THE CURRENT STATE OF THE ART OF ROCK CUTTING AND DREDGING

by

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United Kingdom

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Final Report

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<td>As shipping channels are deepened to accommodate the increasing size of oil and container carriers, dredging requirements are changing from predominantly excavation of alluvial sands and silts to strongly cohesive clays and rock, an area of dredging on which there is little published information or guidelines for contract preparation. As many excavation problems in tunneling, on which there is much more information, are interrelated to rock dredging particularly in the soft to medium rock range, this report is concerned with... (Continued)</td>
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the possible technology transfer to be gained from the tunneling industry that may assist the growth of technology in the dredging industry. Considered are site investigation, rock cutting, and contracts.
PREFACE

This report was prepared by Dr. H. J. Hignett, Camborne School of Mines, Cornwall, Great Britain, under Contract R&D 4594-EN-09 established by the U. S. Army Research, Development and Standardization Group-UK. Funds were provided from the ongoing work unit "Special Studies for Civil Works Rock Problems" under the CWIS Material-Rock Research Program, CWIS Work Unit 31200, sponsored by the Office, Chief of Engineers (OCE), U. S. Army. The technical monitor of the Program is Mr. Paul R. Fisher, OCE; the program manager is Dr. Don C. Banks, U. S. Army Engineer Waterways Experiment Station (WES). Preparation of this report was under the general direction of Dr. Banks and Dr. W. F. Marcuson III, Chief, Geotechnical Laboratory, WES.

The report draws heavily upon the author's experience, knowledge of tunnelling and dredging, European practice and site visitation to the U. S. Army Engineer Division, South Atlantic, and U. S. Army Engineer Districts, Savannah, Jacksonville, and New York.

Commander and Director of WES during the preparation of this report was COL Tilford C. Creel, CE. The Technical Director was Mr. Fred R. Brown.
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1. Claims, counterclaims, and the threat of litigation are all too prevalent these days in dredging. As shipping channels are widened and deepened to accommodate the increasing size of oil and container carriers, dredging requirements are changing from predominantly excavation of alluvial sands and silts to excavation of strongly cohesive soil and rock. Development of dredging equipment appears not to have kept pace with requirements; most research and development is carried out in isolation with all the usual competitiveness and secrecy of vested commercial interests. Hence, in the present state of the art the paucity of data makes it difficult for judgement to be based on sound technical assessment.

2. When the plant supplied fails to meet the planned production, more often than not the contractor claims for adverse site conditions, claiming that conditions were far more difficult than was distinguishable at the time of tendering from the site investigation data supplied. The contractor invariably wins his claim, or a substantial portion of it, as he has one major advantage over the client, or the client's agent. The contractor has incrementally removed all the ground in question; he has the advantage of hindsight. Moreover, as very few contracts are monitored daily for the client and standard agreed tests carried out, or an operational norm established for the plant, the contractor's claims are difficult to refute.

3. A further drawback to the client is that, in the preparing of contract documents, the assessing of ground conditions from a dredging point of view is usually made from an extrapolation of only a few cubic metres of samples to hundreds of thousands, or even millions, of cubic metres. Moreover, current dredging site investigation practice appears to still place an emphasis on alluvial sands and silts rather than on strong cohesive soil and rock, where, of course, the problems lie.

4. How then can a client protect himself at the time of tendering with a reasonable confidence that the tender price is a fair indication of the final cost of the job. The first prerequisite is to ensure that a thorough site investigation is carried out prior to tendering. The question is, what
constitutes good site investigation practice and where are current practices lacking. Second, it should be ensured that the contractors who are bidding do, in fact, have the necessary expertise and equipment for the task. Again, how does the client or his agents assess the capabilities of the contractor and his plant, there being no guidelines and very little published data. Third, the performance contract should state the zones of possible ground difficulties, and the price per unit quantity at each zone should be agreed upon beforehand. The problem is defining the zones, the transitions, and how they should be measured. And finally, monitoring and obtaining site agreement of the dredged material on a daily basis, which requires an agreed-on procedure, site technicians and laboratory facilities. What procedure should be adopted? Will the client pay for this service—in the past, perhaps not, but today as claims are escalating into the $10 to $20 million range, it would undoubtedly be in the client's interest and possibly in the long term the contractor's, as claims procedures can stretch on for years; whereas, on-site agreement could clear matters up in days.

5. There is no doubt, at present, that the current contract, control, and dredging practice falls far short of that outlined, mainly due to the paucity of information readily available to the client for tender preparation, site control, and monitoring. This is reminiscent of the tunnelling industry through the 1950s and 1960s; since then several million dollars have been spent worldwide on research and development and exchange of information is now at an international level, through the International Tunnelling Society and the many Institutions and Universities specializing in tunnelling and mining. Specialist papers are in fair abundance on most aspects of tunnelling. A data-bank search specifically in ground excavation carried out at the U. S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, produced over 25,000 references, over a thousand pertaining to rock excavation. A similar search on rock dredger cutting produced less than a dozen references. In contrast, it is estimated that over 450,000,000 cubic metres of dredging are now being carried out annually in the United States (Mohr 1980)—possibly an order of magnitude higher than the volume extracted by tunnelling. One obvious conclusion is that in the field of Civil Engineering, dredging has been a very much neglected area of research and development. Moreover, the potential for benefits to accrue to the client, which is more often than not
the nation, are perhaps more cost effective at present than those from tunneling and allied research. However, many excavation problems between dredging and tunnelling excavation are interrelated and possible technology transfer may aid the dredging industry, particularly in the area of soft to medium hard rock, where today in tunnelling, excavation is almost totally mechanical with a very small amount of blasting carried out, with a consequential reduction in excavation costs.

6. This report is concerned with the possible technology transfer from the tunnelling industry that may assist the growth of technology in the dredging industry. Considered are site investigation, rock cutting, and contracts. The purpose of this report is to provide some initial impetus to producing fruitful areas of investigation, or application, enabling a higher degree of confidence to be applied to the dredging project, with a resulting reduction in risk and cost.
7. If a new dredging project is to be assessed properly, there is no alternative to a careful and thorough investigation of every facet (Dumbleton and West 1976, t'Hoen 1979, and The Permanent International Association of Navigation Congresses (PIANC) 1972). From the point of view of the dredging technique, the properties of the material largely determine the degree of difficulty in the work, and are of immense importance in relation to several factors, including the type and capacity of the dredger to be used. The taking of representative and/or undisturbed samples and the analysis of these, or of the ground in situ, must be done by generally recognized methods. Only then can reproducible results be expected, allowing optimum interpretation for the purposes of the projected job. Superficial analysis, or shortcomings in the number of samples or the method of obtaining them, can lead to errors in the prescription or choice of the dredging method. Broadly speaking, the ground can be divided into three main groups:

a. Coherent, ductile material, such as clays with a particle diameter up to 16 μm.

b. Incoherent material, such as sand and gravel, with a particle diameter between 63 μm and 64 mm.

c. Coherent, consolidated material, such as slate, granite, and coral, with particle size in excess of 64 mm. Between these main groups lie broad areas of transition. For example, between the first and second groups lie silt and mud, which have a particle diameter between 16 μm and 63 μm. Between the incoherent material and the coherent, consolidated material can be found, for example, cemented sands and gravels and very soft types of stone. In addition to these main groups and transitional areas there are many mixtures.

Investigation on the Site

8. Thorough and detailed investigation of the site will usually obviate unexpected problems during the progress of the work. However, the cost of such an investigation must in all cases bear a reasonable relationship to the overall cost of the project. Many authors have advocated at least 1 percent of the total estimated cost should be allocated for site investigation. For tunnelling, in difficult mixed ground containing, say, sand, clay, rock, and boulders, some authors advocate up to 3 percent for site investigation. Even
so, this is rarely the case in practice. As discussed by Ash and Russell (1974), an increase in the extent and cost of the geotechnical investigations is normally associated with a decrease in the cost and risk and effort involved in the tunnel construction. Ash and Russell do, however, show that there is a finite limit to the desirable extent of the geotechnical investigations after which little or no overall economy can be achieved. At present, such a limit generally occurs at an investigation cost of 1 to 3 percent, although figures as high as 7 percent have been quoted (Construction Industry Research and Information Association (CIRIA) 1978).

9. However, the geotechnical investigation for any dredging operation need not and should not stop at this stage. The construction stage provides a chance to compare "predicted" to "actual" geology. This comparison in areas already dredged may provide information on the accuracy of predicted geology and method of excavations in future sections. Obviously, the investigation must be related to the problems which will arise during the work. This implies that there must be close cooperation from the outset between the expert who will conduct the site investigation and those who, on the basis of his findings, will determine the method of operation, prepare the cost estimates, draw up the specification and perform the actual operation. This is, of course, difficult at the prebidding stage. The manner in which the investigation is carried out also depends to a large extent on the nature of the main types of ground encountered on the site and on other available data relating to the prospective work area.

10. The main problem at the outset of site investigations is that of assessing ground from a dredging point of view: it will usually be necessary to extrapolate from a few cubic metres of sample to hundreds of thousands, or even millions, of cubic metres of spoil; hence the investigation needs to be well planned.

Extent of Ground Sampling

11. The extent of a given ground sampling operation depends to a large extent on the purpose for which the samples are taken. The uniformity or lack of uniformity of the soil stratification, the risks which one is prepared to take, and the experience previously obtained in the area concerned also influence the extent of the operation. The more complex the ground structure, the
more comprehensive will the site investigation have to be; if when it is
decided to take soil samples, little is known about the ground stratification,
a general investigation can be carried out, the results of which may enable
the nature and extent of the overall soil investigation to be determined.

12. For dredging purposes, the distance between the boreholes, measured
from center to center, should as a rule be between 50 and 200 m. For other
purposes, for example, foundations, dams, and dikes, it may be necessary to
take a substantially larger number of samples.

13. The depth of boring should as a rule be a few metres greater than
the dredging depth. Besides usually affording a better picture of the position
of the various strata, this can be useful for other purposes, for example,
subsequent dredging to a greater depth.

Treatment of Samples

14. The samples obtained, together with the results of any tests carried
out in situ, provide data concerning the properties of the soil. If the
sample is to remain representative after transport and storage, several meas­
ures must be taken immediately after the sample is obtained. This implies
that the sample must immediately be placed in a well sealed container, prefer­
ably of metal or plastic.

15. Samples of cohesive ductile material (clay, silt or peat, but also
sand with particles of cohesive ductile material) must be packed with as
little air as possible, and in any case in an airtight container in order to
prevent drying out. In many cases, coherent consolidated material must also
be packed in an airtight container, and preferably under water, immediately
after sampling. The properties of soil of this type can undergo considerable
change if it is exposed to air (drying out or crumbling). Every sample must
immediately be labelled for identification purposes, the label showing the
sample and boring numbers.

16. Immediately after the boring has been carried out, experts can
provide a description of the various soil strata. This enables an impression
of the composition of the various strata to be obtained while sampling is in
progress, so that if necessary the boring pattern can be modified or the
number of boreholes increased. The description is also a valuable aid to the
laboratory in determining the definitive extent of the examination of the samples (selection and possibilities for consultation).

17. The description of the sample may include the following:
   a. Details of structure, compactness, color, and smell.
   b. Estimated grain size, and, in the case of incoherent soils, the grain shape.
   c. For the harder types of coherent consolidated material, the degree of weathering, the number and nature of the strata, discontinuities, and fracture lines.

A number of tests can be carried out on site immediately after the sample has been obtained. Among these are the measurement of the density and, with the aid of the torvane, the shear resistance of coherent ductile materials, and the point load test on rock materials.

18. It is also extremely important that a number of other data which may be of significance for the further treatment of the sample be clearly provided during the sampling operation and any supplementary measurements. These include the following:
   a. Stating the place where the sample was taken (the site of the borings may be shown on a map).
   b. The depth with respect to a given reference plane such as the ground-water level or the ground level.
   c. If possible, particularly in the case of coherent consolidated soils, details of the position of the nucleus of the boring with respect to, say, the north-south axis.
   d. The date on which the sample was taken.
   e. The names of the company and the person responsible for obtaining the sample.
   f. The manner in which the sample was taken and the diameter of the boring.
   g. Data concerning the hardness of the material as measured on the site, for example, the standard penetration test (SPT) value.
   h. The water table, tidal range, and current velocities.
   i. If possible, the extent to which the sample obtained is considered to be representative.
   j. A brief résumé of the comments and statements made about the sample during the sampling operation.
Size of the Sample

19. Where incoherent material is concerned, samples must at least be of the following sizes if the principal characteristics, such as grain distribution, grain shape, density, and mineralogical composition, are to be determined:
   a. Fine-grain types: at least 500 g.
   b. Medium-grain types: at least 5 kg.
   c. Coarse-grain types: at least 30 kg.

20. If the suction properties have to be determined in order to assist in determining the attainable solids concentration of the dredged mixture, a somewhat larger sample is recommended to permit the permeability to be measured; in the case of the fine-grain material, this should be of between 1 and 2 kg.

21. To provide an indication of the wear which may be expected on the various components of a dredging installation, comparative tests can be carried out against a standard reference material. For these tests, at least 50 kg of fine-grain soil and at least 1 m³ of coarser-grained materials (coarse sand/fine gravel) are required.

22. For coherent ductile material (clay, loam, silt, or mud) the minimum quantity required for a satisfactory laboratory investigation is approximately 2 kg. The internal angle of friction and the shearing strength are extremely important factors in determining the power required to dislodge the material. To measure these by means of the cell test or the triaxial test, an undisturbed, cylindrical sample 36 mm in diameter and 72 mm long must be available for each test.

23. A complete laboratory investigation of a sample of coherent consolidated material (rock or coral) requires a core 100 mm in length and at least 50 mm in diameter. An investigation of this nature includes determining the tensile strength, compression strength, bending strength, modulus of elasticity, and density, possibly in the three principal directions. It is often impossible to provide complete cores of the material which is to be investigated. In such cases, the quantity provided must be at least sufficient for three standardized compression and tensile tests in each main direction. The test cores used for this purpose are at least 50 mm in diameter and 100 mm in length, and thus it must be possible to prepare at least 18 specimens of this size from the pieces of material.
24. One person alone cannot adequately assess a complete dredging job. The task demands specialists in many fields including geology, geophysics, soil mechanics, hydrography, and civil engineering. Moreover, theoretical knowledge is not enough; a great deal of practical experience is also demanded, and, for this reason, prospective contractors should be encouraged to take part in site investigation.
25. The method of analyzing soil and the elaborateness of the process are governed by the material itself and the purpose for which the analysis is performed. Specific tests for determining the properties of each type of soil or rock are well documented (Attewell and Farmer 1975); the method of analysis is determined by the nature of the material. Thus, for example, a Brazilian fracture test, to determine tensile strength, would only be performed on coherent, consolidated soil, while the tests carried out on clay will differ from those on sand. The method of analysis is also governed by the purpose for which it is undertaken. Thus, the tests conducted to determine the rate of wear of the various dredger components which are in contact with the material will not be the same as those carried out, say, to determine the suction-ability of the material. For the former purpose, measurements of the hardness and the size and shape of the particles are of primary importance; in the latter case, it is necessary to determine the particle size and shape, but also the permeability and the pore volume of the soil.

Range of Tests

26. The range of possible tests is determined by the nature of the soil. For example, many more specific tests can be performed on coherent, ductile material than on sand.

27. The scope of the investigation will also be determined in part by the ultimate aim. In order to determine structure, strength, and fabric of a rock, generally a large number of specific tests must be performed. To establish the critical velocity of a dredged mixture, on the other hand, only a few specific tests are important.

28. When dealing with coherent, ductile material, the most important physical properties from the point of view of loosening, or breakdown of the cohesion, are the cohesion, the internal angle of friction, the shearing stress, and the penetration resistance (cone value).

29. With incoherent material (sand and gravel), the penetration resistance (cone value) is the most important parameter. With both coherent, ductile material and incoherent material, the grain distribution, specific
gravity, and pore volume should be determined in situ, and the lime and humus contents ascertained.

30. With coherent, consolidated material, the principal properties from the point of view of breaking down the cohesion are:

a. The number and nature of the strata.
b. The porosity and specific gravity.
c. The dynamic modulus of elasticity.
d. The tensile, compressive, bending, and shearing strengths.
e. The toughness.

Properties Required from Site Investigation

31. The following paragraphs summarize the principal physical and mechanical properties that should be measured or ascertained for a complete analysis of the selection of the dredging process.

32. SPT with a spoon and geology mapping on structure genetics, texture, and color are considered to be supplementary; and on their own, inadequate.

33. For coherent, ductile material, including clay and loam less than 16 μm the following properties should be measured: penetration resistance, specific gravity, shearing stress, cohesion, internal angle of friction, pore volume in situ, grain distribution, lime content, humus content, water content, permeability to water, specific gravity of particles, viscosity (mixture), and plasticity limits.

34. For the area of transition between ductile and incoherent soils, 16 μm to 63 μm, the following properties should be measured: penetration resistance, specific gravity, shearing stress, cohesion, internal angle of friction, pore volume in situ, grain distribution, lime content, humus content, water content, permeability to water, specific gravity of particles, shearing stress, internal angle of friction, viscosity (mixture) particle shape, and natural slope beneath the water.

35. For incoherent material (sand, gravel 63 μm to 64 mm), the following properties should be measured or ascertained: penetration, resistance, specific gravity, pore volume in situ, lime content, humus content, water content, permeability to water, specific gravity of particles, grain distribution, shearing stress, internal angle of friction, natural slope beneath the
water, and particle shape.

36. For coherent, consolidated material (slate, granite > 64 mm), the following properties should be measured or ascertained: specific gravity; tensile strength; compressive strength; number and nature of strata; shearing strength; toughness; dynamic modulus of elasticity; porosity; size, shape, and distribution of the particles; and grain hardness.

37. For the careful analysis of the properties of the soil, it is vital that the sampling procedure; the on-site investigation; and the packing, transport, and storage of the samples be undertaken with proper attention to detail.

38. The requirements imposed in respect of the method of boring are determined not only by the purpose for which the samples are taken, but also by the nature of the soil. The degree to which the sample is disturbed is governed by the method employed. In a completely undisturbed sample, the situation of the material obtained is unchanged. This implies that the position and distribution of the particles and the tension in the soil are as they were in situ. It is, however, virtually impossible to obtain so perfect a sample.

**In-situ Soil Sampling**

39. By employing specially designed apparatus, it is possible to determine certain soil properties in situ. The properties concerned include:

   a. The cone resistance.
   b. The N value (SPT value).
   c. The shear strength.
   d. The density of the soil in situ.
   e. The permeability.

The stratification of the soil can also be determined, albeit less accurately, by seismic studies.

**Penetration Tests**

40. A distinction is made between static and dynamic penetration tests. In these tests, the force or energy, respectively, required to press or drive a cone or sampling spoon into the soil is measured. There is, however, no
truly reliable method of analysis for obtaining data relating to the certain properties of the material. Moreover, major differences exist in the manner in which penetration tests are carried out and the results presented.

41. Efforts to standardize penetration test procedures have been made at an international level. A number of methods have been incorporated in the Standards of the American Society for Testing and Materials (ASTM). As regards Europe, the International Society for Soil Mechanics and Foundation Engineering (ISSMFE) has adopted a proposal to standardize, among other things, the dynamic penetration test, the static (or Dutch) penetration test, the standard penetration test, and the so-called Scandinavian weight penetration test.

Dynamic Penetration Tests

42. In these tests, the strength of soil strata is determined on the basis of the pressure required to press a cone into the soil. Cone resistance is now defined as the quotient of this force and the projected surface area of the cone expressed in meganewtons per square metre.

43. By adding a so-called side friction sleeve, consisting of a cylinder which is capable of being pressed into the soil independently of the cone, the local friction-quotient of the measured force and the surface area of the cylinder, expressed in meganewtons per square metre, can also be measured.

44. In many cases, the total friction is also measured. This is obtained from the force needed to press the jacket tubes into the soil. The data obtained are not, however, easily reproducible, and in view of the unreliability, it is not advisable to attach too much importance to the results of the total friction tests.

45. The relationship between the cone resistance and the local adhesion enables a general description of the type of soil in the various strata to be given without the necessity for drilling. This relationship, however, is dependent upon the shape and dimensions of the cone and the friction sleeve. Moreover, the relationship established for a given cone is valid only for water-saturated, undisturbed soils which have been deposited naturally.

46. Residual stresses in the soil strata, which, for example, can be a result of earlier soil loads, can result in higher measured cone values. Such residual stresses can disappear under the influence of vibration or excavation,
causing considerably lower measured cone values. In practice, this can mean that during dredging the compaction of the soil is less than had been assumed on the basis of cone resistance measurements.

47. The cone used in the cone penetration test (CPT) has an apex angle of 60 deg and a diameter of 35.7 mm. The cylindrical portion above the cone must be of the same diameter as the base of the cone for 1,000 mm of its length. If a sleeve is employed to measure adhesion, it must have an area of 15,000 mm² and be situated immediately above the cone. The complete system of cone and tubes must be pressed into the soil at a constant speed of 20 mm/sec.

48. A distinction can be made between mechanical and electrical static penetration tests. In the former, the pressure applied to the cone is transmitted to a pressure gage at ground level via internal rods. This test can be continuous or discontinuous. With the continuous method, the cone and the jacket tubes are pressed into the soil simultaneously by means of the internal rods. In the discontinuous tests, pressure is applied alternately to the cone (via the internal rods), to the jacket (via the system of tubes), and to the cone and jacket together (via the system of tubes).

49. With the electrical test, the pressure on the cone is measured by a sensor located immediately above the cone. Where friction is measured, this is also done electrically. The measurement signals are transmitted by cable to the surface, where they are amplified and read off or registered. Electrical penetration tests are always continuous.

50. Penetration tests can be divided into a number of categories on the basis of the method of execution and the manpower required.

**Manual Tests**

51. With this method, the cone is pressed into the soil manually. The penetration depth and force are very limited.

**Medium and Heavy Penetration Tests**

52. In tests in these categories, the pressure is applied by mechanical or hydraulic means. The apparatus used for this purpose may be portable or may be mounted on, say, a truck or an amphibious vehicle. The forces measured in tests under this heading may be as high as 200 kN. Where forces of this
magnitude are anticipated, the penetration apparatus must be secured in position with screw anchors or ballast.

Underwater Penetration Tests

53. For investigating soil which lies beneath water, penetration tests, alone or in conjunction with drilling, can be carried out from a specially equipped pontoon. Where substantial water depths and/or adverse weather conditions are encountered, the pontoon will have to comply with certain requirements. A ballast block can be used to absorb the reactive forces generated during the test. To reduce the motions of the pontoon under the influence of waves, etc., it will be necessary to employ a swell compensator. At very great depths, penetration tests can be carried out from a diving bell.

Dynamic Penetration Tests

54. Here, too, a number of variants have been developed, chief among which are light and heavy dynamic tests and a standard dynamic test. Although the dynamic penetration test affords less accurate measurements than the static test, the SPT is employed on a fairly wide scale in the dredging industry. The use of the term "standard" here is, in fact, misleading, since various types of apparatus, each with its own dimensions, have been developed in different countries. In most cases, however, the American standard penetration tests developed by Terzaghi and Peck (1967) is used.

55. The cones used in the light and heavy penetration tests have areas of 10 and 15 cm² and apex angles of 60 and 90 deg, respectively.

56. Weights of 10 and 50 kg, respectively, are employed at a fall height of 50 cm. The unit of measurement for these tests is the number of blows required to cause the cone to penetrate the soil to a depth of 20 cm.

57. The most commonly used (American) SPT, developed by Terzaghi and Peck, is based on the use of a penetration tube of standard dimensions. The test is carried out in a borehole after the removal of loose material from the bottom. First, the tube is driven in to a depth of 15 cm. Then the test proper commences. This consists of determining the number of blows required to achieve a penetration of 30 cm. For this purpose, a 63-kg weight is dropped from a height of 75 cm. The number of blows constitutes the N value.
It is usual to limit the blows to 50. An added advantage of this test is that the relevant sample core is obtained simultaneously.

58. In the SPT, the compactness of the soil is chiefly determined by the degree to which the material adheres to the inner and outer surfaces of the tube. Because the friction is governed by the nature of the material, among other factors, the matter of soil composition should be included in the analysis.

59. An approximate relationship between soil compactness, N value, and cone value, depending on the type of soil, is given as follows:

### Incoherent Material (Sand)

<table>
<thead>
<tr>
<th>Soil Condition</th>
<th>N Value (Spt According to Terzaghi and Peck)</th>
<th>Cone Resistance (Dutch Cone Penetration Test), bars</th>
<th>Relative Density (D_r)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very loose</td>
<td>4</td>
<td>25</td>
<td>0.15</td>
</tr>
<tr>
<td>Loose</td>
<td>4-10</td>
<td>25-50</td>
<td>0.15-0.35</td>
</tr>
<tr>
<td>Medium dense</td>
<td>10-30</td>
<td>50-100</td>
<td>0.35-0.65</td>
</tr>
<tr>
<td>Dense</td>
<td>30-50</td>
<td>100-200</td>
<td>0.65-0.85</td>
</tr>
<tr>
<td>Very dense</td>
<td>50</td>
<td>200</td>
<td>0.85</td>
</tr>
</tbody>
</table>

### Coherent Material (Clay)

<table>
<thead>
<tr>
<th>Soil Condition</th>
<th>N Value (SPT according to Terzaghi and Peck)</th>
<th>Unconfined Compression Strength, bars</th>
<th>Torvane Cohesion, bars</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very soft</td>
<td>2</td>
<td>0.25</td>
<td>0.13</td>
</tr>
<tr>
<td>Soft</td>
<td>2-4</td>
<td>0.25-0.5</td>
<td>0.13-0.25</td>
</tr>
<tr>
<td>Plastic</td>
<td>4-8</td>
<td>0.5-1</td>
<td>0.25-0.5</td>
</tr>
<tr>
<td>Stiff</td>
<td>8-15</td>
<td>1-2</td>
<td>0.5-1</td>
</tr>
<tr>
<td>Very Stiff</td>
<td>15-30</td>
<td>2-4</td>
<td>1-2</td>
</tr>
<tr>
<td>Hard</td>
<td>30</td>
<td>4</td>
<td>2</td>
</tr>
</tbody>
</table>
Relationship between N Value and Cone Resistance

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Cone Resistance, $N_{30}$ (Normalization At 30 Blows)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravel</td>
<td>5.5-8</td>
</tr>
<tr>
<td>Coarse sand</td>
<td>4-5.5</td>
</tr>
<tr>
<td>Fine sand</td>
<td>2.5-4</td>
</tr>
<tr>
<td>Clayey sand</td>
<td>6</td>
</tr>
<tr>
<td>Sandy loam</td>
<td>5-6</td>
</tr>
<tr>
<td>Sandy clay</td>
<td>3-4</td>
</tr>
<tr>
<td>Clay</td>
<td>2</td>
</tr>
</tbody>
</table>

60. Both the water table and the depth at which the dynamic penetration test is carried out can greatly influence the N value. Where N values are measured beneath the water table, the tension of the subsoil water can lead to excessively high values being observed. At greater depths, the vertical and horizontal particle tension will increase under the weight of the higher strata, and this can also raise the apparent N value. At N values in excess of 50, the heavy penetration test is sometimes resorted to. This is then often referred to as the SPT with cone.

The Vane Test

61. The shear strength of the soil can be measured with the aid of the vane apparatus. This test is principally carried out in coherent, ductile materials. The vane apparatus consists of four vertical blades, or vanes, which are pressed into the soil vertically to a depth at least equal to the length of the vanes, the pressure being applied via a vertical shaft embodying a rod mechanism.

62. Torque is then applied to the shaft, and the maximum moment at which the cylinder of soil between the vanes shears is a measure of the shear strength.

Density

63. The density (or specific mass) of the soil in situ, at the point of reclamation or in the hopper, is an extremely important factor in determining
the quantity of material to be dredged or to be transported hydraulically as fill. For example, the amount of energy required to dredge a given mass of soil will be less if it is of low density than if it is of high density.

64. Taken in conjunction with the nature of the material, the density affords important information concerning its compactness and permeability, and thus about its workability (disruption of cohesion) and amenability to suction during the dredging process, among other factors.

Permeability

65. Permeability is an important factor in relation to the disruption of cohesion (dislodgement) and suctioning of the soil. Knowledge concerning this aspect is therefore indispensable in assessing the suctionability of a given type of soil, both in terms of lifting it from the bottom and evacuating it from hoppers, etc.

Other Measurements

66. Among the indirect methods of establishing how the soil strata are made up, is geophysical investigation (The British Geologist 1979 and Welkie and Meyer 1982). This involves determining physical properties such as electrical, seismic, acoustic, magnetic, and gravimetric values. These research methods afford only a global impression and are in many cases undertaken only as a part of a preliminary investigation. It is of importance that the geophysical investigation be supplemented by an adequate number of test drillings in order that the findings can be correctly interpreted.

67. Geophysical investigation is a useful method of site investigation for "filling in" detail between borings and drilling. However, such methods still require improvement and very careful interpretation; it is necessary to have relatively simple soil/rock conditions for success with this method (i.e., soft alluvium over rock), where only slight changes in strata density occur. The method may be misleading for establishing actual boundary conditions, but does have merit in assessing rippability and will be discussed later.
PART IV: ASSESSMENT OF PROPERTIES OF ROCK THAT AFFECT EXCAVATION

68. There is no single measurement that can be taken which will provide a proper assessment of the ease with which a particular area of rock can be excavated. This is because there is no single property of rock which in itself renders the rock easy or difficult to excavate, but rather a series of properties, all of which affect the ease of excavation of the rock in varying degrees.

69. The properties of rock which most affect excavation can be divided into two separate groups. The first group consists of those properties inherent in the type of the rock, the rock material properties, such as grain size, grain bonding, and mineral composition, while the second group is more concerned with the rock as it exists in a particular place at a particular time. These are the properties of the rock mass. They include such factors as the degree of lamination, the angle of dip of strata, and probably the most important, the degree of fracture.

70. The properties of the rock material are more important when the rock to be removed is massive. If the rock is already fractured, the properties of the rock mass become much more critical.

71. Except from some oblique references to dredger performance in trade journals, little has been published on the behavior of rock in underwater conditions. Therefore, much of the following has been derived from work originally directed to dry land conditions since it is considered that rock properties do not differ significantly in the marine and land environments, especially as much of the land rock is tunnelled or mined beneath the water table.

The Rock Material

72. Rock is a natural aggregate of mineral grains connected by strong and permanent cohesive forces. Both the type and size of the mineral grains and their bond can influence the breaking of the rock. These factors affect the different methods of penetrating and breaking the rock to different degrees.

73. Mineral composition is of prime importance when the drillability and machineability of rocks is being considered. Fowell (1970) identified two
distinct rock groups which affected mechanical excavation: (a) those containing quartz in appreciable amounts, and (b) those containing little or no quartz. The quartz content of a rock governs its abrasiveness, and for a given compressive strength, those rocks with high quartz contents are shown to be more difficult to drill or machine.

74. When drilling abrasive rocks, greater efficiency can be expected from percussive drills rather than rotary types. Other factors, however, such as Mohr hardness, which is only indirectly related to abrasiveness, also have to be considered; rocks are grouped according to their abrasiveness in each of the three major rock types (igneous, metamorphic, and sedimentary).

75. Grain size particularly affects drillability and ease of fragmentation. Coarse, even-grained rocks can be drilled faster than fine-grained or inequigranular rocks (Paone, Madson, and Bruce 1969). Coarse-grained rocks can also be blasted more easily than fine-grained rocks.

76. The bonding of the mineral grains in a rock determines its toughness. In igneous and metamorphic rocks, the grains are usually welded together because of the greater pressures and temperatures which they have suffered. Sedimentary rocks are either cemented or simply compacted under gravity and are thus generally weaker. The grain bonding has an important influence on the strength of the rock as the bonds are usually the weakest part of a rock. Coarse-grained, weakly cemented rocks need special consideration when being blasted because they tend to form chambers rather than fragment.

77. Rock strength is generally defined as the uniaxial compressive strength of a coherent rock sample. Tsoutrelis (1969) and also Brown and Phillips (1977) showed that, when using a rotary diamond drill at constant rotation speed and thrust, the drilling rate decreased with increasing uniaxial compressive strength. Franklin, Broch, and Walton (1971), however, state that drilling and blasting efficiency depends largely on the tensile strength of the rock; this is certainly true for blasting but is not considered so for cutting. The most probable explanation is that a cutting mode can create a certain amount of tension, going on to a compression mode, creating tension cracks and finally shearing in which case the internal angle of friction may have some bearing on failure of the discrete cutting clip. Nevertheless, a compressive test index would give an approximation to the cutting difficulty involved.
The Rock Mass

78. The discontinuous nature of the rock plays an important part in deciding the strength of a rock mass; Barton, Lien, and Lunde (1974) and Bieniawski (1973) have considered this aspect for excavation support in mining and tunnelling. Discontinuities include faults, joints, bedding planes, and fissures which may have been formed in a variety of ways. In a highly fractured rock mass, the strength is derived mainly from the strength (cohesive and frictional) across the fracture planes; whereas, in massive rock the strength is that of the intact rock material.

79. It follows that when a rock mass is fractured, it is essential that the fractures are described fully so that their effect on its strength may be properly assessed. The following aspects of fracture planes should be considered in any such assessment:

- Spacing.
- Orientation.
- Continuity.
- Tightness.
- Surface texture of fracture plane.
- Type of infilling material, if any.

All of these govern, to some degree, the overall strength of a rock, and thus the ease with which it may be broken up.

80. Laubscher and Taylor (1976) show that the structural pattern in a rock has a major influence on the fragmentation obtained by blasting. The greater the fracturing of the rock mass, the smaller the amount of explosive that will be needed; alternatively, a cheaper explosive may be substituted. On the other hand, clay infillings in joints and faults make breakage more difficult because the clay absorbs the blast energy. Fractures, particularly open fractures, also reduce the drilling rates of all types of drills and increase the risk of breaking the drill. Percussive drills are especially affected by fractures with clay infilling (McGregor 1968).

81. Uchibayashi (1970) considers that hard brittle rocks which contain fissures are not too difficult to excavate with a cutter suction dredger, as more cracks can be propagated quite easily. Tough unfissured rock is usually more difficult. Layered rock where the underlying rock is weaker can also usually be removed without too much difficulty, but massive, homogeneous rocks...
do not lend themselves to dredging without pretreatment.

82. Although the rock mass quality is very significant for support, ripping, and blasting, cutter tools on a rock dredging head are unlikely to benefit from fractures over 50 mm spacing, especially if the jointing is well cemented or tight.

83. Most of the considerations concerning the rock mass are restricted to that part of the rock which has been exposed to the agents of weathering. The thickness of the zone of weathering can vary from a few centimetres to hundreds of metres, depending on the rock type and the degree of fracturing. This zone is characterized by a decrease in seismic velocity compared with the unweathered rock, as the fractures have usually been opened by the weathering process. Below the zone of weathering, the fractures are generally tight and the rock reacts much as a completely massive rock where ease of removal is dependent on the rock material.

**Excavation Characteristics**

84. It is clear that both the properties of the rock mass and the rock material must be considered in order to obtain an overall idea of the ease with which the rock may be broken up and removed. The importance of any one property is very difficult to define, and various authors have attempted to catalog the most important properties.

85. It has even proved difficult to establish direct correlations between any physical rock property and the drillability of the rock. This led White (1969) to produce his empirical Drillability Index based on actual drilling performance achieved in 98 rock types with the three basic drilling methods. Slightly better correlations have been obtained when rock machineability and explosive performance have been compared with certain rock properties.

86. Gnirk and Pfleider (1968) attempted a correlation between crater formation and rock properties using constant explosive charges in five different rocks. They determined the following properties for each rock type:

a. Unconfined tensile strength.
b. Unconfined uniaxial compressive strength.
c. Young's modulus.
d. Longitudinal wave velocity.
e. Density.
87. The only conclusions Gnirk and Pfleider could make were:
   a. Any increase in any one of the five rock properties resulted in a decrease in the maximum crater volume.
   b. Bedding and jointing substantially increase the volume of rock broken and craters are sometimes delineated by joints.
   c. The smallest fragmentation of the rock was obtained when the charge depth was at 70 to 80 percent of the depth for maximum crater volume.

88. Johnson and Fischer (1963) obtained reasonable correlations between crater dimensions and the various mechanical properties of the rocks they tested. The correlation of tensile strength and crater volume had the least scatter and they concluded that reflected strain pulses were the main cause of cratering, with tensile strength being an important parameter.

89. The Caterpillar Tractor Co. (1978) drew the following conclusions concerning the ripping of rock by their machines. Physical characteristics of rock favorable to ripping were as follows:
   a. Fractures and faults and any other planes of weakness.
   b. Weathering resulting from temperature and moisture changes.
   c. Brittleness and crystalline nature.
   d. A high degree of stratification or lamination.
   e. Large grain size.
   f. Permeation by moisture.
   g. Low compressive strengths.

90. Caterpillar Tractor Co. (1978) listed the following physical characteristics of rocks as unfavorable to ripping:
   a. Massiveness and homogeneity.
   b. Noncrystalline and therefore nonbrittle nature.
   c. Fine grain size with solid cementing agent.
   d. Clay origin where moisture may impede ripping because it makes the material plastic.

91. Duncan (1969) used saturation moisture content, $i_s$, as an index for drilling and blasting, and stated:
   i, 4 percent needs drilling and blasting
   i, 4-8 percent needs close inspection
   i, 8-12 percent may be excavated with a face shovel
   i, 12 percent may be classed as a soil
He has correlated many of the physical and mechanical properties of rocks with one another and also says further that a homogeneous rock with laboratory seismic velocity of 2,100 m/sec, or a uniaxial compressive strength of 55 MPa or Schmidt Rebound Number 40, will probably need drilling and blasting before it can be removed. This is interesting as it is the only reference found for the arbitrary threshold between "soil" and "rock."

92. Furby (1964) obtained a straightline plot for rebound number against percussive penetration rate for a limited number of rocks, and the U. S. Army Corps of Engineers states that rocks can be classified by their seismic velocity, stronger rocks having higher velocities. Both of these give support to Duncan's theories.

93. Franklin, Broch, and Walton (1971) have correlated increases of point load strengths of Pre-Cambrian rocks in Norway with decreases in the rate of percussive drilling. They also found evidence of a relationship between the resistance to blasting and point load strength. They concluded that soft rocks with a point load strength of less than 1 MPa are not easily loosened by blasting because of crushing in the vicinity of the borehole which absorbs much of the energy produced.

94. Tarkoy and Hendron (1975) carried out a fairly exhaustive investigation (about 850 references) into rock hardness index properties and geotechnical parameters for predicting tunnel boring machine performance. Good correlation was established between compressive strength, rate of excavation, and cutter wear using an index known as total hardness, \( H_T \), defined as:

\[
H_T = H_R \sqrt{H_A}
\]

where

\( H_R \) = rebound or mass hardness correlating to compressive strength
\( H_A \) = abrasion or small scale hardness, developed from weight loss on abrasion tests

This index was originally proposed by Deere (1968).

95. Farmer, Hignett, and Hudson (1979) carried out field trials on three separate tunnelling projects, in a similar manner to Tarkoy and Hendron. They came to the conclusion that the compressive strength of the rock was the main factor in determining excavation rates, with abrasion of secondary importance.

96. Fracture spacing is probably the most important of the factors for ripping and various means of expressing this have been put forward. The
most common one is Deere's rock quality designation, for which a completely massive rock has a value of 100 percent and a completely fragmented rock (i.e., a soil) has a value of 0 percent.

97. The fracture state of the rock mass has also been expressed by Duncan (1969) by means of the three ratios below, which he states will be substantially equal for any particular rock.

\[
\frac{V_{\text{field}}}{V_{\text{lab}}} = \frac{q_j}{q_u} = \frac{j_c}{j_1}
\]

(2)

where

- \(V_{\text{field}}\) = field seismic velocity
- \(V_{\text{lab}}\) = laboratory seismic velocity
- \(q_j\) = the uniaxial compressive strength of a rock sample containing joints
- \(q_u\) = the uniaxial compressive strength of a coherent rock sample
- \(j_c\) = the real area of contact across the joint zones
- \(j_1\) = the apparent total area of contact across the joint zones

As the ratios increase toward unity, so the effect of jointing in the rock mass becomes less and the strength of the rock mass approaches that of the intact rock material.

98. The comparison of the field and laboratory seismic velocities of a rock mass may be used as preliminary assessment. This is probably the most promising of the various tests proposed, in that the laboratory velocity can be correlated in some degree with the compressive strength, and the field velocity gives an indication of the degree of fracturing present in the rock mass.

99. It is clear that, as yet, no single satisfactory test exists of how to identify the dredgibility of rock. PIANC produced a report (1972) on soil and rock classification for dredging, but this can only be regarded as a preliminary step. A group of Dutch contractors are believed to be sponsoring work at Delft University on this and allied problems. As yet, there is no apparent record of published literature.

100. There is a growing need to excavate channels into rock below the silt levels. The high cost of blasting and mobilization costs if the initial
approach was incorrect all point to an apparent requirement for further research and development to produce codes of practice on interpretation of site investigation results as applied to dredging methods and equipment selection.
101. The pretreatment of rock prior to dredging is still normally a prerequisite in all but weak rocks. There is a growing tendency, however, to direct dredge increasingly harder rocks so that the expense of pretreatment is dispensed with.

102. Machines for the mechanical breaking of rock fall into three groups distinguishable from each other by their mechanical action: breakers, splitters, and rippers.

103. All three methods have been proved on land, although they may be restricted to certain rock formations. Only rock breakers and, to a limited degree, rippers have been used for the pretreatment of rock underwater.

**Rock Breakers**

104. Rock breakers break rock by repeated blows on the rock surface which propagate fractures in the vicinity of the applied blows. A heavy chisel may be allowed to fall under its own weight, be assisted by a diesel explosion, or driven by compressed air. In all cases, the procedure is to apply a certain number of blows at points on a fairly tight grid (often 0.6 to 1.5 m). Fragmentation is generally good, but of very limited penetration. The method, however, requires that the chisel strike in the same local area each time, which, even with guide tubes, is difficult to attain underwater in any but the calmest conditions. Rock breaking is a very slow and expensive method of fragmenting rock. Various sources quote between 4 to 20 m$^3$/hour, depending on the rock and the type of rock breaker used.

105. Rock breakers reached their peak of usefulness in the early 1950's, but have been largely replaced by drilling and blasting since the development of the overburden drill, which substantially cut drilling costs. Until recently it has been thought unlikely that a rock breaker will now prove an economic tool for fragmenting rock, except on a very small contract where the mobilization of a drilling rig would be unjustified. However, Mohr* reports that a contractor is manufacturing a large-scale rock breaker for use at Port Everglades, Florida. The results of such an exercise should be interesting.

* Personal communication, A. W. Mohr, 1984, U. S. Army Engineer Division, South Atlantic, Atlanta, Ga.
106. Rock splitters act directly on the rock and need to be powerful enough to overcome the tensile strength of the rock. A hole of a certain size and depth is drilled into the rock and the double wedge of the splitter inserted. A piston is then forced hydraulically into the wedge, expanding it and thereby exerting the necessary pressure on the rock.

107. Two companies, Emaco Inc. (United States) and Atlas Copco (Sweden), produce rock splitters of similar design. Both makes are claimed to split rock in seconds and to be capable of handling by one operator. They are best suited for confined areas where blasting is prohibited. They also enable controlled, directional splitting. They do, however, require a free rock face to allow splitting to occur.

108. As far as can be ascertained, rock splitters have not been used for underwater dredging, presumably, because of access and obvious slowness.

Rippers

109. Rippers provide a more economical method of rock pretreatment than drilling and blasting in some rock formations. Rippers are best suited to brittle or fractured rock where the teeth can penetrate the rock. The main source of information for ripper design and capability can be obtained from the Caterpillar Tractor Co. (1978), which produces a ripping chart based on seismic velocity measurements for different rock types. The main disadvantage of rippers, when used off a dredging pontoon as opposed to being towed by a large tractor, is their slow rate of excavation. Ten to fifteen cubic metres per hour is considered to be a reasonable performance. On land, this figure is easily increased by tenfold in suitable rock. Obviously, a heavy investment in a development program may well produce a much faster ripper dredger. The nub to such development is tying up a very large investment in a one-off dredging unit, with the consequential heavy mobilization costs for each job. A more cost effective approach would be development of the cutter section dredger, where only the cutting head requires development and many cutting heads can be produced to fit existing cutter suction dredgers.
110. The cutter suction dredger is, in essence, a multiblade ripper, integrated into a rotating suction head. The essential features are shown in Figure 1.

111. The cutter rotates about an axis parallel to the line of the ladder. Some examples of cutters are shown in Figure 2. Cutters may rotate clockwise or counterclockwise, but always in one direction for a particular cutter. This leads to a measure on built-in inefficiency as the teeth or bars are always either overcutting or undercutting, depending on whether the dredger is swinging with, or away from, the direction of cut. Cutters may be straight arm or basket, and may have fixed or removable teeth, depending on the material which is to be cut and the power available. Cutter manufacturers recommend that each dredger keep a considerable variety of cutters of different sizes and shapes for meeting various conditions. The rotating suction pipe cutter drive is a hollow drive shaft which also becomes the suction pipe. This has a number of advantages. The weight of the whole ladder unit can be considerably reduced, and the wear from material friction which normally takes place predominantly in the lower half of the suction tube is spread evenly around the tube, while the rotation also tends to prevent material from settling out and reduces cavitation. From the point of view of rock dredging, however, advantage comes from the strength of the drive shaft for a given weight and the overall size of cutter that can be fitted.

112. An innovation of recent years, and one which can often be fitted to existing plant, is the jet pump booster. The suction available in the main pump cannot exceed the atmospheric pressure, and, in practice, is limited to about 8.5 m of water. This has to overcome entrance restrictions and friction in the pipe itself and has also to raise the water in the pipe to the working velocity. These three factors do not alter appreciably with depth, but the energy needed to lift the spoil in suspension does vary directly with depth. At greater depths, therefore, although the pump can continue to pump water, the material content must drop off. The jet booster injects water at high velocity into the suction tube as close to the entrance as possible. This reduces the amount of pump energy required to accelerate the water column and thereby increases that available for lifting the spoil.
Figure 1. General arrangement, suction cutter dredger
Figure 2. Rock suction cutter dredging head
Rock Suction Cutter Heads

113. Makers of suction cutter heads claim the ability to cut rock up to 70 MPa compressive strength and some much higher. But many cases, although not published, have been reported of cutter heads having difficulty in rock strengths well below 50 MPa. Overall confidence in the dredging industry of the rock cutter head is somewhat mixed, depending on the particular contractor's experience. Unfortunately, there is insufficient published data from field work to substantiate any claim by manufacturers.

114. The author, applying tunnelling rock cutting principles to one manufactured cutting head, at the request of a dredging firm, came to the conclusion that 25- to 30-MPa limestone would be the maximum capability of the dredger. The manufacturers claimed 70 MPa without recourse at all to dredger compliance. Subsequent trials proved the lower figure correct.

115. Unfortunately, there is a paucity of data on the rock excavation industry. The only information readily available in the technical press are generalizations such as "dredger does 250 m³/hour in limestone," which does not add much to the state of the art. Hence, to provide a base for future development, there is a requirement to obtain basic information on field performance of current production rock dredgers with geology, rock indexes, power requirements, and other factors as indicated.
116. In the tunnelling industry, rock cutting machines are usually designed by applying standard engineering methods in conjunction with experience gained during the evolution of successive generations of machines. This is a very sound approach for gradual progressive development, but it may not be appropriate when there are requirements for rapid development involving radical departures from established performance characteristics. A distinct alternative is to design more or less from first principles by means of theoretical or experimental methods.

Approach to Cutter Design

117. Numerous difficulties arise in attempting a strict scientific approach to the design of rock-cutting machines. The relevant theoretical rock mechanics involves controversial fracture theories and failure criteria, and calls for detailed material properties that are not normally available to a machine designer. Direct experiments are costly and time-consuming, and experimental data culled from the literature may be unsuitable for extrapolation. Comprehensive mechanical analyses for rock-cutting machines have not yet evolved, and while established design principles for metal-cutting machine tools may be helpful, they do not cover all pertinent aspects. For example, there are usually enormous differences in forces and power levels between machine tools and excavating machines, and force components that can be almost ignored in a relatively rigid machine tool may be crucial design factors for large mobile rock cutters that are highly compliant.

118. Hence empirical methods have evolved and the most suitable shape and size of rock-cutting tools for use on a range of rocks can best be determined from tests in the laboratory, using an instrumented shaping machine or similar system. The cutter is mounted on a triaxial dynamometer on the machine head and the rock under investigation is fixed to the machine bed while tests are carried out using a range of tool shapes and sizes.

119. By monitoring the forces and the amount of rock excavated, the tool shape with minimum cutting energy requirement per unit quantity excavated, within the range of depths of cut under consideration, can be determined, as well as an optimum depth of cut and spacing between adjacent cutters. A
cutting tool breaks out more rock than the volume swept by its projected area. Additional breakage occurs at the sides of the cut and is termed "side-break." Correctly spaced cutters maximize this "side-break" effect.

120. Within limits, the spacing of tools, S, to achieve maximum interaction between adjacent cuts is a function of the depth of cut, D, so that the ratio of tool spacing to depth of cut is the important consideration when deciding on arrangements of cutting tools. The spacing of cutters is usually fixed on the machine, so changes in the S/D ratio are obtained by altering the depth of cut. The depth of cut, D; the rate of advance of the machine, V; the rotational speed of the cutter head, \( w \); and the number of cuts on the same path per revolution, C, are related by the equation:

\[ V = DwC \quad \text{or} \quad D = V/wC \] (3)

Hignett and O'Reilly (1977) have examined the interactive effect of tool placement and design procedure for placement of cutting tools and consider the geometric placement and operation of the cutting head of equal importance to that of the selection of the individual cutting tool.

Available Mechanical Cutters

121. The general types of cutters available for mechanical rock excavation are shown in Figure 3. Descriptions of each cutter follow, and their applicability to rock strengths is given in Figure 4.

Drag picks

122. Tools which range from a simple chisel form to pointed attack are assembled in array on the peripheral surface of a rotating drum or face plate of a soft ground mole. They are also sometimes used for trepanning, planing, or milling a rock face. This tool is widely used in the coal mining industry. It is generally recognized that the upper limit for using drag picks is 70 to 80 MPa unconfined compressive strength, although some success has been reported in rock of higher strengths.
Figure 3. Rock-cutting techniques
Soft 0 - 100 MPa
(S) - Salt
(S) - Coarse grained, weakly cemented sandstones
(S) - Fossiliferous limestones
(I) - Altered igneous rocks
(S) - Claystones, shales
(S) - Coal

Medium 100 - 170 MPa
(S) - Marlstones, Limestones
(M) - Marble
(S) - Shales, siltstones, sandstones
(M) - Phyllites
(M) - Highly micaceous schists
(M) - Altered intrusive igneous rocks
(M) - Altered metamorphic rocks

Hard 170 - 240 MPa
(M) - Slates
(S) - Crystalline limestones
(I) - Diabase
(S) - Silicious, cemented sandstones
(M) - Gneisses and schists
(I) - Pyroxenites
(I) - Coarse grained granites

Very Hard 240 MPa
(M) - Quartzites
(M) - Amphibolites
(S) - Dolomites
(I) - Fine grained granites
(I) - Basalt, diabase
(I) - Syenites
(I) - Gabbros

Rock classes:
(S) Sedimentary
(I) Igneous Rocks
(M) Metamorphic

Note: Current on-going research suggests both thresholds of picks and discs could be increased significantly with the use of high pressure water jet assisted cutting.
123. Their main use in tunnelling is on partial face or "road heading" machines; they have an obvious similarity of the cutting head to that shown in Figure 2 of the cutter suction dredger, and technology transfer is possible.

**Disc cutters**

124. This is a solid disc with a pointed circumferential edge. The disc operates as a free-rolling wheel. High applied thrust forces the disc into the rock in much the same way as a heavily loaded wheel rolling over yielding ground. Figure 3 shows the disc in its simplest form, but it can take on a multiple-edged or asymmetric configuration. Tunnel boring machines (TBMs) equipped with discs have very few cutting problems in rock up to an unconfined compressive strength of approximately 170 MPa. For rocks with higher strengths, penetration rates that can be achieved are lower and the cutter costs are higher. Drill-and-blast methods are usually favored for driving hard rock tunnels. However, recent improvements in disc bearing design and development of stiffer machines are increasing the maximum rock strength for practical mole excavation toward 240 MPa.

125. The problem in applying discs to a rock cutter dredging head is one of compliance. Extremely high thrust forces of over 6 tonnes would require substantial anchoring of the cutter head, and therefore this type of cutter would be unsuitable for use of rock cutter dredging heads. Obviously, a radical design is possible, but this would amount to a completely new one-off type dredger.

**Roller cutters**

126. This is a roller or star-wheel cutter, similar in concept and design to the disc cutter. However, its circumference is equipped with teeth, having an appearance very similar to that of a simple gear wheel. As each tooth engages the rock during free rotation of the wheel, a rock fragment is chipped away. This tool is extensively used in the oil industry for drilling large-diameter boreholes to great depth. High rotational speeds required for effective operation and the requirement for periodic flushing between cutting segments effectively preclude their use on moles.
127. It is believed by the writer that the application of this type of cutter to rock cutter dredging heads has merit, cutter head speed and slewing thrust having less constraint in dredging. There is, however, very limited data on groove spacing; size and yield and some basic research and development would be necessary before the full potential could be predicted.

**Carbide button cutters**

128. This grinding bit usually takes the form of a free-rolling cylinder or cone frustum, the surface of which is studded with tungsten-carbide buttons. It is operated in a fashion similar to the disc and roller cutter. A high thrust force is applied to the rock surface, and torque is applied to the cutter head to cause rock degradation by grinding and pulverization. This type of tool is suitable for cutting rocks with unconfined compressive strengths in the range of 170 to 200 MPa. Because of the high forces involved, the application of this type of cutting tool to dredger cutter heads is considered unsuitable.

**Rock-cutting Theories**

129. Experimental data on drag picks have been collated and analyzed by Mellor (1972). This report contains 46 references and is an excellent reference base for further research into single-tool laboratory research. The overriding conclusion of Mellor is that there are major shortcomings in rock-cutting theories and that results from experimental studies are essential. This is interesting, as the same conclusion was reached by Summers (1983) on water jet cutting and drag picks assisted by jet cutting. Another interesting, if not critical, point is that the intrinsic properties of the rock appeared largely to be ignored in any analysis. Moreover, it appears that even today very little is known of the mechanics of rock fracture initiation that affect mechanical cutting. Perhaps dynamic rather than static testing is the answer, which is, in fact, single-tool cutting research. However, it is fairly clear that uniaxial compressive strength is the major contributing factor to tool forces, and hardness of the constituents that make up the rock contribute to wear.
Theoretical Rock-cutting Models

130. A rock-cutting tool can be seen as an indentor that is scraped or rolled continuously across the advancing surface of the rock while being held forcibly into the rock. The force imposed on the rock and the equal and opposite reaction on the tool itself can be resolved into three orthogonal forces—a tangential component in the direction of cutting, a component normal to the plane of cutting, and a sideways component at right angles to both the tangential and normal forces. These components of cutting force are respectively known as cutting, normal, and sideways forces.

131. Over the years many researchers have tried to produce an analytical model involving property indexes of the rock and resolution of the induced forces on the rock-cutting tool. The basic problem in applying rigorous mathematical analogy to rock-cutting tools is the complexity of the structure of rock itself. Most theories make the basic assumptions of a linear elastic, isotropic, intact material but, in reality, very few rocks fall into this category. The second assumption is to ignore wear or change of shape of the cutting tool with time; this can, in practice, be the overriding consideration of cutter tool selection. The third assumption is to consider only an infinitely stiff cutting tool, ignoring machine compliance, which again can have a significant effect on tool selection. The use of a "soft" as opposed to a "rigid" mounting of the cutting tool allows the tool to partially ride over the rock rather than shearing or breaking discrete chips of debris.

132. As stated, Mellor (1977a) and also Phillips (1975) have published critical reviews of analytical theories of rock cutting in which some 60 references are cited. Both have come to the conclusion that the various cutting theories for rocks are useful in bringing disciplined thought to bear on the problem, but so far have not been directly usable for practicable design. This is not to say that work should not be encouraged in this area, but rather theories should be developed further to include the complexities of rock failure and machine and rock interaction.

133. The alternative, therefore, to a strict analytical approach is to assemble experimental data that can be used to develop a rational empirical approach. There has been, over the last decade or so, much interest shown in an empirical approach, and rock-cutting data can be found for most cutters.
However, there is a danger in applying "raw" data to machine design as most cutters examined during single-tool rock-cutting research are usually in a pristine condition. It is only when applied in the field for a few hours that their true merit becomes obvious. This is clearly shown in a critical examination of a report by Hurt and Evans (1980) on V-faced, round-nosed, and point attack picks cutting 70-MPa dale sandstone. On the initial few metres of cutting, the V-faced and round-nosed picks appear superior. But, after 600 m of cutting, the pointed attack pick had, by a significant margin, a greater overall cutting efficiency. This has been known for some years by users of partial-face machines, but has only recently been applied in basic rock-cutting investigations.

134. The above leads to the obvious conclusion that empirical cutting research should put as much emphasis on cutter tool life as cutter tool efficiency. Valuable research could be continued in collecting data on cutter tool life and increased specific energies with wear related to specific rock properties, especially for dredging where much higher hourly rates of excavation are expected and indeed obtainable.
135. Given the correct parameters, there appears no reason why knowledge gained in the tunnelling industry should not be directly transferable to the design of rock-dredging excavation equipment. The most obvious area of transfer is the application of drag pick data to the design of rock-cutting suction dredging heads. The mode of operation of a partial-face tunnelling machine and a dredging head are very similar, with the exception that a dredging head has to clear the cut debris and therefore requires an open head design to include suction ports for hydraulic disposal of spoil.

Cutter Head Design

136. Hignett (1979) examined the possibility of direct technology transfer, initially considering changing the type of cutting tools employed on a conventional dredging head as shown in Figure 2. This cutting head was having difficulty in tackling rocks whose compressive stress was much over 34 MPa. The task was to design a cutting head for limestone with a reported compressive strength of 80 MPa.

137. The initial approach was to establish why the original cutting head failed. Peak force cutting data was obtained for the 75-mm-wide cutting tools being employed, and this information was used to calculate the stress distribution and deflections that could arise on the helical cutter tool mounting arm. It was considered that this arm was in fact a fairly soft spring relative to the forces being applied and was considered unsuitable for the mounting of the rock-cutting tools. Further designs were considered using the existing cutting head providing stiffening braces but this was found to be incompatible with the other prime aim of suction dredging. Finally, it was decided to change to a radical design, using pointed attack picks and a very stiff open shell "monocoque" construction with scoops and space for hydraulic dredging as shown in Figures 5 and 6. Although a model of this design was built to study fabrication difficulties, because of a turndown in the dredging market no prototype was ever made and used. Indeed the provisional patent has now lapsed. Since the above design, the state of the art has improved with the advancement of high-pressure water jet assisted cutting. There appears to be
Figure 5. Dredging head "monocoque" construction
Figure 6. Cutter tool arrangement on "monocoque" constructed cutter head
no sound reason why mechanical dredging heads should not be designed to be put confidently to work on rock in the 100- to 140-MPa range. Development of such cutting heads associated with trials, given information on dredging rates, tool life, power levels, and other factors with rock classification, would provide a firm footing on which to base contract procedures and settle disputes.

138. It is ironic that for the price of just one average claim over and above the contract sum, say $10,000,000, most of the information presently lacking in dredging thresholds could be provided.

**Measured Performance of Rock Dredgers**

139. It was proposed by Gaye (1972) that the specific energy for a single cutting tool can be normalized with respect to the strength of the work material in order to obtain a dimensionless index that characterizes the efficiency of the tool in the cutting mode, viz:

\[
Es = \frac{FL}{L^3} = \frac{F}{L^2}
\]

where

- \(Es\) = specific energy
- \(F\) = force
- \(L\) = length
- \(\sigma_c\) = compressive stress

Hence, a plot of \(Es\) against the uniaxial compressive stress, \(\sigma_c\), will produce a dimensionless index.

140. The specific energy of a complete rotating rock-cutting dredging head is a function of the specific energy for its component tools, but is not necessarily simply related, since all tools are cutting at different depths. Nonetheless, a correlation between machine specific energy and rock strength can be expected.

141. Mellor (1977b) and Athorn, Farmer, and Glossop (1983) have amplified the relation proposed by Gaye by considering case histories of excavation machine performance, and suggest a possible basis for correlation of fracture
Figure 7. Dimensionless index, $E_s - \sigma_c$
mechanics and cutting performance. It is, however, stressed that much more reliable data are required before \( E_s/\sigma_c \) can be confidently used as a basis for a predictor equation. Nonetheless, the index does appear admirably suited for application to rock-cutter suction dredging, once reliable data are obtained. The application is illustrated in the following paragraphs.

142. As shown in Figure 7, Mellor (1977b) has plotted the specific energy, \( E_s \), against uniaxial compressive stress, \( \sigma_c \), for a variety of transverse rotation machines working in various materials. These machines include large disc saws, road planers, drum ice breakers, and mining and tunnelling machines.

143. The plot indicates that a realistic design goal is to aim for an \( E_s/\sigma_c \) value of 0.1. Also to be seen is that values of \( E_s/\sigma_c \) between 0.1 and 1.0 are readily obtainable. If analysis of performance for a machine shows \( E_s/\sigma_c \) approaching 10, there is good reason to suspect a serious fault in the design or the operating procedure.

144. The dimensionless performance index is useful in preliminary design for estimating power or performance and could be one method of indicating the current advancement in rock-dredging cutter head design and the possible improvement capable by technology transfer from tunnelling and mining research effort.

145. The basis of analysis is very simple and with minimal disturbance to production dredging a data bank of known performance could easily be established.

146. Once sufficient information has been collected, it is quite conceivable that on each dredging contract a base performance index could be established for a known rock. This could be used as a proportional payment basis for other rock formations. Obviously, other factors such as wear would have to be taken into account. It is also conceivable to be able, with experience, to judge the threshold between cutting and blasting for any particular machine, once the performance index, \( E_s/\sigma_c \), is established. This may well save much speculation both on site and through the courts. Machine evaluation does, however, require considerable experience.

147. The following is an example of calculations taken from information provided by Clark (1983). Typical values of output and cutter head power supplied are: 240 \( \text{m}^3/\text{hour} \) using a 75-kw motor on the cutter head. The 240 \( \text{m}^3/\text{hour} \) is the lowest excavation rate claimed by the manufacturer, presumably
at the strongest rock strength. Assume \( Es/\sigma_c = 0.25 \), which would be obtainable, but one has doubts, looking at the photograph supplied of the cutting head, however.

Let the volume per minute, \( V \), equal 240/60 or \( 4 \text{ m}^3/\text{min} \).

Let \( Pr = \text{power to cutting head} = 75 \text{ kw} \), neglecting frictional losses.

\[
Es = \frac{75 \times 60}{4} = 1125 \text{ KJ/m}^3 = 1.125 \text{ MJ/m}^3
\]

As stated, \( \text{MJ/m}^3 = \frac{FL}{L^3} = \frac{F}{L^2} \) = Same units as \( \sigma_c \) compressive strength.

\[
\sigma_c = \frac{Es}{0.25} = \frac{1.125}{0.25} = 4.5 \text{ MPa} \text{ which is typically a strong cohesive soil or a very weak rock}
\]

This would be the maximum compressive strength of the rock that the machine could excavate at--\( 250 \text{ m}^3/\text{hour} \).

148. As another example, from site investigation, there is a rock estimated at 34 MPa. The contractor claims he can dredge at a rate of \( 240 \text{ m}^3/\text{hour} \). A check can be made to establish if the cutter head is powerful enough for the task.

Assume again \( Es/\sigma_c = 0.25 \) or carry out tests on site to find ratio.

\[
\sigma_c = 34 \text{ MPa} \quad Es = \sigma_c \times 0.25
\]

\[
Es = 34 \times 0.25 = 8.5 \text{ MJ/m}^3
\]

\[
Es = \frac{Pr}{V} = \frac{\text{Power at head}}{\text{Volume removed}}
\]

\[
Pr = \frac{8.5 \times 240}{60} \times \frac{1}{60} \times 1000
\]

\[
= 566 \text{ kw}
\]

Below this power requirement, the contractor would have to blast. Obviously, this approach requires data and refinement, but once established, could be a very powerful approach for settling disputes and also provide a base for dredging head improvements.
PART VIII: WATER-ASSISTED DRAG BIT CUTTING OF ROCK

149. The basic limitations of using pure mechanical breakage of rock are the high cutting and thrust forces encountered. As Mellor has demonstrated, the energy required to be transmitted by the cutting head is proportional to the strength of the rock. Hence, if stronger rocks are required to be cut by conventional rock cutter suction dredging heads, then dredging power and overall size and weight of the barge will have to be increased proportionally to the increase in rock strength, or alternative methods of excavation will have to be considered.

150. One promising development in the last decade is the application of water-assisted drag bit cutting of rocks. It is the only technology at the present time that appears to be capable of improving the performance or reducing the weight of existing machines. Both field and laboratory experiments have been carried out since 1974 and have demonstrated the feasibility of such a technique.

Approaches

151. There are two distinct approaches that have been considered in the past. Initially studied in the United States (Wang, Robbins, and Olsen 1976 and Hustrulid 1976), then in the Federal Republic of Germany (Baumann and Heneke 1981), the first approach consisted of a separate action of a water jet and a cutter tool, taking advantage of a slot previously kerfed by the high-pressure water jet (up to 420 NPa), which resulted in a substantial reduction of the thrust force and torque in the machines in question. The second approach was first tried experimentally in South Africa (Hood 1976), and more recently in the United States (Wang and Wolgamott 1978) and Switzerland (O'Dubugon 1981), and consists of a combined action of a drag pick and water jet of a moderate pressure (140 MPa). The complexity of the water-assisted mechanism requires the consideration of a large number of parameters in any water jet study and consequently most experimental efforts to date have been kept to a moderate size, usually using a "standard" cutting tool and varying nozzle spacing and angle of attack. Nonetheless, the results appear very promising and it has been found that the power ratio between the water jets

51
and the tool and the rock play a dominant role in the process, while the tool geometry and the jet arrangement are of secondary importance.

Effect

152. The apparent success of jet-assisted cutting is explained by the postulation that the high-pressure water jet is injected into the cracks driven by the drag pick, producing a hydraulic fracturing effect, and in addition the rock surrounding the tool tip is highly stressed so that the inelastic dilatary effect generates a large increase of pore water volume, the pressurization of this porosity by the water significantly reducing the rock strength. It is in essence Griffiths' (1921) theory in reverse, where the tension cracks are initiated by positive pressure induced from within. However, whatever theories are put forward, akin to mechanical rock cutting, the proposals put forward for jet application to drag pick cutting are empirical and there is an obvious need to produce much more significant data, based on a large range of cutter tool applications on a variety of rocks and field applications. As low-pressure water jets are used for underwater cleaning of ships' hulls and sea structures (Summers 1983), presumably they will work on a submersible dredging head.

Trial

153. A current research project using a mining partial-face rock head with water jet assisted cutter is being carried out by the National Coal Board (UK) (Walker 1983) at their Middleton experimental mine. They report that limestone of 140 MPa unconfined compressive strength is being cut fairly easily using a 150-MPa water jet placed just in front of the drag picks.
154. Bray (1979) has discussed the writing of dredging contracts in detail, freely quoting from Wallace (1974) and Oosterbaan and Bean (1977). A reading of Bray gives the impression that it should be relatively straightforward to prepare a contract suitable to both parties, leading to a smoothly run operation, with minimum of claims, arbitration or protracted legal maneuvering. Unfortunately, this is rarely the case today.

Disagreements

155. Conflict of opinion often centers on what parties to the contract understand dredging conditions to be prior to the commencement of dredging compared with the actual conditions encountered. The purpose and effectiveness of special expedients also give rise to disagreements.

156. Resulting administration and staff costs dealing with such issues are incurred on all sides. In addition, the loss of working capital, with an attendant need for overdrafts or other financial arrangements (which have to be serviced), increases the contractor's financial burdens, all of which have to be borne by the industry and ultimately passed on to the promoters.

Parallel with Tunnelling

157. Possibly accentuated by a long period of low demand, bidding by contractors has been extremely competitive in dredging contracts. Much the same situation as dredging, regarding contracts and claims, has been experienced in recent years by the United Kingdom and United States tunnelling industry. Much dissatisfaction has been expressed with the method of writing and administering contracts for underground construction (CIRIA 1978; Dellaire 1976; Kellogg 1976; Kuessel 1977; Mayo, Barrett, and Jenny 1976; and U. S. National Committee on Tunnelling Technology 1974). Much of the criticism in both the United Kingdom and the United States has been directed against the lack of adequate definition of risk and apportionment of liabilities among the parties under contract. In the United Kingdom, the Construction Industry Research and Information Association (CIRIA) commissioned a report (CIRIA 1978) in which the definition of various contracts are given as follows.
Types of Contract

158. The types of contract most often used in tunnelling may be arranged in a hierarchy dependent on the way they allocate risk:

<table>
<thead>
<tr>
<th>PROMOTER'S RISK</th>
<th>TYPE OF CONTRACT</th>
<th>CONTRACTOR'S RISK</th>
</tr>
</thead>
<tbody>
<tr>
<td>100%</td>
<td>Turnkey</td>
<td>100%</td>
</tr>
<tr>
<td></td>
<td>Lump sum fixed price</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Lump sum - escalation</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Admeasurement</td>
<td></td>
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<tr>
<td></td>
<td>Target</td>
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<tr>
<td></td>
<td>Cost reimbursable</td>
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<td></td>
<td>Directly employed labor</td>
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</tbody>
</table>

Contracts where the contractor has the responsibility to design and construct and lump sum contracts offer the promoter few risks and may well be appropriate. Directly employed labor, by the elimination of the contractor, transfers the whole risk to the promoter. In these cases, the allocation of risk is clear and no secondary uncertainty is normally introduced by obliqueness or vagueness in the contract forms. However, for most tunnelling situations, an intermediate choice will be indicated by the aims of the promoter.

Admeasurement Type of Contract

159. Underground construction is normally carried out in the United Kingdom under the admeasurement type of contract. Two conditions having an important bearing on this choice of contract are:

a. The risk element arising from the ground should be capable of reasonable evaluation. If this cannot be done, then it should be withdrawn from the competitive element, or an alternative type of contract considered.

b. The engineer should be independent. If he is subject to pressures such that independence cannot be maintained, an alternative form of contract should be adopted.
160. When these conditions are satisfied, the admeasurement type of contract combines a number of requirements, which motivate the contractor to use his engineering skills and resources in estimating and to maintain effort and efficiency during the construction of the works. These requirements include the following:

a. The contractor is bound to rates quoted in competition, and so takes the risk of underpricing and bears the cost of his own inefficiency.

b. The contractor must provide for the risks allocated to him under the contract.

The contractor can escape from his price only by satisfying the engineer or arbitrator that extra costs are within a risk for which the promoter is liable.

161. The tendency over the years has been to lessen the risks for which the contractor is contractually liable. The financial risks placed on the contractor are mainly restricted to costs relating to the following:

a. Foreseeable ground conditions.

b. Weather.

c. Sufficiency of labor and plant.

d. Labor disputes.

e. Delays by, and defects in, work or materials.

f. Delays due to suppliers or subcontractors (excluding nominated suppliers or subcontractors).

g. Insurable risks.

h. Adequacy of bill rates.

i. Some effects of inflation.

Cost Reimbursement and Target Types of Contract

162. The use of a cost-reimbursable type of contract may be considered for the whole or part of the work in the following circumstances:

a. When insufficient information is available about the ground.

b. When there is insufficient time to prepare an admeasurement contract.

c. When it is desirable to use innovative methods for which little or no cost experience is available.

d. When contractors are not willing to respond to a high-risk venture.

163. A target type of contract may be preferred, which increases the
risk to the contractor to the extent of some restricted portion of the costs. The use of time targets may also be considered.

164. The responsibilities between the parties and the engineer undergo significant changes in these types of contract when compared with the admeasurement contract, particularly with respect to the clear-cut responsibilities which are traditionally those of the contractor: labor, plant, and the method of carrying out the work. The same is to be said of the relationship between their respective site staffs. It is a matter of primary importance in these types of contract that these responsibilities and relationships are explicitly stated and understood, and reliance is not placed upon traditional practice. Care should be taken not to obscure the responsibility of the contractor for the safety of his work force.

Working Relationships

165. A summary of recommendations for improved tunnelling contract practice, showing comparison between the United Kingdom and United States practice, is shown in Figure 8, of which most are applicable to dredging. Some of the differences arise because attitudes are noticeably different between the United States and the United Kingdom regarding the structure of responsibilities under contract and the control of disputes. In particular, the working relationship between engineer and contractor is affected by two aspects of construction practice, which deserve special attention: (a) the role of the engineer, and (b) the settlement of disputes.

Role of the Engineer

166. The engineer has traditionally been vested with broad power within the United Kingdom approach to tunnelling. The current Institute of Civil Engineers United Kingdom Contract Form (Institute of Civil Engineers 1973) recognizes these powers and their attendant responsibilities. The engineer is considered to be an independent arbiter of the rights of both promoter and contractor under the contract. This impartiality is taken seriously; it is institutionally part of the contract process. In United States practice, the engineer is regarded as the promoter's representative. If the engineer were to act in a dual capacity, there would be fear of a conflict of interest.
<table>
<thead>
<tr>
<th>US Recommendations</th>
<th>Current UK practice</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Rights of Way, etc.</td>
<td>Conforms to UK practice</td>
<td>No change</td>
</tr>
<tr>
<td>2. Disclosure of subsurface information etc.</td>
<td>Amount of information and its status left to individual judgement. Information given is 'taken account of' by tenderers</td>
<td>Procedure advised for tenders to be made on a common basis</td>
</tr>
<tr>
<td>3. Disclaimers</td>
<td>Spirit of Clause discourages disclaimers</td>
<td>Disclaimers should not be made</td>
</tr>
<tr>
<td>4. Changed conditions</td>
<td>UK practice Clause similar</td>
<td>Procedure to attempt to reduce risk by defining 'foreseeable'</td>
</tr>
<tr>
<td>5. Extraordinary water problems</td>
<td>UK practice Clause similar – common practice to bill pumping separately and this is encouraged by the CESMM</td>
<td>CESMM recommendations amplified</td>
</tr>
<tr>
<td>6. Types of contract – cost-reimbursable form</td>
<td>Conforms to growing practice</td>
<td>Practice encouraged for special cases</td>
</tr>
<tr>
<td>7. Bidder prequalification</td>
<td>Common</td>
<td>Recommended</td>
</tr>
<tr>
<td>8. Bid pricing</td>
<td>Conforms to UK practice in CESMM</td>
<td>No change</td>
</tr>
<tr>
<td>9. Alternative bids</td>
<td>Alternative bids are often made in current UK practice</td>
<td>No change</td>
</tr>
<tr>
<td>10. Escalation</td>
<td>UK practice includes recovery of a large part of the escalation in contracts for periods in excess of 12 months</td>
<td>No change</td>
</tr>
<tr>
<td>11. Wrap up insurance</td>
<td>Used increasingly for complex schemes</td>
<td>No change</td>
</tr>
<tr>
<td>12. Tunnel support</td>
<td>Conforms to UK practice</td>
<td>No change</td>
</tr>
<tr>
<td>13. Change negotiations</td>
<td>Becoming more common</td>
<td>Recommended</td>
</tr>
<tr>
<td>14. Value engineering</td>
<td>No formal procedure but principles understood and accepted</td>
<td>No change</td>
</tr>
<tr>
<td>15. Publication of Engineer's estimate</td>
<td>Not UK practice</td>
<td>No change</td>
</tr>
<tr>
<td>16. Contractor's financing costs</td>
<td>Conforms to UK practice</td>
<td>No change</td>
</tr>
<tr>
<td>17. Arbitration</td>
<td>Conforms to UK practice</td>
<td>Revised procedure advised</td>
</tr>
</tbody>
</table>

* Civil Engineering Standard Method of Measurement (Institution of Civil Engineers, 1976).

Figure 8. Comparison between United Kingdom practice and U. S. recommendations
167. In United Kingdom practice, serious contract disagreements are decided by arbitration. Engineers with experience in contractual matters are chosen as arbitrators. Their decisions are binding and final, except where points of law are involved. The procedure for arbitration can be arranged by the parties under contract and, thus, the practice carries a potential advantage in that procedure can be tailor-made to fit the particular work and disposition of the job participants.

168. Serious disputes on United States projects are settled in court. This frequently results in protracted legal maneuvering, which has been outlined and discussed by Kellogg (1976).

169. Arbitration is no guarantee that legal assistance will not be necessary nor that the loss of time and money will be averted. However, arbitration does keep the settlement of disputes within the circle of those familiar with and responsible for the working of the industry.
170. The following discussion has been modified from "Tunnelling - Contract Practice" (Construction Industry Research and Information Association 1978). The incidence of risk in dredging has been considered. These abridged principal proposals made in the report for action by the engineer, promoter (client), and contractor, are arranged in the sequence in which they would normally occur in practice:

- Formulation of project.
- Design and preparation of documents for the invitation to bid.
- Tendering and tender assessment.
- Dredging and postdredging.

**Formulation of Project**

**Promoter (Client):**
1. Will define aims.
2. Appoint an experienced engineer for the work.
3. Ensure the engineer has the necessary authority and independence.

**Engineer:**
1. Will only accept the appointment if his experience is adequate.
2. Will only accept the appointment if his independence is assured, or if any restrictions on that independence are explicitly stated.

**Design and Preparation of Documents for the Invitation to Bid**

**Promoter (Client):**
1. Will decide on the extent of the risk he is willing to take.
2. Determine the type of contract to be used.
3. Provide adequate funds for a comprehensive site investigation.
4. Select the list of contractors (preferably not more than six) to be invited to negotiate.
5. Allow an adequate negotiation period.
6. Not require contractors to carry out below-ground site investigation.
7. Provide adequate contingency funds.
Engineer:

1. Will advise the promoter (client) on the decisions to be taken on the proposals contained in (1) to (7) above.
2. Ensure the site investigations are carried out with both the requirements of design and construction in mind.
3. Ensure that site investigations are directed by engineers with wide dredging experience and geological expertise.
4. Give prospective contractors the opportunity to participate in the site investigation where appropriate.
5. Make final assessment on site investigation specialist's reports.
6. Formulate the reference conditions.
7. Give the basic design criteria for and specify such parameters as required.
8. Determine what information is to be presented by the contractors, dependent on the risk allocation between the parties.
9. Inform contractors of any limitations on the engineer’s powers.
10. Prepare the bills of quantities.
11. Place no disclaimer on the factual accuracy of site investigation information provided.
12. Place no requirement on contractors to make their own below-ground site investigations.
13. Provide full access to contractors to the relevant site investigation information.

Tenderers (Contractors):

1. Will be sure they have the necessary resources and experience to carry out the work.

Site investigation contractors:

1. Will ensure that they have the resources and experience to carry out the planned work.
2. Carry out work and prepare reports in accordance with recognized codes of practice.

Tendering and Tender Assessment

Promoter (Client):

1. Will make the award to the contractor most likely to fulfill the
aims of the project.

2. Satisfy himself that acceptable insurance has been offered.

Engineer:

1. Will make further site investigation information, arising during the negotiation period, available to all tenderers (contractors).
2. Not discourage tenderers (contractors) from offering alternative geotechnical solutions.

Tenderers:

1. Will challenge the ground reference conditions if they believe them to be misleading.
2. Indicate understanding and use of site investigation information.
4. Use the reference conditions as the basis for the physical conditions expected.
5. Consider the adequacy of the public liability insurance specified.

Dredging and Postdredging

Engineer:

1. Will monitor the ground against the reference conditions and direct dredging accordingly.
2. Set up and supervise a fact-agreement procedure.
3. Implement dilapidation schedule preparation and damage monitoring.
4. In the event of disagreement or dispute, consider early recourse to an independent expert, fact-finder, or arbiter.

Contractor:

1. Will set up procedures to agree on facts with the engineers.
2. Supervise and change staff where personalities persistently clash.
3. Cooperate with promoter (client) and insurers on third party damage resolution.
4. Make relevant disclosures of material changes to insurers.

Insurers:

1. Will make prior agreement with promoter (client) and contractor on third party damage procedure.
Arbitrator or independent expert:
1. Will encourage use of written evidence procedure.
2. Invite both parties to be present during his site investigation.
3. Provide copies of his reports to both parties not later than the preliminary pleadings (negotiations).

171. Standard forms for use with cost-reimbursable and target contracts should be prepared.

Discussion

172. Perhaps the most significant recommendation for improving United Kingdom contract practice has been the introduction of ground reference conditions as a basis for both contractor estimates and payments during tunnelling (Construction Industry Research and Information Association 1978), which could equally be applied to dredging. This would, in effect, expand the bill of quantities by setting definitions and bounds on various types of ground and associated rates of payment before construction. In this way, there is a clear allocation of risk between the contractor and promoter. The problem here is choosing a definition of the ground that reasonably reflects dredging requirements and rates of excavation.

173. The concept of ground reference conditions has generally been favorably accepted by participants in the United Kingdom tunnelling industry with the recognition that exceptionally complex ground behavior may be best treated by target or cost-reimbursable contracts (Mayo, Barrett, and Jenny 1976); whereas, relatively straightforward projects are best negotiated with the admeasurement contract as it currently stands. The U. S. National Committee on Tunnelling Technology (1974) has recommended that cost-reimbursable contracts should be explored as a possible solution to tunnelling under conditions of excessive risk.

174. Significant changes in the dredging industry of both countries would apparently result from changes in contract practice. Some of these changes are as follows:

a. Full disclosure of both geological data and interpretation in terms of dredging difficulty.

b. Increased awareness of different types of contract and their varying suitability for projects with different subsurface conditions.
c. Recognition of the risk elements inherent in a given contract and a corresponding attempt to distribute liability previous to the development of problems on site.

175. Many of the frustrations and attendant risks in dredging apparently stem from a tendency to categorize the problems peculiar to the client, engineer, and contractor as separate, self-interest concerns. In this regard, the most significant changes in dredging are likely to be changes in attitude. Improved dredging requires an emphasis on cooperation and mutual respect, especially between contractor and engineer. Projects are most likely to run smoothly when there is a clear apportionment of liabilities under contract and control exercised over job disputes with minimal litigation.

176. Obviously, with the paucity of data or reference bases available to both engineer and contractor, the initial impetus must surely be to encourage the exchange of information through a recognized forum and to research and develop areas where specific data is unavailable or suppressed by vested interests.

177. Finally, a similar position to dredging regarding control and monitoring through the contract stage is the application of the New Austrian Method of tunnel support. This method requires third-party site supervision, monitoring, and control, and although it has been in operation in Europe for over 20 years, the contracting system in the United States has negated similar application. In the United States, recent approval of the system has been reported by Whitney and Gilbert (1983); they state:

Owners must come to understand that the basic risk of variable conditions on a project belongs to them and the transfer of this risk is accomplished only through payment at a price relative to the magnitude of the risk. Some changes in United States contracting practice are necessary to achieve full benefit of response to changing ground conditions. It is possible to work toward the concept within the basic framework of the present United States contracting practice.
PART XI: CONCLUDING REMARKS

178. There appears to be no sound reason why mechanical dredging heads should not be designed to be confidently put to work on rock up to the 100-MPa range. Development of such cutting heads associated with trials, giving information on dredging rates, tool life, power levels, and other factors with rock classification would provide a firm footing on which to base contract procedures and settle disputes.

179. A dimensionless performance index is useful in preliminary design for estimating power or performance and could well be one method of indicating the current advancement in rock dredger cutter head design, and also the possible improvement capable by technology transfer from tunnelling and mining research effort. When developed, this index may also help to rationalize a particular dredger's performance.

180. There is an urgent requirement for research and development in a number of areas of dredger performance, correlation with ground conditions, and machine interaction and compliance. The basic requirement is to obtain case histories. On-going site studies could provide a most valuable source of practical knowledge for future assessment of dredging projects.

181. Site investigations should be considered as three phases: (a) preliminary--to establish a base for detailed studies, (b) secondary--involving, if necessary, geophysics, core logging and testing, and laboratory testing of samples, and (c) on-going--during the dredging process, with such site and laboratory tests as deemed necessary by both client and contractor.

182. Contrary to current practice, SPT and geological assessment by visual and scratch hardness are but a preliminary assessment for site investigation. Values of N over 50 for 30-cm penetration indicate that core taking and testing would be a wise precaution.

183. Contracts are extremely difficult to prepare and administer when high risks are involved. In present state of the art of rock dredging the paucity of data makes it difficult for judgement to be based on the criteria of sound technical assessment. Hence, at present, consideration should be given to admeasurement or target contracts until such times as reliable performance data are established.

184. Some changes in United States contracting practice are necessary to
achieve on-line reaction to changing ground conditions. It is considered possible to work toward this concept within the basic framework of the present United States contracting practice.
REFERENCES


