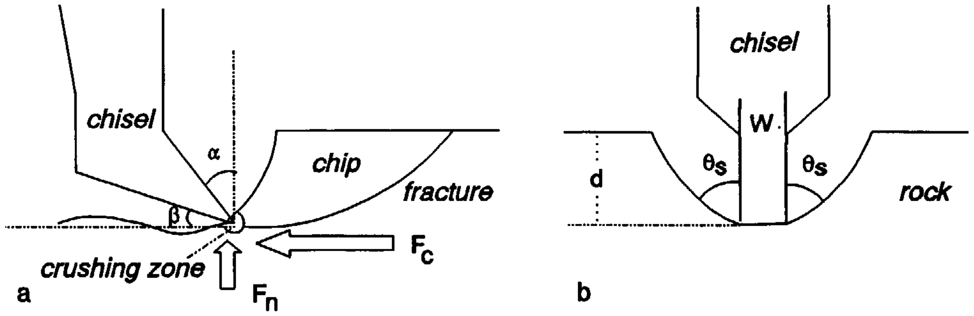

Figure 9.7 a. Drag tool, parallel to cutting direction, α: rake angle, β: clearance angle. b. Normal to cutting direction, W: chisel width, θ_s: breakout angle.

Whittaker & Frith 1990). At the tip of the penetrating wedge, *a*, tensile cracks are observed to be horizontal. The assumption Evans made was that the circular arc is tangential to the bisectrix plane of the tool wedge at the tip of the wedge. The midpoint of the circular failure plane is at *O*, at right angles to the cutting direction. Evans considered that the forces acting on the chip are the resultant cutting force *R* acting in a direction normal to the face of the wedge and the resultant tensile force *T* which is the sum of all tensile forces acting along the curved failure plane. In order to maintain equilibrium of forces he considered that a third force *S* acts through the hinge point *b*. Evans determined *T* by assuming plane strain conditions and summing tensile strength along the failure arc. He found a relation between *R* and *T* by taking moments about *b*. The minimum cutting force F_c was determined by finding the corresponding forward breakout angle θ_f ($\delta F_c/\delta\theta_f=0$). It appeared that the condition is fulfilled when $\theta_f = ½\alpha$ (i.e. the forward breakout angle is half the rake angle). Many cutting tools have the shape of a *half-wedge* (the shaded area in Figure 9.8) and for this case the following relationship for the cutting force F_c (which is the horizontal component of the reaction force R) was derived:

$$F_c = \frac{2\sigma_t d W \sin\frac{1}{2}(90-\alpha)}{1-\sin\frac{1}{2}(90-\alpha)} \quad (N) \tag{9.1}$$

where σ_t is tensile strength (MPa), d is depth of cut (mm), W is the width of the chisel shaped tool (mm) and α is the rake angle (°). Evans (1965) mentions limitations of the theory:

- it is two-dimensional
- it deals with a material of uniform properties, neglecting the effects of anisotropy or weakness planes
- the penetration of the chisel is assumed negligible with respect to the cutting depth (Evans 1962).[43]
- the clearance angle β is assumed zero (fig. 9.7a).

[43] The original theory of Evans includes the assumption that the penetration of the wedge is proportional to the unconfined compressive strength of the rock. The ductility ratio σ_c/σ_t is of influence; higher values give higher cutting forces (Evans 1962, p.769-770).

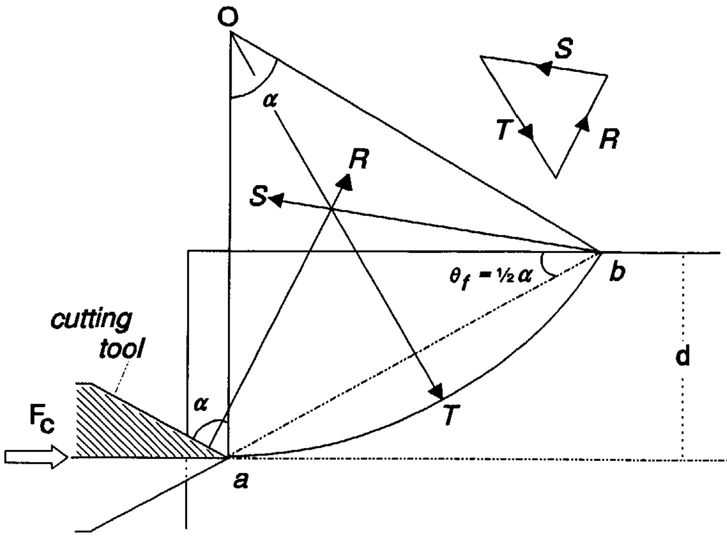

Figure 9.8. Evans' tensile cutting theory (after Evans & Pomeroy, 1966).

The assumption of two-dimensionality leads to an under-estimation of the peak cutting force. One effect of three dimensionality, the chip formation, has been examined by Verkaik (1982) in an MSc study on the cutting of rock with a dredging wheel. The surface area of the tensile fracture of a chip can be regarded as a measure of the energy needed to fracture the rock. Correction of the cutting force calculated by Evans' Equation 9.1 for the surface area did lead to higher cutting forces, but these were still 40 % off the measured value.

Evans (1965) explains that the effect of a positive clearance angle is rapidly annihilated by wear, which produces a wear flat parallel to the direction of motion (i.e. effective clearance angle is zero).

According to Roxborough & Sen (1986), as long as rock failure occurs in tension, this equation matches observed trends almost exactly and gives force values that are in tolerable agreement with measured data. In fact, Roxborough (1973) notes that in many cases rocks agree better with Evans' theory than coal, for which the model was developed. A comparison of experimental data on Dunhouse sandstone, showing the effect of rake angle α, is given in Figure 9.9. Evans' equation underestimates the cutting force, but the decrease of cutting force with increasing rake angle α matches the trend indicated by the equation. In Figure 9.10a a comparison of calculated and measured data of cutting tests on Hawkesbury sand- and siltstones is given. Typically the measured cutting forces are higher than the calculated ones. The cutting force estimated by Equation 9.1 is the minimum peak cutting force needed to fracture the rock. Evans (1962, 1965) showed that the friction between the tool wedge and the rock can be taken into consideration, by including the angle of sliding friction of rock against tool material $\phi^{r,t}$:

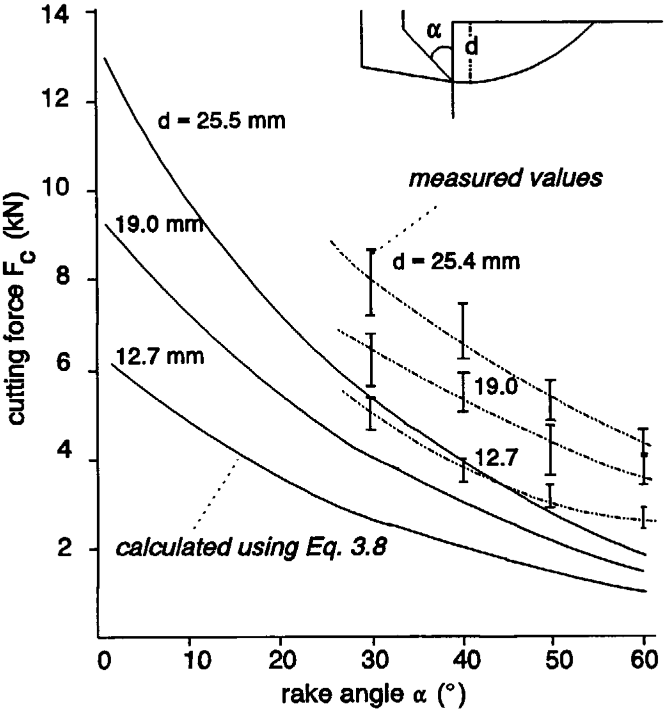

Figure 9.9 Variation in cutting force with rake angle α. Cutting experiments on Dunhouse sandstone compared with results of Evans' model, for three cutting depths (Roxborough 1973).

$$F_c = \frac{2\sigma_t dW \sin\{\frac{1}{2}(90-\alpha)+\phi^{r,t}\}}{1-\sin\{\frac{1}{2}(90-\alpha)+\phi^{r,t}\}} \quad (N) \tag{9.2}$$

Table 9.2 gives some values from literature on measured friction angles of tool steels against geological materials[44] and in Figure 9.10b the effect of including $\phi^{r,t}$ is shown. But not only friction can explain the low values found by Equation 9.1. The effect of three dimensionality has been mentioned. The tensile strength is commonly determined by tests under static conditions, whereas the impact velocity during the cutting may be orders of magnitude higher, which is known to lead to higher strengths.[45]

For the normal component F_n of the reaction force R (Figure 9.8), in the case of a clearance angle of zero and taking rock-tool friction into consideration Evans (1962) finds:

[44] These values only give an indication. As explained in Chapter 4.1.2, Equation 4.5, the friction coefficient, or $\phi^{r,t}$, depends on the actual wear mechanism taking place during the particular test at which the friction was determined.

[45] This is a complicated topic, which is further discussed in Chapter 9.4. Tensile strength will be higher at higher impact velocities, but the static test which is normally used to determine tensile strength, the Brazilian test, is known to overestimate tensile strength by a factor of 2-4 (Appendix C).

Table 9.2 Values of friction angle of geological materials against cutting tool steel (Schatzov 1964).

Rock / Soil type	Surface dry	Surface wet
clay	8.0 - 10.2	4.6 - 6.8
sandy clay	14.0 - 15.6	11.3 - 14.6
slate	11.3 - 14.0	8.5 - 11.3
marl	11.3 - 15.1	10.2 - 14.0
limestone	19.3 - 21.8	18.3 - 20.8
dolomite	20.8 - 22.3	20.0 - 21.8
anhydrite		21.3 - 24.2
weak sandstone, angular quartz	17.7 - 22.3	15.1 - 21.8
weak sandstone. round quartz	12.4 - 18.8	11.3 - 16.7
sandstone	23.3 - 25.6	23.3 - 24.2
quartzite	24.7 - 25.6	25.6 - 26.6
granite	25.1 - 28.8	24.7 - 27.9
coal	20.8 - 22.8	18.3 - 20.0

$$F_n = \frac{F_c}{2\ \tan(90-\alpha+\phi^{r,t})}\ (N) \tag{9.3}$$

Except for very small and very large values of α, the normal force calculated this way is of the same order of magnitude as the cutting force.

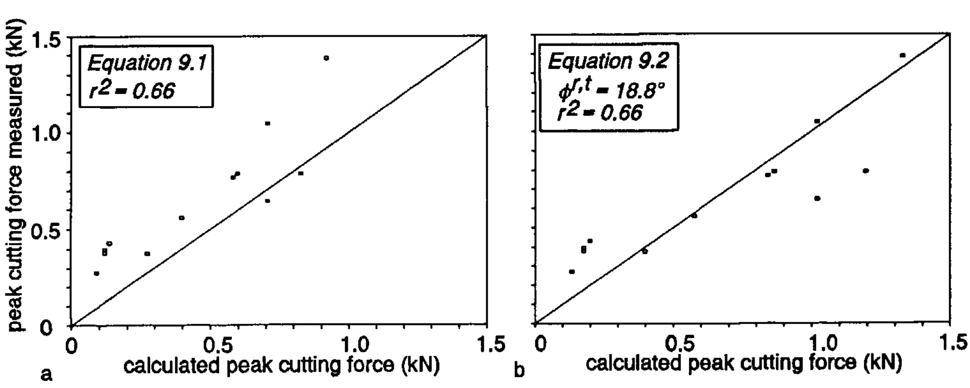

Figure 9.10 Comparison of measured peak cutting forces with calculated forces using Evans' tensile failure model; r^2 = coefficient of determination (data Table 9.3, p.112).

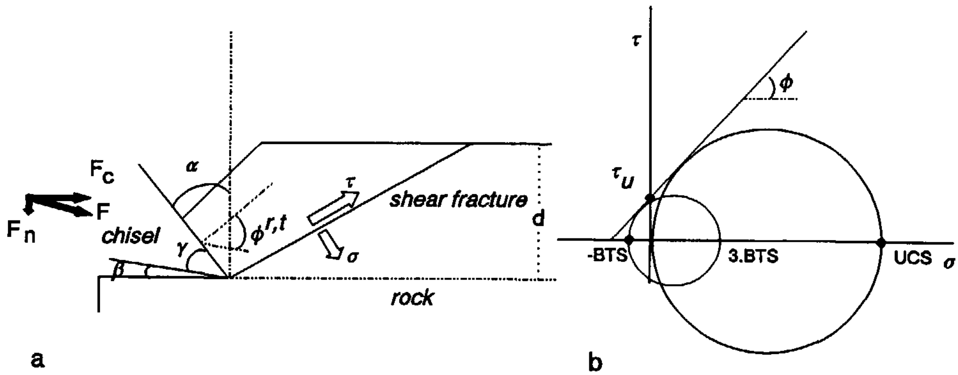

Figure 9.11 a. Shear failure model of Nishimatsu (1972). b. Method of calculating τ_u from UCS and BTS.

9.2.2 *Nishimatsu's shear failure model*

In practice it is observed that when the rake angle reduces, tensile failure may give way to a process of shear failure (Roxborough & Sen 1986). Nishimatsu (1972) has developed a model for this situation (Figure 9.11a). He arrives at the following resulting force *F*:

$$F = \frac{2\tau_u d W \cos\phi}{(n+1)(1-\sin(\phi^{r,t}+\phi-\alpha))} \quad (N) \tag{9.4}$$

where τ_u is the unconfined shear strength (MPa), d is depth of cut (mm), W is the width of the chisel shaped tool (mm), $\phi^{r,t}$ (°) is the angle of sliding friction rock-tool, ϕ (°) is the angle of internal friction of the intact rock material, α is the rake angle (°); n is a stress distribution factor, which approximates $11\text{-}\alpha/5$, according to test results. Nishimatsu determined τ_u and ϕ by a goniometric consideration of the Mohr-Coulomb failure criterion (Figure 9.11b):

$$\tau_u = \frac{\sigma_c\sigma_t}{2\sqrt{\sigma_t(\sigma_c-3\sigma_t)}} \quad (MPa) \ ; \ \tan\phi = \frac{\sigma_c^2 - 4\tau_u^2}{4\sigma_c\tau_u} \tag{9.5}$$

σ_c is the unconfined compressive strength, σ_t the tensile strength of the rock.

The cutting force F_c and the normal force F_n components of the resultant force F are (Figure 9.11):

$$F_c = F\cos(\phi^{r,t}-\alpha) \ ; \ F_n = F\sin(\phi^{r,t}-\alpha) \tag{9.6}$$

The assumptions made by Nishimatsu are summarized by Whittaker et al. 1992:

- the rock cutting process is brittle, without any accompanying plastic deformation (no ductile crushing zone)
- the cutting process is under plane stress condition
- the failure is according to a linear envelope (Figure 9.11b)
- cutting speed has no effect on the process and mechanism of cutting (which, within limits, has shown to be valid in many experiments, Evans & Pomeroy 1966, Roxborough 1973, Roxborough & Phillips 1975).

Roxborough & Sen (1986) state that Evans' and Nitshimatsu's equations are complementary and can be used in conjunction to determine the conditions under which cutting in a given rock would be by a process of dominantly tensile or dominantly shear failure.

9.2.3 *Cutting model for pick points*

The above equations are for chisel shaped cutting tools. For a pointed pick Evans (1984) derived the following equation:

$$F_c = \frac{16\pi\sigma_t^2 d^2}{\sigma_c \cos^2\theta}\ (N) \tag{9.7}$$

where Θ is the conus angle of the pick point.

9.2.4 *Relevance of simple rock cutting models*

All models discussed above make simplifying assumptions, the most important of which is that the rock is considered as a homogeneous, continuous material with a certain *unconfined tensile-, shear- or compressive strength.* The models are essentially geometric analyses of force distributions. In principle it is possible to considerably improve these rock cutting models and adapt these for certain tool designs. In reality such attempts are frustrated due to several factors. Rock fracturing is a complicated microscopic process taking place in extremely complicated materials. Detailed studies on rock fracturing suggest that factors such as mineral composition, grain size and shape, microscopic structure, cementing or bonding characteristics determine the way in which fractures are initiated and propagate through the rock. These characteristics probably influence the shape of the failure curve (Figure 9.2 & 9.4). Similarly these factors will influence the real cutting behaviour. Hoek & Brown (1980) have shown that a good approximation of the failure curve can be obtained when, next to the UCS, the rock constant m is known (see Appendix D). This rock constant is nearly equal to the ratio σ_c/σ_t (ductility number). Many of the cutting force equations cited above contain both σ_c and σ_t. When interpreting rock mechanical data for excavation projects the implementation of more sophisticated rock failure criteria into cutting equations can be of benefit (Appendix D).

New approaches to understand cutting behaviour probably will benefit from the use of numerical modelling tools, which are able to model complex chisel shapes and anisotropic rock properties in two and three-dimensions. Such models calculate stress distributions in the rock and make use of failure criteria such as mentioned above (e.g. Pierry & Charlier 1994, Van Kesteren 1995). Whittaker et al. (1992) describe application of fracture mechanics analysis to the cutting process. Direct application of fracture toughness indices does not give any improvement over other index values such UCS or UTS. For numerical methods use of fracture mechanics concepts seems promising, however. Van Kesteren (1995) gives an estimation of the stress intensity at the tip of the shear crack in the chip formation model (Figure 9.2), using fracture mechanics. The bifurcation of the shear crack, induced by shear strain localisation in the crushed zone, into a tensile crack is regarded crucial to chip formation by Van Kesteren. Numerical modelling supports this hypothesis (Van Kesteren 1995).

Including the numerical methods, the rock cutting models are static. Rock cutting is a dynamical process, with the propagation of stress waves playing a role. Such waves are known to reflect on free surfaces causing tensile stresses that also may fracture the rock. These considerations illustrate the difficult task to predict cutting forces from a theoretical point of view.

9.3 SPECIFIC ENERGY, CUTTING DEPTH AND SPACING OF CUTTING TOOLS

9.3.1 *Specific energy and cutting depth*

Roxborough (1973) extended the theory by considering the work done during rock cutting. It was observed that when a cutting pick took a deeper cut, the breakout production increased. While the resulting inclined sides of the chips formed were irregular, it was possible to calculate an average breakout angle θ_s from measurement of the volume of the cut rock. Within reasonable limits in stronger rocks this angle appeared constant for different depths of cut. Roxborough realized that measurements of forces on a pick in a certain rock cutting system were of little value unless related to the volume of rock excavated. Therefore the parameter *specific energy* was introduced, which was defined as the work done in excavating unit mass or volume of rock.

Consider Figure 9.7b, p.90. The area A of the excavated groove can be approximated from: $A = Wd + d^2 \tan\theta_s$. And the volume $V = L(Wd + d^2 \tan\theta_s)$, where L is the length of the groove. The work done (E) in excavating the groove is length times average cutting force: $E = F_{c,m}.L$. The specific energy SPE is given by:

$$SPE = \frac{F_{c,m}.L}{A.L} = \frac{F_{c,m}}{Wd + d^2\tan\theta_s} \quad (MN/m^2) \tag{9.8}$$

Since it is the *average* cutting force that determines the energy needed for excavation (compare Figure 9.3, p.83, brittle cutting), the cutting force equations of the previous section cannot be used, since these determine the peak cutting force needed to cut a rock. Roxborough (1973) points out that in many experiments the ratio R between peak and mean cutting force is fairly constant. For weak rocks, like coal, this ratio is often 2, for strong rocks it may be 3 or more. If this ratio R is known, the specific energy can be approximated using one of the equations estimating cutting force from the previous section and combine it with Equation 9.8. Equation 9.8 shows that, for a given rock and cutting tool, the specific energy decreases as depth of cut increases. Figure 9.12 shows one of the characteristic experimental test results of Roxborough (1973). The cutting force increases with depth of cut, the specific energy decreases. Decrease in specific energy levels off at greater cutting depths.[46]

[46] At depths greater than the chisel width, the *SPE* will increase again.

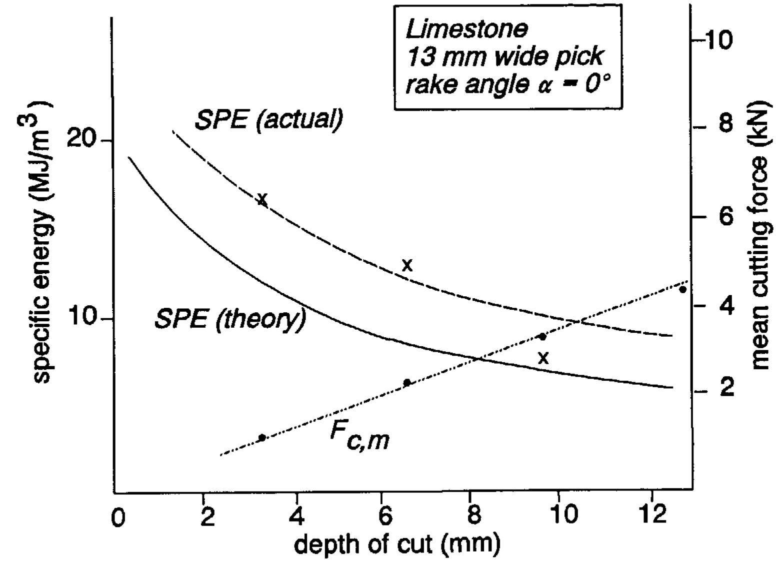

Figure 9.12 Effect of depth of cut on mean cutting force and specific energy (Roxborough 1973).

9.3.2 *Spacing of cutting tools*

In cutterheads the spacing of the tools is such that interaction is likely to occur between adjacent tools. If the breakout angle is constant, the spacing s between adjacent tools at which interaction will begin increases with increasing depth of cut. Evans (1972) estimates the optimum spacing of chisel-shaped cutting tools using the width of the chisel and the cutting depth by:

$$s_t = \frac{W}{2}\left[1 + \sqrt{\left(1 + \frac{20}{(\frac{W}{d})^2}\right)}\right] \; (mm) \tag{9.9}$$

where s_t is the optimum spacing in mm, W is the width of the tool (mm) and d is the depth of cut (mm).

For pointed picks, Evans (1984) found the following approximate relation:

$$s_t = 2d\sqrt{3} \; (mm) \tag{9.10}$$

Roxborough (1973) used Eq. 9.7 to illustrate chisel interaction (Figure 9.13). Interaction begins when the ratio of spacing and cutting depth, s/d, is smaller than $2tan\theta_s$.

A discussion of the influence of the ratio s/d on the breakout pattern and production of cutterheads is given by Knissel (1990). Knissel argues that both the outbreak angle θ_s and the optimum ratio, s_t/d, have to be determined experimentally for each rock. The complication with cutterheads is that, due to the cycloidal cutting paths of the chisels or pick points, cutting depths and cuttings lengths are not constant. The cutting angle will vary as well, being different when entering and leaving the rock.

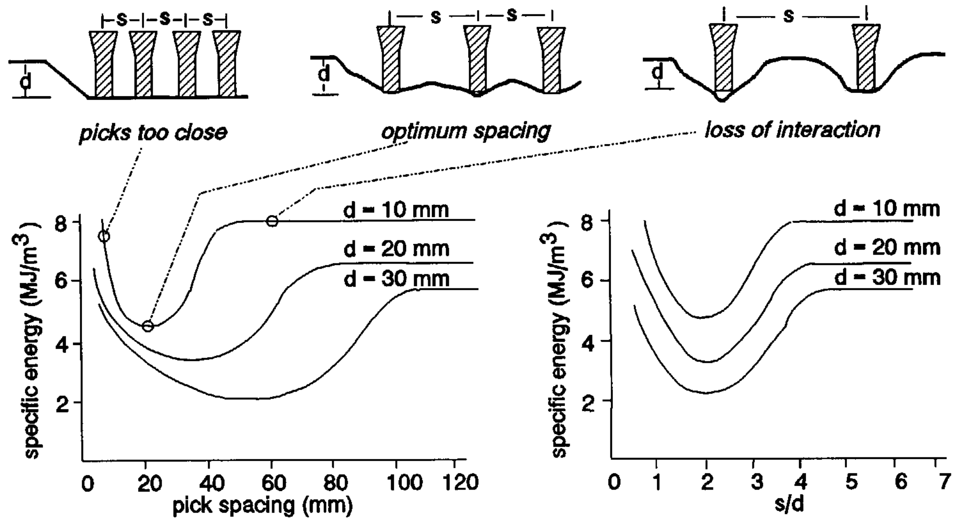

Figure 9.13 Effect of pick spacing on specific energy (Roxborough & Sen 1986).

9.4 CUTTING VELOCITY AND HEAT DEVELOPMENT

In early literature most authors indicate that cutting forces are independent of cutting velocity, within the ranges tested, which were mostly up to 3 m/s (Evans & Pomeroy 1966, Rix 1971, Roxborough 1973). The tested rocks were coal, anhydrite and gypsum, all known to be non-abrasive to steel. Roxborough (1973) investigated the influence of cutting velocity (up to 0.6 m/s) on the forces during rock cutting in anhydrite. No influence was found, but the remark was made that so-called abrasive rocks[47] were expected to influence results, due to the effect of chisel blunting. Gregor (1968, 1970) performed tests on coal and gypsum with cutting speeds, ranging from 10^{-4} to 10 m/s. Interestingly an increase of cutting force was found between 1 - 10 m/s. This increase in cutting force matched a similar increase in unconfined compressive strength (Figure 9.14). Gregor's data indicate that for these rocks significantly higher forces are measured above the velocities normally used in rock cutting experiments (> 3 m/s).

Nishimatsu (1979) gave a theoretical discussion on the effect of cutting velocity, trying to explain the observed effects. Three physical effects were examined:

- the rate of tensile crack propagation compared with cutting velocity
- the effect of stress rate on rock strength
- the effect of increase in velocity on heat generation

During rock cutting, failure in the rock will take place due to the propagation of *tensile cracks* in the rock. The velocity of crack propagation varies from rock to rock, but will be mostly higher than 1000 m/s, which is at least 100 times higher

[47] Abrasiveness is a relative term. It should always be used as "abrasive with respect to" the counter body material, like cutting tool steel or tungsten carbide. When the term abrasive rock is given normally quartz-rich rock is meant. Non-abrasive rocks are rocks consisting of low-hardness minerals like calcite.

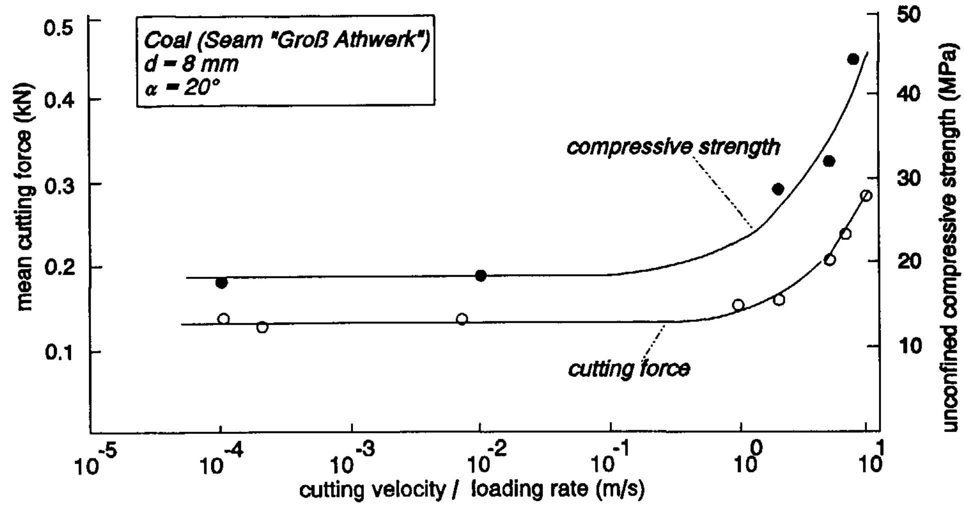

Figure 9.14 Influence of testing velocity on rock cutting force and on unconfined compressive strength of coal (Gregor 1968).

than common cutting tool velocities. Therefore Nishimatsu discards this effect.

Stress rate is known to have a profound effect on rock strength. Apart from the data given in Figure 9.14, well known test results are given by Goodman (1989). A rough rule of thumb indicates that a ten times increase of stress rate increases strength by 10% (Nishimatsu 1979). Such an increase will not be noticed within the ranges of velocities normally used in rock cutting experiments (1 - 3 m/s), due to the inherent force fluctuations during the cutting of brittle rock (compare Figure 9.3). Gregor (1970) attributes the increase of cutting force noted in his experiments at higher velocities (Figure 9.14) largely to the stress rate dependence of rock strength.

Nishimatsu (1979) also gives considerations as to why specific energy should increase with increasing tool velocity and that *heat* should be generated during the cutting process. The increase of tool temperature would depend on tool velocity and cutting force. Models have been developed that calculate the increase of tool temperature due to friction while drag cutting (Van der Sman 1988, Glowka 1989). The models are based on the calculation of the heat flux generated in the contact area of tool and rock due to friction. The amount of heat generated is assumed to be proportional to cutting velocity. These models indicate that steep thermal gradients are present, both in the tool and rock. Van der Sman (1988) made a calculation of the effect of repeated heating and cooling of the teeth of a cutterhead cutting in rock under water. The assumption was made that the cutterhead was cutting in rock half of its diameter; a tooth would cut in rock one half of a revolution, and cool off in water the other half. Heat flux calculations were made, where the frictional heat was divided over the steel and the rock during the cutting and over the steel and the water during the cooling part of a cycle. A cutting tooth would adsorb part of the heat and divide this over the tooth, resulting in a change of the temperature distribution in the tooth. During the non-cutting part of the cycle in which the tooth is rotating in water, part of the heat is conducted to the water and the heat content of the tooth will be reduced by a certain value. After a certain amount of cycles, a stationary situation exists, in which the amount of heat energy that is added to the

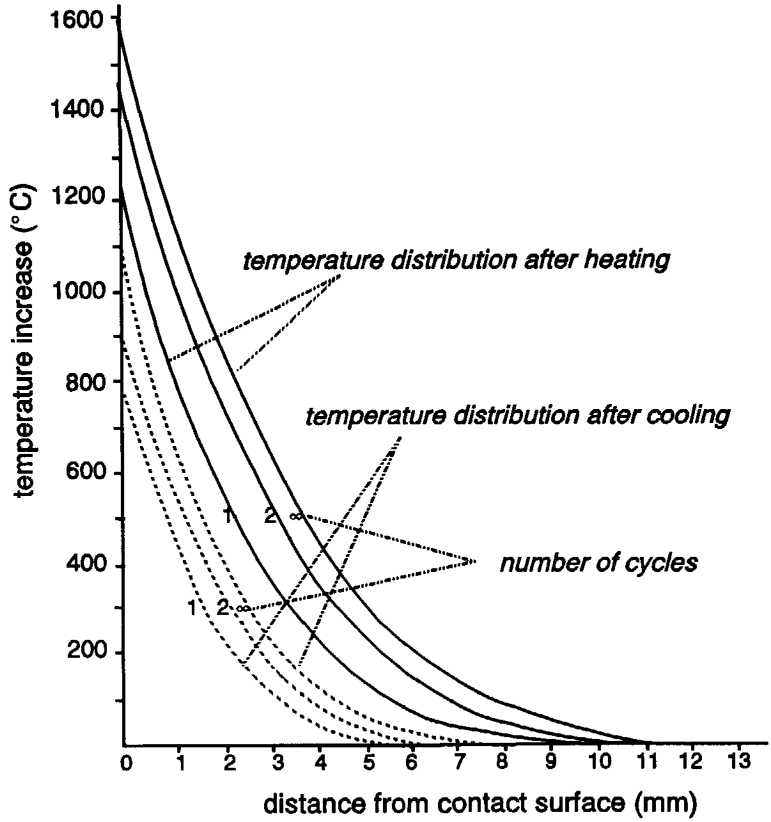

Figure 9.15. Calculated temperature increase after cycles of heating and cooling during rock cutting by cutterhead under water (Van der Sman 1988).

tooth steel body behind the wear flat is equal to the amount of heat released to the water. The calculations showed that already after a few cycles this stationary state was reached. The result of the calculation is shown in Figure 9.15. This result, though based on a calculation in which several simplifying assumptions had to be made, matches the observations made on the temperatures present in chisels while cutting (Cools 1993) or determined indirectly by observation of phase transformations in steel (Figure 9.16) or measurements of Hardness gradients in previously heated chisels (Deketh 1995). Figure 9.16 shows a polished section of a carbon steel chisel used in cutting experiments, described in Chapter 9.7. Near the cutting surface, the primary martensite structure of the hardened steel has been affected by secondary heating during the cutting. The coarsening has taken place over a width of 60 - 80 μm at temperatures estimated at 300 - 350 °C. At the cutting surface plastic deformation has taken place and the martensite structure is lost. At the carbon content of the steel used, martensite starts to form below 400 °C. The plastic deformation must have occurred above this temperature (P. Colijn pers. comm.).

Increasing cutting velocity would therefore have two effects, namely an effect of increase of cutting force due to the stress-rate dependence of rock strength, and an increase of heat generation which leads to temperature increase in the tool.

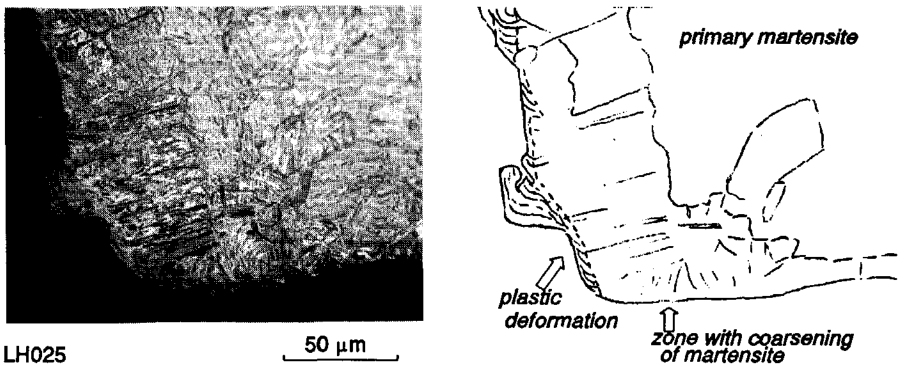

Figure 9.16 Photomicrograph of polished surface of steel cutting tool showing narrow zone of plastically deformed steel, inferred to have been heated above 400 °C (see text).

The temperature increase may lead to a sudden increase of wear rate of tools, which has been repeatedly reported in literature (Krapivin et al. 1967, Osburn 1969, Schimazek & Knatz 1970, 1976, Deketh 1995). In sandstones the experiments indicated critical velocities at which wear rates start to increase. Krapivin et al. (1976) found that this critical velocity is related to the temperature reached at the

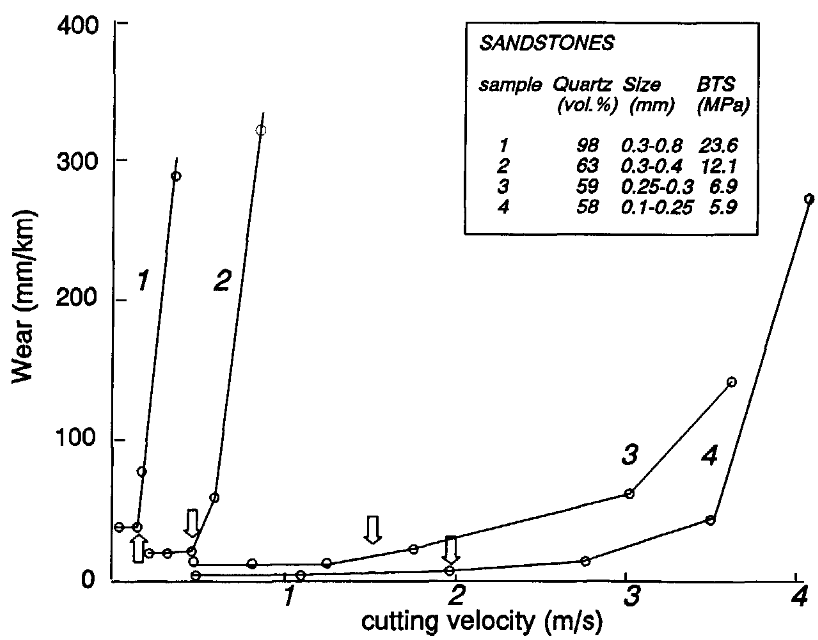

Figure 9.17 Data from Krapivin et al. (1967), showing increase of wear at a critical velocity, indicated by arrows (after Schimazek & Knatz, 1970).

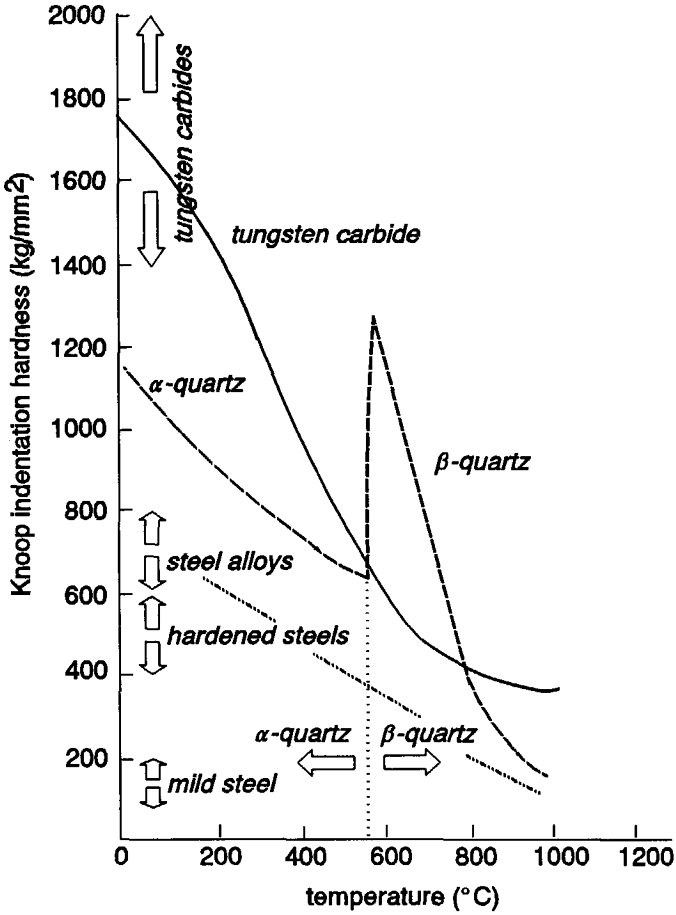

Figure 9.18 Range of hardness of steels and quartz and the change of hardness of tungsten carbide and quartz with temperature (after Osburn 1969).

contact area between the cutting tool and the rock. For the tool they used, hard steel, this temperature was 550 - 600 °C. They noted that quartz content, quartz grain size and strength influenced the critical velocity (see Figure 9.17), a fact later used by Schimazek & Knatz (1970, 1976) to develop a wear factor (see Chapter 11.3). The explanation for the increase in wear can be found in the softening of tool hardness with increasing temperature (Figure 9.18). Krapivin et al. (1967) noted that in their experiments at the temperature of 550 - 600 °C the hardness of the steel and rock would be about equal (hardness contrast = 1, Figure 4.4). At higher velocities than the critical, temperatures of 1200 - 1400 °C were reached and the metal was softer than the rock (Krapivin et al. 1967).[48] If the rock is quartz bearing, wear can be

[48] These temperatures were determined by measuring the temperature of the sparks by an optical pyrometer. These measurements were calibrated against the temperatures near the wear flat of the chisel, which were measured by thermocouples. At 600 °C the sparks consisted of quartz particles, at 1200 °C both quartz and metal particles were present.

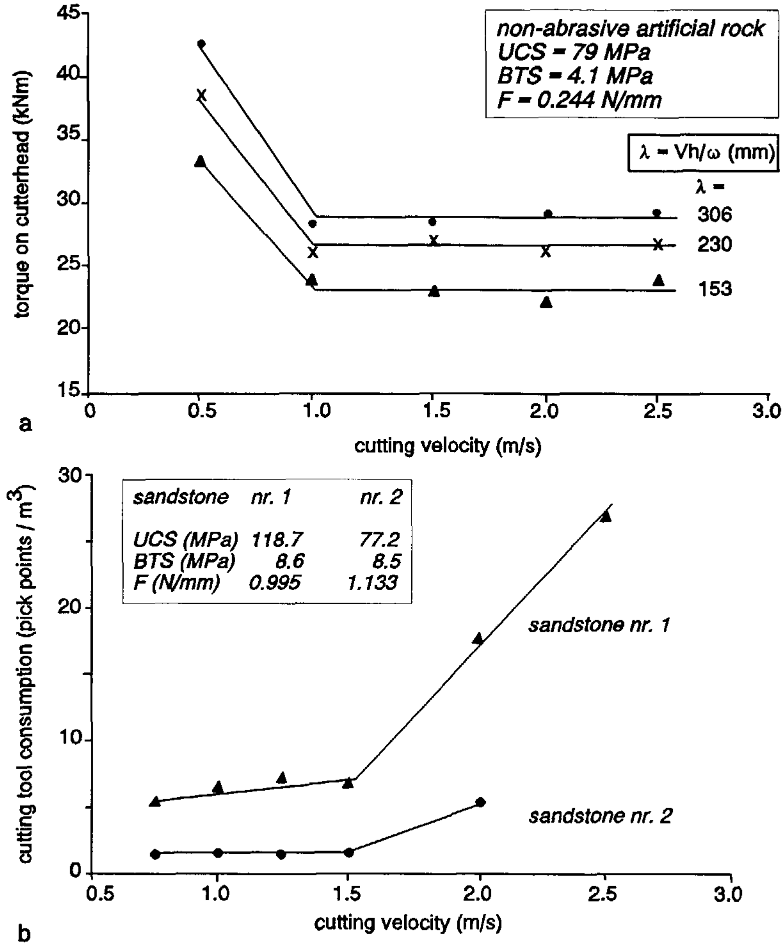

Figure 9.19 Effect of cutting velocity on a. torque and b. tool consumption of cutterhead (Driesch 1994).

dramatically increased by the sudden increase in the hardness of quartz, due to the α - β crystal structure transition taking place at 573 °C (Figure 9.18).

Also in so-called "non-abrasive rock" a sudden increase of wear can occur, if temperatures increase at the tool-rock interface. Due to the heating of the tool steel, this may soften in the thin zone (order of 0.1 mm) that is heated to high temperatures (see Figure 9.16). The cutting experiments of Cools (1993) in limestone are an example (Chapter 9.1.2).

Laboratory tests use mostly single tools, but the results of these tests are matched by tests with cutterheads mounted with chisels or picks. Recent experiments with a cutterhead cutting in sandstone to examine the effect of velocity[49] are shown in Figure 9.19. In these tests, the ratio of haulage velocity and rotation velocity (λ in Figure 9.19) was kept constant to assure constant penetration depth. The results show that cutting velocity has no influence on torque (Figure 9.19a; the higher

[49] The tangential velocity, V_t, of the teeth on the cutterhead.

torque at low cutting velocity was due to vibrations of the bearings of the cutterhead). However, the abrasive rocks tested cause increased wear above a critical velocity of 1.5 m/s (Figure 9.19b; Driesch 1994).

The experimental work suggests that within the range of velocities normally used by rock cutting machines, 1-3 m/s, hardly an increase in cutting forces is expected, despite the stress-rate dependent increase of strength of rock. However, the higher velocities may lead to increased wear due to frictional heating. The resulting blunting of the tools effects the cutting forces. In quartz-bearing rocks, increase in cutting forces can be expected within the range of velocities used by cutting machines.

9.5 CUTTING UNDER WATER

Since dredging occurs below water level, the influence of water on the cutting process has to be examined. There are several effects to be considered[50]:

- the increase of hydrostatic pressure (height of water column above cutting level)
- the effect of porewater pressures and possible cavitation during the cutting process

These effects are discussed in this chapter.

Another effect, the cooling of heated chisels or pick points by the surrounding water has been has been discussed in Chapter 9.4. As remarked there, it is expected that a stationary situation may develop regarding the temperature distribution in the chisel (Figure 9.15). The source of the heat is the frictional heat developing at the contact between rock and tool. The ambient water temperature governs the amount of cooling that can be achieved. The cooling effect of water is low and comparable to air. More important than the cooling by water is the geometry and the mass of steel of the chisel itself. Steel has a much higher thermal conductivity than water and is therefore a better coolant than water.

9.5.1 *Effect of pore water pressure during rock cutting*

In sands the effects of water pressure are important and have been examined by Miedema (1987). The increase of hydrostatic pressure due to the water column (0.1 MPa per 10 m) effects the pore pressure and is not negligible with respect to the strength of soils (unconfined shear strengths in the order of 0-0.5 MPa). During the cutting of water saturated sands the effect of dilatation dominates the cutting process. Due to dilatancy water under-pressures are generated, which could reach cavitation pressures at high cutting velocities (at cavitation pressure the water evaporates). If cavitation occurs the cutting forces increase with a factor $z+10$, where z is the height of the water column and 10 is the addition due to atmospheric pressure.

With respect to the strength of most rocks being dredged, which normally have an unconfined compressive strength of 0.5-30 MPa, the changes in hydrostatic

[50] Not mentioned here is the effect of chemical corrosion due to water composition (see Chapter 4.1.1).

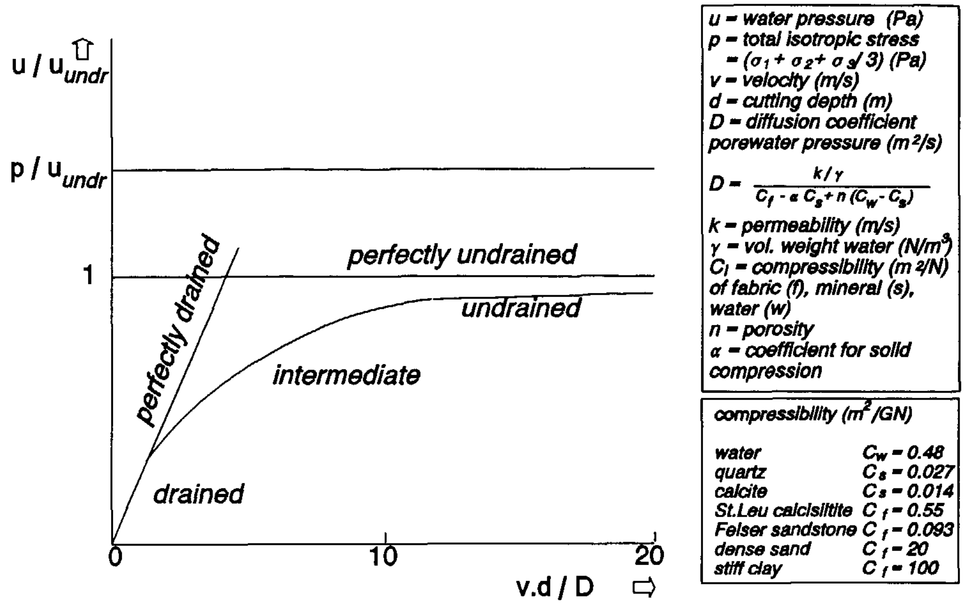

Figure 9.20 Effect of pore pressure dissipation due to isostatic compression as explained by Van Kesteren (1995), see text.

pressure in the order of 0.1 MPa may not seem important, considering the primitivity of the models used for predicting the cutting forces. Most rocks are less porous than sands and have much lower permeabilities. In porous rocks, with regard to the rock cutting model of Chapter 9.1 (Figure 9.2), pore water pressures must play a role in the formation of the crushed zone, considering the high confining stresses present near the tip of the cutting tool. Van Kesteren (1995) has examined the effect of pore water pressures in rocks, by distinguishing two limiting conditions: drained and undrained. In the *drained* condition pore water flow due to pore water pressure gradients is possible, without affecting the porous system itself. In the *undrained* condition, pore water is not allowed to flow through the pores and pore water pressures will affect the stress state in the rock fabric. Van Kesteren has calculated the pore water pressures caused by an external isotropic stress p as a function of the pore water pressure dissipation (Figure 9.20). The pore water pressure is caused by the change in total isotropic strain of the rock, due to compression of the rock fabric, the water in the pores and the mineral grains. As the value of the pore pressure dissipation ($v.d/D$; Peclet number), is larger than 10, undrained behaviour will occur, see Figure 9.20. In that case the limit pore water pressure can be estimated from:

$$\frac{p}{u_{undr}} = 1 + n \, \frac{C_w - C_s}{C_f - \alpha C_s} \quad (-) \qquad (9.11)$$

where p is the total isotropic stress $(\sigma_1+\sigma_2+\sigma_3)/3$ (MPa), u_{undr} is the undrained pore water pressure (MPa), n is the porosity, C_w is the compressibility of water, C_s is the compressibility of the mineral grains and C_f is the compressibility of the rock fabric (m^2/GN). α is a coefficient reflecting the compression of the solids due to the deformation of the rock fabric, α approaches 1 in the case of rock pores.

An example is given by Van Kesteren: For St. Leu limestone, the following parameters are known: $n = 0.39$, $k = 10^{-7}$ m/s, $C_f = 0.55$ m^2/GN, $C_s = 0.014$ m^2/GN, $\alpha = 1$. Therefore D (see the Diffusion coefficient equation in Figure 9.20) is about 0.014 m^2/s. During the cutting, the behaviour of the crushed zone will be drained when $v.d$, the cutting velocity times the cutting depth, is smaller than 0.014 m^2/s and undrained when $v.d > 0.14$ m^2/s (see Figure 9.20). From Equation 9.11 can be calculated that the generated pore pressure is about 75% of the total isotropic stress in this case.

In Van Kesteren's paper cone penetration tests in St.Leu limestone are discussed which confirm the effect of pore water pressure on the cutting mechanism. At low penetration velocity (0.01 m/s) pore water can drain from the deformation zone around the cone. Pore compaction balances the indented volume in the rock. At intermediate velocity the size of the crushed zone is reduced due to the resistance of the pore water flow. The cone volume cannot be balanced any more by pore compaction and shearing occurs towards the free surface with chip formation. At a velocity of 1 m/s the undrained condition around the cone occurs. Due to the high pore water pressures, liquified crushed rock is squeezed to the free surface along the cone face. Similar transitions were observed during cutting tests of the St. Leu rock. A decrease of the friction angle between the chisel and the rock is noted when $v.d > 0.02$ m^2/s. This decrease can be explained by the high contribution of the pore water pressure to the total stress in the crushed zone around the chisel.

Van Kesteren also explains that pore water pressure is of influence on crack initiation and propagation. He argues that at the strain rates occurring during dredging (order of 10^3 s^{-1}), the pore water is able to flow towards crack tips (drained case; the transition towards undrained occurs at strain rates > 10^5 s^{-1}). This implies that crack initiation is not impeded by resistance of the pore water. During crack propagation away from the crushed zone, the pore water pressure is in balance with the hydrostatic head (height of the water column). The maximum pressure difference is determined by the water depth and the possible occurrence of cavitation in the crack and is in the order of 0.2 - 0.4 MPa (10 - 30 m below sea level). This is in the order of the tensile strength of weak rocks (0.5 - 2 MPa). The effect of cavitation on the crack initiation just outside the crushed zone can therefore be neglected, but it will be of influence when the crack propagates towards the free surface: cavitation may then inhibit unstable crack growth.

9.5.2 *Effect of water saturation on the strength of rocks*

It is known that the presence of water in rock has a weakening effect. Compared with dry rocks, water bearing or saturated rocks may have strengths which are 30 - 90 % of the dry strength measured. This reduction in strength occurs independently of stress- or loading rate and is not a mechanical effect (due to pore pressures), but an electrochemical effect, as explained by Vutukuri (1974), who tested limestones immersed in different types of fluids. Water has a pronounced weakening effect compared to other fluids with lower dielectric constants and weakening may occur already at low degrees of water saturation. The conditions of testing of rocks should therefore always be specified (see Appendix C). For rock dredging saturated rock strengths should be used in rock excavatability assessments. In Figure 9.21 the effect of water wetting is illustrated by cutting tests in Bunter sandstone, where mechanical weakening due to water resulted in lower wear (tests with CH chisels).

9.6 EFFECT OF WEAR OF TOOL ON CUTTING PERFORMANCE

Much research has been done, especially in the mining and tunnelling industry, on the wear of rock cutting tools (e.g. Kenny & Johnson 1976a & 1976b, Phillips & Roxborough 1981, Yardley et al. 1981, Clark 1987). Commonly these studies emphasize the abrasive wear of the tool (catastrophic failure or breakage of the tool is disregarded). In most studies different types of tool material have been tested against a limited amount of rock types, often classified as *abrasive* and *non-abrasive*.[51] The wear of rock-cutting tools has a definite influence on their cutting efficiency. Most tools, including drag picks, have the shape of a wedge. Wear results in the removal of a prism of tool material from the cutting edge of the bit (Figure 9.22). The resulting wear flat forms an area of zero or negative clearance, and crushing of rock beneath the wear flat increases the tool forces. A wear flat of a few millimetres increases the tool forces by several times their initial values (Figure 9.23).

It is important to note here that it is *the change in shape* of the tool that determines the rise in forces (Yardley et al. 1981), see Figure 3.11. If a tool could keep its shape during the wearing process, no change in forces would result. With the design of disc cutters, commonly used on full-face tunnel boring machines, use is made of *self sharpening* during wear (Büchi 1984). The robust design necessary for drag cutting tools with pick- or chisel shape always results in wear flat development. Figure 9.21 shows that already after a few hundred meters of distance cut a considerable wear flat may be present. This distance is reached within a few minutes of operation of a cutterhead. The effect of change of shape during wear is also illustrated by the experiments of Kenny & Johnson (1976a & 1976b), which show that large clearance angles reduce the effects of wear, even when the rake angle is small or even negative.

In general pick wear has no influence on optimum pick spacing or production, if cutting depth can be maintained (Roxborough & Sen 1986). It is noted that wear has a much more deleterious effect on normal force than it has on cutting force (Phillips & Roxborough 1981, Roxborough & Sen 1986). For a sharp pick the ratio of normal to cutting force is typically 0.5 - 0.7. This ratio increases to a value more than twice as high at even a moderate amount of wear (Figure 9.23). This general effect, which occurs at a fairly early stage in the pick wear process, results in the cutting machines being thrust rather than torque limited.

It has been found that *depth of cut* is an important factor. With increasing depth of cut less wear occurs and also the effect on cutting forces decreases (Yardley et al. 1981). This effect has been explained by Deketh (1995) and is discussed in Chapter 12.

The effect of blunting of the tools due to wear on the cutting forces could, apart from experimental measurement, also be estimated using an empirical-theoretical

[51] see footnote 47.

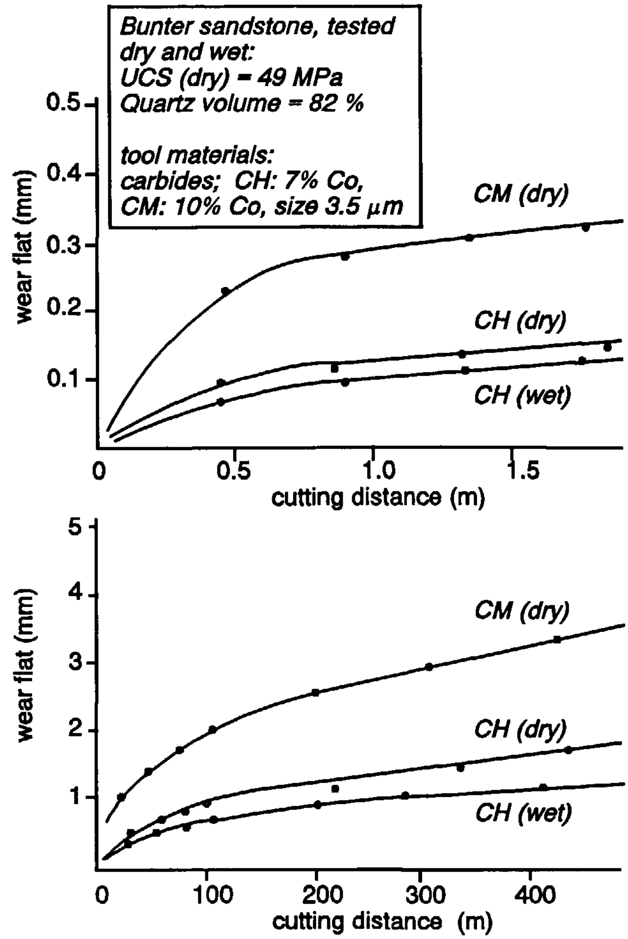

Figure 9.21 Effect of cutting distance on pick wear, linear cutting tests. Tool material: tungsten carbide, rock: abrasive sandstone (Phillips & Roxborough 1981).

approach. The cutting force equations discussed in Chapter 9.2 were developed for sharp wedges, which will only be valid for laboratory cutting tests, considering the high rate of wear in abrasive rocks (Figure 9.21). Evans (1965) developed an adaption of his cutting force equation for blunted tools.

9.7 TEST METHODS: THE CORE CUTTING TEST

Many test methods can be envisaged to obtain information on rock cutting performance. By now it will be clear that cutting tests carried out in the laboratory on small rock specimens, or on blocks of rock, will not necessarily be indicative of the cutting performance of the full-scale cutting machine in the rock mass. This especially pertains to the rock dredging problem. The size of the pick points or chisels on the cutterheads of cutter suction dredgers is much larger than the test

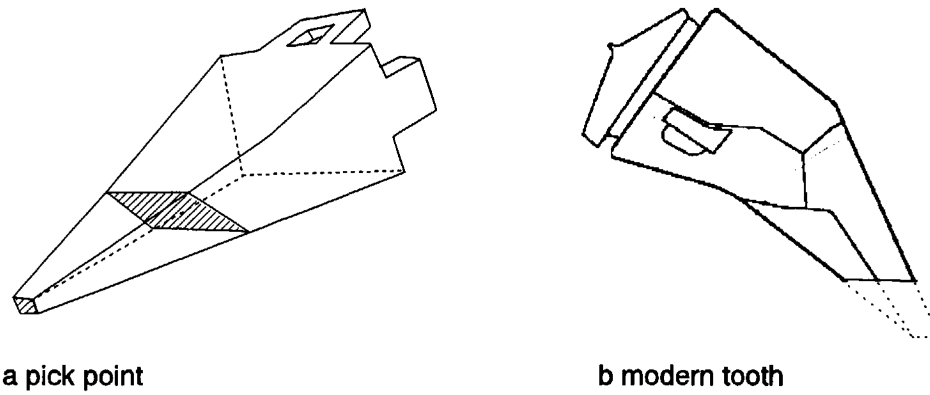

Figure 9.22 Wear flat development on a typical dredger pick point and on a modern tooth.

chisels used in most laboratory tests. Roxborough (1987) considered the following parameters to describe the *cuttability* of intact rock material:

1. specific cutting force: $F_{c,m}/d$ (N/m); the mean force acting on a pick or chisel in its cutting direction ($F_{c,m}$) per unit depth of cut (d).

2. specific normal force : $F_{n,m}/d$ (N/m); the mean force acting on a cutting tool acting normal to its cutting direction per unit depth of cut (d).

3. specific energy: $SPE = F_{c,m}*L/V$ (N/m^2); the energy or work required (cutting force Fc,m) times distance travelled L) to cut a unit volume of rock (V).

4. cutting tool wear rate: the rate at which a cutting tool wears in a given rock, measured as the weight loss of tool material per unit cutting distance (mg/m).

Roxborough and co-workers, at the University of Newcastle-upon-Tyne (UK) developed the *core cutting test* to determine the four machine performance parameters on core or block samples (Figure 9.24).[52] The test is standardized to exclude machine design influences. The test consist of cutting a groove 12.7 mm wide and 5 mm deep along the surface length of a rock core sample parallel to its axis. The core is then rotated by 180° to make a similar parallel cut. If the core has not been broken, it may be rotated again to make a third and fourth cut. If the core is 250 mm long a maximum length of 1 m can be tested.

The test arrangement is placed on a shaper. For each rock core tested a new tungsten carbide cutter insert is used. This insert has a chisel shape, is 12.7 mm wide, has a front rake angle of 0° and a back clearance angle of 5°. The principal forces acting on the chisel are measured by means of a triaxial dynamometer to which the cutting tool is attached. The forces are measured continuously during the cutting and assembled on computer. The amount of rock cut from each groove is measured and used to calculate the specific energy. The tungsten carbide insert is weighted before and after the set of up to four cuts and its weight loss is used to determine cutting wear. The specifications of the cutting test are given in Figure

[52] This cutting test has been used in experiments discussed in the previous paragraphs (Figure 9.10, 9.12, 9.13, 9.21 & 9.23).

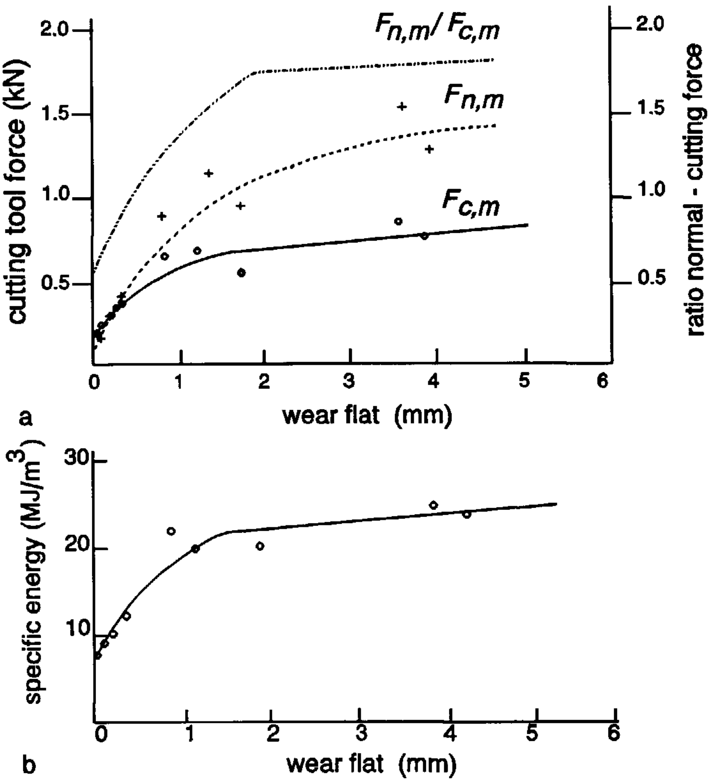

Figure 9.23 Effect of wear on average pick forces (a) and specific energy (b). After Phillips & Roxborough (1981).

9.24. The limiting diameter of 50 mm is required, because the breakout angle of rock chips is dependent on the radius of the core. With small diameter cores rock yield is relatively lower and the specific energy higher than for a larger core of the same rock.

According to Roxborough the wear of the carbide inserts can be attributed to two sources, namely due to abrasion and due to brittle chipping of the metal. The latter mainly occurs when rock of high strength is tested. The impact chipping usually takes place at the cutting edge and corners of the inserts in the cutting test. Significant chipping relates to the magnitude of the peak component forces measured during the test. To investigate the contribution of abrasive wear, the *core abrasion test* was developed, which described in Chapter 11.1. The abrasive wear can be expressed in weight loss of carbide divided by cutting length (mg/m) and this number may be compared with the cutting wear loss determined by the cutting test, to appreciate the relative contribution of abrasive wear to the latter.

On rock cores of Hawkesbury Sandstone the cutting and the abrasion test have been carried out. Since the tests were used to evaluate a dredging project the cores were saturated. The engineering properties of the rocks which have been tested are described in Chapter 16, some properties are given in Table 9.3. Apart from using the usual tungsten carbide inserts, also some tests were performed using S1045 hardened steel, heat treated up to 830 degrees Celsius and quenched in water, to

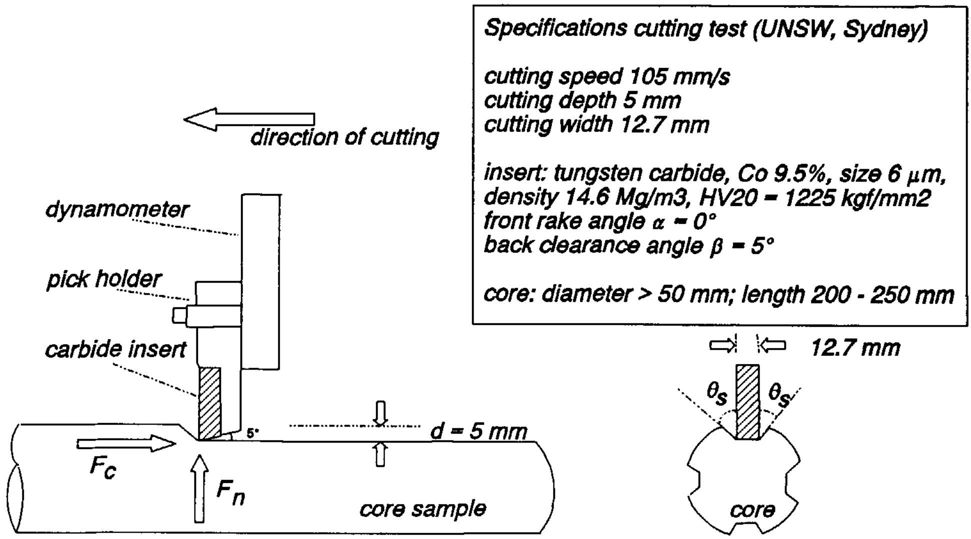

Figure 9.24 Test arrangement of the Newcastle-upon-Tyne cutting test.

obtain an estimated hardness of Rockwell C58 (corresponding to a Vickers Hardness of HV 700).[53] These bits have been examined later with a metallurgical microscope to obtain information on the heat distribution near the wear flat and estimates of temperature that existed on the wear flat during the tests (Figure 9.16).

The cutting tests were performed on cores of preferably 200 to 250 mm long. The diameter of the cores was 53 mm. During the cutting test commonly at the end of a cutting run the end piece of the core broke of. This part was not considered for the determination of the volume of rock cut. The volume was determined by measuring the weight of the cutting debris, which was carefully assembled after each run. It could have been possible to cross check the weight determined on the debris by weighing the core before and after each run, but this was realized only after the performance of the tests. The volume cut was calculated by dividing the weight of debris by the density of the rock. The density was determined before the start of the cutting test on the cores by weighing and measuring the volume of the core. After each run the core was turned and another test was performed. The tungsten carbide inserts were carefully cleaned, using detergent, before they were weighed to assure accurate weight measurement. After the series of cutting tests on each core the test bit was cleaned again and weighed with an accuracy of 0.1 μg. The steel test bits used for some of the cutting and abrasion tests were treated the same way. The forces determined by the dynamometer were assembled by a data acquisition system and the calculations were done using an interactive computer program.

From the stored force data, the peak cutting and normal force and the mean cutting and normal force were established. The peak cutting forces can be compared with calculated ones, like for example Equation 9.1 (Table 9.3). Figure 9.10, p.93 shows that Evans' equation follows the trend of the measured cutting forces quite

[53] This steel is the well-known high quality construction steel, known in Europe as 2C45 (EURO-norm).

Table 9.3 Results of tests on samples of Hawkesbury sandstone from over-water boreholes of Sydney Harbour.

Core Nr.	Description	Cutting Force Mean kN	Cutting Force Peak kN	Normal Force Mean kN	Normal Force Peak kN	Spec. Energy MJ/m^3	Cutting Wear mg/m	Abr. Wear mg/m	BTS MPa	F_c kN
3001	sl.w. SST	0.27	0.40	0.12	0.18	2.28	1.61	0.59	0.4	0.12
3002	fine gr. SST	0.36	0.79	0.31	0.55	4.39	2.52	0.32	2.7	0.83
3201	med.gr. SST	0.29	0.65	0.40	0.57	-	2.53	0.88	2.3	0.71
3211	med.gr. SST	0.27	0.43	0.07	0.10	2.55	1.35	0.40	0.5	0.14
3212	sl.w. med.gr. SST	0.29	0.38	0.11	0.12	3.13	0.32	-	0.4	0.12
3213	med.gr. SST	0.50	0.79	0.27	0.43	8.53	1.46	0.36	2.0	0.59
3233	med.gr. SST	0.68	1.39	0.57	0.89	10.09	1.80	-	3.0	0.92
3241	SST	0.25	0.37	0.13	0.16	3.52	1.00	0.49	0.9	0.28
3252	SST	0.58	1.05	0.54	0.75	9.23	-	1.00	2.3	0.71
3261	sl.w. SST	0.20	0.27	0.07	0.10	2.79	0.41	0.51	0.3	0.10
3262	SST	0.37	0.56	0.13	0.20	4.65	0.69	0.63	1.3	0.40
3281	Siltstone	0.55	0.77	0.06	0.16	-	0.83	-	1.9	0.58

sl.w. = slightly weathered; med. gr. = medium grain size; SST = sandstone; F_c = cutting force calculated using Equation 9.1

well. The mean cutting forces were used to calculate the specific energy for the cutting test.

To have an indication of the accuracy of the tests, fourteen cutting tests have been done on cores from a sandstone block, sampled at Lucas Heights (Sydney). An additional six tests were done using steel test bits. The results are given in Table 9.4.

Some cutting and abrasion tests on the Lucas Heights sandstone were carried out using carbon steel inserts, to obtain in an indirect way some information about the heat development near the cutting surface in the chisel. The observations show that high temperatures can develop in very narrow zones near the cutting surface of the chisels, and that rapid cooling occurs when the cutting or grinding stops. Evidence for temperatures above 400 °C, but below 800 °C, is present for the cutting test (Figure 9.16). The abrasion test resulted in temperatures above 800 °C during the abrasive grinding. The observation that the zone of significant temperature increase is very narrow, in the order of 100 μm, and that the thermal gradient is very steep, agrees with both theoretical and experimental results (thermocouple measurements) obtained elsewhere (Chapter 9.4; Van der Sman 1988, Cools 1993).

Table 9.4 Cutting and abrasion test results of experiments carried out on cores from sandstone block (Lucas Heights), using tungsten carbide and steel test bits.

Carbide	Cutting wear	Abr. wear	Cutting force		Normal force		Spec. Energy
	mg/m	mg/m	Mean (kN)	Peak (kN)	Mean (kN)	Peak (kN)	MJ/m^3
average	2.84 (0.19)[1]	0.43 (0.03)[1]	0.26	0.36	0.13	0.15	3.00
std	1.23		0.06	0.08	0.04	0.05	0.72
COV %	43		21	22	34	30	24
n	14	2	14	14	14	14	12
Steel							
average	3.54 (0.45)[1]	8.96 (1.14)[1]	0.30	0.41	0.13	0.16	3.31
std	1.19		0.07	0.10	0.03	0.03	0.77
COV %	34		22	24	22	20	23
n	2	1	6	6	6	6	6

[1] wear expressed in rate of volume loss (10^9 m^3/m). std = standard deviation, COV = coefficient of variation, n = number of tests.

9.8 ROCK MATERIAL PARAMETERS RELATING TO CUTTABILITY

From the beginning of the development of the cutting test, Roxborough and co-workers have performed a suite of index tests on the rock materials used, including petrographic examination (mineralogy, grain size and shape, cementation), physical properties (density, porosity), UCS, BTS, Shore hardness, NCB cone indenter and Schmidt hammer rebound. Results of these index tests have been correlated with the parameters obtained from the cuttability test by means of multiple linear regression techniques (McFeat-Smith & Fowell 1977, Deketh 1995). Some of these parameters relate to the rates of abrasive wear occurring during rock cutting testing. These will be examined in Chapter 11, 12 & 13. The strength parameters UCS and BTS (or the index tests related to strength like Point Load strength, Shore hardness, NCB cone indenter and Schmidt hammer rebound) are clearly related to the cutting forces measured in rock cutting tests, as shown in this chapter. Apart from strength, the specific energy involved in cutting proved to be a valuable measure of the cuttability of rock.

Attempts were made to relate index tests and the rock cutting parameters to the actual performance of tunnelling machines. McFeat-Smith and Fowell (1977) found linear correlations between laboratory specific energy and in-situ specific energy and between laboratory cutting wear and pick replacement rate for a Dosco road header in relatively massive rock (no significant discontinuities). In addition a relation between *SPE* and field cutting rate was found (see also Speight and Fowell, 1987).

However, Roxborough (1987) points out that, although these relationships look promising, the major drawback of the method is the difficulty of performing sufficient tests for the results to become representative. Hence relationships of cutting parameters with index tests are important. For the laboratory cutting test, Roxborough found:

$$SPE = 0.25\ UCS + C\ (MN/m^2) \tag{9.12}$$

where C is a constant found by linear regression and dependent on rock type. This relationship allows cutting tests to be used and advantage taken of the more numerous data on UCS to make an analysis of the variability of strength in the rock mass to be excavated, which can be applied to the excavation performance prediction. Equation 9.12 cannot be used without consideration. According to Roxborough it is basically sound, but would apply only to machines which have a constant depth of cut.[54]

The most relevant tests have been found to be:

- *Unconfined Compressive Strength*; if this test is performed it is advisable to also determine the deformation modulus of the rock, see Chapter 22.1, Appendix C.

- *Brazilian Tensile Strength* (or Point Load test, Chapter 22.1, Appendix C)

Important further considerations are:

- *Ductility number* (UCS/BTS) gives indication on chance on ductile cutting mode.

- *Rock type* and the *m*-value of Hoek-Brown also give an indication of the ductility of the rock (Appendix D).

9.9 CONCLUSIONS

Two questions were asked in the introduction to this chapter:

1. What intact rock properties that relate to cutting should be assembled in the site investigation for a rock dredging project?

2. How can these properties be used to predict the production of a cutter suction dredger?

Ad 1: During the site investigation for a rock dredging project, assessment of the excavatability of rock for machines necessitates an evaluation of the properties of the rock mass. Excavatability is a function of the rock materials present and of the pattern and density of the discontinuities transecting the rocks. As discussed in Chapter 8, the spacing of the discontinuities and their orientation are important factors. The spacing of the discontinuities may be compared with the size of the cutterhead and the expected penetration depth of the cutting tools (Chapter 8):

- *If the spacing is smaller than the expected penetration depth of a cutting tool, the material properties of the rock are less relevant. Production is mainly determined by the discontinuities.*

[54] Although not investigated in this chapter, besides the laboratory cutting specific energy, other tests measuring energy during rock comminution may be of use to evaluate cuttability. Such as rock toughness or work of destruction that are determined from unconfined compressive strength tests; see Appendix C.

- If the spacing is of the size (diameter) of the cutterhead or smaller, the production will be a function of the properties of the discontinuities (shear strength and orientation) and part of the cutting will be in intact rock.

- If the spacing is larger than the size of the cutterhead, the cutting will be mainly determined by the intact rock.

The cuttability of rock is describing the facility of intact rock material to be cut by mechanical tools. This property is partly dependent on the characteristics of the machine and the conditions under which the cutting takes place. Important rock properties are the *strength* of the rocks and tests measuring the unconfined compressive and tensile strength are highly relevant. Very high normal pressures may occur near the cutting tool wear flat. Some rocks will behave in a ductile way during cutting, which can lead to high temperatures at the wear flat, with resulting increased wear. The *ductility* of rock can be guessed at, by using the ductility number (UCS/BTS), or the m value of the Hoek-Brown failure criterion for intact rock and estimate the brittle-ductile transition stress (Appendix D).

Apart from the intact rock parameters just mentioned, those that relate to adhesive and abrasive wear, discussed in the Chapters 10-13, should be assembled during the site investigation.

Ad 2: The prediction of the performance of a cutter suction dredger will need the properties describing the excavatability and cuttability of rock just discussed. Machine settings and tool geometry and design can influence the rock cutting process. Higher penetration depths are favourable as indicated by the lower specific energy measured in cutting tests. Cutting velocities higher than 3 m/s tend to lead to higher cutting forces and high temperatures developing at the wear flat of the tool. Blunting of the tools, either by high temperature adhesive wear, or by abrasive wear, leads to higher cutting forces and higher energy consumption. In Chapter 18 prediction methods for mechanical rock excavation are considered.

CHAPTER 10

Hardness of rocks and minerals

Hardness of rock is generally recognized as an important property of influence on the cutting process and tool replacement rate. Hardness is a loosely defined term, referring to the resistance of a rock or mineral against a cutting tool or another object attacking the geological material. Figure 10.1 shows that the resistance that a rock may offer depends on the rock texture (microscopic structure) and the minerals present. Each mineral has its own stress-strain behaviour under the impacting force acting on it. The *hardness* experienced is therefore the result of the type of impact process (geometry and mechanical properties of the impacting tool, direction, nature and magnitude of impact force, impact energy, ambient temperature and pressure) and the rock petrography (orientation and size of minerals, grain shape, presence of microcracks). Considering the variables, is not surprising that no definite hardness test is known to date which is commonly used in engineering practice. Table 10.1 lists a number of tests, which are grouped according to the type of impacting action used.

The mineralogical composition of rocks is important, because the minerals largely determine the hardness of the rock. Often the rock hardness is estimated by summing

Table 10.1 Common hardness tests.

Impact action	Test	Reference
Dynamic rebound	Shore scleroscope Schmidt hammer Equotip	ISRM (Brown 1981), Janach & Merminod 1982 Verwaal & Mulder 1993
Static indentation	Vickers Knoop Brinell Rockwell	ASTM, Uetz 1986
Scratching	Mohs	Bowden & Tabor 1964
Grinding	Rosiwal	Rosiwal 1916
Erosion	Sandblast	Verhoef 1987

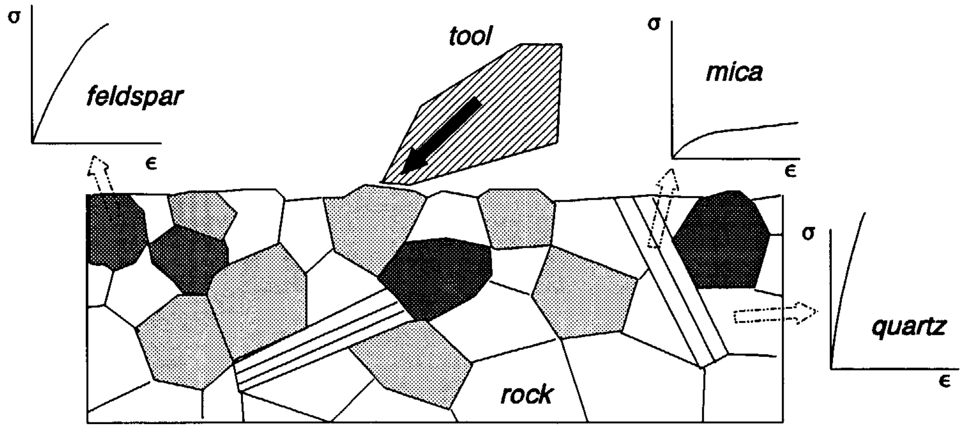

Figure 10.1 Hardness is a strength and stiffness property, which depends on the mechanical properties of the minerals constituting the rock and on the method of loading the rock.

the hardness of the constituent minerals, known from microscopy or mineralogical analysis, by consulting tables of well-known hardness scales, like Vickers Hardness or Rosiwal Hardness. A linear relationship between rock hardness and mineral hardness is assumed implicitly and the influence of grain size, bonding and structure is neglected using this method.

Hardness is determined using rebound tests (Chapter 10.1), indentation tests (Chapter 10.2) or scratch tests (Chapter 10.3). Since the Rosiwal hardness is used commonly in assessments of abrasiveness of rocks, it is discussed in Chapter 10.4. A comparison of hardness scales is given in Chapter 10.5.

10.1 REBOUND HARDNESS

Rebound tests, such as the Schmidt hammer and Shore scleroscope are well known and commonly used to indicate the unconfined compressive strength of rock. In fact they measure the rebound response due to the impact of a steel body on the surface of the rock. Both methods have been recently described by Atkinson (1993). The use of these tests to specify abrasiveness of rock is limited. They tend to relate to the average strength of the rock material. The hardness property of the individual mineral may be assessed by studying the normally high scattering of scleroscope values, determined on different spots on the rock surface. In this respect the electronic Schmidt hammer, the Equotip, may fulfil the same role. The impacting device of the Equotip, a 3 mm diameter tungsten carbide sphere, is also very small (Verwaal & Mulder 1993). One should take care to grind the rock surface before using the Equotip, for which a portable drill with grinding disc can be used.

Janach & Merminod (1982) used a type M Schmidt hammer (impact energy 30 J) equipped with a roller bit (Rockwell hardness 62 HRC) to assess abrasiveness, by

Table 10.2 Schmidt hammer abrasivity test after Janach & Merminod. Values of M-hammer from Janach & Merminod (1982).

Rock type	Number of impacts, M-hammer	Number of impacts, N-hammer	Mass loss (mg)	Average abrasivity (mg/kJ)
Granite (Bohus, Sweden)	20		17.1	
	20		16.3	30
	20		20.0	
	20		18.7	
Gneiss (Washington DC, USA)	50		25.3	19
	50		31.7	
Gneiss (Domodossola, Italy)		100	5.8	
		200	9.2	22
		300	12.6	
Sandstone (Albringhausen, Germany)	20		3.9	
	20		4.9	
	20		3.2	7.4
	20		4.8	
	40		9.7	
Mica schist (Atlanta GA, USA)	30		6.0	4.6
	50		5.1	
Siliceous Limestone (Balmholz, Switzerland)	40		2.4	2.0
Dolomitic Limestone (Chicago IL, USA)	500		3.5	0.23

measuring the loss of mass of the bits after a number of impacts on a certain rock surface. The method proved useful, as the results correlate with the mini-disc wear test which relates to Tunnel Boring Machine performance. For the author's experiments it was tried to use the smaller N-type Schmidt hammer (impact energy 2.25 J) equipped with bits of weaker steel types for this purpose. However wedge steel (St 37) or hardened C45 steel plastically deformed under the blows, with no resulting loss of mass of the bit and rock particles were enclosed by the bit. The bits had to be of the roller bearing quality as used by Janach & Merminod. The mass loss of the hard bits was only measurable after more than 100 impacts even in strong abrasive rock. The results of the Schmidt hammer abrasiveness are expressed in mass loss per total impact energy (number of blows times impact energy of the hammer). The N-type hammer equipped with roller bits would need $30/2.25 = 13.3$ times[55] the amount of blows to have the same mass loss on the bits as the M-type hammer (Table 10.2), which makes the N-type hammer impractical for this purpose.

Tarkoy (1973) has developed a method to determine abrasiveness of rock by using the combination of Schmidt hammer rebound and Tabor abrasion test values to predict tunnelling machine performance.

[55] Assuming a linear relation between impact energy and loss of mass.

10.2 INDENTATION HARDNESS

In tribological engineering, Vickers-, Knoop-, Brinell- or Rockwell *static indentation tests* are used to give a measure of hardness of materials. The difference between the methods is mainly in the shape and size of the indentation body used. For minerals the Knoop and Vickers method are much applied. It is not uncommon that only tables with ranges of Vickers Hardness of minerals are consulted (e.g. Uetz 1986), instead of performing a hardness measurement on mineral or rock samples. Whether this is justified can be judged from the following discussion. Measurement of Vickers Hardness on minerals is more problematic than on metals. Due to the lower plasticity of minerals *micro-hardness* measurements have to be performed, using low pressures on the indentation diamond. If higher loads were to be used the minerals would fracture, making the measurement unsuitable. In metals the impression of the Vickers diamond is largely permanent (plastic), in rocks and minerals an important part of the deformation during indentation is recoverable (elastic). Vickers Hardness (HV) is defined by:

$$HV = \frac{load}{area\ indentation} = 1854.4\ \frac{G*M}{d^2}\ (MPa) \qquad (10.1)$$

where M is the mass applied to the indenter (g), G is the gravitational constant (9.81 m/s^2) and d is the diameter of the permanent imprint in the metal (μm). The units of HV are MPa, but commonly the unit kg/mm^2 is still used. Very often no units are mentioned and one refers to the Vickers Hardness number (given in kg/mm^2).[56]

Microscopic determination of indentation hardness, using loads of 0.01-3 N (mass applied: 1-300 g), is used to measure the indentation hardness of minerals. This method was applied by the author to measure the hardness of some minerals, using a *Durimet* micro-hardness meter. The minerals used were a single crystal of Quartz, where the hardness was measured parallel to the [0001] axis, Chalcedony (cryptocrystalline SiO_2), fresh Microcline feldspar and weathered Plagioclase from a granite. Masses varying from 25 g to 300 g were used. When fracturing occurred a measurement was discarded. In the weathered Plagioclase, many measurements were discarded because of fracturing. Those that were good were apparently in a fresh part of the mineral. This indicates that the weathering of feldspar proceeds by destroying local parts of a crystal, the intact parts retain the hardness of the fresh mineral, no gradual decrease of hardness was measured. The accuracy of the micro-indentation measurement depends mainly on the accuracy of the measurement of the indentation diagonal. The results of the measurements in Table 10.3 show that the coefficient of variation (COV) is quite high, in accordance with the accuracy expected by Bowie & Simpson (1977). They point out the decrease in accuracy when lower loads are used in the measurement. Often when the loads are higher than 1 N silicate minerals already are shattered, hence one is restricted to the low accuracy range of measurement.

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[56] The constant in this Equation is due to the geometry of the indenter, which is a square pyramid with a 136° angle between opposite faces, so that a perfect indentation is a square with equal diagonals seven times the depth of penetration. $H_v = (2 \times \sin(136/2)\ M/d^2 = 1854.4\ M/d^2$ (kg/mm^2), with M in g and d in μm; HV = 9.81 H_v MPa.

Table 10.3 Results of micro hardness measurements on minerals (5 determinations per loading mass).

Mineral	Mass applied (g)	HV (MPa)	COV (%)	HV corrected (MPa)
Quartz	50	13204	9.7	13320
(single crystal, // [0001])	100	15597	11.1	
	200	15147	4.5	
	300	12737	6.8	
Chalcedony	25	10762	17.0	10100
	50	17069	5.5	
	100	14244	9.7	
	200	11880	6.9	
	300	10595	9.7	
Microcline	100	8535	10.3	4340
(single crystal)	200	7387	15.3	
	300	5572	7.9	
Plagioclase	50	9667	15.4	7300
(weathered granite)		10517	10.8	
	100	7877	7.9	
		8896	5.6	
	200	9280	11.1	
		9667	6.7	

COV = coefficient of variation (percentage standard deviation)

Engelhardt & Hausstühl (1965) have pointed out that an important part of the indentation deformation in a brittle mineral is elastic, contrary to metals or ore minerals. The size of the indentation, measured after the removal of the indenter, would then be too small. They proposed a correction, to be able to compare the indentation data of brittle minerals with that of metals. This correction is shown in Figure 10.2, where the results of the micro-hardness measurements on the minerals of Table 10.3 are plotted. The correction assumes that the elastic contraction of the indentation diameter is δ μm. The correction leads, in most cases, to a reduction of Vickers Hardness when compared with single measurements at one load. The correction of Vickers Hardness for minerals has not found wide application, mostly uncorrected HV values are cited in literature.

This small survey into the Vickers Hardness measurement has shown that much less reliability can be given to Vickers Hardness numbers of silicate minerals, compared with that of metals or ore minerals. The method gives only an indication of hardness.[57]

[57] This remark refers mainly to the usage with respect to wear. In itself the method provides much information. It is used frequently as an aid to ore mineral identification. It has shown the importance of hardness anisotropy in most minerals.

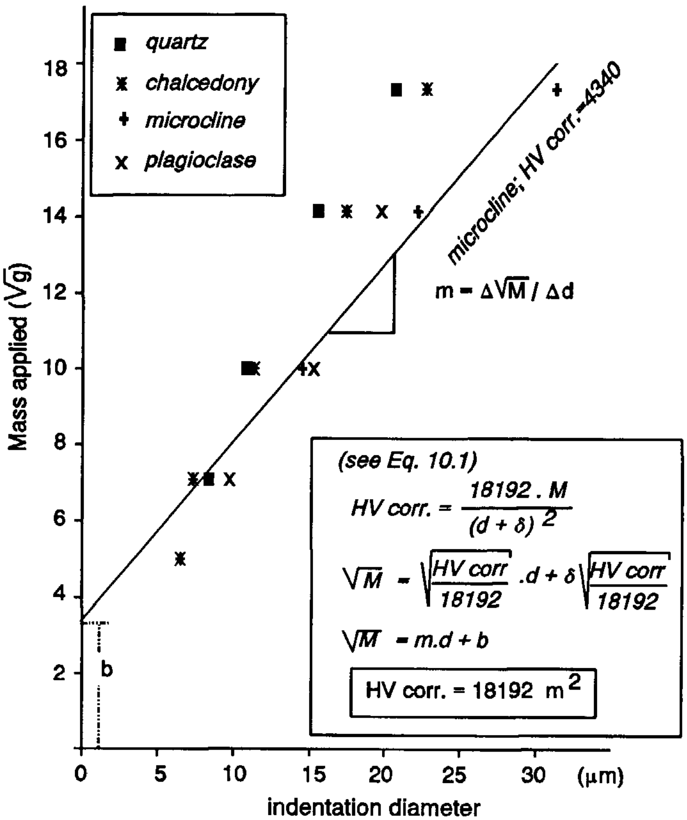

Figure 10.2 Correction for elastic rebound δ of Vickers indentation, using micro-hardness measurements at different loads. The line for microcline is shown. See Table 10.3 (after Uetz 1986, p.7).

10.3 SCRATCH HARDNESS

Table 10.1 refers to *scratching hardness* as well. *Mohs' scale* was made in the early 19th century and published in 1824. A mineral higher in the scale can scratch a mineral lower in the scale:

1. Talc	
2. Gypsum	*(a fingernail can scratch gypsum)*
3. Calcite	*(a brass pinpoint can scratch calcite)*
4. Fluorite	
5. Apatite	*(a knife usually has a hardness of about 5)*
6. Orthoclase	*(window glass has a hardness of about 5.5)*
7. Quartz	
8. Topaz	
9. Corundum	
10. Diamond	

It appears to be a rough scale, but Bowden & Tabor (1964) point to the remarkable fact that each Mohs' scale increment corresponds to a 60 % increase in indentation

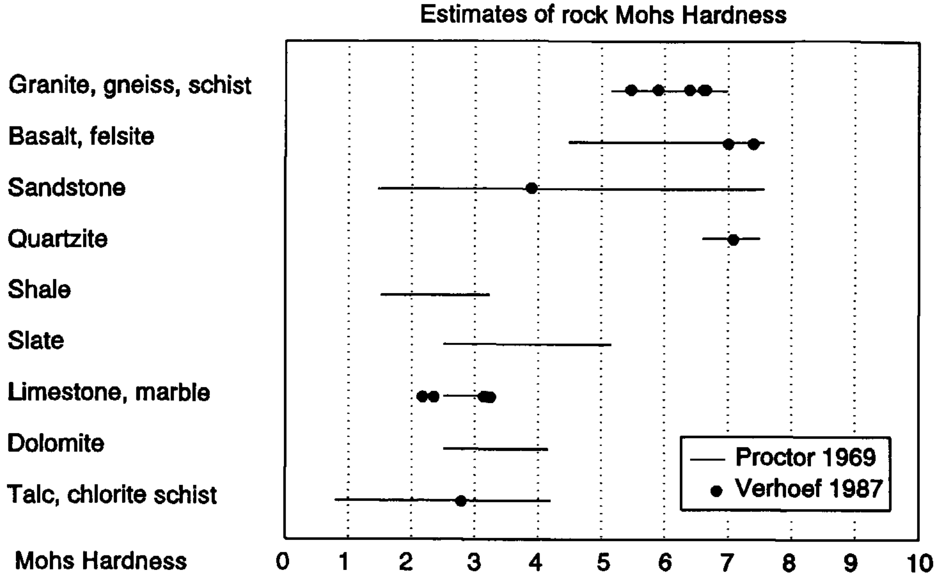

Figure 10.3 Estimates of the Mohs Hardness of rocks using the correlation of sandblast erosion resistance with Mohs Hardness (after Verhoef 1987).

hardness, showing that Mohs had carefully selected the scratching minerals.[58]

It sometimes occurs that in tender documents for a project a required rock hardness is stated in terms of Mohs Hardness, for example when it concerns armouring stone. It is peculiar to ask for a Mohs Hardness of a rock, since the scale is typically intended for single minerals. One could estimate the Mohs hardness of the rock by proportionally averaging the hardness of the minerals constituting the rock (the hardness of the minerals can be found in any book on mineralogy), but then the influence of the grain size and the bond between the grains (tensile strength) is neglected. Ranges of Mohs Hardness estimates for rocks by Proctor (1969) are given in Figure 10.3. Another way is to perform a hardness test and relate that to the Mohs scale. This has been done with the Cerchar scratch test (West 1986) and the sandblast test (Verhoef 1987). In the sandblast test a surface of rock is sandblasted under particular testing conditions. A reference glass plate is tested under the same conditions. The ratio of volume loss of rock and glass is the *sandblast index*. The test has been done both on minerals and rock. In this way the equivalent Mohs Hardness of rock has been determined, see Figure 10.3.

10.4 ROSIWAL GRINDING HARDNESS

The *Rosiwal hardness scale* is based on a test which nowadays would be regarded as a measure of the resistance of rock or a mineral against abrasive wear. Rosiwal

[58] A rough relation between Mohs hardness (MH) and Vickers hardness (HV): $HV = 31\ MH^3$ (MPa) (Uetz 1986).

Table 10.4 Hardness of common rock forming minerals and tool materials.

Mineral	Mohs H (-)	Vickers H (MPa)	Rosiwal H (-)
Talc	1	200	0.03
Gypsum	2	400	0.3
Calcite	3	1250	3
Fluorite	4	1750	4.2
Apatite	5	5500	5.5
Window glass	5.5	6000	15
Orthoclase	6	8000	35
Quartz	7	11000	100
Topaz	8	18500	150
Corundum	9	23000	850
Feldspars	6	8000	35
Clay & Mica's	2-3	600-1200	4
Carbonates	3	1000-2000	3
Dredger teeth		6000	
Tungsten Carbide		10.000 - 18.000	
pipe steel		2000	

(1896) employed a grinding test on a metal or glass disc. He used an abrasive, 0.2 mm size corundum powder (or in some cases dolomite- or quartz powder, the workability of which was calibrated against the corundum powder), which was used in very low quantities. The test specimens were pressed by hand against a rotating grinding disc (comparable to the preparation of a thin section of a rock for microscopic examination) until the abrasive had lost its workability, usually after 5 to 8 minutes. Later a grinding time of 8 minutes was taken as standard and the amount of corundum powder was specified at 100 mg (Rosiwal 1916). The test specimens had a specified surface of 400 mm^2. Nowadays also a specified normal pressure would have been used. Rosiwal expressed the abrasive loss of the numerous rocks and minerals tested in volume units and derived a hardness scale relative to corundum. This scale is still used nowadays, although the Rosiwal grinding test is not performed any more. Using the hardness data of Rosiwal (1896, 1916), the following linear regression equation resulted:

$$MH = 2.53 + 0.906 \ln RosH \qquad (10.2)$$

The coefficient of determination (r^2) is 0.88 and the standard error of the MH estimate is 0.6, the number of data pairs was 50.

10.5 COMPARISON OF HARDNESS SCALES

Table 10.4 gives indicative values of hardness that can be used to compare the hardness of minerals (or rocks consisting of these minerals) with the hardness of tool materials. The *hardness contrast* (H_a/H_m) between abrasive and tool material indicates whether a high level or a low level of abrasive wear can be expected (see Figure 4.4).

Table 10.5 compares test results of common minerals. This table is mainly presenting the data of Rosiwal (1896, 1916), which are not readily accessible. Following Schimazek & Knatz (1970), the Rosiwal hardness is given with respect to quartz = 100. When studying this table, the large spread in testing results of the hardness values is striking.

Table 10.5 Hardness of minerals, comparison of test results. Data from Rosiwal (1896, 1916), Salminen & Viitala (1985), Tourenq (1966), Uetz (1986).

Mineral	Density (Mg/m^3)	Mohs H (-)	Rosiwal H Quartz = 100 (-)	Vickers H (MPa)	Vickers H corrected (MPa)
Andalusite	3.2	7.5	*177*		
Amphibole group					
Hornblende	3.22	5-6	18-30	5990-7160	
Anhydrite	2.9-2.3	3-3.5		980	
Apatite	3.16	5	3.3-5.5	4450-5950	
Aragonite	2.95	3.5	5.1-9.9	2745	2910
Biotite	3.01	2.5-3	1.8-5.6	880-1080	
Calcite	2.72	3	1.9-3.9	1030-1690	1215
Cassiterite	6.84	6-7	*136*	6915	
Chlorite	2.78	2-2.5	0.9	490	
Cordierite	2.64	7-7.5	*45*		
Corundum	3.95	9	326-1765	17950-25485	
Dolomite	2.85	3.5-4		2140-5640	
Epidote	3.39	6-7	54-69	6670	
Feldspar group	2.6-2.7	6			
Adularia		6	*37*		
Albite	2.62	6			
Anorthite	2.76	6			
Labradorite	2.71	6	28-52		
Microcline	2.55	6		5570-8535	4340
Oligoclase	2.64	6	23-38	7875-1050	7300
Orthoclase	2.57	6	17-40	6300-9150	

Fluorite	3.18	4	2.8-4.4	1705-1775	1295
Garnet group	3.8-4.2	7-7.5	203-210	10400-12410	
Goethite	4.37	5-5.5		4975-12635	4610
Glass (window)	2.51	5.5		4905-6375	4905-5790
Gypsum	2.32	1.5-2	0.3-0.6	305-470	
Halite	2.16	2.5	1.2-1.4		
Hematite	4.9	5.5-6.5	25	8025-13479	6915
Ilmenite	4.7	5-6		4915-10760	9055
Kyanite	3.5-3.7	5-7	*53*	4905-21030	
Leucite	2.47	6	*40*		
Magnetite	5.15	5.5	34	4870-9280	5700
Muscovite	2.85	2-3	1.5-6.1	835	
Olivine group	3.41	6.5-7	80	8045-9610	
Pyroxene group	3.2-3.5	5-6			
Augite	3.2-3.4	5-6		6280-6375	
Diopside	3.2	5-6		7850	
Hypersthene	3.4-3.5	5-6		5885	
Pyrite	5.1	6.25	56	10075-32330	9515-14060
Quartz	2.65	7	74-127	10810-15600	8830-13320
Rutile	4.25	6-6.5	116-143	9150-14085	
Scapolite	2.65	5.25	12-19		
Serpentine	2.50	3-4	*4*	1715	
Spinel	3.5	8	*389*	13515-14765	
Staurolite	3.74	7-7.5	83	12950	
Talc	2.74	1	*0.03*	20-460	
Titanite	3.48	5.25	*48*	3965-6670	
Topaz	3.56	8	87-148	10890-17835	18150
Tourmaline	3.15	7-7.5	102	11645-13520	
Zircon	4.45	7.5	*367*	10928-14765	

Rosiwal Hardness with respect to quartz = 100. Single numbers are in most cases single test results, averages are indicated by italics.

CHAPTER 11

Tests to determine the abrasiveness of rock

Although *abrasiveness* is a generally used word and certain rock types are described as *abrasive*, the implication is not straightforward. Abrasiveness is an *interaction* property. Whether a rock is abrasive or not depends on both the properties of the rock (the *counter body* in tribology terms, Figure 4.1) and on the properties of the tool (the *solid body*) under the prevailing conditions of temperature and pressure during cutting. When abrasiveness is discussed, always this combination should be kept in mind. For example, quartz is abrasive with respect to steel, but not to tungsten carbide (at room temperature and pressure). The latter statement is based on the *hardness contrast* (Figure 4.4) that exists between quartz and steel, respectively tungsten carbide (Figure 9.18). The harder the constituent minerals of a rock, the higher the chance that the rock will be abrasive to certain tool materials. Since abrasiveness is considered to be an interaction property, it may also be defined as *abrasive capacity*.

The common approach to date to assess abrasiveness of rock is by performing tests on the rock material and measure the wear of the tool material used in the test. Many types of test have been developed. The problem with these tests is that the results are dependent on the experimental conditions, as was concluded in Chapter 4. In the initial stage of the research on this subject, it was assumed that it would be possible to select suitable wear tests, addressing either two-body abrasion or three-body abrasion, see Figure 11.1. The abrasive capacity of the rock would then be assessed by evaluating the results of these tests (Verhoef 1988). Research carried out in Norway to determine the drillability of rock for Tunnel Boring Machines (TBM's) acted as an example of this approach. A set of experiments was used to assess drillability, Figure 11.2:

1. A laboratory drilling test (Siever's test, which relates to the rotation cutting action of the TBM).
2. A determination of brittleness index by a dynamic crushing test (relating to percussion action and impact).
3. A bit wear test (a three-body abrasion test).

A data base has been built up and a set of rules was derived that is used in a model to predict TBM performance (Bruland et al. 1988, Johannessen 1990; see Chapter 18.1)

In Delft the author attempted to use the pin-on-disc test as a basic tool to measure

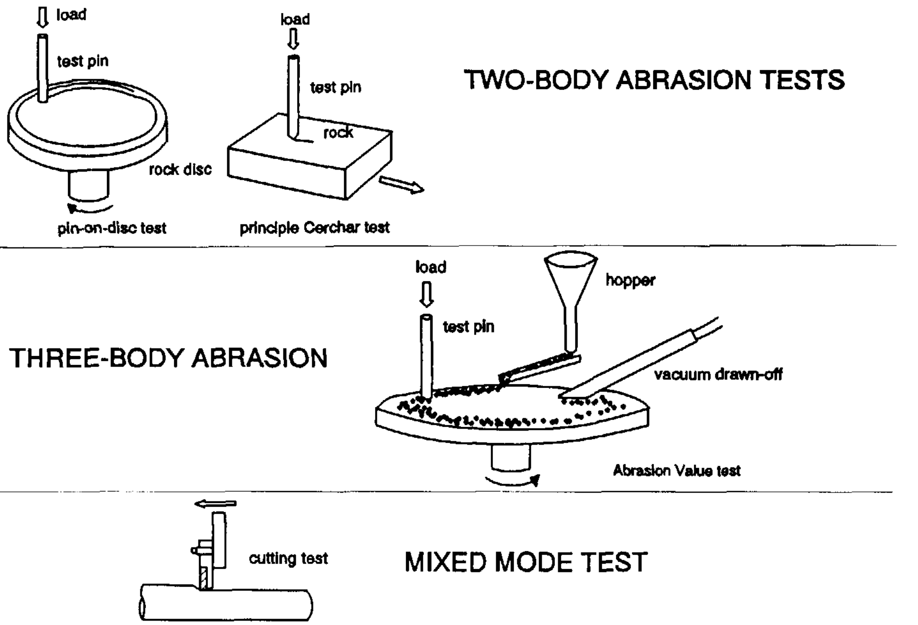

Figure 11.1 Test methods used to assess abrasiveness of rocks arranged according to wear mechanism.

both two-body- (pin directly tested on rock) and three-body abrasion (pin tested against rock powder). Typical two-body wear tests, like the Cerchar test (Chapter 11.2), Schimazek's test (Chapter 11.3) and pin-on-disc test (Chapter 11.4) have been studied. In these tests (Figure 11.1) a pin of a specified steel type is pressed against a rock surface. The slice of rock is placed on a turning table, or is moved linearly under the pin. Commonly the load, the (rotation) velocity and length of scratching is specified and the mass loss of the pin is measured. Three body wear is occurring when abrasive grains are present between the test body of steel and the counterbody. Examples of three-body wear tests are the Rubber Wheel Abrasive test (ASTM G65-81, Mishra & Finnie 1980) and the Abrasive Value test (Movinkel & Johannessen 1986), see Figure 11.2. It is possible to use ground rock powder and test this against standard steel or tungsten carbide pins or test bodies. While performing the research on the two-body abrasion tests, cutting tests of chisels with machined wear flat were performed at the Mechanical Engineering Faculty of Delft University of Technology (Chapter 11.5).

These tests have been used to study the influence of rock properties. Artificial rock has been used in order to be able to study the effect of certain rock properties, keeping other rock properties constant. It is generally known that the *abrasive capacity* of a rock is related to *petrographic* and *rock mechanical properties*, such as:

- Mineralogical composition (hardness of the minerals).
- Grain size and grain shape of the constituent minerals.
- Microscopic structure (grain configuration, cracks, anisotropy).
- Bonding between grains (related to the strength of the rock).

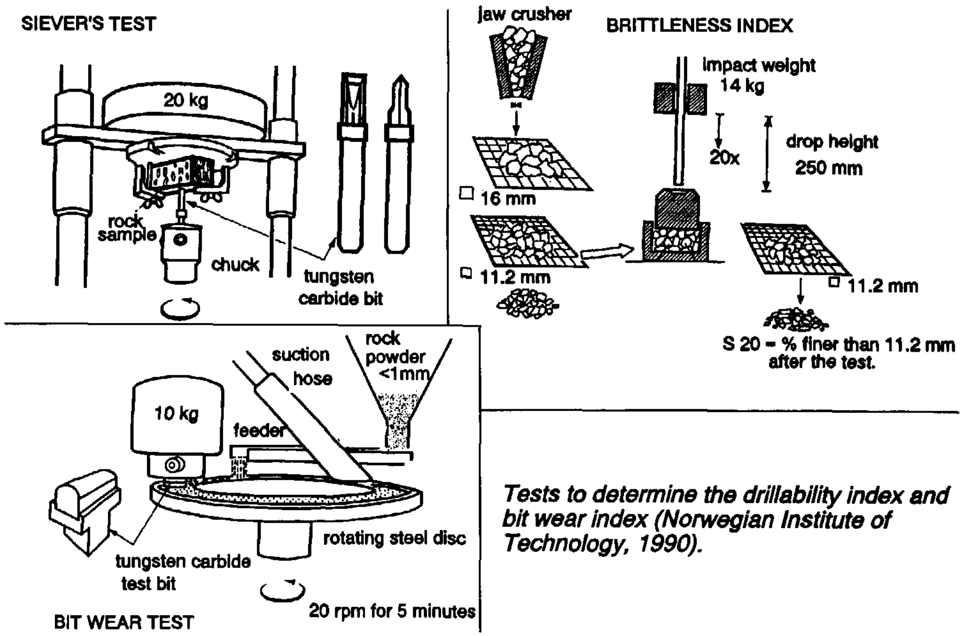

Figure 11.2 The laboratory tests developed in Norway to assess drillability of rock (after Johannessen 1990).

The first three items listed are petrographic properties, they can be described using microscopic observation techniques. The last property is usually assessed by rock strength testing (Chapter 22, Appendix C).

11.1 THE CORE ABRASION TEST

To investigate the contribution of abrasive wear to the total chisel wear in the rock cutting test (Chapter 9.7), the *core abrasion test* was developed (Roxborough 1987). The test is performed on a core of the same rock as tested in the core cutting test. The core is placed in a lathe rotating at 50 rpm and feeding the tungsten carbide insert which was used during the cutting test at an angle of about 45° axially along the outer surface of the core (Figure 11.3). The forward feed is taken at 0.1 - 0.2 mm per revolution, which ensures that virtually all wear is the result of abrasion, because the depth of cut used and the forces are very low. This way lengths of 25 up to 100 m of rock surface may be abraded. The abrasive wear can be expressed in weight loss of carbide divided by cutting length (mg/m) and this number may be compared with the cutting wear loss determined by the cutting test, to appreciate the relative contribution of abrasive wear to the latter.

As described in Chapter 9.7, the abrasion test was performed on rock cores of the Sydney Harbour project, adjacent to the ones of the cutting test. It was attempted to do the tests also on the cores remaining from the cutting test, but it was necessary to first grind down these cores to remove the grooves remaining from the cutting

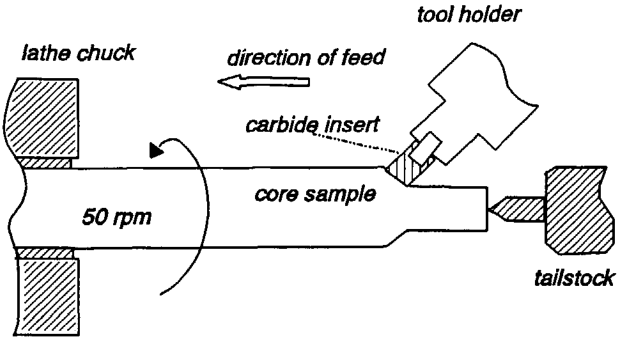

Figure 11.3 Test arrangement of the core abrasion test.

test. Generally the cores did not survive this procedure. Carbide test bits used for the cutting test were taken for the abrasion test. The minor wear that resulted from the cutting test compared to that of the abrasion test made this allowable. The results of the tests are summarized in Table 9.3 & 9.4. In examining the cutting and abrasive wear values, one should realize that the density of tungsten carbide is about twice as high as that of steel (14.59 versus 7.85 Mg/m^3). The difference in wear due to the use of another tool material can be seen if volume loss per cutting length is compared for the steel and carbide bits. Therefore in Table 9.4 the rates of volume loss are also given. The wear rate of the steel bits that occurred during the cutting test was about 2.4 times higher and during the abrasion test about 38 times higher compared to the tungsten carbide bits. This clearly illustrates the role of the tool material and the actual wear mechanism operating. Table 9.4 also shows that the magnitude of the forces and specific energy measured on the same rock is about identical when different tool materials are used, despite the difference in wear rate. The cutting forces measured using the steel inserts are somewhat higher, which can be attributed to an increased size of the wear flat of the chisel (blunting of the tool).

During the execution of the core abrasion test, the feeding was carried out by hand. This is an unsatisfactory procedure, as the experiments reported by Deketh (1995) show that at low values of feed changes of wear mechanism (from adhesive to abrasive) may rapidly occur (Chapter 12).

11.2 THE CERCHAR SCRATCH TEST

The Cerchar test was developed by the French coal mining research institute *CERCHAR* (Centre d'études et recherches des charbonnages de France). It was meant to give a measure of the abrasiveness of rock. The test was not meant to determine rock hardness (in the sense of *strength*), for this purpose another test determining the penetration rate of an indenter in rock was used.[59] The test has been used extensively both in the coal mining industry and in the tunnelling industry.

[59] The hardness test and the abrasiveness test were used to examine the *workability* of the rocks with respect to machine excavation (CERCHAR brochure, 1980).

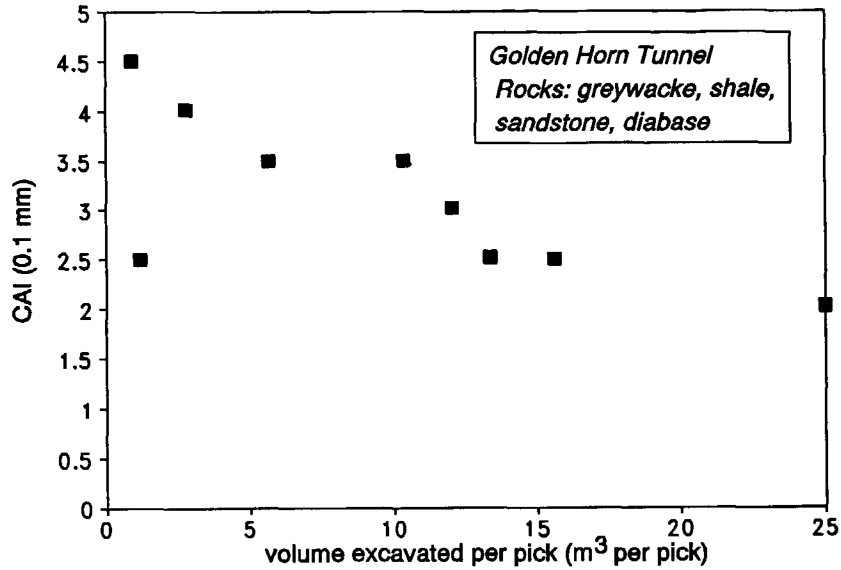

Figure 11.4 Tool consumption of roadheader compared with Cerchar test values for a greywacke, shale, sandstone and diabase rock mass (Golden Horn tunnel, Turkey; Bilgin et al. 1988).

West (1989) describes the testing procedure and gives many test results on different rock types. A sharp steel conical point (90° conus angle) is pressed into the surface of a rock specimen under a load of 70 N. For soft rocks the rock surface is prepared with a file, for harder rocks a saw cut is used (West 1989). A convenient size of sample is clamped into the vice, taking care that the surface is level. The test pin (stylus) is placed in the holder and loaded with a mass of 7 kg. The rock is then slowly linearly displaced by 10 mm. After cleaning any rock debris from the conical tip of the stylus, its wear flat is measured using a microscope fitted with a micrometer. Two measurements across opposite diameters are made and the mean value is taken. The unit of abrasiveness, CAI (Cerchar Abrasiveness Index), is defined as a wear flat of 0.1 mm diameter. Five tests are normally carried out on each rock specimen to give a reasonable average value for samples with a grain size less than 1 mm. Some rocks are so hard that the stylus is unable to cut a groove and although the steel is blunted, it has not interacted properly with the rock to form a genuine wear flat. Examination of the groove, to ensure that the tool has bitten into the rock, can be done by hand lens. If no groove is formed, the rock should be reported as too hard to test (West 1989). The steel of the test stylus was defined by *CERCHAR* as steel of 2000 MPa tensile strength.

The Cerchar test has been used in rock tunnelling practice, to describe abrasiveness of rock and its usefulness for this purpose has been advocated by Suana & Peters (1982) and Büchi (1984). An example of a relationship between tool consumption rate and CAI value is given in Figure 11.4

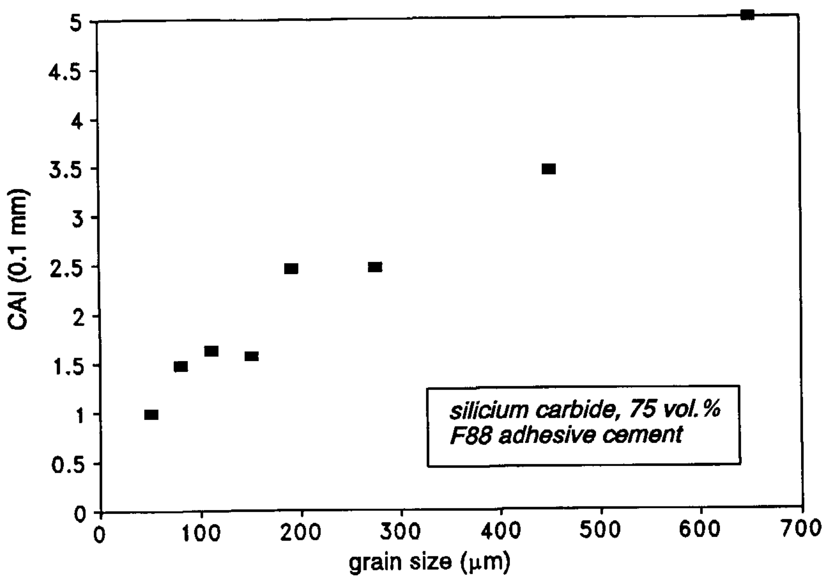

Figure 11.5 Effect of grain size on the Cerchar Abrasiveness Index (CAI).

11.2.1 *Comparison of Cerchar tests results from different laboratories*

Test apparatuses using this principle have been built in various institutions. One problem that has been noted was that, for comparison of values, *exactly the same steel type* should be used. This has proven to be a problem. Suana & Peters (1982) use pins of 2000 MPa steel with a Rockwell hardness of 54-56. West (1986, 1989) and Al-Ameen & Waller (1994) used Steel EN 24, heat treated to a Rockwell hardness RH_c of 40, which value was chosen to obtain similar results on granite as reported by *CERCHAR*. In Delft 42CrMo4 steel was used, heat treated to 54-56 RH_c (tensile strength 1000 MPa). Very often the steel type is not specified accurately enough. This makes that values measured with different apparatus cannot be compared without discrimination. After testing the pins are sharpened on a lathe for renewed testing. This should be done carefully under controlled temperatures and with oil as coolant, otherwise the hardness would be affected. Al-Ameen & Waller (1994) noted that the EN 24 steel pins they used were not of constant hardness, a very large spread around the expected value of 6000 MPa (HV) was found (3400 - 7850). Probably poor quality control during heat treatment was the cause of this variation. Since the weak coal measure rocks they investigated had low CAI values (below 0.15), they choose the softer steel type EN 3 to perform the test. This steel had a much lower variation of HV (2200 MPa $\pm$ 40). Using this steel type good results were obtained.

In Delft, when freshly machined points were examined by microscope before testing it was found that not all points were sharp. The bluntness varied from 5 microns up to 70 microns. This affects the variation in test results. It is good practice to examine the points before testing with a hand lens and have bad points resharpened. Another problem that occurs is the development of an irregular wear flat, or wear flats occurring at several depth levels. In such a case it is arbitrary which diameter should be measured.

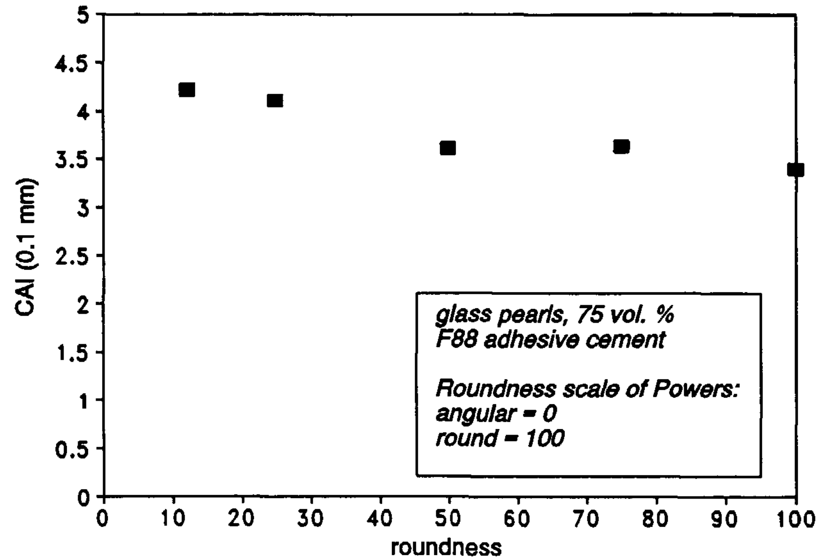

Figure 11.6 Effect on roundness of grains on the Cerchar Abrasiveness Index (Jager 1987).

11.2.2 *Influence of rock parameters on Cerchar test results*

Experiments were made to show the effect of some rock properties on the outcome of the Cerchar test (Jager 1987 and Reinking 1989). The first rock property examined was the *grain size* of abrasive minerals. Well sorted size fractions of silicium carbide grains were used, which were mixed with non-epoxy adhesive cement F88. The volume percentage SiC was 75%. Cerchar measurements were made on surfaces of this material, see Figure 11.5. An increase of CAI with grain size is noticed. With increasing grain size, the rate of increase of the wear flat size declines.

The second rock property examined was the *roundness* of the grains (Figure 11.6). One would expect that angular grains give more wear than rounded grains. This was examined by preparing mixtures of mortar and glass particles of varying roundness. A slight decrease in wear was measured with roundness, less than expected. The explanation is that if the pin cuts the grains, roundness may not much affect wear. The scraping experiments of Deketh (1995) confirm that once the tool starts to cut the rock, there is no influence of grain angularity.

The effect of *rock strength* was examined by performing tests on a hardening mortar-quartz mixture of constant mineralogical composition. Figure 11.7 shows that the CAI increases linearly with strength. The conclusion from this is that, apart from mineralogy, strength indeed influences the result of the measurement. Al-Ameen & Waller (1994) concluded in a comprehensive study on the application of the Cerchar

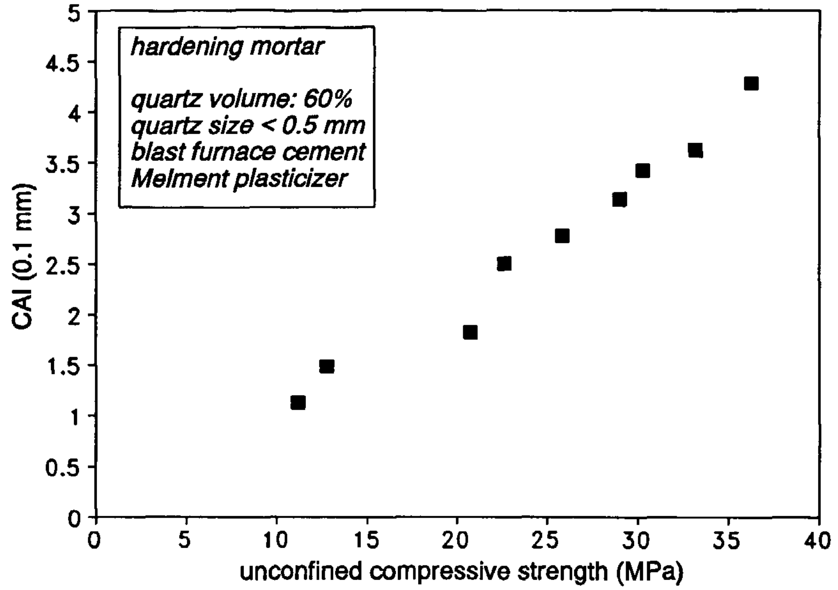

Figure 11.7 Effect of strength on Cerchar Abrasiveness Index, determined on a hardening mortar-quartz mixture (Jager 1987).

test to natural rocks that the CAI is related to rock strength and to *abrasive mineral hardness*[60] in the following manner:

$$CAI_{(EN3,10mm)} = f\,(UCS\ (1 + \sum Mineral\ Hardness)) \qquad (11.1)$$

Note that the experiments were done with the soft EN3 stylus, while the other test conditions were the same (7 kg mass, 10 mm sliding). The rocks were limestones, ironstones, some mudstones and seatearth. Most of the abrasive rocks were fine grained, so probably the effect of grain size on wear is not noted in these test results (Figure 11.8).

11.2.3 *Critique on the Cerchar test*

Some results of CAI measurements that are published in literature are given in Table 11.1. The CAI values in this table are all obtained using the hard type stylus. It must be clear by now that these results should be treated with caution, because they are dependent on test apparatus and steel type used.

Lislerud (pers. comm. 1990) stated that the Cerchar test was not used in Norway (University of Trondheim, Construction Engineering), because not enough differentiation between hard rocks (granite, gneiss, quartzite) was found using this

[60] The mineral hardness is determined by proportional summation of the Mohs Hardness of the abrasive minerals in the rock. Abrasive minerals are defined as minerals in the host rock whose hardness equals or exceeds that of the tested material (i.e. mild steel). The mineral composition is determined by petrographic examination (Al-Ameen and Waller 1992).

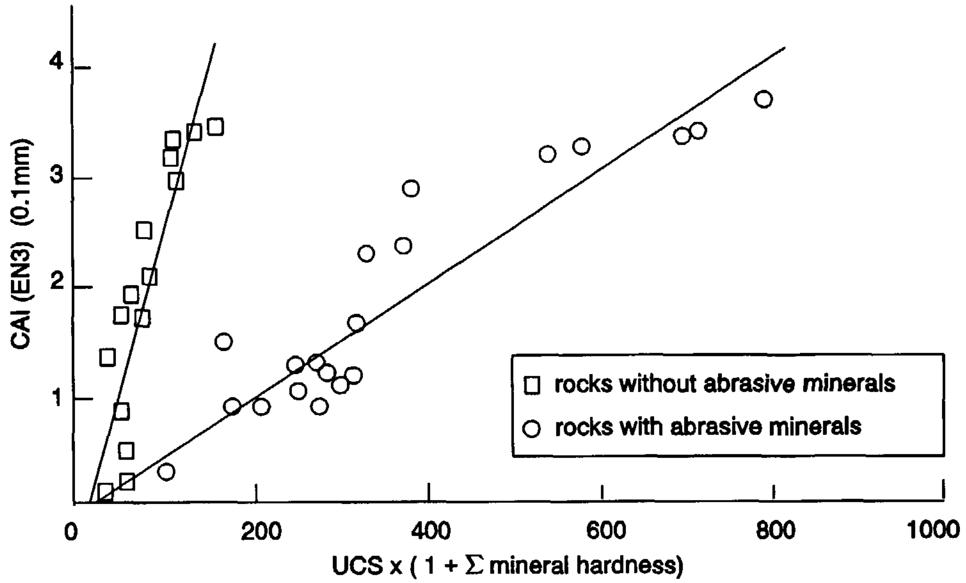

Figure 11.8 Results of Al-Ameen & Waller (1994) with adapted Cerchar test, showing effect of strength on the CAI value (see text).

test. Sindhusen (1991) did not get good results on hard rocks as well. She attributes this to the inability of the tool to cut a groove in these rocks. Al-Ameen & Waller (1994) note that the stylus tends to slide over the rock surface of hard rocks. The surface roughness then determines the amount of wear occurring, normally leading to CAI values lower than expected. Another remark made by Sindhusen (1991) was that the Cerchar abrasiveness indices have to be viewed with caution, because the stylus tends to dig in, particularly if the matrix material is soft and easily penetrated. Consequently, the conical stylus sinks deeper into the specimen, redistributing the load onto the sides of the cone away from the apex, thus indicating an abrasive index value lower than expected. Similar observations by Al-Ameen & Waller (1994) led them to compare the CAI after 1 mm sliding with the CAI after the standard 10 mm of sliding. They found that 70 % of the mass loss occurred in the first mm of sliding, probably due to breakage of the point at the digging-in phase and rapid development of a wear flat. With increasing size of the wear flat of the pin, the pressure under the flat will decrease, which is a problem that the Cerchar test has in common with the Schimazek test discussed in the next section.

Examination of this simple laboratory test reveals problems that are typical for wear studies. Apart from problems that may be easily overlooked in the laboratory (like varying hardness of the hardened test styluses, or lack of calibration of the measuring microscope), comparison of test results made in different laboratories is often not allowed, because different steel types are used for the stylus.

Table 11.1 Ranges of CAI results for different apparatus using the Cerchar test principle.

Mineral	Suana & Peters (1982)	West (1989)	Sindhusen (1991)	Reinking (1989)	Al-Ameen & Waller (1994)
Quartz	5.6-6.0	4.7-6.9		3.0	
Feldspar	4.2-4.8	4.8-5.9		3.9	
Glass				1.8	
Apatite		2.7-3.7			
Fluorite		1.4-2.2		2.1	
Calcite	1.0-1.2	1.0-1.8		1.8	
Gypsum		0.1-1.7			
Rock type					
Quartzites	5.6-6.0	too hard	too hard		
Anorthosite (Feldspar rock)	4.2-4.8				
Dunite (olivine rock)	3.4-3.6				
Pyroxenites	3.0-3.2				
Amphibolites	2.8-3.2				
Serpentinites	1.4-1.8	1.8			
Claystones	<2.5	<2.6			0.1-0.7
Siltstones					0.3-0.6
Sandstones		1.3-6.3	1.6-8.3	1.0-1.4	0.7-3.3
Limestones	1.0-1.2	<2.4	4.3	0.3-0.8	0.5-1.1
Siliceous Limestones				0.5-2.7	
Granites		4.5-7.3	too hard		
Dolerites		3.8-5.4			
Basalts, Andesites		3.5-5.3			

11.2.4 *Conclusions on the Cerchar test*

- The test result depends on steel type used for the stylus and when hardening is used a high variation in steel hardness might be present.
- The pin should dig into the rock while sliding. If the rock is too hard, rock surface roughness determines the wear of the pin. CAI values tend to be too low and the test is unsuitable. The pressure applied by the stylus should exceed the strength of the rock.
- During the development of a wear flat, or when the rock is very weak, the contact pressure will vary during the test, due to the increasing contact area between pin and rock surface.
- A major factor contributing to wear is the strength of the rock, wear is not solely due to the abrasive mineral content and grain size of the abrasive minerals.
- The relationship of the Cerchar index with wear of cutting tools of excavation machines like dredgers will be very tenuous, since many factors affecting the

behaviour of the rock differ in practice. For example, the cutting mechanism, the loading-displacement rate, the temperature during the cutting process etc.

11.3 SCHIMAZEK'S TEST AND THE WEAR FACTOR F

11.3.1 *Schimazek's pin-on-disc test*

Another method that is considered to be of value was developed by Schimazek and co-workers (Schimazek and Knatz 1970, 1976, Paschen 1980). The test performed by Schimazek is based on the use of a turning table, as commonly applied in grinding and polishing. A disc of rock is placed on the table. The rock surface first is polished with 240 SiC powder. A 10 mm diameter pin made of readily wearing St 50 wedge steel of 700 MPa tensile strength with a 90° conical point flattened to 0.3 mm, is placed in a holder and loaded with a mass of 4.5 kg. The pin holder moves radially outward during a test, which results in the pin describing a spiral path on the rock surface, cutting or scratching continuously in fresh rock. The distance between the cutting grooves is 0.5 mm and the number of revolutions of the disc is 100. The rotation rate is 25 rpm. The total distance travelled by the pin is 16 m. After a test the mass loss of the pin is determined. On each rock type 10 tests are conducted and the average mass loss is reported. Contrary to the Cerchar test, the pin is made of weak steel, to increase the amount of wear to be measured.

Schimazek & Knatz (1970) performed this test on artificial rock (concrete with a variation in quartz content and grain size). Comparing the measured mass loss of the pins with the variation in tensile strength, quartz content and grain size of the artificial rocks, they found no correlation with either tensile strength or hardness, but the product of the above factors had a linear correlation with the mass-loss of the steel pins. They called this product the wear factor "*F*", where *F* is defined as follows:

$$F = \frac{Qtz\ eq\ *\ \varnothing\ *\ BTS}{100}\ (N/mm) \tag{11.2}$$

where *Qtz eq* = mineral content expressed in hardness of Quartz using Rosiwal's hardness scale (in vol.%), ∅ = mean diameter of mineral grains[61] (in mm) and BTS = Brazilian tensile strength (in MPa). Although no statistical information is given in the original paper, the fit of the data for the artificial samples is particularly good and holds at least for range of rock properties used (Quartz content: 0 - 70 vol.%, Quartz grain size: average ∅ = 0.02 - 0.45 mm, BTS: 1.6 - 4.4 Mpa).

The Schimazek test has been performed on natural rocks as well (Schimazek & Knatz 1970). The rocks were Carboniferous sandstones and shales from the Ruhr-

[61] Schimazek & Knatz (1970) determine the average grain size using Rosiwal's line intercept method, by measuring the number of intersections of grain boundaries along parallel measuring lines laid over a thin section studied by microscope. The average intersection length is multiplied by a factor of 1.4 to estimate mean grain size. In this study the author used a factor of 1.5, following DIN 22021 (draft, 1981). Paschen (1980, p.181) uses the mean intersection length to calculate F, without correction, hence he gets lower F-values.

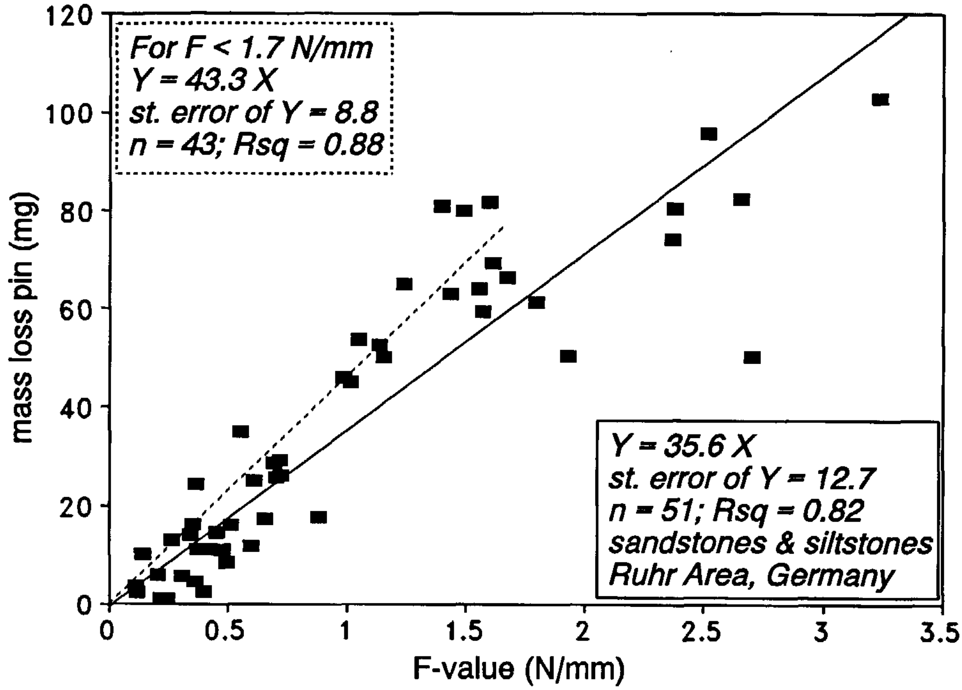

Figure 11.9 Results of the Schimazek test on Carboniferous rocks from the Ruhr area. Data from Paschen 1980.

area, the data on these rocks were kindly provided for this study by Dr. Paschen. Paschen (1980) examined the statistical value of Schimazek's *F*-factor and found that the *F*-value correlated well with the wear measured with Schimazek's pin-on-disc experimental set-up (Figure 11.9). Paschen (1980) noted that the deviation from linearity at high *F*-values is caused by a group of strong sandstone rocks. If these were left out, what actually means leaving out the *F*-values > 1.7 N/mm, a better linearity and a higher correlation was found (Figure 11.9).[62]

11.3.2 *Determination of the F-value and some applications*

To determine the wear factor, thin sections of the rock materials under consideration have to be prepared for microscopic examination. Using the petrographic microscope, the mineral content and the grain size of the minerals can be determined. Thin sections can be made of the same rock disc on which the Brazilian tensile strength test is carried out. For example, a sandstone may have the following composition:

[62] F-values were calculated using the original data (applying a correction factor of 1.5 for the grain size estimation from the line intercept method, see previous footnote). Hence, these values are 1.5 times higher than calculated by Paschen (1980) and 1.5/1.4 times higher than those published by Schimazek & Knatz (1970).

Mineral	*Volume (%)*	*Grain size (mm)*	*Rosiwal Hardness (Table 10.4)*
Quartz	*20*	*0.5*	*100*
Feldspar	*30*	*0.4*	*35*
Calcite	*20*	*0.1*	*3*
Clay minerals	*25*	*<0.05*	*4*
Porosity	*5*	*<0.05*	*0*
	100%	*BTS rock (saturated) = 15 MPa*	

The F-value is then calculated as follows (Equation 11.2):

$$F = \frac{[(20*1*0.5)+(30*0.35*0.4)+(20*0.03*0.1)+(25*0.04*0.05)+(5*0*0.05)]*15}{100} = 2.1\ N/mm$$

Note that the Rosiwal hardness of Quartz is set to 1. That of Feldspar is then 0.35, Clay 0.04 and Calcite 0.03; Table 10.4, p 123. It is clear that the hard minerals of larger grain size determine the value of F. In many cases it suffices to use only the percentage of hard minerals in the rock and their mean diameter to calculate F. Apart from Quartz and Feldspar, minerals like Pyroxenes, Amphiboles (Hornblende) and Pyrite are examples of hard minerals that commonly occur. Their Rosiwal Hardness may be found in Table 10.4 & 10.5.

A suggestion of a classification of the abrasive capacity of rock, based on the F-value, is given in Table 11.2.

The microscopic study of thin sections of the rocks to be cut can provide further information related to the machinability of the rock. Information like:

- distribution of minerals, pores and structures (the *microscopic structure* or *texture*), giving information on homogeneity, anisotropy
- presence of microcracks, fissures
- observations giving information on the type of bonding of grains, the presence of cementing minerals etc.

The F-value has become a popular parameter used to assess abrasiveness of rock. The performance of road headers and other mining machines in rock were compared with the wear value F and this proved useful. *Voest-Alpine*, manufacturer of road headers (Alpine Miner), has developed graphs that estimate pick point consumption based on the F-value of Schimazek. Two major causes of pick point consumption are

Table 11.2 Classification of the abrasive capacity of rocks based on the F-value. The order of pick point consumption of cutterheads on CSD's is based on limited data known to the author.

F-value Schimazek	Abrasiveness	Order of pick point consumption
$F < 0.05$ N/mm	low abrasiveness	100 m^3/p.p.
$0.05 < F < 0.1$	abrasive	
$0.1 < F < 0.5$	highly abrasive	
$F > 0.5$ N/mm	extremely abrasive	10 m^3/p.p.

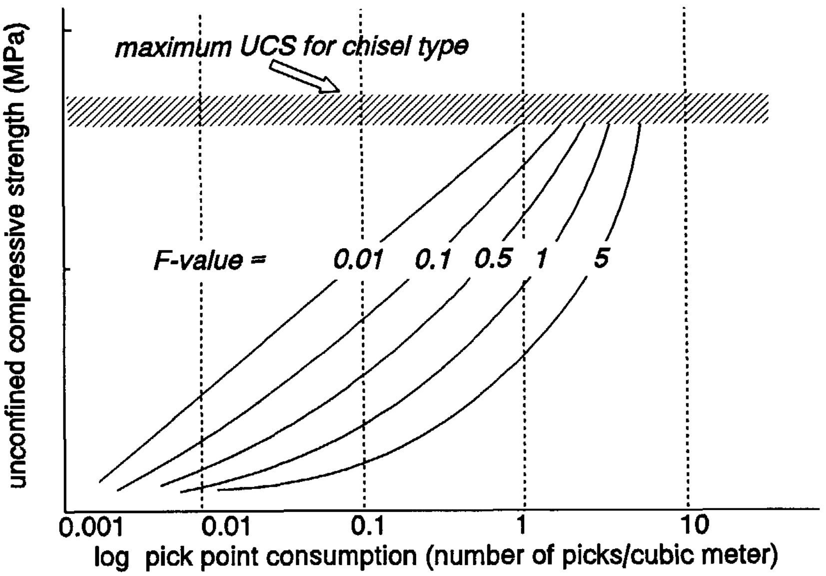

Figure 11.10 The type of graph Voest-Alpine is using to give predictions of tool consumption. Graphs exist for different types of pick points on various machines (Gehring, pers.comm. 1991).

considered: breakage due to high rock strength and abrasive wear. Breakage is evaluated against the Unconfined Compressive Strength of the rocks; depending on the robustness of a particular pick point type, a maximum rock strength can be ascribed to each type of pick point. A data base of excavation cases enabled the company to design graphs showing the increase of pick point consumption due to abrasive wear with increasing *F*-values (Gehring 1991, pers. comm.), see Figure 11.10.

Other work has confirmed the utility of the *F*-value as an aid in evaluating abrasiveness of rock. Its main value lies in the combination of petrographic and rock mechanical information contained in the factor. However, if only abrasion tests and rock mechanical tests were carried out to give information on the expected rock cutting characteristics, it would be easy to overlook other important information.[63]

Schimazek & Knatz (1970) suggested a relation of the *F*-value with the critical velocity, above which excessive wear of cutting tool material occurs. They used the data provided by Krapivin et al. 1967, which are presented in Figure 9.17. The expression of this relationship (Figure 11.11):

$$v_{crit} = k\ e^{-F}\ (m/s) \tag{11.3}$$

where v_{crit} is the critical velocity in m/s, k a coefficient that contains chisel geometry and the critical temperature of the chisel material. A practical application of this suggestion has not been encountered as yet. Closer examination of the data on which the relationship was based shows why. Figure 11.11 illustrates that the error in the estimate of k for the chisels used by Krapivin et al. (1967) is so large, that the relationship cannot be of practical value.

[63] The use of the F-value alone would disregard important factors such as discontinuity spacing or excavation machine characteristics.

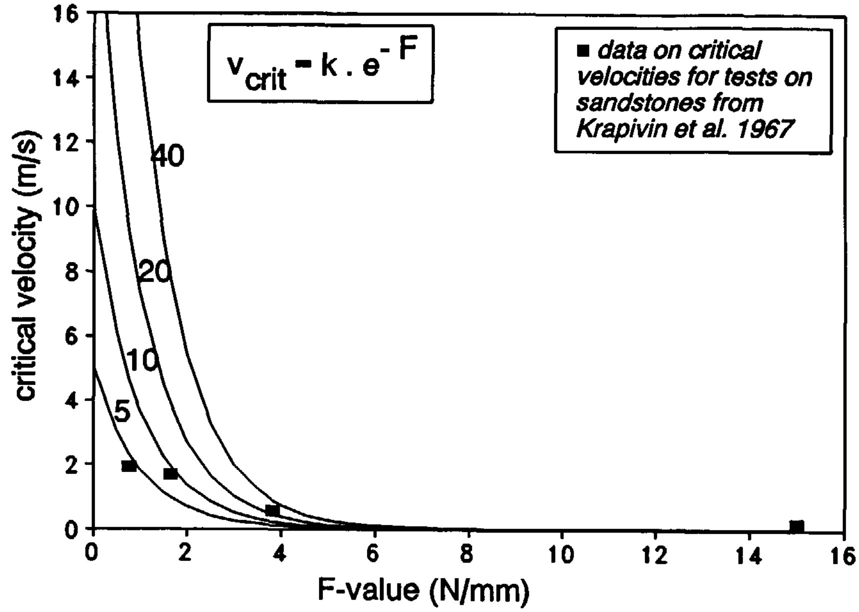

Figure 11.11. Estimate of critical velocity by Schimazek & Knatz (1970). Data from experiments of Krapivin et al. 1967 (see Figure 9.17). Curves for different values of k are shown.

The use of the *F*-value as a number indicating the abrasiveness of rock is further examined in this study. Contrary to the popularity of the *F*-value, the *pin-on-disc test* of Schimazek is not commonly used.

11.4 DEVELOPMENT OF A PIN-ON-DISC TEST

The pin-on-disc test was chosen for further examination. Some problems with the Schimazek pin-on-disc test had been identified. Paschen (1980) started to use a lathe, where the pin would horizontally press into the rotating rock disc (Figure 11.12). Test conditions were changed to a constant normal pressure on the pin of 8 bar (78.5 kPa) and 8 m sliding distance. The reason to choose a lathe was to prevent rock particles loosened by the pin interfering with the pin wearing process (Paschen 1980), but the presence of loose rock particles was not the only problem. Due to the conical shape of the pin, the contact pressure under the pin would change, when the wear flat increased in surface area during the test. To have relatively uniform contact stresses under the pin, that would remain constant during wear, a pin of cylindrical shape and flat surface area was used in a test series carried out in Delft (Van den Bold 1986, Vermeer 1989, Verhoef et al. 1990). A test arrangement using a lathe following the example of Paschen was used, but a constant force was applied to the sample and the sliding length was 36 m. Van den Bold (1986) and Vermeer (1989) carried out tests on artificial rock specimens made from mortar-glass mixtures. The first type of pin used was made of the soft steel type St 37 (tensile strength 360 MPa) and had a square (3.95 x 3.95 mm) prismatic shape. The following variables were investigated:

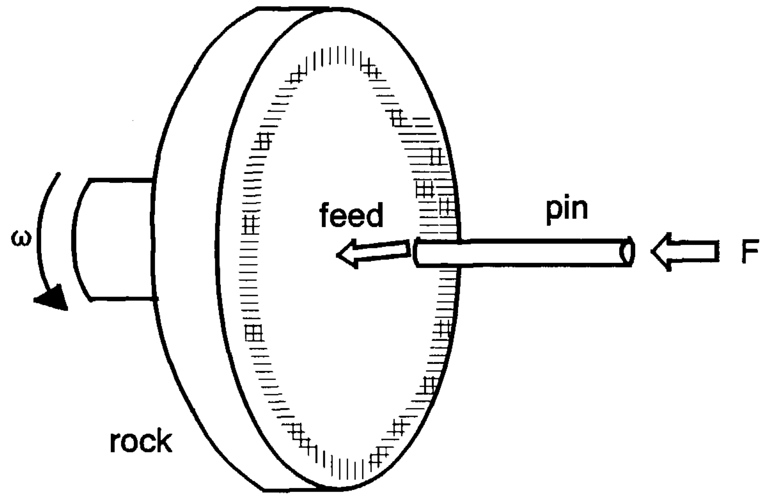

Figure 11.12 The principle of the pin-on-disc test on a lathe.

- Grain size of glass particles
- Volume percentage of glass particles
- Roundness of particles
- Velocity of pin against the abrading surface
- Applied normal pressure of the pin against the sample

The results of the experiments carried out using this test arrangement have been reported in Verhoef et al. (1990). Several problems were revealed by these experiments, related to both the experimental design and the properties of the artificial rock samples:

- The flat pin surface was sliding over overlapping area and polishing of the surface occurred by steel particles.

- The wear mechanism on the wear flat surface was not homogeneous; wear in the frontal part was different from the rear end.

- The test velocity had to be kept below a threshold value, to keep the temperatures below the softening threshold for the steel.

- Roughness of the surface influenced the result (sample preparation method was of influence).

- The normal stress that could be applied was limited by the strength of the sample.

Due to these problems the results of the tests are only of limited value. The purpose of the tests was to study the difference in wear due to the shape of the abrasive grains (angular or round) in the rock. It was intended to study the effect of grain shape by only varying this property, while keeping the other properties of the rock constant. The strength properties of samples with round or angular grains (with all other properties equal) differed, however, with the angular glass concrete being stronger in compression and weaker in tension than the round glass concrete (see Verhoef et al. 1990). As with Schimazek's test, the F-value showed a linear correlation with the mass loss measured on the pins. Figure 11.13 illustrates that no difference is apparent between concrete with round and angular glass grains at 145 rpm. Interestingly, concrete containing round grains appears to give more wear at

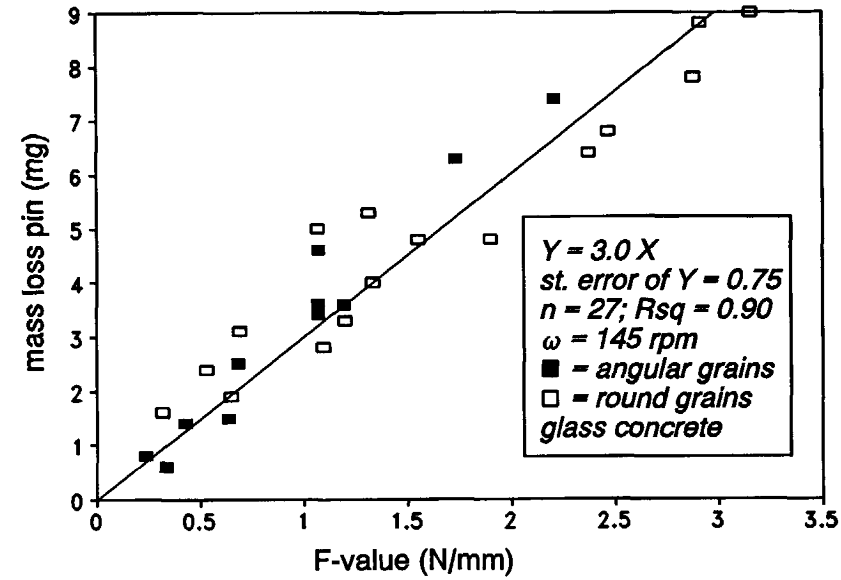

Figure 11.13 Results of pin-on-disc tests on glass concrete with angular and round glass grains, rotation rate lathe 145 rpm (Verhoef et al. 1990).

a rotation velocity of 275 rpm, at lower values of *F*, Figure 11.14. The interpretation of these results is hampered by the experimental problems and errors summed up before, but it is shown that a small change in test conditions, namely rotation speeds of 145 rpm (mean tangential velocity 0.2 m/s) and 275 rpm (mean tangential velocity 0.3 m/s), can lead to variation in response of rock types only differing in microscopic structure. These observations indicated that the *F*-value as such does not describe wear completely. The use of a factor like *F* assumes homogeneity of the rock. Structure (anisotropy) or inhomogeneity on sample scale is not represented in the factor, but is very probably of influence on both cutting and wear behaviour.

These pin-on-disc tests showed that the attempt to work with constant stress under the pin and thus ensuring constant test conditions with respect to this variable led to a new range of problems. In particular the overlapping of sliding paths had to be avoided and also the effect of surface roughness. Nevertheless it was thought that the test would be useful to measure the effects of a controlled two-body wear process.[64]

11.4.1 *Optimization of the pin-on-disc test*

A study was performed to optimize the pin-on-disc test for the purpose of studying abrasiveness of rock (Deketh 1991). A special lathe was used with a provision to maintain a constant tangential velocity under the pin, while it moves inward. In all

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[64] It was thought that, by feeding abrasive rock particles between pin and rock, also a three-body wear situation (see Figure 11.1) could be created using the same test configuration.

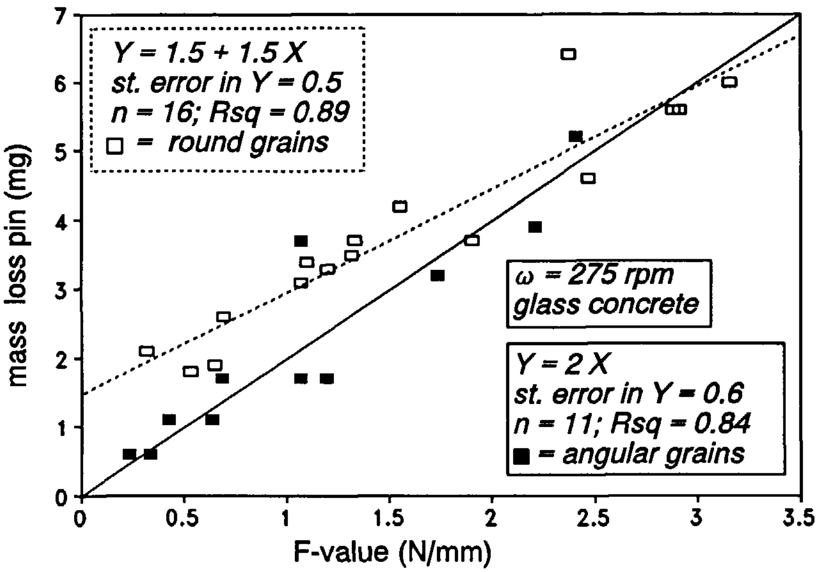


Figure 11.14 Results of pin-on-disc tests on glass concrete at a rotation speed of 275 rpm (Verhoef et al. 1990).

the previous tests, the velocity under the pin would vary considerably.[65] In the new test arrangement, when the pin moves inward to the centre of the rock specimen during a test, the angular velocity of the disc of rock increases, keeping the tangential velocity under the pin constant. The normal pressure under the pin was kept constant, by using a constant force (dead weight) on the pin. First a two-level full factorial analysis was carried out to find out which variables were of influence on the test result. The following factors were considered:

- 1. The effect of the steel type and hardness of the stylus
- 2. The diameter of the wear flat of the pin
- 3. The feed or radial velocity of the pin
- 4. The normal pressure of the pin on the sample
- 5. The tangential velocity of the sample under the pin
- 6. The effect of polishing
- 7. The effect of sample preparation

The effects of these parameters were determined by a series of tests on one rock material, artificial lime-sandstone (calcareous sandstone) (Type H, Table 11.3). A summary of the results of the factorial analysis is given:

- The normal pressure of the pin on the rock disc had the greatest effect on the amount of wear measured, Figure 11.15.

- Too high velocities of the pin relative to the rock disc caused such high temperatures that the pin hardness changed. It was found that the best type of steel to use was C45 steel, without any temperature treatment. It appeared that a Vickers Hardness of about 2000 MPa was needed and that hardening had no effect on the amount of wear. Steel C45, when hardened to 3000 MPa or more showed about the

[65] In the test described by Verhoef et al. 1990, at a rotation speed of 145 rpm the velocity under the pin varied from 0.137 - 0.273 m/s; at 275 rpm from 0.259 - 0.419 m/s.

Table 11.3 Description of the rock types used for testing.

Code	Rock description	Mineralogy
A	Artificial calcareous sandstone, consisting of relatively large quartz grains in a carbonate matrix with very fine grained quartz. Average grain size 0.57 mm.	Quartz 53 vol.% Carbonate-Quartz matrix 22% Calcite, Mica, Opaque, Glauconite Void space 30%
B	Idem, average grain size 0.20 mm	Quartz 46 vol.% Carbonate-Quartz matrix 16% Calcite, Mica, Opaque, Glauconite Void space 38%
H	Idem, average grain size 0.42 mm	Quartz 49 vol.% Feldspar 4% Carbonate-Quartz matrix 21% Calcite, Mica, Opaque, Glauconite Void space 26%
Euv	Calcarenite from Euville (Belgium). Bioclasts (fossil remains) in a matrix of recrystallized calcite and carbonate mud. Size grains varies from 0.3 - 5 mm.	Fossil grain fragments 57 vol.% Matrix 18% Void space 25%
Fel	Felser sandstone (Germany). Partly rounded quartz grains in a fine grained matrix of quartz, chlorite and mica. Average size quartz grains 0.4 mm.	Quartz 56 vol% Feldspars 9% Mica 3% Rock fragments 2% Opaque min. 4% Void space 26%
Leu	Calcilutite (St. Leu, Belgium). Fine grained mud and bioclasts, with some larger bioclasts. Minor amount of subangular quartz. Average grain size 0.11 mm.	Quartz 4 vol.% Fossil grain fragments 3% Matrix of carbonate 50% Void space 42%

same amount of wear, Figure 11.16. The reason was that at the wear flat temperatures in the order of 700 °C developed, which is the critical temperature of the C45 steel.

- Due to the fact that worn-off metal fragments polish the grooved surface, that pin must not slide over previously tested rock. The feed of the lathe must be minimally set at the pin diameter.

- The smaller the diameter of the wear flat, the lower the variance in test results. A longer wear path is possible at low diameter, when the pin always slides over fresh rock (no overlapping of sliding area). The limiting factor is the strength of the pin, which decreases with diameter (bending).

- Sample preparation influenced test results. It was not possible to test saturated samples; only dry samples can be tested with this method. The surface roughness of the sample is of influence, identical sample preparation is necessary.

- The best outcome of the test is when the pins have substantial loss of mass. This occurs at relative high contact pressures and when the pin is grooving into the rock.

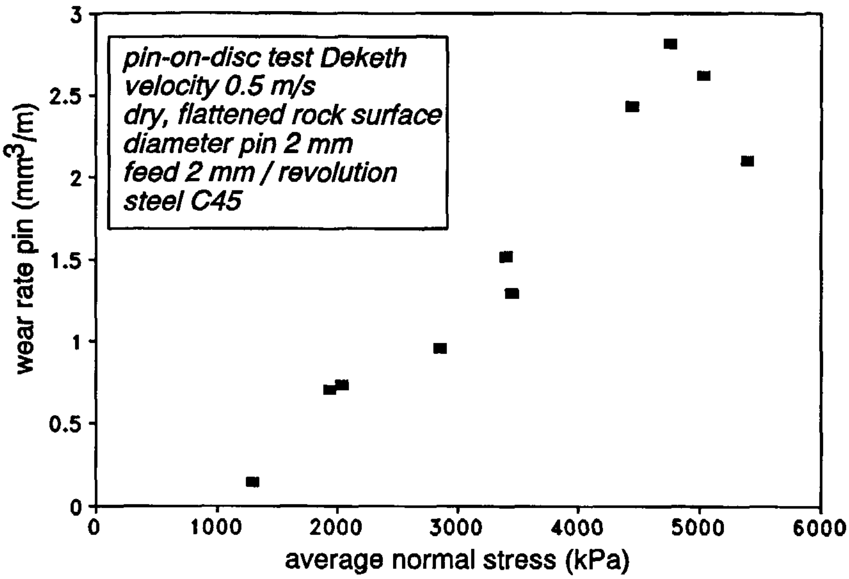


Figure 11.15 Typical example of the effect of normal pressure of the pin on the rock on the rate of wear during pin-on-disc testing (Deketh 1991).

Six rock types were used to investigate the pin-on-disc test. These rocks were also used as test material for the shaper tests that were carried out concurrently (Chapter 11.5). A description of these rocks is given in Table 11.3 and a summary of the engineering properties determined is given in Table 11.4. The settings of the

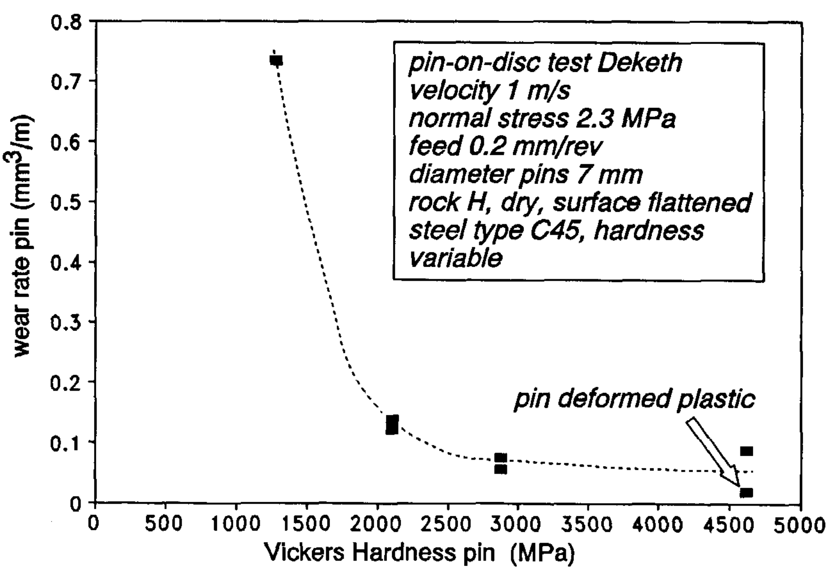


Figure 11.16 Rate of wear of the pins related to steel hardness (Deketh 1991).

Table 11.4 Properties of the rocks used by Bisschop 1991 and Deketh 1991 for testing (see also Table 11.5).

Rock type	UCS (MPa)	BTS (MPa)	E_{t50} (GPa)	ρ (Mg/m³)	n (vol.%)	grain size (mm)	Qtz eq (vol.%)	F (N/mm)
A dry	16.3	1.47	5.9	1.86	28	0.57	54	0.452
A sat	12.8	1.12	5.8	2.17	28	0.57	54	0.345
B dry	9.1	1.44	3.5	1.59	39	0.20	47	0.135
B sat	7.1	1.06	3.0	1.98	39	0.20	47	0.100
H dry	15.3	1.53	5.9	1.71	34	0.42	51	0.328
H sat	11.3	1.22	5.5	2.05	34	0.42	51	0.261
Euv dry	25.0	1.74	16.6	2.17	27	1.47	2	0.051
Euv sat	25.2	1.50	20.1	2.44	27	1.47	2	0.044
Fel dry	28.6	1.85	7.3	2.03	27	0.38	59	0.415
Fel sat	19.5	1.66	6.5	2.30	27	0.38	59	0.372
Leu dry	5.6	0.82	2.5	1.56	41	0.11	6	0.005
Leu sat	4.7	0.58	4.8	1.97	41	0.11	6	0.004

E_{t50} = tangential elasticity modulus at 50 % of UCS stress; ρ = density; n = porosity

Table 11.5 Results of pin-on-disc tests (Deketh 1991) and shaper abrasion tests on dry rock (Bisschop 1991).

settings pin-on-disc test: pin steel C45 (VH 1885 MPa); diameter pin 2 mm; velocity 0.5 m/s; normal stress 5 MPa; feed 2 mm/rev
settings shaper abrasion test: chisel steel C45 (VH 4610 MPa); chisel width 5 mm; cutting depth 0.4 mm (see Figure 11.18)

Rock type	pin-on-disc $W_{V,s}$ (mm³/m)	shaper $W_{V,s}$ (mm³/m)	BTS (MPa)	Qtz eq (vol.%)	Mean grain size (mm)	F (N/mm)
A	2.49	1.00	1.47	54	0.57	0.452
B	1.90	0.31	1.44	47	0.20	0.135
H	2.12	1.13	1.53	51	0.42	0.328
Fel	3.60	0.69	1.85	59	0.38	0.415
Euv	0	0	1.74	2	1.47	0.051
Leu	0	0	0.82	6	0.11	0.005

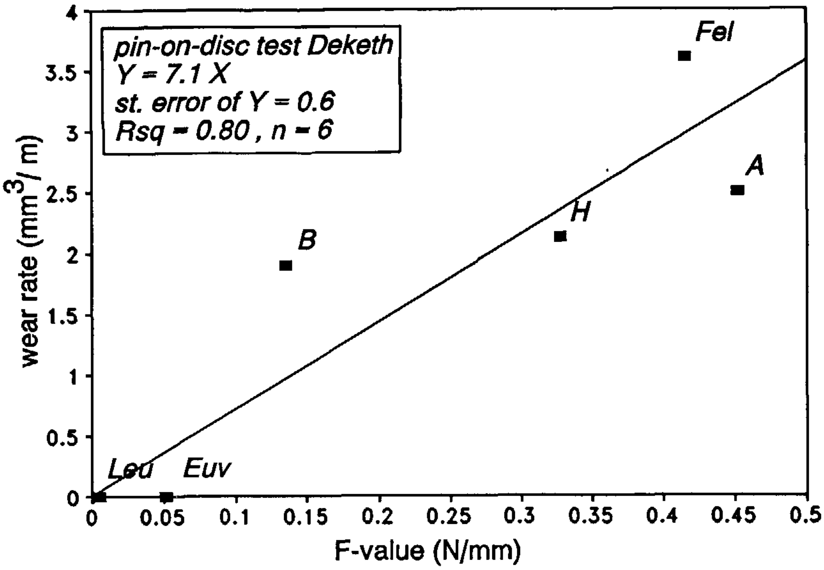


Figure 11.17 Rate of wear related to F-value (Deketh 1991).

pin-on-disc test that gave optimal results are given in Table 11.5, which also gives the test results.

Many wear tests have standard conditions to be able to compare test results. Schimazek's test, for example, has a set sliding distance of 16 m. Paschen (1980) used a sliding path length of 8 m, with a feed of 0.5 mm per revolution. In his test, Deketh chose to describe the amount of wear in volume loss per sliding distance (*rate of wear*), to be able to test without being bounded by a fixed length:

$$W_{V,s} = \frac{\textit{volume loss of the pin}}{\textit{length of sliding path}} \; (mm^3/m) \qquad (11.4)$$

It was not possible to determine the volume loss of the rock material, by either assembling the worn-off rock powder or by weighing the rock (due to the adhered metal particles), otherwise the *specific wear* (volume of metal worn divided by volume of rock cut) could have been determined. Results of the tests carried out by Deketh are shown in Table 11.5 and the relation of the *F*-value with the rate of wear is shown in Figure 11.17. Comparing this with Figure 11.9, we see a similar increase of wear rate with *F*-value as was obtained with the Schimazek test.

11.5 CUTTING TEST ON SHAPER

Cutting and abrasion tests using a shaper were performed on the same materials tested with the modified pin-on-disc test. The test method using the shaper was developed by Van der Sman (1988, 1990) and Davids and Adrichem (1990). The shaper test follows the concept of the core cutting test described in Chapter 9.7 (Figure 11.18). The cutting forces on the chisels could be measured using a three component dynamometer system, the cutting velocity (up to 1.4 m/s) could be

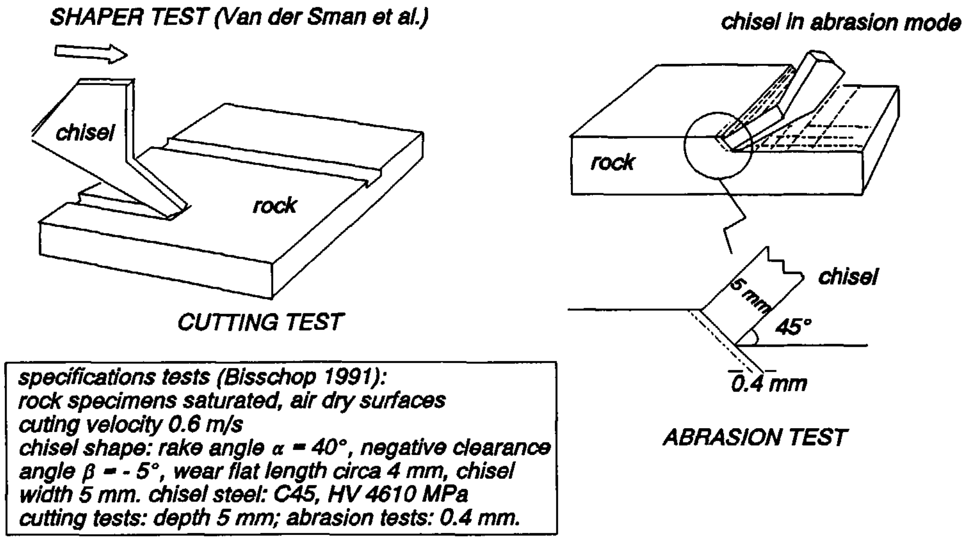


Figure 11.18 Characteristics of the shaper cutting and abrasion tests.

measured as well. Bisschop (1991) reported test results on the same six rock types as were tested by Deketh. Both cutting tests and abrasion tests were performed with specially designed chisels with wear flats (Figure 11.18). The cutting tests were carried out with a cutting depth of 5 mm. The abrasion tests were well controlled, by keeping the cutting depth constant at 0.4 mm.[66] The test results of the abrasion test, expressed as $W_{V,s}$ (Equation 11.4) are shown in Table 11.5.

For the purpose of comparison, the relation of F-value with the wear rate in this test is shown in Figure 11.19. Apart from abrasion tests on dry rock, that can be compared with Figure 11.17, Bisschop also tested the rocks saturated with water. Normally strength of wet rocks is lower, and the F-values of the wet rocks are therefore lower as well (Table 11.4), but the wear rate obtained was considerable higher for the saturated rocks. Another wear mechanism must be the cause of this. Bisschop (1991) assumed that crushed particles more readily break away when the rock is dry and thus less abrasive grains are attacking the tool. Also from practice is known that wet rocks, although weaker, can give more wear. Roxborough (1973) mentions that the development of a wet abrasive paste is the cause of much wear in weak sandstones: *"Because the forces involved in cutting weak rock are low, strain energy at failure is also low and the debris tends not to be propelled away from the cutting area in large fragments, but small sandy material clings together in the presence of water to form a highly abrasive paste near the cutting edge. Pick wear under these conditions will be excessive".*

[66] The core abrasion test (Chapter 11.1) is less accurate, because the test chisel is fed by hand.

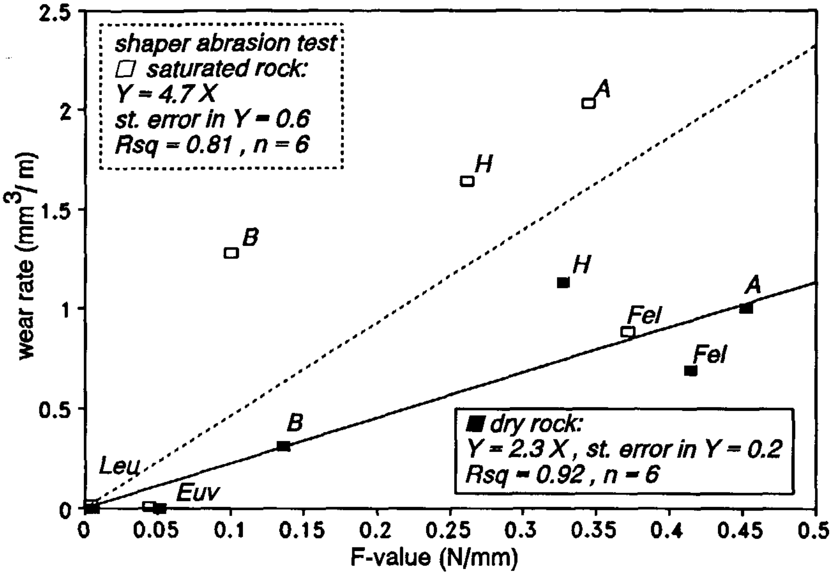


Figure 11.19 Results of the shaper abrasion tests on saturated and dry rocks (Bisschop 1991) related to the F-value.

11.6 COMPARISON OF CERCHAR, PIN-ON-DISC AND SHAPER ABRASION TESTS

The results of the test programmes carried out in Delft on the Cerchar test (Jager 1987), the pin-on-disc test (Van den Bold 1986, Vermeer 1989, Verhoef et al. 1990, Deketh 1991) and the shaper tests (Van der Sman 1988, 1990, Davids & Adrichem 1990, Bisschop 1991) were not easy to interpret. The main conclusion drawn was that further work was necessary to show that any of these tests can be used for routine testing to give a reliable indication of potential abrasiveness. This conclusion did not only hinge on the principle of wear processes being dependent on test conditions. The test results proved to be difficult to interpret. For example, optimal conditions for one rock type were different for an other type. Even when test conditions were slightly changed results could differ (compare Figure 11.13 & 11.14). Some test methods may work for weak sedimentary rocks, but not for strong granites.

Although in principle the tribological approach chosen for this study, which can be summarized as "wear is a process depending on the system in which it operates", is correct, it is not easy to apply this to determine the abrasive capacity of rocks. Commonly tribological tests are used to compare the performance of different tool materials against a rock or mineral abrasive. The results of such tests are used to select tool materials. When using such tests to assess rock properties, the usual high variability of rocks obscures the test results.

The *wear rates* derived from the Cerchar-, pin-on-disc- and shaper abrasion tests are plotted in Figure 11.20. The graph illustrates that the rock types are not ranked in the same order when the wear rates of the three tests are compared. Attempts to

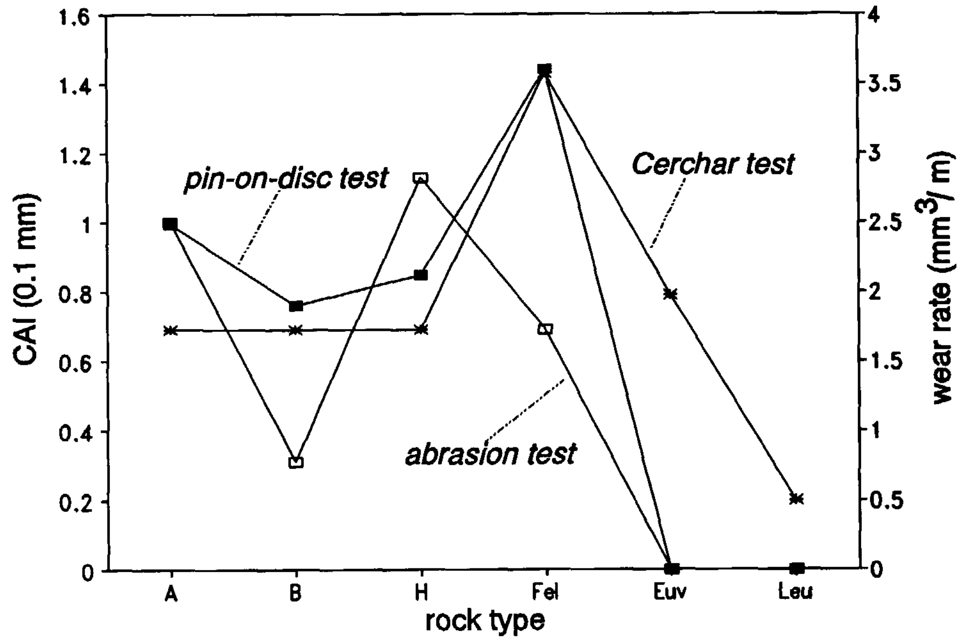


Figure 11.20 Comparison of the wear rates obtained by the Cerchar, pin-on-disc and shaper tests. Note that the Cerchar test was performed on one, different, calcareous sandstone sample.

derive wear prediction statements or formula from the test results have been done using statistical correlation analyses. These will be examined in Chapter 13.

It became clear that standard tests had important limitations, making them of limited use for purposes of prediction of wear of cutting chisels of rock excavation machines. They might be used as index test, giving an idea of the relative abrasiveness of a rock, but only under the conditions that apply to the test (see discussion in Chapter 4.4).

11.7 CONCLUSION

Although many more tests do exist, the results of the work done show that *tests with a fixed setting*, or so-called standard tests can only serve for reasons of comparison of one rock type with another. They may give erroneous expectations about abrasiveness during rock cutting with a machine. Each test has its own particularities and problems. The Cerchar test, for example, has been used to compare laboratory data with practical data. The treatment of the test in Chapter 11.2 shows that besides the abrasive minerals present in the rock, strength alone has a large influence on the outcome of the test. Strong limestones, without any abrasive minerals, may have relatively high Cerchar values, while on the other hand rocks like granite or gneiss, which are known to be abrasive, may have very low values, because the test stylus cannot penetrate the rock surface.

A comparison of the tests shows their limited value (Chapter 11.6). The attempts to improve the pin-on-disc test (Chapter 11.4) have shown that many factors influence test results. No real improvement compared to the Schimazek pin-on-disc test with fixed settings has been obtained. The test showed, however, that with increasing normal pressure on the pin, a change in wear mechanism occurred. This observation led to the development of the scraper test, which was specifically intended to study changes in wear mechanisms with increasing depth of cut into the rock. The theory developed from these tests is summarized in Chapter 12.

It became clear that the approach of using several laboratory tests to establish abrasiveness of rock would not be helpful, without a proper understanding of the processes operating during laboratory tests and during real scale rock cutting.

CHAPTER 12

Wear mode theory

12.1 DEKETH'S SCRAPING TEST

While the usage of laboratory tests for determining abrasiveness of rocks is of limited value, they might be helpful in understanding the wear mechanisms operating during rock cutting. One of the findings of the experiments with the pin-on-disc test was that with increasing normal pressure the rate of wear increases up to a maximum value, after which a decrease in wear rate is noted (Figure 11.15). At higher normal pressures crushing of rock occurs below the pin. Apparently when crushed rock material is present between pin and rock, less wear occurs. This led to the development of a hypothesis of different wear mechanisms, or *wear modes*, occurring. The decrease in wear rate at higher normal pressures could be explained by a transition from a two-body abrasion, or an adhesion wear type of mechanism, to a three-body abrasion mechanism.

Deketh (1995) developed the shaper test to study the transition of wear modes while testing rocks under different conditions. The principle of the test is shown in Figure 12.1. At shallow cutting depth only scraping of the rock will occur and the test resembles abrasion tests like the shaper abrasion test or the core abrasion test. With increasing *feed* (radial displacement of the support of the lathe per revolution of the turning head of the lathe) the transition to cutting can be studied.[67] The samples have a diameter of 140 mm and a thickness of about 80 mm. At both sides of the cutting path grooves are sawn into the rock discs before testing, to allow cracks to propagate to free surfaces. The grooves simulate the free surfaces that would be created in rock cutting practice by other chisels, travelling at either side of the chisel. The fundamental difference between a pin-on-disc type of experiment and a rock cutting type of experiment is that the former is force controlled (the force on the pin remains constant) while during a laboratory cutting experiment the

[67] The cutting depth is defined as the thickness of the rock cut per revolution of the turning head of the lathe. Thickness is determined by measuring the mass loss of the test rock disc and calculating volume loss. The cutting depth can be compared with the feed. The ratio of depth of cut to the feed (*cutting mode ratio*) indicates whether the chisel cuts (value near to 1) or scrapes (value near to 0) the rock.

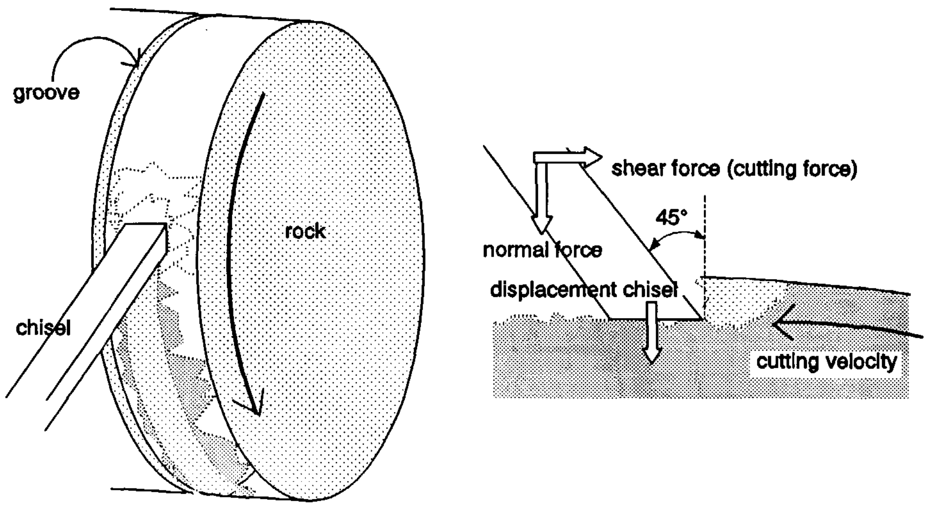


Figure 12.1 The principle of the scraping test experiments (Deketh 1995).

displacement of the chisel through the rock controls the process. The forces developing are a function of the rock properties and are measured during the experiments. On one rock type a series of experiments is performed, starting with tests at low feed (resulting in a shallow cutting depth) and increasing the feed values in subsequent experiments to conclude the testing on a particular rock at a large value of feed (high cutting depth). At low feed only scraping occurs, at high feed the chisel is cutting the rock.

Experiments were carried out with the following objectives:

- to study the effect of increasing feed on the wear mechanisms operating.
- to develop and verify a wear mode hypothesis.
- to investigate the influence of some rock properties on the different wear modes identified.
- to study the effects of cutting velocity on the rate and type of wear.
- to investigate the influence of some rock properties on the rate and type of wear at different cutting velocities.

The test procedure is described in Deketh (1995). Two types of steel were used for the experiments, steel Fe60K (HV 3000 MPa) and hardened steel machined from dredger teeth (SRO 57N, HV 6000 MPa). Testing was undertaken at a constant value of feed and when the chisel moved to the centre of the disc of rock, the angular velocity increased automatically to keep the cutting velocity constant. Most experiments were executed at a cutting velocity of 0.4 m/s. About a thousand tests were carried out on natural and artificial rocks. Artificial rock (mortars) allowed for controllable variation of one rock property at a time. Rock strength, grain size, volume percentage of abrasive minerals and shape of abrasive minerals were varied. Natural sandstones and limestones were tested, to find out to which natural rock types the test results could be applied. Apart from the rock properties the machine settings feed and cutting velocity were also varied (Table 12.1). The measured test parameters were described by the rate of wear, $W_{M,s}$ (mass loss of chisel per m of

Table 12.1 Variables in the scraper tests carried out by Deketh (1995).

Rock parameters		Machine parameters	
Artificial rock (mortar: cement + sand)	Range of values tested	Variable	Range of settings
vol.% abrasive minerals	0 - 60 %	cutting velocity	0.3 - 3.3 m/s
grain size abrasive minerals	0.125 - 2 mm	feed	0.05 - 2.5 mm/rev
grain shape of abrasive minerals	rounded - angular	tool material: wedge steel dredge steel	 HV = 2950 MPa HV = 6000 MPa
strength (UCS) of rock	9 - 74 MPa		
abrasive mineral	quartz, feldspar, glass		
natural rocks			
sandstones	5 types		
limestones	2 types		

cutting distance), and by the *specific wear, SPW* (mass loss per unit volume of rock cut). *SPW* shows the efficiency of the cutting process with respect to wear.

The results of these tests showed that indeed different wear mechanisms occur at different cutting depths. The magnitude of the *rate of wear* or the *specific wear* varies and depends on cutting depth. The scraping tests illustrate why tests with standard settings, such as described up to now, never can be used to compare abrasive capacity of different kinds of rock.

12.2 WEAR MODES

The displacement-controlled scraping test was developed to study the transition of wear modes while increasing the feed from zero to higher values. The machine settings given in Table 12.1 show that the experiments were confined to relatively shallow cutting depths. Hence, the experiments concentrate on the initial or penetration phase of cutting with tools mounted on rotary cutterheads, which have to penetrate the rock face at each revolution of the cutterhead. Rocks which are relatively strong with respect to the cutting machine will allow only shallow penetration (i.e. the operating parameters (haulage velocity and cutting velocity) and the cutterhead design are not tuned to the rock). For such conditions (penetration of fractions of millimetres to a few millimetres in the rock) the scraping test is directly relevant. Relatively weak rocks may allow penetrations that exceed the range of values of feed examined with the scraping test. However, during the initial penetration of a chisel into rock, the chisel will pass the zone examined by the test.

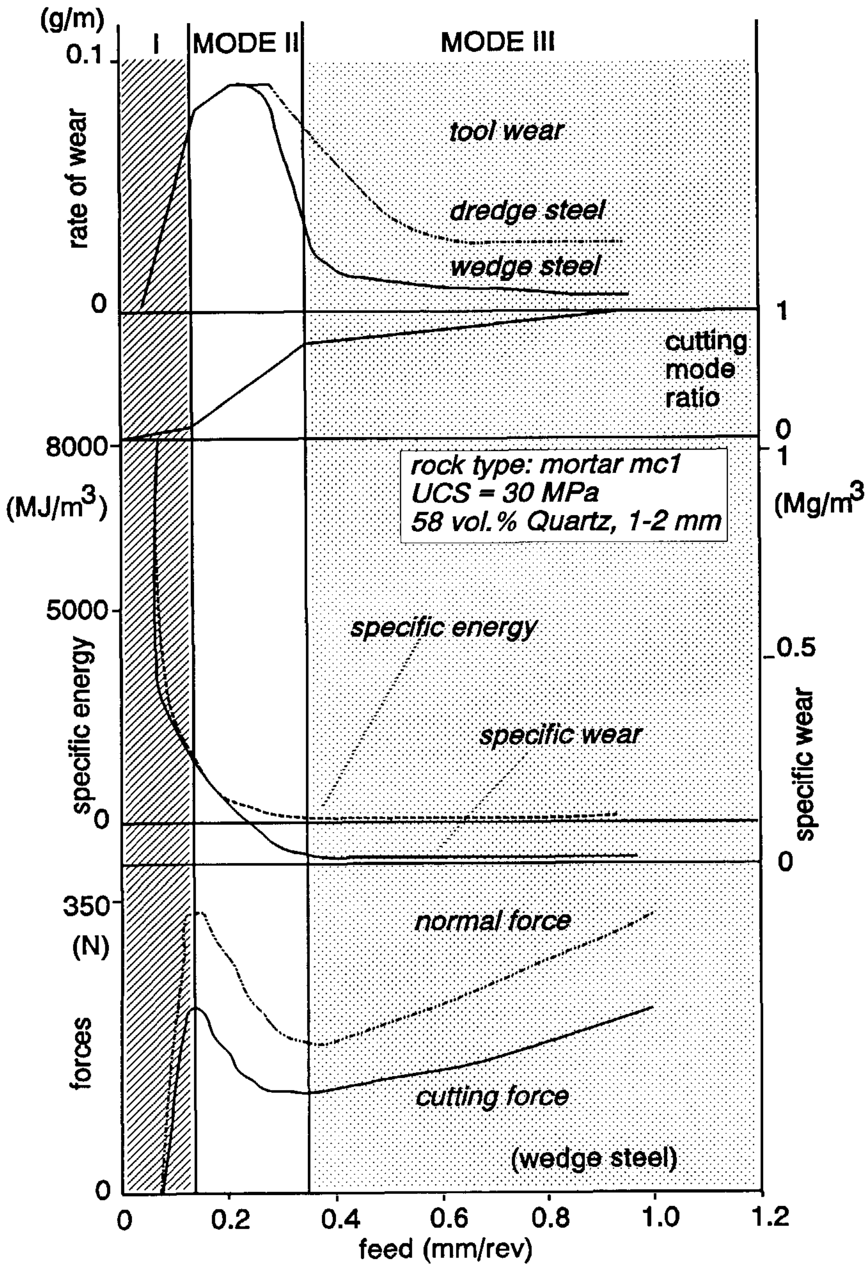


Figure 12.2 Typical result of scraping tests on mortar rock. See text for explanation. After Deketh (1995).

The interesting outcome of the tests on sand-cement mixtures (mortar), was that during the initial penetration of the chisel, at low values of feed, very high wear rates occur. When the chisel is cutting at higher depth, rates of wear are lower. The experiments confirmed the idea that at shallow depth two-body abrasion and adhesive wear processes are dominant, while at greater depth three-body wear is the major process. The artificial mortar rock models *sandstones*, i.e. rocks with abrasive grains embedded in a finer grained matrix, or in grain-to-grain contact with each other.

In this chapter the results of the experiments will be examined. The implications for the practice of rock cutting dredging are discussed.

12.2.1 *Wear processes at shallow cutting depth*

To examine the events that occur when a chisel enters the mortar rock, a simplified set of graphs is prepared. These are shown in Figure 12.2 & 12.3. It should be clear that the graphs show lines drawn along trends indicated by point data representing the results of many experiments, each carried out at one particular value of feed.

Figure 12.2 is a compilation of the experiments done on a mortar rock, consisting of 58 vol.% of 1-2 mm sized quartz grains embedded in cement, that had a UCS of 30 MPa. The top graph shows the rate of wear (loss of mass per unit of sliding distance), which is negligible when the chisel barely touches the rock, increases to a maximum and then drops to a level that remains constant in the test range of feed. Interestingly, the dredge steel, with higher initial hardness, has higher wear rates than the wedge steel.

The graph below gives the *cutting mode ratio*, which is the ratio of the depth of cut and the feed. A value of 1 indicates cutting, a value of 0 indicates that scraping, without formation of rock fragments, is taken place. A cutting mode ratio of 0 means that the chisel wears at the same rate as it is displaced into the rock, resulting in a net cutting depth of 0 mm/rev. A cutting mode ratio of 1 indicates that no wear is taking place, the feed equals the cutting depth. In this case at a feed of 1 mm full rock cutting starts.

The graphs of specific energy and specific wear versus feed are of particular importance, because they indicate the economy of the cutting process. Both values significantly drop when the chisel starts to produce rock fragments and cuttings.

The lowest graph displays the measured averaged normal and cutting forces for the wedge steel chisels. The forces increase to a high value at shallow depth, decrease and then increase again with cutting depth.

Deketh (1995) describes that the chisel temperatures in the initial shallow cutting stages are very high and rise above 600 °C in a narrow, 300 μm wide, zone at the wear flat of the chisels. The wear flat surfaces of the chisels showed parallel grooves (indicating microcutting) and sometimes a 2 μm thin band of steel has plastically flown into the direction of movement of the chisel. (Deketh, 1995, Figures 36 & 43). These observations indicate that the wear mechanisms involved were two-body abrasion and adhesion.

Temperatures near the wear flat of chisels cutting at higher feeds reach a maximum of 400 °C. The wear flats of these chisels have irregular, abruptly ending, grooves and have much rock gouge embedded in the steel of the wear flat.

These observations led to the model of three modes of wear taking place during the penetration of a chisel into this type of rock. These three wear modes relate to three cutting mechanisms:

- *Wear mode I*: The scraping process occurring at low feed is associated with two-body grooving abrasion. Clean, parallel continuous grooves on the wear flats of

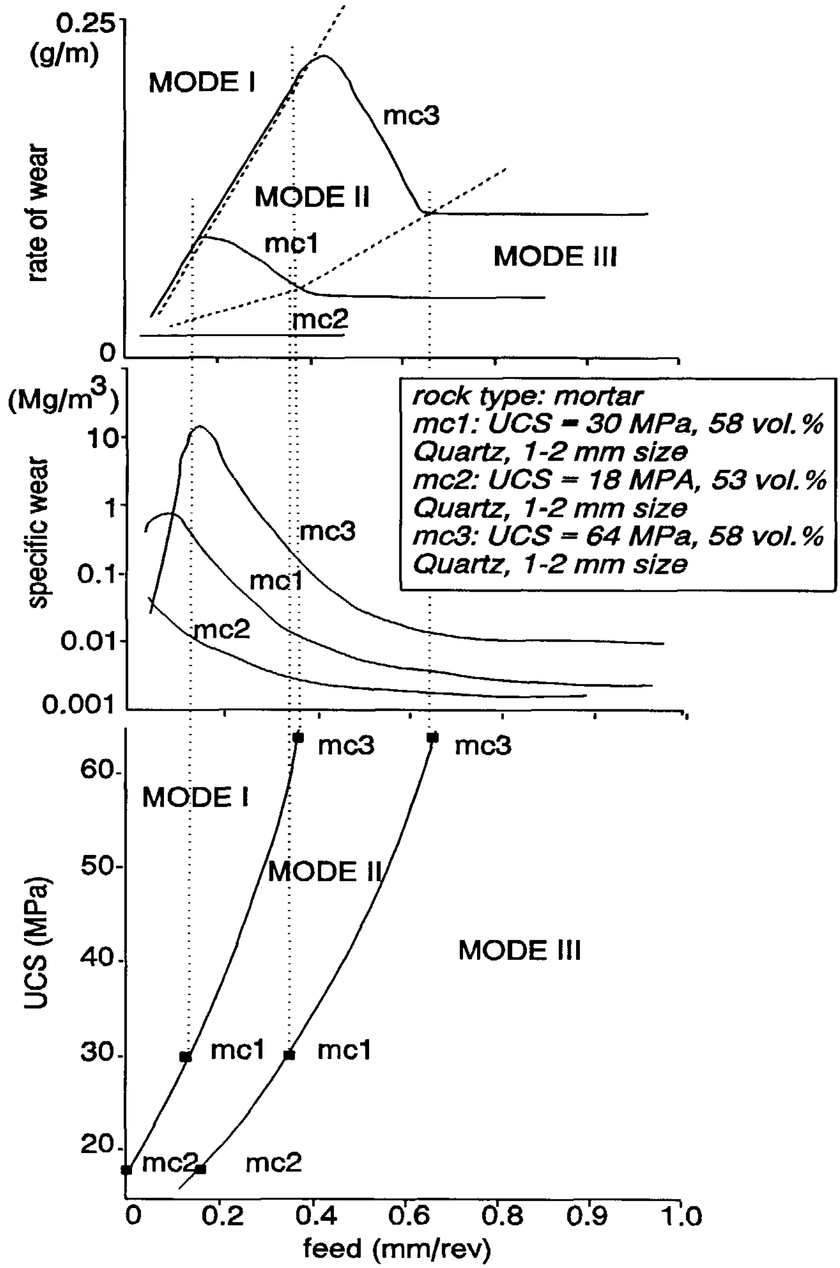


Figure 12.3 Effect of unconfined compressive strength on the transition of wear modes at increased feed during scraping tests. Summary graphs. After Deketh (1995).

the chisels affirm this. High temperatures may develop leading to plastic deformation of a thin zone near the wear flat, that significantly contributes to wear. The latter process is termed adhesion in this study.

- *Wear mode II*: The transition from scraping to cutting is associated with the same wear mechanisms taking place while scraping. With increasing cutting depth, a change towards mode III takes place, associated with a fall of the wear-flat temperature and the rate of wear.

- *Wear mode III*: The chisels are cutting into and crushing the mortar rock. Observations of the chisels indicate that three-body abrasive wear is dominant, crushed rock gouge is present between wear flat and intact rock. Temperatures of the wear flat are relatively low.

Referring to Figure 10.2, some additional remarks can be made:

- In these experiments, the wear rate of the dredge steel is about 75% higher than that of the softer wedge steel in wear mode III. Deketh (1995) notes that this difference also holds for tests carried out on a stronger mortar. The explanation must be found in a different reaction of the two steel types on the wear process imposed.
- The rise in cutting forces with increasing feed, or cutting depth, will probably lead to a rise in the specific energy and specific wear. In the shallow depths investigated, however, this is not occurring as yet.

12.3 ROCK PROPERTIES INFLUENCING WEAR

Figure 12.3 is a compilation of the test results of mortars which have an almost identical mineralogical composition and differ only in strength. The top graph shows that for the strongest mortar rock the feed range at which the unfavourable wear mode I is occurring is larger. Correspondingly the depth of cut at which relatively high wear rates are present increases as well. The transition to wear mode III also occurs over a wider feed range. Contrasting with this is the behaviour of the weakest rock. No transitions in wear rate are indicated, but observations of the worn chisel surfaces indicated that at a feed of 0.18 mm/rev probably two-body grooving abrasion was still occurring, while at higher feed mode III prevailed. The lowest graph in the figure indicates the trend of shifting transition zones with rock strength.

The graph in the middle shows the data on the specific wear (note the logarithmic scale; specific energy follows a similar trend).

The rock parameters *average grain size* and *vol.% Quartz* had an important influence on the rate of wear. The transition zone (mode II) that separates the fields where mode I and mode III wear occurs is given in Figure 12.4. These graphs can be used to guess the likely wear mode that would operate at a certain feed in a mortar of a composition different from that tested.

The rock parameters *strength, quartz content* and *grain size* influence, as shown by these tests, the rate of wear. Increase of the value of these parameters causes an increase of the rate of wear and a shift of the transition towards the more favourable wear mode III to a larger cutting depth.

Deketh also investigated the effect of *grain shape*. Only at very small feeds, while scraping, did sharper grains result in more wear. When the transition to cutting took place, in wear mode III, no difference in wear rate was found. The crushing process occurring in wear mode III explains why initial shape of grains has no influence on the wear of the chisels at greater cutting depth.

Finally, samples of mortar rock were prepared with *Feldspar* and *fused silica glass*. Silica glass has a hardness lower or equal to that of hard steel (HV 7500 MPa), Feldspar has a hardness of about HV 8000 MPa. It was expected that the

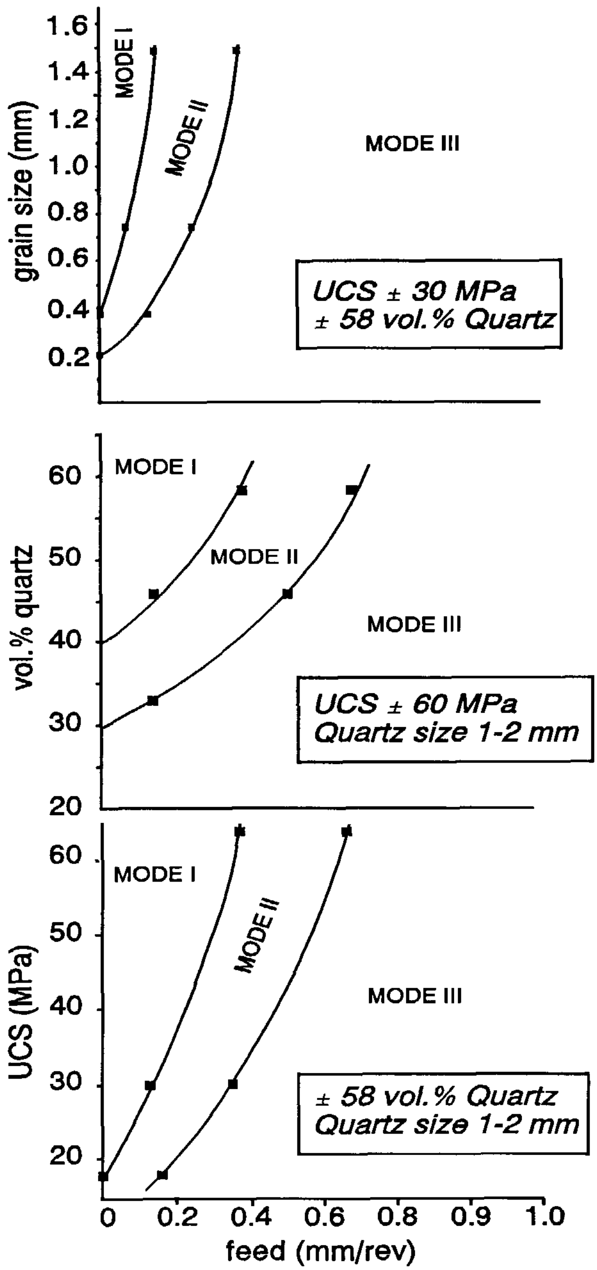


Figure 12.4 Influence of grain size, quartz content and strength on the transition of wear modes at increasing feed for the experiments on artificial rock (mortar) of Deketh (1995).

cleavage present in Feldspar grains might have influence on its hardness during cutting (as found by the grinding experiments of Rosiwal, resulting in a low Rosiwal hardness, see Table 10.4, p123). But no significant difference in behaviour of these abrasive minerals was found, compared to the experiments with Quartz mortar. Note that the test chisels used had a lower hardness (3000 MPa and 6000 MPa) than the abrasives, so wear in all experiments was in the high wear mode (Figure 4.4). Apparently the high difference in hardness between Quartz and the other two abrasive minerals did not result in significantly different results in the scraping tests. This is an important result, because many wear prediction equations assume a linear relation of abrasive mineral hardness with amount of wear (see Chapter 13).

Natural rocks were tested as well. The *sandstones* behaved much in the same way as the artificial mortar rock. The *limestones*, however, behaved differently. The wear rate was much lower and a different type of wear process (sliding wear) operated. These limestones lacked abrasive minerals (i.e. during the scraping tests the calcite hardness remained lower than the steel hardness).

12.4 MACHINE PARAMETERS

When the *cutting velocity* is increased, the scraper tests show a sudden increase in wear rate above a critical velocity, similar to that of earlier experiments (see Chapter 9.4). When the experiments are repeated at different feeds, at increased depth of cut, the more favourable, mode III, wear conditions reduce the wear rate. The effect of velocity is to broaden the zone of initial penetration of the chisel where mode I conditions are present (Figure 12.5). The most important cause of the wear at high velocities is the high temperature that develops at the wear flat, causing steel softening.

The effect of *feed* has been central in the description of the test results. Feed is a machine parameter. The displacement per revolution is imposed on the rock by the machine. The actual cutting depth is a result of the breakout pattern occurring (depending on the rock type) and the retreat of the wear flat, which is a result of the combined response of the chisel and the rock to the wear process operating.

Tool material is a factor of importance. In Chapter 4 has been pointed out that most of the efforts done to improve wear resistance of cutting tools have concentrated on improving tool materials. Deketh tested wedge steel and a dredger tooth steel. In his experiments the wear rate of the dredger steel proved to be higher than that of the wedge steel. No direct conclusions can be drawn from this result, regarding practice. The scraper test is of test category VI-V (Figure 4.13). If tests were done at a larger scale, at dynamic impact conditions comparable to practice (category III-I), wedge steel chisels would probably readily deform under the high energy impact conditions, despite their abrasive wear resistance being perhaps better than that of the hardened dredge steel chisels.

12.5 SPECIFIC WEAR AND SPECIFIC ENERGY

Specific wear and specific energy follow the same trend from scraping to cutting. The process is very unfavourable at shallow depth and when cutting starts both parameters level off to a constant low value, much as was found by Roxborough in his cutting experiments (Figure 9.12). An equation that describes the *SPW* of the mortar rock and the sandstones may be derived from test data by multiple regression

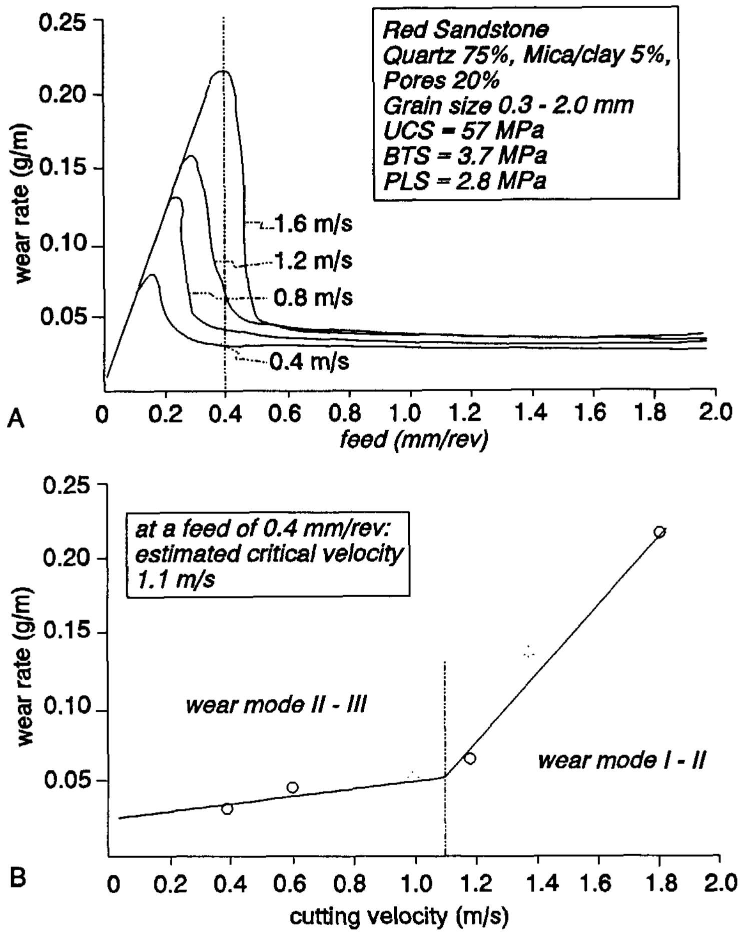


Figure 12.5 A. Scraper tests on sandstone at varying velocity, showing transition zone of wear mode I to III (Deketh 1995, p.123). B. Estimation of the critical velocity at a feed of 0.4 mm/rev.

analysis to give (Deketh 1995):

$$SPW = (A * 10^{-a * feed} + B) * 1000 \ (g/m^3) \tag{12.1}$$

$$A = 10^{-6.554 + 0.054\ UCS + 0.108\ vol.\%\ ab.\text{min.} + 1.489\sqrt{\phi}}$$

$$B = -0.026 BTS + 0.016 vol.\%\ abr.\text{min.} + (0.0078 * BTS * \phi * vol.\%\ abr.\text{min.})^2$$

$$a = 5.5 \textit{ for mortar artificial rock}$$

$$a = 7.5 \textit{ for sandstones; if vol.\% abr.min.} > 60\%, \textit{ use } 60\%$$

UCS and BTS are expressed in MPa, the grain size ∅ in mm. Vol.% abrasive minerals refers to all minerals harder than the tool material at testing conditions. The first half (A) of the equations refers to the specific wear of the mode I & II process (scraping). The second part (B) describes the specific wear of the mode III process (cutting). The shape of this equation is shown in Figure 12.6.

Roxborough (1987) explained that the specific energy of a cutting experiment is a most useful parameter of a laboratory experiment to relate to the specific energy of larger cutting machines (see Chapter 9.8). Since not many cutting experiments

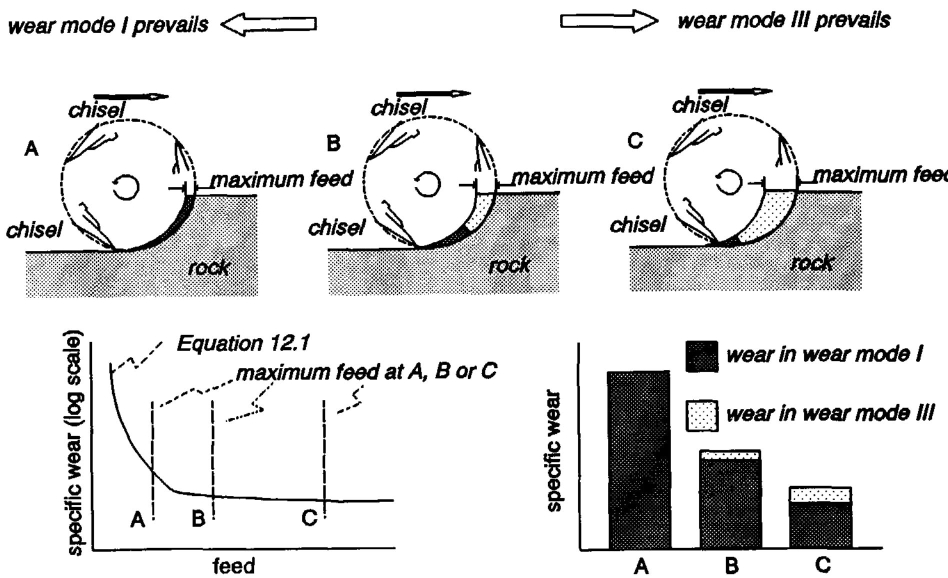


Figure 12.6 Wear mode model for rock cutting dredging, see text for explanation (Deketh 1995).

may be carried out, the relation of the laboratory specific energy with index tests are important. The core cutting test (cutting depth 5 mm, which will for many rocks be in mode III) appeared to have a linear relation with UCS (Chapter 9.8, Equation 9.11).

The scraping experiments show that specific wear relates to the rock parameters present in Equation 12.1.

12.6 APPLICATION OF WEAR MODE THEORY TO ROCK CUTTING DREDGING

Figure 12.6 gives the essential contribution of Deketh's wear mode theory to the understanding of chisel wear in machine rock cutting. If the feed, or cutting depth, remains relatively shallow the wear mode I (scraping) will occur during the full cutting part of the cycle. If the machine is stronger and can exert a greater thrust, a thicker slice is formed and part of the cut will be in mode III. The larger the penetration depth, the smaller will be the initial area where mode I and II wear processes occur. However, since the wear rate in mode I & II is exponentially higher, the contribution of the initial penetration part of a rock cut to the total wear is still very large, even in mode III (case C of Figure 12.6).

To reduce the amount of wear, it is favourable to cut at high feed (high cutting depths) and reduce the occurrence of wear mode I & II (i.e. cutting at shallow depths) as much as possible. For cutter suction dredgers this implies:

- Increasing the power of the cutterhead and winches of the dredger, resulting in higher penetration depths of the teeth.
- Increasing the ratio of haulage velocity over rotation velocity of the cutterhead.

This results in higher penetration depth of the teeth (Figure 8.4). A lower rotation velocity of the cutterhead also will keep the contact temperatures of the tool-rock contact lower.

- Adaption of cutterhead design to rock type, to ensure maximum feed for each tooth. A reduction of the number of pick points in an identical position (Equation 3.4) results in higher penetration depths of the individual pick points.

The above listed measures should be regarded with judgement. For example, Roxborough & Sen (1986) comment with respect to the deep cutting principle in mechanised rock cutting: *"Deeper cuts with more widely spaced picks mean that fewer picks are cutting at any one time and so the associated higher transitory cutting forces are being applied at fewer points in the array. In the extreme, this gives rise to cutterhead vibrations. There should always be enough picks in an array to enable the cutting sequence to be balanced sufficiently well to avoid wide fluctuations in the magnitude and location of the resultant force acting on the cutterhead."* Moreover, a reduction of the number of pick points is associated with increase of pick point spacing. The interaction of the pick points stops. This leads to an unfavourable cutting mode, accompanied by the use of more energy per unit of rock cut.

The above practical considerations have been taken into account by Deliac (1993), who has developed models that simulate the cutting action of cutterheads in rock.

CHAPTER 13

Assessment of the abrasiveness of rock

The development of the research into wear processes during rock cutting leads away from relying entirely on the results of laboratory abrasion tests on rock samples. Although many remain positive about using abrasion tests for the assessment of abrasiveness of rocks (Büchi et al. 1995), most would probably agree with Roxborough (1987) who states that correlation of laboratory test results with real scale machine cutting is fortuitous. An overall assessment involving rock mechanical and petrographic properties of the intact rock and considerations on the type of machine used, the expected depth of penetration and the type of cutting process, will play an important part in future discussions. Almost all the tests described in the preceding chapters have been conducted on *intact rock materials*; each rock cutting machine, or each cutter suction dredger, will have its own wear characteristics in a certain *rock mass*.

The conclusion drawn is that laboratory tests are helpful in assessing the abrasiveness of rock, but that the hope to be able to predict the amount of wear during rock cutting from tests is not justified by the results.

The Norwegian "Hard rock tunnel boring" programme that has been used as an example for this research, also found that the drillability indices DRI and BWI, which are based on laboratory tests (Figure 11.2), did not correlate better with the wear data (cutter disc life) than the Vickers Hardness number of the rock (VHNR) (Johannessen at al. 1990). The VHNR is determined by proportional summation of the Vickers Hardness of the constituent minerals (Table 10.4 & 5). To determine the VHNR only the petrography of the rock is needed. The Norwegian programme shows that in retrospect, simple petrographic indices may perform just as well, or even better, as laboratory tests.

However, the laboratory testing has helped to understand what rock parameters definitely relate to abrasiveness. These are:

- Strength
- Mineral content
- Grain size of (abrasive) minerals

To assemble the information needed, strength tests on rock samples can be carried out (tests: Unconfined Compressive Strength, Brazilian Tensile Strength, Point Load Strength). Grain size and mineral content can be established by microscopic examination. The hardness of the minerals is important in this respect. This is sometimes expressed in a total hardness of the rock, such as the Quartz equivalent volume percentage (proportional summation of Rosiwal Hardness), the VHNR just mentioned, or by the volume percentage of Abrasive Minerals (Minerals harder than steel = Mohs Hardness 5).

It is clear that the parameters listed above should be assembled during site investigation. The influence of the parameters on potential wear, however, depend on the circumstances. Therefore, for each project the data should be evaluated and trial tests should reveal the abrasiveness. In the remaining of this chapter common relationships found by correlation analysis of laboratory tests are examined more closely. It might be possible that these relationships can give further insight, at least with regard to the relative significance of the parameters involved. The influence of the parameters in full scale mechanical excavation of rocks is examined in Part C (Chapter 15-19).

13.1 WEAR PREDICTION EQUATIONS

Despite the fact that the laboratory tests examined in Chapter 11 are not considered generally applicable to assess abrasiveness of a rock, the wear factor F, derived by Schimazek & Knatz from pin-on-disc tests, has sometimes proved successful in practice (Chapter 11.3). The Equation 11.2 is reproduced here:

$$F = \frac{Qtz\ eq\ *\ \varnothing\ *\ BTS}{100}\ (N/mm) \qquad (13.1)$$

where $Qtz\ eq$ = mineral content expressed in hardness of Quartz using Rosiwal's hardness scale (in vol.%), $\varnothing$ = mean diameter of mineral grains (in mm) and BTS = Brazilian tensile strength (in MPa).

Paschen (1980) performed multilinear regression analysis on the data of Schimazek & Knatz (1970). The loss of mass of the pins was best described in terms of summations and products of mineral content, grain size, tensile and compressive strength. The data of Schimazek and Knatz (1970) would have an improved fit with the following estimate of mass loss (coefficient of discrimination r^2 of 0.96):

$$\Delta M = -21+493\varnothing_q-17938(Qtz\ eq)*\varnothing_q-2.6V_{ph}*BTS+0.02BTS*UCS\ (mg) \qquad (13.2)$$

where $\varnothing_q$ is grain size of Quartz (mm) and V_{ph} is volume percentage of phyllosilicate minerals (mica and clay). This result can be compared with Figure 11.9, where the linear regression line of the loss of mass of the pins with the F-value (Equation 13.1) is shown (coefficient of discrimination r^2 of 0.82).

Generally, relationships derived by multilinear regression are dependent on the data sets used and the experimental circumstances. This is shown by the work of Paschen. The regression equation he found for experiments performed using a modification of the pin-on-disc test on a lathe, but with the same rocks, is different.

The wear loss of pins or chisels in the pin-on-disc tests by Van den Bold (1986), Vermeer (1987), Bisschop (1991) and Deketh (1991) has been analyzed using regression techniques as well. The Schimazek F-value showed reasonable to good relationships with wear loss in all these tests. The refinements by multilinear regression (Bisschop 1991), however, were always limited to the one series of test data at hand and could not be transferred to other test results.

A modification of the Schimazek & Knatz equation, based on a number of mini-disc cutting tests, was proposed by Ewendt (1989):

$$F_{mod} = \frac{Qtz\ eq\ *\ \sqrt{\varnothing}\ *\ Is_{50}}{100}\ (N/mm^{1.5}) \tag{13.3}$$

where Is_{50} is the Point Load Strength (MPa). If the grain size is smaller than 1 mm, Ewendt found no influence in his tests and 1 mm is used. The Point Load Strength (MPa) is used as a measure of tensile strength instead of BTS (see Chapter 21). This modified wear factor can be considered as well in wear studies. It assumes that wear is proportional to the square root of grain size and that grain sizes smaller than 1 mm have no further effect.

Deketh's Equation 12.1 has been derived by applying statistical techniques on the data of the scraping test. It is repeated here for convenience:

$$SPW = (A*10^{-a*feed} + B)\ *\ 1000\ (g/m^3) \tag{13.4}$$

$$A = 10^{-6.554\ +\ 0.054\ UCS\ +\ 0.108\ vol.\%\ ab.\text{min.}\ +\ 1.489\sqrt{\phi}}$$
$$B = -0.026BTS + 0.016vol.\%\ abr.\text{min.} + (0.0078*BTS*\phi*vol.\%\ abr.\text{min.})^2$$
$$a = 5.5\ \textit{for mortar artificial rock}$$
$$a = 7.5\ \textit{for sandstones; if vol.\% abr.}\text{min.} > 60\%,\ \textit{use}\ 60\%$$

UCS and BTS are expressed in MPa, the grain size $\varnothing$ in mm. Vol.% abrasive minerals refers to all minerals harder than the tool material at testing conditions. The first half (A) of the equations refers to the specific wear of the mode I & II process (scraping). The second part (B) describes the specific wear of the mode III process (cutting). The shape of this equation is shown in Figure 12.6. The *SPW* equation 13.4 describes the wear rate per volume of rock produced with steel test chisels. The second part of the equation (B) is composed of the same parameters as are present in the *F* value of Schimazek.

13.1.1 *Sensitivity of prediction equations to variation of index parameters*

Even within a single rock block sample considerable variation in lithological properties is common. From a block sample of Hawkesbury sandstone (Lucas Heights, Sydney, Australia), on which cutting and wear tests have been performed (Chapter 16, Verhoef 1993), two parallel cores were taken. Each core was sawn into discs of about 2 cm thickness that were used for Brazilian tests (necessarily performed parallel to bedding) and the preparation of thin sections for petrographic study. Volume estimates of minerals were obtained by point counting (750 points per thin section). Equivalent quartz percentage (*Qtz eq*) was calculated assuming clay and mica to be the only other minerals besides quartz (in fact an average of about 2 vol.% ironhydroxides was present). The mean value for each parameter determined is given in Table 13.1, together with the standard deviation (σ) and coefficient of variation (COV = standard deviation/mean*100%). The variation of the values around the mean includes the variation due to lithological differences in the rock block of Hawkesbury sandstone.

Table 13.1 Rock parameters from samples of a Hawkesbury Sandstone block (Lucas Heights, Sydney) and calculated wear prediction factors.

Sample description	BTS MPa	Qtz vol. %	Qtz Eq. %	Grain size mm	F N/mm	F mod N/mm$^{1.5}$	SPW[1] 0.1mm g/m^3	SPW[1] 1mm g/m^3
1 white-grey sst with red band	0.75	68	69	0.27	0.14	0.35	3943	949
1	0.82	66	67	0.22	0.12	0.37	3764	946
2 red sst	1.53	75	76	0.35	0.41	0.78	14503	983
2	1.02	73	74	0.31	0.24	0.51	6217	955
3 light red sst	1.63	68	70	0.27	0.31	0.76	13376	960
3	1.21	68	69	0.32	0.27	0.56	8336	961
4 white-grey sst with red haze	0.89	63	65	0.30	0.17	0.38	5086	952
4	1.03	66	67	0.29	0.20	0.46	5975	953
5 white-grey sst	0.87	61	63	0.20	0.11	0.37	3780	944
5	0.82	64	65	0.20	0.11	0.36	3560	945
6 light stained white-grey sst	0.84	69	70	0.36	0.21	0.39	5519	958
7 white-grey sst with red band	0.68	64	65	0.22	0.10	0.30	3195	947
7	0.79	56	58	0.20	0.09	0.30	1802	880
Average	0.99	66	68	0.27	0.19	0.45	6081	949
Standard deviation	0.28	5	5	0.06	0.09	0.15	3708	22
Coeff. of variation (COV) (%)	29	7	7	20	48	34	61	2
COV calculated by Eq.13.5					37	29	37	0.9

[1] Assumptions: UCS = 13*BTS; PLS = BTS/1.5 (see Chapter 16.2), feed = 0.1 mm and 1 mm.

Using the data, Schimazek's wear value F (Equation 13.1), Ewendt's modification F_{mod} (Eq. 13.3) and Deketh's *SPW* (Eq. 13.4) were calculated for each sample and the mean value and standard variation for the results were calculated as well.

Using the mean value of the rock parameters and their standard deviations, it is possible to calculate the coefficient of variation of the wear prediction equations by Equation 13.5 (Ang & Tang 1984):

$$COV \approx \frac{\sqrt{\sum_{i=1}^{n}\left(\left(\frac{\delta g}{\delta x_i}\right)^2 * \sigma_i^2\right)}}{\mu} * 100\ (\%) \qquad (13.5)$$

where COV is the coefficient of variation of the predicted value, g is the prediction function, x_i are the rock parameters in the equation, μ is the mean prediction value, and σ_i are the standard deviations of rock parameters i. This equation may only be used when the parameters are independent, which assumed here. The calculated coefficient of variation for the wear prediction functions is also given in Table 13.1. The mean value of F, F_{mod} and *SPW* for the block sample has a higher coefficient of variation than calculated by Equation 13.5. This higher value is expected since it is based on n=14 data points whereas the analytical estimate is for n=∞.

Equations 13.3 and 13.4 both assume a constant value for one of the parameters, when a threshold is passed. Equation 13.3 takes a value of 1 mm for all grain sizes below 1 mm. Equation 13.4 takes a volume percentage of abrasive minerals of 60 for all values above 60%. Both rules apply in the case of the Hawkesbury sandstone block (Table 13.1).

The sensitivity of the Equations 13.1, 13.3 & 13.4 to variation of rock properties was determined for the Lucas Heights Hawkesbury sandstone block (Table 13.1) and for a hypothetical medium strong sandstone. The hypothetical rock type has been used, because its properties are in such a range that the exponential part of Equation 13.4 plays a significant role. Also grain size and volume percentage abrasive minerals have been chosen so that the threshold values of Equations 13.3 and 13.4 are not surpassed. The standard deviation of the rock properties is of the same order of magnitude as in case of the Hawkesbury sandstone. The rock properties of the hypothetical sandstone are:

Unconfined Compressive Strength	*50 (σ=10) MPa*
Brazilian Tensile Strength	*4 (σ=1) MPa*
Point Load Strength index	*4 (σ=1) MPa*
Equivalent quartz volume percentage or volume percentage abrasive minerals	*50 (σ=5) %*
Grain size abrasive minerals	*1.5 (σ=0.3) mm*

The coefficient of variation COV has been used to compare the relative sensitivity of the three wear prediction equations to a ten percent variation of the standard deviation and of the mean value of the rock parameters. Spreadsheet calculations of COV using Equation 13.5 were done. The prediction value of Equations 13.1 & 13.3, which are simple products, varies proportionally with changes of the values of the parameters. The COV of the exponential function 13.4, however, increases rapidly at smaller values of feed. The exponential function with the rock property parameters implemented in the exponent has the highest coefficient of variation at low values of feed. Changes in Quartz content, UCS and grain size have a large influence on the numerical value of the *SPW* in the exponential part. The wear prediction is therefore less reliable at small penetration (advance) rates of the cutting machine. At higher feed, the second part of the Equation 13.4, shows a behaviour which resembles the product Equations 13.1 & 13.3. This implies that at higher feeds it would make no difference which equation is used to make a prediction.

13.1.2 *Discussion*

Various authors showed that wear of rock cutting tools in laboratory experiments or in field observations is an exponential or power function of some rock properties (McFeat-Smith & Fowell 1977, Johannessen 1988). The *SPW* equation that describes the wear rate of the scraper tests is exponential. The exponential part of the wear rate equation is related to the penetration phase of the cutting tool into the rock, once the cutting process has commenced, mode III wear occurs and a linear function may be used to describe the wear rate (part B of the *SPW* equation).

Referring to the outline of wear theory presented in Chapter 4, the exponential nature of the wear rate during rock cutting can also be understood in the light of *wear mechanisms*. During rock cutting, while penetrating the rock wear mechanisms are changing continuously from mode I towards mode III. With each change of mechanism, the rate of wear would differ. If a steady state position would be reached, then the wear rate possibly could be described by a linear, Archard-type equation:

$$V_w = K.\frac{F_N.s}{H}\ (m^3) \tag{13.6}$$

where V_w is the volume loss of the tool material along the contact surface, s is the sliding distance in m, F_N is the normal force on the surface (N), H is the hardness of the tool material (N/m^2). The constant K is the wear coefficient, which depends on the actual wear process operating. The equation states that a linear relation exists between volume loss and sliding distance, which depends on the normal force on the contact surface and the hardness of the tool material. The constant K depends on the relative contribution of two-body grooving abrasion, adhesion and three-body abrasion at steady state. When test conditions are changed, it is known that K is often related to V_w in a non-linear fashion (Landheer 1983). This shows that it is necessary to perform many wear tests, carried out under different physical conditions, to map out the variation in rate of wear.

13.2 CONCLUSION

The examination of the laboratory test methods in Chapter 11 has shown that each test has its own complex relation of rock properties with test results. The wear factors derived from these tests indicate that a combination of rock strength, rock hardness and mineral grain size describes the wear rate reasonably well. Wear rate is a non-linear process, as illustrated by the *SPW* equation of Deketh, developed for the scraper test. The non-linear part is related to the penetration phase of the tool, where changes in modes of wear processes occur. When cutting in the wear mode III, the wear rate may become constant. In that case the simpler *F-value* may correlate just as well with wear rate.

The assessment of abrasiveness of rock can be based on a combination of rock strength testing and petrographic examination. Rock hardness is expressed either as a proportional volumetric summation of the hardness of the constituent minerals or by the volume percentage of abrasive minerals (i.e. the minerals harder than cutting tool steel at the testing conditions). In each project, the rock parameters, or the wear prediction equations, must be related to the actual wear occurring.

CHAPTER 14

Conclusions of Part B: Rock properties influencing cutting and wear

In Part B the influence of rock properties on cutting and wear of tools is examined. It has been pointed out that the nature of the rock mass influences excavation. The frequency and geometry of fractures (discontinuities) in the rock mass determines whether the cutterhead is able to act as a ripper, by loosening the rock blocks, or will have to partially or completely cut into the rock material. In order to predict whether ripping or cutting will occur, the size and geometry of the rock blocks building the rock mass should be compared with the size and design of the cutterhead of the cutter suction dredger.

The rock material properties relevant during cutting have been established mainly by tests carried out in the laboratory. Simple rock cutting theories use only the unconfined tensile or compressive strength, or a combination of these two, to predict cutting forces.

Work by the Delft Hydraulics Laboratory has shown that the complete failure envelope of the rock is addressed. Near the cutting tip of the tool, extremely high confining pressures exist, causing compressive crushing of the rock. In brittle rock, away from the crushing zone, shear failure and tensile failure occur during chip formation. This implies that, to properly describe rock failure during cutting, attempts should be made to predict the complete rock failure envelope. One possible way to do this, is to follow the approach taken by Hoek and Brown (1980) and try to predict the failure envelope from values of the UCS and BTS test. The transition stress from brittle to ductile failure is an important parameter in this respect. From mechanical excavation practice there are indications that rocks, which are ductile during cutting (large crushed zone; no chip formation), may cause extreme heating of the cutting tool resulting in adhesive wear of the tool surface. Whether this type of wear will occur depends on the strength of the rock with respect to the power and design of the cutting tool.

The scraper test of Deketh has shown that during the penetration of the cutting tool into the rock, the tool at first scrapes over the rock surface and experiences a transition towards real cutting. In the initial stage, very high wear rates occur and chisels are heated to such a degree that apart from abrasive wear, a high amount of the total wear can be attributed to high temperature adhesive wear (wear mode I).

After a threshold cutting depth chip formation occurs and the wear rate is lower and is dominated by abrasive wear (mode III).

If adhesive wear occurs, the tool surface is weakened due to temperature dependent phase transformations in the tool material. Also tungsten carbide undergoes weakening at elevated temperatures. The contrast of hardness between the tool material and the rock mineral grains may diminish, or even reverse, causing the high rates of wear.

High temperatures can also occur at increased cutting velocity. Many experiments show a distinct critical velocity, above which a sudden increase in wear rate occurs.

At temperatures below the tool material weakening threshold, wear will be mainly by abrasion. Abrasive wear is mainly due to the contrast in hardness between the mineral grains and the tool material.

At the start of this study, in 1989, it was thought that tool wear might be predicted by a combination of rock strength parameters and some suitable wear tests. These wear tests should address two-body and three-body abrasive wear. Based on our examination of pin-on-disc type tests (Cerchar, Schimazek, flat pins), however, it appeared that the wear rates obtained by these tests were dependent on a large number of factors, the most important being rock strength, hardness of the constituent minerals and grain size of the minerals of the rock. This dependency is such, that tests carried out at standard conditions may not induce identical wear mechanisms on different rock types. It is concluded that the tests themselves may not be so valuable in a site investigation for a rock dredging project. The main reason for this statement is that the tests should simulate a wear process identical as that which would occur in practice, with preferably real size tools. The rock parameters that relate to abrasive wear in the laboratory are important, however. It is suggested that these parameters should be evaluated against the performance of real cutting machines. The performance of the machines might be calibrated against these rock parameters.

The laboratory tests examined illustrate the importance of several rock properties, that can be determined by index tests. It is also clear that no complete picture of wear during rock cutting can be given. This research has been restricted to *abrasive wear processes* and most results apply to rocks that contain abrasive minerals set in a matrix of weaker material (the *sandstone* rock type). The parameters strength (measured by the index tests UCS, BTS and PLS), volume percentage abrasive minerals and grain size are important. Abrasive wear is sensitive to the following rock index parameters:

1. At shallow rock cutting (scraping), or during initial penetration of the cutting tools, the unconfined compressive strength (UCS), the volume percentage abrasive minerals and the grain size.

2. During rock cutting at higher cutting depths the Brazilian tensile strength (BTS), the volume percentage abrasive minerals and the grain size.

These parameters should be measured in sufficient quantity during a site investigation. Part C examines whether the approach of relating basic rock parameters to mechanical rock excavation projects is promising or not.

CHAPTER 15

Introduction to Part C: Application to practice

The question how to apply the knowledge gained during this study to practice was not easy to resolve. Rock dredging projects could not be monitored, which was one problem, but even if that would be possible, the assembling of data of the rock mass before, during and after dredging would be difficult. It was, however, possible to study the site investigation reports and the dredging logs of the Sydney Harbour project. This involved rock cutting in partially weathered Hawkesbury sandstone. In this project it could be assumed that the cutting was mainly in rock material, considering the massive nature of the rock. Near the Harbour side at the Opera House, markings in the rock, left by the cutter head of the dredger, confirm that real rock cutting took place. On these data it has been attempted to see whether the laboratory core cutting and abrasion tests or the rock wear factor F developed by Schimazek showed correlation with tool consumption (Chapter 16).

Within this project verification or confirmation of the ideas developed on the factors involved in tool wear and mechanical rock excavation was sought by studying the performance of rock cutting trenchers. The large T-850 trencher of Vermeer International can cut trenches of a few meters deep and a meter width. Such trenches are ideal for geological mapping and sampling purposes. The performance of the machine and the geology could thus be compared. The machine excavates by the cutting action of bits. When cutting in massive rock, cutting depths in the order of fractions of a mm to a few mm are common. This makes comparison with lab experiments possible. In Chapter 17 the result of the monitoring of 16 projects is discussed. Many of the outcomes of the research of Part B are confirmed by the observations of the trencher excavation. The rock mass structure is important. The frequency of discontinuities (fractures) in the rock influences the excavation rate. The block size resulting from the discontinuity pattern is crucial in this respect. If the dimensions of the trench and the block sizes are such that cutting of massive rock occurs, the same factors as involved in laboratory cutting, such as rock strength and petrographic characteristics, are important. It was also attempted to find out whether ductile rock cutting occurred and if this type of cutting would have influence on tool wear.

The next Chapter 18 is devoted to the methods of assessment of rock in order to predict excavation performance. The excavatability of a rock mass is mainly determined by the strength of the rock material and the density and orientation of

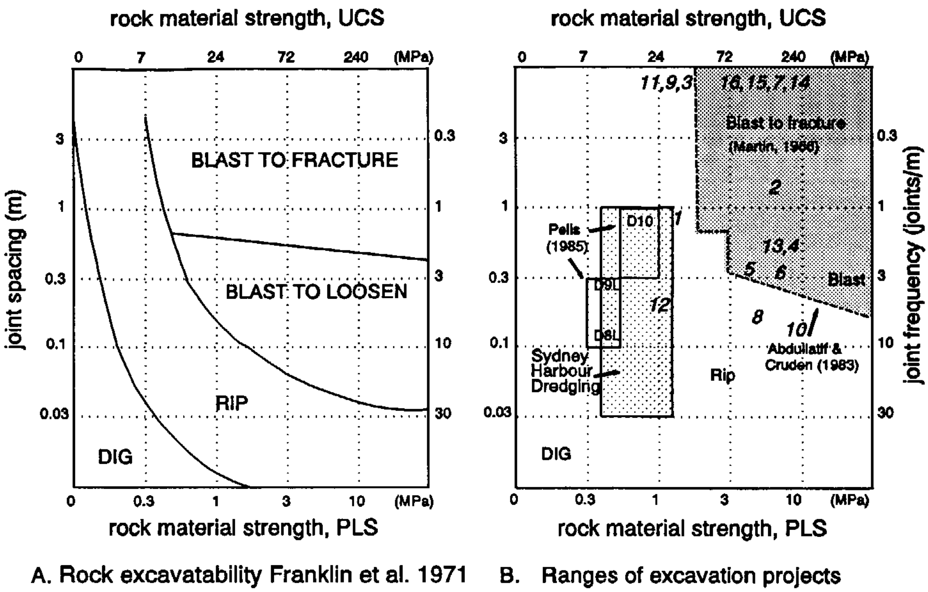


Figure 15.1 Rock excavatability graphs based on material strength and characteristic spacing of the discontinuities in the rock mass.

discontinuities in the rock mass. The performance of rock cutting machines is closely related to the mode of excavation. The trencher observations clearly show that two modes of excavation can be distinguished: cutting and ripping. Ripping occurs when the cutting tool (bit or pick point) is loosening the blocks of rock bounded by the discontinuities of the rock mass. When the spacing of the discontinuities or joints becomes wider, the bits start to cut into the rock material (Chapter 17). It is thought that such a transition from a cutting mode, where the bits are cutting into the rock material, and a ripping mode, where the bits mainly function as looseners of rock blocks, will exist for any rock cutting machine using the drag bit principle. Regarding rock properties, this statement emphasizes the importance of rock material properties when cutting occurs and rock discontinuities (fractures, joints etc.) when ripping occurs.

Franklin et al. (1971) used joint density and unconfined rock material strength to design an excavatability graph (Figure 15.1A). The boundaries on the graph are obviously machine dependent, as is illustrated by Figure 15.1B. In Figure 15.1B, data on ripping is plotted. Two boxes indicate ranges where Caterpillar D8, D9 and D10 have excavated Hawkesbury Sandstone in the Sydney region. Work by Martin (1986) and Abdullatif & Cruden (1983) gives an indication of the boundaries of rippable rock in the early 1980's (Braybrooke 1988). The blasting region is shaded. The suction cutter dredger Kunara (7438 HP) excavated Hawkesbury Sandstone for the Sydney Harbour Tunnel, with properties lying in the stippled box of Figure 5.2B. Present limits for underwater ripping and cutting dredging are difficult to delineate. A new graph of rippability assessment for bulldozers by this method is given in Chapter 18.2 (Figure 18.3). Of course it would be interesting to plot more data from dredging practice, but regrettably information is not available. The positions of the 16 trencher projects which are discussed in Chapter 17 are also plotted in Figure 15.1B.

The Franklin method and also the rock mass classification systems currently used in Engineering Geology site investigation are intended to give an indication of the way a rock can be excavated. It was examined whether these methods can be directly used to predict rippability (Chapter 18.2), or could be of use in dredging (Chapter 18.3). But it was found that the analysis of many data is necessary to come to some form of prediction. The example of the tunnel boring machine model of the Norwegian Technology Institute is discussed in Chapter 18.1, the example of ripping in Chapter 18.2. In Chapter 18.4 is outlined how we think the data of the trencher project can be used for prediction purposes by using fuzzy expert modelling.

CHAPTER 16

Abrasiveness of Hawkesbury Sandstone (Sydney Harbour tunnel dredging)[68]

In this chapter the tool consumption data of a rock cutting dredging operation are discussed. It concerns the excavation of the trench for the Sydney Harbour Tunnel, which was partially cut in rock (Hawkesbury Sandstone). The tool consumption data were compared with the wear factor *F* developed by Schimazek (Chapter 11.3), the *SPW* (specific wear rate) equation developed by Deketh (Chapter 12.5) and with the cutting- and abrasive wear rates as determined with the core cutting- (Chapter 9.7) and core abrasion test (Chapter 11.1). Also the data on the performance of roadheaders in the sandstones and siltstones of the Sydney Ocean Outfalls tunnelling operation were studied.

Both *F*-value and *SPW* are composed of easy to measure rock parameters (Brazilian tensile strength, mineralogical composition and grain size). The data needed can be obtained from a rock core. The Brazilian test is executed on a rock disc. The data on mineralogy and grain size can be obtained from examination by microscope of a thin section made from the same rock core. Microscopic examination is, up to now, not routinely done. If the *F*-value or *SPW* would give reliable information on wear, however, the data acquisition needed to determine these values on a large number of samples is probably easier and cheaper than performing laboratory cutting and abrasion experiments.

There are two main reasons why an abrasiveness number based on rock engineering index tests (like BTS) and petrography is preferred. Firstly the difficulty of performing sufficient cutting and abrasiveness tests to cover the variability inherent to a rock mass, and secondly the impossibility of performing laboratory tests that are truly representative of the actual cutting and abrasion mechanism operating.

In this chapter the data available on the excavation projects and on the rock properties are presented. The proposition of using rock strength index tests and petrography instead of, or in conjunction with, laboratory cutting and abrasion tests is examined.

[68] Parts of this chapter were published in the Quarterly Journal of Engineering Geology, London: 26, 5-17, 1993, and are reproduced with permission.

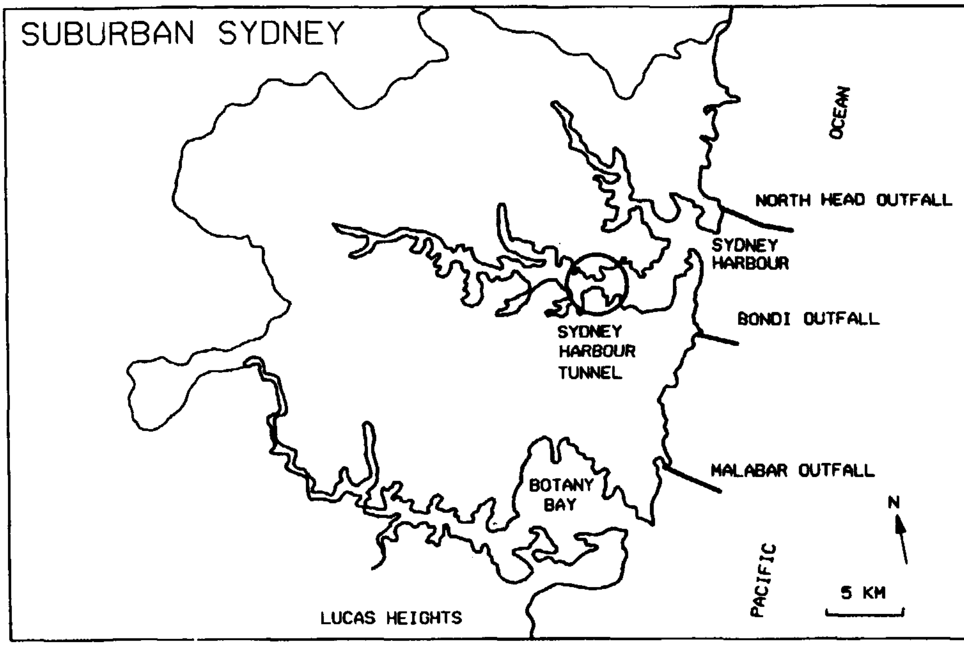


Figure 16.1 Location map Sydney area.

16.1 ROCK EXCAVATION PROJECTS IN THE SYDNEY AREA

Large parts of the Sydney area are underlain by the Hawkesbury Sandstone, a weak to moderately strong quartz-rich rock. The Hawkesbury Sandstone is of Triassic age and up to 290 m thick. Major tunnelling projects have been carried out in this rock and increasingly use has been made of mechanical excavation methods. For two recent projects carried out in the Sydney region, the Sydney Harbour Tunnel and the Sydney Ocean Outfall tunnels (Figure 16.1), considerable attention has been paid to the abrasiveness of the Hawkesbury Sandstone. On two of the Outfall tunnels reports have been published with data on the cuttability and abrasiveness of the rocks excavated (Lowe and McQueen 1988 and 1990). The Malabar Ocean Outfall Decline passes through Hawkesbury Sandstone and was partially excavated by mechanical means. Lowe and McQueen (1988) give data on rock properties and tool consumption rate and an analysis of roadheader performance.

For the Sydney Harbour Tunnel, consisting partly of land tunnels and partly of immersed reinforced concrete tube elements in the harbour, the trench dredged for the tunnel elements was partially excavated in rock, using a rock cutter suction dredger. Unusually the contractor, Westham Dredging Company Pty Ltd Sydney, made the excavation data available for analysis. In the following a comparison is made between the site investigation and laboratory data available and the excavation performance of the dredger.

For both projects results of the core cutting and abrasion test developed by Roxborough (1987) were available. Some additional tests have been performed. To examine whether wear factors derived from laboratory tests relate to tool

Table 16.1 Mineralogical composition of the Hawkesbury Sandstone (average of 42 samples from 16 locations in the Sydney Basin).

Mineral	Fractional (%)	Stand. dev. (%)
Quartz (sand size)	58.4	13.0
Others (rock fragments, feldspar and silt size particles	3.5	2.8
Matrix clay (kaolinite, illite, mixed layer clay)	24.2	7.1
Secondary silicates	8.4	4.4
Dry bulk density (Mg/m3)	2.37	0.13
Porosity (%)	16.1	3.5

Data from Robson, 1978; from Pells (1985)

consumption, the wear factor *F* of Schimazek and the *SPW* of Deketh have been chosen. For the determination of the wear factors, literature data and borehole records had to be studied by the author. As the wear factors were not used during the site investigation programme of both projects, the necessary parameters were not readily obtainable and had to be partially inferred. The rock abrasiveness data were compared with tool replacement data. The data of the Malabar project came from Lowe and McQueen (1988). The data of the dredging project were compiled by Watson (1990). Only tool replacement data were used that were considered representative of normal rock cutting dredging operation. Extremely weathered rock was discarded, as was also the head-on dredging performed at the harbour sides.

16.2 ENGINEERING PROPERTIES OF THE HAWKESBURY SANDSTONE

Pells (1985) gives a comprehensive review of the properties of the Hawkesbury Sandstone in the Sydney area. The rock mass consists dominantly of massive sandstone beds, typically 2-5 m thick, but locally up to 15 m. The mass structure is a mega cross-bedded structure, which has been described by Herbert (1976) to be the result of braided river deposition in a Triassic environment comparable to the present River Bramaputra in Bangladesh. Locally mudstone or shale is present in layers up to 2 m thick, usually interbedded with the sandstone. The mega cross-bedded structure results in a spatial variability of the rock material on outcrop scale. The sandstone is composed of subangular quartz grains in an argillaceous matrix, with some siderite. Table 16.1 gives data on the mineralogical composition assembled by Robson (1978) and cited with other data by Pells (1985). The variability of quartz content, expressed by the coefficient of variation COV, is somewhat more than 20% according to the data given in Pells' paper.

As part of the present study 27 samples coming from cores of 10 overwater boreholes for the Sydney Harbour project were examined in thin section. Two main groups of rock types were present: sandstones and siltstones-shales (laminites). The

Table 16.2 Geotechnical properties of the Hawkesbury Sandstone at the Sydney Harbour Tunnel site.

	Northern Tunnels		Southern Tunnels		Over-water boreholes	
Test	No. (n)	Mean (Range)	No. (n)	Mean (Range)	No. (n)	Mean (Range)
UCS dry (MPa)	70	42.9 (16.2-93)	18	42.6 (29-67)	-	
UCS sat (MPa)	55	22.8 (4-46)	14	21.0 (8-40)	26	22.6 (9.0-34.6)
BTS dry (MPa)	45	4.6 (1.5-9.8)	16	3.6 (1.4-6.0)	-	
BTS sat (MPa)	-		-		26	3.3 (1.4-5.4)
Ductility number UCS/BTS	-		-		6	9.2 (7.8-10.2)
Qtz %	10	70 (40-82)	10	76 (66-80)	13	60 (19-82)
Qtz eq %	-		-		13	62 (24-83)
SPE dry (MJ/m3)[1]	10	10.6 (8.3-13.9)	10	11.6 (10.4-13.9)	-	
SPE sat (MJ/m3)[1]	3	8.5 (6.8-10.7)	-		4	11.3 (10.0-12.3)
Cutting Wear dry (mg/m)[1]	4	1.70 (1.44-2.11)	-		-	
Cutting Wear sat (mg/m)[1]	-		-		4	4.25 (3.83-4.98)
Abr. Wear dry (mg/m)[2]	9	1.14 (0.57-1.80)	9	1.13 (1.0-1.2)	-	
Abr. Wear sat (mg/m)[2]	-		-		6	0.87 (0.11-1.15)
Cerchar Abr. dry (0.1 mm)	19	4.8 (2.8-8.7)	6	4.0 (2.0-5.7)	-	
Cerchar Abr. sat (0.1 mm)	-		6	4.0 (2.0-6.3)	-	
F-value sat (N/mm)	-		-		10	0.61 (0.03-1.07)

[1] Core cutting test, [2] Core abrasion test. Sources: Pells (1990), Coffey & Partners reports (1987). F-values calculated from data site investigation reports.

major minerals present were quartz and clay with subordinate iron hydroxides and carbonate (possibly siderite; in some samples dolomite crystals have been observed). White mica grains occur in minor amounts and occasionally microcline feldspar is found. Accessory minerals present are tourmaline and zircon. In certain zones, iron hydroxides stain the rock through thin layers or laminae and may act as cement. Disregarding the siltstone-shale laminites, the modal quartz content of the 19 sandstone samples estimated with the use of density diagrams was 74 vol.% (range 71-89%, COV 14%). The modal clay content was 13% by volume (range 5-20%, COV 34%). Mean grain size was 0.32 mm (range 0.25-0.46 mm, COV 22%).

The compilation of data on strength variation in the Hawkesbury Sandstone in the Sydney area given by Pells (1985) is relevant for the present study. Most unconfined compressive strength (UCS) values fall in the moderately strong group (12.5-50 MPa), with UCS_{dry} values also occurring in the strong (50-100 MPa) range. Moisture has a pronounced effect on strength; UCS_{sat} values being only 30-67% of the dry strength values measured. The variability of UCS values of the Hawkesbury Sandstone is about 35%, which is rather high for sandstones (a value of 20% is common; cf. Roxborough, 1987). Pells gives correlations between (diametral) point load strength (PLS; Is_{50}) and UCS and Brazilian tensile strength (BTS) and UCS: UCS=20*PLS; UCS=13*BTS (modal values; range: 12 to 15*BTS) (data from Ferry, 1983, cited in Pells, 1985). From these data the correlation between point load strength and Brazilian tensile strength (necessary for this study to estimate *F*-values from the borehole records) has been derived as: BTS=1.5*PLS (1.3 to 1.7*PLS).

Table 16.2 gives a summary of the data on engineering geological properties of the Hawkesbury Sandstone at the site of Sydney Harbour Tunnel. The data fit the range of properties as described by Pells (1985), although the ductility number (UCS/BTS) seems lower, being in the range 7-12. Unfortunately no correlations between PLS and UCS or BTS could be derived from the site investigation reports. Table 16.2 shows that in 10 instances *F*-values could be calculated for samples collected from the over-water boreholes and relevant for the rock dredging. In each case saturated BTS, mineralogy and grain size were determined on one sample and reported in the site investigation report.

For the present study additional tests on cores from the over-water boreholes have been performed. The data have not been included in Table 16.2, because the cores had suffered from ageing which would influence the test results. The results of these tests are discussed in the following section.

16.2.1 *Laboratory test results*

During the site investigations for the Sydney Harbour Tunnel and the Ocean Outfall tunnels, the core cutting and core abrasion tests developed at the University of Newcastle-upon-Tyne were performed (Chapter 9.7 & 11.1). The *F*-value of Schimazek and the *SPW* (Chapter 13) were calculated from the available petrographic and rock strength data. To be able to use and compare data, it is necessary that these are obtained in an identical standard way, using similar testing procedures. Schimazek used the following methods to obtain the necessary parameters for the *F*-value:

- Petrographic examination. The mineral contents were obtained from thin section study, using point counting techniques. Relative hardness of minerals with respect to quartz were determined using Rosiwal's hardness scale (Table 10.4 & 5). Grain size was determined using the line intercept method under the microscope. The average grain size was estimated by multiplying the average intercept length by 1.5 (Chapter 11.3).

- Tensile strength was determined using the Brazilian split test on dry specimens. The following adaption has been made with respect to the standard practise of determining the *F*-value. For dredging projects the use of dry tensile strengths was considered not relevant and from the beginning of the application of the *F*-value to rock dredging saturated Brazilian tensile strengths were used to determine the wear factor. This implies that the *F*-value data used for rock dredging may not directly be

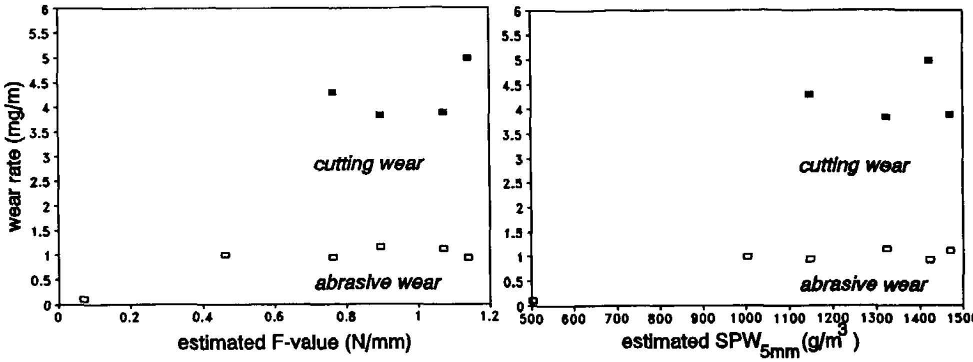


Figure 16.2 Results of the cutting and abrasion tests on samples of the overwater boreholes (Table 16.2) compared with estimates of the F-value and the SPW (feed 5 mm).

compared with *F*-values cited elsewhere; they should be corrected for type of strength testing (wet or dry).

Table 16.2 shows the mean results of the cutting and abrasion tests carried out on the over-water boreholes. For the rocks tested mineralogical and strength data were available, and *F*- and *SPW* values could be calculated. In Figure 16.2 the results are given. The figure shows no apparent relationship between either the *F*- or the *SPW* value and the cutting wear rates (wear of the chisels during the cutting test) and abrasive wear rates (wear of the test chisels due to the abrasion test).

Additional tests were carried out in October and November 1990 on old rock cores from the Sydney Harbour tunnel project by the author. The tests were performed in the Laboratory of the Department of Mining Engineering of the University of New South Wales, where Professor Roxborough has set up the apparatus for the cuttability assessment tests.

The samples were provided by the Geotechnical Consultant to the Sydney Harbour Tunnel project, Coffey Partners Pty Ltd. They were taken from the over-water boreholes in the Harbour, to assemble additional data relevant to the rock cutting dredging. It should be emphasized that these cores have been stored in metal boxes in the open air for more than two years. This must have had detrimental influence on their quality and weakened them.

On the rock cores both the cutting test and the abrasion test have been performed. Since the cuttability of the rock was studied in relation to the rock cutting dredging done for the Sydney Harbour tunnel, all cores have been tested in a water saturated condition. Water saturation was obtained by immersion in water for 24 hours. From each core tested, slices were taken to perform the Brazilian tensile strength test (saturated). From the slices tested thin sections were made for microscopic study, with the purpose of determining the wear factor *F* of Schimazek.

The results of the laboratory tests are given in Table 16.3 & 16.4 and Figure 16.3. The values of cutting forces, strength and cutting wear are generally lower than those measured during the site investigation on fresh cores (compare Figure 16.3 with Figure 16.2). This reflects the deterioration of the quality of the cores during storage. Again, the conclusion can be drawn that there is no clear relationship between *F*-value and laboratory wear rates as measured with the cutting and abrasion test. Plots of *SPW* against wear rate give a similar result.

Table 16.3 Data of overwater borehole samples tested. Wear factors calculated assuming PLS = 0.67*BTS, UCS = 9.2*BTS.

Description	No.	BTS_{sat} (MPa)	Qtz.eq (%)	Gr.size (mm)	Clay (%)	F (N/mm)	Fmod ($N/mm^{1.5}$)	SPW_{5mm} (g/m^3)
sl.w. sst	3001	0.4	86	0.39	10	0.12	0.21	955
finegr.sst	3002	2.7	39	0.09	25	0.09	0.70	560
laminite	3021	1.9	62	0.06	25	0.07	0.80	913
med. gr. sst	3201	2.3	82	0.35	5	0.67	1.28	1045
med. gr. sst	3211	0.6	85	0.27	10	0.14	0.35	950
	3211	0.3	85	0.31	10	0.09	0.19	954
sl.w. med. gr. sst	3212	0.4	78	0.30	20	0.08	0.18	953
med. gr. sst	3213	2.2	75	0.36	15	0.59	1.10	1039
	3213	1.7	80	0.46	10	0.63	0.92	1051
sl.w. sst	3231	0.8	81	0.28	10	0.17	0.41	950
med.-coarse gr sst	3232	1.2	86	0.26	10	0.26	0.67	950
med. gr. sst	3233	3.1	76	0.40	15	0.94	1.57	1214
	3233	3.3	80	0.48	15	1.26	1.76	1421
	3233	2.7	84	0.40	10	0.89	1.50	1139
sst	3241	0.9	71	0.25	20	0.17	0.44	948
	3251	0.8	76	0.25	20	0.16	0.43	948
sl.w. sst	3251	1.6	89	0.25	0	0.34	0.92	953
sst	3252	2.3	75	0.30	20	0.52	1.16	1005
sl.w. sst	3261	0.3	84	0.25	10	0.07	0.17	953
	3261	0.3	83	0.25	10	0.06	0.17	953
sst	3262	1.3	71	0.30	20	0.28	0.62	960
laminite	3281	1.9	51	0.08	30	0.08	0.66	769
	3282	5.9	75	0.07	15	0.31	2.97	844
	3282	6.8	56	0.06	30	0.23	2.57	750
laminite	3282	6.1	47	0.09	40	0.26	1.91	634
siltst/lam.	3301	3.2	58	0.10	35	0.19	1.24	866
siltst/lam.	3302	2.5	46	0.09	40	0.11	0.78	677
Average		2.1	73	0.25	18	0.33	0.95	939
St. dev.		1.7	14	0.13	10	0.31	0.71	169
COV (%)		82	19	50	57	95	75	18

sl.w. = slightly weathered; med.gr. = medium grain size; sst = sandstone; lam. = laminite = laminated siltstone.

Generally the wear rate during rock cutting is higher. The wear is due to chipping of fragments of the tool during impact. Examination of Tables 16.3 and 16.4 shows that high values of cutting wear relate to stronger rock, i.e. cutting wear relates to rock strength in particular.

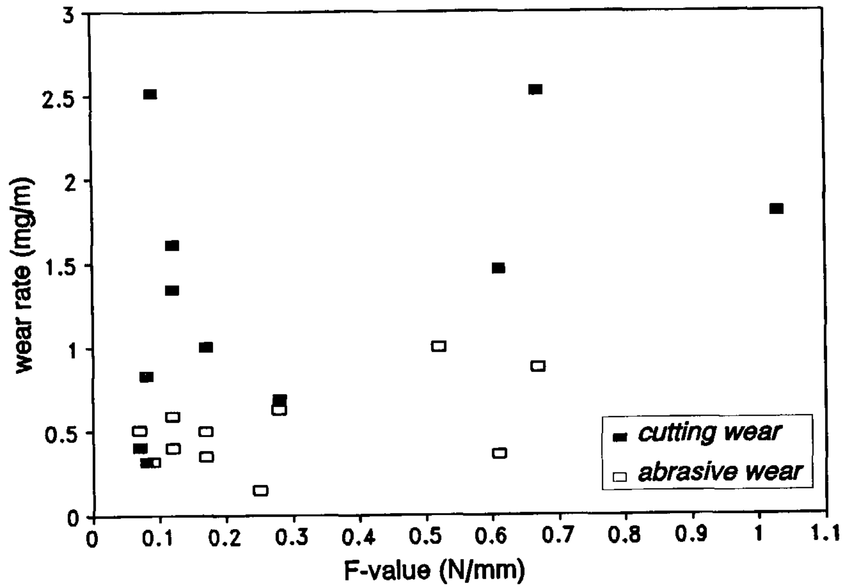


Figure 16.3 Results of cutting and abrasion tests on aged cores from the overwater boreholes, Sydney Harbour tunnel, compared with the F-values of the samples.

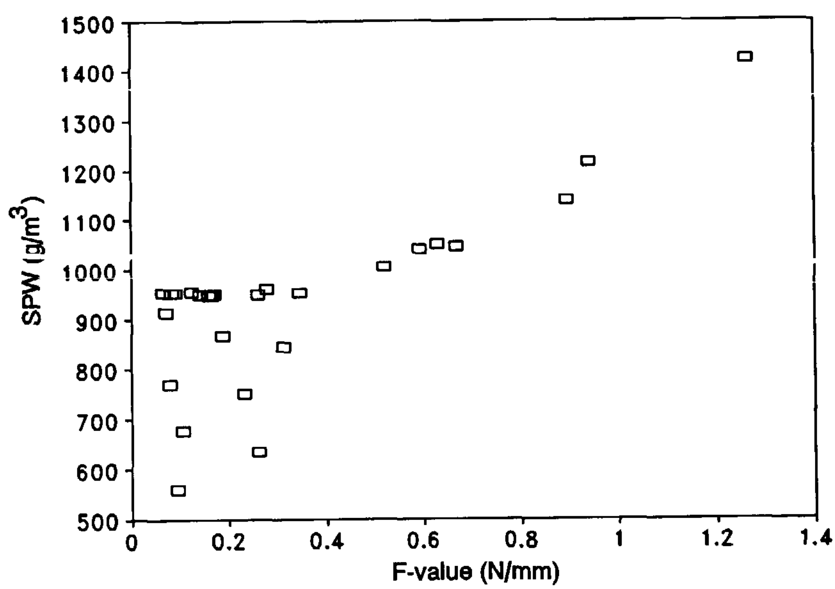


Figure 16.4 Specific wear (Equation 13.3) calculated for a feed of 5 mm, compared with the F-value (data from Table 16.3).

Table 16.4 Results of cutting and abrasion tests on aged cores of the overwater boreholes and calculated F, F_{mod} and SPW values.

Core Number	Cutting wear (mg/m)	Abr. wear (mg/m)	F (N/mm)	Fmod ($N/mm^{1.5}$)	SPW_{5mm} (g/m^3)
3001	1.61	0.59	0.12	0.21	955
3002	2.52	0.32	0.09	0.70	560
3201	2.53	0.88	0.67	1.28	1045
3211	1.35	0.40	0.12	0.27	952
3212	0.32		0.14	0.35	953
3213	1.46	0.36	0.61	1.01	1045
3231		0.35	0.17	0.41	950
3233	1.80		1.03	1.61	1258
3241	1.00	0.50	0.17	0.44	948
3251		0.15	0.25	0.68	951
3252		1.00	0.52	1.16	1005
3261	0.41	0.51	0.07	0.17	953
3262	0.69	0.63	0.28	0.62	960
3281	0.83		0.08	0.66	769

The SPW-value has been calculated for a feed of 5 mm, at which the Equation 13.3 predicts that wear occurs in the mode III (Figure 16.4). The relation of SPW at higher feed with the F-value is not surprising, since the same rock parameters are involved in both rock wear factors (Chapter 13). Therefore we continue by only referring to the F-value as a factor describing abrasiveness.

16.3 SYDNEY HARBOUR ROCK DREDGING

Sydney Harbour is an old river valley partially filled with Quaternary sediments, deposited during the Pleistocene periods of high relative sea level. The central part of the trench excavation for the Sydney Harbour Tunnel therefore was in soft sediments. Only at the ancient valley sides -partially weathered- Hawkesbury Sandstone had to be excavated. From the site investigation reports and the over-water borehole records estimates of F-value have been made (see Chapter 16.2).

It was attempted to estimate an average F-value (F-mass) for the section of the borehole actually excavated by the dredger. This was done by considering the variation in strength (PLS) and petrography recorded in the relevant section of the borehole. The results are given in Table 16.5 and Figure 16.5.

Figure 16.5 shows plots of estimated F-value considered representative of the excavation zone in the boreholes and the tool consumption recorded near the

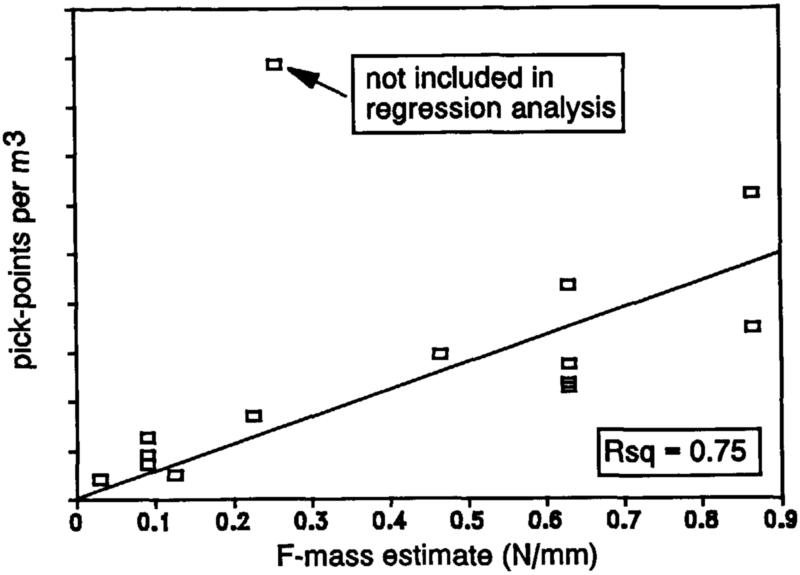


Figure 16.5 Tool consumption of the suction cutter dredger compared with the estimated F-mass value for the rock excavated.

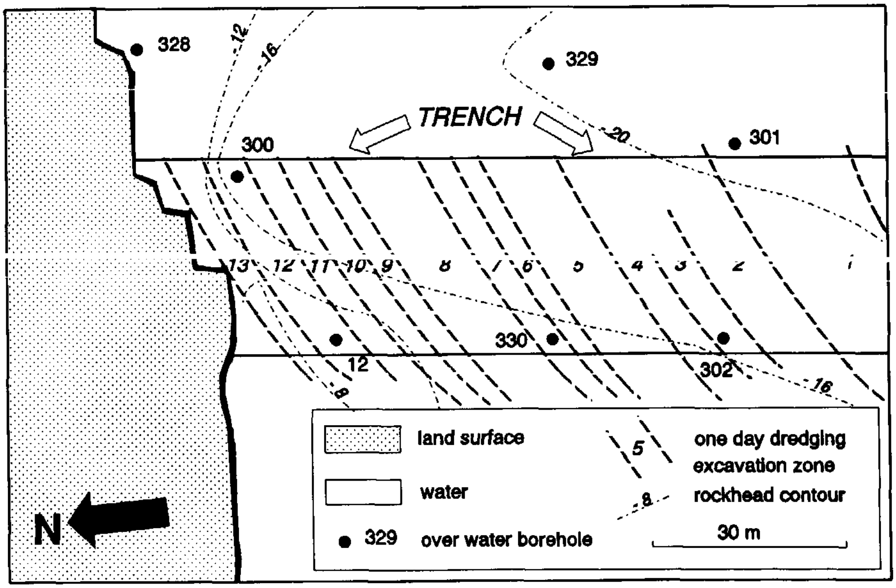


Figure 16.6 Part of the trench for Sydney Harbour Tunnel. Daily excavation zones and distribution of site investigation boreholes.

Table 16.5 Estimate of average strength, quartz content and grain size for the length of the borehole actually excavated for the trench, to determine a characteristic F-mass.

BH no.	Length (m)	PLS (MPa)	BTS(est) (MPa)	Quartz (vol. %)	Size (mm)	F-mass (N/mm)
302 (N)	7.2	0.6	0.9	65	0.10	0.06
330 (N)	5.2	0.3	0.4	60	0.10	0.02
300 (N)	1.6	0.3	0.4	60	0.10	0.02
12 (N)	13.0	0.2	0.3	70	0.35	0.07
320 (S)	1.8	0.3	0.5	68	0.35	0.12
321 (S)	4.0	0.3	0.5	80	0.50	0.18
322 (S)	0.5	-extremely weathered rock-				
323A (S)	5.0	1.0	1.5	77	0.60	0.69
324 (S)	2.7	0.8	1.2	67	0.60	0.48
325A (S)	2.9	1.4	2.1	60	0.40	0.50
326A (S)	4.0	0.7	1.1	71	0.50	0.37

boreholes. The problem with tool consumption data is that these are not always related to normal practice rock cutting dredging. Only reliable tool consumption data were used and only F-mass estimates of boreholes sufficiently close to the excavated rock zone. The tool consumption data were compiled by Watson (1990). He took care that the data on tool consumption were representative of normal practice and were not based by dredging operator factors or exceptional dredging conditions. The F-mass estimates were made as follows. First an average strength (PLS) was calculated for the section of the borehole which was in the excavation zone (Table 16.5). Also an estimate of average quartz content and grain size was made. Then the daily excavation reports were examined and it was examined which boreholes were close to the excavated zone (Figure 16.6). If no boreholes were close, the data were discarded. With respect to excavation volume, the density of borehole data is very small. The estimated means of F-mass and the tool consumption, given in Figure 16.5, show a linear trend. Since production rate is related to tool consumption, it is not surprising that also production relates to the F-mass estimates (Figure 16.7).

The conclusion of this survey is that:

1. The cutting and abrasion wear rates do not correlate clearly with the F-value (or *SPW*) for the Hawkesbury sandstone.

2. The F-value correlates with tool consumption (and thus abrasive wear of the pick-points).

3. The cutting- and abrasive wear rates as determined with the cuttability tests do not correlate with actual tool consumption in this case.

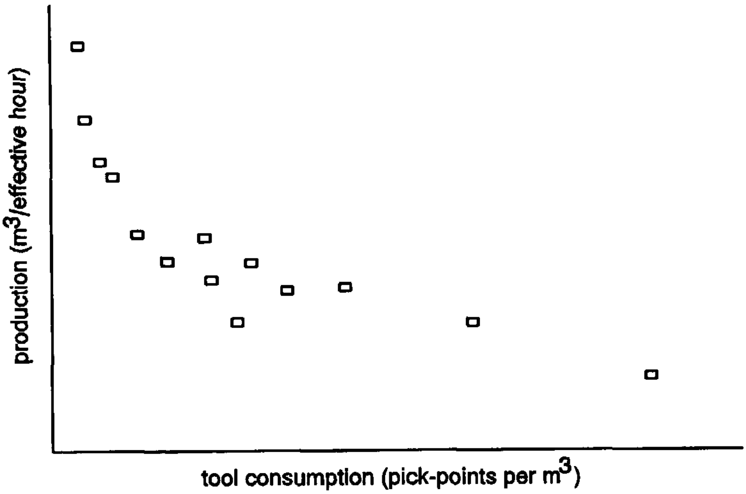


Figure 16.7 The production of the suction cutter dredger Kunara was clearly related to the tool consumption.

16.4 MALABAR OUTFALL DECLINE TUNNEL

The decline tunnel traverses two sedimentary rock formations, the Hawkesbury Sandstone and the underlying Newport formation, which consists mainly of siltstone and sandstone units. Lowe and McQueen (1988) give an extensive description of the engineering geological properties of these rocks. They have divided the Hawkesbury Sandstone into 5 types, based on petrographic characteristics:

- Type 1: fine to medium grained sandstone, average quartz content 63 vol.%.
- Type 2: fine to medium grained, quartz content 78 vol.%.
- Type 3: medium to coarse grained, quartz content 78 vol.%.
- Type 4: medium to coarse grained to conglomerate, quartz content 76 vol.%.
- Type 5: fine to coarse, carbonate cemented, 57 vol.% quartz.

Average data necessary to estimate the *F*-value are given in Table 16.6. As regards the strength of type 4, the unconfined compressive strength considered characteristic by Lowe and Mc Queen is 55 MPa (Table 16.6). The strength measured by Roxborough on the sample used for the cutting test was 26 MPa, however. If this value is used for the calculation of *F*, an *F*-value of 0.91 is obtained, which would give the SST 4 a more logical position in the graph of Figure 16.8. In Table 16.6 the average wear values derived from the cutting and abrasion test of Roxborough (1987) and the pick consumption data of the two roadheader types used (Alpine Voest AM75 and AM100) are given. Since no tensile strength test results were reported BTS is estimated to be 1/13 of UCS, following Pells (1985). Since the Newport (Nwp) rocks, siltstones and sandstones, were described as being very similar to type SST 1 Hawkesbury Sandstone and also pick consumption data were given, these have been included.

Table 16.6 Summary of geotechnical data for the Malabar Decline Tunnel (Lowe & McQueen 1988).

SST type	Mass vol.%	Grain size mm	Qtz vol.%	UCS MPa	BTS est. MPa	F est. N/mm	Abr. wear mg/m	Cutting wear mg/m	AM75 pps/m³	AM100 pps/m³
1	39	0.30	63	32.0	2.5	0.47	1.09	2.16	0.302	0.866
2	*	0.30	78	45.0	3.5	0.81	2.19	3.13	1.189	0.950
3	*	0.50	78	45.0	3.5	1.35	2.45	4.10	1.189	0.950
4	7	0.60	76	55.0	4.2	1.93	1.30	3.93	-	-
5	1.5	0.40	57	-	-	-	2.35	3.07	-	-
Nwp.sst	23	0.13	60	38.0	2.9	0.23	0.42	1.30	-	0.535
Nwp.siltst	74	0.09	45	60.0	4.6	0.19	0.26	0.92	-	0.206

* Type 2&3 make up 50% of the rock mass excavated; they could not be distinguished in the tunnel, AM75 & AM100 are roadheader type tunnelling machines (Alpine Miners).

Figure 16.8 shows how the estimated average *F*-value relates to the laboratory wear data available. Apparently a relationship with the cutting wear and abrasive wear rates determined in the laboratory tests exists.

Tool consumptions of the Voest Alpine roadheaders are plotted against the *F*-value in Figure 16.9. For both an increase of tool consumption with *F*-value is shown. Due to the fact that no distinction could be made between type 2 and type 3 sandstone in the tunnel, the pick consumption data are for type 2&3 combined. Also the relative abundance of these two types in the boreholes apparently could not be estimated. This should be borne in mind when examining Figure 16.8 & 9. Type 2 differed from type 3 mainly in grain size, but this is an important property with regard to abrasiveness as can be deduced from their *F*-values.

Considering the manner in which the data have been obtained, any conclusion can only be tentative. In this case it is suggested simply that the *F*-value and the laboratory cutting- and abrasive wear rates relate to actual tool consumption.

16.4.1 *Discussion*

Several attempts have been made to relate laboratory - or in-situ derived rock parameters to actual excavation performance. Lack of sufficient data, which are difficult to extract from real projects and lack of sufficient borehole-derived information will always be a problem. Speight and Fowell (1987) explain that due to this it is necessary to use full-scale laboratory machines to obtain sufficient data. From the performance data of their laboratory road header they found, using sufficient data, that all excavation relationships investigated by them had power functions. The linear, or linear-with-intercept relationships found by earlier workers can be attributed to lack of sufficient data. The application of the cutting and abrasion parameters derived from Roxborough's cuttability and abrasion tests to performance prediction in the case of the Malabar Outfall by Lowe and McQueen (1990) has shown that such predictions are not accurate. If we use the Voest Alpine

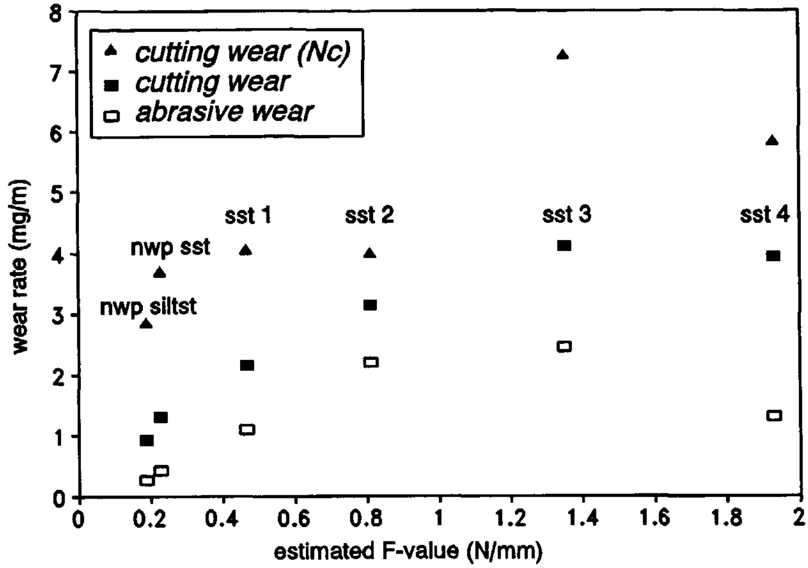


Figure 16.8 Estimated F-values of the sandstone types at Malabar tunnel and wear rates determined by the core cutting (Nc: carried out at Univ. of Newcastle-upon-Tyne, UK) and abrasion test.

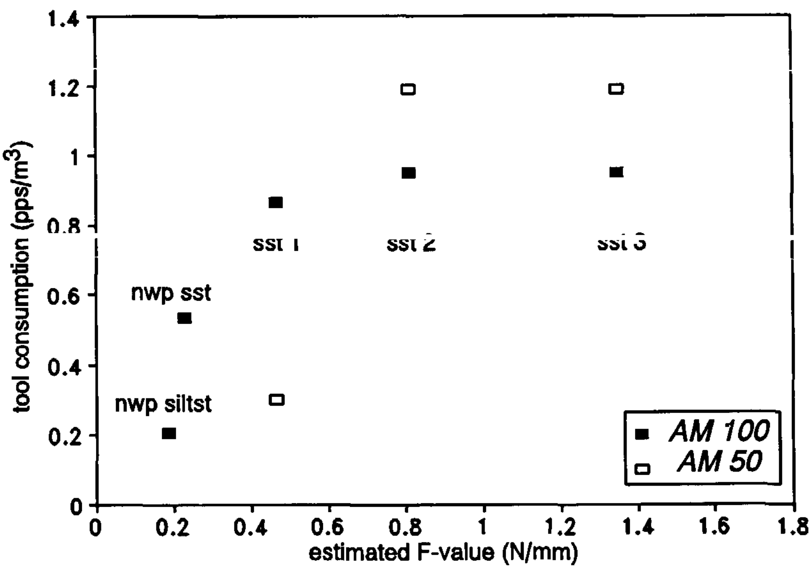


Figure 16.9 Tool consumption of Voest-Alpine roadheaders AM 75 & 100, compared with the representative F-values of the Malabar sandstone types.

graphs (Gehring 1991, pers. comm.; Braybrooke 1988), which are based on *F*-value and UCS, only in one case out of four is a proper estimate of pick consumption is obtained. The others fall outside the 10-30% accuracy boundaries mentioned by Gehring (1987). In the case of Malabar, the data show that the *F*-value and the cutting- and abrasive wear rates derived from the cuttability tests are related. Both methods also relate to pick consumption. In the case of the rock cutting dredging operation it is striking that the estimated *F*-mass value shows an apparently linear correlation with actual pick consumption, but that the cutting- and abrasion wear rates are not related to both *F*-value and pick consumption.

16.5 CONCLUSION

For the first time relatively accurate tool consumption data of a rock cutter suction dredger could be studied. In this case the wear factor *F*, derived by Schimazek, related reasonably well with tool consumption. The *SPW* relation of Deketh, also derived from laboratory tests gives comparable results. Another method commonly applied in rock tunnelling, the cutting wear and abrasive wear rates derived from the core cutting and core abrasion tests, did not correlate with the tool consumption of the rock cutting dredger. Both *F*-value and the cutting- and abrasion wear rates were related to the tool consumption of the roadheaders used to excavate the Malabar Outfall Decline in Sydney.

The amount of cutting or abrasion tests which can be performed on rock cores will always be limited. Therefore in site investigation practice use should be made of rock index tests which do relate to cutting performance and abrasive wear. Such tests include UCS, BTS, PLS, thin section examination (mineralogy, microscopic structure, grain shape, grain size), E-modulus and so on. For wear prediction it is necessary to calibrate the results of such tests against a trial-excavation with a real cutting machine at the site.

CHAPTER 17

Field observations of the performance of rock cutting trenchers

Although a general idea exists which rock properties influence tool consumption and production, studies which are devoted to back analysis of cutting or drilling projects are scarce. The validity of the ideas developed during this research was examined by studying the performance of rock cutting trenchers. The mechanical excavation occurs in dry condition and the larger cutting machines excavate trenches that are easily accessible for the geologist to survey the rock and soil properties. The results of this study are presented in this chapter. Most probably the general model developed from these observations will apply also to other excavation machines working on the drag cutting principle.

17.1 FIELD OBSERVATIONS OF ROCK CUTTING TRENCHING

The performance of T-850 rock cutting trenchers (Vermeer Manufacturing Inc. Pella, Iowa) has been monitored for this study in 16 projects (Giezen 1993, Den Hartog 1995, Deketh 1995c, Deketh et al. 1996).[69] These machines can cut trenches in rock up to about 1 m width and 3 m depth. The trencher excavates the rock using a boom of a length of 3.7 m.[70] A double chain runs over the boom. The chain carries base plates, which can be of variable size, according to the desired width of the trench. The base plates are mounted with bits, that normally are arranged in a V-pattern. The angle of the boom with the surface determines the depth of the trench (Figure 17.1). The boom with the rotating chain is lowered into the surface to the desired depth and the trencher moves backwards. The bits which cut through the rock can rotate freely in their pockets, ensuring a symmetric wearing of the bits. The excavated rock material is removed by the chain to a conveyer belt

[69] The monitoring of the T-850 trenchers was done by a team involving Dr. H.J.R. Deketh and M. Alvarez Grima MSc; M. Giezen, M.H. den Hartog and I.M. Hergarden performed MSc thesis studies related to the subject. Mario Alvarez Grima provided the data used in this Chapter.

[70] Booms can be replaced. Standard lengths are 6, 8, 10 and 12 feet.

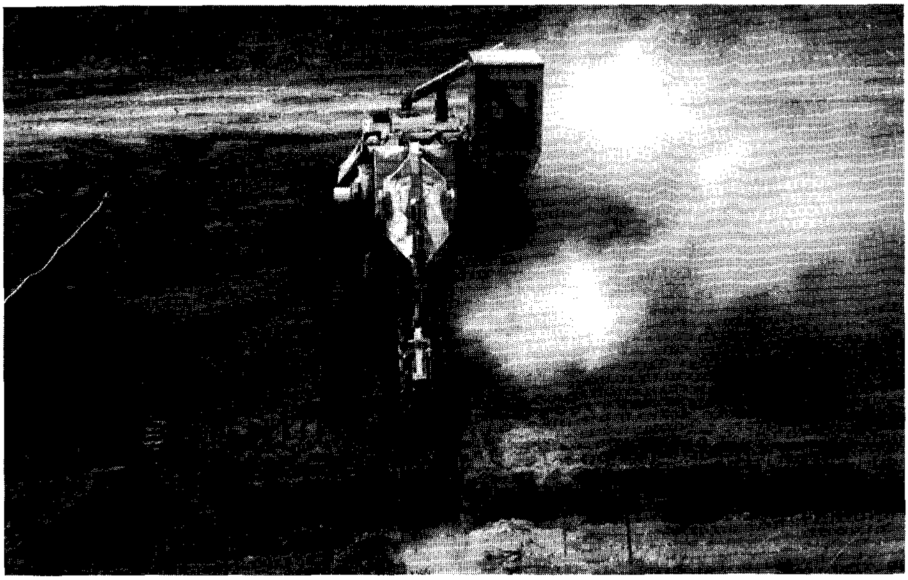


Figure 17.1 Vermeer T850 trencher, working at project 9, Langmeil Germany, in sandstone.

which transports and disposes the muck sideways of the trench.

The trencher is operated at full capacity when cutting through the rock. The hydraulic power (260 kW) is divided over the tracks and the cutting chain. The chain rotates with a velocity of about 3.3 m/s (19 rpm) that ideally remains constant. Generally the trencher is operated automatically by means of an auto-creep function, which holds the advance rate of the trencher at a maximum level. The advance rate is restricted by the dimensions of the trench and the resistance of the rock mass to excavation. The advance rate can vary between 0 and 27 m/min (1600 m/hr). The automatic operation reduces the influence of the operator on the way the trenching is performed. Comparison of different T-850 trenchers operated by different drivers is therefore more reliable than with other excavation machines, such as bulldozer rippers or roadheader tunnelling machines.

In most of the 16 projects the trenchers have been monitored for one complete working day. The advance rate of the trencher was recorded by clocking the progress of the trencher and all stops to obtain the effective working hours over the observed length of the trench. The bit consumption due to breakage and wear was determined. To establish bit wear, the exact position of each bit on the chain within a selected V-pattern section was noted as well as the type of bit. Bits were weighed before and after the work. Only already worn bits were used for the wear assessment, since new bits wear relatively fast in the beginning, until a steady shape has been reached. All bits which were changed or lost during the monitored period were recorded. The dimensions of the trench were measured and the geology of the trench was surveyed. The size of the trenches facilitated the geological surveying. In the trench the distribution of rock and soil types, weathering and the discontinuity orientation and density were mapped and recorded. A typical illustration of such a survey is given in Figure 17.2. The rock types of the observed trenches are given in Table 17.1 and the trencher performance data are given in Table 17.2.

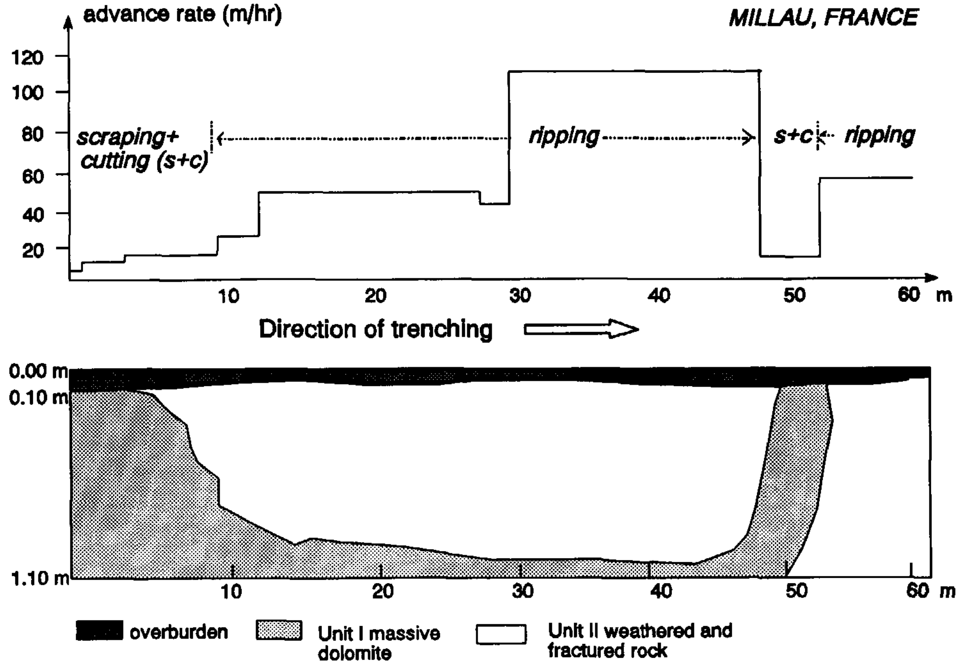


Figure 17.2 Example of the relationship of excavation mechanism (scraping & cutting and ripping) with rock type and advance rate of the trencher.

In Table 17.1 data on the trench dimensions and the rock characteristics are given. The trench dimensions varied for each project. This has a bearing on the interpretation of the rock mass data. The block size of the rock mass excavated should be compared with the cross section of the trench, to be able to judge whether a block could be easily dislodged. Dislodging will be easy when the dimension of the rock blocks is smaller than the cross section of the trench face. In the table the column block size gives the indication *massive* (m.) when the block size is larger than the trench dimension and dislodging cannot occur. To be able to study the rock cutting performance, the assumption was made that soil overburden or highly to completely weathered rock would not contribute significantly to the wear of the cutting bits, as compared with the contribution of massive rock.

The volume of rock cut in each trench was estimated and is given as a percentage of the trench dimensions in Table 17.2. The excavation rate in rock was established by multiplying the advance rate of the trencher (m/hour) by the width of the trench and the thickness of the rock unit. The total excavation rate was obtained by multiplying the advance rate by the width and the total depth of the trench. The bit consumption rate was calculated by dividing the total number of bits replaced or lost by the rock volume excavated.

Breakage mainly occurred due to high impact forces exceeding the strength of the bits or the tungsten carbide insert. Loss of the insert led to very rapid wear of the steel body of the bit. Failure of the steel body of the bit occurred mostly at the position where the shaft of the bit is not supported by the bit pocket. The *specific wear* for all bits on the chain was calculated by extrapolating the mass loss of the

Table 17.1 Characteristics of the observed trenches.

	Project name/ location	Trench dimensions[1] (m^2)	Overburden type	Rock type	Strength UCS (MPa)	Block size (dm^3)
4	Avoriaz (F)	0.77 (29%)	wr. & b.	limestone	163	<280
6	Billingsbach (D)	3.62 (40%)	clay	limestone	173	36
8	Fabrègues (F)	0.56 (75%)	s. & b.	limestone	107	14
5	La Lauze (F)	0.36 (24%)	s. & b.	limestone	80	4-m.
7	Monaco (F)	1.21 (100%)	-	limestone	143	*m.*
13	Castelneau (F)	1.84 (19%)	wr. & b.	limestone	104	105-m.
10	Wölland (D)	2.16 (100%)	-	limestone	258	<4
14	La Couronne (F)	1.89 (100%)	-	limestone	212	*m.*
15	Millau (F)	0.38 (33%)	clay	dolomite	100	*m.*
16	Büschdorf 1 (D)	1.22 (78%)	-	dolomite	80	5-m.
9	Langmeil (D)	1.54 (100%)	-	sandstone	41	*m.*
3	Oberndorf (D)	1.81 (31%)	wr. & s.	sandstone	47	*m.*
1	Schnarrtanne 1 (D)	2.71 (59%)	wr.& rb./p.	granite	37	500-m.
2	Schnarrtanne 2 (D)	3.16 (54%)	wr.& rb./p.	granite	110	1400-m.
11	Sybillenbad 1 (D)	3.74 (100%)	-	schist	17	m.
12	Sybillenbad 2 (D)	2.58 (100%)	wr. & b.	schist	10	<0.5

(F)= France;(D)= Germany; m.= behaved massive during trenching; *m.* = massive; wr.= weathered rock; b.= boulders; s.= soil; rb./p.= road-base/pavement.

[1] The percentage between the brackets represents the relative amount of hard rock encountered in the trench.

bits in the measured section and divide by the rock volume excavated. The type of wear marks on the bits were observed. Parallel grooves indicate two-body abrasive wear, irregular grooves indicate three-body abrasive wear (Figure 17.3). Damage on the tungsten carbide inserts occurred by micro-chipping. The surface temperature of the bits was also estimated during monitoring. When the bits were very hot (could not be touched by hand) parallel grooves were normally present (two-body abrasive wear) and it is inferred that the steel surface was weakened as well (adhesive wear, see Figure 9.16). Apart from breakage and wear as cause for bit replacement, sometimes bits were lost due to loosening of the attachment clips of the bits.

17.1.1 *Excavation modes during trenching*

The trencher observations clearly show that two main modes of excavation can be distinguished: cutting and ripping. Ripping occurs when the cutting tool (bit or pick point) is loosening the blocks of rock bounded by the discontinuities of the rock mass. When the spacing of the discontinuities or joints becomes wider, the bits start

Table 17.2 Excavation performance and tool consumption data of the trenching operations.

Project	Trencher advance rate (m/hr)	Rock excavation rate (m^3/hr)	Total excavation rate (m^3/hr)	Bit wear (g/m^3)	Bit consumption (no./m^3)	Type of bit wear	Estimated bit temp.
4 Avoriaz	9.7	2.1	7.2	8.57	4.5	b. mc.	hot
6 Billingsbach	4.0	5.6	13.9	0.007	0.3	b.	warm
8 Fabrègues	53.2	23.6	30.8	0.028	0.2	b.	warm-hot
5 La Lauze	92.8	7.7	32.1	0.01	0.3	b.	warm
7 Monaco	9.7	7.17	7.17	0.08	1.2	b. 2bw.	hot
13 Castelneau	12.15	5.5	10.80	0.061	0.29	b. 3bw.	warm
10 Wölland	62.0	134.8	134.8	0.002	0	n.v.	warm
14 La Couronne	1.9	4.47	4.47	0.16	3.66	b.	hot
15 Millau	23.55	5.80	10.42	0.71	0.39	2bw. mc.	hot
16 Büschdorf 1	7.40	8.19	9.24	0.0187	0	n.v.	hot
9 Langmeil	7.9	11.9	11.9	0.006	0	2bw.	warm
3 Oberndorf	15.2	8.1	26.3	0.25	0	3bw.	cool
1 Schnarrtanne 1	4.6	7.4	12.5	0.21	0.2	2bw.	hot
2 Schnarrtanne 2	2.6	4.6	8.48	0.47	0.4	2bw.	hot
11 Sybillenbad 1	2.4	8.7	8.74	0.1	0.1	2bw.	hot
12 Sybillenbad 2	25.9	60.2	60.2	0.0183	0	3bw.	cool

b. = breakage; 2bw. = two-body abrasive wear (parallel grooves); 3bw. = three-body abrasive wear ; mc. = micro-chipping; n.v. = not visible; hot = at least 600 °C at the bit surface during trenching.

to cut into the rock material, Figure 17.4. It is thought that such a transition from a cutting mode, where the bits are cutting into the rock material, and a ripping mode, where the bits mainly function as looseners of rock blocks, will exist for any rock cutting machine using the drag bit principle. Regarding rock properties, this statement emphasizes the importance of rock material properties when cutting occurs and rock discontinuities (fractures, joints etc.) when ripping occurs.

If the rock material is very strong, the feed of the bits of the trencher will be low and scraping occurs instead of real cutting (see Chapter 12.2). Changes of excavation mode are reflected in the advance rate of the trencher. Sudden jumps in the advance rate indicate transitions from cutting-scraping to ripping in the section of the trench at Millau (Figure 17.2).

Independent of the actual excavation mode of the trencher (ripping or cutting), the maximum *feed* of the bits into the rock mass can be calculated (Giezen 1993, Deketh 1995):

$$feed_{per\ bit} = \frac{advance\ rate}{rpm_{chain} * 60 * N_R} * \sin\alpha \ \ (mm/rev) \qquad (17.1)$$

where the advance rate is given in m/hour, the chain speed in the auto creep function

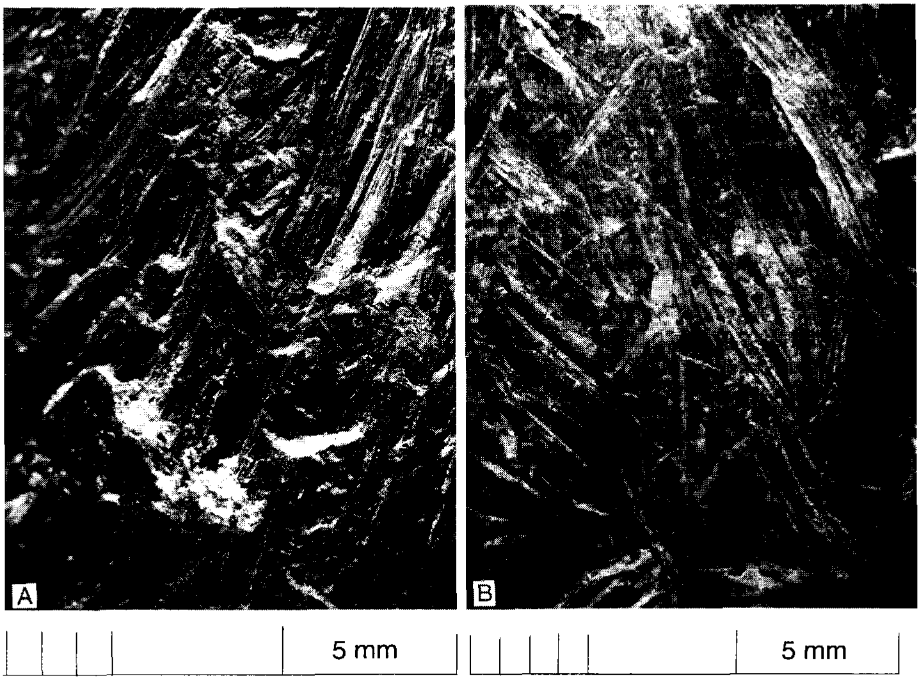


Figure 17.3 Wear marks on bit surfaces. A. Sybillenbad, project 11, parallel grooves and plastic deformation. B. La Lauze, project 4, irregular grooves (three-body abrasion), local plastic deformation.

is 19 rpm, N_R is the number of bits acting in one vertical line over the chain and α is the angle of the boom with the horizontal. Figure 17.5 shows a plot of the excavation rate against the feed calculated according to Equation 17.1. The high feed in projects 5, 8, 10 & 12 can be attributed to the relatively small block size (Table 17.1), leading to a ripping excavation mode. The relatively lower excavation rate of projects 5 & 8 is related to a small size of the trench and the block size.

It appeared that the *block size* effected the excavation mode, as illustrated by Figure 17.6. In Figure 17.6 block size is described by the orthogonal volume (multiplication of the average spacing of the three major discontinuity sets).If the blocks were small enough (relative to the trench dimension and the trencher chain), a ripping mode of excavation occurred. Otherwise the bits were scraping or cutting the rock. A transition size of blocks gave particular problems. Blocks of the size class 0.04 - 0.3 m^3, when detached from the excavation face, tended to remain tumbling between the chain and the face. The blocks were too large to be transported and needed to be fragmented to a smaller size. This situation led to decrease of excavation rate and breakage of bits (high bit consumption) in the case of strong rock (for example project 4, Table 17.1 & 2). Similar transport problems occurred with other rocks in this size class, but their lower material strength facilitated size reduction (for example project 5, La Lauze). *Block shape* influences excavation (Den Hartog 1996). Two shapes have been encountered in the projects, most rocks are blocky, some tabular (i.e. projects 9, 10, 11, 12). If favourable, the orientation of the tabular shape can assist excavation (Chapter 8.2.2).

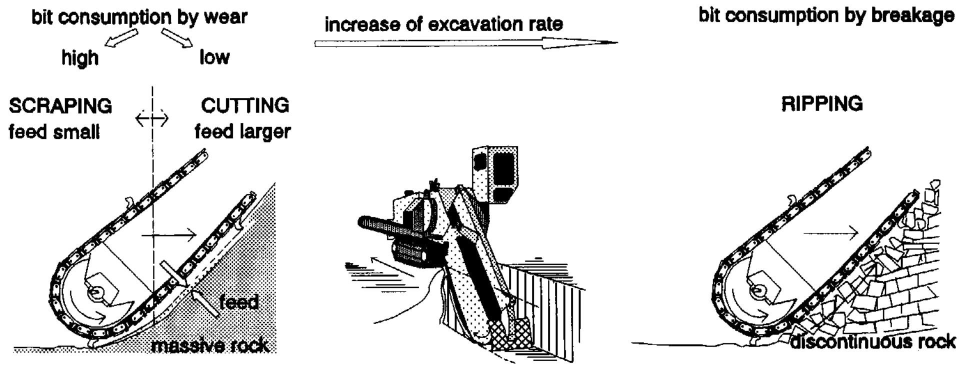


Figure 17.4 Tool consumption of rock trenchers is determined by wear when rock cutting takes place and mainly by breakage when ripping occurs (Deketh et al. 1996).

The mode of excavation is also reflected in the type of muck that is disposed. Blocks of rock result when ripping, fine sand to gravel sized gouge result when cutting.

17.2 ROCK PROPERTIES INFLUENCING TRENCHER PERFORMANCE IN MASSIVE ROCK

In *massive rock*, the major excavation mode is scraping and cutting. The performance in massive rock is interesting, because it can be compared with the results of the laboratory experiments of Deketh (1995), described in Chapter 12. The bit enters the rock at shallow depth to reach a maximum penetration after a certain displacement (compare Figure 12.6 & 17.4). The advance rate (travel speed) and excavation rate are considerably lower when the trencher has to cut the rock (Figure 17.2 & 17.6).

17.2.1 *Unconfined compressive strength*

If the UCS of the rock material is compared with the *bit consumption* (Figure 17.7 and Table 17.1 & 2), it is obvious the bit breakage is important at UCS values above 100 MPa. The projects 7 (Monaco) and 14 (La Couronne) involved strong limestones. The boom angle was only 20° in Monaco (small trench dimensions), but the normal 60° in La Couronne. Calculated average maximum feed values were 0.9 mm/rev and 0.3 mm/rev per bit respectively. Due to the low feed, a high amount of scraping is expected. The bits were very hot and the surface showed two-body abrasion grooves and relatively high wear rates for limestones (wear mode I) (Figure 17.8). When operating in these rocks, the boom vibrates heavily and apart from scraping also dynamic impact occurs. Loss of the tungsten carbide inserts and breakage of the bits resulted. Breakage is the major contributor to bit consumption (Table 17.2) and the causes for breakage, very high rock strength and unfavourable block size, should be considered in prediction models.

The relationship of *specific wear* and UCS, given in Figure 17.8, illustrates

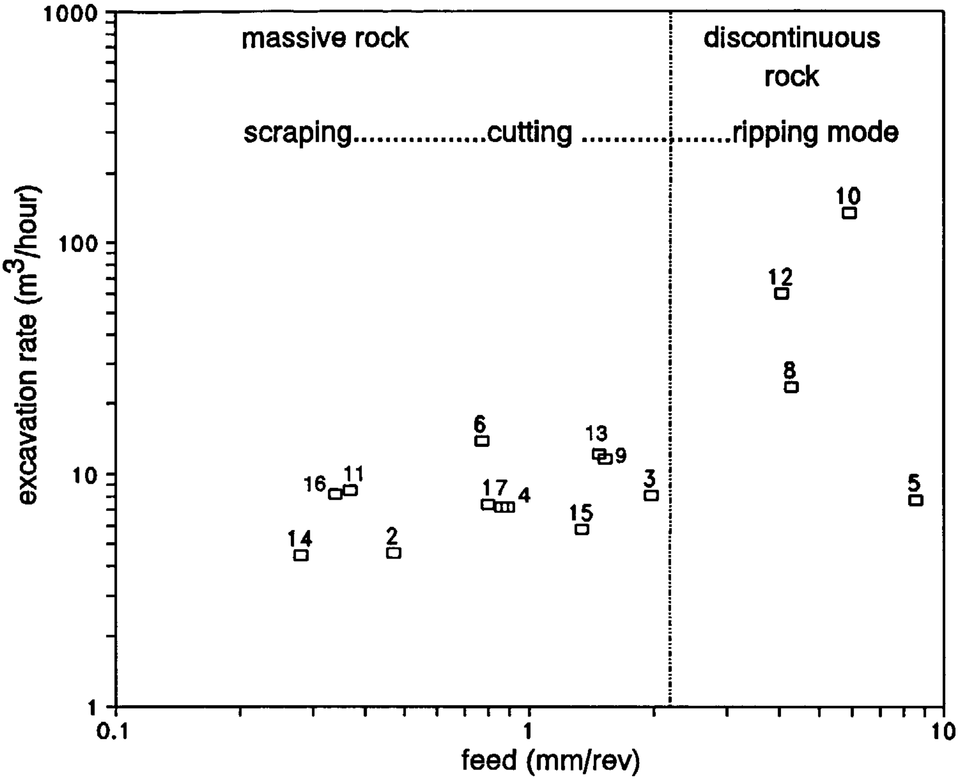


Figure 17.5 Calculated maximum feed per bit compared with the rock excavation rate of the T-850 trenchers in the 16 projects (see Table 17.1).

distinct trends for two rock groups: the *carbonate rocks* (limestones and dolomites) and the *silicate rocks*. With increasing UCS the specific wear of the carbonate rocks increase. This can be explained by the increasing amount of scraping, resulting in the combined two-body abrasion and adhesion wear of wear mode I type. The massive silicate rocks in general show a higher specific wear and a higher increase of specific wear with strength of the rock. The available data point to a linear relationship of specific wear and UCS. The silicate rocks differ from the carbonate rocks by their high content of minerals harder than 5 on the scale of Mohs (Table 17.3, Figure 17.9). Two rocks do not fit the two linear trends visible in Figure 17.8. Rock type 15 (a dolomite from Millau) showed a particularly high wear of the bits. In this project microchipping occurred on the bits, which may explain the higher amount of wear. The average maximum feed for this rock was 1.4 mm/rev per bit and the excavation rate low (Figure 17.5). The ferruginous sandstone 9 gave hardly any wear. This sandstone differs from the other sandstones and the igneous and metamorphic rocks by the presence of a weak iron hydroxide cement, that should explain the low wear rate, despite the high content of quartz in this rock (Figure 17.9, Table 17.3); the average maximum feed was 1.55 mm/rev per bit, Figure 17.5. Both rock 9 and rock 15 behaved contrary to the expectation based on content of hard minerals and rock strength characteristics alone.

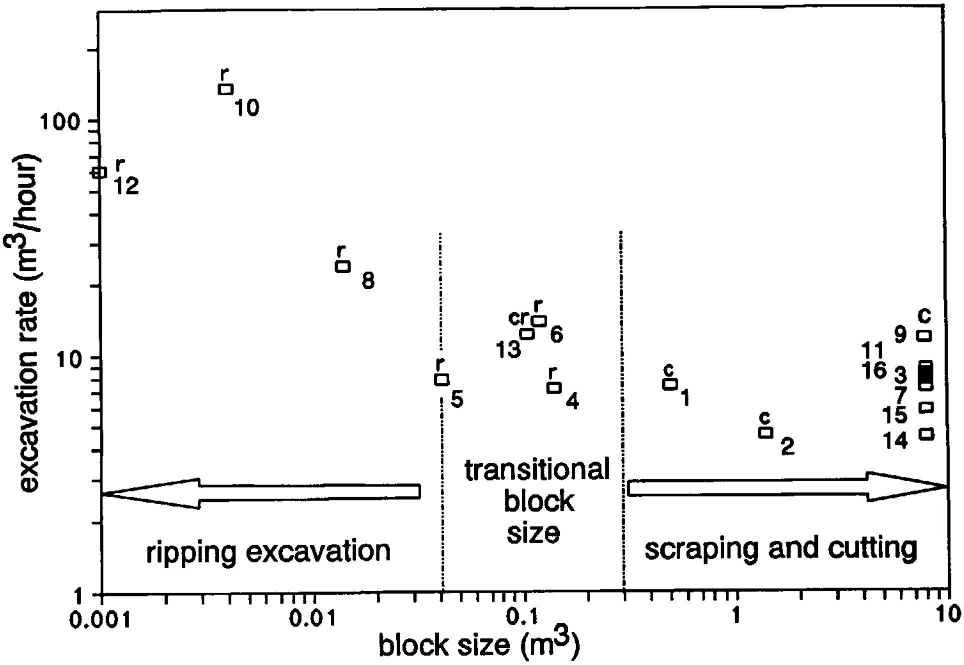


Figure 17.6 Excavation rate is assisted by small block size (r=ripping), when block size is large scraping and cutting (c) occurs. The range of block sizes causing problems with transport is indicated.

17.2.2 *Indicators of abrasiveness*

Mineral hardness (Figure 17.9), grain size and strength (UCS, BTS, PLS; Table 17.3) are the rock parameters known to relate to abrasive wear (Chapter 13). Figures 17.10 - 12 illustrate the results of the abrasiveness indicators F-value (Equation 13.1; Figure 5.11), modified F-value (Equation 13.2; Figure 17.11) and SPW (Equation 13.3, mode III; Figure 17.12). The SPW mode III and the F-value give similar results, as was found before (Chapter 16). The modified F-value gives a better indication of specific wear for the sandstones. The latter may reflect the general better correlation of PLS with trencher data, compared to BTS (Table 17.3).

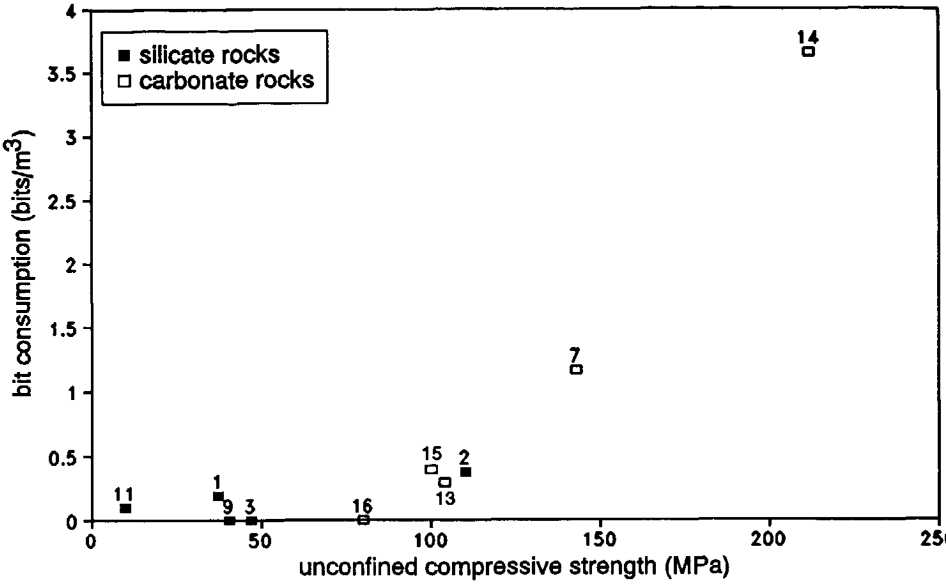


Figure 17.7 Bit consumption by T 850 trenchers of the massive rocks compared with the unconfined strength of the rock materials.

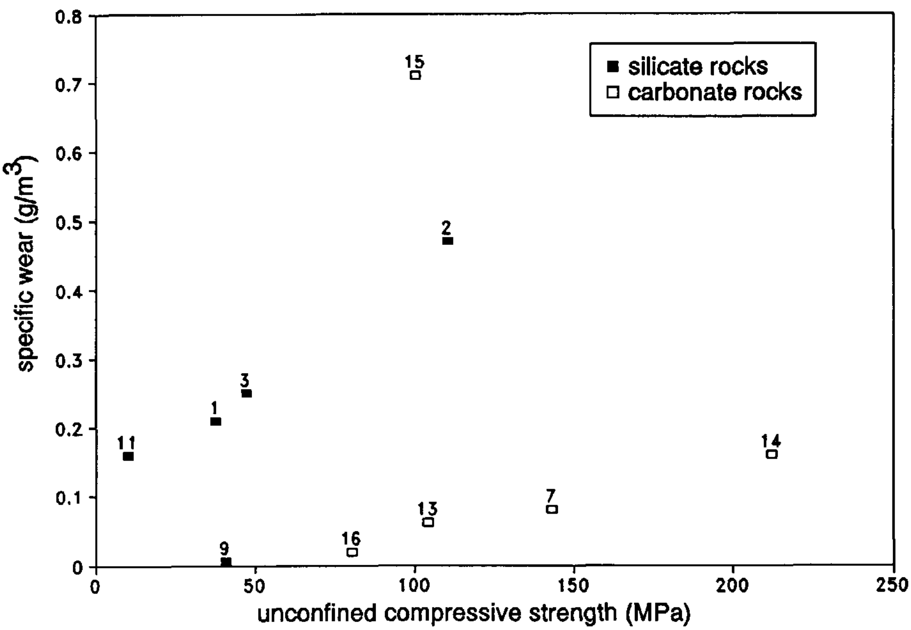


Figure 17.8 The specific wear of the bits of the trencher compared with the unconfined strength of the massive rocks.

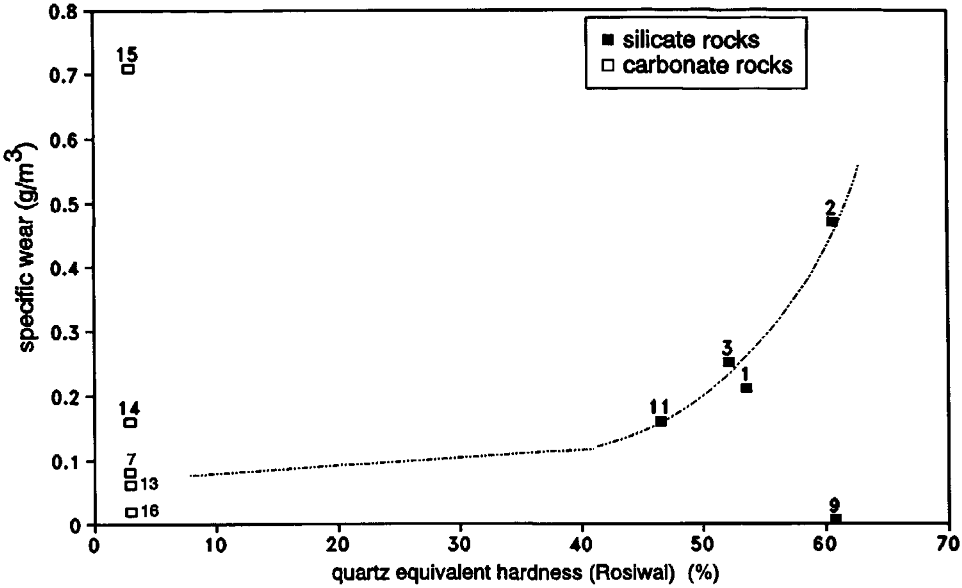


Figure 17.9 Specific wear of the trencher bits compared to the Rosiwal Hardness of the rock material of the massive rocks.

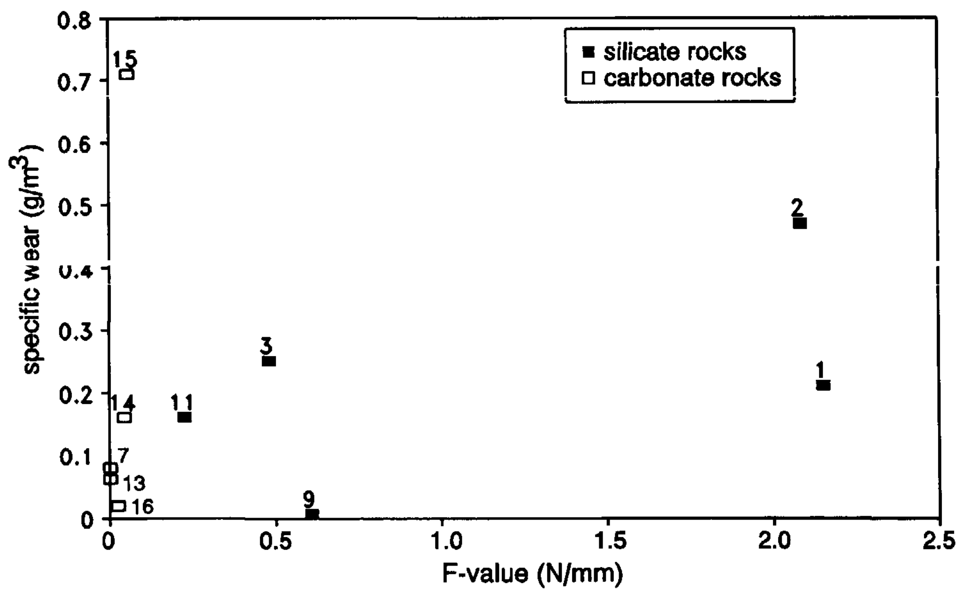


Figure 17.10 Comparison of the F-value abrasiveness number with the specific wear of the bits in the massive rock types.

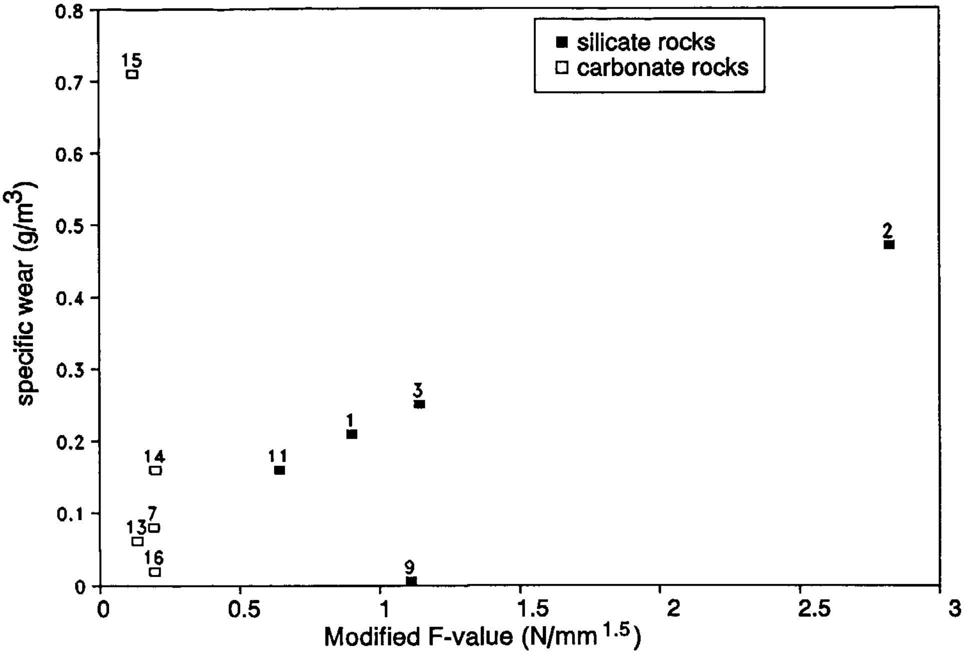


Figure 17.11 The modified F-value compared with the specific wear of the trencher bits.

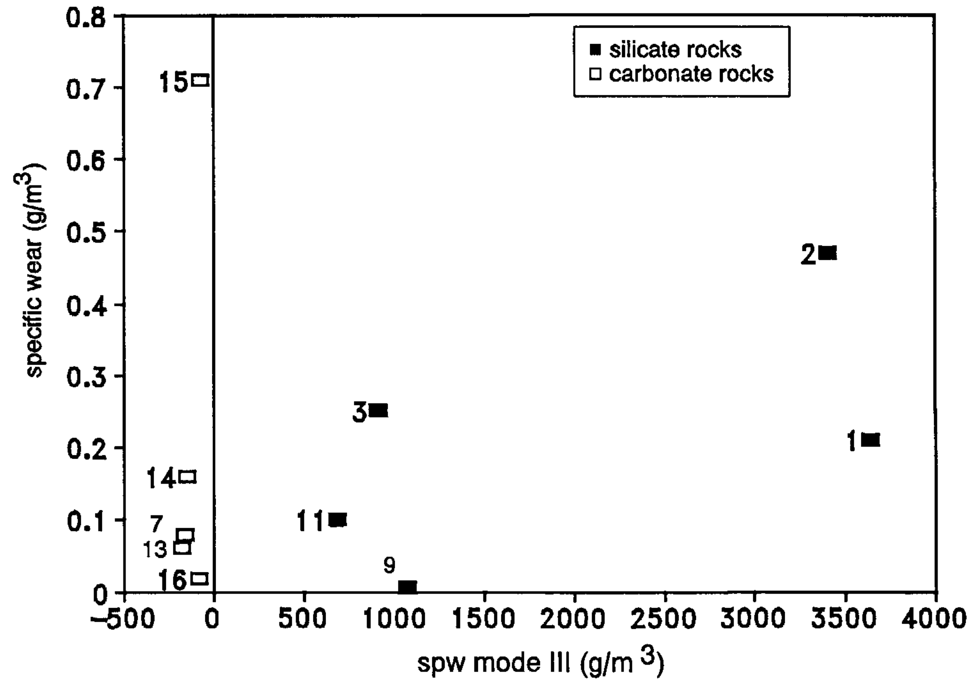


Figure 17.12 The SPW prediction for mode III (Deketh 1995) shows a similar pattern as the F-value (see Figure 17.10).

Table 17.3 Material properties of the rocks at the trenching sites.

	Project name/location	Rock type	BTS (MPa)	UCS (MPa)	PLS (MPa)	Grain size (mm)	Qz.eq. (vol. %)
4	Avoriaz (F)	limestone	8.6	163	5.60	0.15(*) - 4.9(**)	12.44
6	Billingsbach (D)	limestone	7.1	173	3.60	< 0.002	3.00
8	Fabrègues (F)	limestone	7.7	107	3.95	< 0..002	3.00
5	La Lauze (F)	limestone	5.5	80	4.50	0.24	4.06
7	Monaco (F)	limestone	8.1	143	5.06	< 0.002	3.00
13	Castelneau (F)	limestone	8.6	104	4.53	< 0.002	3.00
10	Wölland (D)	limestone	8.6	258	6.26	< 0.002	2.85
14	La Couronne (F)	limestone	7.5	212	6.70	0.20	3.00
15	Millau (F)	dolomite	4.7	100	4.10	0.40	3.00
16	Büschdorf 1 (D)	dolomite	5.1	80	5.18	0.10	3.00
9	Langmeil (D)	sandstone	4.0	41	1.83	0.25	60.79
3	Oberndorf (D)	sandstone	2.3	47	2.20	0.40	52.02
1	Schnarrtanne 1 (D)	granite	2.7	37.1	1.40	1.50	53.50
2	Schnarrtanne 2 (D)	granite	4.9	110.2	4.70	0.71	60.60
11	Sybillenbad 1 (D)	schist	3.4	37	1.38	0.14	46.49
12	Sybillenbad 2 (D)	schist	3.5	10	1.38	1.18	83.25

(*) average grain size of the silica.
(**) average grain size of the breccia particles.

17.2.3 *Rock ductility*

In order to get an idea whether the ductility of the rocks plays a role in the wear of the bits, Hergarden (1997) performed a study on some of the projects where cutting occurred in massive rock. She determined the dependence of the area of contact of the bits with the rock of the feed, for each project and each brand of bit used. Also an estimate of the force on the bits was made. In this way estimates of contact stresses were made. Furthermore the bit surfaces were studied on wear phenomena and polished sections were made of the bits. The tungsten carbide inserts and the steel top part were studied by reflection microscopy. From the microstructural changes and phase transformations that occurred near the contact surface, Mr. Colijn (Dept. of Materials Science, Delft University) interpreted minimal contact temperatures. Finally triaxial tests were performed to determine the brittle-ductile transition stresses of the rocks. The data are summarized in Table 17.4 & 5. From the data no clear conclusions can be drawn as to the effect of the influence of ductility on the rate of wear. The data is further discussed in Appendix D.

Table 17.4 Estimated bit-rock contact stresses, the rock failure mode at that stress as determined by Hergarden (1997), compared with the bit wear and consumption of trencher projects in massive rock.

Project	rock type	contact stress MPa	stress region	surface temp. °C	Bit wear g/m³	Bit cons. no./m³	Wear type	Excav. rate m³/hr
1 Schnarrtanne I	granite	158-308	-	>600	0.21	0.2	2 bw	7.4
2 Schnarrtanne II	granite	205-732	B	hot	0.47	0.4	2 bw	4.6
3 Oberndorf	sandstone	56-177	BD	<500	0.25	0	3 bw	8.1
9 Langmeil	sandstone	161-323	BD/D	<200	0.0062	0	2 bw	11.9
11 Sybillenbad I	schist	295-1093	D	hot	0.08	0.1	2 bw	8.7
7 Monaco	limestone	145-283	B/BD	~600	0.08	1.2	2 bw	7.2
13 Castelnau	limestone	119-345	BD/D	>500	0.061	0.29	3 bw	5.5
14 La Couronne	limestone	428-855	D	~700	0.16	3.66	breakage/ 3 bw	4.5
16 Buschdorf I	dolomite	533-1066	D	hot	0.0189	0	not visible	8.2

B = brittle failure regime; BD = transition brittle-ductile; D = ductile failure; 2 bw = two-body wear; 3 bw = three-body wear. Stress region is determined by comparing with BD stress found by triaxial testing, see Table 17.5.

Table 17.5 Results of rock tests carried out by Hergarden (1997), showing parameters relating to ductility; compare with Table 17.4.

Project	rock type	UCS (MPa)	Destruction work (kJ/m³)	UCS/BTS	m-value	BD - σ_1 (MPa)	stress σ_3 (MPa)
1 Schnarrtanne I	granite	37	113	13.7	-	-	-
2 Schnarrtanne II	granite	110	-	22.4	29.9	>600	>60
3 Oberndorf	sandstone	47	131	20.4	9.6	180	35
9 Langmeil	sandstone	41	148	10.3	15.8	200	45
11 Sybillenbad I	schist	17	67	4.9	10.2	450	65
7 Monaco	limestone	143	212	17.4	10.6	165	45
13 Castelnau	limestone	104	134	16	15.8	300	45
14 La Couronne	limestone	212	364	28.3	8.3	450	70
16 Buschdorf I	dolomite	105	239	14.2	1.8	235	35

For the determination of destruction work, m-value and BD transition stress, see Appendix C & D.

17.3 CONCLUSIONS

The 16 projects that are studied up to now do not give sufficient data to make definite conclusions. Some important general points can be derived from the discussion in the previous section.

1. Dimension of the trench and block size and shape determine whether ripping (dislodging and transport of blocks) occurs. This is favourable for production. Production rates above 10 m^3/hour indicate ripping. Calculated maximum feeds (Equation 17.1) in the order of several mm/rev also indicate ripping (Figure 17.5, p.197). A certain size range of blocks may give problems with transport. When rock strength is high, breakage of bits may occur.

2. Massive rocks are being scraped or cut by the bits. Excavation rates are below 10 m^3/hour. Calculated maximum feed values below 2 mm/rev indicate massive cutting, but some rocks that were ripped (projects 4,5,6,13) had low feeds due to transport problems with blocks (Figure 17.5 & 6). A definite relationship of feed with rock strength exists. Above a UCS of 150 MPa excessive bit breakage can be expected. Very strong rocks have a low feed and scraping with resulting mode I wear (hot bits, two-body abrasive wear and adhesive wear) occurs.

3. Massive silicate rocks and carbonate rocks (with no quartz) have clearly different relations of specific wear with rock strength (Figure 17.8, p.199).

To estimate trencher performance for new projects, first an evaluation of the likelihood of ripping can be made. If massive rocks are present an estimate of feed may be made based on rock strength. Deketh (1995) gives a formula to estimate the maximum feed on the basis of trench dimension, based on only three projects (project 1,2,3). Using the data of the massive rocks, two prediction equations, one for silicate rocks and one for carbonate rocks (without quartz) can be derived:

$$feed_{perbit} = \frac{C}{PLS * trench\ width * massive\ rock\ thickness}\ (mm/rev) \qquad (17.2)$$

where the units of Point Load Strength are MPa, the thicknesses are in m. The constant C = 3.8 MNmm/rev for silicate rocks (based on projects 1,2,3,9,11; linear regression, r^2 = 0.89). C = 2.1 MNmm/rev for carbonate rocks without quartz (based on projects 7,14,15,16; linear regression, r^2 = 0.79). The feed calculated using this equation may compared with the data base of excavation rate of massive rocks (Figure 17.5), to estimate the production.

A guess of SPW in massive rock can be obtained using rock material strength and again differentiate between silicate and carbonate rock types (Figure 17.8). As pointed out in the previous section, bit consumption increases exponentially above UCS values of 80 - 100 MPa (Figure 17.7), due to breakage.

The rock properties that influence trencher performance are summarized in Figure 17.13. For a trencher, the boundary between a broken rock mass and a massive rock mass is determined by the trench dimensions. If rock blocks cannot be removed, the rock will be massive for the trencher and cutting will occur, even in rocks that are jointed or fractured. In the rare occasion that ductile cutting occurs in strong rock (perhaps this occurred in project 14, a dolomite), high temperatures may lead to adhesive wear. Adhesive wear may also occur in all rocks at low feed values, which is more likely when the rock becomes stronger (Equation 17.2).

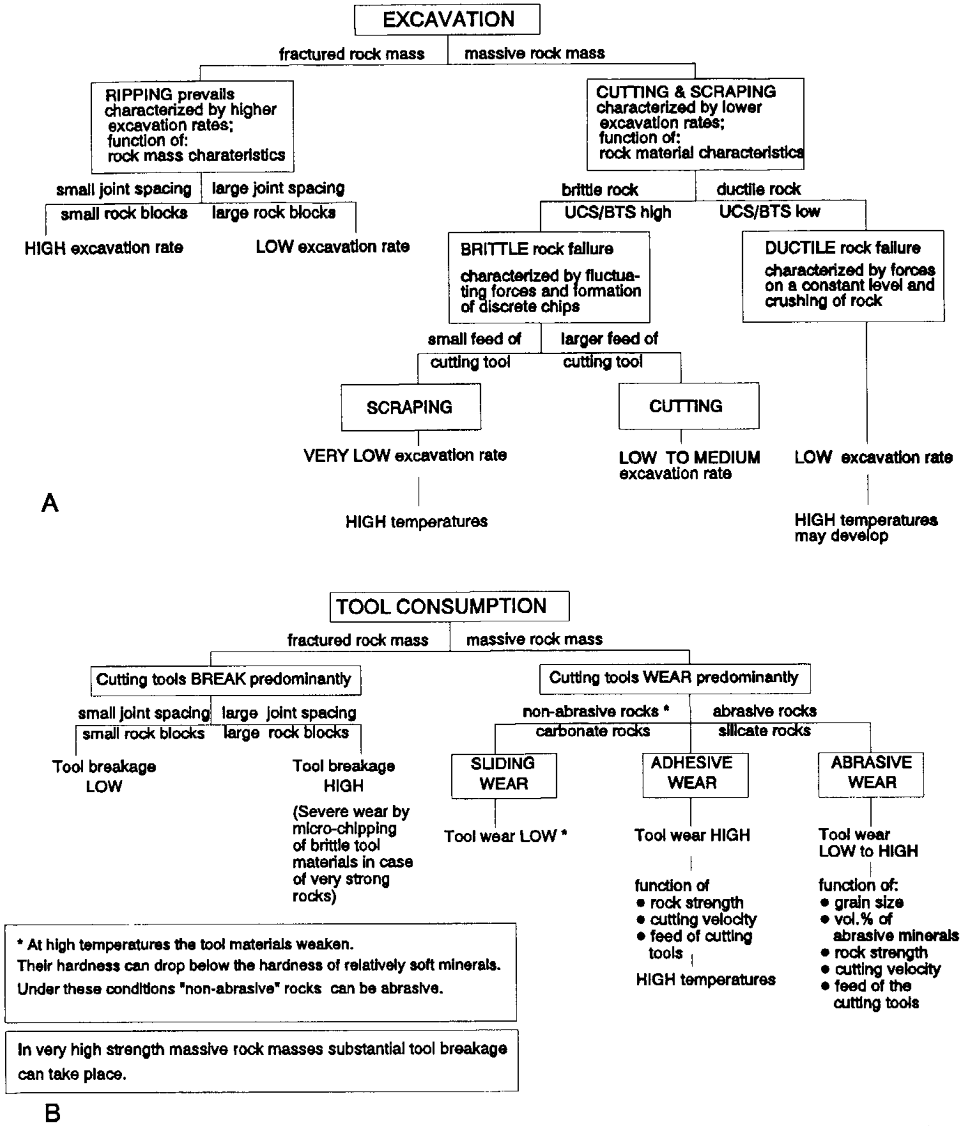


Figure 17.13 Scheme depicting various excavation- (A) and tool consumption (B) mechanisms deduced from observations of lab experiments and the performance of Vermeer T-850 rock cutting trenchers.

In Chapter 18 the possibility of predicting trencher performance using a fuzzy logic model based on the data obtained by the field monitoring projects is described.

CHAPTER 18

Assessment methods for excavation performance

This chapter examines methods that have been developed to predict rock excavation by machines. First a summary of the method developed in Norway to predict full-face tunnel machine performance is given. Then the work on prediction of rippability by bulldozers. Also for rock dredging approaches of prediction, using conventional rock mass classification, have been used. A recent dredging project in Quatar (Ra's Laffan) has been analyzed this way. It appears that a general approach to excavation prediction, valuable for a broad group of excavation machines, is too simple to be useful. The rock trencher projects discussed in Chapter 17 have shown that the performance of the machine relates with rock properties. For the trencher projects, the performance of one type of excavation machine is examined. To develop valid prediction models for this machine requires more data. Instead of the classical rock mass classification approach, the potential of fuzzy model expert systems has been examined to develop better predictions of excavation rates and bit consumption.

18.1 ASSESSMENT METHODS USED FOR TUNNEL BORING MACHINES

The Norwegian Institute of Technology (University of Trondheim) has developed a system to predict drilling rate which is well published and widely known (Movinkel & Johannessen 1986, Bruland et al. 1988, Johannessen 1990). The Drilling Rate Index (DRI) was originally developed by Reidar Lien in the late 1950's to estimate penetration rates of light percussive drifters. The method is still used in rock drilling, Tamrock and Atlas Copco are two manufacturers known to use the system to estimate the *drillability* of rocks for percussive rock drilling. For this purpose the Drilling Rate Index (DRI) and the Bit Wear Index (BWI) are determined on rock material in the laboratory, see Chapter 11, Figure 11.2. The influence of discontinuities or fractures in the rock mass is ignored (Tamrock 1983).

Since 1975 systematic collection of data and rock samples from Tunnel Boring Machine (TBM) projects in Norway and abroad has taken place in order to find a correlation between the DRI value and TBM advance rates. The last publication of the prediction model for Tunnel Boring Machines is the project report *Hard Rock Tunnel Boring* (Bruland et al. 1988). A large data base with test results exists

Table 18.1 Geological parameters and machine factors determining net advance rate and cutter disc consumption of tunnel boring machines (Bruland et al. 1988).

Rock parameters	Machine factors
fracturing	cutter shape and size
drillability, DRI	cutterhead velocity (rpm)
abrasiveness	cutterhead curvature
	number of discs
	installed power and thrust

(Johannessen 1990). The current prognosis model has been developed based on data from 26 sites with 150 km of bored tunnel, mainly in Norway (Bruland et al. 1988). Net advance rate and cutter consumption of a full face tunnel boring machine (TBM) depends on both rock parameters and machine factors (Table 18.1). The rock parameters include fracturing, drillability and abrasiveness.

For TBM's it is clear that fractures and joints in the rock mass influence the advance rate. The size of the TBM (diameter of the cutterhead) determines the maximum spacing of joints which start to influence the advance rates. Fracturing of the rock mass has to be established during the site investigation by examining outcrops and rock cores. The frequency or fracture index (number of fractures or joints per meter) or its inverse (average spacing) is used. The fracturing of the rock is classified in Fracture Classes (Table 18.2). The rock material properties needed to assess the *boreability* of rock for a TBM are:

- drilling rate index DRI
- bit wear index BWI
- cutter life index CLI

Table 18.2 Fracture Classification used by Norwegian Institute of Technology (Bruland et al. 1988).

Fracture Class	Spacing of weakness planes (m)
0	-
0-I	1.6
I-	0.8
I	0.4
II	0.2
III	0.1
IV	0.05

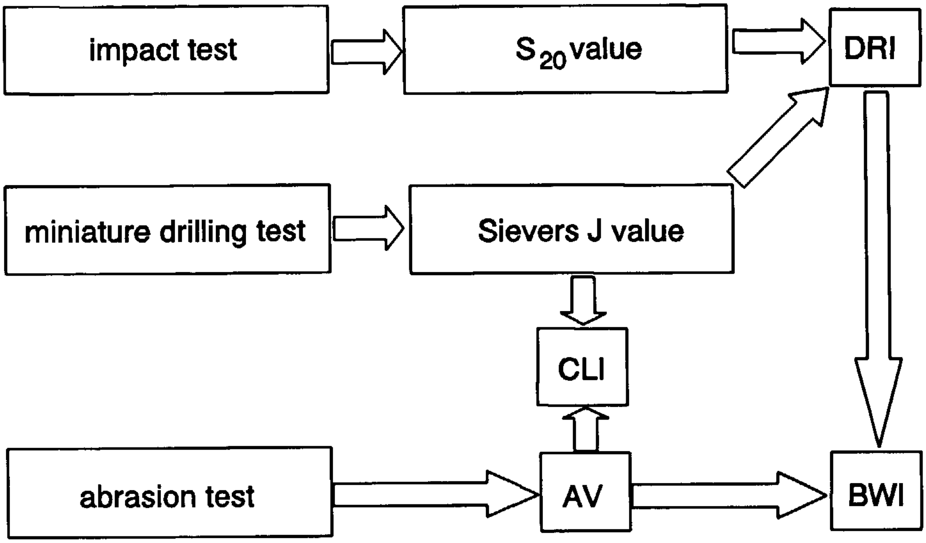


Figure 18.1 Derivation of the Norwegian Institute of Technology indices for rock drillability (after Blindheim et al. 1990).

DRI is calculated on the basis of two laboratory tests, the Swedish brittleness test (S_{20}) and a miniature drill test (Sievers J value). BWI is calculated on the basis of the DRI and the result of an abrasion test (Abrasion Value), see Chapter 11, Figure 11.2. CLI is calculated on the basis of the Sievers' drill test and the Abrasion Value for Steel (AVS). The testing scheme is given in Figure 18.1.

The estimate of the advance rate of a TBM can be obtained by using a series of empirical graphs, published by the Norwegian Institute of Technology, Figure 18.2. Most of the empirical relationships are non-linear.

An example of the application of this prediction method to the Lesotho Highlands tunnelling projects is published by Blindheim et al. (1990, 1991). This work may act as an example of how geological information, a testing programme on rock samples and the application of linear cutting tests aid to make an intelligent guess of the behaviour of the rock formations considered tunnelling excavation. Since similar information is needed for rock dredging projects, the type of data analyzed for this tunnelling project is summarised here.

- Geology. An interpretation of the thicknesses and internal variation in properties of the rock units present, considering weathering, jointing, contacts of rock units and internal rock stresses.

- Rock properties. Variation in strength properties (UCS, expressed in average and standard deviation). Rock Mass Classification (RMR, Q-system), variation in fracture spacing.

- Boreability was assessed using both direct methods (in this case large scale laboratory linear single cutter tests) and indirect methods (in this case UCS, PLS, BTS, Schmidt hammer, Cerchar abrasion, Norwegian Institute of Technology tests and the Total hardness tests of Tarkoy 1973).

Blindheim et al. (1990) point out that the laboratory tests performed have proved useful in the preconstruction phase of the project, enabling independent evaluations

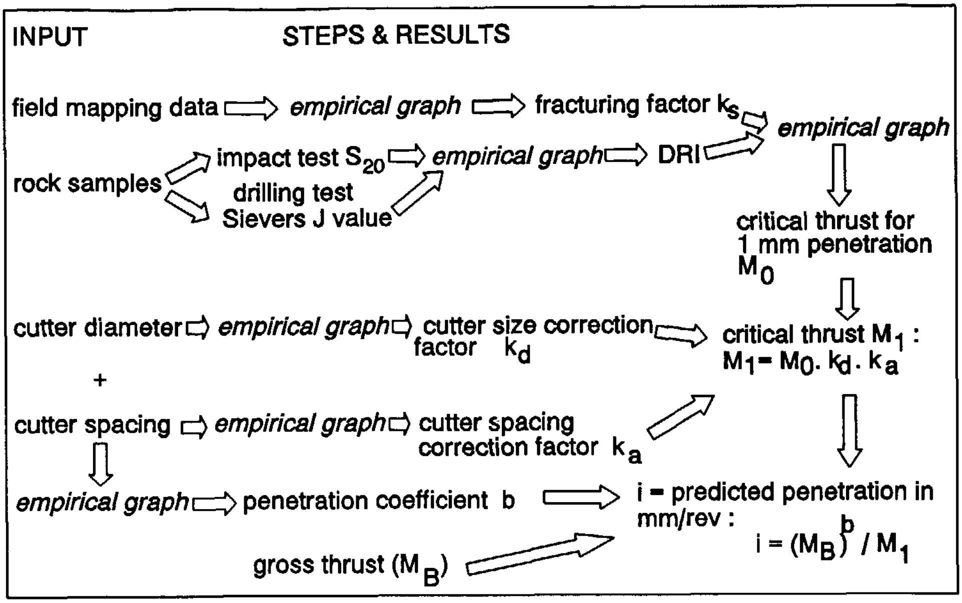


Figure 18.2 The steps taken to predict the penetration rate of TBM's in the Norwegian Institute of Technology model (after Blindheim et al. 1990).

of TBM performance to be made. They emphasize that the interpretation of the test results requires experience. Interpretation has to take into account the characteristics of the TBM and the properties of the rock mass, where adjustment of the rock mass properties can be significant. The combination of geological site investigation, testing of rock samples and execution of large scale trial cutting, either in the laboratory or in the field is advocated.

According to Nelson (1993), who gives a review of TBM performance analysis with respect to rock properties, the Norwegian Institute of Technology system is the most complete model publicly available which includes all aspects of TBM design and performance considerations. Nelson (1996) emphasizes the need of studies of tunnel boring machine performance by comparing the data with the rock mass properties of the sections excavated. This is the way the Norwegian prediction model has been developed.

Concluding, at present assessments on the production or performance of tunnel boring machines can best be made using the method of the Norwegian Institute of Technology, which is based on the analysis of a large data base. Both machine characteristics (cutter diameter, cutter spacing, gross thrust) and rock properties (brittleness, drillability and abrasiveness, to be determined by special tests) are considered.

18.2 ASSESSMENT METHODS USED FOR ROCK RIPPING

One of the earliest methods used to assess excavatability by ripping is the seismic velocity of the rock mass as determined in the field by a seismic refraction survey.

The Caterpillar Company regularly publishes excavatability graphs for its various tractor models. Field seismic wave velocity depends on both the rock material density and is discontinuity density of the rock mass. Discontinuities reduce the seismic wave velocity. The nearer the measured velocity approaches the material velocity, the harder a certain rock is to rip.

Another bulldozer company, Komatsu, publishes graphs of machine productivity against unconfined compressive strength of the rock material.

Recently two studies on rippability assessment have been published (MacGregor et al. 1994, Pettifer & Fookes 1994). Both papers review and comment on earlier literature concerning rippability prediction or assessment and the reader is referred to these papers for detailed information.

Pettifer & Fookes have made a revision of the graph published by Franklin et al. 1971 (Figure 15.1). More than 100 case studies have been used. This graph can be used in the early stages of a project in which rock excavation is considered and data on rock material strength and discontinuity density is available. The graph is reproduced in Figure 18.3. The rock strength is determined by point load testing. The discontinuity spacing index I_f is calculated from the frequencies or average spacings of the major discontinuity sets in the rock mass. Commonly three main sets of discontinuities are present, the spacing index is the mean value of the average spacings of these sets: $I_f = (s1 + s2 + s3)/3$. The diagram also gives a verbal definition of block size, which is defined by the maximum spacing s1 (BS 5930:1981, Code of Practice for Site Investigations, see Pettifer & Fookes 1994). The purpose of the study of Pettifer & Fookes is to aid in the selection of excavation machinery in the early stages of a project.

In Australia, MacGregor et al. (1994) have recently carried out research into the *prediction of bulldozer ripper production* from geological data. The study involved more than 500 sets of ripping data from locations in New South Wales (Australia). All rock types: sedimentary, igneous and metamorphic rocks were involved, but sandstones occupied half of the data set. MacGregor compared several rippability prediction methods. The single factors that correlated best with the production data were seismic velocity and strength (UCS as well as PLS and BTS). But the spread of production values found for each value of seismic velocity or strength is so high, that no sensible predictions can be made from these values alone. A conclusion made on the data from MacGregor et al. was that the production prediction charts of the manufacturers tend to be overly optimistic. The data were also plotted on a Franklin diagram, but no satisfactory differentiation between easy and difficult ripping conditions could be made.

MacGregor et al. proceeded by examining whether the existing classification systems to estimate rippability would give better results. Many attempts have been made to develop useful classification systems besides the Franklin diagram and the seismic velocity measurement. The well-known systems use the so-called rock mass classification systems as a base. The two best known rock mass classification systems, Bieniawski's Rock Mass Rating (Chapter 8.2.3) and Barton's Q-system Barton et al. 1974, (Bell 1992) have been adapted to assess excavatability, again for ripping purposes. Weavers's rippability rating chart (1975) was developed from Bieniawski's Rock Mass Rating (RMR) system by adding a rating for the seismic velocity instead of the RQD. A weathering parameter is included and an adjustment for the effects of orientation of the discontinuities on ripping. Kirsten's excavatability

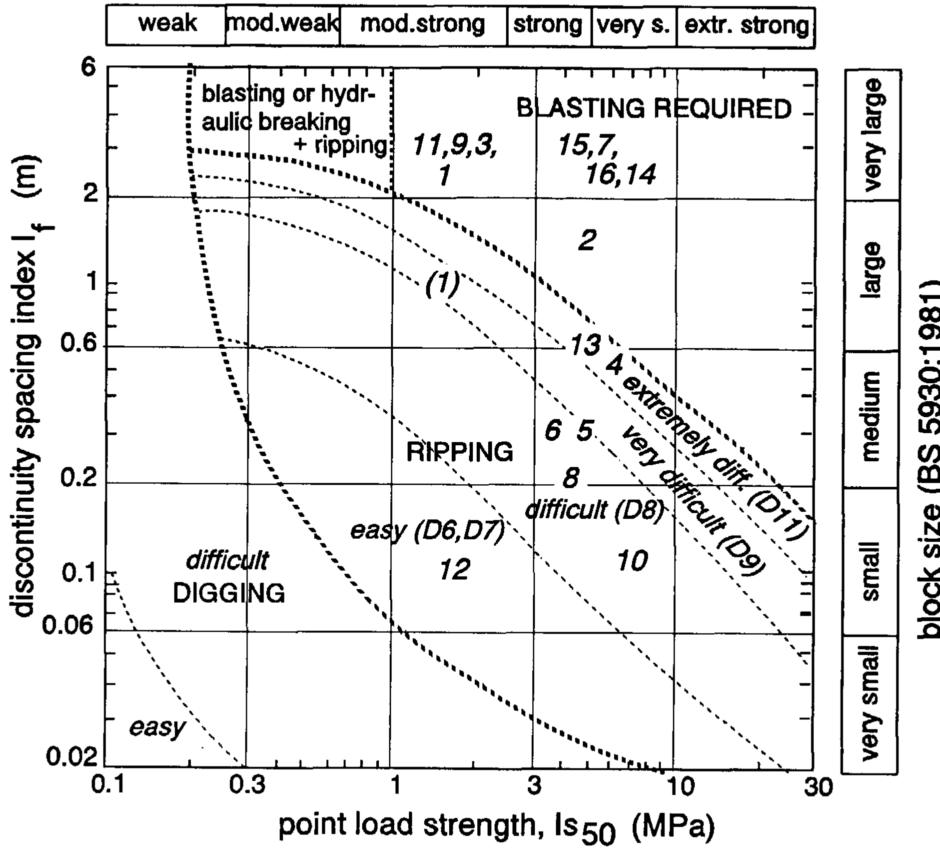


Figure 18.3 Rock excavation diagram developed by Pettifer & Fookes (1994) using 120 case data on ripping projects. Numbers correspond to the trencher projects of Chapter 17.

index (1982) was developed using Barton's Q system. Besides these two systems, MacGregor et al. examined those of Minty & Kearns (1983), Scoble & Muftougly (1984), Hadjigeorgiou & Scoble (1990), Smith (1986) and Singh et al. (1987).

The main conclusion from the extensive study of MacGregor et al. 1994 was that none of the present methods are capable of accurate ripper production prediction. In terms of a statistical (linear) regression analysis on the data from ripping by a Caterpillar D10 dozer, the coefficients of determination r^2 were all below 0.26.

MacGregor's study clearly shows that individual rock parameters cannot predict productivity with any accuracy. She therefore examined the data using a multiple regression analysis. Best results were obtained when the square root of the effective production was divided by the mass of the bulldozer used and this value compared with a set of rock factors. The most general equation has the following form:

$$\frac{\sqrt{PRODU}}{MASS} = C - A*UCS + B*WEATH - D*SIZE - E*SEIS + F*ROUGH + G*SETS + H*STRUC \quad (18.1)$$

where PRODU = the effective production (m^3/hour), MASS = bulldozer mass (in tonnes including ripper), A-H are constants obtained by the regression, WEATH = a weathering rating, SIZE = a grain size rating, SEIS = seismic velocity (m/s), ROUGH = a roughness rating, SETS = number of discontinuity sets, STRUC =

a structure rating related to block size (see MacGregor et al. 1994). Apart from the general equation, other regression equations are given which can be used for specific rock types, such as igneous rocks, or when data is lacking, such as for example seismic velocity.

The conclusion that can be drawn from MacGregor's study is that production assessment from rock data cannot be reliably made using single rock factors, such as seismic velocity or rock strength. Also rock classification systems or rating systems specially made for rippability assessment prove unreliable. Allowing for machine characteristics (in this case bulldozer mass), reasonable assessments can be made using the regression equations of MacGregor et al. (1994), which are derived from a large data base.

18.3 ASSESSMENT METHODS USED FOR ROCK DREDGING

Dredging contractors are known to have developed in-house methods to estimate production and tool consumption from rock data. Probably the most common factors used are UCS and a measure of discontinuity density, like RQD and FI[71]. Both UCS and RQD and/or fracture index are used to derive a factor related to the specific energy required to excavate the rock mass. This is used to estimate production and tool consumption for the various cutter suction dredgers available. It is not known how reliable and evolved these systems are, but similar problems as with the tunnel boring machines and bulldozer rippers exist. The aim of this work is to come up with a list of rock parameters that ought to participate in a database of rock dredging excavation, in order that the contractors may improve their in-house assessment methods.

Also for dredging a rating system has been proposed in literature. Weaver's (1975) method has been modified by Smith (1987) to an underwater rippability rating chart, by leaving out seismic velocity (Table 18.3). The information on rock strength has been expanded to include weathering grade. The information on joints also includes orientation. Joint spacing is considered with respect to ripping depth or the size of the cutting tool. Smith considered his Rippability Rating (RR) more appropriate to dredging. According to Smith (1987) seismic refraction data are seldom available[72], but also the observations on joint continuity and joint gouge, that can easily be made in on-land excavations or tunnels, are often not available for dredging. Smith omitted this information, stating that his index would give information on relative ripping difficulty, but not on productivity. Joint orientation is most adverse for vertical or horizontal bedding and favourable near 45 degrees. The rating number for orientation can therefore be expressed by $|\alpha\text{-}45|/3$, where α is the angle of the joint with the horizontal into the direction of ripping. The most advantageous direction of ripping is normal to the strike, in which case α is equal to the dip of joint or layer. The usefulness of the RR method has to be proven in

[71] FI = fracture index = number of fractures per meter (identical to joint frequency).

[72] Nowadays overwater seismic surveys (reflection measurements) are routinely carried out, velocities may be derived with some extra effort (refraction measurements).

Table 18.3 Underwater Rippability Rating Chart (Smith 1987).

RELATIVE DESCRIPTIVE CLASSIFICATION	VERY HARD RIPPING OR BLASTING	HARD RIPPING	AVERAGE RIPPING	EASY RIPPING	VERY EASY RIPPING
Rock strength[1]	Very strong > 70 MPa	Strong 25-70 MPa	Medium strong 10-25 MPa	Weak 3-10 MPa	Very weak < 3 MPa
Rating	≥ 10	5	2	1	0
Rock weathering[2]	Unweathered	Slightly weathered	Weathered	Highly weathered	Completely weathered
Rating	10	7	5	3	1
Orientation	Very unfavourable	Unfavourable	Slightly unfavourable	Favourably	Very favourable
Rating: $\lvert\alpha-45\rvert/3$	15	13	10	5	≤ 3
Joint spacing (expressed in ripping depth, D)	> 3D	D to 3 D	D/3 to D	D/20 to D/3	< D/20
Rating	30	25	20	10	5

[1] In the original table the term *hardness* is used for rock strength. Such usage should be avoided.
[2] For a strength and weathering classification see Appendix A, Table A5 & A6.

practice, but its success in prediction of production can be doubted considering the results of MacGregor et al. (1994), see the previous Chapter 18.2.

De Wit (1996) has tried several of the above methods on data from a dredging project carried out by Jan de Nul and Boskalis in Ra's Laffan (Qatar)[73]. Three main geological units are present in the area: 1. dolomitic breccia limestones (collapse breccia's) overlain by 2. calcarenites, both units are capped by 3. a duricrust (caprock) of 0.1 - 0.5 m thick. The dolomitic limestones consist of limestone breccia clasts occurring in a matrix of dolomitised limestone. The rock is moderately strong when fresh, but when weathered the matrix is weaker than the hard limestone breccia clasts. The moderately strong calcarenite generally consists of cemented shell fragments within a relatively fine grained matrix. The caprock everywhere covers underlying rock and is moderately strong to strong. The UCS of the major part of the Ra's Laffan rocks varies from 10 - 20 MPa. Only the caprock part and the bottom part of the dolomitic limestone have higher UCS values (up to 70 MPa).

[73] The data is confidential, therefore in the figures production and tool consumption numbers have been omitted.

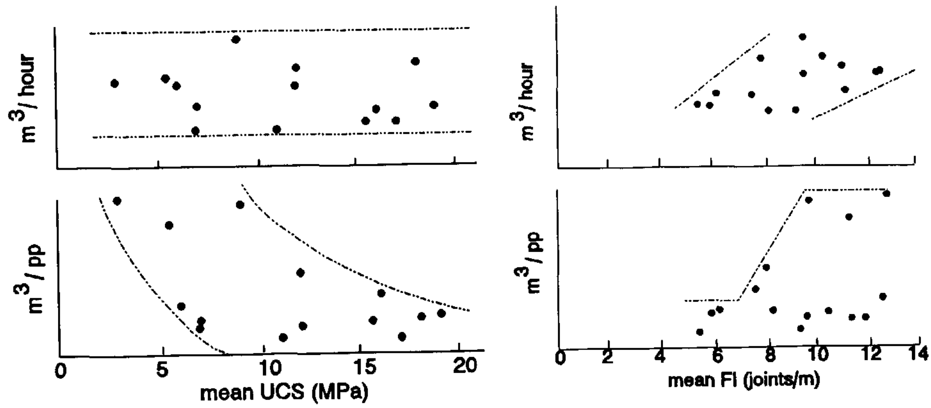


Figure 18.4 Effective production (m^3/hour) and tool consumption (m^3 per pick point) data versus mean UCS and FI of the limestone rock mass (data from De Wit 1996).

The site investigation involved a seismic reflection survey in a grit of longitudinal lines every 100 m and cross lines every 300 m. 60 boreholes were sunk to 1 meter below the intended dredging depth, which meant a maximum penetration depth of 6 m into the seabed in water depths of 5 - 15 metres. The core diameter was 100 mm. The following tests were carried out on the core materials: UCS, BTS, PLS, specific gravity and water absorption (BS 812), dry and wet density, chemical analysis, sodium sulphate soundness and Los Angeles Abrasion.

Seismic reflection and borehole data were used to develop a three dimensional model of the site, using geostatistical modelling (BLUEPACK computer programme). A total of 11 geotechnical units were distinguished, which coincided with layers present in the geological units. The 11 units were defined on the basis of strength (UCS) and fracture index (FI).

De Wit had to make several steps to be able to use the data for a production and tool consumption evaluation. The site was dredged by three different cutter suction dredgers. Production and tool replacement data were available for certain sectors in the dredged area. From the 58 sectors that were related to a borehole log and a production and tool replacement number, only 25 were dredged by a single dredger. Because of the influence of the excavating dredger on the production and tool consumption, only the performance of the CSD *Leonardo da Vinci* was studied, because most data (14 data sets) were available for this cutter suction dredger. Figure 18.4 shows the data of production (m^3 per effective working hour) and tool consumption (m^3 per pick point) compared with the average UCS and FI of the 14 sectors. The average UCS and FI were calculated by averaging the values of UCS and FI of the borehole record, proportional to the thickness of the layers from which the values were derived. Figure 18.4 shows no relation between production and UCS and a trend of increasing production with increasing fracture frequency. The effect of these parameters on tool consumption is more clear. A decrease of production per pick point with increasing UCS is indicated and increase with fracture frequency. Although the tool replacement remains high in most sectors with an average FI > 9, in three sections the tool replacement has reduced significantly. This may indicate a transition to (easy) ripping above an FI of about 9, which corresponds to an average fracture spacing of 0.11 m.

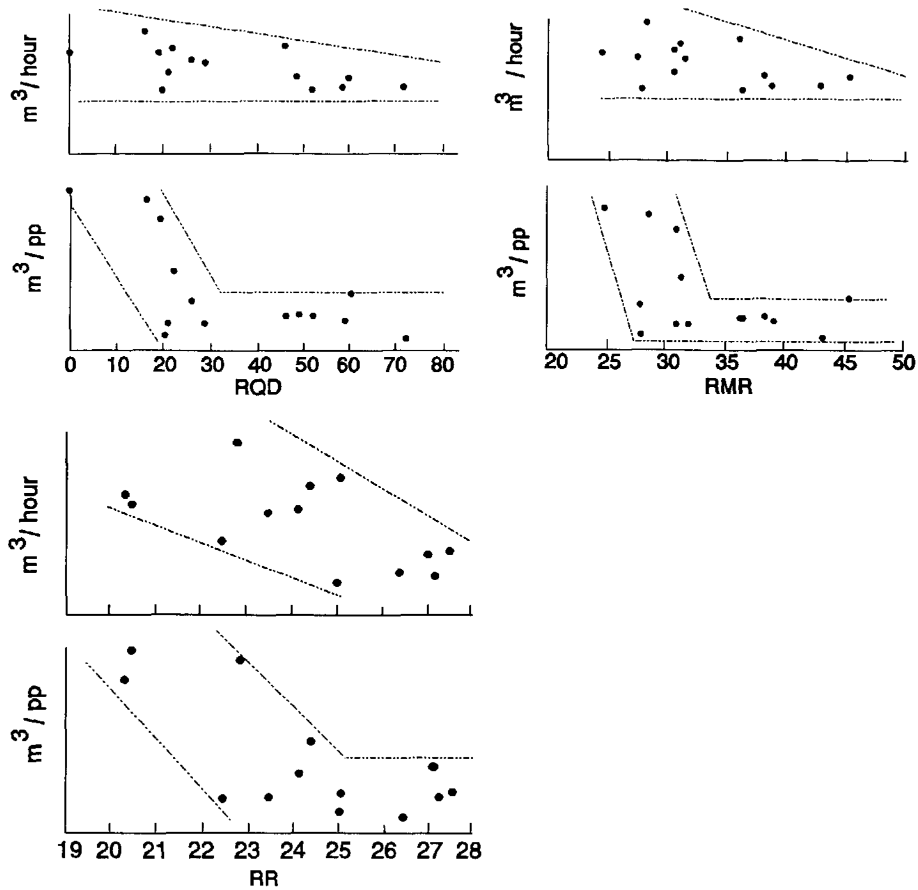


Figure 18.5 Production and tool consumption of Ra's Laffan limestone related to the rock mass descriptors RQD, RMR and RR (data from De Wit 1996).

Figure 18.5 shows plots of the RQD (Rock Quality Designation), the RMR (Rock Mass Rating) and the RR (underwater Rippability Rating). Both rock mass parameters point to a change in tool consumption at a threshold value, at an RQD of about 30 and an RMR of 35. Possibly this corresponds to a transition from mostly ripping to mostly cutting as well. Smith's underwater rippability rating, RR, shows similar trends. The decrease in production with increasing rock mass quality is somewhat better indicated by Smith's system.

De Wit has tried combinations of rock parameters, like layer thickness and reduced mass strength factors (products of RQD or FI with UCS) into the rock mass classification. No real improvement was obtained, but the general trends of decrease of production rate and increase of tool consumption with increasing rating number were confirmed. Comparison of all classification systems showed that:

- Absolute layer thickness is a very important factor.
- Material strength is only important as far as the degree of fracturing or weathering of the rock mass is limited.

The study of De Wit has revealed a problem common to many dredging projects, namely the limited usefulness of the available data (De Wit 1996):

"Examining the data of Quatar could give no decisive answer to many interesting questions: Does the compressive strength (UCS) describe the mechanical resistance of the intact rock material better than the tensile strength (BTS, PLS)? Is the FI more appropriate to measure the degree of fracturing of the rock mass than the RQD? What is the influence on dredgeability of mineralogy, Young's modulus, joint orientation and joint condition? All these questions remain unanswered because of various reasons:

- *Important data is lacking*. From only seven rock samples the mineralogy and Young's modulus have been determined. During core logging, the FI was not registered as regularly as the RQD. Detailed information on joint orientation and condition is always hard to get during marine site investigations. Seismic surveys only give a general view on the main geological structures.

- *The usefulness of the available data is limited.* Since for a whole section only one wear and production figure is available, it is very difficult to assess the specific contribution of each individual layer, let alone to evaluate the importance of its different properties. One could relate the (properties of one) type of rock much better to the obtained production, if during each swing made by the cutter suction dredger (CSD), the following data would be collected:

    -cutting depth (d) and face (D) of the cut;
    -momentary position of the CSD and the step taken;
    -production made during that swing

- *There is little variation in the geology of the dredged area.* In Ra's Laffan mainly dolomitic limestone and calcarenites occur. The variety in properties (e.g. ductility, strength, mineralogy) of the encountered layers is therefore limited."

Two main points follow from the above:

1. Rock mass classification systems can only give a general indication of excavatability of rock.

2. A larger variety of relevant rock data should be assembled both in the site investigation and during dredging.

18.4 DISCUSSION OF PRESENT ASSESSMENT METHODS FOR ROCK EXCAVATION

From the previous Sections in this chapter may be deduced that modelling of the mechanical excavation process may only be done with a reasonable chance of success when a large data base exists of rock excavation projects carried out with a certain machine. The Norwegian model for Tunnel Boring Machines in hard rock (Chapter 18.1) may act as example. This prediction model is mainly based on regression analysis of the data from which relationships between performance and rock properties could be established (Figure 18.1 & 2). Much less successful appear the rock mass classification systems in this respect. The study of bulldozer ripper data by MacGregor et al. (1994) illustrates this. Rock Mass Ratings give only a general idea of production and tool consumption and are too crude to act as real predictors. This was also found when the data assembled on the performance of the T-850 trencher were examined (Giezen 1993). Probably the description of the rock mass by the current classification systems is not accurate enough. They do not give a good idea of the spreading of properties within the rock mass, even when the range of rating values is taken into consideration. The character of geological information is often vague. Therefore the possibility of applying fuzzy logic to process the geological data was examined, as described in the next chapter.

18.5 ASSESSMENT OF TRENCHER EXCAVATION PERFORMANCE USING FUZZY EXPERT SYSTEM

18.5.1 *First indication of trencher excavatability*

Two rock factors of importance are the discontinuity density and the material properties of the rock. Trenching has shown to be sensitive to this. Excavation occurs by ripping or cutting. In the rippability diagram of Figure 18.3 the 16 trencher projects have been plotted, using point load strength and block size. The position of the projects indicates that the Pettifer diagram is useful. Trencher projects in which ripping occurred fall in the field of rippable rock. Project 13, in which both ripping and cutting occurred falls in the field of extremely hard ripping. Project 4, in which ripping occurred and very high bit breakage, plots also in this area. The projects within the blasting field, all involve cutting action during trench excavation. For trencher projects, however, the dimensions of the trench should be compared with the block size. If the trench dimensions are smaller than the block size, the trencher excavates in effectively massive rock (see Table 17.1). The projects 1, 2, 11, 16 that have been plotted in the top of the diagram, above the 2 m discontinuity spacing line belong to this category. The projects 3, 7, 9, 14, 15 above the 2 m line have joint spacings wider than 2 m. Project 1, which has a bock size of 0.5 m^3 and a discontinuity spacing index of 1 m, is an example of effectively massive rock. Projects 11 & 12, in Sybillenbad were both in the same rock, a schist. In the case of project 12 this rock was weathered and ripping took place along the discontinuities in the rock. In unweathered schist (project 11), cutting took place.

18.5.2 *Fuzzy expert system*[74]

Den Hartog (1996) made a study of the trencher data of the first 11 projects. Fuzzy logic has been used to model the trencher performance for three reasons:

- The limited number of data does not allow the use of conventional statistical methods to predict trencher performance.
- Most of the factors involved in the excavation process do not have well defined boundaries.
- Fuzzy logic allows to use subjective (non-numerical) information in models of prediction performance.

The application of fuzzy logic to this problem is described by Den Hartog et al. 1997. Den Hartog (1996) developed a preliminary scheme for the data on production

[74] For the development of the fuzzy system, the support of Dr. R.Babuska, TUD, Faculty of Electrical Engineering is acknowledged.

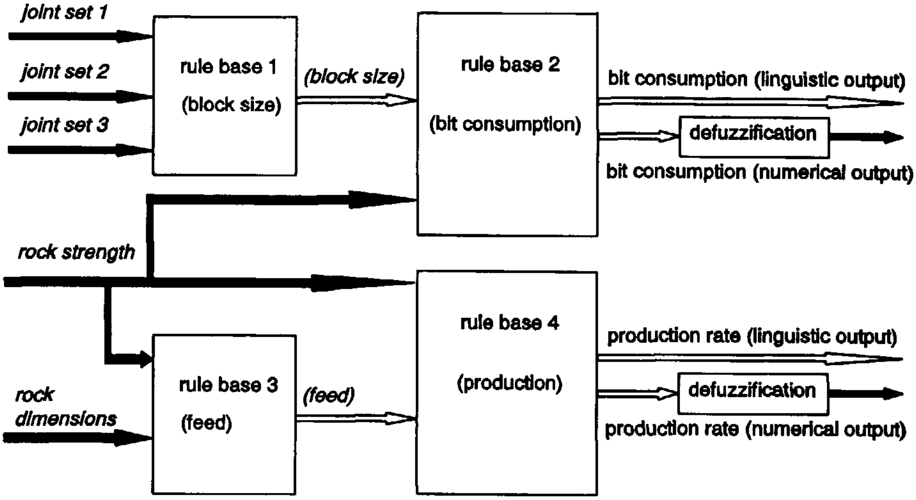


Figure 18.6 Organisation of rule bases for the model of bit consumption and production of the T-850 trencher. Full arrows are numerical values, open arrows linguistic terms (Den Hartog et al. 1996).

and tool consumption of the rock cutting trenchers, to define the rock properties that determine trencher performance. The model developed uses six input variables: rock strength, fracture spacing of three joint sets, joint orientation and dimensions of the hard rock body in the trench. Other factors such as mineralogy of the rock, grain size of abrasive minerals, rock ductility, angle of the boom etc. were not treated in the model. The main reason for this is the limited amount of data sets.

Based on the knowledge obtained in the trencher project, as laid down in Figure 17.13, *if-then rules* have been formulated. With the help of these rules four hierarchically organised *rule bases* constitute the organisation of the knowledge, see Figure 18.6. Each rule consists of two parts, the *if* part with single or multiple premise and the *then* part with a single conclusion:

If x_1 is $A_{i,1}$ **and** x_2 is $A_{i,2}$ **and** .... $x_{i,p}$ is $A_{i,p}$ **then** y is B_i

where x_1, x_2, ..., x_p are the input variables (premise), p the number of input variables and y is the output variable (conclusion). $A_{i,j}$ represents the *reference linguistic terms*, where j is the number of terms. B_i is the set of reference terms of the output variable y. The four rule bases for the trencher performance, Figure 18.6, are defined by in total about 100 rules. An example of how rules are formulated:

The rate of bit consumption is mainly due to breakage of bits (Chapter 17.1). Breakage occurs when the impact forces exceed the strength of the bits. The impact forces are determined by the material strength of the rock and also by the size of the rock blocks in the mass. If blocks are small the bits do not break, regardless of rock strength. The larger the blocks are, the more the strength of the rock will affect the bit consumption rate. When the rock is massive, rock cutting will occur and breakage will depend entirely on the strength of the rock. These observations can be expressed in rules that are used in the knowledge base (Den Hartog et al. 1997):

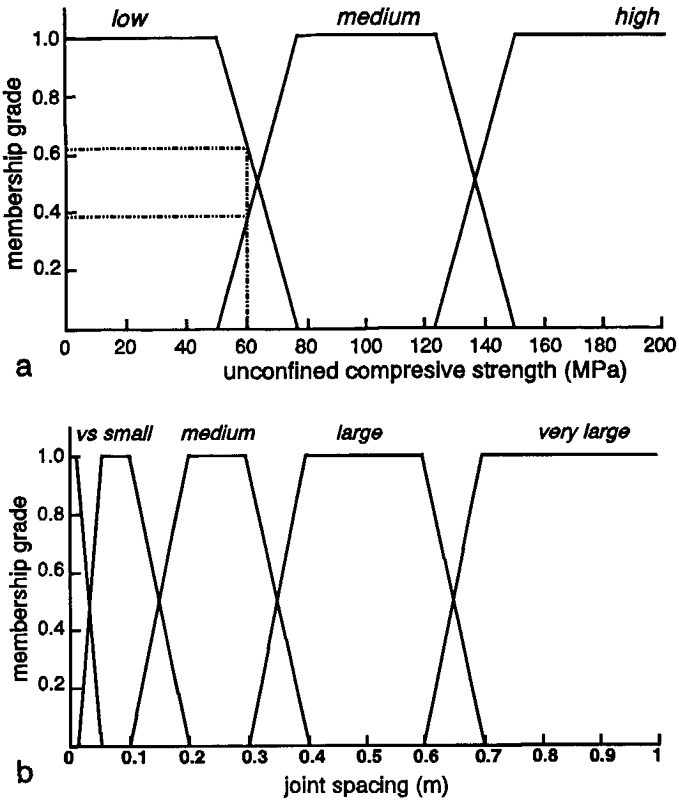


Figure 18.7 Membership functions for the variables strength and joint spacing (Den Hartog 1996).

- **If** *Block Size* **is** *Small* **then** *Bit Consumption* **is** *Very Low*
- **If** *Block Size* **is** *Medium* **and** *Strength* **is** *Medium* **then** *Bit Consumption* **is** *Medium*
- **If** *Block Size* **is** *Large* **and** *Strength* **is** *High* **then** *Bit Consumption* **is** *High*

The reference linguistic terms are of two types, numerical linguistic variables (e.g. the input variable strength, the output variable bit consumption) and qualitative linguistic variables (block size).

Strength of the rock material has been divided into three fuzzy sets: low, medium and high. Trapezoidal membership functions have been chosen (Figure 18.7a).

The block size is not directly measured, but is determined from the spacing of three joint sets. Den Hartog (1996) divided the spacing of the joint sets into five classes, ranging from very small to very large. The boundaries of these classes were based on the observations of the 11 trencher projects considered (Figure 18.7b). Eleven linguistic terms are used to describe the block size and the block shape (5 size classes for blocky rocks, 3 classes for tabular rocks and 3 classes for columnar rocks). The shape is included, because it is relevant for the production rate. The number of rules to describe block size and shape (rule base 1) is 25.

The bit consumption has been divided into five output sets or classes (Figure 18.8a). After a first trial of the fuzzy model, these classes were adapted, to improve the result of the calculation (Figure 18.8b).

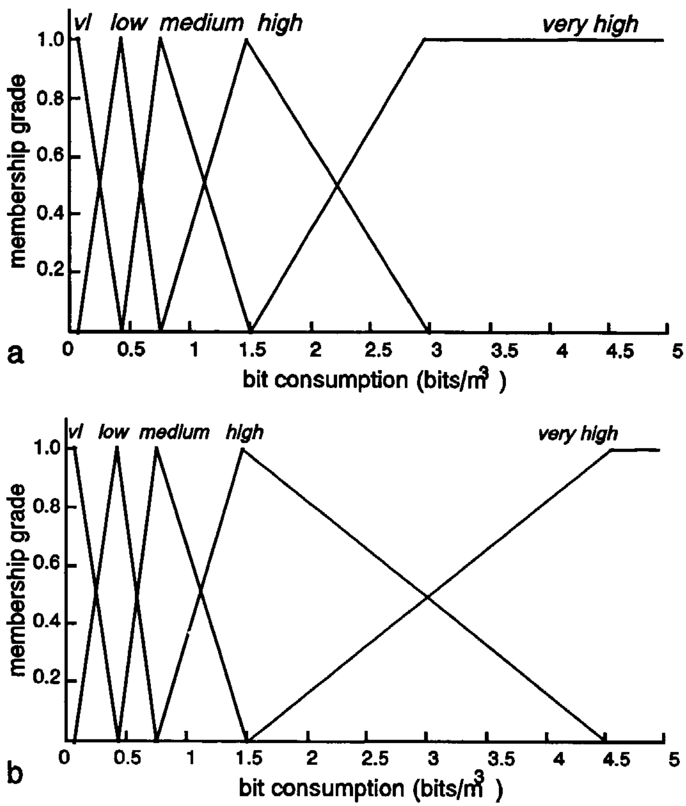


Figure 18.8 The output classes for bitconsumption (a) were redefined after a first trial of the model (Den Hartog, 1996).

The computation of a fuzzy logic model generally proceeds in five steps: fuzzification, computation of degree of fulfilment, inference, aggregation and defuzzification (Babuska 1995, Den Hartog 1996):

1. In the fuzzification step the numerical input values of the different parameters are translated into linguistic terms, with their corresponding membership grades. In Figure 18.7a the example of a rock with a UCS of 60 MPa is given. The linguistic class is medium strength, its membership grade is 0.38.

2. The computation of the degree of fulfilment of the input variables (premise) of each rule proceeds by combining the membership grades of the different variables by the logical operators (**and**, **or** and **not**).

3. In the inference step the output fuzzy set (conclusion) of each rule is modified using the degree of fulfilment of the premise of that rule. In this case the max-min inference method is used. The minimum operator clips off the conclusion fuzzy set at a height corresponding to the degree of fulfilment of the premise of the rule. (Figure 18.9 gives an imaginary example where the effect of two rules, based on strength and block size on production are illustrated; the minimum operator ensures that the premise with the lowest membership grade $\mu(x)$ clipps off the corresponding conclusion: the medium production for the first rule, the low production class for the second rule).

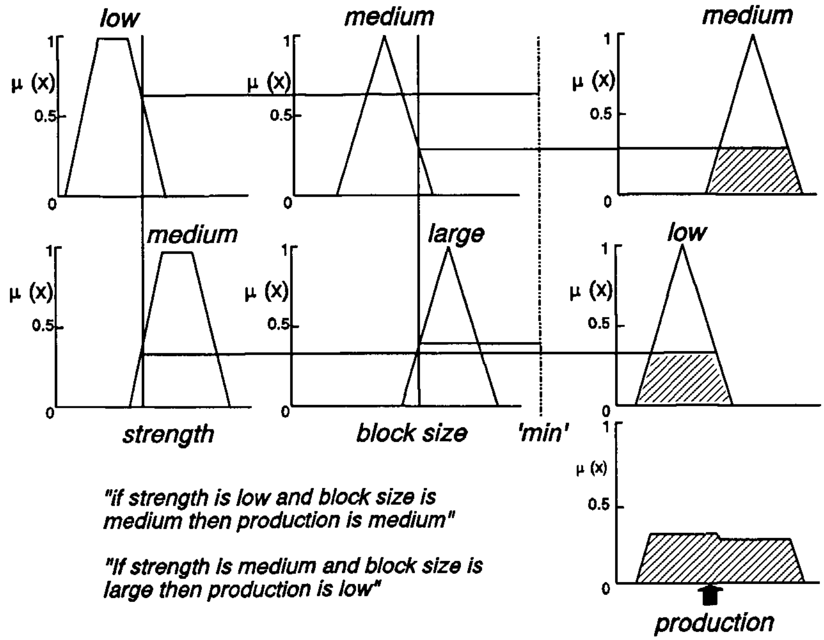


Figure 18.9 Example of the computation of a fuzzy logic model (adapted from Babuska 1995), see text for explanation.

4. The aggregation step combines the different conclusion fuzzy sets into a single fuzzy set (Figure 18.9; in the example the resulting production diagram at the bottom right).

5. The defuzzification step converts the resulting fuzzy set into a crisp value (a single numerical value). There are several defuzzification methods. The best known is the centre-of-gravity method. (Figure 18.9; indicated by the arrow in the bottom right diagram)

The results of the application of fuzzy set theory to the trencher project data are promising. Figure 18.10 shows a comparison of the measured bit consumption data and the figures predicted by the model. The model was based on the first 11 projects, the results of the 16 projects discussed in this chapter are plotted in the figure. Two projects, number 4 (Avoriaz) and number 14 (La Couronne) are definite outliers. These projects both had extreme bit consumption due to breakage (Table 17.2). In Avoriaz this was the result of very strong breccia blocks embedded in a weaker matrix. The very strong massive dolomite rock of La Couronne was a rock outside the capacity of the trencher: it should not have been trenched by this type of trencher (compare Figure 17.7). Figure 18.11 gives the outcome of the model for the production rate. The fuzzy model and its preliminary verification is described in detail by Den Hartog et al.(1997). Further work on the modelling of the trencher data has been done by Alvarez Grima, in which also bit wear is included (Alvarez Grima & Verhoef 1997). When more data becomes available the model can be improved. It is but one example of the application of artificial intelligence methods. Siezen (1996) applied an expert system to develop a dredgeability clasification system, using data of the Øresund Link project provided by a contractor.

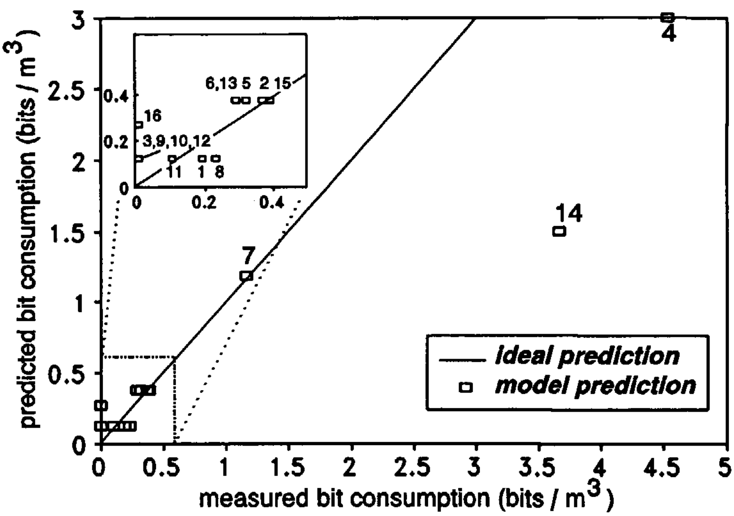


Figure 18.10 Results of the fuzzy model calculations for the bit consumption of the first 16 trencher projects (after Den Hartog et al. 1997).

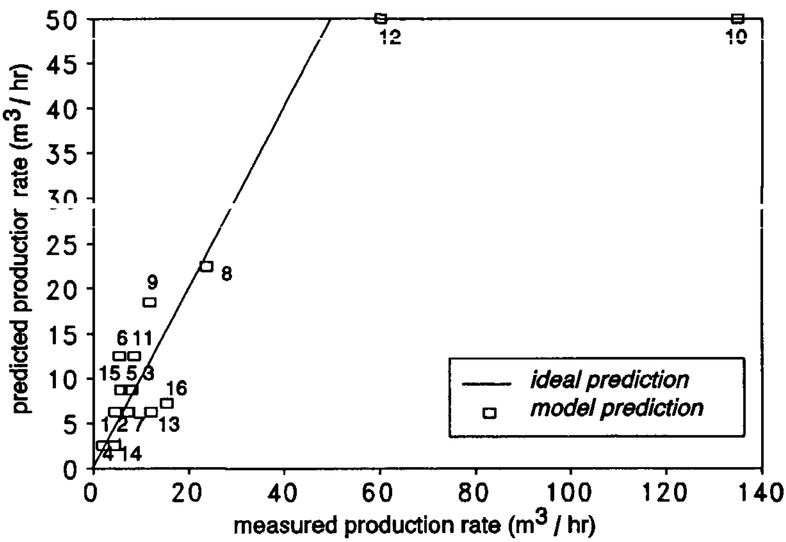


Figure 18.11 Comparison of the predicted production rates and the measured ones of the first 16 trencher projects (after Den Hartog et al. 1997).

CHAPTER 19

Conclusions of Part C: Application to practice

In Part C the proposition that mechanical rock excavation performance should be related to basic rock parameters is examined. Especially for rock dredging projects this is the way to proceed, because of several reasons that have been addressed in the previous chapters:

- the size of the rock cutting tools (pick points) is too large to perform full scale laboratory tests
- sample problems for testing
- other cutting or abrasion tests in the laboratory do not have to relate to the cutting process that will take place in practice

The analysis of the dredging project in Hawkesbury sandstone for Sydney Harbour (Chapter 16) confirms that the laboratory core cutting and abrasion test are not related to the tool consumption and that the rock factors based on BTS, hardness of minerals and grain size, such as the *F*-value or *SPW* are better in this respect. Both laboratory cutting and abrasion tests and the wear factors show relation with the tool consumption of roadheader tunnelling machines in Hawkesbury sandstone.

The trencher excavation projects discussed in Chapter 17 gave the opportunity to assemble data on a number of excavation projects in different rock types. The importance of the discontinuities in the rock mass on the excavation mechanism, either ripping or cutting, was clearly shown in the rates of excavation experienced. In effectively massive rock, the trencher would either cut or scrape the rock material and it was possible to relate the bit consumption (due to breakage and due to wear), the bit wear *sensu stricto* and the excavation rate to rock properties. In this respect it proved important to distinguish silicate rocks and carbonate rocks. Figure 17.13 summarizes the mechanisms involved. Concluding, the following rock properties are of importance to assess trenching projects:

1. The discontinuity frequency in the rock mass: fractured or massive with respect to the trench dimensions. Determines whether ripping or cutting occurs, or problems with block transport.
2. The strength of the rock. The UCS is of importance for possible bit breakage (important above 80 MPa), and tool wear in silicate rock is highly influenced by UCS.
3. The ductility of the rock has been examined, but its influence on wear or excavation has not been clearly demonstrated.

4. The abrasiveness indicators F-value, F_{mod}, and SPW_{III} are very comparable and relate to bit wear.

Considering the prediction of excavation performance, a method using an expert system based on fuzzy modelling is considered useful for this problem. A model for the T-850 Vermeer trencher has been developed and predicts excavation rate, bit consumption and bit wear, using information on the discontinuity system of the rock mass, the rock material strength and the mineralogy and grain size.

One of the goals of this research project was how to improve the prediction of tool consumption for rock cutting dredgers. For the development of prediction models a similar approach as used with the trenchers, using expert systems, is thought useful. For each cutter suction dredger a data base and a performance model should be developed. This implies that it is important that rock properties such as strength (UCS and BTS), petrographic composition (mineralogy and grain size) of the rocks dredged by a CSD should be constantly monitored and compared with the production and tool replacement data. If a good geotechnical model of the underground exists before the dredging starts, this model can be used during the dredging operation.

CHAPTER 20

Introduction to Part D: Site investigation for rock dredging contracts

One of the aims of this work was to outline how the site investigation for a rock dredging contract should be performed, bearing in mind the factors that are involved in the wear of the rock cutting tools. In Part D therefore the general outline of such a site investigation is given, by focusing on the goal of the site investigation, the development of a *geotechnical model* of the subsurface.

It is emphasized that the consulting engineer should aim at the development of a three-dimensional model of the subsurface, based on a good understanding of the geology. Nowadays 3D geoscience modelling is supported by computer assisted information systems and imaging methods. To have all information available in digitized form can be of great help during the execution of a project. The general procedure to come to such a model and the type of data that is needed for a rock dredging project is discussed in Chapter 21. The subsurface is divided into *engineering geological units*, which are soil- and rock mass volumes that have distinguishable ranges of geotechnical properties that relate to the dredging excavation process. These units are distinguished based on both lithological and geotechnical information.

In Chapter 22 emphasis is made on the characterisation of the rock mass for engineering purposes. Rock mass is described in terms of rock material properties and the presence and properties of the rock discontinuities in the mass. Some relevant features of the existing methods are discussed, mainly to point to the type of information to be gathered during the site investigation. Therefore also some short remarks are made on weathered rock mass and the presence of cementation. Both soils and weathered rock can become cemented by precipitation of cementing material from saturated water. Since in many potential dredging locations in coastal areas conditions for weathering and cementation are present or have occurred in the recent past, during the site investigation attention should be given to these features.

In Chapter 23 the conclusion of the work is given. Based on the review of rock abrasiveness and testing methods and mainly on the laboratory tests and observations of the rock cutting trencher operations models of the excavation mechanisms of rock mass and the factors of that determine tool consumption are given. These models are the point of departure for further work on this subject (Chapter 24).

CHAPTER 21

The construction of a geotechnical model for a rock dredging contract

The objective of the site investigation for a rock dredging contract is the production of a *geotechnical model.* This geotechnical model of the subsurface consists of *engineering geological units*[75], which are volumes of soil or rock that have well defined ranges of geotechnical properties related to dredging. Within each engineering geological unit the properties relevant to dredging are, within established limits, approximately equal. The model should be such that the contractor can design an excavation plan and choose the appropriate dredging method. The geotechnical model should give information related to excavatability (for production estimation) and wear (tool consumption). Other aspects related to the dredging operation, such as estimates of the bulking factor, hydraulic transport of the slurry, slope stability of a trench should be retrievable from the model or its data base.

Figure 21.1 illustrates the concept of the geotechnical model. The engineering geological units are described following the definitions given in Appendix B, Table B1. From the verbal descriptions a clear picture of the geotechnical properties of the engineering geological unit can be obtained. The following information is used to characterise an engineering geological unit:

1. Description of the rock material:

colour, fabric, grain size, important minerals, ROCK NAME, ductility, strength, abrasiveness, weathering state. For example:

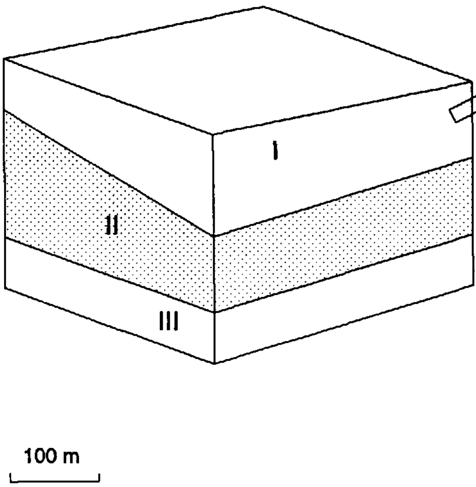
yellow - thickly laminated - medium grained - quartz bearing - DOLOMITE - ductile -moderately strong - low abrasiveness - fresh

2. Description of the rock mass:

layer thickness, discontinuity spacing in three dimensions, block shape, number of discontinuity sets

medium bedded - medium block size - tabular block shape - some dissolution along the two joint sets, with clay infill; three discontinuity sets: bedding and two subvertical joint sets

[75] In this text the term engineering geological unit is used, to stress the geological (lithological) basis of the concept. The term geotechnical unit is commonly used and is synonymous (see for example Hack 1996).



Engineering geological unit I

Description:

grey thinly laminated coarse grained clay-bearing SANDSTONE, ductile, weak, extremely abrasive, partly decomposed

thin bedded, medium block size, tabular block shape, highly weathered

Test data on rocks from unit I:

Test	*average*	*range*	*n*
UCS (MPa)	*10*	*2 - 20*	*10*
UCS //	*15*	*14-16*	*3*
BTS (MPa)	*1.5*	*0.2 - 4*	*35*
BTS //	*1.0*	*0.1 - 2*	*30*
FI (joints/m)	*11*	*8 - 20*	*5*
Qtz eq. (%)	*60*	*40 - 65*	*10*
grain size (mm)	*1*	*0.5 - 2*	*10*
F-value (N/mm)	*0.9*	*0.3 - 1.1*	*10*
density (s.) Mg/m^3	*2.1*	*1.9 - 2.2*	*10*

Figure 21.1 Geotechnical model with engineering geological units.

From these descriptions the approximate geotechnical properties can be derived (compare Table B1): The rock is a dolomite, which contains up to 10% quartz, has a grain size of 0.06 - 2 mm, is anisotropic and laminated on a scale of 6-20 mm, the strength is in the order of 12.5-50 MPa, the rock is ductile (UCS/BTS < 9), the F-value is lower than 0.05 N/mm and the rock is not weathered, but karstic dissolution has occurred along the joints, in which clay is present. The block size is from 200-600 mm and the rock blocks have a tabular shape (thickness much less than length or width).

The method of description can be used in bore core logging, but then the information on the discontinuities will be one dimensional and only spacings will be recorded (see however Chapter 22.2). Many of these descriptions will lead to an overall assessment and definition of the properties of the engineering geological unit. The results of the laboratory tests can be used to add numerical information to the linguistic terms used.

The end product of a site investigation for rock dredging should be a geotechnical model of the area to be dredged. The information obtained during the site investigation should be arranged in a data base with the three dimensional spatial coordinates as a reference. This will allow the use of 3D-GIS (Geo Information Systems) as a tool for the project (Houlding 1994, Siezen 1996, De Kok et al. 1997). The contractor can use the model to build up a data base on the production and tool consumption of the cutter suction dredger and develop expert systems for performance prediction.

The three dimensional framework of the model should be based on the geological structure and stratigraphy of the site. The starting point for the model is therefore a study of the local geology and geomorphology, which is based on sea bed observations, boreholes, on-land observations, using also geophysics and other site investigation techniques. Additional information comes from adequate sampling and

Table 21.1 Type of geological and geomorphological information relevant to rock dredging projects.

Geology	Purpose
Existing maps	Prediction of geology underwater at site
Maps made of surface natural & man made outcrops, aided by air photo's etc.	Structural geology; orientation of discontinuities, spacing, block size and shape
	Samples for petrography, mineralogy, testing etc.
Geomorphology	
Coastal and hinterland landform	Connection with buried underwater landform, e.g. landslides
Weathering & erosion	Changes in material properties, lateritization
Submarine landform (side scan sonar maps)	Reefs
Mineralogy of recent deposits	Mineralogy of coastal deposits, e.g. quartz in duricrust

testing, to be able to determine the range of geotechnical properties of potential units. Detailed studies of certain problematic aspects encountered at the potential dredging site, such as for example local cementation in soil units (beach rock or duricrust formation) or problematic boundaries of units add to the quality of the model.

21.1 SITE INVESTIGATION

The site investigation commonly can be divided into a *preparatory phase* and the *main investigation.*

The *preparatory investigation* is intended to discover geological and geomorphological information that will aid the design of the main site investigation. During this stage a visit to the site is essential. The engineering geologist should examine the information listed in Table 21.1. The information obtained from the data available on the local geology and geomorphology should lead to an appraisal of possible geotechnical problems and difficulties of obtaining adequate data during the coming main investigation. The design of the main site investigation follows.

The purpose of the *main investigation* is to construct the geotechnical model, which should contain sufficient information to allow reasonable estimates of production and tool wear. The site investigation normally starts with geophysical surveying overwater (Table 21.2). Usually this survey involves reflection seismic profiling, side-scan sonar and sonar techniques. The choice of equipment depends on the local conditions, which should be defined during the preparatory study phase. Ideally the survey should lead to a three dimensional framework of the subsoil on

Table 21.2 Overwater investigation techniques.

Boundary data and sampling		
1. Geophysics	reflection	geological boundaries
	refraction	velocities as a measure of rock quality
2. Boring	probe core (at appropriate sizes) geophysical well-logging	rockhead depth samples for geology, boundaries, testing
3. Sampling underwater by divers		

which the positioning and the number of boreholes and probes (cone penetration or vibrocoring for the soil above the rockhead) can be based. The framework can calibrated using the results of the bore campaign.

21.1.1 *Geological and geomorphological investigation*

To obtain a clear picture of the rock and soil mass to be dredged, the geology and geomorphology of the area around the dredging site should be carefully examined. This work should preferably be done by an engineering geologist, which is trained in the conversion of geological to geotechnical information. Apart from assembling published literature and maps of the area, it is advisable to consult geologists familiar with the area. These often have insight and experience of the particular characteristics of the soils and rocks present, the extent of weathering features in the rock mass, the likely presence of cemented zones or duricrusts in the soils or the local history of sea level changes.

The engineering geologist is occupied with the development of a geological model of the area. In order to understand the distribution of rocks and soils in the dredging area, it is commonly necessary and useful to pay attention to the coastal area on land as well. Observations of the landscape and inspection of rock outcrops can give valuable information, which can be compared with maps of underwater surface morphology.

During the geological survey, standardized descriptions of the soils and rocks should be used. It is advised to use the procedures and definitions given by BS5930:1981. The PIANC 1984 document, developed for dredging site investigation is based on the BS standard, but the rules for rock description are not given in the PIANC document.

A first model may be made of the different geological formations or units to be expected in the dredging area. On this model, the locations of boreholes are based. The geophysical surveying which is normally done overwater, may also help do delineate different units in the subsurface. Refraction seismics is particularly helpful in indicating the boundary of rock overlain by soil.

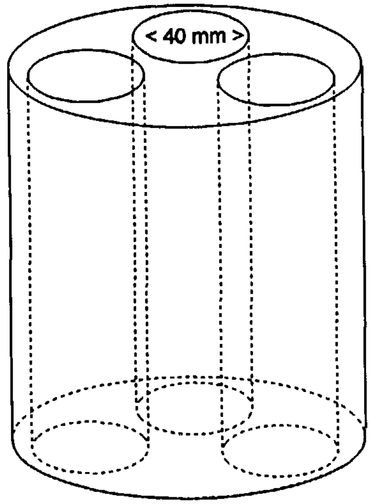


Figure 21.2 Samples for rock testing require large diameter bore core.

21.1.2 *Boring and sampling*

The boring campaign is of great importance. Boring is done not only to obtain information on the location of different geological units, but also to obtain samples for testing. When rock is involved, requirements for the size of the cores are based on two considerations:

1. To recover the rock without damage, high quality drilling is required. In other words, the solid core recovery should be high and no fractures should be induced by the drilling process. This requires often the use of triple core barrel equipment and large diameter coring, when weak rocks are concerned. For example, weakly cemented rock may be described as soil, sand or gravel for example, while during excavation it turns out that it concerns a weak sandstone or a conglomerate. Poor coring may destroy the bonding by the cement and only careful high quality drilling can reveal the true identity of the rock. Important to notice in this case is that petrographic examination of the materials, including sands and gravels, will show the presence of cement on the sand or gravel grains.

2. Size requirements for strength testing. It is customary to core about 60 mm diameter rock cylinders in stronger rock and use 100 mm core for the weaker rocks. The investigation into the abrasiveness and cuttability of rock has shown that it is important to have information on the strength of the rock and the mineralogy, and that it is important to have this information from the same sample. Strength should be obtained from triaxial testing, or the combination of a UCS and BTS test (Chapter 22.1 & Appendix C & D). Mineralogy can be obtained from a slice of rock that can be machined from the samples used for rock testing (Appendix E). For the rock testing, the diameter of the cores should be at least ten times the diameter of the grain size in the rock. Most standards for rock testing prescribe a minimum diameter

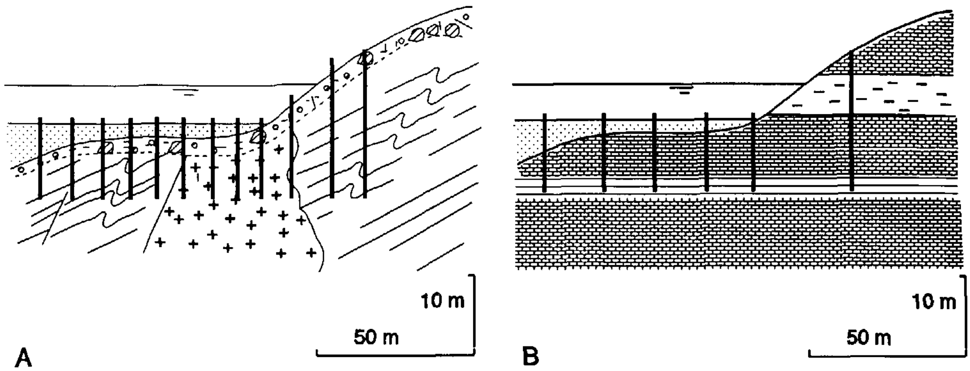


Figure 21.3 Borehole spacing depends on complexity of geology in the subsurface.

of 50 mm. Rocks which contain large pebbles of quartz or shells in the cm size range should have core diameters of 100 - 200 mm or more to fulfil the standard. On the other hand, cores of a diameter of 30 mm or 40 mm may be used for testing, provided the grain size is sufficiently small. To be able to test samples of layered rock, preferably the samples should be taken side by side (Figure 21.2).

These remarks lead to a preferred large diameter of core of 100 mm or larger. Although may be not customary nowadays, one should aim at more high quality, large diameter boring for dredging site investigation, for reasons of improved recovery and good sampling.

The number of boreholes needed depends on the variation in the geology. Geostatistical procedures may be used to determine the density of boreholes to obtain a certain degree of reliability. The principle is indicated by Figure 21.3. Also the number of samples needed for testing can be determined using statistical techniques. Or, if the number of boreholes and samples are limited by the circumstances, the accuracy of the data can be assessed by these methods. In many cases, a proper construction of the three dimensional model of the subsurface geological structure requires boreholes that are not necessary within the zone of dredging. This especially pertains to areas where the rock layers are folded, or when buried channels or valleys are present. Geophysical methods may be used to join the information from boreholes.

The *borehole record* should contain information that can be used to predict the production (excavatability), the tool consumption (wear) and other aspects of relevance for dredging, such as bulking factor or slope stability of the cut. The record should contain information on:

- the drilling progress (advance rates, information on tool wear)
- rock & soil description, according to standards (BS 5930:1981)
- discontinuity description, to be able to derive information on block size (see Chapter 22.2).

An example of a borehole record is given in Figure 21.4.

During the drilling operation, the geologist should log the cores, mark locations for samples for testing and may perform Equotip tests to log the strength variation in the rock. Attention should be given to the description of the discontinuities.

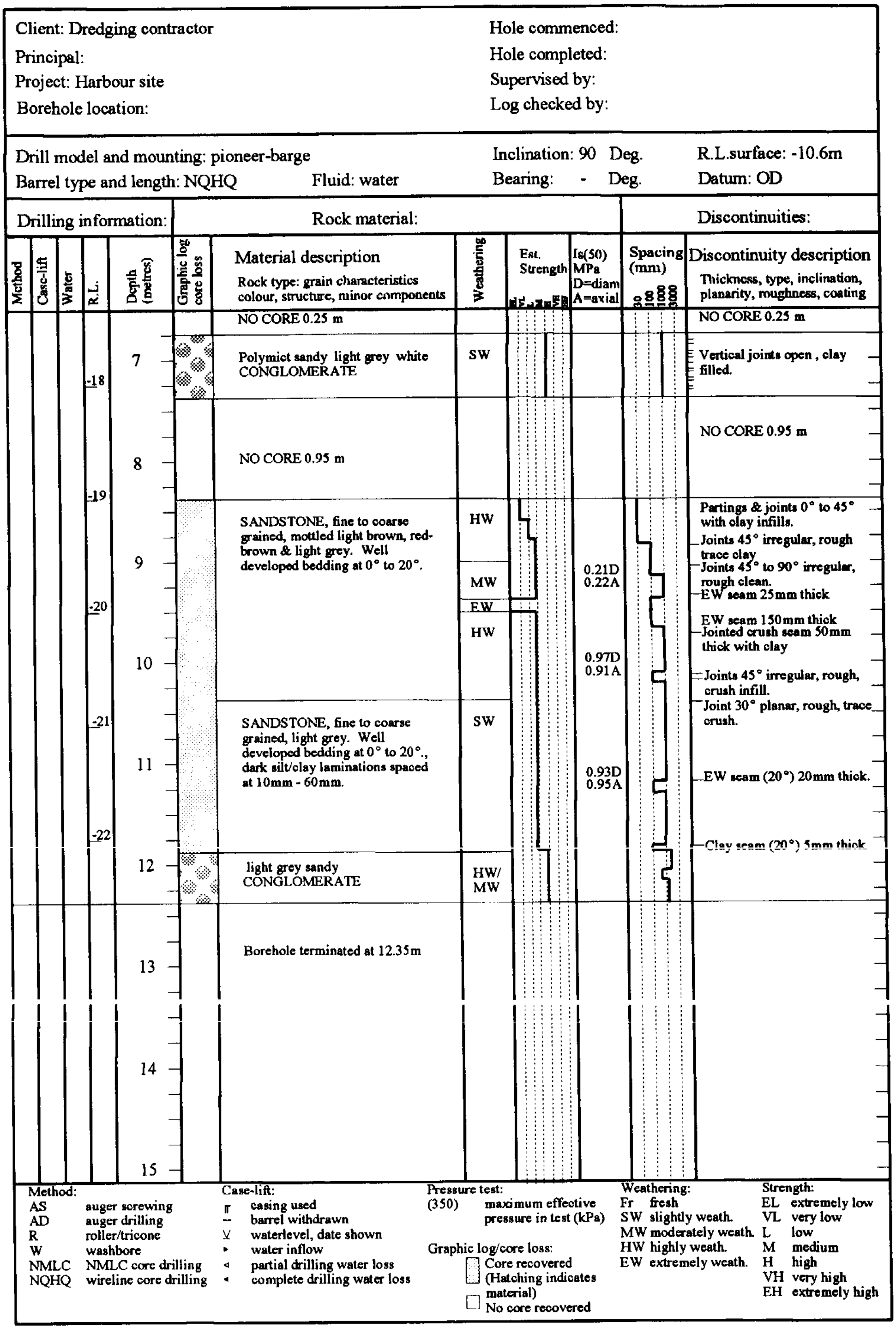


Figure 21.4 Example of a borehole record (after Coffey & Partners).

Table 21.3 Main site investigation programme for rock dredging contracts.

1. Field investigation	Engineering geological mapping on land and, if possible, under water Borehole drilling campaign, in-situ testing Geophysical survey Rock & soil description and classification Quantitative description of discontinuities (following ISRM suggested methods)	Geology and geomorphology of area, structural geology Determination of inhomogeneity and anisotropy of rock and soil units Weathering features (presence of clay, rock core stones in weathered soil) Discontinuity frequency; orientation, spacing, RQD, block size and shape estimation
2. Rock material properties	Sampling from drilling cores or outcrops on land	Unconfined Compressive Strength (UCS), Triaxial testing Deformation modulus Unconfined Tensile Strength (BTS, PLS) Ratio of UCS/BTS Density Porosity Influence of anisotropy or inhomogeneity on mechanical properties
3. Petrographic description	Mineral composition Microscopic structure	Equivalent quartz content (mineral hardness) Grain size and shape Cementation micro-cracks anisotropy, homogeneity
4. Geotechnical modelling	Two- or three dimensional geological-geotechnical modelling	Estimation of quantities (volumes) of engineering geological units to dredge
5. Performance prediction	Excavation model	Choice of dredger Estimate of production and tool consumption

Sometimes the orientation of discontinuities with respect to the core axis should be logged, in order to be able to obtain information on block size and shape and joint set orientations (see Chapter 22.2). When the laboratory tests are done, spreadsheet databases may be set up with the most important information can be compiled in borehole description forms. The combination of all data available may be used to delineate the engineering geological units at the site.

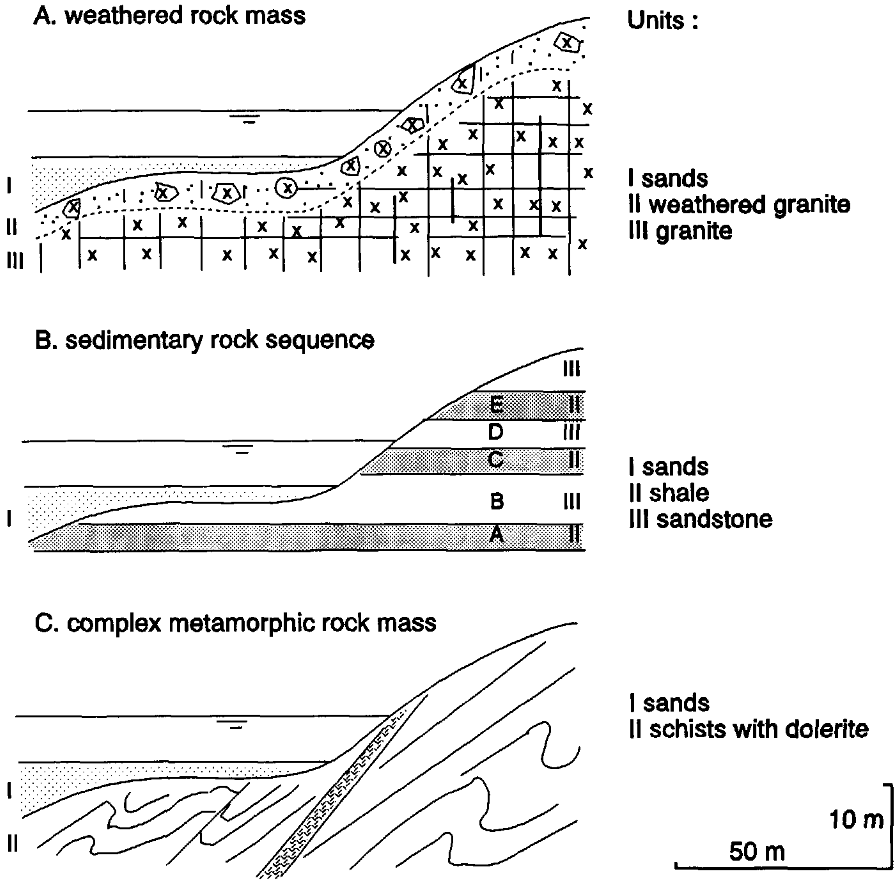


Figure 21.5 Examples of geotechnical units in different geological situations.

21.1.3 *Data to be assembled during the main site investigation*

Table 21.3 summarises the information to be gathered on the rock mass characteristics during the main site investigation.

In many current site investigation reports for rock dredging projects emphasis is on borehole record listings and summation tables of test results. In the case of rocks this pertains to Point Load tests and UCS tests. Commonly no effort is made to base the geotechnical model, when present, on a geological model. Petrography is often limited to visual descriptions and name giving. Mineralogical examinations and microscopic studies are seldom performed. Discontinuities are commonly described in the borehole reports in the form of RQD and FI (fracture index) numbers. This practice needs to be improved, because a sound geological model is a necessary prerequisite for a sound geotechnical model. It is necessary to delineate lithological boundaries of both rock and soil units and the discontinuity patterns to describe the complete mass to be excavated. Focus should be on the proper presentation of the data, by distinguishing lithological units that also have geotechnical significance. Part of the mass may be satisfactorily described using mechanical properties of the materials present. Insight into the geological history of the site can help to understand the structure of the rock mass and the effect of weathering processes on the rock properties (Chapter 22).

The material properties needed for a proper analysis should consist of information on the density, porosity and mechanical strength of the rocks present. The variation of these properties should be recorded as well. The tests on strength should preferably be performed on saturated samples. As is obvious from the trend of this research, a petrographic study of the nature of the rock types is essential to establish the presence and nature of potentially abrasive minerals (Table 21.3).

21.2 ENGINEERING GEOLOGICAL UNITS

The subsurface will be divided into units that are defined on the basis of two characteristics related to the type of soil or rock: the lithology (geology) and the geotechnical properties. Therefore these units are termed *engineering geological units*. Each unit will be defined by a characteristic range of properties. Figure 21.5 gives some examples, which show that the local circumstances dictate the division. The division should be such that the units are meaningful with respect to dredging excavation. Not only the geology, but also the dredging requirements are used to make the subdivision in units. The last remark refers to the *scale* of the units. For a cutter suction dredger, layers with a thickness of 30 cm or more can be considered as separate units.

It is advisable to maintain the geological subdivision as a basis throughout the project, to avoid confusion, such as for example existed in the Port Hedland project (Chapter 5, compare Table 5.3, p.56 with Table 5.4, p.61).

In Figure 21.5B, for example, on the basis of the stratigraphy the layers A, B, C, D & E are a normal succession. Layers B & D have the same range of geotechnical properties, as have layers A, C & E. Now it would be unwise to distinguish only two engineering geological units (this was done in Table 5.3). Better is to divide the rock mass into the engineering geological units AII, BIII, CII, DIII & EII.

Geological and geotechnical properties that would be used to define the engineering geological units are:

- layering/anisotropy (layer thickness)
- material strength properties: strength and ductility
- block size and shape; discontinuity patterns and spacings
- abrasiveness, as determined by strength and petrography/mineralogy

The description of the units should follow standards, as outlined in Chapter 22 & Appendix B. In fact, any information that relates to engineering properties of rock mass can be used to define the units (Table 21.4).

The units themselves can be chosen in such a way that they relate to dredgeability. Each layer or unit should behave in a characteristic way with respect to excavation. The contractor should be able to earmark the engineering geological units as *rippable*, or *massive*, or *mixed*, based on block size estimation. If massive,

Table 21.4 Information that contributes to the definition of an engineering geological unit.

Drilling progress	Advance rate is related to rock properties Information on strength and wear	(Thuro & Spaun 1996)
Rock description	Following standard procedure and standard rock nomenclature Weathering	BS 5930:1981 Appendix B
Discontinuity description	Orientation and density & information on friction properties	Chapter 22.2, ISRM (Brown 1981)
Tests results	Rock density, porosity, Mineralogy, UCS, BTS, E (anisotropy) Derived indicators such as F-value	BS, ASTM, ISRM
Engineering Geological unit	Description Ranges of material and mass properties	
Expert system	derived estimates of dredgeability, tool consumption, pipe and pump wear and others	(Siezen 1996)

production and tool consumption estimates should be derived from the strength and mineralogical information (Chapter 23).

21.3 GEOTECHNICAL MODELS

Once the engineering geological units are established, these are used as basis for the geotechnical model of the dredging location. The boreholes, the information from geophysics and geology serve as the basis for the 3D framework of the geotechnical model. The engineering geological units are volumetric units of a certain range of properties that relate to dredgeability. Once such a system is established, analysis of expected production & tool replacements can be made. But the geotechnical model may also be used to relate to other factors of importance, such as the stability of the cut slopes of a trench, for example.

At present a rapid development of geological data presentation takes place. Usage is made of 2D- and 3D GIS (Geo-Information Systems) and geostatistical modelling (Houlding 1994). Computerized data manipulation and graphic presentation seem ideal for site investigation purposes and will increasingly be used in dredging projects. Examples of the use of 3D GIS systems in the modelling of the subsurface for geotechnical purposes are given by Houlding (1994), Orlić & Rösingh (1995) and Orlić (1997). Siezen (1996) and Brugman (1997) have shown the potential of involving expert systems in the data interpretation (Figure 21.6). The advantage of these systems is that, using the geotechnical model with its engineering geological units as a basis, different purposes can be served. A model for the dredgeability by a CSD can be made, but also a model for cut slope stability. Areas of difficult rock cutting can be made visible and volumes calculated. Another important advantage is

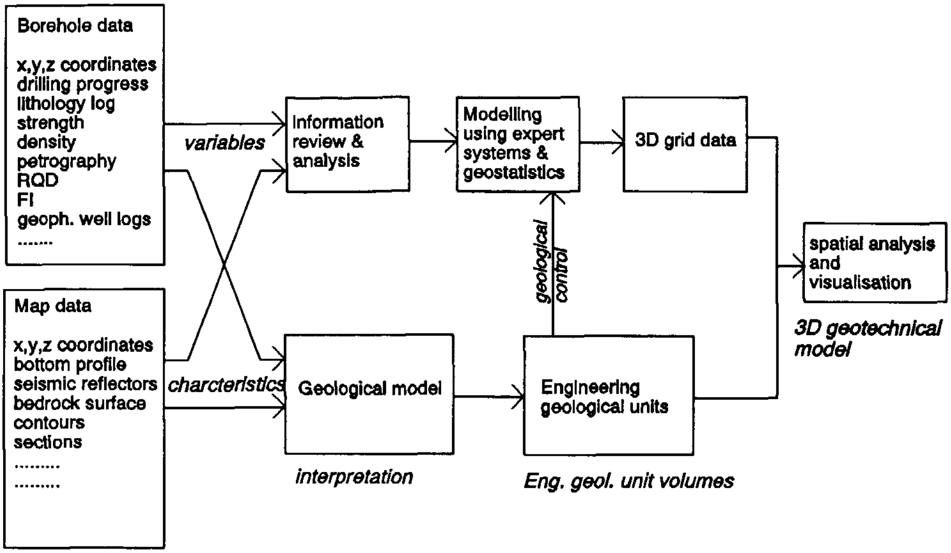


Figure 21.6 Scheme of the information sources, data handling and operations for the development of a 3D geotechnical model, using computer systems (after Houlding 1994).

that new information can be readily included and the model adapted. In the Øresund project, dredging is performed using this method (De Kok et al. 1997).

CHAPTER 22

Rock mass characterisation for rock dredging projects

In this chapter general aspects of the characterisation of rock for dredging projects are treated. Each project will have its own special geological situation and merits careful investigation by the engineering geologist. The information that is needed to arrive at meaningful engineering geological units for a project relates to useful description methods of the rock mass. The advise is to follow the procedures laid down in professional standards. These have been developed for site investigation as such, but also for the rock mass characterisation and the description of weathered rock mass standard or suggested methods exist.

Concerning the production and tool consumption, it has become clear from this work that intact rock strength and the density of fractures in the rock mass are important parameters. The rock types, the extent of weathering, the possible presence of cemented horizons in the soils to be dredged in an area are typical aspects that differ from project to project. Many rock dredging projects occur in areas that have been transgressed by the sea, during the most recent sea-level rise. In such cases dredging occurs in areas previously exposed on land, where rocks may have undergone weathering processes, or soils may have been partly or completely cemented by cementation processes that are common in certain coastal areas. Many dredging projects take place on the boundary of weathered rock and overlying soil. In this chapter therefore attention is given to weathering and also to the cementation that may be present in the soils of a coastal zone (beach rock and duricrusts). Sometimes these cemented rocks have strength higher than the cutter suction dredging limit of 30-50 MPa.

The following subjects are treated: intact rock strength (Chapter 22.1), discontinuities and block size (Chapter 22.2), rock mass strength (Chapter 22.3), weathered rock mass (Chapter 22.4) and rock cementation (Chapter 22.5). The treatment will be concise, but is regarded relevant for this work. The aim is to convince the consultant and contractor of the importance of a thorough engineering geological site investigation for dredging contracts involving rock.

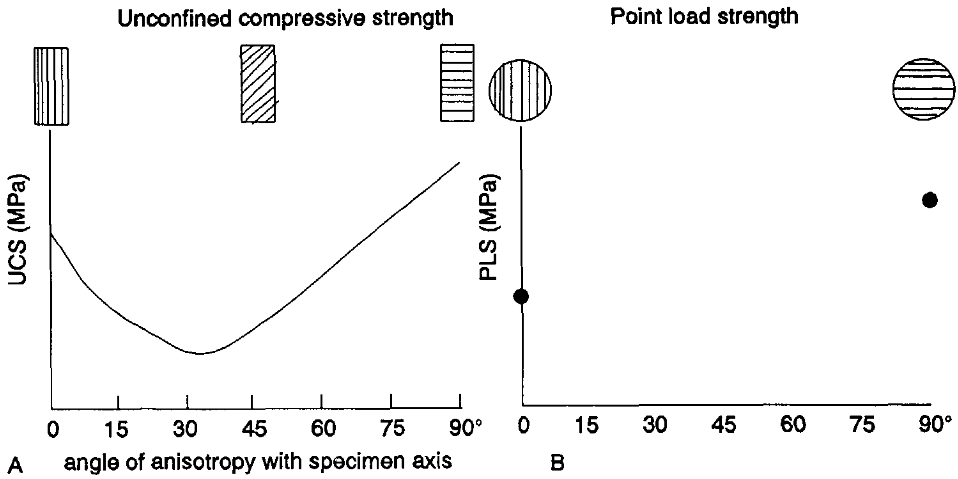


Figure 22.1 Anisotropy of rock strength of laminated rocks in UCS test and in PLS test.

22.1 INTACT ROCK STRENGTH

The strongest cutter suction dredgers are capable of cutting into massive rock materials which are up to moderately strong (UCS values of 30 to 50 MPa, see Chapter 2). Many rock types occur in this strength class, especially sedimentary rocks. Table A6 of Appendix A mentions common weak to moderately strong rocks. Many sedimentary rocks, such as shales, siltstones, sandstones and limestones fall into the weak to moderately strong class, but their range of strength can extend into the strong to extremely strong class as well. It is not possible to derive an idea of strength just from the rock name alone, although special engineering classifications use strength to name the rock. The limestone classification of Clark & Walker, given in Table A2 of Appendix A, is an example of such a classification.

In Chapter 8.2.3 the *integral discontinuity* is described, being a plane of relative weakness in anisotropic rock. Commonly such planes are parallel to a lamination or foliation in the rock. If integral discontinuities occur at sufficiently small spacing (closely spaced to extremely closely spaced, see Table B1, Appendix B1), they can have a marked reducing effect on the strength. This is the reason why anisotropic rock should be tested in directions parallel and normal to the anisotropy. Rocks which can be anisotropic in this sense are sedimentary rocks such as shales and laminated siltstones, sandstones and limestones and the foliated metamorphic rocks such as slates, phyllites and schists (see Table A1, Appendix A). Often the UCS parallel to the direction of anisotropy is lower than normal to it (Figure 22.1). Commonly, when strength values are given, rocks have been tested normal to foliation or lamination. If the integral discontinuities are favourably oriented with respect to the cutting direction, rocks which have UCS values higher than 50 MPa might be dredgeable. This way strong schists and slates have been dredged by cutter suction dredger.

In site investigation for rock excavation projects, normally the Unconfined Compressive Strength (UCS) of rock is the standard used to define rock strength.

Table 21.1 Recommended test methods for intact rock strength determination.

Test	Parameters	Standard
Unconfined Compressive Strength	UCS, E	ISRM 1979 ASTM D-2938, D-3148
Brazilian Tensile Strength	BTS	ISRM 1978 ASTM D-3967
Point Load Strength	PLS (Is_{50})	ISRM 1985
Triaxial testing of rock		ISRM 1978 ASTM D-2664

The core material from borings is often utilised to give material for the test. In many cases the presence in the rock mass of fractures and joints, weak fissile rocks or weathered rock, gives problems in selecting samples from the bore core which fulfil the requirements of a sample for the UCS test. Stronger rocks have a higher possibility of providing good samples. There is normally a bias towards stronger rock being sampled in a site investigation campaign. For the purpose of the study of rock excavation projects this is not such a disadvantage, since the stronger rocks present in a rock unit will determine the performance of the excavation machine.

In practice the Point Load Strength (PLS; Is_{50}) is the most common test used to estimate rock strength. This test can also be carried out on lumps of rock and is known to correlate with the UCS. The popularity of the test is due to the portable equipment that allows measurements taken on the drilling site, for example. The third material strength test is the Brazilian Tensile Strength test (BTS), carried out on discs of rock. If a diamond saw is available, this test can also easily be performed on portable equipment.

Rather than describing the test procedures for these tests, in Appendix C some observations concerning the tests methods and the use of the test results are given. It is strongly advised to perform the tests according to the prescriptions of standards. Table 22.1 gives the recommended standards to carry out these tests.

Both the PLS and the BTS are used to make estimates of the UCS, since the UCS is the standard by which the test data can be compared with the results of other projects or rocks from which UCS data are known. Although still done, it is unwise to use the commonly cited correlation numbers (UCS = 10*BTS, UCS = 24*PLS) to estimate UCS. These numbers should be established by testing and correlation. If such correlation is available, the portable PLS and BTS tests can be done in the field to establish a large data base of strength values for a project.

Other possibilities exist to estimate UCS, for example by using the Schmidt hammer, or the Equotip apparatus (Verwaal & Mulder 1993). The Equotip has potential to be used while logging rock cores.

It is emphasized that, for the purpose of mechanical rock excavation, *both* the UCS and the BTS and/or PLS test should be carried out. The reason is that information on the ductility (brittleness) of the rock is required. This can only be obtained when both the UCS and the BTS test are carried out on the same rock sample (Appendix D). For the same reason it is advisable to have the UCS tests

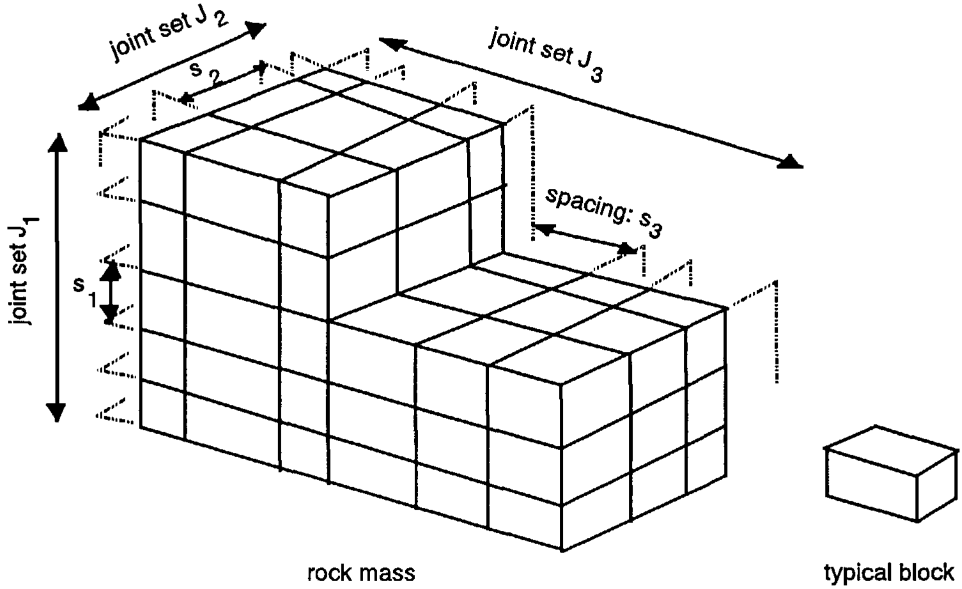


Figure 22.2 Illustration of a rock mass having a discontinuity system of three joint sets.

carried out on a servo-controlled test apparatus, in order to be able to determine the deformation modulus and the work of failure (Figure C1, Appendix C).

Finally the effect of mineralogical composition, rock microstructure, homogeneity and anisotropy on the mechanical behaviour of intact rock should be considered. If the rock is anisotropic, strength tests should always be carried out in at least two directions: normal and parallel to the anisotropy (Figure 22.1).

The description of the intact rock material should be carried out according to standards, such as the suggestions given in the BS 5930:1981, see Appendix B. This way of describing the rocks is informative, since geotechnical information is included in the terms referring to strength, grain size and weathering state. Note that during the boring campaign the rocks can be described in simpler terms, while after the laboratory tests have been done a fuller description can be made.

22.2 DISCONTINUITIES AND BLOCK SIZE

Concerning the description of rock masses, it is recommended to follow the suggested methods developed by the ISRM (International Society of Rock Mechanics, Brown 1981) and the BS 5930:1981.

The recording of the patterns of the discontinuities of the rock mass is not such an easy task if only information can be obtained from over-water surveys. Also when discontinuity surveys can be conducted on-land, where outcrops or quarries are available, it remains to be established whether the patterns recorded are representative for the rocks to be dredged. A *discontinuity* is defined as a significant mechanical break or fracture of negligible tensile strength in a rock. Discontinuity is the general term used, and makes no distinctions concerning the age, mode of

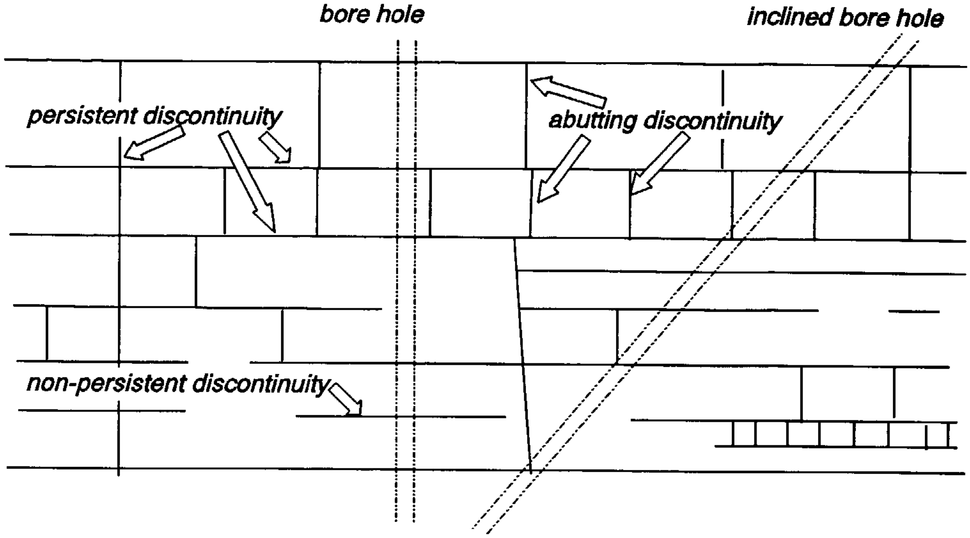


Figure 22.3 Example of a layered rock sequence with persistent, abutting and non-persistent discontinuities. Inclined boreholes are needed to cover the subvertical discontinuity sets.

origin or geometry of the feature (Priest 1993). *Singular* discontinuities and *systematic* discontinuities can be distinguished.

Singular discontinuities are a local event. Examples are faults and shear zones. The systematic discontinuities can be described as one or more *sets* that describe a *system* or *network*. A set consists of a regular assemblage of discontinuities that have a common orientation (are more or less parallel). Examples of discontinuities that occur in sets are layering or bedding discontinuities and joints. *Joints* are tensile fractures that are very common in rocks. They normally occur in a system of two or more sets. Figure 22.2 illustrates the concept of a rock mass with a system of three joint sets. Joints are the most common discontinuities in a rock mass. Figure 22.3 illustrates that in the case of layered rock, joints are often confined to the layers. In terms of discontinuity description such joints are called *abutting* joints. Discontinuities that end within the rock are described as being *non-persistent* (Hack 1996).

In borehole records commonly two terms related to discontinuities are recorded. The *frequency* or *Fracture Index (FI)* and the *Rock Quality Designation (RQD)*. Both relate to the density of discontinuities in the rock. *Frequency* is the number of joints per metre recorded. Such recordings should be made of continuous sections in the bore core (i.e. within one rock unit), to be significant. The *Fracture Index (FI)* is synonymous to frequency. *RQD* (Rock Quality Designation) was the first rock mass qualifier (Deere 1964) that gained wide popularity, and RQD is still used in most rock mass classification systems. RQD is the percentage length of a given length of core consisting of intact rock pieces longer than 0.1 m. Some criticism can be given to the usage of the RQD. The point is illustrated by Figure 22.4. Due to the threshold value of 0.1 m solid core length, rock sections that do contain discontinuities and rocks that do not can both have an RQD of 100%. Rock with a frequency of 11 joints per metre may have an RQD of zero.

By logging the intersections of the discontinuities with the bore core axis and the

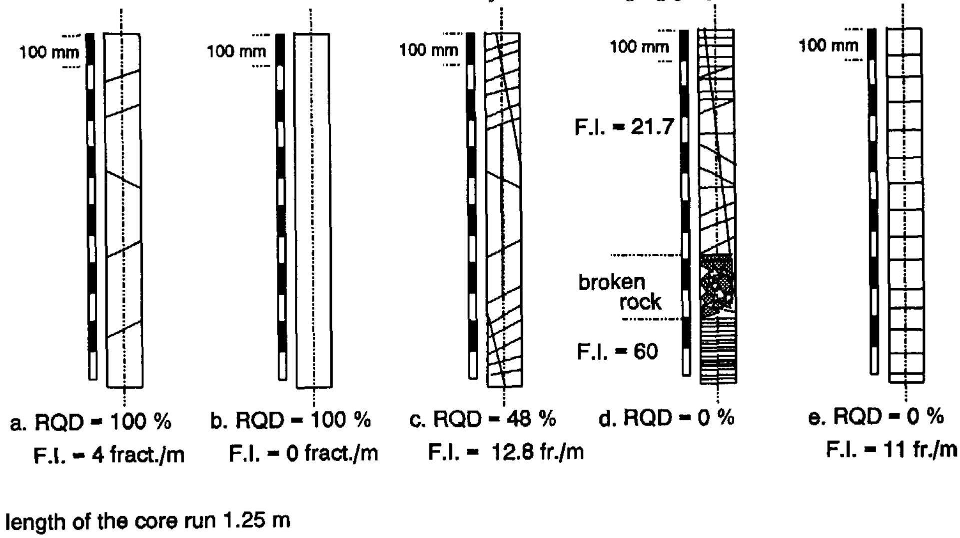


Figure 22.4 Illustration of the concepts of RQD and Fracture Index (frequency).

angle that the discontinuity surface makes with the borehole (Figure 22.3), all relevant data, such as joint frequency, RQD, can be derived later during the analysis phase of the data. Such procedures are not so common in site investigations for dredging, but are recommended if a dredging project occurs in jointed rock. From bore core information, the discontinuity system or network cannot directly be derived. Since the core can rotate during the extraction, special sampling and analysis techniques are needed to determine the true orientation of the discontinuity. If more boreholes are made into bedded rock, for example, and the bedding can assumed to be of constant orientation, graphical techniques, using stereographic projection, can be used to find the orientation of the measured discontinuities and to construct a representative discontinuity system (Priest 1985, 1993, Van Staveren 1987).

If it is thought that the rock mass discontinuity system is relevant to a dredging project, attention should be given to the borehole survey. Figure 22.2 & 3 illustrate that vertical boreholes in this case have a very high chance of missing the important subvertical joint sets. Information of these sets can only be assembled using inclined boreholes on land, or by direct observation of submerged or on-land outcrops. The three dimensional framework can be modelled by:

- geometrical analysis of discontinuity systems (joint systems consisting of joint sets)
- analysis of discontinuity density (RQD, FI)
- locating singular discontinuities (faults, shear zones)

For each discontinuity (joint) set, acquisition of the following data are recommended by Priest (1993):

1. number of sets, mean orientation and statistical variation around the mean (Fisher's constant), for each set
2. mean normal frequency for each set plus isotropic frequency for fractures not belonging to sets

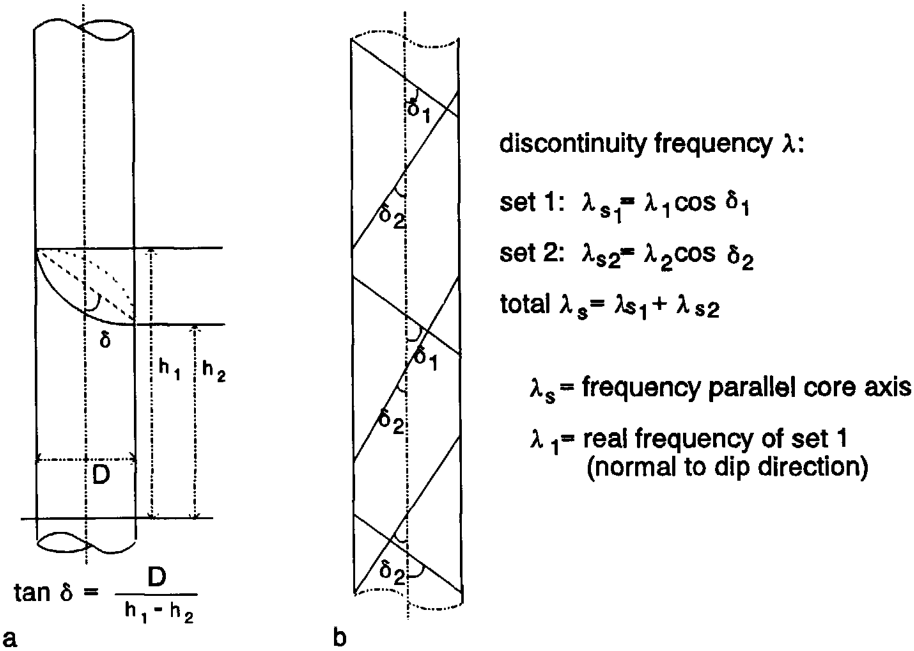


Figure 22.5 The angle δ of discontinuity plane and bore axis can be used to derive parameters such as the real orientation and the real frequency of a set or the total frequency in any direction.

3. mean size (trace length or disc diameter) plus isotropic size for fractures not belonging to sets
4. mean shear strength characteristics

Other information (such as mean normal and shear stiffness for each set and mean physical or effective hydraulic aperture for each set is not considered relevant for excavation purposes. For dredging mainly the geometry characteristics (points 1-3) are important, although the other factors relate to the strength of the rock mass and must have influence on the rippability. In many cases only data coming from bore cores can be studied. In that case the statistical treatment of data that can be derived from bore cores is probably interesting. For example corrections can be made of the Fracture index, by giving the sets that make an angle with the bore axis a weighting factor, as shown in Figure 22.5 (Ruth Terzaghi method, Priest 1993), to derive at a better estimate of the density of joints in the rock mass. If both the orientation and the frequency of the sets of discontinuities building up the network are known, information regarding excavation performance, such as favourable excavation directions can be derived. If only bore core information is available the task of deriving the orientations of the different sets of joints is difficult, but not impossible when specially oriented boreholes can be made (deviating from the vertical in many cases). If outcrops of the same rock units are present on land, it is likely that the block size within the different geotechnical units can be estimated (see Brown 1981), perhaps in excavated trial trenches.

One of the purposes of the discontinuity survey is to establish the distribution of the potential block sizes and block shapes of a discontinuous rock mass. Block size can be described according to BS 5930:1981 (Appendix B, Table B3) in terms of being *very large, large, medium, small* or *very small*. The dimension of the discontinuity set with the largest spacing is chosen for the block size classification and the second term used (*blocky, tabular, columnar*) gives an indication of the shape (and the dimensions) of the other discontinuity sets involved. Pettifer & Fookes (1994) used the discontinuity spacing I_f to give the block size (the mean value of the average spacings of the three major discontinuity sets), see Chapter 18.2. Price has developed a ratings system to indicate the relative size and shape of rock blocks. This system might be of use in excavation projects and is therefore described in Appendix B. The ratings system gives high values to large, cubic blocks. The ratings diminish as the blocks become smaller and change in shape from cubic to tabular and then columnar (Figure B2).

22.3 ROCK MASS STRENGTH

It is clear by now that the presence of discontinuities in the rock mass effectively reduces the strength of stronger rocks. Strength reduction by discontinuities, possibly in combination with weathering, may be such that the rock may become dredgeable.

In Chapter 8 & 18, the rock mass classification systems that are used in engineering geology have been discussed. In has been shown that these systems indicate the rock mass quality. The RMR system, for example, has shown to relate to excavation performance (Chapter 8.2.4; 18.3). From this information it appears that rock with a RMR below 45 can probably be excavated by mechanical means. In Chapter 18.4 it is concluded, however, that the systems are too crude to be used in production estimation. During the site investigation, the systems can be used to evaluate the rock mass.

In rock engineering RMR values are used to make estimates of the strength of the rock mass, among others by using the rock failure criterion of Hoek & Brown (see Hoek & Brown 1988).[76] Estimates of the rock mass strength by this method, however, give unrealistically low strength values. Probably the constants have been chosen on the conservative side, since the estimations are commonly used for underground construction in rock masses or for slope stability calculations. The values of rock mass strength calculated with this method would give an unjustified optimism with regard to machine excavation.

In the site investigation stage, it is helpful to use Figure 18.3 (Rippability chart) in considerations of excavatability.

For improved predictions and usage during dredging projects, however, many more case studies should be assembled to improve the knowledge of dredgeable rock masses (see Chapter 23).

[76]See Equation C3, Appendix C. Parameter m can be estimated from: $m = m_i \exp\{(RMR-100)/28)\}$, where m_i is the m-value for intact rock. Parameter s can be estimated from $s = \exp\{(RMR-100)/9)\}$. The UCS of the rock mass ($\sigma_3 = 0$) is solely depending on the factor for fracture density s: $UCS_{mass} = s^{0.5}.UCS_{material}$.

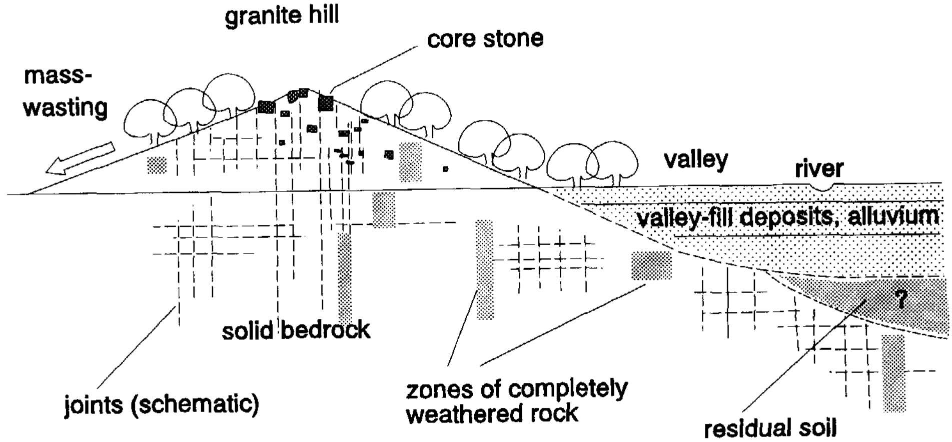


Figure 22.6 Cross section through typical granite hill, Chonbury area, with tentative interpretation of weathering features below surface.

22.4 WEATHERED ROCK MASS

Many dredging projects occur in zones where rocks have been partially to completely weathered as a consequence of their previous exposure on land. Appreciation of the effect of weathering on the rock properties is important. Rocks can be reduced in strength by weathering, so that originally strong rocks can become dredgeable. In silicate rocks the effects on strength, but also on cutting behaviour may be dramatic. However, the weathered rock may cause greater difficulties than the fresh source rock, if it contains clay and quartz. Also the odd *core stone* (a weathering resistant remnant of relatively fresh rock) may give problems.

In many cases weathering intensity may be inhomogeneously distributed in the rock mass. Attempts have been made to design special classification systems to describe weathered rock masses for engineering purposes (Anon. 1995, Price 1993, 1995).

In BS 5930:1981 particular attention is given to the description of the weathering state of the rock material or rock mass under consideration (see Appendix B). Weathering of rock is due to a combination of processes, which have been broadly grouped into mechanical disintegration and chemical decomposition. Generally these act together, while the intensity depends on the climatic conditions occurring during the geological history of the rock. Mechanical weathering describes the set of processes that lead to fracturing of the rock, by opening up of discontinuities on the rock mass scale, or inducing new fractures on the rock material scale. Chemical weathering refers to the processes that occur mainly due to the interaction with atmospheric water or ground water percolating through the rock. This may lead to redox reactions or dissolution processes between the often somewhat acid water and some or all of the minerals constituting the rock. Silicate minerals tend to react towards clay minerals and Fe- or Al-oxides. The end product of chemical weathering of silicate rock is the well-known lateritic or bauxitic soil, consisting solely of clays and iron or aluminium oxides. Complete weathering, however, is only known to occur under wet tropical conditions. In most situations, not all minerals are decomposed by weathering reactions. Quartz is known to be resistant to chemical weathering. It has been found that biological processes often assist weathering. Mechanical weathering may be assisted by the action of plant or tree roots, chemical weathering by the catalytic action of specialized bacteria.

Chemical weathering reactions involve changes in volume of primary and secondary minerals. In silicate rocks the effects on the strength, and also on cutting behaviour, may be dramatic. Weathering reactions involve the transformation of minerals like feldspar to clay minerals. Many of such reactions involve volume changes that may result in high internal stresses in the rock. Some weathered granite rocks have internally completely crushed minerals, including quartz, which become very angular in the process. Weathered granite soils may therefore be very abrasive (see Chapter 22.4.1).

Whereas the silicate rocks weather due to a combination of chemical reactions (including dissolution), the limestone rocks and evaporitic rocks mainly dissolve during weathering. If no other minerals, like clay, are present in limestone, the material properties, such as strength, will not be affected by weathering. Such a rock will mainly have dissolution holes or widening of discontinuities, for example along the bedding or joint planes.

For the general description of weathered rock, the tables in Appendix B may be consulted. Rock material can be described as fresh, discoloured, decomposed or disintegrated. Normally the major strength reduction occurs in decomposed rock and disintegrated rock. Completely decomposed or disintegrated rocks have the characteristics of an engineering soil. In many cases such soils have voids both within and between mineral grains, may be partly filled up with weathering products such as clay or Fe-hydroxides, but these products may also have been removed by leaching. Locally the bonding between mineral grains may be still very strong (Anon. 1995).

Rock mass weathering is described with respect to the weathering state of the rock materials in it and the effect of weathering on the discontinuities. BS 5930:1981 distinguishes six grades, that are commonly used (Appendix B).

22.4.1 *Example of dredging in soils derived from weathered granite.*

Some dredging works have been carried out in soils that were derived from weathered granite bedrock. One example is the Port Hedland project (Chapter 5). Another is the Chonbury coast of Thailand, where dredging works are regularly carried out in an area underlain by granite bedrock. The recent sea-level rise was such that dredging off today's shore-line occurs in a drowned granite hill landscape. Shallow depth seismic reflection profiles show this hilly landscape covered by more recent horizontal sediment deposits. On land this landscape can still be observed (Figure 22.6).

In some cases the soils to be dredged were recognized to be derived from granite,

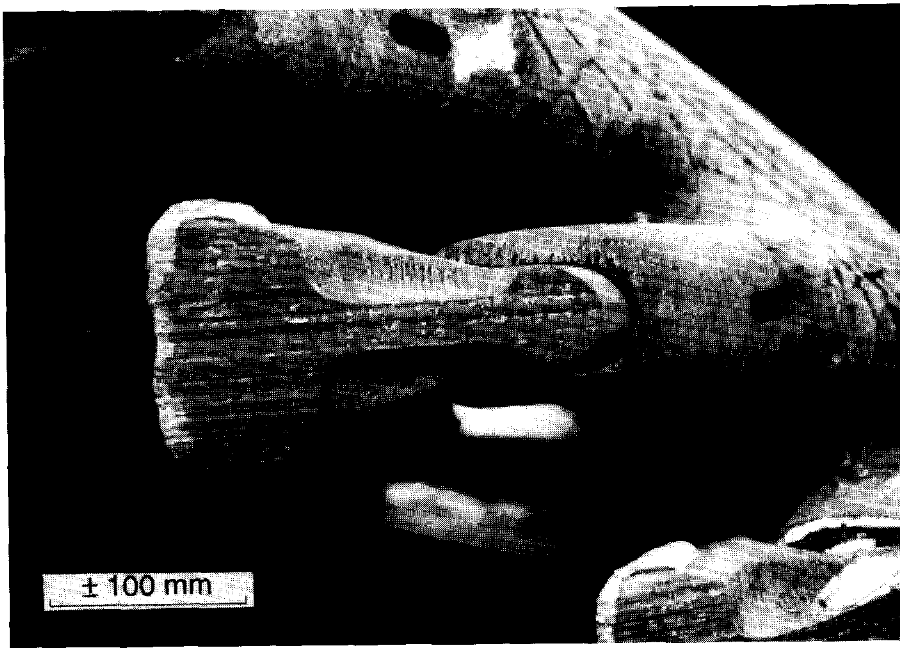


Figure 22.7 Grooves cut in dredger chisel, by angular quartz in stiff sandy clay.

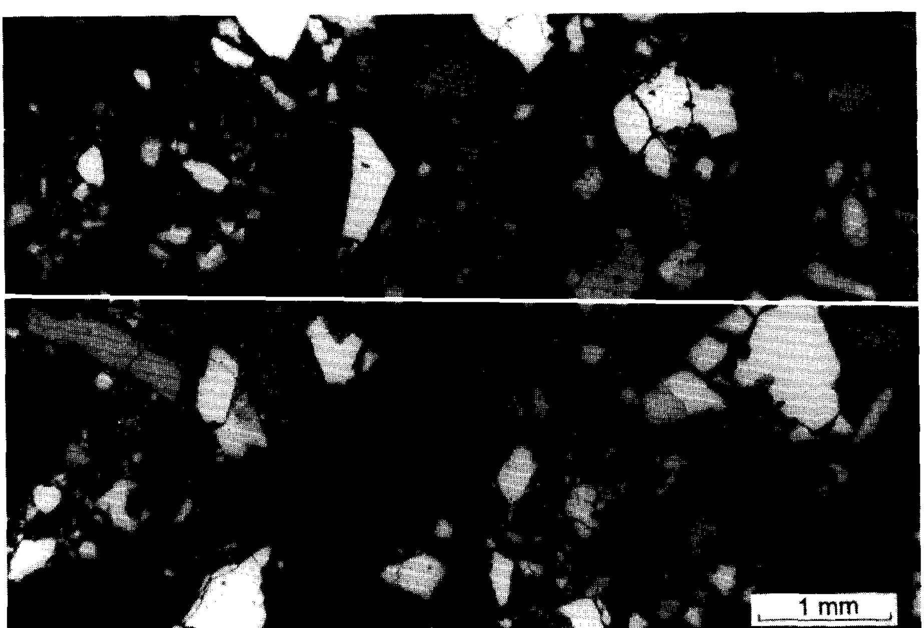


Figure 22.8 Angular quartz embedded in clay matrix (polarizing microscope, crossed nicols).

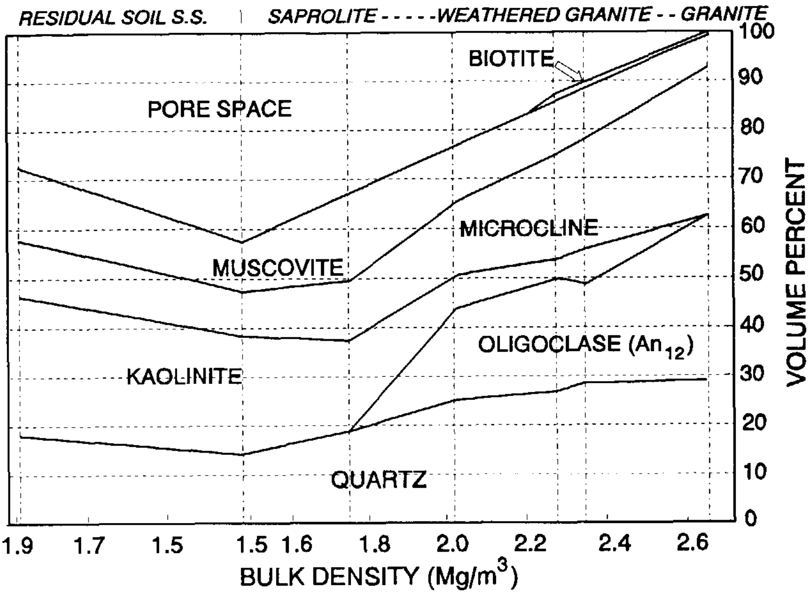


Figure 22.9. Mineralogical composition of weathered Stone Mountain Granite, determined on various samples of weathered rock and residual soils, by point counting of thin sections (after Grant, 1963).

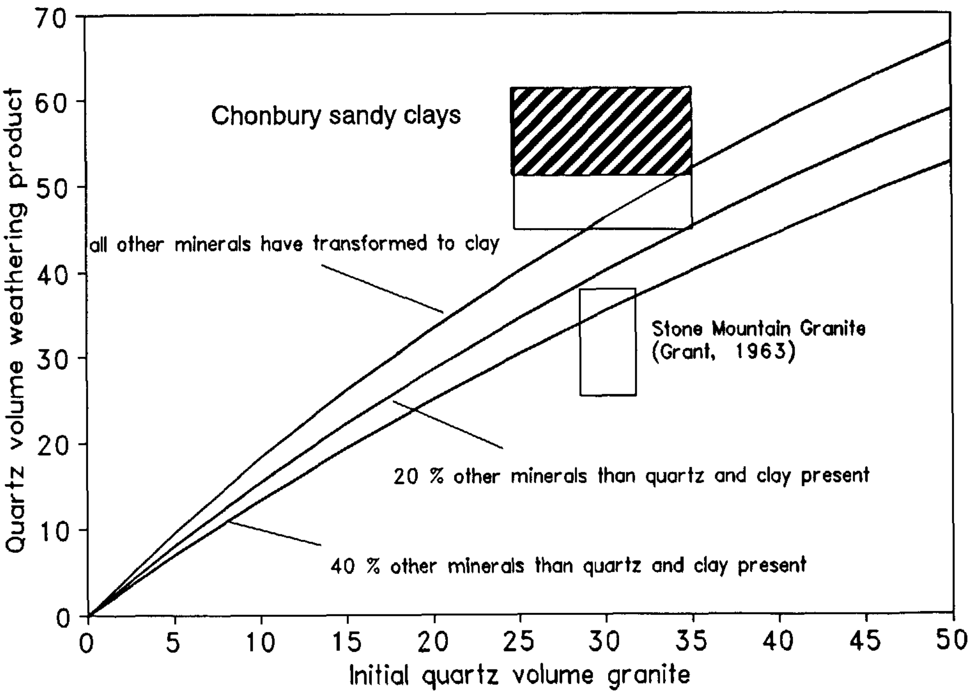


Figure 22.10. Estimated quartz contents of weathered granitic rock. Solid line: complete weathering of non-quartz minerals to clay; broken line: 10% of these minerals still occur (see text).

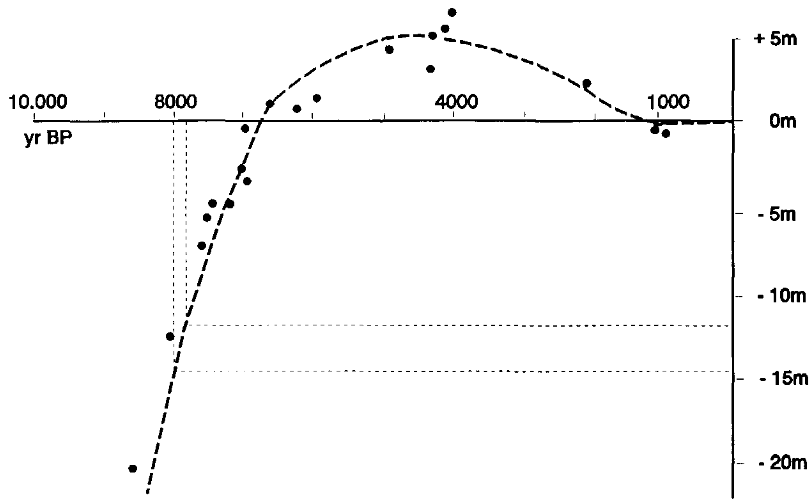


Figure 22.11 Sea level curve for the past 8000 years of the Strait of Malacca region, Malaysia (Geyh & Kudrash 1979).

but considered to be *residual soils*, which strictly means non-transported weathering soils still in contact with the source rock. The clayey sands dredged, however, are very rich in quartz. They cause high wear on the cutting chisels, because the stiff clay firmly holds the angular quartz grains ranging in size from silt to small gravel, causing grooving wear (Figure 22.7). A thin section prepared from the clayey soil causing the wear is shown in Figure 22.8. Thin sections of several soil samples of a dredging project have been examined. The mineralogical composition of undisturbed samples of clayey sands showed a rather high quartz content. The modal volume percentages of the constituent minerals was determined by point counting and most of the samples studied had percentages of quartz higher than 50%. If the volume of voids is discarded, a weathered granite would have a volume of about 20-35% of quartz (Grant, 1963; Irfan, 1988). Figure 22.9 shows the classic diagram of Grant (1963), which gives the mineral content of granite in stages of increased weathering grade. From this diagram it is obvious that the volume content of quartz does not increase in the soils compared to the source granite. Using the volume changes observed by Grant, the graph of Figure 22.10 was constructed and the range of quartz contents of the samples studied is given. Most samples fall into the hatched area. Figure 22.10 indicates that the Chonbury clayey soils have a quartz content that is equal or higher than could be obtained by in-situ weathering alone.

When studying the granite occurrence on land, it is clear that much of the weathered soil will be transported away from the source. Figure 22.6 illustrates transport along the slopes (mass wasting) and transport by surface water is also feasible. During the recent sea-level rise in this area (Figure 22.11), coastal erosion processes must have occurred as well. These type of processes can be envisaged to explain the high quartz content of the granite-derived soils. Quartz enrichment may occur by washing-out of clay particles.

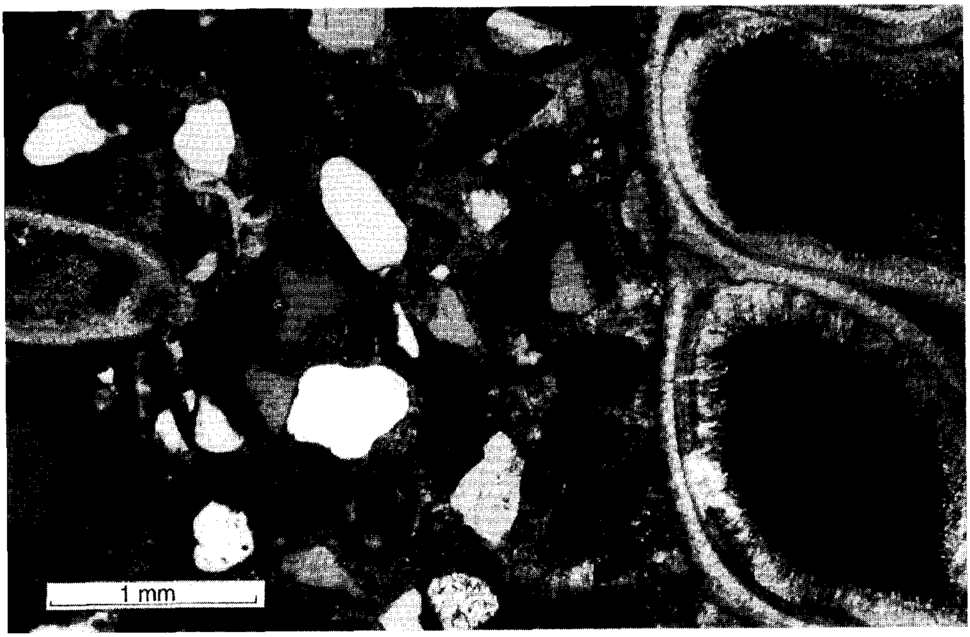


Figure 22.12 Fotomicrograph of cemented sands from Jubail (Gulf area). Note drusy calcite cement in gastropode shell and blocky calcite cement (sparite).

22.5 ROCK CEMENTATION

In many dredging projects in coastal areas the phenomenon of cementation of soils is encountered. Sometimes unexpectedly, parts of a sand mass turn out to be cemented to strong rock, that may occur locally in small patches or in thin or thick layers. This cementation might be the result of processes that are geologically recent. For example, so-called *beach rocks* are found in which the sand has been cemented together with modern bottles and other garbage (Prothero & Schwab 1996). Cementation can occur related to weathering processes or present-day top-soil formation processes. Well-known are the *duricrusts* (also called *hardgrounds* and when related to top-soil formation *pedocretes*). Duricrust formation is related to processes acting within the zone of weathering. The products of chemical leaching are commonly transported by groundwater. Precipitation of cementing agents, such as iron- and aluminium sesquioxides, silica and calcium carbonate occurs at specific horizons or sites, depending on chemical environmental conditions. Duricrusts or pedocretes (Netterberg 1984) are common in tropical and subtropical environments and named according to the nature of the cement involved: calcrete (Wright & Tucker 1991), ferricrete (laterite; Aleva 1990), silcrete, phoscrete, gypcrete. Such near-surface cementation is possible under any climatic condition, but most common in warm climate areas.

If dredging occurs above areas which have been subjected to relative sea-level rise and concern drowned ancient landscapes (such as the Port Hedland case of

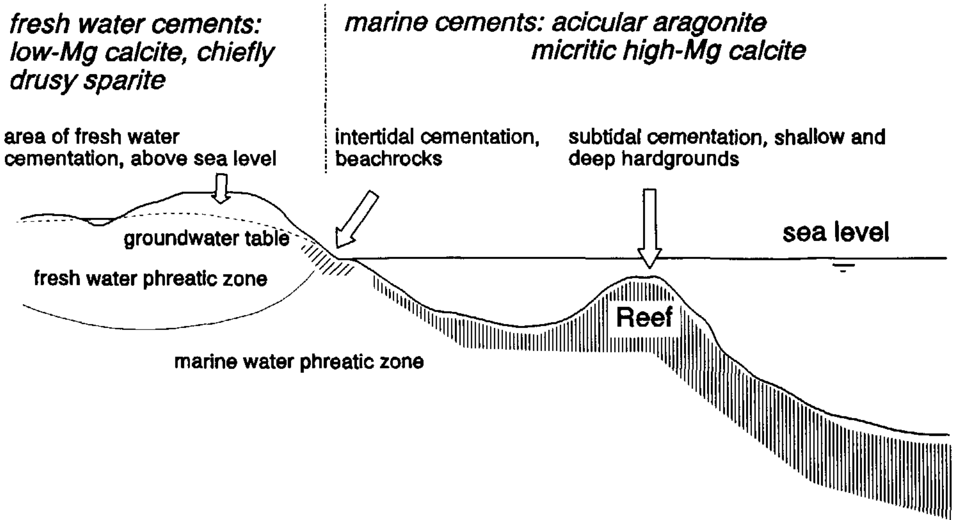


Figure 22.13 Principal environments of cementation in carbonate rocks, after Tucker (1981) and Tucker & Wright (1990).

Chapter 5), study of the rock weathering history might reveal the likelihood of hardgrounds like ferricrete (laterite), silcretes and the like, besides the presence of other weathering products like clays. Again, the better the picture that is obtained by the geological study, the better the contractor can be prepared for the rock dredging work.

Cases are known in which unexpectedly extensive zones of cementation were met in dredging projects. In certain areas such cementation is likely to be present. For example in the regions of carbonate platforms. But also in more temperate climates carbonate cementation is known to have occurred in sands. These observations all the more support the thesis defended in this work: that geological examination of the area at hand is absolutely necessary. Petrographic examination of soil samples can reveal the presence of cementation (Figure 22.12). If cement is present in the soils, then one is warned of the possible presence of beach rock or zones of hardground in the sediments.

22.5.1 *Cementation in carbonate rocks*

In areas where limestone occurs, or in regions with present day carbonate sediments (carbonate platforms, reef areas), cementation is very common. Calcium carbonate is soluble, therefore groundwater in these areas is normally saturated with calcium carbonate. Calcium carbonate precipitation from such saturated waters readily occurs if the chemical environmental conditions are favourable. Cementation occurs when carbonate is precipitated in a pre-existing void space. Cementation often starts with tiny needle like crystals that radiate away from the rim of a void space. Crystals tend to nucleate on the surfaces of sand grains or shells (*drusy cement*, Figure 22.12). If cementation continues, void spaces tend to fill up with crystalline calcite (*sparry* calcite, or *sparite*). The most extensive early carbonate cementation occurs in the areas above sea level, in zones where fresh rainwater or run-off surface water penetrates the ground above the groundwater level. The area around the groundwater level (the *phreatic zone*) is also prone to cementation (Figure 22.13). The typical cement is low Mg-calcite, which can be distinguished by mineralogical methods from the high Mg-calcite variety that precipitates typically from marine pore water. In the shallow subtidal zone often a crusty surface known as a *hardground* is present. Such hardgrounds are usually cemented mostly at the interface of sediment and seawater, and the cementation degree decreases within centimetres from the surface. The type of cement is commonly very fine grained (muddy cement: *micrite*). Seawater is supersaturated with calcium carbonate and it takes just a slight change in water chemistry, or impurities and sites of nucleation for marine cementation to occur. In the intertidal zone usually such fluctuation of water chemistry occurs, which results in *beach rock*, which can form in a matter of years under the right conditions. The cements are composed of drusy needles of aragonite and high-Mg calcite. The cement is apparently precipitated when sea water evaporates (Prothero & Schwab 1996).

CHAPTER 23

Conclusion: Wear assessments within site investigations for rock dredging

This research started with the aim of improving the prediction of tool consumption for rock dredging contracts (Verhoef 1988). The approach taken was to view the problem this time from the perspective of the rock mass to be excavated. Earlier, most attention was given to the improvement of the cutting machines and cutting tools. Two aspects can be distinguished: The proper geological and geotechnical description of the rock mass and determination of rock parameters (to be done at the site investigation) and the prediction of the tool consumption and production of the rock cutting dredger.

It has been found that the site investigation should aim at a well developed 3D model of the subsurface, with different engineering geological units delineated. Good understanding of the geological history of the area around the site is necessary and the geotechnical interpretation should be done by professionals able to communicate the geological information to the dredging engineers. The special requirements of dredging should be understood by the professionals involved in the site investigation.

To improve tool consumption prediction, the methods of laboratory testing of rock samples for this purpose were examined. It was attempted to use tests that examined fundamental wear mechanisms, such as two-body wear or three-body wear. However, the laboratory testing program learnt us that test results aimed at wear are not easily transferred to other scales. Cutting and wear tests with tools that are much smaller in size than the real tools are not likely to be relevant. However, insight into the mechanisms operating have been obtained and the tests indicate that a combination of rock strength, mineral hardness and mineral grain size describe the wear rate reasonably well. Wear rate is a non-linear process, and the scraper tests of Deketh have revealed that the non-linear part is related to the penetration phase of the tool, where changes in modes of wear process occur. When the tool is able to reach a cutting depth sufficiently deep to reach wear mode III (three-body abrasive wear), the wear rate may become constant.

The conclusion drawn is that, even more important than special cutting or wear tests, the more fundamental rock parameters strength and mineralogy should be used in assessments of potential wear. These fundamental parameters can be compared with the performance of, for example, the exploration drilling equipment (drilling progress, information on bit wear), the observations during sample preparation in the laboratory, the performance in laboratory cutting or wear tests and - of course - during the actual excavation with the excavation machine/dredger. All this information helps to obtain a picture of the abrasiveness of the rock concerned.

Regarding the interpretation of the site investigation results, the conclusion is drawn that the general rock classification systems known in Engineering Geology, can only give a first indication of the excavatability of rock. Instead of proposing yet another classification system, it is thought that another type of modelling should be applied. The factors involved in rock cutting performance and tool consumption can be formulated into rules that may be used in expert systems using fuzzy logic, as already applied on the data assembled of the rock trencher projects (Chapter 17 & 18.4).

In Chapter 23.1, based on both the laboratory work and the observations on the trencher performance, the rock factors of importance are outlined. These rock parameters are discussed with respect to site investigations for rock dredging contracts. This information is mainly relevant for the consultants that are responsible for the site investigation.

In Chapter 23.2 suggestions are made how contractors can improve the production and tool consumption prediction by improving their in-house data base on dredger performance in rock dredging projects.

23.1 ROCK FACTORS INVOLVED IN EXCAVATION AND TOOL CONSUMPTION

Dredging works normally comprise a complexity of operations consisting of soil excavation and movement and in some cases rock excavation and transportation. The client, usually a Port Authority, carries out the site investigation, ideally through a firm of consulting engineers who act on their behalf. The consulting engineer prepares the site investigation contract document and selects, through a tendering procedure, a firm of specialist site investigation contractors to carry out the site investigations. The client should be aware of the peculiarities of dredging. The contractor that is going to carry out the work will base the type of dredging equipment to be used on the data obtained by the site investigation contractor. If inadequate, may be the wrong equipment is applied and contrary to on-land engineering works it will be often impossible to bring in other better suited equipment. Such instances usually result in claims and associated extra work and costs if such claims are disputed, despite clauses in the main works contract stating the dredging contractor has to verify or supplement any geotechnical information contained in the tender document. The client will find little sympathy in court with such clauses especially when the main works tendering period is too short to carry out an adequate site investigation to supplement or verify previous investigations.

Several working party reports have summed up the techniques involved in geotechnical site investigation. The PIANC document (1984) is applied specifically to dredging. Another useful document is the Report of the IAEG commission on site investigations (Price, 1981), which gives an outline of most methods, procedures and

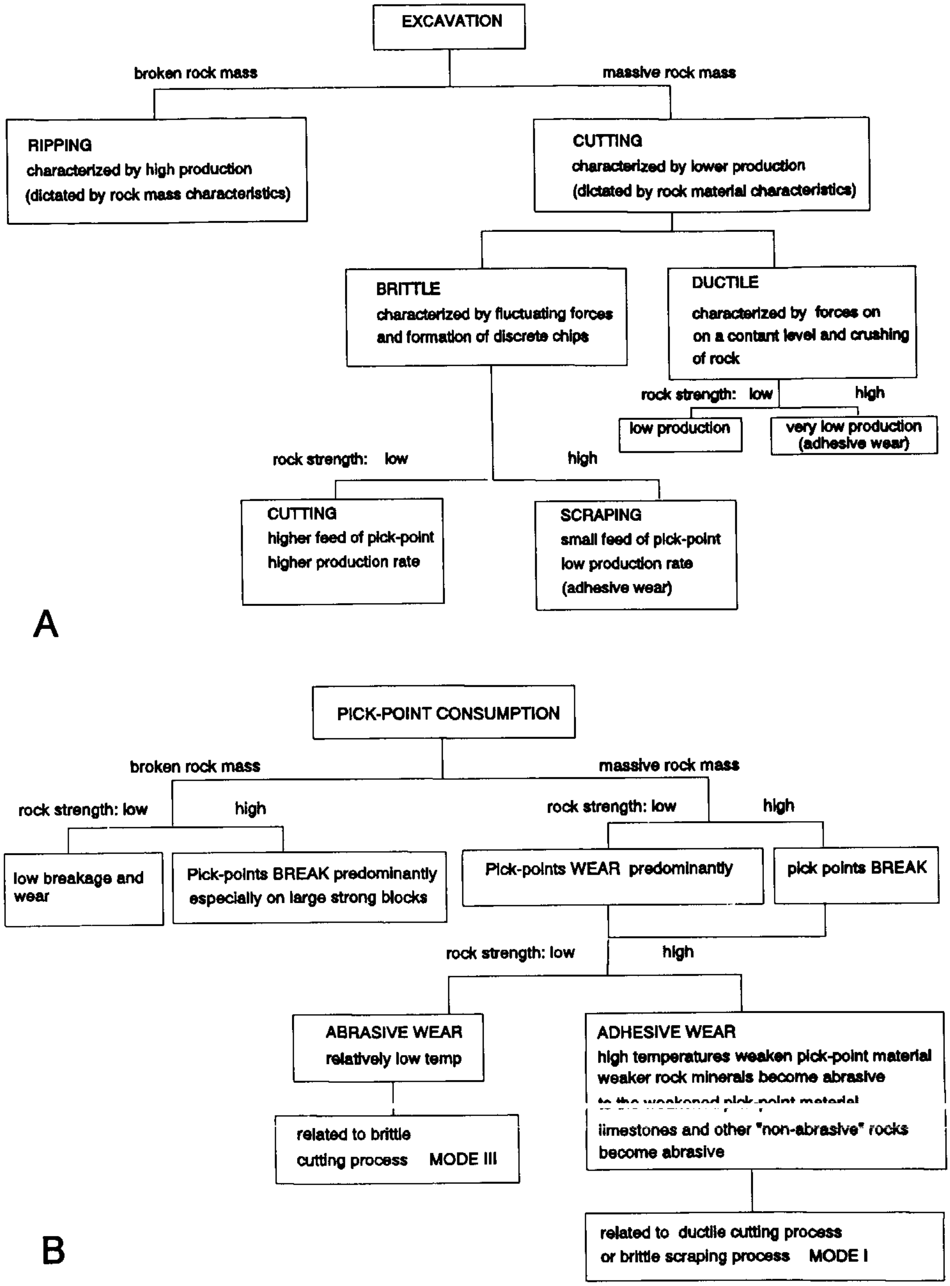


Figure 23.1 Rock-tool interaction, excavation and tool consumption mechanisms.

testing techniques involved in a site investigation. The British Standards Code of Practice for site investigations, BS 5930:1981, is the most comprehensive document at present covering site investigation, mainly discussing on-land techniques. This document is followed in the suggested classifications of Appendix A & B.

When rock is involved, the first choice to be made is between fragmenting the

Table 23.1 Rock properties of importance in mechanical rock excavation.

ROCK MASS		ROCK MATERIAL			MACHINE PERFORMANCE	
Nature of rock mass	Block size/ joint spacing	Strength	Ductility	Abrasiveness	Excavation rate	Tool consumption
massive	large	high	brittle	relevant	feed small: very low	tool breakage likely, very high to high wear (mode I)
			ductile	relevant	feed small: very low	high wear likely (mode I)
	large	low	brittle	relevant	feed high: low	low wear (mode III)
			ductile	relevant	feed high: low	low to high wear (mode I or III)
fractured	large blocks/ joint spacing	high	brittle	relevant	low	tool breakage high, high wear likely
			ductile	relevant	low	tool breakage high, high wear likely
		low	brittle	may be relevant	medium	low to normal breakage and wear
			ductile	may be relevant	medium	low to normal breakage and wear
	small blocks/ joint spacing	high	brittle	not relevant	high	low breakage and wear
			ductile	not relevant	high	low breakage and wear
		low	brittle	not relevant	high	low breakage and wear
			ductile	not relevant	high	low breakage and wear

rock mass by drilling and blasting methods (see Bray et al. 1997) or mechanical excavation. The basic information to decide on this is the rock mass structure and the strength of intact rock. Figure 8.1 shows the principles involved. If the rock mass is well described, by giving information on the network of discontinuities, the size and shape of the rock blocks and the properties of the rock materials that built up the rock mass, the contractor can decide what method to use.

As discussed in this work, the rock mass classification systems, such as RMR (Figure 8.8), can give a rough idea on the likeliness that mechanical excavation is possible.

Based on the observations on trencher performance, as outlined in Chapter 17 and summarized in Table 17.13, the scheme of Figure 23.1 is given. In this work we have not been able to prove beyond doubt the importance of the processes mentioned in Figure 23.1 and Table 23.1. The role of ductile rock in wear has not been completely clarified.

When examining the two flow charts of Figure 23.1, the boundaries between massive and fractured rock, high and low strength of rock material and high/low production are relative, because these depend on the type of rock excavation machine (dredger) involved. The scheme intends to point to the type of information that needs to be gathered:

Rock mass properties. Most relevant are the discontinuity patterns and estimations of block sizes (see Chapter 8 & 22). The cutting forces will also relate to the inter block shear strength.

Rock material properties. With regard to rock cutting, the *rock type* is important. Carbonate rocks differ in behaviour from silicate rocks (Chapter 17). The *strength* of the rocks is very important. It has been shown that both the UCS test and BTS test (or PLS test) have to be performed on the samples. Not only to be able to correlate the simpler Point Load or Brazilian split tests to the standard UCS, but also because the ratio UCS/BTS relates to the *ductility* of rock during failure and cutting (Chapter 9 & Appendix D). It is thought that more extensive rock mechanical testing would benefit the understanding of the mechanical behaviour of the rock. Carrying out UCS tests, with information on the deformation behaviour (cyclic loading, determination of deformation modulus) and registration of the complete failure curve, gives an indication of the work of destruction. Triaxial tests are helpful to determine the brittle-ductile transition stress and give a determination of the *m*-value. Low *m*-values also point to ductility of the rock (Appendix D). Unfortunately the data gathered up to now are insufficient to really prove this statement (Table 17.4). *Cutting tests* are considered less relevant for the dredging application. Calibration with practice is cumbersome.[77] *Petrography* becomes relevant when real rock cutting takes place (Table 23.1). The *abrasiveness* of the rock can be assessed by the combination of rock strength, mineral hardness and grain size (Chapter 10-13). When rock scraping occurs, or when ductile cutting occurs in relatively stronger rocks, the tool-rock contact surface can reach high temperatures due to frictional heating, the *adhesive wear* mechanism operates and the steel or carbide is much weakened, such that even soft rock as limestones become abrasive. Limestones can, sometimes unexpectedly, contain quartz. Also for this reason petrographic examination is really needed (Chapter 5). In fractured rock, when ripping occurs, the relevance of abrasiveness is lower, depending of the amount of cutting or scraping that still occurs.

In reality the characterisation of the groundmass to be dredged can be complicated by many geological factors. The mass can be heterogeneous, consisting of layers of highly different nature. It can be attempted in that case to distinguish units of rock or soil that have rock properties within a certain geotechnical range. In many cases rocks are weathered or weathering zones are present. Often clay is formed, which can give a ductile component to the mix of material dredged. This is known to influence the amount and types of wear occurring (Chapter 5 & 22.5).

Concluding, to make better assessment of potential wear, the site investigation of rock that is considered to be dredged should not be confined to the execution of Point Load tests and Unconfined Compression tests, but also triaxial tests are advised to be carried out, and petrographic examination is needed as well. The variation in

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[77] For rock drilling purposes this is different. In this case the bits that are used in a laboratory cutting test are similar to the ones used in the real drilling tool (same scale).

the groundmass needs to be well documented and a sound geological model of the subsurface is needed.

23.2 MONITORING OF EXCAVATION AND TOOL CONSUMPTION DURING DREDGING

In the ideal situation, the contractor has at his disposal a model of the ground to be dredged. In this model, the distribution of different layers with defined properties are given. In recent years many projects have already been executed with ground models presented in computer aided graph format and supported by geostatistics programs (Chapter 17.3).

To improve the present situation in prediction of production and tool consumption, it is strongly advised to develop performance monitoring programmes during rock dredging projects. These should aim at checking the geotechnical model of the subsurface during the dredging operation and, by regular sampling of the rock dredged, control the basic rock parameters. It is important to check regularly the wear patterns on the dredging teeth or pick point surfaces and advisable is to have the tooth steel examined microscopically, to derive an estimate of the tool surface temperature (Chapter 9.4). The data base thus built up for each cutter suction dredger in different projects, can be used in models that will predict the performance in future projects.

This can be done in the way that is used for the rock trencher projects studied in this project (Chapter 17, Table 17.4 & 5). The modelling could be done by using expert systems and fuzzy systems. This approach appears promising for this purpose (see Chapter 18.5). Once developed, these systems may be used on board of the CSD vessels during an operation and give direct information to the operator.

Probably in the near future the three-dimensional geotechnical model, with built-in fuzzy expert models that describe production and wear, can be compared directly on board of the vessel with data on specific energy. In this way the information needed on the tribological wear system is generated and can be used to make much more reliable predictions about production and tool consumption for the remainder of the dredging contract.

CHAPTER 24

Closing remarks and recommendations

This research, which has taken part of the time of the author over a period of more than ten years, has been rewarding in the sense that insight has been gained in what turned out to be a complex problem. It is hoped that this work can be of assistance to the dredging contractors and consulting engineers involved in rock dredging contracts. Being aware of the geological factors involved in the mechanical excavation of rock is the most important notion that can benefit and improve the quality of site investigation reports.

Many aspects have not been resolved that certainly can be clarified in future research. For example, the role of ductility of rock in cutting and wear could be studied in a laboratory testing program involving a selection of rock types, a triaxial testing program and a series of cutting experiments. This would be a major research program on its own.

A task of the Engineering Geologist is to make structure and composition of the groundmass to be dredged transparent to the dredging engineers and provide useful geotechnical data. Three dimensional imaging of the shallow subsurface is important in this respect, and is improving continuously these years, through the improvement of geophysical sounding tools and the rapid development of data processing and imaging techniques. In Engineering Geology, much effort should be directed to the development of more reliable geotechnical models of soil and rock masses. It is thought that the rock mass classification systems, which have been a major aid since the early 1970's in rock mass mechanics, should be replaced by models based on expert systems and fuzzy systems, that are able to combine expert opinion, hard data and fuzzy information. Apart from representations of the spatial distribution of geotechnical parameters, the accuracy of the knowledge should also be indicated. This type of systems can only be developed with any reliability, if sufficient data is available. The assemblage of data of production and tool consumption during excavation projects is therefore very important. Such data are only valuable if the rock mass parameters that relate to excavation and tool consumption are gathered at the same time.

APPENDIX A

Rock identification and classification procedure for engineering purposes[77]

A1 INTRODUCTION

In rock engineering practice, engineers are often confronted with the task to estimate or determine the engineering properties of rock. The first step needed is to recognize the type of rock one is dealing with. Often this is not an easy task. It is not without reason that within the science of Geology special branches have developed that study the different types of rock (sedimentary rocks by 'sedimentologists', metamorphic and igneous rocks by 'petrologists'). The classification discussed here was specially developed for the use of engineers and is an evolvement from the classification given in BS 5930:1981, which is also used by PIANC (1984). It is evident that for dredging projects specialists should be consulted concerning rock identification.

The engineering behaviour of rock is recognized to be intimately related to its constitution, which is reflected in its name[78]. The classification procedure presented here aids persons with limited geological knowledge to allocate a rock name to a hand specimen of rock or a fresh outcrop surface. Several flowcharts are presented which follow a logical path and are easy to use. The names of the most common rock types are given in Table A1. In Section A2 terms and definitions of mineral and rock descriptors are listed. The user is expected to take notice of these terms and definitions before starting to classify. In section A3 the classification procedure is explained (Flowchart A, p.275).

Before starting to classify a rock it is necessary to identify the rock-forming minerals. Descriptions of minerals, of which there are many hundreds, can be found in any mineralogy book, but fortunately the list of the most common rock-forming minerals is rather short, about 10-20. The common rock-forming minerals belong to the groups of the *silicates, carbonates and the evaporites (salt, sulphates and chlorides*). Some silicate minerals are for example quartz, feldspar (alkali feldspar and

[77] By Mario Alvarez Grima and Peter N.W. Verhoef. An Expert System of this classification is available on request.

[78] It should be appreciated that the rock classification presented here is not intended to group rocks with similar engineering properties. Its main objective is to group rocks of similar origin. However, a rock name with a short description of the origin, texture and structure of particles and minerals, can indicate much that is of practical value for engineering geological purposes.

Table A1 Aid to identificatic n of rock for engineering purposes (after BS 5930:1981).

GRAIN SIZE (mm)	Bedded rocks SEDIMENTARY: clastic	rocks with mixed composition (Clark and Walker 1977)**	calcareous	CLASTIC LIMESTONE	CRYSTALLINE LIMESTONE & Dolomite	EVAPORITES	COAL & LIGNITE	Foliated rocks METAMORPHIC	Sometimes massive	Massive and crystalline rocks IGNEOUS				GRAIN SIZE (mm)
Coarse 2	RUDACEOUS: CONGLOMERATE (rounded particles in a finer matrix); BRECCIA (angular particles in a finer matrix)		calcirudite	conglomerate limestone		Halite, gypsum, anhydrite		GNEISS Well developed but often widely spaced foliation sometimes with schistose bands; MIGMATITE mixture of gneiss and veins of igneous rock		GRANITE (1,2)	DIORITE (1,2)	GABBRO (1,3)	PERIDOTITE	Coarse 2
0.6 – Medium 0.2 – 0.06	ARENACEOUS: SANDSTONES (Quartz) ARENITE Quartz grains and siliceous cement; (Quartz) ARKOSE Many feldspar grains usually with some mica; (Quartz)(grey) WACKE Many rock chips; TUFF (CEMENTED VOLCANIC ASH)		calcarenite	detrital limestone				SCHIST Well developed foliation generally much mica	METAMORPHOSED LIMESTONE (Marble); SERPENTINITE; AMPHIBOLITE; QUARTZITE; HORNFELLS	MICRO-GRANITE (1,2) (Porphyry)*	MICRO-DIORITE (1,2) (Porphyry)*	DOLERITE (3) (Porphyry)*		Medium 0.06
Fine 0.002	ARGILLACEOUS: SILTSTONE Mostly silt; CLAYSTONE Mostly clay (massive texture); SHALE (fissile texture)		calcilutite calcisiltite	fine grained limestone				PHYLLITE Closely spaced foliation, mica luster, but crystals not visible with handlens; SLATE narrow spaced well developed plane of foliation, (mica is absent)		RHYOLITE (3,4)	ANDESITE (3,4)	BASALT (3,4)		Fine 0.002
Amorphous	Flint, Chert		weak ROCK	strong ROCK				Mylonite		Obsidian				Amorphous
	CLASTIC: SILICEOUS		CALCAREOUS		CRYSTALLINE		Organic: Carbon-aceous	Foliated rocks may be layered or banded. This banding may be undulating or contorted, except in the case of slates where it may be exactly parallel. Metamorphic rocks often split easily along foliation planes.		CRYSTALLINE: Light ⟸ COLOUR ⟹ Dark; Quartz rich … Quartz poor				

GENERAL NOTES: Bedding in sedimentary rocks may not, because of its spacing, be seen in hand specimen but only in outcrop. Fossils may be found in sedimentary rocks. The mineral calcite, in calcareous rocks, may be scratched with a knife, and will react with dilute hydrochloric acid. Quartz scratches steel. Broken crystals in crystalline rocks reflect light.

** siliceous and calcareous components are present (e.g. siliceous fine grained limestone)

MODE OF OCCURRENCE OF IGNEOUS ROCKS: 1. Batholiths 2. Stocks 3. Sills and dykes 4. Lava flows

* Porphyries are rocks in which some mineral grains are very much larger than the surrounding matrix. All igneous rocks can be "porphyritic".

plagioclase), mica (biotite and muscovite), chlorite, amphibole (e.g. hornblende), pyroxene (e.g. augite), and olivine. Calcite and dolomite belong to the carbonate mineral group.
The minerals are identified and classified through the use of Flowchart B (p.276) After identification of the minerals, the rock identification can be carried out (Flowchart C, D, and E). Finally the rock name is retrieved. The rock name is based on the rock classification for engineering purposes, Table A1.

It is important to point out here that the rocks that are most difficult to identify are those without visible grains or minerals. For example, rocks composed uniformly of very fine grains like basalts, shales, some slates, and some fine-grained limestones and dolomites can provide difficulties when the hardness and structure are overlooked. Associated rocks and structures which can be studied in the field usually make rock identification easier.

A2 TERMS AND DEFINITIONS

There are common terms frequently used in any rock identification procedure. These terms are related to minerals and rocks respectively. It is necessary to become familiar with these terms before starting to classify any type of rock.

A2.1 *Minerals*

A mineral can be defined as a natural inorganic solid substance having a particular chemical composition or range of composition and a regular atomic structure to which its crystalline form is related.

A2.1.1 *Physical properties of the minerals*

Properties included in the classification are: colour, lustre, crystal form, hardness, cleavage, fracture, strength, and specific gravity. It is not necessary to know all these properties to identify a mineral; two or three of them taken together may be enough. In a few cases the taste (for example rock - salt) and touch (for example talc, and serpentine, feels soapy) give useful indications.

A2.1.1.1 *Hardness*

Hardness is measured relative to a standard scale of ten minerals, known as Mohs' Scale of Hardness (Table A3, A4). These minerals are chosen so that their hardness increases from 1 to 10. Hardness is tested by attempting to scratch the minerals of the scale with the sample under examination. Hardness should be tested on a crystal face or a cleavage plane. For rapid identification relative hardness can be determined as shown in flowchart B (p.276). For example talc, gypsum, chlorite, and serpentine can be scratched by the fingernail, a steel knife will scratch calcite, dolomite, biotite and

Table A2 Aid to identification of rock for engineering purposes: Mixed calcareous-siliceous rocks.

Degree of INDURATION	Approximate Unconfined Compressive Strength (MPa)	ADDITIONAL DESCRIPTIVE TERMS BASED ON ORIGIN OF CONSTITUENT PARTICLES: NOT DISCERNIBLE		BIOCLASTIC (organic) / OOLITE (inorganic)	SHELL (organic) / CORAL (organic) / ALGAL (organic) / PISOLITES (inorganic)	TOTAL CARBONATE CONTENT % (constituent particles plus matrix)
		INCREASING GRAIN SIZE OF PARTICULATE DEPOSITS →				
		< 0.002	0.002 – 0.060	0.060 – 2	2 – 60 mm	
Non-Indurated	Very soft to hard, < 0.036 to > 0.3 MPa	CARBONATE MUD	CARBONATE SILT	CARBONATE SAND	CARBONATE GRAVEL	90
		clayey CARBONATE MUD	siliceous CARBONATE SILT (1)	siliceous CARBONATE SAND (1)	mixed carbonate and non-carbonate GRAVEL (2)	50
		calcareous CLAY	calcareous SILT (1)	calcareous silica SAND (1)		10
		CLAY	SILT	SAND	GRAVEL	
Slightly indurated	Hard to moderately weak, 0.3 - 12.5 MPa	CALCILUTITE	CALCISILTITE	CALCARENITE	CALCIRUDITE (calc. congl. or breccia)	90
		clayey CALCILUTITE	siliceous CALCISILTITE	siliceous CALCARENITE	conglomeratic CALCIRUDITE (2)	50
		calcareous CLAYSTONE	calcareous SILTSTONE	calcareous SANDSTONE	calcareous CONGLOMERATE	10
		CLAYSTONE	SILTSTONE	SANDSTONE	CONGLOMERATE OR BRECCIA	
Moderately indurated	Moderately strong to strong, 12.5 - 100 MPa	fine grained LIMESTONE		detrital LIMESTONE	CONGLOMERATE LIMESTONE	90
		fine grained argillaceous LIMESTONE	fine grained siliceous LIMESTONE	siliceous detrital LIMESTONE	conglomeratic LIMESTONE (2)	50
		calcareous CLAYSTONE	calcareous SILTSTONE	calcareous SANDSTONE	calcareous CONGLOMERATE	10
		CLAYSTONE	SILTSTONE	SANDSTONE	CONGLOMERATE OR BRECCIA	
Highly Ind.	Strong to extremely strong	CRYSTALLINE LIMESTONE OR MARBLE (tends towards uniformity of grainsize and loss of original texture)				50
		conventional metamorphic nomenclature applies in this section				

(1) Non-carbonate constituents are likely to be siliceous apart from local concentrations of minerals such as feldpar and mixed heavy minerals.
(2) In descriptions the rough proportions of carbonate and non-carbonate constituents should be quoted and details of both the particle minerals and matrix minerals should be included. (3) Calcareous is suggested as a general term to indicate the presence of unidentified carbonate. Where applicable, when mineral identification is possible, calcareous is referring to calcite or use alternative adjectives such as dolomitic, aragonitic, sideritic. (After Clark & Walker 1977)

muscovite (Mohs' hardness 3) and apatite (Mohs' hardness 5) but not feldspars (Mohs' hardness 6) and quartz (Mohs' hardness 7). The hardness test is very easy, it is straightforward and useful to distinguish the hardness of common minerals like quartz and calcite.

A2.1.1.2 *Cleavage*

Several minerals have a tendency to split easily in certain regular directions (one, two or three directions), and yield smooth failure planes called cleavage planes. These directions depend on the arrangement of the atoms of the mineral, and are parallel to definite crystal faces. Terms used frequently to describe the cleavage are: perfect, good, distinct, and weak.

Table A3 Mohs' Scale of Hardness

Hardness	Mineral	Can be scratched by
1	Talc	fingernail
2	Gypsum	fingernail
3	Calcite	brass pinpoint
4	Fluorspar	
5	Apatite	steel
6	Feldspar	
7	Quartz	
8	Topaz	
9	Corundum	
10	Diamond	

A2.1.1.3 *Lustre*

Lustre is the appearance of a mineral surface in reflected light. It may be described as metallic, as in pyrite or galena; glassy or vitreous, as in quartz; resinous or greasy, as in opal; pearly, as in talc; or silky, as in fibrous minerals such as asbestos, serpentine. Minerals without lustre are described as dull.

A2.2 *Rocks*

Rock[79] is defined as material of the earth's crust, composed of one or more minerals strongly bound together that are so little altered by weathering that the fabric and the majority of the parent minerals are still present. The classification of rock is based on the examination of intact rock material (intact rock refers to the rock material on the

[79] In this definition we follow the engineering usage. Geologists tend to include soils into the classification of rock. Here loose or cohesive aggregates of minerals are excluded. Only cemented and or solidified crystalline mineral aggregates are considered as rock.

scale of a hand specimen), excluding fractures or other discontinuities like joints, bedding planes in sedimentary rocks or foliation planes in metamorphic rocks.

A2.2.1 *Rock types*

Rock types are named in relation to a defined mineralogical composition, predominant grain size, matrix composition, texture and genetic origin.

A2.2.1.1 *Sedimentary rock*

Sedimentary rocks are formed from sediments which have been transported and deposited, sometimes as chemical precipitates, or from the remains of plants and animals which have been lithified under the heat and pressure of overlying sediments or by chemical reactions. Characteristic is the bedded nature of most sedimentary rocks.

A2.2.1.2 *Metamorphic rock*

Metamorphic rocks are formed from other rocks by deformation due to tectonic processes which cause solid-state plastic flow, in combination with heat and water, or by the heat of molten rock injected into adjoining rock, which causes changes and produces new minerals. Characteristic for many metamorphic rocks is the penetrative foliation that develops due to deformation (in rocks like slate, phyllite, schist and gneiss).

A2.2.1.3 *Igneous rock*

Igneous rocks are formed by crystallization of masses of molten rock (magma) originating from below the earth's surface (the earth's interior).

A2.2.2 *Texture*

The texture or fabric of the rock refers to the size and shape of individual minerals (crystal or grains) or rock particles in the rock. The major division is between *crystalline* and *clastic* texture. Many geologists use the term texture interchangeable with fabric or microstructure (see A.2.2.3).

A2.2.2.1 *Crystalline texture*

Crystalline rocks have an interlocking fabric of minerals. Examples: granite, diorite, basalt, marble and some limestones (Photo 1 and 2).

A2.2.2.2 *Clastic texture*

Clastic rocks consist of a collection of minerals and rock particles bound together by cement (e.g. carbonate, silicate, Fe-(hydro) oxides). These rocks are mainly composed of particles of pre-existing rocks, minerals or organic remains and of particles newly formed during weathering and erosion such as clays (examples: claystones, siltstones, sandstones, conglomerates, breccias) (Photo 3, 4, 5 & 6).

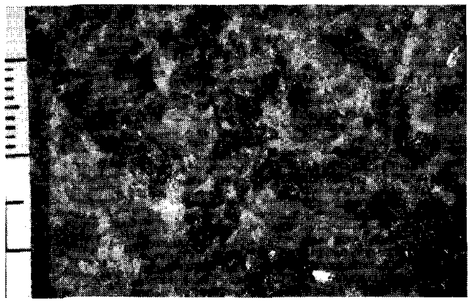


Photo 1 Granite

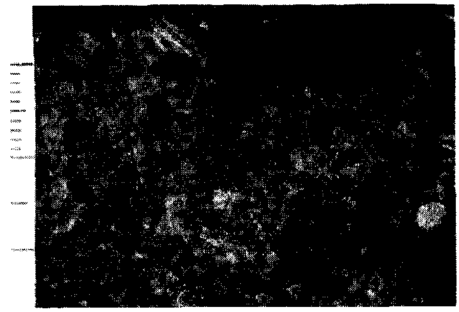


Photo 2 Gabbro

Photo 3 Conglomerate

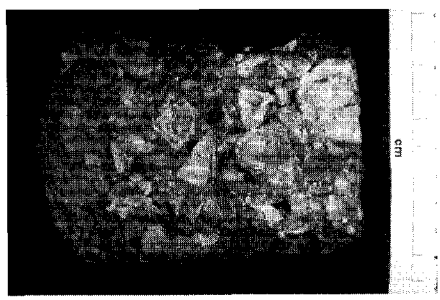


Photo 4 Breccia

A2.2.3 *Microstructure or fabric*

Microstructure or fabric refers specifically to the arrangement of the constituent minerals or rock particles. In sedimentary rocks, microstructure is the orientation in space (or lack of it) of the discrete particles, minerals, and cement comprising the rock. The term is used in igneous and other crystalline rocks and in metamorphic rocks for the pattern produced by the shapes and orientation of the crystalline and non-crystalline parts of the rock. It depends on the relative sizes and shapes of the grains or crystals and their orientation with respect to one another and to the matrix.

A2.2.3.1 *Isotropic structure*

The structure of the rock is isotropic if the mineral grains have a random orientation and if from an engineering view point the mechanical and physical properties of the rock are similar in all directions (Photo 1 and 2).

Photo 5 Sandstone

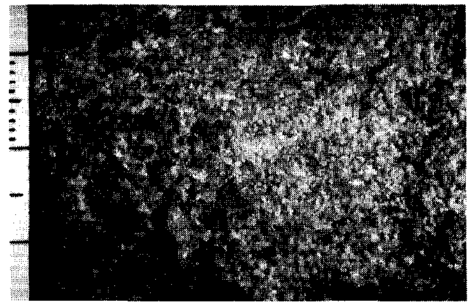


Photo 6 Oolitic Limestone

A2.2.3.2 *Anisotropic structure*

The structure of the rock is anisotropic if the rock has planar or linear elements, like lamination, (cross) bedding in sedimentary rocks, or foliation (penetrative, planar deformation structure in deformed rocks, often called (slaty) cleavage in slates, schistosity in schist and gneissic foliation or-layering in gneiss) and if from an engineering view point the mechanical and physical properties vary with the fabric orientation (Photo 7 and 8).

Photo 7 Gneiss

Photo 8 Rhyolite

A2.2.4. *Grain size*

Grain size refers to the average dimensions of the minerals or rock particles comprising the rock. It is usually sufficient to estimate the grain size by eye, or with the aid of a hand lens. A general way to define the grain size is: coarse (grain size larger than 2 mm), medium (grain size from 0.06 to 2 mm), fine (grain size from 0.002 to 0.06 mm) and very fine or amorphous (smaller than 0.002 mm).

A2.2.5 *Matrix*

The term 'matrix' describes the fine grained, glassy or amorphous groundmass of a rock containing also larger mineral grains or rock particles.

A3 ROCK IDENTIFICATION AND CLASSIFICATION PROCEDURE FOR HAND SPECIMENS

In this Section the rock classification procedure for a hand specimen is explained in detail.

A3.1 *To give a rock a name*

For this task the necessary tools, the important factors, and the flowcharts needed to identify minerals and rocks are given.

A3.1.1 *Identification tools*

The necessary identification tools are:

- A knife blade
- Diluted hydrochloric acid (5 %)
- A hand lens (8-10 x)
- A geological hammer
- Flowchart A: Basic rock classification chart
- Flowcharts B: Mineral identification, common rock-forming minerals
- Flowchart C: Aid to identification of common isotropic crystalline rocks
- Flowchart D: Aid to identification of common anisotropic crystalline rocks
- Flowchart E: Aid to identification of common clastic rocks
- Table A1 & A2 Aid to identification of rocks for engineering purposes
- Table A3 Mohs' scale of hardness
- Table A4 Characteristics of some common rock-forming minerals
- Table A5 Terms to describe weathered state of rock material
- Table A6 Terminology for the description of strength

A3.1.2 *Important factors in identification of minerals and rocks*

The factors important in identification of minerals and rocks are:

- Mineralogical composition
- Texture of the rock (Crystalline or Clastic)
- Structure (fabric) of the rock (Isotropic or Anisotropic)
- Hardness of the minerals or of the matrix
- Grain size of rock particles and minerals
- Reaction of the minerals when treated with cold diluted hydrochloric acid (5%)

A3.1.3 *Identification of common rock forming-minerals in hand specimen*

Flowchart B and Table A4 will assist to identify the common rock forming-minerals. Commonly it is necessary to observe the rock using a hand lens, since minerals forming the rock fabric have often a size in the order of the resolution of the human eye. Flowchart B presents the three physical properties of the minerals considered for identification: (*hardness, cleavage and colour*).

First the hardness of the mineral(s) is determined. Rocks are divided into those containing minerals which can be scratched by a fingernail, those containing minerals which can be scratched by a knife blade and those containing minerals which cannot be scratched by a knife blade. On Mohs' scale of relative hardness, Table A3, a finger nail usually has a hardness between 2 and 2.5, while the average knife blade has a hardness between 5 and 5.5. More information about physical properties of individual minerals is given in Table A4.

The second division is based on mineral cleavage. The presence or absence of cleavage is one of the diagnostic features of minerals. Cleavage surfaces are smooth and uniform and reflect incident light uniformly at one orientation. When the size of the mineral is very small (less than about 1 mm), as in some fine grained crystalline rock (e.g. basalt, rhyolite and andesite), it is difficult to note the cleavage even with the aid of a hand lens. In this case other methods, like microscopic examination of thin sections is suggested. The third division is based on the colour of the mineral, (Flowchart B).

Example 1: As illustration of how the flowchart B works, you can compare calcite, feldspar and quartz, three common minerals that are frequently confused when they are present in a rock specimen. Quartz has no cleavage and will not be scratched by a knife blade, but will scratch the blade itself. Quartz does not react with diluted hydrochloric acid (5%). Feldspar is harder than a knife blade, although sometimes it can be scratched. It presents two good directions of cleavage. Calcite has three good cleavage directions, and can be scratched by a knife blade. It also shows vigorous effervesce when treated with diluted hydrochloric acid. Moreover, calcite presents rhombohedral angles between the cleavage surfaces (75° and 105°), whereas feldspar cleavage faces have approximately 90° angles between them.

A3.1.4 *Identification of common rocks in hand specimens*

In Table A1 the names of common rock types found in engineering projects are present. Using this table the first step is to decide whether the rock is igneous, sedimentary or metamorphic. For someone with limited geological knowledge using Table A1 is rather difficult. Therefore to facilitate the rock classification several flowcharts were created (Flowcharts B, C, D and E).

Four of the many attributes presented by a rock specimen are singled out dominantly in this classification: *texture, structure, hardness and grain size*. The first division given is between crystalline and clastic texture (Flowcharts C, D and E).

The crystalline rocks like granite present an interlocking structure of minerals with very little or no pore space. There may be grain boundary cracks and other fissures that can weaken the rock, and minerals themselves may be deformable (e.g. calcite in marble) but the structure is generally a strong one.

Contrary, the clastic rocks (Flowchart E) consist of a collection of minerals and rocks particles with pore space filled with cement. Some clastic rocks are so fine grained that the grain particles or minerals cannot be seen with the naked eye (e.g.

claystone or calcilutite). In this case the determination of the matrix composition using diluted hydrochloric acid (calcareous or siliceous), and the hardness test are very useful indicators to give the rock a proper name.

In crystalline rocks (Flowcharts C and D), the second classification index takes into account the structure or fabric of the rock specimen. Is the structure of the rock isotropic or anisotropic? For example metamorphic rocks such as slate, schist, phyllite, gneiss, etc, have an incipient parting tendency parallel to the foliation, consequently these rocks present a distinct anisotropy (i.e. directionality) of all physical and mechanical properties. In other rocks the structure is massive on the hand specimen scale (e.g. granite, gabbro, peridotite, etc.), so that the specimen appears to be isotropic.

The third division is based upon the grain size, this attribute is useful for all types of rocks. To illustrate how it works, some individual rock groups will now be considered (Flowchart C). The hard isotropic crystalline rocks (igneous rock) present three series depending on the relative sizes of the minerals: the coarse grained varieties are of plutonic igneous origin (e.g. granites, gabbros). Those with relatively coarse minerals embedded in a medium fine grained matrix ("porphyres") are intrusive in origin (e.g. dolerites) and usually of dyke origin, having cooled at moderate depth. Rocks that are uniformly fine grained, or porphyritic with a fine grained ground mass and commonly porous are extrusive (lava flows, etc.,). To illustrate how the flowcharts B, C and Table A4 work, the following example is given:

A3.2 *Assessment the engineering properties of the rocks*

Example 2: Suppose that you have a rock sample in your hand and you have found that the texture of the rock is crystalline, it has an isotropic structure, which means a random orientation of the minerals, also the minerals can be scratched by a knife blade. You applied some drops of cold diluted hydrochloric acid (5%) noting that the minerals react. At this moment you are able to distinguish the probable genetic origin of the rock. In this example the rock specimen is either a sedimentary or a metamorphic rock; (Flowchart C). Furthermore, with the aid of the flowchart B and Table A4, you can find the common mineral (s) and finally give the rock a name. In this case the rock specimen can be either a limestone, a dolomitic limestone or a marble.

Once the rock name is established, other aspects such as *weathering* and *strength* of the intact material should be considered to complete the classification. The description of the weathering and intact material strength is of particular importance to assess the engineering properties of the rock. It should be appreciated that the rock mass characteristics such as the pattern of fracturing and jointing ("discontinuities") and the distribution of weathering intensity are not included in this classification. It is advised to follow the proceedings of the Code of practice for site investigation (BS 5930:1981) for this; see Appendix B.

A3.2.1 *Weathering* (BS 5930:1981)
Weathering results in an alteration of the original rock. There are two important aspects of weathering: one dominated by mechanical disintegration and the other by chemical decomposition, including solution. Generally, mechanical and chemical effects act together and depending on climate regimes a certain weathering process prevails.

Mechanical weathering results in the opening of microcracks, the formation of new microcracks by rock fracture, the opening of grain boundaries, and the fracture or cleavage of individual mineral grains. Chemical weathering leads to the eventual decomposition of silicate minerals to clay or other secondary minerals. Some minerals, notably quartz, resist this action and may remain unchanged. It should be pointed out that early stages of chemical weathering result in discolouration of the affected rock. Solution is an aspect of chemical weathering which is particularly important to carbonate minerals, rock salt, gypsum and anhydrite.

On the basis of these changes, a descriptive scheme for weathering of rock material is presented in Table A5. This table does not present a weathering scale. The stages of weathering described in Table A5 may be subdivided by using qualifying terms, for example, partially discoloured, wholly discoloured and slightly discoloured, which will aid the description of the material being examined. These descriptive qualifying terms may be quantified if necessary.

A3.2.2 *Strength*
The strength of the intact material refers to the maximum stress level which can be carried by a specimen in a specific test. The strength of the rock material can be subdivided, based on for example the unconfined compression test, the point load test or by geological hammer blows, as presented in Table A6.

Table A4 Characteristics of some common minerals

Mineral	Colour	Mohs' Hardness	Specific gravity	Characteristics
Alkali feldspar $KalSi_3O_8$	pink, white, green	6 to 6.5	2.5	cleavage 2 directions at 90°
Anhydrite $CaSO_4$	white brown, grey	3	3	cleavage
(Hornblende) Amphibole Na,Ca,Mg,Fe,Al silicate	dark green, black, brown	6	3 to 3.5	cleavage, 2 at 60° and 120°
Biotite $K_2(MgFe)_6(SiAl)_8O_{20}(OH)_4$	dark brown, dark green	2.5 to 3	3 to 3.5	perfect cleavage, thin sheets
Calcite $CaCO_3$	white	3	2.7	perfect cleavage 3 directions approx. 75°, react with cold diluted acid (HCL, 5%)
Dolomite $CaMg(CO_3)_2$	white or pink, light brown	3.5 to 4	2.8	cleavage as in calcite Effervesces in cold HCL only when powdered, effervesces in warm diluted acid (HCL, 5%)
Gypsum $CaSO_42(H_2O)$	white, grey, red	2	2.3	perfect cleavage in three directions.
Halite NaCl	variable	2.5	2.2	saline taste
Muscovite $Kal_2(Si_3Al)O_{10}(OH)_2$	light coloured,	2 to 3	2.8	perfect cleavage one direction, producing thin elastic sheets
Olivine $(Mg,Fe)_2SiO_4$	olive to greyish green, brown	6.5 to 7	3.27 to 4.37	usually in disseminated crystals in basic igneous rocks. May be massive granular
Plagioclase $NaAlSi_3O_8$ to $CaAl_2Si_2O_8$	white, grey or pink	6 to 6.5	2.5	cleavage 2 directions, striations on some cleavage planes
(Augite) Pyroxene Ca,Mg,Fe,Al silicate	dark green, black	6	3.5	cleavage 2 at 90°. Prismatic 8 sided crystals
Quartz SiO_2	grey or white	7	2.65	conchoidal fracture 6-sided crystals, transparent to translucent
Serpentine $Mg_3Si_2O_5(OH)_4$	olive to blackish-green,yellow-green	2 to 5	2.2 to 2.65	massive. Fibrous in the asbestos variety, chrysotile. Frequently mottled green in the massive variety

Table A5 Terms to describe weathering of rock material (terms according to BS 5390:1981)

Term	Description
Fresh	No visible sign of weathering of the rock material
Discoloured	The colour of the original fresh rock material is changed. The degree of change from the original colour should be indicated. If the colour change is confined to particular mineral constituents this should be mentioned.
Decomposed	The rock is weathered to the condition of a soil in which the original material fabric is still intact, but some or all of the minerals grains are decomposed
Disintegrated	The rock is weathered to the condition of a soil in which the original material fabric is still intact. The rock is friable but the mineral grains are not decomposed

Table A6 Terminology for the description of strength (After Hoek et al. 1995)

Term	UCS MPa	PLS MPa	Field estimate of strength	Example
Very weak	< 1.25	< 0.05	indented by thumbnail	clay gouge
Weak	1.25 to 5	0.05 to 0.2	material crumbles under firm blows of geological hammer, can be shaped with knife	highly weathered or altered rock
Moderately weak	5 to 12.5	0.2 to 0.5	thin slabs broken by heavy hand pressure	rocksalt, calcarenite
Moderately strong	12.5 to 50	0.5 to 2	hand held specimens broken by light hammer blows	schist, shale, siltstone, claystone, coal, concrete
Strong	50 to 100	2 to 4	hand held specimens broken by heavy hammer blows	limestone, marbles, phyllite, arenite, schist, shale
Very strong	100 to 200	4 to 8	lumps only chip by heavy hammer blows (Dull ringing sound)	basalt, gabbro, gneiss, limestone, schist, shale
Extremely strong	> 200	> 8	rocks ring on hammer blows. sparks fly.	fresh basalt, chert,, gneiss, granite, quartzite

note 1 UCS means Unconfined compressive strength

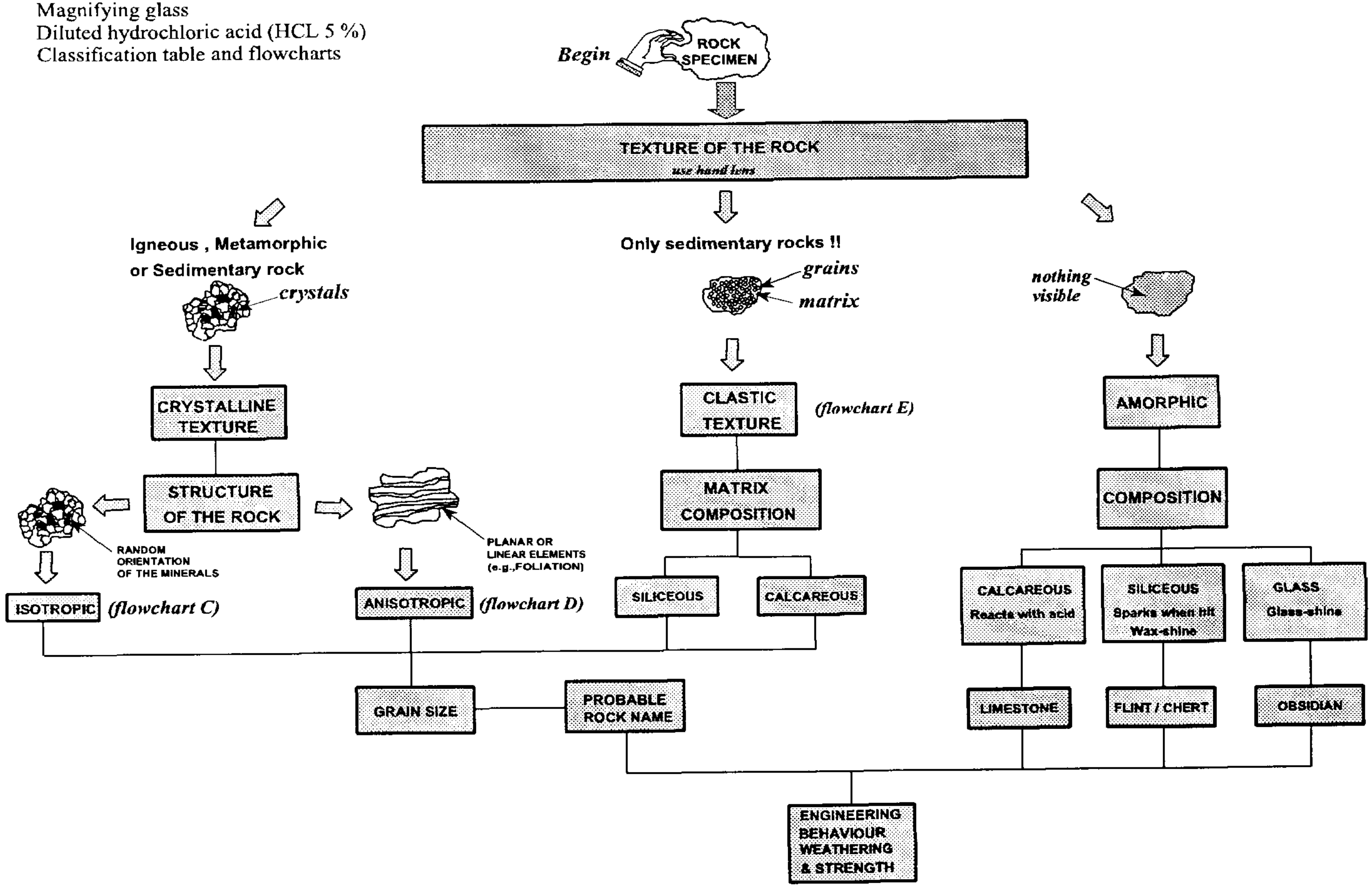
note 2 PLS means Point load strength.

note 3 All rock types exhibit a broad range of UCS value which reflect the heterogeneity in composition and anisotropy structure. Strong rocks are characterized by well interlocked crystal fabric and few voids.

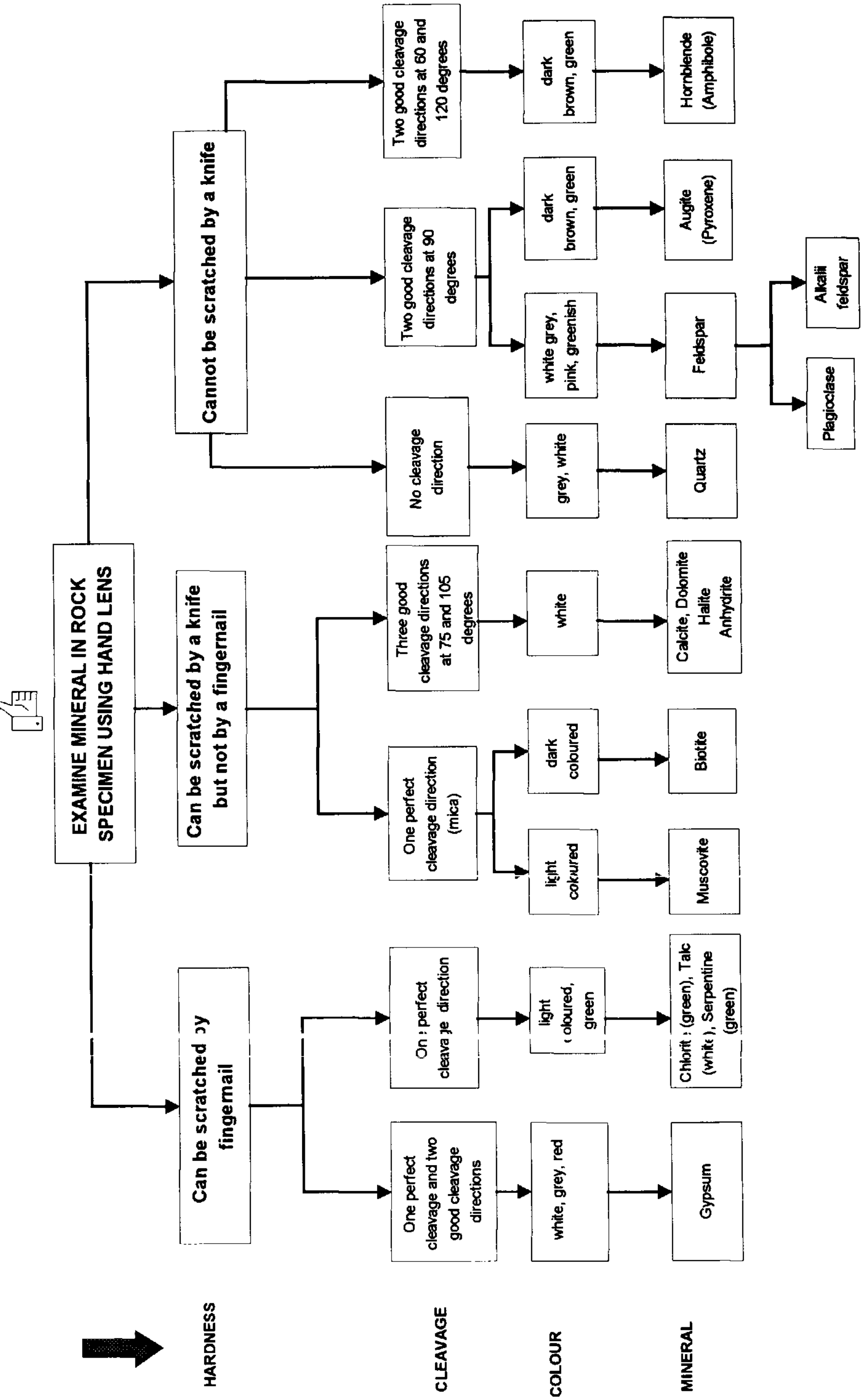
note 4 Terms according to ISRM (1981).

Necessary:
- Hammer
- Magnifying glass
- Diluted hydrochloric acid (HCL 5 %)
- Classification table and flowcharts

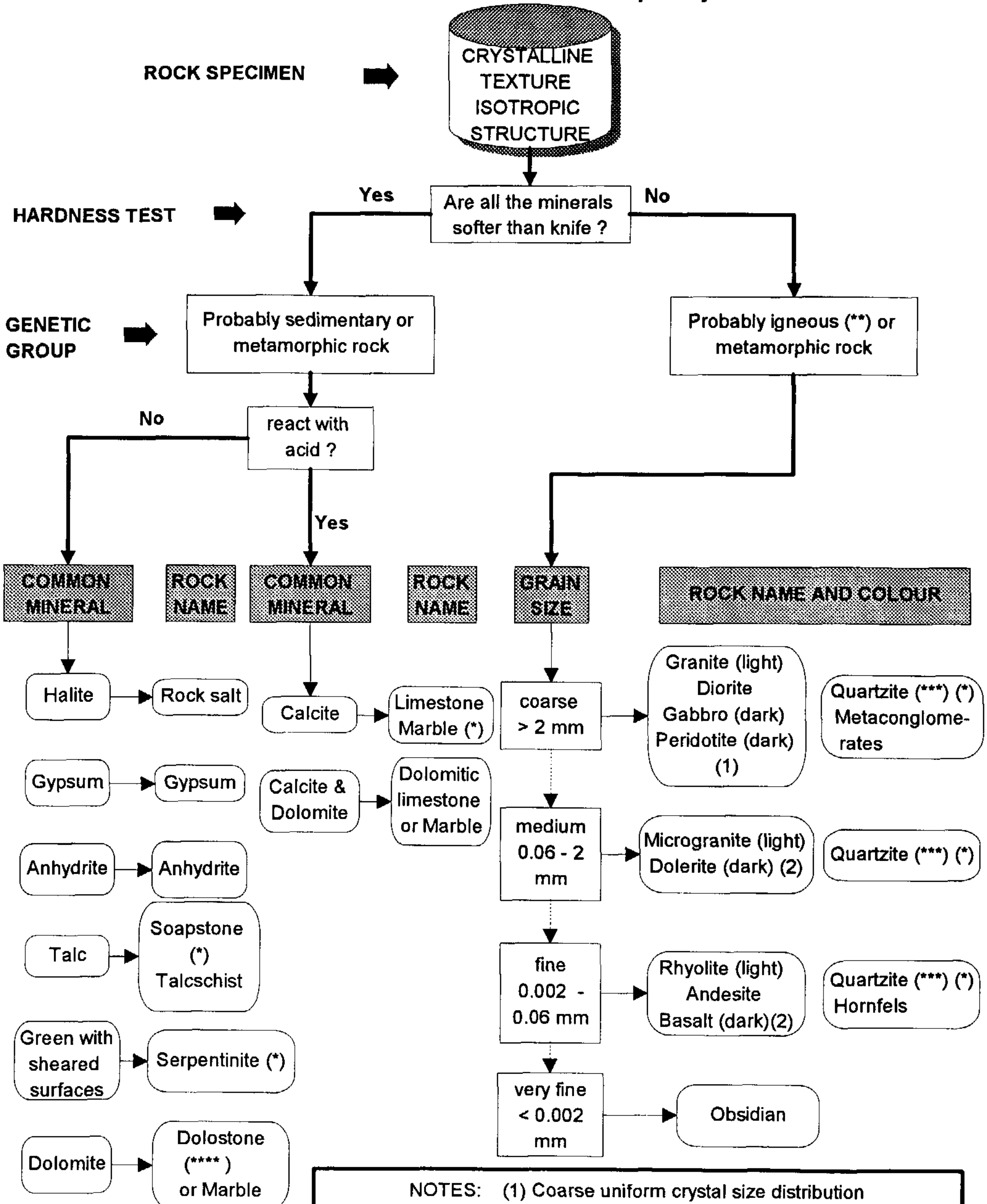
Flowchart A: Basic Rock Classification Chart



Flowchart B: Mineral Identification, Common Rock Forming Minerals

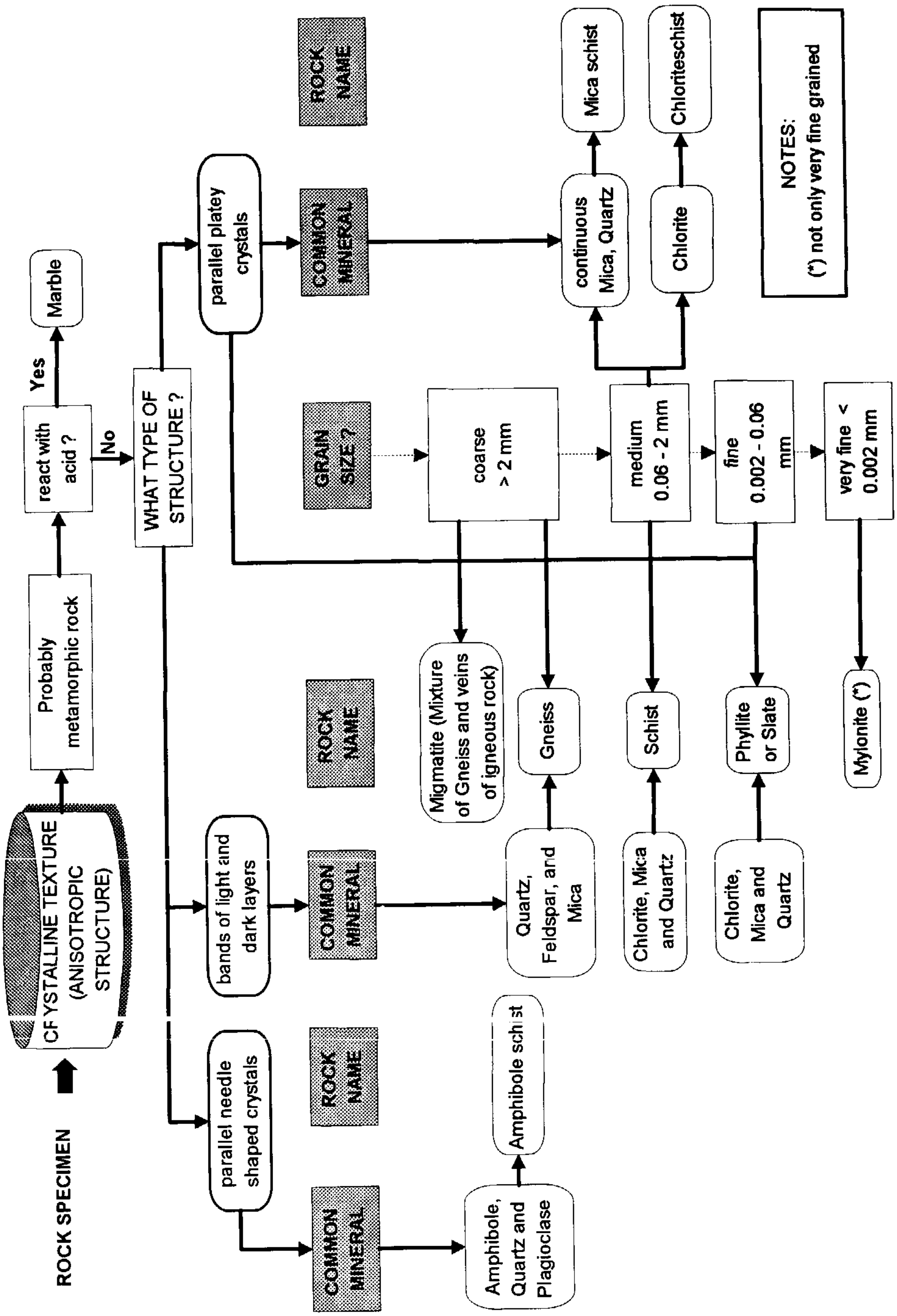


Flowchart C: Aid to identification of common isotropic crystalline rocks

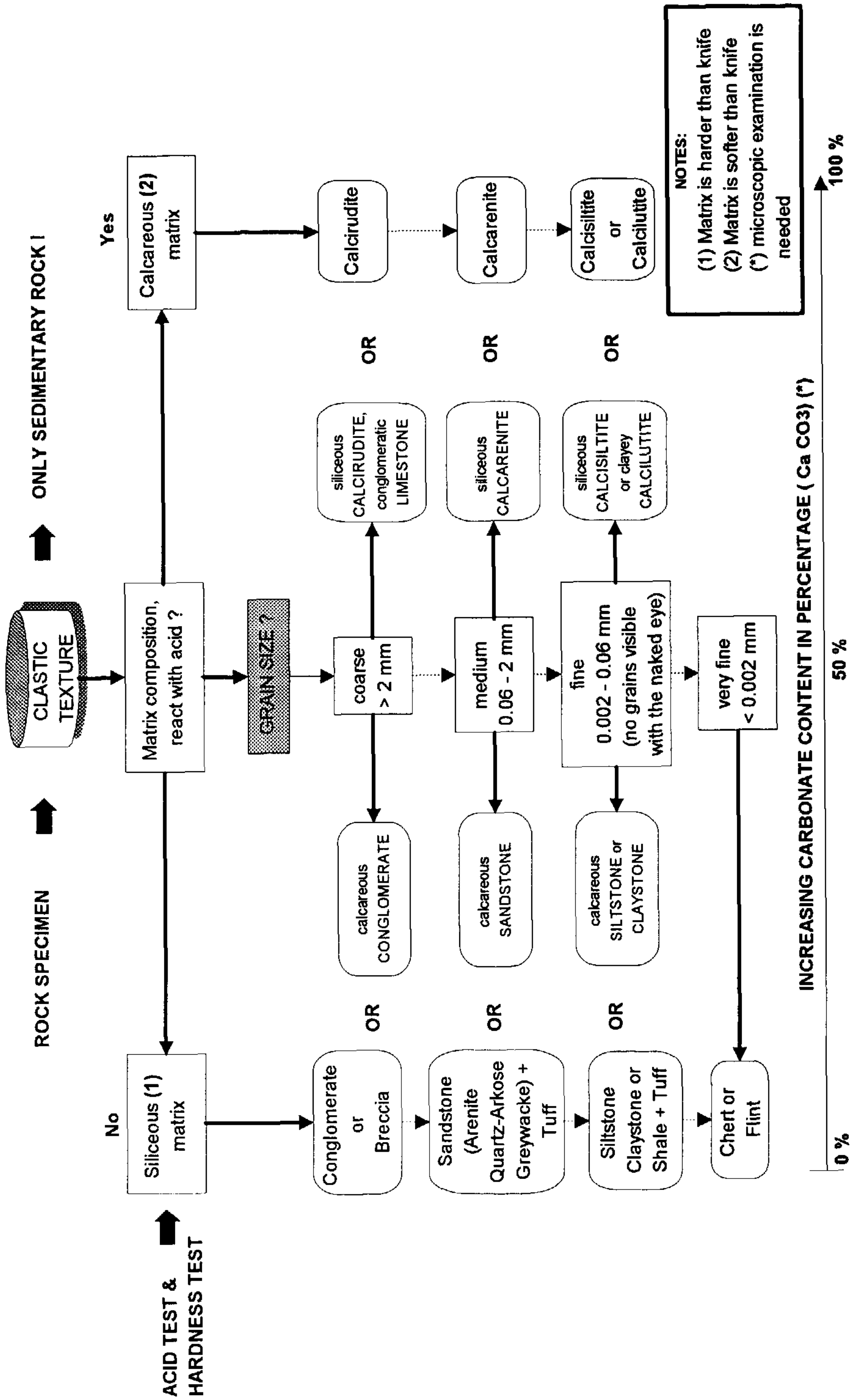


NOTES:
(1) Coarse uniform crystal size distribution
(2) Mixed sizes: Coarse crystals in fine or very fine matrix (porphyre)
(*) May be anisotropic in hand specimen
(**) Softer minerals may be present (e.g. mica)
(***) > 90 % Quartz
(****) 100 % Dolomite no or slight reaction with acid

Flowchart D: Aid to identification of common anisotropic crystalline rocks



Flowchart E: Aid to identification of common clastic rocks



APPENDIX B

Definitions for rock mass description

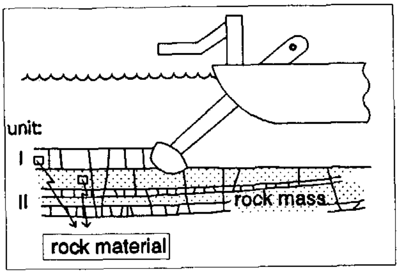
In this Appendix, the definitions used to describe rock masses for engineering purposes are given. Bell (1992, Chapter 3) gives a comprehensive review of the description of rock masses for engineering purposes, which can be consulted if more information on the background of rock description and classification is needed. Another useful document is the *Guide to Rock and Soil descriptions* of the Geotechnical Engineering Office of Hongkong (1994). It is obvious that describing the rock mass in well defined terms supports computerised processing of data. This has been the purpose of this type of description from the onset (Anon. 1977). For each project the most convenient way of representing the data can be developed. But it is advised to adhere as much as possible to internationally accepted standards. The BS 5930:1981 *Code of practice for site investigations* should be consulted for the recommended classification of soils and rocks for engineering purposes. This standard is the basis for the PIANC (1984) document for site investigation for dredging projects.

B1 DESCRIPTION OF ROCK MATERIAL

Rocks are described by their colour, the grain size of the constituent minerals, the texture (grain size and shape of the constituent minerals), fabric (the arrangement of the mineral grains on microscopic scale and structure (arrangement of the fabric elements on mesoscopic scale), the state of weathering, the rock name (given in capitals), the strength and other characteristics and properties, see Figure B1 and Table B1. The rock name may be derived from Table A1 and A2 of Appendix A or by petrographic examination (Appendix E). Table B1 also gives recommended descriptive terms for weathered rock material, which are defined in Table A5.

Table B1 (heading 2b, structure) gives the descriptive terms for integral discontinuities in rock material. This concerns bedding and lamination in sedimentary rocks; banding, foliation or cleavage in metamorphic rocks or flow banding in igneous rocks. Integral discontinuities may be planes of relative weakness of the rock material. However, such features are not necessarily weak and strength testing of the rock is needed to establish this. *Massive* is used as a term to describe rock material without such structures.

Description of rock material from Engineering Geological Unit



Rock material:
1. colour: yellow, grey / greenish / light, dark ...
2. texture and fabric: layering or lamination / crystalline or clastic / grain shape ....
3. grain size
4. mineral content (quartz, clay ....)
5. ROCK NAME
6. ductility
7. strength
8. abrasiveness
9. weathering state

TABLE B1

Rock mass:
1. layer thickness (Table B1, 2b)
2. block size estimate (Table B3), or if not possible spacing of discontinuities (Table B2)
3. block shape (Table B3)
4. description of discontinuity sets
5. description weathering state of the mass (Table B4)

Figure B1 Description of engineering geological units for rock dredging purposes.

B2 DISCONTINUITY DENSITY AND BLOCK SIZE

Discontinuities are fractures in the rock mass (joints, fissures, faults, shear planes, bedding). Borehole cores provide one dimensional information on the presence of discontinuities. To describe spacing of discontinuities in cores, or in special scanline studies in a rock mass with several discontinuity sets, the terms of Table B2 are used. If the rock mass can be observed, the terms of Table B3 are used to describe the block size and shape of the mass, using the spacing of the largest discontinuity set as an indicator of size and the second term is used to describe the shape of the blocks.

Price has developed a ratings system that might be applied to estimates of excavatability of rock. In this system the three major discontinuity sets are used and the dimensions plotted on the graph of Figure B2. Examples how to use the graph are also given in the figure. The idea of this system is that cubic and large blocks compose the most stable discontinuous rock mass and are the most difficult to excavate. Such rock masses will get the highest rating number. The ratings diminish as the blocks become smaller and change in shape from cubic to tabular and then columnar.

Table B1 Descriptive entries for the engineering geological description of rock material.

Rock material description	Descriptive term	definition	reference
1. colour	Value: light, dark Chroma: pinkish, reddish, yellowish, brownish etc. Hue: pink, red, yellow, orange, brown, green, blue, purple, white, grey, black	Munsell colour chart may be used (Geol. Soc. of America)	BS 5039 Geol. Soc. of America
2a. texture and fabric	layered, laminated, cross-bedded, massive, crystalline, porphyritic, cryptocrystalline, granular, clastic, amorphous, micritic etc.	refers to properties of individual grains (texture) and their structural arrangement (fabric)	BS 5039
2b. structure (terms)	bedded, laminated, massive massive, flow-banded foliated, banded, cleaved	sedimentary rocks igneous rocks metamorphic rocks	Geot. Engn. Office (1994)
2c. structure (spacing of planar structures)	Very thick Thick Medium Thin Thickly laminated (sedimentary) Narrow (metamorphic and igneous) Thinly laminated (sedimentary) Very narrow (metamorphic and igneous)	> 2 m 600 mm - 2 m 200 mm - 600 mm 60 mm - 200 mm 6 mm - 20 mm 6 mm - 20 mm < 6 mm < 6 mm	BS 5039
3a. grain size (general)	Very coarse grained Coarse grained Medium grained Fine grained Very fine grained	> 60 mm 2 - 60 mm 0.06 - 2 mm 0.002 - 0.06 mm < 0.002 mm	Anon. 1977 Bell 1992
3b. grainsize (sandstones)	medium grained: coarse medium fine	0.6 - 2 mm 0.2 - 0.6 mm 0.06 - 0.2 mm	BS 5039
4. content of important minerals	Minerals that are known to have influence on the dredging operation (hard minerals, plastic minerals): e.g. quartz, clay	e.g. quartz-bearing: < 10 vol.% quartz prefix to rock name: 10 - 50 vol.% (quartz calcarenite) > 50 vol.% presence must be clear from rock name.	Table A2 Appendix E
5. rock name	based on simple identification or petrographic study		BS 5039 Table A1 Appendix E

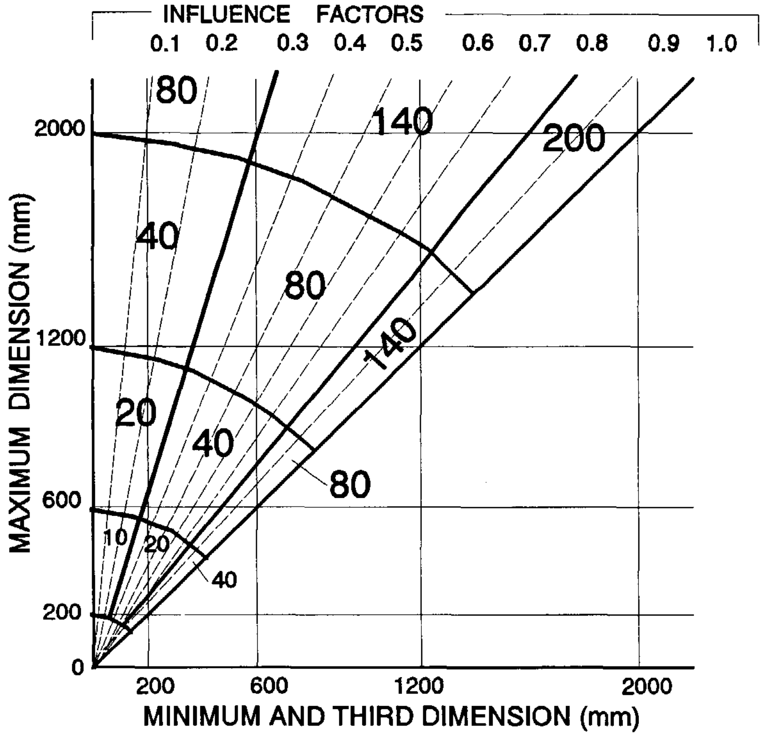
6. ductility	ductile average brittle	UCS/BTS < 9 9 - 15 > 15	Table 9.1
7. strength	very weak weak moderately weak moderately strong strong very strong extremely strong	UCS < 1.25 MPa 1.25 - 5 5 - 12.5 12.5 - 50 50 - 100 100 - 200 > 200 MPa	Table A6
8. abrasiveness	low abrasiveness abrasive highly abrasive extremely abrasive	F < 0.05 N/mm 0.05 - 0.1 0.1 - 0.5 > 0.5 N/mm	Table 11.2
9. weathering state	fresh discoloured decomposed disintegrated	see Table A5	Table A5

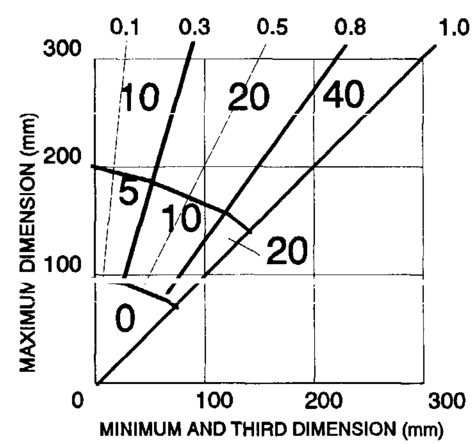
Table B2 Spacing of discontinuities in one dimension (rock core or scan line) (BS 5039:1981).

Term	Spacing
Very widely spaced	> 2 m
Widely spaced	600 mm - 2 m
Medium spaced	200 mm - 600 mm
Closely spaced	60 mm - 200 mm
Very closely spaced	20 - 60 mm
Extremely closely spaced	< 20 mm

Table B3 Spacing of discontinuities in three dimensions (rock mass) (BS 5039:1981).

First term	Maximum dimension
Very large	> 2 m
Large	600 mm - 2 m
Medium	200 mm - 600 mm
Small	60 mm - 200 mm
Very small	< 60 mm
Secondary term	**Nature of block**
Blocky	Equidimensional
Tabular	Thickness much less than length or width
Columnar	Height much greater than cross section





TO CALCULATE RATING:

1. Crossplot maximum and minimum dimensions to give max/min rating.
2. Crossplot maximum and third dimension to give influence factor to one decimal place (0.1 to 1.0)
3. Multiply max/min rating by influence factor to give final rating.

EXAMPLES:

Block: 1200 X 1200 x 1200 mm
Max/min rating = 140
Influence factor = 1.0
Final rating = 140

Block: 1000 X 400 x 800 mm
Max/min rating = 40
Influence factor = 0.9
Final rating = 36

Block: 1500 X 600 x 1000 mm
Max/min rating = 80
Influence factor = 0.7
Final rating = 56

Figure B2. Rating system for rock mass block size and shape, developed by D.G. Price.

B3 DESCRIPTION OF THE WEATHERING STATE OF THE ROCK MASS

Weathering of rock is commonly caused by the action of meteoric water on the rock, causing chemical reactions. Such reactions lead to deterioration of the rock. Since the groundwater is mainly percolating along the rock discontinuities, weathering begins along these surfaces. Table B4 gives the weathering grades that are internationally used. Further references to the methods used to describe and classify weathered rock mass: Anon. 1977, Bell 1992, Anon. 1995, Price 1993.

Table B4 Weathering grades of rock mass (BS 5039:1981).

Term	Description	Grade
Fresh	No visible sign of rock material weathering; perhaps slight discolouration on major discontinuity surfaces	I
Slightly weathered	Discoloration indicates weathering of rock material and discontinuity surfaces. All rock material may be discoloured by weathering.	II
Moderately weathered	Less than half of the rock material is decomposed or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.	III
Highly weathered	More than half of the rock material is decomposed or disintegrated into soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.	IV
Completely weathered	All rock material is decomposed and/or disintegrated into soil. The original mass structure is still largely intact.	V
Residual soil	All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.	VI

APPENDIX C

Intact rock strength

The most common index tests that are used to estimate the material strength of rock are the Unconfined Compressive Strength (UCS) test, The Brazilian Tensile Strength test (BTS) and the Point Load Strength test (PLS). These tests are carried out under standard conditions, because the results depend on various factors, such as test specimen shape and size, loading rates and other test conditions. Current procedures follow commonly the recommendations given by the ISRM (International Society for Rock Mechanics; Brown 1981). Further background information on these tests can be found in Brook (1993) and Pells (1993).

C1 UNCONFINED COMPRESSIVE STRENGTH

The Unconfined Compressive Strength test is the general accepted standard of engineering strength for intact rock. The test is carried out on cylinders of rock. Apart from the requirements mentioned in the ISRM 1979 standard, some comments are made on this test. To perform the test according to the standards accurate sample preparation is required. The length/diameter ratio should preferably be larger than 2 (The ISRM suggests 2.5), to avoid friction at the end platens affecting the stress condition in the centre of the specimen. The specimen ends should be flat, parallel and at 90° to the long axis of the core. Tests are normally carried out in a special rock mechanics laboratory, which has the facilities of specimen preparation. Relatively simple testing frames exist to perform the test. Nowadays many laboratories have stiff servo-controlled testing machines that provide the possibility of accurate deformation monitoring during the test, also after failure. If available, with little extra effort, the tests could be performed on such a testing frame. The complete stress-strain curve that is obtained this way gives additional information besides the unconfined strength, namely the deformation modulus and the specific work of failure. The latter is larger in ductile rock and has successfully been related to rock drillability by Thuro (Thuro 1996, Thuro & Spaun 1996). Figure C1 illustrates the concept.

Thuro (1996) calculates the specific work of failure (specific destruction work) using the complete stress-strain curve:

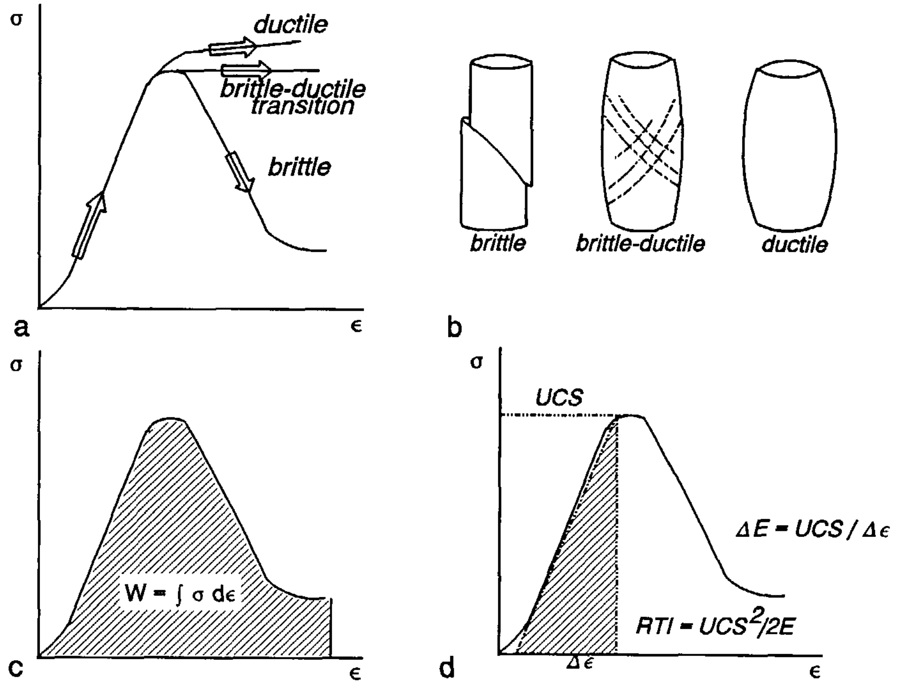


Figure C1 Concepts of brittle, brittle-ductile and ductile failure behaviour in unconfined compression tests.

$$W_z = \int \sigma \; d\epsilon \; (kJ/m^3) \tag{C1}$$

It is obvious, that at a similar level of failure stress (UCS), the work of failure for brittle rock is less than for ductile rock. This difference in estimate of work of failure does not come out when UCS value and the deformation modulus (E) are used, as in traditional estimates of work of failure. For example, Singh et al. (1986) defined the Rock Toughness Index (RTI) for this purpose:

$$RTI = \frac{UCS^2}{2E} \; (MPa) \tag{C2}$$

In Figure C1c & d the two concepts are compared. In the hypothetic example of Figure C1a, the Rock Toughness Index would be similar for the brittle, brittle-ductile and ductile case, whereas the specific work of failure would differ. It should be noted that the complete stress-strain curve does not, as is sometimes thought, describe a material property. The curve is dependent on specimen shape (geometry), rock material properties and on machine stiffness (loading system), see Pells (1993). Thuro's proposal, of determining the specific work of failure, therefore gives results which are test machine dependent.

C2 BRAZILIAN TENSILE STRENGTH

The second relevant rock parameter is the Unconfined Tensile Strength (UTS). Direct tensile tests are not commonly performed, because these require much effort in sample preparation and testing procedure[80]. The Brazilian split test is by far the most common test used to estimate tensile strength. The test is easy to perform and can be used in the field as well, if a small diamond saw is available to cut parallel slices from rock core. The test can be done on short length of core (less than 25 mm) and no strict requirements regarding test disc preparation are needed. The small size of the samples makes it an excellent test to perform on many samples and to assess any anisotropy in strength of the rock. The diameter of the disc should preferably be 50 mm, the grain size of the rock is important in this respect (10 times the grain size is the minimum requirement). Very coarse rock (like limestones containing quartz pebbles or large shells) may preclude valid testing, since the tensile crack growth should start from the centre of the specimen. According to Pells (1993), the BTS test is generally valid for rocks which have a UCS > 5 MPa.

Once for each rock type within a project site the ratio UCS/BTS is established (which is important with regard to rock cutting), the test may be used as an estimate of UCS as well.

There is much discussion about the validity of the Brazilian test with regard to the estimation of the direct Unconfined Tensile Strength (UTS) value. The discussion on the validity of the BTS test as estimate of the direct UTS stems from several concerns:

- Often crack growth starts directly below the platens of the compression jig in stead of the centre of the specimen. This can be prevented by using cardboard strips between the platens, which cause some circumferential confinement.

- The equation used to calculate BTS (ISRM suggested method 1978, in Brown 1981) assumes the rock to be homogeneous, isotropic and elastic. Most rocks are anisotropic and this is known to affect the test result. Chen & Stimpson (1993) have examined the effect of the modulus of deformation (E) being unequal in tension and compression. For most rocks the ratio E_t/E_c is smaller than 1 and the UTS is overestimated. For example for a ratio of 0.5, the BTS overestimates the UTS by 15%.

- In a study of the comparison of the BTS test, the direct UTS test and the Hoop test (a promising new test to estimate UTS) Shulin Xu et al. (1988) found that the BTS of Penrith sandstone was four times higher than the direct UTS. Another sandstone gave a BTS three times higher than UTS! (The Hoop test gave nearly the same result as the direct UTS test). These observations suggest that the data in literature on tensile strengths of rocks could have a bias towards too high values. Pells (1993) states that for most applications in practice this discussion is largely academic.

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[80] Direct tensile tests apparently give more variation in test results due to this. Compare the results of Shulin Xu et al. (1988) on Penrith sandstone: Coefficient of variation (COV) of BTS test: 15%, COV of Hoop test: 15%, COV of direct UTS: 25%.

C3 POINT LOAD STRENGTH

In rock dredging up to present the Unconfined Compressive Strength is used as the measure of strength of intact rock and the main parameter to assess dredgeability. Often the portable Point Load test is used to determine the strength of a large number of samples. Originally the PLS test was a development from the Brazilian test and used to determine the tensile strength of brittle rocks (Hoek 1977). But in (very) weak rocks the test becomes inaccurate. The conical points penetrate into the rock before failure and the failure itself may occur by a diffuse zone of multiple fractures and not by a vertical tensile fracture. Hoek (1977) suggests that, for this reason, the test may not be valid for rocks which have a UCS/BTS ratio of less than 5. Rocks with such low ratios are common in dredging projects (weak limestones, claystones etc.). But for more brittle rocks, when the test is carried out in diametric loading, the Point Load test might give a value reasonably close to the BTS value. A value of PLS=0.8*BTS (or BTS=1.25*PLS) is suggested by ISRM (1985), but it is advised to establish a correlation by comparing test results on samples of rock. Point loading resembles the wedge penetration of a cutting tool and although the test occurs in a static mode, sometimes it correlates better with cutting tool performance than either the UCS or the BTS test. If the point load test is carried out on rock cylinders (diametrical test), the coefficient of variation (COV), which gives an indication of the spreading of test results, is commonly somewhat lower than that of the BTS test (Piepers 1995).

The Point Load test can also be carried out on irregular lumps of rock. In fact, the test was originally meant as an aid in rock strength classification (Franklin et al. 1971). The results of PLS index tests are commonly used to estimate UCS. The test can be used on site, which is an advantage. It is well established that each rock type has its own correlation of PLS with UCS. In other words for each project calibrations should be made[81]. Normally a certain spreading of data points around the calibration line is common. The standardized procedures for this test are given by the ISRM suggested method (1985). As described there, the test can be used to determine the strength anisotropy of the rock, which is significant for the cutting process.

C4 EFFECT OF LOADING RATE AND MOISTURE

Loading rate influences the result of strength tests. This effect is discussed in relation to the stress rate dependence of cutting forces in Chapter 9.4 (see Figure 9.14). At high loading rates strength tends to increase exponentially. The reason for the strength increase is not clear yet (see Goodman 1989).

[81] Note that in the Franklin diagrams (Figure 15.1) the factor 24 is used (UCS = 24 PLS), Pettifer & Fookes (1994) used the factor 20, which was the average for the rocks in their data base (Figure 18.3).

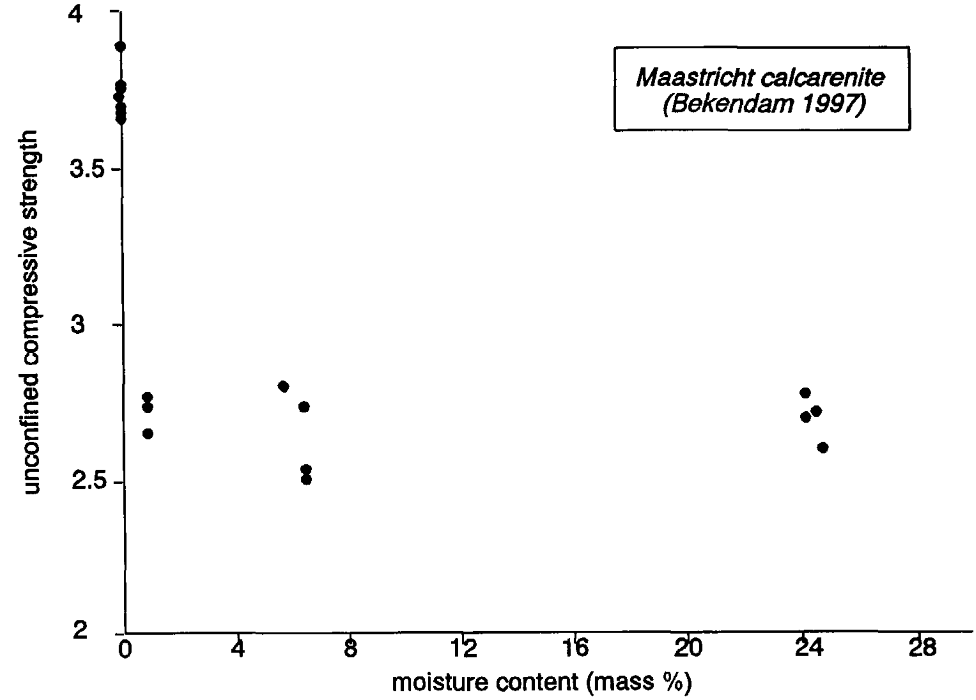

Figure C2 Influence of moisture on the unconfined strength of calcarenite rock (Bekendam 1997).

The presence of moisture is known to influence strength. A number of influencing factors are known, such as:

1. a physical-chemical effect
2. pore pressure changes, including negative pore pressures (capillary tension)
3. friction reduction

The effects of water on rock cutting have been discussed in Chapter 9.5. The first effect mentioned above is important, because it applies generally. It is known that the presence of water in rock has a weakening effect. Already at relatively low moisture contents the reduction in strength is known to occur, Figure C2 (Bekendam 1997, Hawkins & McConnell 1992). Compared with dry rocks, water bearing or saturated rocks may have strengths which are 30 - 90 % of the dry strength measured. This reduction in strength occurs independent of loading rate and is not a mechanical effect (due to pore pressures), but an electrochemical effect, as explained by Vutukuri (1974), who tested limestones immersed in different types of fluids. Water has a pronounced weakening effect compared to other fluids with lower dielectric constants and weakening may occur already at low degrees of water saturation. The effect is reversible in rocks without swelling minerals (like smectite clays). Pells (1993) tested Hawkesbury sandstone after 1, 4 and 8 cycles of saturation and oven drying and found the strength continued to vary between the saturated and dry values. The conditions of testing of rocks should therefore always be specified and moisture contents should be measured. For rock dredging saturated rock strengths should be used in rock excavatability assessments.

APPENDIX D

The brittle-ductile transition

The importance of the brittle-ductile transition for rock cutting is discussed in Chapter 9.1 and by Verhoef et al. (1996). In the rock cutting model developed by Uittenbogaard et al. (in prep.), see Figure 9.2, it is emphasized that a crushed zone is present below the tip of the cutting tool. This implies that the stress field near the tip is in the ductile cataclastic failure field. During rock cutting therefore the complete failure envelope for intact rock is addressed.

The stress level at which the transition from brittle to ductile failure occurs is of interest. It occurs at the apex of the failure envelope, as illustrated by the results of three series of triaxial tests carried out by Ockeloen (1997). Figure D1 shows typical stress-strain curves for tests done with increasing confining pressure on Felser sandstone. The surface area below the failure envelopes relates to the amount of work involved in the failure process, in analogue with the stress-strain diagram of an unconfined test (Figure C1 & D1). The failure stresses are plotted in a p-q diagram in Figure D2, together with the results of tests on mortar and calcarenite rock. The failure stresses of the triaxial tests define the failure envelope of the three rocks (compare with Figure 9.2). The diagram of Figure D2 indicates that the brittle-ductile transition of the Felser sandstone is at a relatively higher normalized stress level than that of the mortar and calcarenite. The two lines shown in the figure are the brittle-ductile transition stress lines found by Mogi (1966). The line $\sigma_1=3.4\sigma_3$ approximates the BD (brittle-ductile) transition stress for silicate rocks, the line $\sigma_1=4.2\sigma_3$ was the best fit for the BD transition stress of carbonate (limestone) rocks. These lines separate the diagram in a brittle failure area (left of the lines) and a ductile failure area (right of the lines), see also Figure 9.2 & 4.

Apart from triaxial tests to establish the failure envelope and the brittle-ductile transition stress, other methods have been mentioned that may indicate ductile or brittle rock:

1. The ratio UCS/BTS (ductility number). Gehring (1987) proposed boundaries (Chapter 9.1.2, Table 9.1). A ratio < 9 indicates ductile rock; > 15 brittle rock.
2. Assessing the complete stress-strain path of a UCS test and estimate the specific work of failure (Figure C1).
3. Estimate the failure envelope using BTS (or PLS) and UCS test and the empirical Hoek-Brown failure criterion.

These three possibilities have been examined and are discussed in the next section.

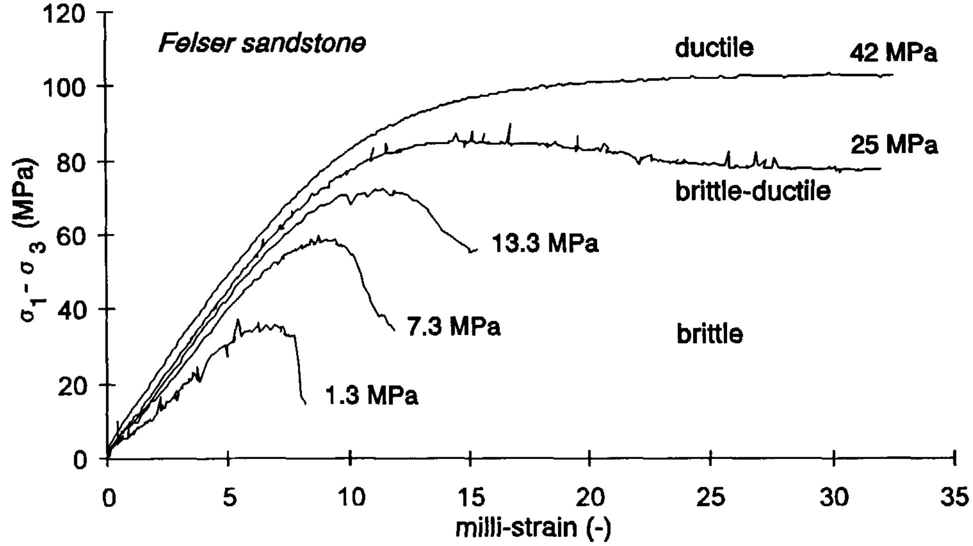

Figure D1 Stress-strain diagrams from triaxial tests illustrating the transition from brittle to ductile failure with increasing confining pressure σ_3.

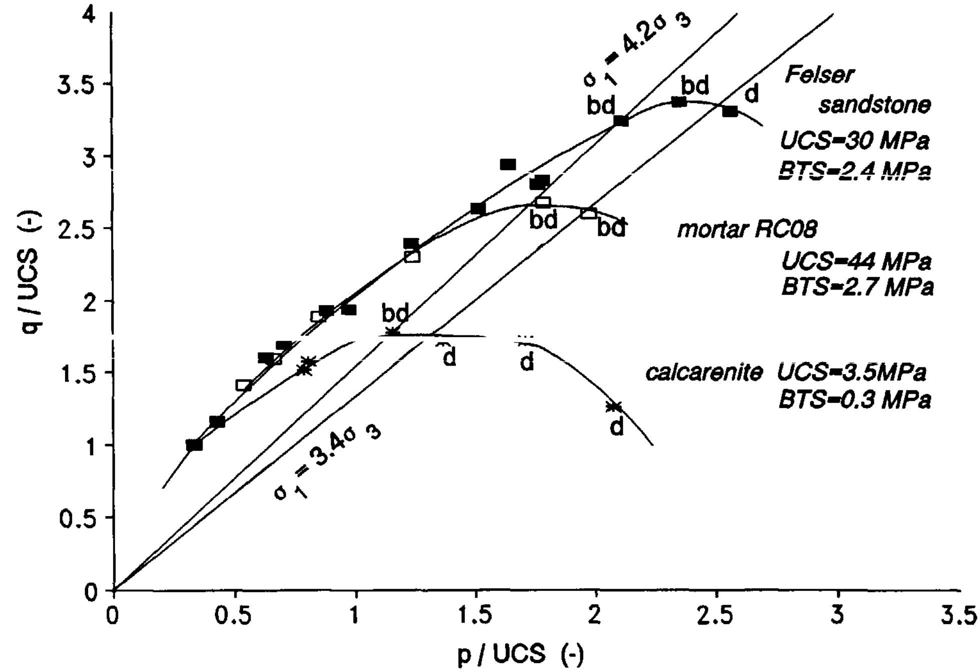

Figure D2 Failure envelopes determined by triaxial testing. d = ductile failure; bd = brittle-ductile transition; p = $(\sigma_1+2\sigma_3)/3$; q = σ_1-σ_3.

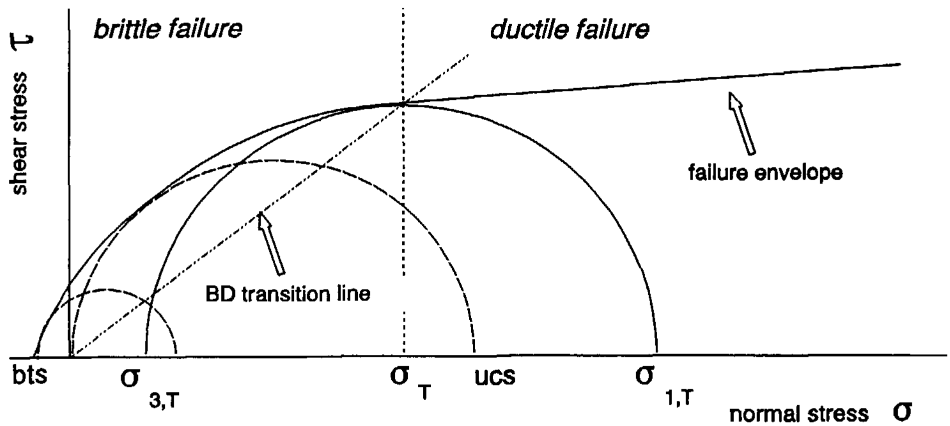

Figure D3 Estimation of the shape of the failure envelope in the brittle field by the BTS and UCS index tests. The Mohr circle for the brittle-ductile (BD) transition stress is given.

D1 INDEX TESTS AND THEIR RELATIONSHIP WITH ROCK MECHANICS FAILURE THEORY

For most rock engineering applications extensive rock testing is precluded for time and cost reasons. Therefore it is examined here how the simple index tests (BTS and PLS) and the UCS test can assist in defining the shape and order of magnitude of the failure envelope (Figure D3). A well established approximation of the failure envelope in the brittle part of the diagram is the Hoek-Brown failure criterion, which was developed after examination of numerous sets of triaxial test data.

Hoek & Brown (1980) compared the test results by normalising the strength data sets by dividing through the UCS value. They studied the shape of the failure envelopes and found that these generally have a parabolic shape in σ_1-σ_3 diagrams. The failure curve can be described by:

$$\sigma_{1n} = \sigma_{3n} + (m\sigma_{3n} + s)^{0.5} \qquad \text{(D1)}$$

where σ_{1n} and σ_{3n} are the normalised major and minor principal stress at peak strength (MPa) (σ_1/σ_c and σ_3/σ_c; σ_c is the UCS (MPa)), m is a constant depending on the rock type and s is a parameter describing the degree of fracturing of the rock (for intact rock $s = 1$, for fractured rock $s < 1$).[82] In the following only intact rock is considered. The data base used by Hoek & Brown could be divided into several rock

[82] The analysis of Hoek & Brown (1980, p.137-151) extends to the strength of rock masses, by including the effect of discontinuities. Interestingly, from RMR (Rock Mass Rating) values estimates of Hoek-Brown rock mass failure criteria can be made (Hoek & Brown 1988).

Table D1 Rock types with similar shape of failure envelope (Hoek & Brown 1980)

Rock type	m-value	UCS/UTS[1]
a. Crystalline limestones (dolomite, limestone, marble)	7	7.1
b. Lithified argillaceous rocks (mudstone, shale, slate, claystone)	10	10.1
c. Arenaceous silicate rocks (sandstone, quartzite)	15	15.1
d. Fine grained crystalline igneous rocks (andesite, dolerite, basalt)	17	17.1
e. Coarse grained crystalline igneous and metamorphic rocks	25	25.0

[1] Calculated from Equation D1

types, each with its own characteristic m-value (Table D1). From Equation D1, the *unconfined tensile strength* (UTS) of the rock may be estimated as well, by choosing $\sigma_1 = 0$. Table D1 gives the calculated UCS/UTS value for each of the rock types. It is striking that the ductility number (UCS/UTS) is nearly equal to the m-value. Brook (1993) explains that the use of the fixed index of 0.5 in the Hoek & Brown criterion is a reasonable approximation, but this makes that *m* is nearly, but not quite the ratio UCS/UTS for intact rock[83].

Hoek & Brown emphasize that their analysis is only valid in the regime of brittle failure. They offered a tentative boundary for the brittle-ductile transition, based on work of Mogi (1966). For silicate rocks the line $\sigma_1=3.4\sigma_3$ divides the stress diagram in a brittle and a ductile field. This line is shown in the Figures 9.4 and D2. Although the transition line used is known to be a simplification, Figure D4 illustrates that rock with a low m-value enters the ductile field at relatively lower stress values.

The empirical rock failure criterion concept of Hoek & Brown (1980) is useful for several reasons:

- it shows that the shape of the failure curve (described by *m*) is related to lithological composition and microscopic structure (i.e. rock type).
- if the *m*-value of the rock type under consideration is known (see Table D2), from the UCS value alone an idea of the shape of the strength envelope may be obtained
- an impression of the ductility of the rock may be obtained

Hoek and Brown (1980) chose the relationship $\sigma_1=3.4\sigma_3$, based on the work of Mogi (1966), as the best approximation of the brittle-ductile transition. With the help of the Hoek-Brown criterion and the m-value (or UCS/UTS) it is possible to make a prediction of the shape of the failure envelope in the brittle field and the approximate position of the brittle-ductile stress state (Figure D3 & D4). The similarity of the

[83] A more general relation between peak stress at failure and the confining stress is given by a power law like: $\sigma_1/UCS=A(\sigma_3/UCS+UTS/UCS)^n$, where $A=(UCS/UTS)^n$, see Brook 1993, p.63.

Table D2 Values of m for the common rock material types. Values in parenthesis are estimates (Hoek et al. 1995).

Rock type	Class	Group	Texture			
			Coarse	Medium	Fine	Very fine
Sedimentary	Clastic		Conglomerate (22)	Sandstone 19	Siltstone 9	Claystone 4
			<----Greywacke----> (18)			
	Non-clastic	Organic	<------Chalk------> 7 <------Coal-------> (8-21)			
		Carbonate	Breccia (20)	Sparitic Limestone[1] 10	Micritic Limestone[2] 8	
		Chemical		Gypsum 16	Anhydrite 13	
Metamorphic	Non foliated		Marble 9	Hornfels (19)	Quartzite 24	
	Slightly foliated		Migmatite (30)	Amphibolite 31	Mylonite (6)	
	Foliated[3]		Gneiss 33	Schist (10)	Phyllites (10)	Slate 9
Igneous	Light		Granite 33 Granodiorite (30) Diorite 28 Gabbro 27 Norite 22	Dolerite (19)	Rhyolite (16) Dacite (17) Andesite 19 Basalt (17)	Obsidian (19)
	Extrusive pyroclastic type		Aggomerate (20)	Breccia (18)	Tuff (15)	

[1] crystalline limestone; [2] micrite = limestone mud; [3] m-values for rocks tested normal to the foliation

brittle-ductile transition state with the critical state of soil mechanics (transition of brittle overconsolidated soil to normally consolidated, ductile, soil) has been noted by many workers (see Farmer 1983, Novello & Johnston 1995). The brittle-ductile transition would then coincide with the critical state line of Schofield & Wroth (1968): $q=Mp$, where $q=\sigma_1-\sigma_3$, *and* $p=(\sigma_1+2\sigma_3)/3$ (Figure 9.2). Barton (1976) proposed that the critical state line would coincide with the residual or frictional strength of failure planes in intact rock ($\sigma_1=2\sigma_3$ for undrained and $\sigma_1=3\sigma_3$ for drained failure; see Farmer 1983).

Verhoef et al. (1996) describe triaxial and compression cell tests that were carried out to study the brittle-ductile transition of several rock types. Hergarden (1997) performed additional triaxial tests on the rocks form the trencher project (Chapter 17). In Figure D5 the brittle-ductile transition (*BD*) points of all tests have been plotted. Linear regression gives a value of 5.2±0.3 for the best fitting factor ($\sigma_1=5.2\sigma_3$) for all data (n=32 data pairs). Mogi (1966) already distinguished between silicate and carbonate rocks. For silicate rocks (n=24) the factor of 3.4 was

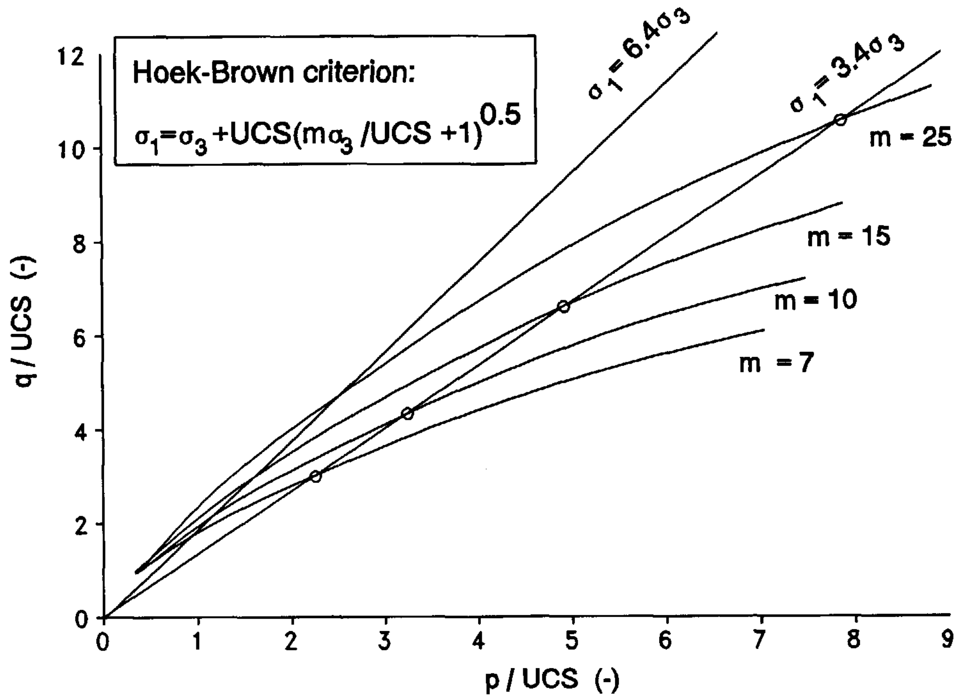

Figure D4 Prediction of the brittle-ductile transition stress, using the Hoek-Brown criterion (*m*-value), Mogi's BD line for silicate rock (3.4) and the BD line for weak carbonate rock (6.4).

found, for carbonate rocks (n=21) a factor of 4.2. Both Mogi's transition lines are plotted in Figure D2. Most of the rocks of Figure D5 are carbonate rocks. Regression on only the carbonate rocks (n=24), gave a factor of 6.4±2 (discrimination coefficient 0.97). The silicate rocks (n=6) gave a factor of 3.9±0.3 (but this regression is inaccurate, with very low discrimination coefficient). These factors are higher than Mogi's. The reason for this discrepancy probably relates to the higher amount of weak rocks tested in Delft. Mogi tested stronger rocks and could reach higher confining stresses with his equipment (up to 500 MPa). Mogi also noted, however, that at lower confining stress, the transition from brittle to ductile was along a steeper transition line. Probably for weaker carbonate rock the transition from brittle to ductile occurs at relative lower confining stress. The 6.4 factor for carbonate rocks, as found by this research, is given in Figure D4, to show the likely area where transition from brittle to ductile failure occurs.

Table 17.5 gives the results of the tests performed by Hergarden (1997). These test results have been analyzed, to see whether any of the parameters known to relate to rock ductility, like UCS/BTS (Chapter 9.1), the *m*-value, the BD transition stress or the destruction work (Figure C1) would relate to the observed bit wear, the production or the estimated stress under the bits of the trencher (Table 17.4). No clear answers can be found. Hot bits are commonly, but not always, related to high wear. Hot bits can occur either with very strong brittle cutting rocks or with less strong ductile cutting rocks. It is thought that more of this type of data is needed and

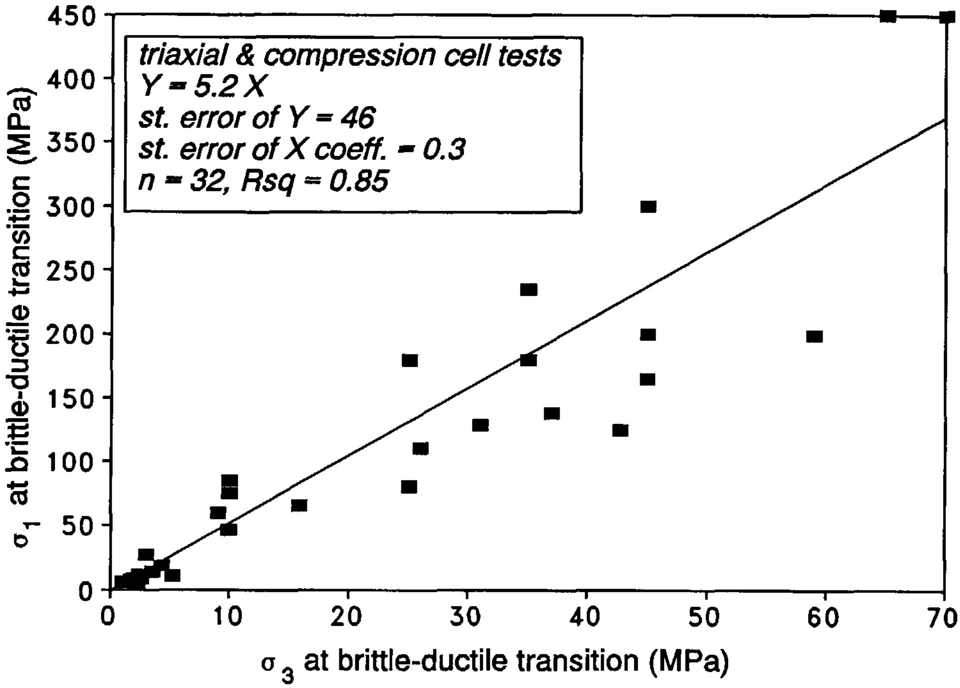

Figure D5 Linear regression of brittle-ductile transition stress found by triaxial and compression cell testing (data from Verhoef et al. 1996 and Hergarden 1997).

that, using neuro-fuzzy logic programming techniques, clarification can be found in this matter.

In Figure D4 the intersection of the failure envelopes of rocks with different *m*-value and the *BD* transition line of Mogi defines the predicted *BD* points. Hergarden (1997) determined the *m*-value from the triaxial experiments (Table 17.5). Note that the experimentally determined *m*-values differ from the UCS/BTS ratio's found for these rocks. Using the *m*-value and a brittle-ductile transition line with a factor of 6, estimates of the brittle-ductile transition stress were made, to test the usefulness of the prediction method illustrated by Figure D4. Predicted transition values are too low for weak rocks and too high for strong rocks, no good estimates can be made.

There is insufficient data available to make firm statements. It appears that attempts to estimate the failure envelope using simple index tests, like *UCS, BTS, m*-value, are less reliable than triaxial tests. The latter tests are worthwhile to perform when considering rock cutting. The tests should be carried out well into the ductile field. The resulting envelope will be helpful in predicting the cutting behaviour of the rocks. A small envelope points to ductile cutting, a large one to brittle cutting (Figure D2).

APPENDIX E

Petrographic description

The microscopic composition and structure of intact rocks have a profound influence on their mechanical behaviour. This is well established and in the context of this work, illustrated by the study of abrasiveness (Chapter 11) and the clearly distinct behaviour of carbonate and silicate rocks during rock trenching (Chapter 17). Therefore it is important that the rocks that are considered in a dredging project are adequately described. It is important that the composition and the texture of the rocks are well known. This implies that a petrographic study of the rock types should be undertaken. Important to know are:

- mineralogical composition
- rock microstructure (texture)
- homogeneity
- anisotropy

During the drilling campaign the rock should be described by a professional engineering geologist, by methods of visual examination. Very commonly this description occurs using standard codes. This practice is encouraged and descriptions which are not done using an established code of practice should not be accepted without confirmation of their quality. The Code of practice for Site investigations (British Standard Institute, BS 5930:1981) is an example of a good standard. The PIANC (1984) classification of soils and rocks to be dredged follows and refers to the BS code.[84]

Besides the visual examination, however, microscopic investigation of typical samples from a site should be undertaken to obtain information vital for the interpretation of potential abrasiveness of rock. Petrographic examination should be carried out by specialists. From the rock a thin section should be prepared, by cutting a slice of rock, attach this to a glass plate and grinding and polishing this down to 0.3 mm thickness. Also from porous sandy soils such thin sections can be prepared. The soil sample should be immersed with epoxy resin before preparation.

[84] For the interested contractor and consulting engineer, a simple scheme to recognize minerals and rock is presented in Appendix A. This resembles the scheme of BS 5930:1981 and the classification of PIANC. Apart from the general classification, a classification of carbonate rocks is included.

PETROGRAPHIC DESCRIPTION **name :** **date :**

Project : location :
coordinates :
spec. no. : thin section no. :
collection :
Rock name :
petrographic classification :
geological formation :

Rock Engineering Test Results :
UCS; $\perp$,$\parallel$; wet/dry: (MPa)
PLS/BTS; $\perp$,$\parallel$; wet/dry: (MPa)
density: (Mg/m^3)
porosity: (%)
water absorption: (%)
MBA test: (meq/l)
other test results:

MACROSCOPIC DESCRIPTION OF SAMPLE	General: Degree of weathering: Structure (incl. bedding): Fractures, micro-cracks:

MINERALOGICAL COMPOSITION AND MICROSCOPIC STRUCTURE

mineral	**vol. %**	**grain size**	**description**

microstructure	general microstructure / texture: fracturing: alteration:
GENERAL REMARKS	Significance of results for rock engineering:

Figure E1 Suggested format for a petrographic description for engineering purposes.

The ISRM has shortly described a *Suggested method for petrographic description of rocks* (Brown 1981). A format for petrographic description is given in Figure E1. An example of a description is given is given in Figure E2.

LABORATORY OF ENGINEERING GEOLOGY, DELFT UNIVERSITY OF TECHNOLOGY

PETROGRAPHIC DESCRIPTION

name : P.N.W.Verhoef
date : 16.5.97

Project : Dredging contractor
location :
coordinates :
spec. no. : 97-32
thin section no. : 97-32c
collection :

Rock Engineering Test Results :
UCS; $\perp$,$\parallel$; wet: $\perp$ 19; // 23 (MP
BTS; $\perp$,$\parallel$; wet: 1 (MP
Schmidt hammer/Equotip:
density/porosity: (Mg/m^3) (%)
water absorption: (%)
MBA test: (g/100g) (me
other test results:

Rock name : slightly weathered SANDSTONE
petrographic classification : quartz wacke
geological formation :

MACROSCOPIC DESCRIPTION OF SAMPLE	General: light brownish yellow, laminated, slightly weathered, porous, mica bearing Quartz-rich SANDSTONE, with local iron-oxide staining along laminations Degree of weathering: slightly to moderately Structure (incl. bedding): laminated, with iron(hydr)oxide staining along lamination Fractures, micro-cracks: -

MINERALOGICAL COMPOSITION AND MICROSCOPIC STRUCTURE

mineral	**vol. %**	**grain size**	**description**
Quartz	65	0.2-0.4 mm	angular, slightly elongated grains
Plagioclase	10	0.2-0.4	Both feldspars are partially altered to
Alkali-feldspar	10	0.2-0.4	sericite/kaolinite clay and iron-oxide
Muscovite	5		partially weathered
Chlorite			
Fe(hydr)oxide + Sec. min.s	10		brownish secondary minerals along grainboundari / cracks
tourmaline			
opaque			
(pore space)			

microstructure	general microstructure / texture: laminated rock. Quartz-feldspar grai slightly elongated parallel to lamination fracturing: Fe-(hydr)oxides and secondary minerals in cracks and along grainboundaries alteration: general discoloration; slightly to moderately weathered
GENERAL REMARKS	Significance of results for rock engineering: This sandstone specimen is slightly to moderately weathered, which probably has resulted in weakening with respect to the fresh equivalent of this rock. abrasiveness: F-value = 0.15-0.30 N/mm (highly abrasive)

Figure E2 Example of a petrographic description.

References

Abdullatif, O.M. & D.M. Cruden 1983. The relationship between rock mass quality and ease of excavation. *Bull Int. Assoc. Eng. Geol.*: 28, 183-187.

Al-Ameen, S.I. & M.D. Waller 1992. Petrographic analysis for assessing the abrasiveness of Coal Measures rocks. *Dep. Min. Resour. Eng. Mag.*: 44, 29-34.

Al-Ameen, S.I. & M.D. Waller 1994. The influence of rock strength and abrasive mineral content on the Cerchar Abrasive Index. *Engineering Geology*: 36, 293-301.

Aleman, V.P. 1983. Prediction of cutting rates for boom type roadheaders. *Tunnels & Tunnelling* Jan 1983: 23-25.

Aleva, G.J.J. 1990. *Laterites. Concepts, geology, morphology and chemistry.* ISRIC (International Soil Reference and Information Centre. pp. 169. Wageningen, The Netherlands.

Alvarez Grima, M., M.H. den Hartog, H.J.R. Deketh, P.N.W. Verhoef & R. Babuska 1996. Uncertainty handling of mechanical excavation processes with fuzzy logic (Geotechnics Congress, Havana, September 1996).

Alvarez Grima, M. & P.N.W. Verhoef 1997. Forecasting of rock trencher performance using a fuzzy logic approach. *Int. J. Rock Mech. & Min. Sci.*: 34, 3-4, paper no. 325 (36th US Rock Mechanics Symposium, NYRocks'97).

Ang, A.H-S & W.H.Tang 1988. *Probability Concepts in Engineering Planning and Design, Volume II-Decision, Risk, and Reliability.* John Wiley & Sons.

Anon. 1977 The description of rock masses for engineering purposes. Working party report. *Q. J. Engng. Geol.*: 10, 355-388.

Anon. 1995 The description and classification of weathered rocks for engineering purposes. Working party report. *Q. J. Engng. Geol.*: 28, 207-242.

Archer, R. 1983. Port Hedland. *Western Australia 1:50.000 Urban Geology Series:* Sheet 2657 III. Geological Survey of Western Australia 66 Adelaide Tce., Perth.

Atkinson, R.H. 1993. Hardness tests for rock characterization. In: J.A. Hudson (ed.) *Comprehensive Rock Engineering, vol. 3, Rock testing and site characterization*: 105 - 117. Permagon Press, Oxford.

Babuska, R. 1995 Fuzzy logic in modelling and control. *Proceedings Symposium Computational Intelligence*, May 10 1995, Delft, The Netherlands.

Barton, N. 1976. The shear strength of rock and rock joints. *Int. J. Rock Mech. & Min. Sci.*: 13, 255-279.

Barton, N., R.Lien & J.Lunde 1974. Engineering classification of rock masses for the design of tunnel support. *Rock Mechanics*: 6, 189-236.

Barton, N. & O. Stephanson (eds.) 1990. *Rock Joints. Proceedings International Symposium, 4-6 June, Loen, Norway*: 814 pp. Balkema, Rotterdam.

Bekendam, R. 1997. *The assessment of stability of the calcarenite room and pillar mines of South Limburg*. PhD thesis Faculty of Applied Earth Sciences, in preparation.

Bell, F.G. 1992. *Engineering in rock masses*. Butterworth, Oxford. 580 pp.

Bieniawski, Z.T. 1974. Geomechanics classification of rock masses and its application in tunnelling. *Proceedings 3rd Congress ISRM, Denver*: vol. 2A, p.27

Bieniawski, Z.T. 1989. *Engineering rock mass classifications*. John Wiley & Sons, New York. 251 pp.

Bilgin, N., T. Seyreck & K. Shahriar 1988. Golden Horn clean-up contributes valuable data. *Tunnels & Tunnelling*: 20, no.6, 41-44.

Bisschop, F. 1991. The analysis of a laboratory cutting and abrasion test to be applied in rock cutting dredging. *Classified internal report, Memoirs of the Centre for Engineering Geology in the Netherlands* 81 105 pp. Delft University of Technology.

Blindheim, O.T., A. Boniface & J.A. Richards 1990. Boreability assessments for the Lesotho Highlands water project. In: R. Brummer (ed.) *Static and dynamic considerations in rock engineering. ISRM Symposium, Swaziland*: 29-39. Balkema, Rotterdam.

Blindheim, O.T., A. Boniface & J.A. Richards 1991. Boreability assessments for the Lesotho Highlands water project. *Tunnels & Tunnelling*: June, 55-58.

Bold, H.J. van den 1986. De abrasieve slijtage van snijwerktuigen door gesteente (The abrasive wear of cutting tools by rock). *Memoirs of the Centre for Engineering Geology in the Netherlands:* 38, 98 pp. Delft University of Technology.

Bowden, F.P. & D. Tabor 1950. *The friction and lubrication of solids. Part I.* Clarenden Press. Oxford. 337 pp.

Bowden, F.P. & D. Tabor 1964. *The friction and lubrication of solids. Part II.* Clarenden Press. Oxford. 544 pp.

Bowie, S.H.U. & P.R. Simpson 1977. Microscopy: reflected light. In J. Zussman (ed.): *Physical methods in determinative mineralogy*: 109-165. Academic Press, London.

Brady, B.H.G. & E.T. Brown (1985). *Rock mechanics for underground mining.* George Allen & Unwin, London. 527 pp.

Bray, R.N. 1979. *Dredging, a handbook for engineers*. Edward Arnold, London. 276 pp.

Bray, R.N., A.D. Bates & J.M.Land 1997. *Dredging, a handbook for engineers* (2nd ed). Edward Arnold, London. 434 pp.

Braybrooke, J.C. 1988. The state of the art of rock cuttability and rippability prediction. *Proc. 5th Australian-New Zealand Conf. on Geomechanics, August 1988, Sydney:* 13-42.

Bree, S.E.M. de 1975. The abrasion behaviour of materials for dredger components, due to sand/water mixtures. *First International Symposium on Dredging Technology, Canterbury. BHRA Fluid Engineering, Cranfield, Bedford, England:* 35-57.

Bree, S.E.M. de, A. Begelinger & A.W.J. de Gee 1980. A study on the wear behaviour of materials for dredge parts in water sand mixtures. *Third International Symposium on Dredging Technology, Bordeaux. BHRA Fluid Engineering, Cranfield, Bedford, England:* 299-314.

Bree, S.E.M. de, W.F. Rosenbrand & A.W.J. de Gee 1982. On the erosion resistance in water-sand mixtures of steels for application in slurry pipelines. *Hydrotransport 8, International Conference on the hydraulic transport of solids in pipes, Johannesburg, South Africa:* 161-180.

Brook, N. 1993. The measurement and estimation of basic rock strength. In: J.A. Hudson (ed.) *Comprehensive Rock Engineering, vol. 3, Rock testing and site characterization*: 41-66. Permagon Press, Oxford.

Brouwer, J. 1984. Gesteentemechanica in de baggertechniek (Rock mechanics in dredging technology). *Nieuwsbrief Ingeokring*: juli, 35-44.

Brown, E.T. 1981. *Rock characterization, testing and monitoring. ISRM suggested methods*. Permagon Press, Oxford. 211 pp.

Brugman, M.H.A. 1997. A three-dimensional production estimation system. Geotechnical aspects of dredging production estimation in the context of the Øresund Link Tunnel Trench dredging project. *Memoirs of the Centre of Engineering Geology in The Netherlands*: 157, 111 pp. Delft University of Technology.

Bruland, A., B.E. Johannessen, A. Lislerud, T. Movinkel, K. Myrvold & O. Johannessen 1988. *Hard rock tunnel boring*. Project report 1-88: 183 pp. Norwegian Institute of Technology, Trondheim.

BS 5930:1981. *Code of practice for site investigations*. British Standards Institution, London. 147 pp.

Büchi, E. 1984. *Einfluß geologischer parameter auf die Vortriebsleistung einer Tunnelbohrmachine*. (Influence of geological parameters on the performance of a tunnel boring machine). PhD thesis University of Bern, Switzerland, 136 pp.

Büchi, E., J.-F. Mathier & Ch. Wyss 1995. Rock abrasivity - a significant cost factor for mechanical tunnelling in loose and hard rock. *Tunnel*: 5, 38-44.

Chappell, J. 1987. Late Quaternary sea-level changes in the Australasian region". In Tooley, M.J. & I. Shennan (eds.) *Sea-level changes*. Blackwell, Oxford.

Chen, R. & B. Stimpson 1993. Interpretation of indirect tensile strength tests when moduli of deformation in compression and tension are different. *Rock Mechanics and Rock Engineering*: 26, 183-189.

Clark, A.R. & B.F. Walker 1977. A proposed scheme for the classification and nomenclature for use in the engineering description of Middle Eastern sedimentary rocks. Technical Note. *Geotechnique*: 27, 93-99.

Clark, G.B. 1987. *Principles of rock fragmentation*. John Wiley & Sons, New York. 610 pp.

Cools,P.M.C.B.M. 1993. Temperature measurements upon the chisel surface during rock cutting. *Int. J. Rock Mech. Min. Sci. & Geomech. Abstr.*: 30, 25-35.

Cottington, J. & R. Akenhead 1984. *Site investigation and the law*. Thomas Telford Ltd, London. 184 pp.

Davids, S.W. & P. Adrichem 1990. The testing of a classification apparatus for the wear of cutter heads during cutting of rock (in Dutch). *Classified internal report, Faculty of Mechanical and Marine Engineering. no. 90.3.GV.2671*: 100 pp. Delft University of Technology.

Deere, D.U. 1964. Technical description of rock cores for engineering purposes. *Rock Mechanics and Rock Engineering:* 1, 17-22.

Degen, B.T.A.J., L. LeBlanc & P.M. Maurenbrecher 1995. Formations and geotechnical parameters from reflection surveys. *Proceedings 11th European Conf. Soil mech. & Foundation Engng., Kopenhagen, DGF Bulletin:* 11, 1.85-1.90. Danish Geotechnical Society, Kopenhagen.

Deketh, H.J.R. 1991. Determination of optimum conditions for the modified pin-on-disc test. *Memoirs of the Centre for Engineering Geology in the Netherlands:* 85, 27 pp. & appendices. Delft University of Technology.

Deketh, H.J.R. 1993. Rock properties relevant for the assessment of abrasiveness and excavatability of rock, applied to dredging. *Proceedings CEDA Dredging Days, November 18-19, Amsterdam*: E4, 14pp.

Deketh, H.J.R. 1995. *Wear of rock cutting tools. Laboratory experiments on the abrasivity of rock.* Balkema. Rotterdam. 144 pp.

Deketh, H.J.R. 1995b. The wear sensitive cutting principle of a cutter suction dredger. *Terra et Aqua:* 60, 3-7.

Deketh H.J.R. 1995c. Wear of the cutting tools of rock excavation machines. *Proceedings of the 35th US Symposium of Rock Mechanics, 5-7 June, Reno, USA:* 342-347. Balkema, Rotterdam.

Deketh, H.J.R. & P.N.W. Verhoef 1993. Fundamental aspects of abrasive wear of cutting tools by rock. *Proceedings XIV Annual Meeting Western Dredging Association, May 25-28, Texas A&M University Centre for Dredging Studies Report*: 327, 281-294.

Deketh, H.J.R., D.D. Genske & P.N.W. Verhoef 1995. Rock properties affecting the sensitivity of prediction equations for rock cutting tool wear. *Proceedings of the 8th International Congress on Rock Mechanics, Tokyo, Japan*: 1, 19-21. Balkema, Rotterdam.

Deketh, H.J.R., M. Giezen, M. Alvarez Grima & M. den Hartog 1996. Performance of rock cutting trenchers at different sites. *Proceedings 2nd North American Rock Mechanics Conference, Montreal:* 2, 669-675. Balkema Rotterdam.

Deliac, E.P. 1986. *Optimization des machines d'abbatage a pics.* PhD thesis, University of Paris VI, 501 pp.

Deliac, E.P. 1993. Theoretical and practical rules for mechanical rock excavation. In: J.A. Hudson (ed.) *Comprehensive Rock Engineering, vol. 4, Excavation, support and monitoring*: 178-227. Permagon Press, Oxford.

Den Hartog, M.H. 1996. T-850 Rock cutting trencher performance and fuzzy logic. *Memoirs of the Centre for Engineering Geology in the Netherlands:* 132, pp.162. Delft University of Technology.

Den Hartog, M.H., R. Babuska, H.J.R. Deketh, M. Alvarez Grima, P.N.W. Verhoef & H.B. Verbruggen 1997. Knowledge-based fuzzy model for performance prediction of a rock cutting trencher. *Int. J. of Approximate Reasoning*: 16, 43-66.

DIN 50 320 1979: Verschleiß. Begriffe, Systemanalyse von Verschleißvorgängen. Gliederung des Verschleißgebietes. (Wear. Definitions, system analysis of wear occurrences. Classification of the wear field). *Deutsches Institut für Normung e.V., Berlin*: 8 pp.

DIN 50 321 1979. Verschleiß - Meßgrößen. (Wear - quantification). *Deutsches Institut für Normung e.V., Berlin*: 6 pp.

Driesch, S. von den 1994. Schneidtechnische und verfahrentechnische Weiterentwicklungen für den maschinellen Vortrieb. (Operational and cutting technical developments for tunnelling machines). *Glückauf-Forschungshefte*: 55, 156-160.

Engelhardt, W. von & S. Haussühl 1965. Festigkeit und Härte von Kristallen. (Strength and hardness of crystals). *Fortschritte der Mineralogie*: 42, 5-49.

Ersoy, A. & M.D. Waller 1995. Textural charcterisation of rocks. *Engineering Geology*: 39, 123-136

Evans, I. 1962. A theory on the basic mechanics of coal ploughing. In G.B. Clark (ed): *International Symposium on Mining Research, February 1961, Missouri*: vol. 2, 761-798.

Evans, I. 1965. The force required to cut coal with blunt wedges. *Int.J. Rock Mech. Mining Sci.*: 2, 1-12.

Evans, I. 1972. Line spacing of picks for efficient cutting. *Int.J. Rock Mech. Mining Sci.*: 9, 355-361.

Evans, I. 1984. A theory of the cutting force for pointed attack picks. *Int. J. Min. Engng.*: 2, 63-71.

Evans, I. & C.D. Pomeroy 1966. *The strength, fracture and workability of coal.* Permagon Press, Oxford. 277 pp.

Ewendt,G. 1989. Erfaßung der Gesteinsabrasivität und Prognose des Werkzeugversleißes beim maschinellen Tunnelvortrieb mit Diskenmeiseln. (Determination of rock abrasivity and prediction of tool wear of disc cutter tunnelling machines). *Bochumer geologische und geotechnische Arbeiten: Heft 33.* Institüt für Geologie, Ruhr -Universität, Bochum.

Fairhurst C.R. 1987. *Theory and practice of enhanced rock cutting picks: the water-jet assisted tool and the vibrating tool.* Thesis L'Ecole nationale supérieure des mines de Paris.

Farmer, I. 1983. *Engineering behaviour of rocks.* Chapman and Hall. London. 208 pp.

Fell, R. 1990. Engineering geophysics- a civil engineer's viewpoint. *Exploration Geophysics*: 21, 25-31.

Fowell, R.J. & S.T. Johnson 1991. Cuttability assessment applied to drag tool tunnelling machines. *Proc.ISRM International Congress on Rock Mechanics, Aachen, Germany, Vol. 2:* 985-990. Balkema, Rotterdam

Franklin, J.A., E.Broch & G. Walton 1971. Logging the mechanical character of rock. *Trans. Inst. Min. Metall.*: 80, Section A- Mining Industry, A1-9.

Gehring, K. 1987. Rock testing procedures at VA's geotechnical laboratory in Zeltweg. *Internal report TZU 41.* Voest Alpine Zeltweg, Austria. 11 pp.

Geyh, M.A., H.R. Kudrass & H. Streif 1979. Sea-level changes during the late Pleistocene and Holocene in the Strait of Malacca. *Nature*: 278, 441-443.

Giersch, D. 1989. Dredge teeth, design and economy. Wilson, J.R. & V.L. van Dam (eds.) *Proceedings XIIth World Dredging Congress, Orlando, Florida, USA*: 870-889.

Giezen, M. 1993. Rock properties relevant to tool wear and production of rock cutting trenchers. A pilot study. *Memoirs of the Centre of Engineering Geology in The Netherlands*: 110, 75 pp. + App. Delft University of Technology.

Goodman, R.E. (1989). *Introduction to rock mechanics.* 2nd ed., John Wiley & Sons, New York. 562 pp.

Grant, W.H. 1963. Weathering of Stone Mountain granite. In: *Clays and clay minerals. Proceedings 11th national conference, Ontario, August 1962*: 65-73.

Gregor, M. 1968. *Experimentelle und theoretische Untersuchungen über den Einfluß der Schnittgeschwindigkeit auf den Zerspanungswiderstand von Kohle.* (Experimental and theoretical investigations into the influence of cutting velocity on the cutting forces of coal). Thesis RWTH Aachen. 88 pp.

Gregor, M. 1970. Der Einfluß der Schnittgeschwindigkeit auf Schnitt- und Andrückkraft beim Zerspanen von Kohle. (Influence of cutting velocity on the cutting- and thrust forces of coal). *Glückauf Forschungshefte*: 29, 179-188.

Habig, K.H. 1980. *Verschleiß und Härte von Werkstoffen.* (Wear and hardness of materials). Carl Hanser Verlag, München. 303 pp.

Hack H.R.G.K. 1996. *Slope stability probability classification SSPC.* PhD thesis. ITC publication number 43. pp. 258. Delft, The Netherlands.

Hadjigeorgiou, J. & M.J. Scoble 1990. Ground characterization for assessment of ease of excavation. In: Singhal, R.K. & M. Varva (eds.): *Proceedings of the 4th Int. Symposium on Mine Planning and Equipment Selection, Calgary:* 323-331. Balkema, Rotterdam

Hata, S. & T. Muro 1978. Relations between wear life of rippertip and rock mass properties. *Proc. of the 6th Int. Conf. Int .Soc. Terrain Vehicle Soc., Vienna:* 951-981.

Hawkins, A.B. & B.J. McConnell 1992. Sensitivity of sandstone strength and deformability to changes in moisture content. *Quarterly Journal of Engineering Geology, London*: 25, 115-130.

Heer, R.J. de & Rochmanhadi 1989. Dredging and dredging equipment. Part 2, dredging equipment. *Lecture notes International Institute for Hydraulic Engineering, Delft:* 353 pp.

Herbert, C. 1976. The depositional development of the basin. In: Branagan, D.F. et.al.. *An outline of the geology and geomorphology of the Sydney Basin. Science Press, Sydney*: 5-38.

Herbich, John B. 1992. *Handbook of dredging engineering*. McGraw-Hill, New York, 600 pp.

Hergarden, I.M. 1997. The influence of the ductility of rock on the performance of rock cutting trenchers. *Memoirs of the Centre of Engineering Geology in The Netherlands*: 143, 130 pp. Delft University of Technology.

Hoek, E. 1983. Strength of jointed rock masses. *Géotechnique:* 33, 187-223.

Hoek, E. & E.T. Brown 1980. *Underground excavations in rock.* Institution of Mining and Metallurgy, London. 527 pp.

Hoek, E. & E.T. Brown 1988. Determination of shear failure envelope in rock masses. *Journ. Geotechn. Engng.*: 114, 371-373.

Hoek, E., P.K. Kaiser & W.F. Bawden 1995. *Support of underground excavations in hard rock.* Balkema, Rotterdam. 215 pp.

Holland, R. 1991. Too much or too little?". *In: "Capital Dredging". Proceedings of conference, Institution of Civil Engineers, Edinburgh, 15-16 May*: 1-13. Thomas Telford Ltd, London.

Hoogerbrugge, W.J. 1980. The influence of discontinuities on the excavation of rock by disc bottom cutter dredging. *Memoirs of the Centre of Engineering Geology in The Netherlands*: 3, 109 pp. Delft University of Technology.

Hornung, J. 1978. Modelluntersuchungen über die Vorgänge beim maschinellen Reissen von Fels (Model tests on the mechanical ripping of rock). *Baumachine und Bautechnik:* 25, 167-173.

Houlding, S.W. 1994. *3D Geoscience modeling. Computertechniques for geologicalcharacterization.* 309 pp. Springer-Verlag, Berlin

Hurt, K.G. & K.M. MacAndrew 1985. Cutting efficiency and life of rock-cutting picks. *Mining Science and Technology*: 2, 139-151.

IHC 1994. How to select stationary dredging equipment. *Ports and Dredging*: 142, 18 -21.

Irfan, T.Y. 1988. Fabric variability and testing of a granitic saprolite. In: *Geomechanics in tropical soils. Proc. 2nd int. conf. on geomech. in trop. soils, Singapore*: 1, 25-35. Balkema, Rotterdam

ISRM 1978. Suggested methods for determining the strength of rock in triaxial compression. *Int. J. of Rock Mech. and Mining Sciences & Geomech. Abstracts*: 15, 47-51.

ISRM 1978. Suggested methods for determining tensile strength of rock materials. *Int. J. of Rock Mech. and Mining Sciences & Geomech. Abstracts*: 15, 99-103.

ISRM 1979. Suggested methods for determining the uniaxial compressive strength and deformability of rock materials. *Int. J. of Rock Mech. and Mining Sciences & Geomech. Abstracts*: 16, 135-140.

ISRM 1981. Basic geotechnical description of rock masses. *Int. J. Rock Mech. Min. Sci. & Geomech. Abst.*: 18, 85-100.

ISRM 1985. Suggested method for determining point load strength. *Int. J. Rock Mech. Min. Sci. & Geomech. Abst.*: 22, 61-70.

Jager W. 1988. An investigation into the abrasive capacity of rock. *Memoirs of the Centre of Engineering Geology in The Netherlands*: 52, 99 pp. Delft University of Technology.

Janach, W. & A. Merminod 1982. Rock abrasivity test with a modified Schmidt hammer. *Int. J. Rock Mech. Min. Sci. & Geomech. Abst.*: 19, 43-45.

Johannessen, O. 1990. *Drillability, drilling rate index catalogue*. Report 13-90. The University of Trondheim, The Norwegian Geotechnical Institute of Technology, Division of Construction Engineering & Department of Geology. 170 pp.

Johnston, I.W. 1985. Strength of intact geomechanical materials. *Journal of geotechnical engineering*: 111, 730-743.

Juhász, A.Z. & L. Opoczky 1990. *Mechanical activation of minerals by grinding. Pulverizing and morphology of particles.* Akadémiai Kiadó, Budapest. 234 pp.

Kenny, P. & S.N. Johnson 1976a. An investigation of the abrasive wear of mineral cutting tools. *Wear*: 36, 337-361.

Kenny, P. & S.N. Johnson 1976b. The effect of wear on the performance of mineral-cutting tools. *Colliery Guardian*: June, 256-252.

Kearey P. & M. Brooks 1984. *An introduction to geophysical exploration.* Blackwell Scientific Publications, Oxford. pp. 296.

Kesteren, W.G.M. van, H.J.M.G. Steeghs, D.R. Mastbergen 1992. Pore water behaviour in dredging processes. *proc. XIIIth World Dredging Congress WODCON, Bombay India*: 597-615.

Kesteren, W.G.M. van 1995. Numerical simulations of crack bifurcation in the chip forming cutting process in rock. In G. Baker & B.L. Karihaloo *Fracture of brittle disordered materials: concrete, rock and ceramics*: paper 33, 505-524. E & FN Spon, London.

Kirsten, H.A.D. 1982. A classification system for excavation in natural materials. *The Civil Engineer in South Africa*: 24, 293-306.

Koert, J.P. 1981. *Trillend snijden van gesteente.* (Vibrational cutting of rock). MSc thesis. Faculty of Mechanical and Marine Engineering, Delft University of Technology, CO/81/126, 88 pp.

Kok, M. de, W. Dirks & R. Hessels 1997. The Oresung fixed link: dredging and reclamation. *Terra et Aqua*: 66, 16-24.

Koning, J. de 1968. Boundary conditions for the use of dredging equipment. Paper *Course of dredging engineering in coastal waters and estuaries, May 1968.* Delft/The Hague.

Koning, J. de 1993. *Terugblik naar de toekomst (Looking back at the future).* Lecture Delft University of Technology. 40 pp.

Krapivin, M.G., V.M. Manakov & V.G. Michajlov 1967. Untersuchungen der Temperaturen und ihres Einflusses auf den Verschleiß der Schneidwerkzeuge von Vortriebsmachinen beim schneiden in Sandstein (translated from Russian into German). *Izvest VUZ, Gorn*: 10, 84-89.

Landheer, D. 1983. Adhesie, wrijving, slijtage. (Adhesion, friction and wear). In: *Tribotechniek en aandrijvingen. Lecture notes Mechanical Engineering 4B51.0.* Eindhoven University of Technology.

Lowe, P.T. & L.B. McQueen 1988. Ground conditions and construction methods in the Malabar Ocean Outfall Decline and observations on rock cuttability. *Proc. Australian Underground Construction and Tunnelling Association Seminar on Cost Effective Tunnelling in the Sydney Basin. IE Aust., Canberra*: 165-203.

Lowe, P.T. & L.B. McQueen 1990. Construction of the North Head Ocean Outfall Tunnel. *Proc. VII Australian Tunnelling Conference, The Underground Domain. IE Aust., National Conf. Publ*: 90/8, 159-172.

MacGregor, F., R. Fell, G.R. Mostyn, G. Hocking & G. McNally 1994. The estimation of rock rippability. *Quarterly Journal of Engineering Geology, London*: 27, 123-144.

Martin, J.M. 1986. Predicting the rippability of sandstone in SE Queensland. *13th ARRB/5th REAAA Conf.*: 119-132.

Mashloosh, K.M. & T.S. Eyre 1985. Abrasive wear and its application to digger teeth. *Tribology international*: 18, 259-266.

Maurenbrecher, P.M., B.T.J.A. Degen, D.G. Price & P.N.W. Verhoef 1992. Marine investigation for dredging works. Course notes. *Memoirs of the Centre for Engineering Geology in the Netherlands*: 99, 90 pp.

McFeat-Smith, I. & R.J. Fowell 1977. Correlation of rock properties and the cutting performance of tunnelling machines. *Proc. Conf. on Rock Engineering, University Newcastle upon Tyne*: 587-602.

Mens, J.W.M. & A.W.J. de Gee 1986. Erosion in seawater sand slurries. *Tribology international*: 19, 59-64.

Miedema, S.A. 1987. *Berekening van de snijkrachten bij het snijden van volledig met water verzadigd zand (Calculation of the cuttingforces during the cutting of completely saturated sand).* PhD thesis, Delft University of Technology, Faculty of Mechanical and Maritime Engineering, 268 pp.

Miedema, S.A. 1990. *The development of an apparatus for the classification of wear while dredging solid rock* (in Dutch). Classified internal report, Faculty of Mechanical and Marine Engineering, Delft University of Technology, no. 90.3.GV.2749, 30 pp.

Minty, E.J. & G.K. Kearns 1983. Rock mass workability. *Geological Society of Australia, Special Publication:* 11, 59-81.

Mishnaevsky Jr, L.L. 1995. Physical mechanisms of hard rock fragmentation under mechanical loading: a review. *Int. J. Rock Mech. Min. Sci. & Geomech. Abstr.*: 32, 763-766.

Mishra, A. & I. Finnie 1980. A classification of three-body abrasive wear and design of a new tester. *Wear*: 60, 111-121.

Mogi, K. 1966. Pressure dependence of rock strength and transition from brittle fracture to ductile flow. *Bull. Earthquake Res. Inst., Japan*: 44, 215-232.

Movinkel, T. & O. Johannessen 1986. Geological parameters for hard rock tunnel boring. *Tunnels & Tunnelling* April 1986: 45-48.

Muro, Tatsuro 1985. Abrasive wear resistance of surface coatings on an excavating tip against a rock mass. *Journal of Terramechanics*: 22, 87-109.

Natau, O., Th. Mutschler & Ch. Lempp 1991. Estimation of the cutting rate and the bit wear of partial-face tunnelling machines. *Proc.ISRM International Congress on Rock Mechanics, Aachen, Germany, Vol. 3:* 1591-1595. Balkema, Rotterdam.

Nelson, P.P. 1993. TBM performance analysis with reference to rock properties. In: J.A. Hudson (ed.) *Comprehensive Rock Engineering, vol. 4, Excavation, support and monitoring*: 261-291. Permagon Press, Oxford.

Nelson, P.P. 1996. Rock engineering for underground civil construction. In: *M.Aubertin et al (eds.) Rock Mechanics, Tools and Techniques. Proceedings 2nd North American Rock Mechanics Conference, Montreal, June 1996:* 1, 3-12. Balkema Rotterdam.

Netterberg, F. 1994. Engineering geology of pedocretes and other residual soils. *Proceedings 7th International IAEG Congress:* p. XIX-XXXVIII. Balkema, Roterdam. >

Nishimatsu, Y. 1972. The mechanics of rock cutting. *Int.J. Rock Mech. Mining Sci.*: 9, 261-270.

Nishimatsu, Y. 1979. On the effect of tool velocity in the rock cutting. *International Conference on Mining and Machinery, Brisbane, 2 - 6 July*: 314-319.

Novello, E.A. & I.W. Johnston 1995. Geotechnical materials and the critical state. *Géotechnique*: 45, 223-235.

Ockeloen, J.J. 1997. Experiments to determine the transition of brittle to ductile rock failure at room temperature. MSc thesis, *Memoirs of the Centre for Engineering Geology in The Netherlands:* 94, pp. 43 + App.

Orlić B. 1997. *Predicting subsurface conditions for geotechnical modelling. PhD thesis.* ITC publication: 55, Delft.

Orlić B. & J.W. Rösingh 1995. Three-dimensional geomodelling for offshore aggregate resources assessment. *Q. J. of Engineering Geology*: 28, 385-391.

Osburn, H.J. 1969. Wear of rock cutting tools. *Powder Metallurgy*: 12, 471-502.

Ottmann, F.G. & G. Lahuec 1976. Dredging and geology. *Terra et Aqua*: 11, 26-34.

Palermo, M.R. & D.F. Hayes 1992. Environmental effects of dredging. In: J.B. Herbich (ed) *Handbook of ocean engineering, vol.3, Harbors, navigational channels, estuaries, environmental effects*: 1169-1175.

Pariseau, W.G. 1970. Wedge indentation of anisotropic media. *Dynamic rock mechanics. Proceedings 12th Symposium on rock mechanics, Rolla, Missouri, USA*: 529-546.

Pariseau, W.G. 1994. On the significance of dimensionless failure criterea. *Int. J. Rock Mech. Min. Sci. & Geomech. Abst.*: 31, 555-560.

Paschen, D. 1980. *Petrographische und geomechanische Charakterisierung von Ruhrkarbongesteinen zur Bestimmung ihres Verschleißverhaltens (Petrographic and geomechanical characterisation of Ruhr area Carboniferous rocks for the determination of their wear behaviour).* PhD dissertation, Technische Universität Claustahl. pp. 202.

PBNA. *Voortgezette opleiding uitvoering baggerwerken, deel 5. Snijkopzuiger, winzuiger, bakkenzuiger (Advanced course execution of dredging works, part 5. Cutter suction dredger, suction dredger).* Vereniging "Centrale Baggerbedrijf". Undated lecture notes, probably early 1970's.

Pells, P.J.N. 1985. Engineering properties of the Hawkesbury Sandstone. In Pells, P.J.N. (ed.). *Engineering Geology of the Sydney Region:* 179-197. Balkema, Rotterdam.

Pells, P.J.N. 1993. Uniaxial strength testing. In: J.A. Hudson (ed.) *Comprehensive Rock Engineering, vol. 3, Rock testing and site characterization*: 67-85. Permagon Press, Oxford.

Petterson, D. & K.G. Wijma 1997. Lowering the overall costs for the use of rock cutterhead systems. *Proceedings CEDA Dredging Days, 20-21 November 1997, Amsterdam.* CEDA, Delft, The Netherlands.

Pettifer, G.S. & P.G. Fookes 1994. A revision of the graphical method for assessing the excavatability of rock. *Quarterly Journal of Engineering Geology, London*: 27, 145-164.

Phillips H.R & F.F. Roxborough 1981. The influence of tool material on the wear rate of rock cutting picks. *Proceedings 34th annual conference, Brisbane, Queensland. Australian Institute of Metals:* 52-56.

PIANC 1984. Classification of soils and rocks to be dredged. Supplement to Bulletin No. 47, *Permanent International Association of Navigation Congresses, Brussels.* 16 pp.

Piepers, T.P.H. 1995. The significance of rock strength index tests applied to rock dredging. *Memoirs of the Centre for Engineering Geology in the Netherlands*: 125, pp. 93. Delft, The Netherlands.

Pierry, J. & R. Charlier 1994. Finite element modelling of shear band localisation and application to rock cutting by a PCD tool. In: *Rock Mechanics in Petroleum Engineering. Proceedings Eurock'94 SPE/ISRM Conference, Delft, 29-31 August*: 199-204. Balkema, Rotterdam.

Pillans, B. 1987. Quaternary sea-level changes: Southern Hemisphere data. In Devoy, R.J.N. (ed.) *Sea surface studies*: Chapter 9.

Price, D.G. 1981. Report of the IAEG commission on site investigations. *Bull. Int. Ass. of Engineering Geology*: 24, 185-226.

Price, D.G. 1990. Panel report: Erosion and sedimentation in harbour projects - dredging and hydraulic filling. *Proceedings 5th Int. Congress Int. Ass. of Eng. Geol., Buenos Aires:* 8, 2699-2705. Balkema, Rotterdam.

Price, D.G. 1991. *Engineering Geology: Site Investigation.* Lecture notes, Delft University of technology. pp. 209.

Price, D.G. 1993. A suggested method for the classification of rock mass weathering by a ratings system. *Q. J. of Engng. Geology*: 26, 69-76.

Price, D.G. 1995. Weathering and weathering processes. *Q. J. of Engng. Geology*: 28, 243-252.

Priest, S.D. 1985. *Hemispherical projection methods in rock mechanics.* George Allen and Unwin, London.

Priest, S.D. 1993. *Discontinuity analysis for rock engineering.* Chapman & Hall, London.

Prudhomme, R.J., M.L. Rizzone, T.H. Schmieman & J.E. Miller 1970. Reciprocating pumps for long distance slurry pipelines. *Hydrotransport 1, International Conference on the hydraulic transport of solids in pipes, Coventry, England:* E4, 49-59.

Prothero, D.R., & F. Schwab 1996. *Sedimentary geology.* W.H. Freeman and Company.

Reinking, M.W. 1989. Unpublished Cerchar test results. Section of Engineering Geology, Delft University of Technology.

Reinking, M.W. 1989. Linear cutting tests in artificial sand-clay mixtures to study the influence of brittleness on abrasive wear of cutting tools. *Memoirs of the Centre for Engineering Geology in the Netherlands*: 75, pp.95.

Reinking, M.W. 1990. Linear cutting tests in artificial sand-clay mixtures to study the influence of brittleness on abrasive wear of cutting tools. *Proc. 6th Int. Congress IAEG, Amsterdam*: 2913-2918. Balkema, Rotterdam.

Rix, P.F. 1971. *Untersuchungen über den Spannwiderstand von spröd elastischen Gesteinen in Abhängigkeit von der Meißelbreite und der Spantiefe unter besonderer Berücksichtigung des Bruchvorganges.* (Investigations into the cutting resistance of brittle elastic rocks, depending on chisel width and cutting depth, with special emphasis on fracture propagation). Thesis RWTH Aachen.

Rosiwal, A. 1896. Neue Untersuchungsergebnisse über die Härte von Mineralien und Gesteinen. (New research findings on the hardness of minerals and rocks). *Verhandlungen der königl. geol. Reichsanstalt, Wien*: 17 & 18, 475-491.

Rosiwal, A. 1916. Neuere Ergebnisse der Hartebestimmung von Mineralien und Gesteinen -Ein Absolutes Maß für die Härte spröder Körper. (New findings on the hardness determination of minerals and rocks - an absolute measure for the hardness of brittle materials). *Verhandlungen der königl. geol. Reichsanstalt, Wien*: 5 & 6, 117-147.

Rossen, H.P.H. van 1987. *Ananlyse van snij- en slijtage processen van beitels bij het snijden van vaste gesteenten.* (Analysis of cutting and wear processes of chisels cutting in solid rock). MSc thesis. Faculty of Mechanical and Marine Engineering, Delft University of Technology, 87.3.GV.2232, 80 pp.

Roxborough, F.F. 1973. Cutting rock with picks. *The mining engineer*: June 1973.

Roxborough, F.F. 1982. *An assessment of the cuttability of rock formations associated with the Sydney MWS & DB Submarine Outfall Tunnels*. Unisearch Limited Report for The Metropolitan Water Sewerage and Drainage Board. June 1982. 81 pp.

Roxborough, F.F. 1987. The role of some basic rock properties in assessing cuttability. *Proceedings Seminar "Tunnels - Wholly Engineered Structures". IE Aust.* April.

Roxborough, F.F. & H.W. Phillips 1974. Experimental studies on the excavation on the rock using picks. *International Society of Rock Mechanics, Advances in Rockmechanics, Vol. II, part B. Proc. of the 3rd Congress, Denver.*

Roxborough, F.F. & G.C. Sen 1986. Breaking coal and rock. In C.H. Martin (ed.) *Australian coal mining practice, AIMM Monograph Series:* 12, 130-147.

Sandbak, L.A. 1985. Roadheader drift excavation and geomechanics rock classification. *Proc. Rapid Excav. Tunnelling Conf., AIME, New York*: vol.2, 902-916.

Schatzov, N.I. 1964. *Bohren auf Erdöl und Erdgas, Band I.* VEB Deutscher Verlag für Grundstoffindustrie, Leipzig. 295 pp.

Schimazek J. & H. Knatz 1970. Der Einfluß des Gesteinsaufbaus auf die Schnittgeschwindigkeit und den Meißelverschleiß von Streckenvortriebs-machinen. (The influence of rock composition on cutting velocity and chisel wear of tunnelling machines). *Glückauf:* 106, 274-278.

Schimazek J. & H. Knatz 1976. Die Beurteiling der Bearbeitbarkeit von Gesteinen durch Schneid- und Rollenbohrwerkzeuge. (The assessment of the cuttability of rocks by drag- and roller bits). *Ertzmetall*: 29, 113-119.

Scholz, C.H. 199O. *The mechanics of earthquakes and faulting*. Cambridge University Press. 439 pp.

Schweins, B. 1986. Verschleiß im Braunkohlentagebau. (Wear in browncoal mining). In H. Uetz, *Abrasion und Erosion:* 598-637. Carl Hauser Verlag, München.

Schofield, A. & P. Wroth 1968. *Critical state soil mechanics.* McGraw-Hill, London. pp. 310.

Scoble, M.J. & Y.V. Muftuoglu 1984. Derivation of a diggability index for surface mine equipment selection. *Mining Science and Technology*, 1, 305-322.

Shulin Xu, De Freitas, M.H. & B.A. Clark 1988. The measurement of tensile strength of rock. In Romana (ed.) Rock Mechanics and Power Plants. *Proceedings ISRM Symposium, Madrid*: 125-132. Balkema, Rotterdam.

Siezen, M.R. 1996. A three-dimensional dredgeability classification system. A case study for the Oresund Link Tunnel dredging project. *Memoirs of the Centre for Engineering Geology in the Netherlands*: 140, pp. 118. Delft, The Netherlands.

Sindhusen, S. 1991. *Rock abrasivity.* MSc thesis University of New South Wales, Department of Applied Sciences. pp. 97. Sidney, Australia.

Singh, R.N., B. Denby, I. Egretly & A.G. Pathan 1986. Assessment of ground rippability in open cast mining operations. *University of Nottingham, Mining Department Magazine:* XXXVIII, 21-34.

Singh, R.N., B. Denby & I. Egretly 1987. Development of a new rippability index for coal measures excavations. *27th US Symposium on Rock Mechanics*: 935-943. Balkema, Rotterdam.

Sman, R.M. van der 1988. *Investigation into the concept of a test apparatus for chisel wear* (in Dutch). Classified internal report, Faculty of Mechanical and Marine Engineering, Delft University of Technology, 38 pp.

Sman, R.M. van der 1988. *Results of a first series of wear tests on chisels with the shaper* (in Dutch). Classified internal report, Faculty of Mechanical and Marine Engineering, Delft University of Technology, 89.3.GV.2667, 67 pp.

Smith, H.J. 1986. Estimating rippability by rock mass classification. *27th US Symposium on Rock Mechanics*: 443-448. Balkema, Rotterdam.

Smith, H.J. 1987. Estimating the mechanical dredgeability of rock. *28th US Symposium on Rock Mechanics*: 945-952. Balkema, Rotterdam.

Speight, H.E. & R.J. Fowell 1987. The influence of operational parameters of roadheader productivity and efficiency with particular reference to cutting pick wear. *Conference on tribology in mineral extraction, wear on wear, Nottingham University:* 175-182. I.Mech.E., London.

Speight, H.E. & R.J. Fowell 1987. Roadheader performance studies using a full scale laboratory research facility. *Proceedings 28th US Symposium on Rock Mechanics, Tuscon*: 973-980. Balkema, Rotterdam.

Staveren, M.T. van 1987. Discontinuity studies for the OPAC project. *Memoirs of the Centre for Engineering Geology in the Netherlands*: 41.

Stone, M. 1991. Soil investigation. In: *Capital Dredging. Proceedings of conference, Institution of Civil Engineers, Edinburgh. 15-16 May 1991*: 47-62. Thomas Telford Ltd. London.

Suana, M. & Tj. Peters 1982. The CERCHAR abrasivity index and its relation to rock mineralogy and petrography. *Rock Mechanics*: 51, 1-8.

Swart, P.D. & T.M. McGee 1993. Marine seismic single-channel investigation techniques to improve dredging performance - a case study. *Proceedings CEDA dredging days, Amsterdam. Central Dredging Association, Delft:* paper F2, pp. 21.

Tamrock 1983. *Handbook of underground drilling.* Tamrock Oy, Tampere, Finland.

Tarkoy, P.J. 1973. A study of rock properties and tunnel machine advance rates in two mica schists formations. *15th Symp. Rock Mech, Custer State Park, South Dakota*: 415-447.

Tarkoy, P.J. 1973. Predicting tunnel boring machine (TBM) penetration rates and cutter costs in selected rock types. In: *Proceedings of the 9th Canada Rock mechanics Symposium, Montreal*: 263-274.

Tucker, M.E. 1981. *Sedimentary petrology, an introduction.* Blackwell Scientific Publishers. Oxford. 252 pp.

Tucker, M.E. & V.P.Wright 1990. *Carbonate sedimentology*. Blackwell Scientific Publishers. Oxford. 482 pp.

Thuro, K. & G. Spaun 1996. Drillability in hard rock drill and blast tunnelling. *Felsbau*: 14, 103-109.

TID (Training Institute for Dredging). Course notes: *Cutters & Some aspects of cutter suction dredging*. IHC. Kinderdijk, Holland.

Tourenq, C 1966. La dureté Vickers des minéraux et des roches, quelques applications (Vickers hardness of minerals and rocks, some applications). *Bul. Liaison Laboratoires des Ponts et Chaussées*: 19, réf 332

Uetz, H. 1986. *Abrasion und Erosion*. Carl Hauser Verlag, München. 829 pp.

Uittenbogaard, R., J. Cornelisse, P.M.C.B.M. Cools & D. Luger, in prep. Report on laboratory research on rock cutting in the context of dredging. Submitted to *Int. J. Rock Mech. Min. Sci. & Geomech. Abstr.*

Verbeek, P.R.H. 1984. Soil analysis and dredging. *Terra et Aqua*: 28, 11-21.

Verhoef, P.N.W. 1986. *The assessment of the abrasive capacity of rocks for dredging works* (in Dutch). Abstract. Lecture Ingeokring/CEDA, 12 May 1986.

Verhoef, P.N.W. 1987. Sandblast testing of rock. *Int. J. Rock Mech. Min. Sci. & Geomech. Abst.*: 24, 185-192.

Verhoef, P.N.W. 1988. Towards a prediction of the abrasive wear of cutting tools in rock dredging. *Delft Progress Report:* 13, 307-320.

Verhoef, P.N.W. 1993. Abrasivity of Hawkesbury Sandstone (Sydney, Australia) in relation to rock dredging. *Quarterly Journal of Engineering Geology, London*: 26, 5-17.

Verhoef, P.N.W. 1994. Wear testing categories for rock dredging projects. *6th. International IAEG Congress, Amsterdam, 1990*: 5, 3289-3292. Balkema, Rotterdam.

Verhoef, P.N.W., H.J. van den Bold & Th.W.M. Vermeer 1990. Influence of microscopic structure on the abrasivity of rock as determined by the pin-on-disc test. *Proc. 6th Int. Congress IAEG, Amsterdam*: 495-504. Balkema, Rotterdam.

Verhoef, P.N.W. & H.J.R. Deketh 1992. Factors determining abrasivity of rock, experience from rock dredging. *First International Symposium on Tunnel Construction and Underground Structures, Ljubljana, Slovenia, September 1992.* pp.: 10.

Verhoef, P.N.W. & H.J.R. Deketh 1994. Rock characterisation for dredging purposes. *Terra et Aqua*: 55, 4-13.

Verhoef, P.N.W., J.J. Ockeloen, W.G.M. van Kesteren 1996. The significance of rock ductility for mechanical rock excavation. In *M.Aubertin et al (eds.) Rock Mechanics, Tools and Techniques. Proceedings 2nd North American Rock Mechanics Conference, Montreal, June 1996:* 1, 709-716. Balkema Rotterdam.

Verkaik C.J. 1982. *Snijden van gesteente met een baggerwiel.* (Cutting of rock with a dredging wheel). MSc thesis. Faculty of Mechanical and Marine Engineering, Delft University of Technology, CO/82/126, 130 pp.

Vermeer, T.W.M. 1989. Investigation of the abrasive- and erosive wear capacity of rock related to rock cutting tools. *Memoirs of the Centre for Engineering Geology in the Netherlands*: 71, pp. 71. Delft, The Netherlands.

Verwaal, W. & A. Mulder 1993. Estimating rock strength with the Equotip hardness tester. *Int.J. Rock Mech. Mining Sci.*: 30, 659-662.

Vutukuri, V.S. 1974. The effect of liquids on the tensile strength of limestone. *Int.J. Rock Mech. Mining Sci.*: 11, 27-29.

Wasp, E.J., J.P. Kenny & R.L. Gandhi 1977. *Solid-liquid flow. Slurry pipeline transportation.* Trans Tech Publications. Clausthal.

Watson, J. 1990. Internal report Westham Dredging Company Pty. Ltd.

Weaver, J.M. 1975. Geological factors significant in the assessment of rippability. *The Civil Engineer in South Africa*: 17, 313-316.

Whittaker, B.N. R.N. Singh & G. Sun 1992. *Rock fracture mechanics.* Elsevier, Amsterdam.

Wit, K. de 1996. *Correlation between rock mechanical properties and dredgeability.* MSc thesis Faculty of Applied Sciences, Mining Engineering Department, Catholic University of Leuven. 113 pp. Leuven, Belgium.

West, G. 1986. An observation on Mohs' scale of Hardness. *Quaterly Journal of Engineering Geology*: 19, 203-205.

Yardley, E.D., S.N. Johnson & P. Kenny 1981. The wear of tools used for cutting coal and rock. *Proc. Int. Conf. on wear of metals, San Fransisco*: 468-477.

Zum Gahr, K.H. 1987. *Microstructure and wear of materials.* Elsevier, Amsterdam. 560 pp.

Notation

A = apparent area of contact (m^2)
A_r = real area of contact, or tribo contact surface (m^2)
AVS = Abrasion Value for Steel
B = bulking factor (-)
BWI = Bit Wear Index
BTS = Brazilian Tensile Strength (MPa)
CAI = Cerchar Abrasiveness Index
TS = unconfined Tensile Strength (MPa)
COV = Coefficient of Variation (standard deviation expressed in percentage) (%)
CSD = cutter suction dredger
c = cohesion (MPa)
C_f = compressibility of the rock material (fabric)(m^2/GN)
C_s = compressibility of mineral grains (m^2/GN)
C_w = compressibility of water(m^2/GN)
CLI = Cutter Life Index
D_c = cutter diameter (m)
D_s = diameter suction pipe (m)
D = face; vertical depth of cut of cutterhead (m)
d = maximum penetration per pick point in rock during cutting
DRI = Drilling Rate Index
E = energy (Nm)
E = modulus of deformation as determined by unconfined compression test (general modulus, includes both elastic and plastic deformation component, if not stated otherwise) (GPa)
F = Schimazek's wear value (Equivalent Quartz content x grain size x tensile strength) (N/mm)
F_c = cutting force (N)
$F_{c,m}$ = mean cutting force (N)
F_n = normal (thrust) force (N)
$F_{n,m}$ = mean normal (thrust) force (N)
F_f = friction force (N)
F_N = normal load on (sliding) surface (N)

F_r = resultant force
F_T = tangential load on (sliding) surface (N)
FI = Fracture Index (= fracture frequency; number of fractures per meter)
H_a = Hardness of abrasive
H_m = Hardness of tool material
HR = Rockwell Hardness
HV = Vickers indentation hardness (MPa), often expressed in "H_v" units (kg/mm^2) (HV = 9.8 x H_v)
H_v = Vickers Hardness (kg/mm^2)
I_f = discontinuity spacing index, calculated from three major discontinuity sets: (s1+s2+s3)/3 (m)
Is_{50} = diametral Point Load Strength normalised to core diameter of 50 mm
J_v = joint volume number (volumetric joint count). Sum of the number of joints per meter of each joint set in a rock mass (joints/m_3)
l = length (m)
m = Hoek-Brown constant describing curvature of failure envelope (-)
n = porosity (volume of voids divided by total volume) (-)
n_R is the number of teeth with identical position R on the cutterhead
P_c is power capacity (kW) of cutter
p = penetration hardness of metal (plastic yield strength) (N/m^2)
p = total isotropic stress $(\sigma_1+\sigma_2+\sigma_3)/3$ (MPa),
PLS = Point Load Strength (MPa)
Q = production (m^3)
Qtz eq = total mineral hardness, where hardness of minerals is expressed relative to quartz, using Rosiwal's hardness scale (vol. %)
R = position vector of tooth on cutterhead
R_n = radius of *n-th* tooth on cutterhead
Rsq = coefficient of discrimination
r^2 = coefficient of discrimination
r = regression coefficient
RMR = Rock Mass Rating
RTI = Rock Toughness Index (MPa)
RQD = Rock Quality Designation. Length of intact rock core > 100 mm divided by total length drilled in section of interest (%)
S_c = length of cutter (m)
s = cut length (m)
s = degree of fracturing of rock mass
s_r = total shear strength of asperities on sliding surface (N/m^2)
SPE = specific energy (MN/m^2), (MJ/m^3)
SPT = Standard Penetration Test
t = time
u = pore water pressure (MPa)
UCS = Unconfined Compressive Strength (MPa)
(U)TS = Unconfined Tensile Strength (MPa)
TBM = Tunnel Boring Machine
V = volume of excavated soil or rock, in-situ (m^3)
V_h = haul velocity (swing speed) (m/min)
V_t = tangential velocity tooth on cutterhead (m/s)

v = velocity (m/s)
VHNR = Vickers Hardness Number of Rock; proportional summation of HV of minerals constituting the rock (MPa)
W_z = Work of failure, specific work of destruction (MPa)
α = rake angle (°) cutting tool
β = clearance angle (°) cutting tool
γ = wedge angle cutting tool (°)
$\gamma_{d,d}$ = dry volumetric weight in disposal area (kN/m^3)
$\gamma_{d,i}$ = dry volumetric weight in situ (kN/m^3)
Θ = conus angle pick point (°)
θ_f = forward breakout angle of chip (°)
θ_s = sideways breakout angle of chip (°)
σ = standard deviation
σ = normal stress (MPa)
σ_c = unconfined compressive strength (UCS) (MPa)
σ_n = normalized normal stress (σ/σ_c) (-)
σ_t = tensile strength (MPa)
σ_u = unconfined compressive strength (MPa)
σ_1 = major principal stress (MPa)
σ_3 = minor principal stress (MPa)
$\varnothing$ = diameter grain, grain size (mm)
μ = friction coefficient
μ = average or mean value
η = efficiency; delivered power / used power (-)
τ = shear stress (MPa)
τ_n = normalized shear stress (τ/σ_c) (-)
τ_u = unconfined shear stress (cohesion) (MPa)
ϕ = angle of internal friction (°)
$\phi^{r,t}$ = angle of friction between rock and tool (°)
ω = angular velocity (revolutions per minute, rpm)

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