Commission of Inquiry
PARADISE DAM

PARADISE DAM COMMISSION OF INQUIRY

Commissions of Inquiry Act 1950

Section 5(1)(d)

STATEMENT OF JOHN YOUNG

Name of Witness:	**John Emmanuel Young**
Date of birth:	[redacted]
Current address:	**C/Stantec** **4934 Portage Road** **Niagara Falls, Ontario** **Canada, L2E6B4**
Occupation:	**Geotechnical engineer**
Contact details (phone/email):	[redacted] **john.e.young@stantec.com**
Statement taken by:	**Jonathan Horton QC and Thea Hadok-Quadrio**

I, John Emmanuel Young, Engineer make oath and state as follows:

Background

1. I am a Geotechnical Engineer and have been employed by Stantec Pty Ltd (**Stantec**) for about 4 years. I was employed by Montgomery Watson Harza (**MWH**) for about 10 years before it merged with Stantec. I am part of the MWH contingent within Stantec.

Witness	Taken by

2. I hold a Bachelor of Science from Memorial University of Newfoundland and a Master of Science Geology and Geotechnical Engineering and have worked as a geotechnical engineer, mostly in the field of hydroelectric energy, for more than four decades.

3. I am a registered professional engineer (P. Eng.) in the Canadian provinces of Ontario and British Columbia and in the Canadian territory Northwest Territories.

4. Virtually all of my work for MWH and Stantec is in the geotechnical aspects of hydroelectric projects in North America, Asia, Africa and South America.

5. A copy of my curriculum vitae is attached to this statement and marked **'JY-1'**.

Technical Review Panel (TRP)

6. From when I joined the TRP, I was the lead geotechnical advisor. I was not involved in the TRP from the beginning. The first meeting of the TRP that I attended was the second session, which took place in the last week of August 2019.

7. I participated in data review for the third session in early December 2019 at a teleconference with the GHD geologist and a subsequent examination of documents which had been provided to TRP members by Sunwater since the earlier meeting. I wrote the geology/geotechnical section in the TRP Report No 3 dated 9 December 2019 (**TRP Report No 3**).

8. I was retained by Sunwater. Peter Foster introduced me to the project. I am part of the Stantec team with Peter Foster and Glenn Tarbox.

9. I am usually referred to as an owner's engineer or expert advisor in projects like this. I would not consider myself a consultant on this job. The references on pages 3 and 4 of

Witness	Taken by

the TRP Report No 2 dated 23 September 2019 (**TRP Report No 2**) to 'the consultant' refers to the geologists from GHD Group Pty Ltd (**GHD**), not me. That is me saying that my opinion is that GHD seems to be doing a good job. I wrote all of section two of the TRP Report No 2.

10. There is no other geotechnical engineer or geologist on the TRP besides me.

11. In terms of TRP Report No 3, I wrote the text of Section 2 (the second-mentioned section 2), 'Geological/Geotechnical Update' (commencing on page 5). I was unable to go to Australia for the third formal workshop meeting of the TRP. My contribution to TRP Report No 3 was based on documents which were sent to me and a telephone discussion that I had with GHD's lead engineering geologist, Christopher Bennett, on 4 December 2019. Craig Hillier of Sunwater also participated in that telephone conversation.

12. Many of my discussions with the other TRP members and GHD engineers was about the bedrock geology mapping and what they had learned about the two faults, the Paradise Fault and the Apron Fault. My understanding of their comments is set out in the bullet points at the bottom of page 5 of TRP Report No 3. Virtually everything in this part of the report is my understanding of what they told me. Further down, I give my own comments towards the bottom of page 6 of TRP Report No 3 in the bullet points. These are my opinions, comments and suggestions of where things are going. On page 8 I express other opinions. I suggest, for example, that GHD use particular methods for assessing the strength of the rock mass and gave them three papers I thought they should think about.

13. The papers I suggested in TRP Report No 3 are:

Witness	Taken by

a. Hoek, E., Carter, T.G., Diederichs, M.S., "Quantification of the Geological Strength Index Chart", 47th US Rock Mechanics/Geomechanics Symposium held in San Francisco, CA, USA, 2013;
Attached to this statement and marked '**JY-2**' is a copy of this paper.

b. Hoek, E., Marinos, P., "Estimating the geotechnical properties of heterogeneous rock masses such as flysch", Bull. Eng. Geol. Env., 2001; and
Attached to this statement and marked '**JY-3**' is a copy of this paper.

c. Marinos, V., Fortsakis, P., Prountzopoulos, G., "Estimation of geotechnical properties and classification of geotechnical behaviour in tunnelling for flysch rock masses", Proceedings of the 15th European Conference on Soil Mechanics and Geotechnical Engineering, Athens, 2001.
Attached to this statement and marked '**JY-4**' is a copy of this paper.

14. Two fault zones are present in the dam foundation. (a) The Paradise Fault is a steeply inclined feature, reportedly about 400 metres long that runs right across the left side of the dam, almost normal to the dam axis, and (b) The gently inclined Apron Fault zone runs parallel to the dam axis along the rock just downstream of the dam, mostly beneath the concrete apron. The two faults come together near the left bank toe of the dam under the apron. These faults are not straight lines, they are zones. There are two intersecting zones of smashed-up rock that has been crushed and sheared. That is where the scour hole formed. Looking at the early reports, there was a hole about 6 metres deep which chewed out a big piece of the apron. This is very serious. If the flood in 2013 had persisted for a significantly longer period of time, erosion could have worked its way to the dam and started to undermine the rock under the dam. That did not happen but that is the kind of thing that, when one sees it, one realises one has a serious problem and one has to do something about it. When fixing it, it is essential to make sure it is not going to happen again.

Witness	Taken by

15. Most of that area of rock (ie where the faults intersect) is described as a geological term called 'melange'. This means a mixture of chaotically placed rock, some quite large - up to 10 or 15 metres - in a matrix of crushed material. So there are some plainer zones which are quite continuous but there are also some zones that have haphazard orientation. The net result is there is a poor quality rock mass that is somewhere between a rock and a soil in many areas. It no longer has the properties of solid rock. A reasonable dam builder and designer ought to have been aware of this melange area prior to construction.

16. It seems to me that the erosive forces of the water overtopping that section of the spillway dam were not taken into account in the dam's design. This is getting to the edge of my expertise as a geotechnical engineer, but the simple solution would have been that the apron should have been much wider so that the water would have slowed down a bit by the time it hit rock. Taking into account the melange area, a larger apron would have been a proxy for hard rock. I have been involved in large dam projects where exactly that has been done. The apron ought to have been significantly wider, extending downstream in that area and probably going up the bank as the rock in the river bank is not very good either. The rock had not been significantly damaged at the time of the 2013 flood, but it could well have been much worse if the flood had been more serious.

17. The Apron Fault lies under the apron. Both faults (this and the Paradise Fault) were probably covered by concrete at the time of the 2013 flood. The Paradise Fault runs upstream-downstream and extends well downstream of the Paradise Dam. Whether it is 200 or 400 metres does not matter greatly: the dam is still within the zone of it. My assessment is the erosion started along the zone of the Paradise Fault and then got much worse when it reached the intersection of the two faults where the zone of poor quality rock was much wider. The Apron Fault is made up of two meandering thrust fault and runs at right angles to the Paradise Fault. It runs under the apron and the toe of the dam.

Witness	Taken by

18. Under the right abutment there is highly weathered rock. I do not have a good sense for that as it is all covered now by concrete. GHD is looking at records. I was informed by them, during a telephone conference with the GHD site geologists on November 12, 2019, that this material is not now visible, as it was covered by the dam structure. Construction geological records show that it is poor quality. I agree that is likely to be correct. There are two concerns there:

    a. one is the strength of the rock mass. This impacts the sliding stability of the dam and possible erosion of the rock mass at the toe of the dam. The water has to be much higher to overtop the right abutment section of the dam (the right has a higher crest elevation than the left side) but if you had a flood much bigger than the one we had in 2013, it could conceivably start overtopping in that area. That is a concern;

    b. the second issue is the characteristic of the concrete/rock contact at the base of the dam. In TRP Report No 2 on page 8 and 9, Table 2.1 mentions 'open contact'. I produced this table from GHD data and I also looked at photographs. There are three possible conditions when you place concrete on rock for a dam:

        (i) first, you get a good, bonded contact. In other words, the concrete sticks to the rock. If you take a core sample through that contact it is like one piece of core. It has a good bond.

        (ii) secondly, the wall is intimately in contact with rock, but it might not stick together. The bond strength is low but you still have intimate contact. The rock surface is always irregular, so you have very good shear strength if you try to shear it;

Witness	Taken by

(iii) thirdly, the contact is 'open' which means it does not appear the contact between the rock and concrete is intimate or tight. You still have significant strength, but it is getting down probably below 45 degrees. It can be worse if the surface was not clean, if they left silt or sand on it, then really that sand is determining the strength at that particular point.

19. In TRP Report No 2, Table 2.1, I refer to there being 'tight contact' in some instances. It was not clear to me that there was good *bonding*, but there seemed to be very good *contact*.

20. In the cases of open contact, it did not appear to me, based on some of the core samples and on core photographs and descriptions, that there was good contact in some areas. That is not necessarily, ultimately, a problem, it just means that the strength will be a bit lower. It is not known how many occasions of open contact there are. This is something to be resolved in the ongoing investigations.

21. I did not have access to core testing or early testing that was done when the dam was designed and built before forming these opinions. Tests were available for this purpose, but to my knowledge there was no testing done of the actual contact, that is, of the rock and concrete.

22. The work I did is limited. I took other experts' opinions as a basis for the views I expressed and also relied on photographs. I also looked at some core samples of the RCC concrete (which is Photo 1 on page 12 of TRP Report No 2). This is probably the more serious issue. The photographs included some of the lift joints. One photo included the report is of a dirty, relatively smooth lift joint. They are not supposed to look like that.

Witness	Taken by

23. Ordinarily, there would not be testing done at the time of construction as to the adequacy of the contact between the rock and the dam foundation post-construction. In my experience, that is typically not done at that time. The designers and geotechnical people – that would include me – usually make a conservative assessment of what it is and assume that if they was good quality control during the concrete pour, that the necessary strengths and contact integrity would be achieved. Geotechnical and civil engineers inspect the rock surface before concrete is placed, but typically there are very few tests of contact (shear strength) between the rock and the foundation done, if any. There have been large dams I have been involved in where no rock/concrete tests were done but we were quite confident we had achieved good contact.

24. Golder Associates (**Golder**) carried out geotechnical evaluations during construction works and supervised the foundation preparation. I would expect, given my past experiences with Golder's methods and staff, that the work was competently done and that foundations, at least those supervised by them, had been properly prepared.

25. It is not possible at present to know the extent of any open contact under the dam wall. This remains a live issue.

26. In TRP Report No 3, Figure 2.1 on page 7 is a scanned portion of one of GHD's drawings. This shows inferred shear planes and breccias that are estimated by GHD geologists based on surface exposures mapped in bedrock outcrops and in the upstream wall of the scour hole. The horizontal feature under the dam is a "red flag" for me. It is indicated by the arrow in Figure 2.1. Such a feature can be a source of sliding instability similar a bad lift joint or a poor bedrock/concrete foundation surface. I included this as something to which consideration should be given. I emphasize that this feature is assumed and is not fully understood at this point. I recommend some analyses be done of that particular

Witness	Taken by

feature. To me, it is a red flag and is something that I do not like to see in dam foundations.

27. None of the geological information available to date means that the dam wall should never have been placed where it is. If the rock were totally inappropriate, cracking and settlements would have been seen already, and these issues do not seem to have occurred. There are ways to deal with weak foundations. For example, one way is to use a wider dam base to distribute the load. Another way is to install shear key into the rock under the dam. Simply speaking, a shear key is constructed by excavating a trench through the dam foundation (in this case a couple of metres past the horizontal feature shown in Figure 2.1 of TRP Report No 3) and filling it with concrete to reinforce the rock. These and other measures are commonly done to strengthen a rock foundation when deemed necessary. At the current time it is not apparent that any of the measures should have been installed for the Paradise Dam rock foundation.

28. I do not say the dam should never have built where it was: I do not suggest that this rock is of such poor quality such that a dam ought never to have been built upon it.

29. Foundation erosion is an important consideration for dam stability. In most places the upper zone of the rock mass is weathered and weakened. This surficial weathered zone, which is underlain by more competent unweathered rock, can be between approximately 2 and 10 metres down. Typically, most weathered rock is excavated from a dam foundation before the dam is built. On the right abutment, it appears that the dam was placed on weathered rock. Sheared and broken, unweathered, rock, is also relatively susceptible to erosion. The variably sheared melange rock of the left abutment is also susceptible to erosion, as was seen in the 2013 flood. A deep hole at the toe of the dam could undercut the dam foundation and seriously reduce sliding stability. If the 2013 flood had persisted for a significantly longer period, the left bank scour hole that occurred

Witness	Taken by

could well have worked its way back to the dam and caused some movement or sliding of it. As it was, it stopped well away from the dam and the dam was safe, but it is very worrying to see a 6 metre hole so close to a dam.

30. I am not an expert in roller compacted concrete (**RCC**) so the RCC aspects of the technical reports were not prepared by me. During my trip I did look at the RCC but it is not my expertise. Glenn Tarbox is an international RCC expert.

31. The designers and construction supervisory body carried out geological and geotechnical investigations before and during construction of the dam. This included various geological mapping efforts. There were drill holes and testing done on core samples during earlier investigations prior to dam construction. In my opinion there would have been difficulties in measuring the compressive strength of intact rock core samples. When testing rock in a melange, a lot of the rock has incipient fractures. In other words, one takes a solid piece of the core but there are little hairline healed fractures throughout. It is very difficult to test it with a conventional method, such as the unconfined compressive strength test, because it just breaks along the fractures and gives you a strength number that is very low. What tends to get done is that one takes the better samples of intact core with no hairline fractures to get test results with conventional testing and the result can be a false picture that the rock mass is actually stronger than it is. I am not saying that is what happened in this case, but it does appear to be something that might be relevant and ought be looked into further.

32. Golder produced detailed foundation geology maps under the footprint of the dam during dam construction. This is normal practice. The portion of the dam, where the concrete was placed, was very well mapped. GHD has been doing, in view of these other issues of erodability and possible shear failures, a lot of detailed geological work downstream of the dam. GHD engineers have been trying to match that work up with Golder's work

Witness	Taken by

to come up with a uniform geological map. I saw that work in progress during my trip there and they are doing a commendable job in my opinion.

33. I have not seen the Burnett River Dam Detail Design Report from 2004. I have seen summaries produced by GHD but I have not looked at the original document. I am not aware of any results of the shear testing of the concrete/rock contact. I have not spoken to Steven Tatro about that.

34. Where base cracking is referred to in the TRP reports, I assume that refers to a basal crack that can form in a gravity structure when it is subjected high loading conditions. The front of the dam, in this scenario, literally lifts off the rock a tiny bit and you get a crack, reducing the stability of the dam. This is built into most design stability analyses. This could potentially happen either at the rock-concrete interface or along one of the lift joints.

35. The GHD geological mapping divides the rock into 'domains', which are zones of relatively uniform rock. I made suggestions in both TRP reports that rock strength parameters must be assigned to each domain and also to specific smaller areas and features within the domains. It is one thing to have explanations of the geology, but in the end if we have a block of rock that extends under most of the foundations of the dam, we have to assess simple strength parameters – shear strength, friction angles, cohesion, and so on. GHD is cognizant of this requirement and is working on it. In both TRP reports I have made various suggestions along those lines pertinent to the rock conditions at the Paradise Dam.

36. I had some discussions in the field with GHD about rock mass strength and noted the incipient fractures in some of the rock which present difficulties in measuring true intact rock material strength. GHD had expressed concerns in deciding which is solid rock or

Witness	Taken by

not. I made comments on that because I felt my experience could be beneficial in this case. In TRP Report No 2, I noted there are three types of rock (see pages 7 and 8). I made a suggestion on treating the first rock type - brecciated material - as more like a soil, so using soil mechanics. I suggested that the second material type, with incipient fractures, is "in-between" material where care must be taken to take the incipient fractures into account when testing core samples for strength. The third type is solid rock, which is amenable to normal rock mechanics testing.

37. In TRP Report No 3, I noted that special considerations are needed for estimating the large scale strength parameters of melange rock masses which consist of a mixture of very solid rock with frequent zones of almost soil-like sheared/broken rock. For large scale strength estimations that cover the width of a dam foundation, the relative strengths of the broken/soil-like and solid rock zones must be integrated to estimate relatively simple average strength parameters. That is why I suggested the three papers given on page 8 of TRP Report No 3 (attachments JY-2, JY-3 and JY-4). This is not a criticism of GHD's work, but, once again, I offered some suggestions that are based on my experiences in similar rock masses.

38. I am unable to say how long GHD will need to complete the rock mapping. They seem very advanced. It ought to take no more than another couple of months. It is being set into a 3-dimensional model simultaneously. I have not seen the 3-dimensional model but I have seen the pencil markup.

Witness	Taken by

Commission of Inquiry
PARADISE DAM

OATHS ACT 1867 (DECLARATION)

I, John Emmanuel Young, do solemnly and sincerely declare that:

(1) This written statement by me dated 3 February 2020 is true to the best of my knowledge and belief; and

(2) I make this statement knowing that if it were admitted as evidence, I may be liable to prosecution for stating in it anything I know to be false.

And I make this solemn declaration conscientiously believing the same to be true and by virtue of the provisions of the *Oaths Act 1867*.

..... **Signature**

Taken and declared before me at Niagara Falls, Ontario Canada **this** ..... 3 ..... **day of** February .... **2020.**

Taken By .....

Justice of the Peace / Commissioner for Declarations / Lawyer

BEV HODGSON
Barrister and Solicitor
6057 DRUMMOND ROAD
NIAGARA FALLS, ON L2G 4M1

Witness	Taken by

JOHN E. YOUNG, M.Sc., P.Eng.

POSITION IN STANTEC: Senior Principal, Geotechnical Engineer

TOTAL YEARS OF EXPERIENCE: 43

ADDRESS

4934 Portage Road
Niagara Falls, Ontario
Canada, L2E 6B4

[redacted]

E-mail: john.e.young@stantec.com

SUMMARY OF EXPERIENCE

Mr. Young has extensive experience in the geotechnical and geological engineering aspects of major hydroelectric and large civil engineering projects in Canada and overseas. He is a geotechnical/geological engineer with over 43 years' experience in the geotechnical and geological engineering aspects for major hydroelectric projects in Canada and overseas. Previous works include performing geological investigations, geotechnical design and construction supervision for dams, tunnels and underground powerhouses, grouting design and supervision. Work on over 40 hydroelectric projects has resulted in a strong blend of field related practical experience combined with theoretical skills in analysis and design.

Mr. Young has worked on geotechnical foundation investigations, geotechnical designs and construction supervision for a variety of concrete and embankment dams. Notable dam projects include arch dams for the Karun 3, Katse and Jinping projects, a CFRD dam for the Genele Project, conventional embankment dams for the Long Pond, Conawapa and Marietta projects and concrete gravity dams for the Huanza, Susitna and Kalai 2 sites. He was the Resident Geotechnical Engineer responsible for all geotechnical construction aspects of the 200-m-high arch dam and 2,000 MW underground powerhouse for the Karun III Hydroelectric Project in Iran.

Project work includes extensive experience in the Himalayas and similar mountainous environments elsewhere. In Nepal, he has worked on feasibility investigations for the Nalsing Gad project, lender's engineer for the Upper Trishuli 1 Hydropower Project and dam safety/slope remediation works for Kali Gandaki hydroelectric station. Other experiences in the Himalayas include the Dasu Project in Pakistan and the Kalai 2, Singoli Bhatwari and Karcham Wangtoo projects in India. There have been numerous projects in the Andes Mountains which include Yaku, Santa Teresa and Huanza in Peru and Cachapoal, Chicayas, La Higuera and La Confluencia in Chile

He has carried out leading roles in the assessment, design and construction of numerous underground facilities, including three underground powerhouse complexes and over twelve hydroelectric water transfer and transportation tunnels. Notable underground powerhouse projects include the Karun 3 Project in Iran, the Santa Teresa Project in Peru and the Genele Hydroelectric Project in Ethiopia. He carried out rock mechanics assessments, design and construction inspections for numerous tunnel projects, including the Katse project Lesotho Highlands water transfer tunnel, the power tunnels of the Karun 3 project, the Genele Hydroelectric project in Ethiopia and various deep tunnels of the Santa Teresa Project and the Huanza Hydroelectric Project in Peru., Cachapoal, La Confluencia and La Higuera projects in Chile and the Chemoga Yeda Project in Ethiopia.

He has worked as owner's engineer and lender's independent engineer to provide due diligence monitoring during design and construction of numerous large hydroelectric and water transfer projects, including the Paradise Dam in Australia, San Gaban Project in Peru, Star Pumped Storage project in Israel, Murum Dam in Malaysia, the Genele Dam in Ethiopia, Muskrat Falls project in Canada and the Huanza dam in Peru and the Lesotho Highlands Water Transfer Project in Lesotho. His field and design experiences include the evaluation and geotechnical design for large projects in karstic limestone terrain. Projects with significant karstic issues included the Urumuka Hydroelectric Project in West Papua, Indonesia, mine facilities foundation design at the Cerra Corona mine in Peru, the Karun 3 arch dam in Iran and the Conawapa Project in Manitoba, Canada.

EMPLOYMENT HISTORY

2006 – Present	Stantec Consultants (MWH Canada Inc.) **Senior Principal, Geotechnical Engineer**
1980 – 2006	Hatch Acres Ltd. (formerly Acres International Limited), Niagara Falls, Ontario, Canada **Senior Geotechnical Engineer**
1976 – 1980	Advocate Mines Ltd, Baie Verte, Newfoundland, Canada **Geotechnical Engineer/Chief Mining Engineer**
1975 – 1976	University of Leeds, England **Graduate Studies**
1972 – 1975	Nolan White and Associates Ltd., St. John's, Newfoundland, Canada **Associate Geologist**

EDUCATION

1972	Memorial University of Newfoundland, Canada **B.Sc., Geology**
1976	University of Leeds, UK, **M.Sc., Engineering Geology and Geotechnics**

PROFESSIONAL REGISTRATIONS

Professional Engineers of Ontario, Member, License Number 90295536

Northwest Territories and Nunavut Association of Professional Engineers and Geoscientists, License Number L4354

Engineers and Geoscientists British Columbia, License Number 48202

PROJECT EXPERIENCE

2006 to 2019: Stantec Consultants (MWH Canada)

San Gaban 3 Project, Peru
Client: *Empresa de Generación Eléctrica SanGabán S.A.-(2019)*

- Member of the Owner's Engineer team to evaluate the geotechnical and geological aspects of the project. The project includes Intake works, a 15 km long headrace tunnel, pressure shaft and an underground powerhouse. It is located on the San Gaban River in the Peruvian Andes mountains.

The work included an assessment of the geotechnical aspects of the designs and walkover inspections of the sites of the surface and underground structures.

Lluclla Hydroelectric Project, Peru
Client: *Inland Energy SAC-(2019)*

- Member of the Owner's Engineer team to evaluate the geotechnical and geological aspects of the project. The project includes Intake works, a 15 km long headrace tunnel, 2 km long surface penstock and a surface powerhouse. It is located on the Huasamayo River in the Peruvian Andes mountains. The work included walkover inspections of the sites of the surface and underground structures, review of drill core samples and an assessment of the geotechnical aspects of the designs.

Paradise Dam Improvement Project, Australia
Client: *Sunwater (2019 to present)*

- Member of the Technical Review Panel to cover geological and geotechnical aspects of the improvement works for the Paradise Dam in support of Sunwater's design processes and studies. Paradise Dam is of RCC construction with a maximum height 53m. The structure, including the primary and secondary overflow spillways, has a total length of 920 m. Investigation works include geotechnical investigations of the RCC structure, bedrock foundation geology and various concrete laboratory tests. Ongoing assessments include overall sliding stability for various flood levels and bedrock scour issues in the area downstream of portions of the spillways. Alternative designs for remedial works are being carried out by the designers.

Cóndor Cliff-La Barrancosa Hydro Project (ex Néstor Kirchner-Jorge Cepernic), Patagonia, Argentina
Client: *EBISA ((2014 to present)*

- Technical advisor for geotechnical aspects of the 76 m high La Condor and 44 m high CFRD dams, both being constructed on the Rio Santa Cruz in Patagonia, Argentina. The dams are founded on river alluvium in a 1100 to 1300 m wide valley. The works include cement-bentonite slurry cutoff walls and large excavations in the glacio-fluvial deposits in the abutments to accommodate the large concrete spillways, powerhouse and intake/tailrace structures. Work included geotechnical evaluation of the overburden rock fill material, foundation requirements, input to grading design work and stability analyses.

Snowy Mountain Pumped Storage Project, Australia
Client: Salini Impregilo (2018 to 2019)

- Geotechnical engineering for contractor's EPC bid for the 2000 MW Underground powerhouse and various rock engineering aspects of the 23 km long headrace/Tailrace tunnel. Input for 3-D and 2-D finite element analyses of the underground power complex and power shaft.

Boundary Dam Power Station – Annual Dam Safety Assessment of Ash Retention Dykes Saskatchewan, Canada; Client Saskatchewan Power (2018)

- Principal Geotechnical Engineer for the geotechnical aspects of a dam safety study of Nipawin Power Station. Fly ash from the power station is discharged into six ash retention lagoons that have a combined surface area of approximately 60 Ha. The lagoons are retained by 6 to 20 m high dykes that are constructed of natural overburden fills and ash waste materials. This study was a mandatory dam safety assessment (DSA) that is carried out every year. The work included a review of existing documentation, a site visit, evaluation of risks and contributions to the DSR report.

Poplar River Power Station – Annual Dam Safety Assessment of Ash Retention Dykes Saskatchewan, Canada; Client Saskatchewan Power (2018)

- Principal Geotechnical Engineer for the geotechnical aspects of a dam safety study of the fly ash retention dykes of the Poplar River coal fired powerhouse. The BDPS ash lagoon facility comprises lagoons that are utilized for storage of ash waste materials. Perimeter dykes, 6 to 18 m high, for each lagoon are constructed of compacted clay (mine spoil) with surcharge dykes and interior dykes are constructed of compacted ash. The work included a review of existing documentation, walkover site visit, instrumentation review, evaluation of risks and preparation of a report. This study was a mandatory dam safety review (DSR) that is carried out annually.

Tajikistan Capacity Building and Technical Assistance Project, Tajikistan - Dam Safety Training Project

Client: European Bank for Reconstruction and Development (2016 to present)

- Geotechnical specialist for dam safety works for the Barqi Tajik, the Tajikistan electrical distribution authority. Conducted dam safety workshops in Dusahnabe and at the Baypaza Hydroelectric site, by making a number of presentations on the geotechnical aspects of dam design and safety assessments. Carried out a review of dam safety studies of the Nurek Dam, a 300 m high embankment dam. Carried out a walk-over dam safety condition assessment of the Baypaza Hydroelectric Project, including the 40 m high embankment dam spillway works, underground drainage galleries and associated civil structures as well as various large landslide features upstream and downstream of the project facilities.
- Further workshops and dam safety assessments will be carried out in 2020.

Asana Barrage, Peru

Client: Anglo American Mining (2016 to 2017)

- Principal Geotechnical engineer for investigations and geotechnical aspects of the construction of a 42 m high hardfill gravity dam at the Quellavaco Mine in Peru. The Asana dam is being built to provide river diversion around the planned open pit mine. The structure has mixed alluvium and bedrock foundations that required specialized analyses and designs.

Star Pumped Storage Project, Israel

Client: Bank Leume B.M. and Bank Hapoalim B.M (2016)

- Lender's Engineer geotechnical specialist to evaluate and monitor EPC design of the project. The project is being developed by Star Pumped Storage and is located on the right slope of the Jordan River Valley, approximately 10 km south of the Sea of Galilee. It consists of an artificial upper and lower reservoirs impounded by earth fill ring dams, water intake structure, 780 m long headrace tunnel, 450 m high vertical shaft, underground powerhouse with 340 MW installed capacity and a 1.6 km long tailrace tunnel. Reviewed the geotechnical aspects of the EPC contractor's designs for the surface and underground structures and participated in review meetings in Tel Aviv and at the site. Participated in assessments of geotechnical risks as applied to the financing of the project.

Kali Gandaki A - Hydropower Plant Rehabilitation Project, Nepal

Client: Nepal, NEA (2015 to 2017)

- Provide geotechnical input for the design of rehabilitation works for the Kali Gandaki Hydropower Plant in Nepal. Geotechnical work includes the evaluation of a large active landslide area near the dam and assessment of the instrumentation in the dam, diversion works and power facilities. The dam consists of 40 m high gravity sections and spillway gated facilities, a large desanding structure and a surface powerhouse.
- The project is constructed on interlayered phylites, slates and limestones. The weak phylites underlie the adjacent slopes and have unique slope stability challenges. The site contains a 90 m high unstable slope upstream of the left bank intake. The slope is actively moving, and more than 1.0 m of deep-seated slumping has occurred during the past decade. This instability threatens the main access road to the dam. Devised a program of drilling, slope movement monitoring instrumentation geophysics and laboratory testing investigations for the unstable slope Supervised

the geotechnical assessment and the design of remedial works. This work program is continuing through 2017.

Nalsing Gad Hydroelectric Project, Nepal
Client: Nepal, NEA (2016 to present)

- Principal Geotechnical engineer for investigations and geotechnical evaluation of the ongoing Nalsing Gad Hydroelectric Project. The project consists of a 150 m high dam, diversion tunnels, power tunnels and a 500 MW underground powerhouse. Responsible for the development and supervision of currently ongoing geotechnical investigations, including drilling, geophysics and rock mechanics testing for the project. Carried out walkover 1/2000 scale geological mapping of the damsite and concrete aggregate source area as well as a walkover reconnaissance of the main reservoir area. Responsible for the geotechnical aspects of dam design including foundation treatment and grouting works. The work includes evaluation of slope stability and watertightness of the 25 km long reservoir. Earthquake induced instability of the some of the high slopes is a concern and current work includes of wave modeling for theoretical catastrophic failure of large slopes.

Yaku Hydroelectric Project, Peru,
Client: Enersur, S.A. (2015)

- Principal Geotechnical engineer for investigations and geotechnical evaluation of the Yaku Hydroelectric Project. The project consists of a 130 m high RCC gravity arch dam, diversion tunnels, power tunnels and a 300 MW underground powerhouse. Responsible for the development and supervision of geotechnical investigations, including drilling, geophysics and rock mechanics testing for the project. Carried out detailed surficial geology mapping of damsite/powerhouse area and medium scale geological mapping of the planned reservoir.
- Identified and classified several instable high slopes in the reservoir and established special mapping and sampling at those locations. Participated in geotechnical design and supervised stability analyses of the dam foundation, tunnels, underground powerhouse and all surface excavations. Carried out stability analyses of critical slopes in the reservoir in order to determine their sensitivity to fluctuating reservoir levels and to establish filling and emptying rates during hydropower operation.

Lower Wanagon Overburden Stockpile Closure Design, West Papua, Indonesia,
Client: *PT Freeport Indonesia (2014 to present)*

- Principal Geotechnical Engineer for geotechnical aspects of the design of the Lower Wanagon Overburden Stockpile Closure Design facility of the Grasberg Mine in West Papua. The stockpile consists of dumped rock, mostly limestone, in the steeply inclined Wanagon River valley. The stockpile face had a vertical height of approximately 1200 metres. Work included geotechnical evaluation of the overburden rock fill material, input to grading design work and stability analyses. Supervised the seismicity evaluation of the project and derived seismic design parameters.

Nipawin 5 Year Dam Safety Review, Saskatchewan, Canada
Client: Saskatchewan Power (2017)

- Principal Geotechnical Engineer for the geotechnical aspects of a dam safety study of Nipawin Power Station. The power system consists of a 45 m high and 850 m long earth-fill dam, concrete spillway/intake structure, powerhouse, underground drainage tunnel, a fuse-plug dike, and an emergency spillway channel. The project has been operational since 1980 and this study was a mandatory dam safety review (DSR) that is carried out every 5 years. The work included a review of existing documentation, a site visit, instrumentation review, evaluation of risks and preparation of a report.

Nipawin Dam Annual Dam Safety Assessment, Saskatchewan, Canada
Client: Saskatchewan Power (2015)

- Principal Geotechnical Engineer for the geotechnical aspects of a dam safety study of Nipawin Power Station. The power system consists of a 45 m high and 850 m long earth-fill dam, concrete spillway/intake structure, powerhouse, underground drainage tunnel, a fuse-plug dike, and an emergency spillway channel. This study was a mandatory dam safety assessment (DSA) that is carried out annually.

Campbell River Annual Dam Safety Assessment, Saskatchewan, Canada
Client: Saskatchewan Power (2015)

- Principal Geotechnical Engineer for the geotechnical aspects of a dam safety study of Nipawin Power Station. The power facilities consist of a 2 km long intake canal, 33 m high and 770 m long earth-fill dam, concrete spillway/intake structure, and a 288 MW powerhouse with eight generating units. This study was a mandatory dam safety assessment (DSA) that is carried out annually.

Upper Trishuli 1 Hydropower Project, Nepal
Client: World Bank. (2015)

- Lender's Engineer geotechnical specialist to monitor EPC design and construction of the project. The project is located on the Trishuli River 50 km to the North of Kathmandu, Nepal. Flow is diverted by a gated concrete weir through an underground desanding structure into 10.4 km of low-pressure headrace tunnel which is connected to an underground powerhouse by a 292 m high pressure drop-shaft and 150 m of high-pressure tunnel. Reviewed the geotechnical aspects of the EPC contractor's designs for the surface and underground structures and participated in review meetings in Kathmandu. Led efforts to revise the seismicity criteria of the project and supervised the preparation of a new seismic rick study that incorporated the data from the 2015 Nepal earthquakes.

Las Bambas Mine, Peru,
Client: Las Bambas Mining Company S.A. (2015)

- Geotechnical engineer to evaluate foundation settlement issues for the two concentrate thickener facilities. These structures were under construction at the time of the evaluation and had suffered some non-uniform vertical settlements. Evaluated existing foundation information and deformation measurements and supervised some new core drilling investigations of critical areas. Recommended future instrumentation and designed micro-pile stabilization measures for one of the structures.

PTFI Pyrite Concentrate Storage Facility (PCON), West Papua, Indonesia,
Client: *PT Freeport Indonesia (2015)*

- Principal Geotechnical Engineer to provide technical review of prefeasibility level geotechnical investigations, foundation conditions and geotechnical designs for retaining ponds to store pyrite rich concentrates. This work was carried out by PTFI's consultants. The facility is founded on thick granular outwash deposits and the review included an assessment of the surficial geology and hydrogeology.

Susitna-Watana Hydroelectric Project, Alaska
Client: Alaska Energy Authority (2012 to 2013)

- Senior Geotechnical Engineer and Reviewer. Review of site conditions and geotechnical design criteria for feasibility design of a 200 m RCC dam, surface powerhouse and associated facilities in Alaska. Supervise geotechnical assessment of the dam foundation the development of finite element models for analysis.

Muskrat Falls Generating Station and Power Transmission, province of Newfoundland and Labrador, Canada:
Client: NALCOR, (2013 to 2017)

- Independent Engineer for geotechnical aspects review of final design and construction compliance of the project. The project, which is currently under construction, consists of a 30 m high North RCC dam, 20 m high south embankment dam, 824 MW surface powerhouse, approximately 1100 km of high voltage transmission line, a 30 km long sub2013 to 2017marine high voltage cable crossing between Newfoundland and Labrador, directional drilling for marine shoreline power cable boreholes. Carried out regular site inspections of the various works and participate in technical review of various design issues.

Santa Teresa II Hydroelectric Project, Peru:
Client: Empresa de Generation Electrica Machupichu, S.A. (2012 and 2013)

- Senior Geotechnical Engineer for prefeasibility studies of the planned 12 km long power tunnel. Carried out filed geological mapping at the damsite, along the 11 km tunnel route and at the surface powerhouse site. Carried out an engineering geology assessment of all of the principal project components and devised rock mechanics parameters for preliminary design. Devised rock support and prepared rock support drawings for design estimates.

Santa Teresa I Hydroelectric Project, Peru:
Client: Empresa de Generation Electrica Machupichu, S.A. (2011 to 2013)

- Senior Geotechnical Engineer for geotechnical evaluations, final design and construction evaluations design of a 300 MW underground powerhouse, and a 10 km long and 4 m wide power tunnel near Machu Pichu, Peru. Prepared design drawings for tunnel rock support, linings and dam foundation excavation.

Kurram Tangi Environmental Assessment, Pakistan:
Client: USAID (2013):

- Lead Geotechnical Engineer. MWH Lead geotechnical engineer for the geotechnical engineering aspects of the environmental assessment of the Kurram Tangi Dam Project. Critically assessed available geological information and geotechnical aspects of the design of the river control and irrigation project. Responsible for geotechnical evaluation of the field investigations and designs for the 90 m high main dam, the three weirs of 6 to 25 m height, 1900 m long water transfer tunnel and 2 power tunnels of 1500 and 100 m length respectively. Provided input regarding the appropriateness of geotechnical designs and recommended further studies for the next stage of project design.

Genale Hydroelectric Project, Ethiopia:
Client: Ethiopian Power Company (2011 to present)

- Owner's senior review geotechnical engineer for the design and construction of the Genele Hydroelectric Project in Ethiopia. The project consists of a 110 m high concrete faced embankment dam (CFRD), an11 km long/8m diameter power tunnel and a 260 MW underground power house. The project is being constructed as a design build project by CGGC of China. The power tunnel is being excavated by a TBM and all other underground excavation is by drill and blast.
- Carried out technical review and provided final approvals for of all aspects of geotechnical design. Participated in regular site inspections and project meetings with the design and construction staff. Notable aspects of my involvement include rock support and excavation considerations for the underground powerhouse, review of the TBM work, grout curtain criteria and AAR suitability of the concrete aggregate.
- Carried out assessment, analyses and design of rock support to stabilize 30 to 40 m high unstable rock slopes in the left side of the tailrace channel.

White River Hydroelectric Project, Canada:
Client: Regional Power OPCO, Inc. (2010 to 2016)

- Principal Geotechnical Engineer for design and construction reviews of the Upper and Lower Generating Stations. Each generating station is a two-unit hydro power plant with an installed capacity of 8.9 Megawatt (MW) and 10.0 Megawatt (MW), respectively. Each plant complex includes a 10 to 20 m high embankment dam and bedrock excavations, some 20 to 30 m deep

each, for the spillway and power canal works. Responsible for geotechnical assessment, construction materials investigations, rock mechanics design of surface excavations, grout curtain layout, dam type considerations, assessment of suitability of construction materials.

- Produced an air photo interpretation surficial geology map of the project area that included both damsites and the reservoir areas. Carried out walkover geology mapping of both damsites and produced 1/2000 scale geology maps. Carried out rock mechanics assessment of the bedrock, for the damsite and developed criteria for the geotechnical designs of the dam foundations, spillway/power channel excavations. Supervised slope stability analyses and the design of rock support for the 20 to 30 m deep rock excavations. Participated in the evaluation of the construction materials and directed alkali aggregate reactivity (AAR) testing of the potential concrete aggregate. Participated in the evaluation of various dam types and the selection of the final dam designs.

Ladore Falls Dam, British Columbia, Canada:
Client: British Columbia Hydro. (2011)

- Senior geotechnical engineer to carry out geological mapping and geotechnical evaluation of the bedrock foundation. This work was carried out as part of ongoing safety evaluations of the 60 years old Ladore Falls gravity dam and its purpose was to formulate shear strength parameters that could be used for new stability analyses. The geotechnical evaluation included the identification and rock mechanics assessment of various potential sliding planes in the dam foundation as well as a review of the concrete/rock contact. The work consisted of a field mapping program and assessment of construction data and earlier geotechnical evaluations.

Urumuka Hydroelectric Project, Indonesia:
Client: Freeport Mining (2011 to 2013)

- Senior geotechnical engineer responsible for the geotechnical aspects for feasibility investigations and geotechnical designs for a barrage, intake structure, 2 km long power tunnel and surface powerhouse in West Papua, Indonesia. This work included field assessments, supervision of field investigation programs and geotechnical design work for the final design of the project. The project is sited in rain forest jungle and is underlain by karstic limestone. The tunnel route traverses a large landside area.

Murum Hydroelectric Project, Malaysia:
Client: Sarawak Energy (2010 to 2011)

- Owner's Engineer for assessment of dam foundation stability for a 150 m RCC gravity dam in Sarawak. The dam, which was constructed under an EPC contract, is underlain by subhorizontal discontinuities and analyses were carried out by the design engineers to assess its long-term stability. As owner's representative I carried out a technical review of the designer's work and performing independent stability analyses for this project.

WAC Bennett Dam –Dam Safety Review, British Columbia, Canada
Client: BC Hydro and Power Authority (2011)

- *Geotechnical Engineer to* identify geotechnically related performance expectations, including flood and earthquake criteria, based on the BC Hydro Dam Safety Management Manual and CDA Guidelines; Review available documents for evidence of conformance with dam safety requirements; Determine the dam's conformance with the set of dam safety expectations; Identify any additional dam safety requirements to enhance risk management and to incorporate appropriate international practices. This was the mandatory dam safety review (DSR) that nut be done every 5 years.

Kalai II Hydroelectric Project, Arunachal Pradesh, India
Client: Larsen and Toubro (2010 to 2013)

- Senior Geotechnical Engineer for feasibility investigations and geotechnical design of a 160 m dam, alternative surface and underground powerhouses and associated tunnels for the 1200 MW hydroelectric project in northern India.

- Responsible for development and supervision of project definition geotechnical investigations for two alternative damsites. Participated in selection for final damsite. Supervised final geotechnical investigations for the final layout, including drillholes, geophysics, six exploration adits and in-situ rock mechanics testing program. Carried out geotechnical design and supervised stability analyses of the dam foundation, tunnels, underground powerhouse and all surface excavations. Supervised geological surveys and stability assessments for the reservoir. Led special studies to assess and design for the 300 m high natural slope in colluvial deposits in the left bank of the project area.

Panama Canal, Third Set of Locks Project, Panama
Client: Impregilo (2010 to 2011)

- Senior Geotechnical Reviewer for ongoing design work for excavation for the locks and lock chambers on the Pacific and Atlantic ends of the Panama Canal. The Lock excavations will be 20 to 40 m deep and will be cut into overburden and bedrock in an area subject to intense earthquake activity.

Oskan and Berkman Hydroelectric Projects, Turkey
Client: AEI Services LLC (2009)

- Lead Geotechnical Engineer to perform due diligence services for the Oskan and Berkman Hydroelectric Projects on the Ceyhan River, in southern Turkey. This project is at an advanced stage of construction and consists of two nearly identical hydroelectric developments that consist of 25 to 30 m high embankment dams, concrete gate structures and surface powerhouses of 25 and 35 MW capacities. Visited the site and reviewed and the design documents of the facilities. Carried out a number of limit equilibrium and finite element analyses to verify various project designs. The work included assessments of the embankment dams which are founded on thick alluvium at both sites, bedrock foundations of the concrete spillways and powerhouses, grout curtain designs, seismicity parameters and the stability of the100 m high excavated rock slope at the Oskan site.

.

Huanza Hydroelectric Project, Peru
Client: Empresa de Generación Huanza S.A (Minera Buenaventura), Peru (2009 to 2013)

- Senior Geotechnical Engineer for geotechnical evaluations and design of a 90 MW surface powerhouse, 10 km long and 4 m wide power tunnel and a 25-m-high concrete gravity dam of the Huanza Hydroelectric Project in the Andes Mountains of central Peru.
- Developed and monitored a program of geological mapping, geology compilation work, structural interpretations and geo-mechanical assessments in a sequence of folded and faulted andesite, pyroclastic and sedimentary rocks.
- Carried out geotechnical assessments and developed geotechnical design parameters for surface and underground works. Responsible for geotechnical design of the power tunnel, dam foundation and surface excavations for the powerhouse and various tunnel portals. Tunnel geology presented a number of tunneling challenges, including faulted and hydrothermally altered andesite and swelling/slaking tuffs.
- Carried out the design and stability analyses for a 120 m slope above the powerhouse in cemented colluvium. Designed and assessed slope monitoring of this and the adjacent slope during the period of construction.

La Confluencia Hydroelectric Project, Chile
Client: Pacific Hydro and the International Finance Corporation (2009-Present)

- Senior Geotechnical Engineer for due diligence review and assessment of potential squeezing rock problems and remedial rock support designs for power tunnel. Review and assessment of remedial rock support design and construction work.

La Higuera Hydroelectric Project, Chile
Client: Pacific Hydro and the International Finance Corporation (2009-Present)

- Senior Geotechnical Engineer for due diligence review and design studies to assess squeezing rock problems and remedial rock support designs for 16 km long and 5.9 m wide power tunnel.

The tunnel is constructed in non-durable argillaceous tuffs that are susceptible to time dependent squeezing deformations.

Waneta Hydroelectric Project, British Columbia, Canada
Client: Kiewit (2008 - 2009)

- Senior geotechnical engineer for geotechnical assessment and design work for design-build contract design of twin power tunnels and a 400 MW surface powerhouse in southern British Columbia.
- Assessed the engineering geology and geotechnical design of the existing gravity dam and power facilities. Directed additional field investigations for the planned new power facilities.
- Produced geotechnical designs for the power tunnel, surface excavations, cofferdams and related facilities.

Cachapoal Hydroelectric Project, Chile
Client: Pacific Hydro (2007-2009)

- Senior geological engineer in charge of geological mapping, drilling and geotechnical laboratory testing in the Andes Mountains of Chile. The planned project consists of four tunnels with a combined length of over 30 kilometres, four river diversion structures and an underground powerhouse with a planned capacity of 300MW.
- Supervised geological mapping, geology compilation work, structural interpretations and geo-mechanical assessments in a sequence of complexly folded and faulted andesite, pyroclastic and sedimentary rocks. Carried out a geological hazard risk assessment of the project. Produced regional and local geological maps and profiles for the various underground and surface structures.
- Participated in value engineering work with the owner's design engineers and the lending bank's review engineers to optimize all aspects of project design, excavation methodology and rock mechanics measures.

Jinping Project, China – Technical Review
Client: ERTAN (2007-2008)

- Senior Geotechnical Engineer. This project includes the world's highest arch dam, which is 230 m high, and underground powerhouse constructed in Sichuan Province.
- Participated in two missions, one-month duration each, to the Jinping project site in China.
- Carried out a review of the rock mechanics designs for the arch dam. Performed a series of finite element and a limit equilibrium analyses to verify project designs and geotechnical parameters for the dam abutments and large surface excavations.

Singoli Bhatwari Project, India
Client: Larsen and Toubro (2007 to 2010)

- Senior Geotechnical Engineer for design and construction review of the 90 MW surface powerhouse, 10 km long power tunnel and a 10-m-high concrete dam of the Singoli Bhatwari hydroelectric project in northern India.

Dasu Hydropower Project, Pakistan
Client: Water and Power Development Authority (2006 – 2009)

- Chief Geotechnical Engineer. Responsible for feasibility investigations and geotechnical design of the dams and underground powerhouse structures in the Himalayan Mountains of the Northwest Frontier Province of Pakistan.
- The project concept consists of 235-m-high concrete-faced rockfill dam (CFRD) and 105-m-long reservoir. The spillway will have six radial gates, each 20 m^2, and design flood capacity 23,000 m^3/s. The project has a 25 m wide, 400 m long, 60 m high underground powerhouse and five 10

m diameter tailrace tunnels that will be 5 to 7 km long. Total installed capacity of the power facilities will be 2,700-MW with 10 units.

- Carried out walkover field assessments of the various alternative dam and powerhouse sites during the Phase 1 site selection studies. Participated in site comparison studies and the selection of a final scheme that was studied to full feasibility level.
- Supervised a comprehensive site investigation program that included regional and detailed geological mapping, drilling of 25 boreholes, construction of 2 exploratory adits, materials testing and geophysical studies. Carried out geotechnical assessments of the investigations results, rock mechanics design of the dam, surface excavations and underground structures. Supervised all limit equilibrium and finite element stability studies for the slopes, tunnels and underground powerhouse.
- Supervised geotechnical design work for the dam foundation, power tunnels, underground powerhouse, tailrace tunnels and all surface excavations for the project.

Cerro Corona Gold Mine, Peru

Client: Gold Fields (2007)

- Principal Geotechnical Engineer to evaluate and carry out remedial designs for karstic cavities in the foundation area of the Mill complex. Carried out a geological assessment of the karstic limestone in the foundations area. Devised and supervised a program of closely spaced rotary percussion probe holes in the foundation footprints of the Sag Mill, Ball Mill and Surge Tank sites. The investigations identified and delineated several medium sized karstic cavities at shallow depts. In critical foundation areas. Carried out design work for remedial works that consisted of dental excavations and concrete back filling of the karst cavities.

.

Dniester Pumped Storage Project, Ukraine

Client: EDF/World Bank (2006 – 2007)

- Lead Geotechnical Engineer to perform due diligence services for the proposed World Bank investment program to complete the first phase (3 units, each 350 MW, out of a potential 7 units) of the Dniester pumped storage project.
- Visited the site and worked with the local Design Institute and Hydro Company to review and assess the design of the facilities. Carried out a number of limit equilibrium and finite element analyses to verify various project designs.
- The project consists of an artificial upper reservoir created by means of a 7 km long ring earth fill dam 26 m high; a water intake structure joining 7 separate intakes; vertical shafts (depth around 100 m and 7.5-m diameter), inclined penstocks (400-m length and 7.5-m diameter); individual pit powerhouses; and downstream inclined tunnels (120 to 150-m length and 8.2-m diameter 8.2 m).

Slave River Project, Alberta, Canada

Client: Trans Canada Pipeline (2006 – 2007)

- Senior geotechnical engineer for hydroelectric planning and feasibility studies on the Slave River in Northern Alberta.
- Carried out geotechnical feasibility level design works and assessments for the 2 km long, 45 m high planned Slave River Dam and 1200 MW surface powerhouse. This site is characterized by granitic riverbed foundation and glacio-fluvial riverbank deposits.

Albany River Project, Ontario, Canada

Client: OPG (2006 – 2007)

- Senior geotechnical engineer for geotechnical design for hydroelectric planning and pre-feasibility studies on the Albany River in Northern Ontario.
- Carried out geotechnical prefeasibility level design works and assessments for the Chard and Hat Island damsites. These sites are characterized by karstic limestone and weak shales foundations. The final dam will be a 25 to 35 m embankment structure with a surface powerhouse.

Khazir Gomel Project, Republic of Iraq
Client: Ministry of Water resources, Republic of Iraq (2006 – 2006)

- Assessed the engineering geology and geotechnical design of the planned 105 m high Bakerman damsite.
- Reviewed geotechnical criteria for arch dam and RCC alternative designs. Established geotechnical design parameters for the new RCC dam design

1980 to 2006: Hatch Acres Ltd. (Formerly Acres International Ltd.)

Karcham Wangtoo Project, India
Client: Jaiprakash Associates Limited (2006 – 2006)

- Senior Geotechnical Engineer for a due diligence review of the 100 MW underground powerhouse, 17 km long power tunnel and a 40-m-high concrete dam of the Karcham Wangtoo hydroelectric project in northern India. Acting as lender's engineer, carried out a site inspection and a detailed geotechnical review of all geotechnical design aspects of the project.

Forrest Kerr Project, British Columbia
Client: Bechtel (2005)

- Senior geotechnical engineer, responsible for feasibility level rock mechanics designs for a small dam, 3 km long power tunnel and 22 m wide underground powerhouse 2 km long access/surge tunnels, 0.3 km long tailrace tunnel and various utility tunnels.
- Carried out a geological and geotechnical assessment of the project participated in layout design work. Performed various stability analyses for the underground excavations and designed rock support for the underground powerhouse and the power tunnel, tailrace tunnel and various access and utility tunnels. Assessed potential hydrojacking issues in the powerhouse area and helped finalize the design of the steel lining in the power tunnels manifold area.

Final Impoundment of the Karun 3 Dam, Iran
Client: Iran Water and Power Authority (2005 – 2005)

- Impoundment Engineer. Supervised geotechnical monitoring and hydro-geological assessments for the filling of the 200 m deep, 50 km long Karun 3 reservoir.
- Carried out hydro-geological assessments, including finite element seepage analyses of various seepage issues in the karstic limestone reservoir rim.
- Prepared design and supervised construction of various flow-monitoring structures.
- Responsible for finalization of civil/geotechnical design and construction of numerous works in the dam, plunge pool, power tunnels and underground power complex.
- Supervised seismic monitoring and interpretation of reservoir induced seismicity, which developed as the reservoir was being filled.
- Carried out regular reviews and walk-over inspections of several critical reservoir slopes for the first year after initial impoundment. These slopes had been identified as having large mass movement features doing design studies of the project.

Dam Safety Condition Inspections – Mactaquac Dam and Milltown Dam
Client: New Brunswick Power (2004)

- Responsible for the geotechnical aspects of condition inspections of the geotechnical aspects of the 55 m high Mactaquac Dam (embankment dam) and the 15 m high Milltown Dam (concrete dam). This work was part of ongoing dam safety reviews of these structures. Both dams have with surface powerhouses. The Mactaquac Dam has a large concrete intake and spillway structure, both of which were suffering the effects of advance concrete aggregate reactivity. Inspections of these structures focused on the expansion cracking and displacements of the concrete structures and the condition of the drainage gallery under the spillway.

Inception Study for Condition Assessment of the Marun Dam, Khuzestan, Iran
Client: Khuzestan Water and Power Authority (2004)

- Carried out a site assessment and data collation for the geotechnical condition of the ten years old Marun dam in Khuzestan, Iran. Work included an assessment of the chemical and physical stability of the impervious core of this 150 m rock fill dam.

Shikwamkwa Dam, Ontario, Canada
Client: Brookfield Power (2004)

- Senior Geotechnical Engineer. Produced a seismicity study and formulated seismic design criteria for the design of the 35-m-high Shikwamkwa embankment dam in northwestern Ontario. Carried out an earthquake liquefaction assessment of the dam foundation.
- Carried out air photo interpretation material investigation for sources of construction materials in the dam site area. Produced designs for the foundation grout curtain and technical specifications for all grouting works.

Dam Safety Condition Assessment of the Kajakai Dam in Helmand Province, Afghanistan
Client: Louis Berger Engineering (for US AID) (2004)

- Senior Geotechnical Engineer. Carried out an on-site condition inspection and assessment of the 50 years old, 90 m high Kajakai embankment dam. The work included a rock mechanics evaluation of the uncompleted spillway excavation and a seismicity review of the project area.
- Performed stability analyses of the embankment dam and produced a dam condition assessment report.

Dam Safety Assessment of Chesterville and Chrysler Dams, Ontario Canada
Client: Second Nation Conservation Authority (2003)

- Senior geotechnical engineer for dam safety assessment of the Chesterville and Chrysler dams. These are 10 to 15 m high concrete water supply dams.

Karun III Hydroelectric Project, Khuzestan, Iran
Client: Iran Water and Power Authority (1997 – 2004)

- Resident Geotechnical Engineer. Responsible for all geotechnical construction aspects of the 200-m-high arch dam and 2000-MW underground powerhouse. The project was constructed in karstic limestone rock formations in the Zagros Mountains of western Iran.
- Duties include redesign of abutment stability measures, underground rock support and groundwater control. Supervised geotechnical aspects of construction of the underground works, including the 250 m long and 25 m wide powerhouse cavern, the 250 m long and 20 m wide transformer cavern, power intake portal works, four 700-m long, 13 m diameter power tunnels, two 14.2 m diameter tailrace tunnels and outlet portals. Supervised all monitoring and stability evaluations of the underground excavations once they were completed.
- Carried out on-site design modifications of the grout and drainage curtains as needed and supervised construction of grout curtain work in karstic limestone dam foundation.
- Designed rock support measures for the 200-m deep, 500-m long plunge pool excavation. Implemented slope stabilization measures for 50 m to 200 m high surface excavations for project facilities and natural slopes in unstable bedrock. The work included long term monitoring and remedial stabilization measures for the 300 m high G2M slope which experienced 20 to 40 cm movement per year during the period of 1997 to 2005.
- Identified potential reservoir rim leakage areas along various structural geology features in the downstream end of the reservoir and implemented drilling investigations and seepage analyses.

Technical Review of the Rafsanjan Tunnel, Iran
Client: Rafsanjan Development Authority (2003 – 2003)

- Senior Geological Engineer. Carried out a review of site conditions and geotechnical data for the proposed 56 km long Rafsanjan tunnel in the High Zagros Mountains of Iran.

- Assessed the engineering geology, rock mechanics issues and construction methodology for the tunnel.
- Produced a report summarizing the design issues with detailed recommendations for further design and investigation work.

Dam Safety Assessment - Caneadea Arch Dan, New York, USA.
Client: a New York power authority (2003)

- Senior Geotechnical Engineer. Responsible for the geotechnical aspects of a safety assessment of the 75 years old, 40-m-high Caneadea Arch dam in upstate New York.
- Carried out a geological assessment of the dam foundation and performed Londe type 3-dimensional sliding block stability analyses on large wedge features which were identified in the dam foundation.

Technical Review of the Shuibuya Project, China
Client: Employer: Acres International Limited (2002 – 2003)

- Senior Geotechnical Engineer. This project includes a 220-m high concrete faced rockfill dam and underground powerhouse constructed in a karstic limestone formation.
- Participated in three missions, one-month duration each, to the Shuibuya project site in China.
- Responsible for review and revision of rock mechanics designs for the underground structures and plunge pool. Carried out a series of finite element and a limit equilibrium analyses

Conawapa Generating Station, Canada
Client: Manitoba Hydro (2002 – 2003)

- Senior Geological Engineer. Carried out an engineering geology comparison of alternative dam sites for the planned Conawapa project in Manitoba. Work included a review of site investigation data, geological assessment and design of alternative grouted cutoffs.

Highway Slopes Evaluations, Ontario, Canada
Client: Ontario Ministry of Transportation (2001 – 2001)

- Project Manager. Slope stability assessments of 50 rock slope excavations along major highways in Ontario. Fieldwork was carried out using the Rock Hazard Rating system of the Ontario Ministry of Transportation (RHRON). The work included field assessments of the excavated slopes, hazard ranking, remedial designs and cost estimates.

WAC Bennett Dam, Canada
Client: British Columbia Hydro (1996 – 1997)

- Geotechnical Specialist. Worked on team of geotechnical specialists retained by British Columbia Hydro from a variety of Canadian consulting companies for investigations, monitoring and remedial grouting of sinkhole damage in the 180 m high WAC Bennett rock-fill dam in British Columbia, Canada.
- Shift Supervisor for round-the-clock surveillance of the dam and abutments. Supervised geophysics field investigations. Carried out design and cost estimating work for a proposed exploration adit in the dam fill. Supervised deep compaction grouting of the dam core.

Lesotho Highlands Development Project, Lesotho
Client: Lesotho Highlands Development Authority (1996 – 1996)

- Principal Geotechnical Engineer. Supervised the geotechnical work of consultants engaged in design and construction of the Lesotho Highlands Development Project. Monitored geotechnical aspects of construction of the 185-m-high Katse Arch Dam, the 55 m high Muela arch dam and approximately 40 km of water delivery tunnels.
- Reviewed and supervised geotechnical work for the 150-m-high Mohale concrete face rock-fill dam, 30 km of 5.0 m to 5.03 m diameter tunnels and the Matsuko Diversion Project.

- Responsible for monitoring reservoir induced seismicity during impoundment of the Katse Reservoir. Carried out a number of walk-over inspections of critical reservoir slopes to assess the impact of reservoir impoundment and reservoir triggered earthquakes.

Mohale Dam, Lesotho
Client: Lesotho Highlands Development Authority (1995 – 1995)

- Geological engineer for the Mohale Dam feasibility study in Lesotho. Responsible for field investigations, test quarry program and geotechnical evaluation of a 140-m-high concrete face rock-fill dam.

Alto Cachapoal Project, Chile
Client: Andrade Gutierrez (1994 – 1995)

- Geological Engineer. Designed and supervised field investigations in the Andes Mountains for six dams, two underground powerhouses and a total of 40 km of power tunnels.
- Supervised rock and soil mechanics laboratory testing and carried out geotechnical design work for various project facilities.

Keele Street Trunk Relief Sewer Project, Toronto, Ontario
Client: City of Toronto (1993 – 1994)

- Geotechnical Engineer. Carried out rock mechanics design work for a 4.3-km long, 5.2-m diameter sewer tunnel and four shafts. Prepared contract documents. The tunnel is in an area of very high horizontal stress and the designs had to cope with anticipated rock squeeze problems.

Takoradi Thermal Generating Plant, Takoradi, Ghana
Client: Employer: Acres International Limited (1993 – 1993)

- Geological Engineer. Foundation investigations and seismicity investigation for thermal generation plant.

Long Term Power Planning Study, Pakistan
Client: WAPDA (1993 – 1993)

- Geological Engineer. Carried out an assessment of the geothermal resources of Pakistan.
- Evaluated the geotechnical and seismological criteria for thermal power siting studies.

Aleltu Hydroelectric Project, Ethiopia
Client: Ethiopian Electric Light and Power Authority (1992 – 1993)

- Geotechnical Coordinator for feasibility study of the Aleltu hydroelectric project in Ethiopia. Responsible for site investigations for a number of earth-fill dams, 20-km of water transfer tunnels and a 1000-m head power facility.

San Gaban Project, Peru
Client: Cisel Engineering (1991 – 1992)

- Geotechnical Coordinator. Carried out geological mapping and established a field investigation program for detailed studies. Carried out geotechnical design for an underground powerhouse and 13 km of water delivery tunnel.

Power Planning Study, Sudan
Client: NEP (1991 – 1991)

- Geological Engineer. Reviewed the geotechnical design aspects of a number of proposed dam sites along the Nile River. Carried out a review of geothermal resources of Sudan.

Karun 3 Project, Iran
Client: Khuzestan Water and Power Authority (1989 – 1991)

- Geotechnical coordinator for Phase 2 geotechnical investigations and final design of the Karun III hydroelectric project in Iran. Duties included liaison with field investigation team, coordination of in situ and laboratory testing, formulation of geotechnical design criteria, supervision of geotechnical design work and preparation of contract documents for the Islamic Republic of Iran, Ministry of *Energy.*
- Supervised assessment of slope stability risk factor and watertightness characteristics of the 30 km long reservoir. Using air photo interpretation and walkover mapping, identified several large mass movement areas that included gravity deformed rock slopes ("sakung" slope sag type features) and relict landslides. Identified the geomorphological and geotechnical features of these features by geological mapping and in, in a few cases, exploratory boreholes. Carried out stability analyses to determine the influence of reservoir impoundment on slope stability. Assessed watertightness of the reservoir by a series of 2-D flow net analyses.

Conawapa Generating Station, Canada
Client: Manitoba Hydro (1989 – 1989)

- Geotechnical Liaison. Supervised test excavation, in-situ rock mechanics testing, temporary grout curtain construction, trial grout tests and geological mapping at the Conawapa Generating Station for Manitoba Hydro.
- The project, which consists of a planned 600-m-long fill dam and 1400-MW surface powerhouse, is founded on horizontally bedded karstic limestone. The test excavation program consisted of a 100-m-long, 30-m wide and 25-m-deep test excavation, an 18-m-deep shaft, a perimeter grout curtain and a number of test grout cells.
- Responsible for blasting design, undisturbed sampling, detailed geological mapping, in situ rock mechanics testing, excavation dewatering, rock slope stabilization, and design and execution of the test grout cells.

Karun Run of River Project, Iran
Client: Khuzestan Water and Power Authority (1988 – 1988)

- Staff Engineering Geologist. Provided technical assistance for pre-feasibility study of run-of-river hydropower tunnel scheme for the Great Hydropower Project of Khuzestan, high-level tunnel scheme for Dez-Ab Consulting Engineers for KWPA, Iran.

Chemoga Yeda Project, Ethiopia
Client: Ethiopian Electric Light and Power Authority (1987 – 1988)

- Supervisor of geological investigation. Supervision of geological investigations and design of geotechnical aspects of Chemoga Yeda, Ethiopia hydroelectric project, comprising six rock-fill dams, an underground powerhouse and 15 km of power tunnel.
- Carried out terrain and geological mapping of six reservoirs that varied in length from seven to eighteen kilometers. Carried out slope stability and reservoir watertightness assessments of each reservoir.
- Carried out pre-feasibility level geotechnical design work for EELPA.

Long Pond Reservoir, Newfoundland, Canada
Client: Newfoundland and Labrador Hydro (1986 – 1987)

- Resident Engineer. Managed and supervised construction work for the Long Pond reservoir improvements project in Baie D'Espoir, Newfoundland. Work consisted of crest raising of six dikes with a combined crest length of 3 km. Supervised pressure relief drilling program at the Salmon River spillway and grouting program at the Long Pond intake structure.

Long Pond Dam Remedial Grouting, Newfoundland, Canada
Client: Newfoundland and Labrador Hydro (1986 – 1986)

- Senior Geotechnical Engineer. Field investigations and supervision of the design and execution of a remedial grouting program to repair the causes of sinkhole development in a 15-m high fill dam in Newfoundland.

Geothermal Assessment, Kenya
Client: Kenyan Power authority (1985 – 1986)

- Engineering Geologist. Carried out detailed analysis of geothermal potential of the Rift Valley for incorporation in the Kenya National Power Development Plan.
- This study recommended the development of additional 300 MW of geothermal by the year 2005. Mr. Young also participated in a seminar on this topic held in Nairobi for Kenya Light and Power Company.

Godar-e-Landar Project, Iran
Client: Khuzestan Water and Power Authority (1985 – 1985)

- Technical Advisor. Served as technical advisor for geological investigations and design of geotechnical aspects of the Godar-e-Landar hydroelectric project, Iran.
- The project involved a 150-m high concrete arch dam and a 15-km power tunnel in sedimentary rocks in a highly seismic area for Ministry of Energy, Iran.

Chamera Hydroelectric Project, India
Client: Central Power Authority (1984 – 1984)

- Staff Engineering Geologist. Carried out rock mechanics design studies and analyses for the 25-m-wide underground powerhouse of the Chamera hydroelectric project in India for the National Hydroelectric Power Corporation, India.

Sentani Lake Project, Acres Niagara Falls Office, Canada
Client: Employer: Acres International Limited (1983 – 1983)

- Staff Engineering Geologist. Performed supervision of rock mechanics design work for the 2-km power tunnel and all surface rock excavations of the Sentani Lake, Indonesia.

Elkem Waste Retention Dam, USA
Client: Elkem Metals (1982 – 1983)

- Site Geotechnical Engineer and Resident Engineer. Supervised construction of the Stage II waste retention dam for Elkem Metals Company in Marietta, Ohio. Work included supervision of the contractor's operations in constructing a 30m-high embankment dam and excavating a 30-m deep by 500-m long spillway channel.
- Responsible for geotechnical quality control, monitoring of dewatering, the execution of a grouting program, and contract administration for all aspects of the work.

Lake Tana Regulating Dam, Ethiopia
Client: Ethiopian Electric Light and Power Authority (1981 – 1981)

- Lead Engineering Geologist. Supervision of geological investigation and geotechnical design of rock-fill dams of the Lake Tana regulating project. Fieldwork consisted of geological mapping, direction of drilling and test pitting operations and supervision of laboratory testing for EELPA

Karun 2 and 3 Feasibility Study
Client: Khuzestan Water and Power Authority (1980 – 1980)

- Lead Engineering Geologist. Engineering geology data assessment and stability analyses for proposed embankment dams of the Karun 2 feasibility study.

1976 to 1980: Advocate Mines Limited

Advocate Mines limited, Newfoundland, Canada

- Slope Stability Engineer (1976-1978) and Chief Mining Engineer (1979-1980).
- Carried out a slope stability study of the open pit mine. Developed and supervised a 3500-m geotechnical diamond drilling program and carried out surface geological mapping throughout the mine. Revised planned slope angles and worked with mine planning engineers in devising new long-range mine plans.
- Worked on the development of a wall control blasting program and developed an EDM slope movement monitoring program.
- Responsible for mine planning, ore grade control and mine engineering functions of a 16,000,000-t/yr open pit asbestos mine. Supervised mine engineering department of 8 engineers and technicians.
- Reported directly to the mine manager and interfaced with the Board of Directors in developing long- and short-range mining strategy.

1972 to 1975: Nolan White and Associates Limited, St John's, Newfoundland

Engineering Geologist

- Worked on a variety of groundwater projects, foundation investigations, drilling and materials investigations within the province of Newfoundland and Labrador

Hoek, E., Carter, T.G., Diederichs, M.S.

Quantification of the Geological Strength Index Chart

This paper was prepared for presentation at the 47th US Rock Mechanics / Geomechanics Symposium held in San Francisco, CA, USA
June 23-26, 2013

COPYRIGHT NOTICE

ARMA 13-672

Quantification of the Geological Strength Index chart

Hoek, E.
Vancouver, British Columbia, Canada
Carter, T.G.
Golder Associates, Toronto, Ontario, Canada
Diederichs, M.S.
Queens University, Kingston, Ontario, Canada

This paper was prepared for presentation at the 47th US Rock Mechanics / Geomechanics Symposium held in San Francisco, CA, USA, 23-26 June 2013.

ABSTRACT:

The original Geological Strength Index chart was developed on the assumption that observations of the rock mass would be made by qualified and experienced geologists or engineering geologists. With the ever increasing use of the GSI chart as the basis for the selection of input parameters for numerical analysis, often by individuals without the strong geologic understanding of rock mass variability necessary to interpret the graphical GSI chart properly, some uniformity and quantification of the chart seems necessary. This paper presents a proposed quantification of the GSI chart on the basis of two well-established parameters - Joint Condition and RQD. Recommendations for future development of more robust scales are presented.

1. INTRODUCTION

The original Geological Strength Index (GSI) chart was developed on the assumption that observations of the rock mass would be made by qualified and experienced geologists or engineering geologists. When such individuals are available, the use of the GSI charts based on the descriptive categories of rock mass structure and discontinuity surface conditions have been found to work well. However, there are many situations where engineering staff rather than geological staff are assigned to collect data, which means that the mapping of rock masses or core is carried out by persons who are less comfortable with these qualitative descriptions.

As part of an ongoing evaluation of the uses and abuses of the Hoek-Brown and Geological Strength Index systems for estimating the mechanical properties of rock masses, the issue of quantifying GSI has been given priority. GSI is the first point of entry into the system and, unless this Index is well understood and applied correctly, the reliability of the estimated properties is open to question.

Figure 1 illustrates the data flow when using the GSI/Hoek-Brown method for estimating the parameters required for a numerical analysis of underground or surface excavations in rock. Depending on whether the users have a geological or an engineering background, there tend to be strongly held opinions on whether the observed geological conditions should be entered either descriptively or quantitatively into the characterization table for GSI. Both of these approaches are catered for in the discussion that follows.

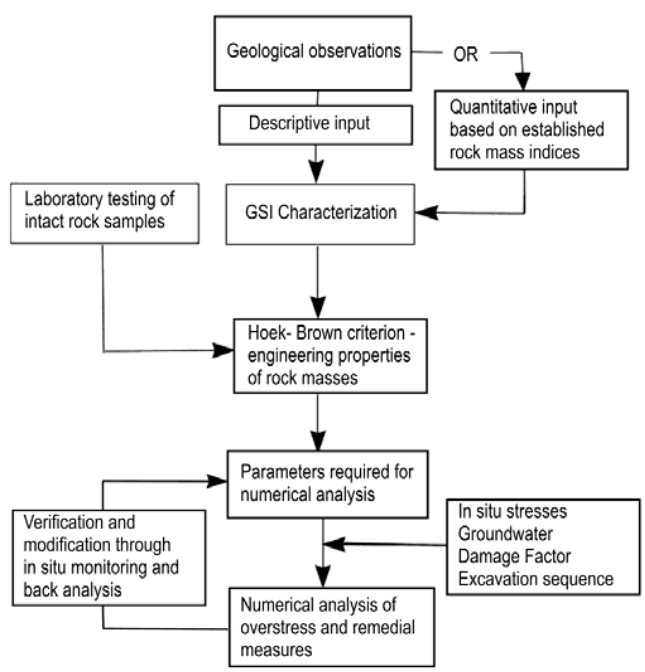


Figure 1: Data entry stream for using the Hoek-Brown system for estimating rock mass parameters for numerical analysis.

2. CONSTRUCTION OF THE BASIC GSI CHART

The GSI chart published by Hoek and Marinos (2000) [1] is reproduced in Figure 2. Scale A has been added to represent the 5 divisions of surface quality with a range of 45 points, defined by the approximate intersection of the GSI = 45 line on the axis. Scale B represents the 5 divisions of the block interlocking scale with a range of 40 points in the zone in which quantification is applied.

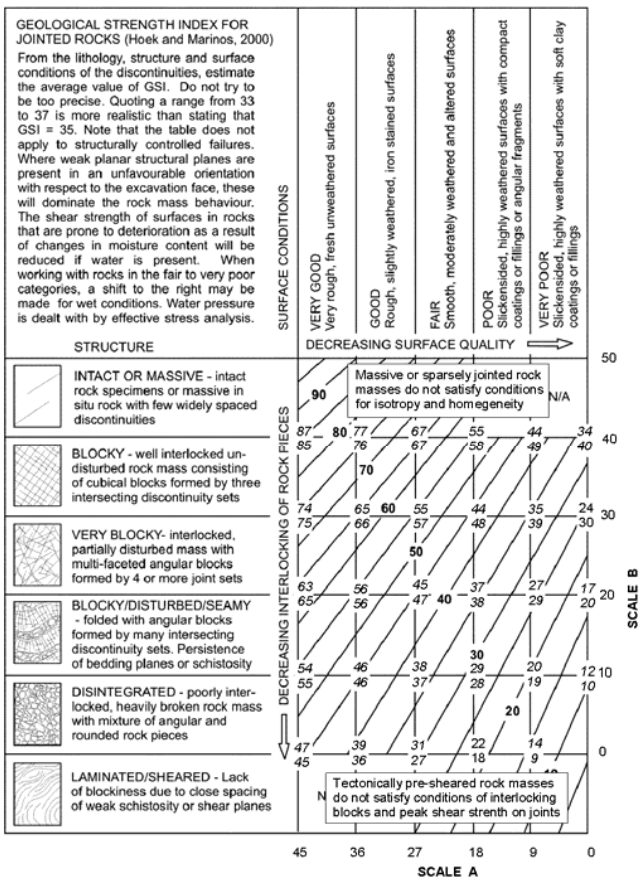


Figure 2: The basic structure of the Hoek and Marinos (2000) GSI chart and possibilities for quantification.

At each intersection of the A and B scales the value of GSI has been estimated from the GSI lines on the chart. These values are shown as the upper italicized number at the intersection point. At the same intersection points the lower italicized number equals the sum of the A and B values. The two numbers at each intersection point are then plotted against each other in Figure 3.

This plot demonstrates that there is a high potential for quantifying GSI by means of two linear scales representing the discontinuity surface conditions (scale A) and the interlocking of the rock blocks defined by these intersecting discontinuities (scale B).

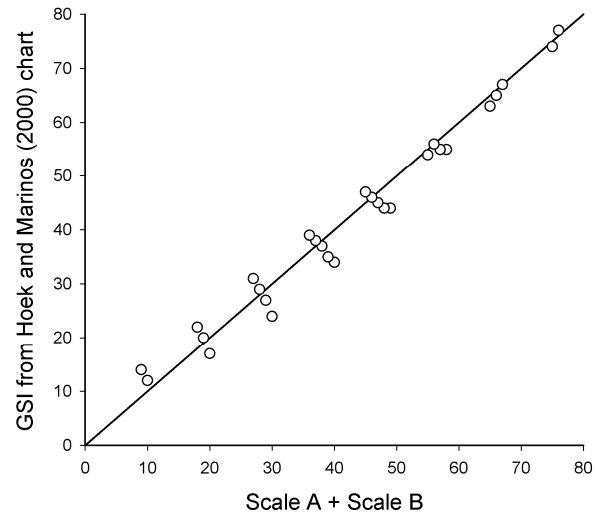


Figure 3: Plot of GSI estimated from the basic GSI chart against the sum of the A and B values.

Figure 3 also shows that there is a systematic trend in each group of plotted points and, from an examination of the chart in Figure 2, it is obvious that this trend is due to the fact that the original GSI lines, which were hand drawn, are neither parallel nor equally spaced.

With a modest correction to the original GSI lines to make them parallel and equally spaced, the error trends in Figure 3 can be eliminated completely. This correction has been applied to Figure 5.

Note that the correction of the GSI lines and the addition of the A and B scales do not change the chart's original function of estimating GSI from field observations of blockiness and joint condition, characterized in terms of the descriptive axis title blocks. Hence the chart shown in Figure 5 has the potential for satisfying both the descriptive and quantitative user camps.

Before proceeding any further with this discussion it is necessary to define a number of conditions and limitations of the proposed quantitative GSI chart.

1. The addition of quantitative scales to the GSI chart should not limit the use for which it was originally designed – the estimation of GSI values from direct visual observations of the rock conditions in the field.

2. A fundamental assumption of the Hoek-Brown criterion for the estimation of the mechanical properties of rock masses is that the deformation and the peak strength are controlled by sliding and rotation of intact blocks of rock defined by intersecting discontinuity systems. It is assumed that there are several discontinuity sets and that they are sufficiently closely spaced, relative to the size of the structure under consideration, that the rock mass can be considered homogeneous and isotropic. These concepts are illustrated diagrammatically in Figure 4.

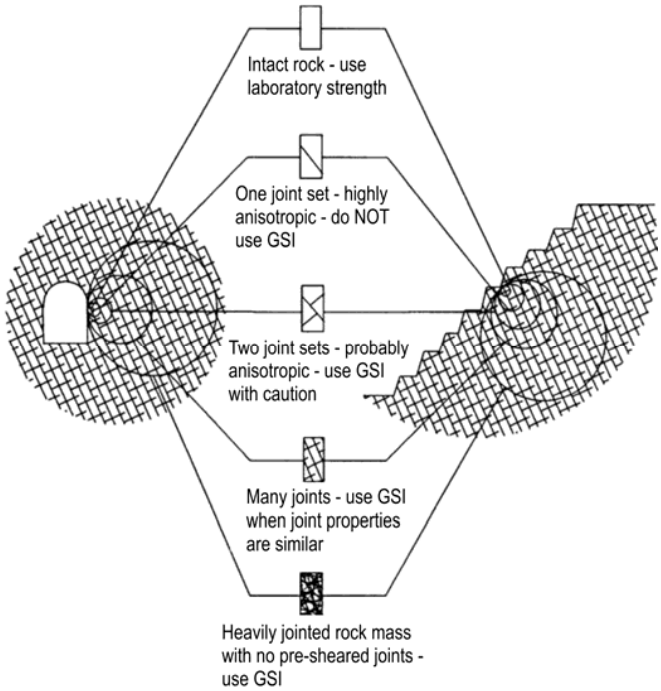


Figure 4: Limitations on the use of GSI depending on scale.

3. For intact massive or very sparsely joined rock, the GSI chart should not be used for input into the Hoek-Brown criterion as shown in Figure 1. This is because there are insufficient pre-existing joints to satisfy the conditions of homogeneity and isotropy described above. Hence, in order to avoid confusion, the upper row of the chart shown in Figure 2 has been removed in the development of the quantified GSI chart. Brittle failure processes such as rockbursts and spalling are specifically excluded from the section of the quantified GSI chart since these processes do not involve the rotation and translation of interlocking blocks of rock as defined in 2 above. Similarly, structurally controlled failure in sparsely jointed rock does not fall within the definition of homogeneity inherent in the definition of GSI.

4. The lower row of the original 2000 GSI chart has also been removed since this represents previously sheared or transported or heavily altered materials to which the conditions defined in item 2 above also do not apply. A second GSI chart for heterogeneous, pre-sheared materials such as flysch has been published by Marinos and Hoek (2002) [2] and Marinos et al (2007) [3]. Where applicable this flysch chart could be used or a similar site specific chart could be developed for rock masses that fall below the last row of the chart given in Figure 5.

Some approaches for tackling both ends of the rockmass competency scale addressed in paragraphs (3) and (4) are suggested by Carter et al, 2008, [4].

5. In order to quantify GSI using the chart, the quantities used to construct the A and B scales have to be practical ratings that are familiar to engineering geologists and geotechnical engineers operating in the field. They should also be well established in the literature as reliable indices for characterizing rock masses intersecting discontinuity systems. It is assumed that there are a sufficient number of discontinuities and that they are sufficiently closely spaced, relative to the size of the structure under consideration, that the rock mass can be considered homogeneous and isotropic.

3. ESTIMATION OF GSI IN TERMS OF RQD AND JOINT CONDITION

Scale A in Figure 2 represents discontinuity surface conditions while Scale B represents the blockiness of the rock mass. Prime candidates for these scales are the Joint Condition ($JCond_{89}$) rating defined by Bieniawski (1989) [5] and the Rock Quality Designation (RQD) defined by Deere (1963) [6]. These ratings are given in Appendix 1.

The $JCond_{89}$ rating corresponds well with the surface conditions defined in the text boxes of the x axis of the GSI chart in Figure 5. This rating parameter has been in use for many years and users have found it to be both simple and reliable to apply in the field.

The RQD rating has been in use for 50 years and some users have defined it as boringly reliable. Hence these two ratings appear to be ideal for use as the A and B scales for the quantification of GSI.

Figure 5 shows a chart in which the A scale is defined by 1.5 $JCond_{89}$ while the B scale is defined as RQD/2. The value of GSI is given by the sum of these scales which results in the relationship:

$$GSI = 1.5\ JCond_{89} + RQD/2 \qquad (1)$$

4. CHECK OF QUANTIFIED GSI AGAINST MAPPED GSI

In order to check whether or not the proposed quantification of GSI works it is necessary to check the values of GSI predicted from equation 1 against field mapped GSI values. At the time of writing only one set of reliable field data, from a drill and blast tunnel, is available to the authors. The GSI values calculated from $JCond_{89}$ and RQD are plotted against mapped GSI values in Figure 6. This plot shows that the correlation between the calculated and mapped GSI values is reasonably close to the ideal 1:1 relationship for a perfect fit. This suggests that, once additional field data are obtained, the application of this quantification of GSI may justify the transition from proposed to recommended.

It is possible that some adjustments in the positions of the $JCond_{89}$ and RQD scales in Figure 5 may be required as more mapped GSI data becomes available and as experience is gained in using this quantification.

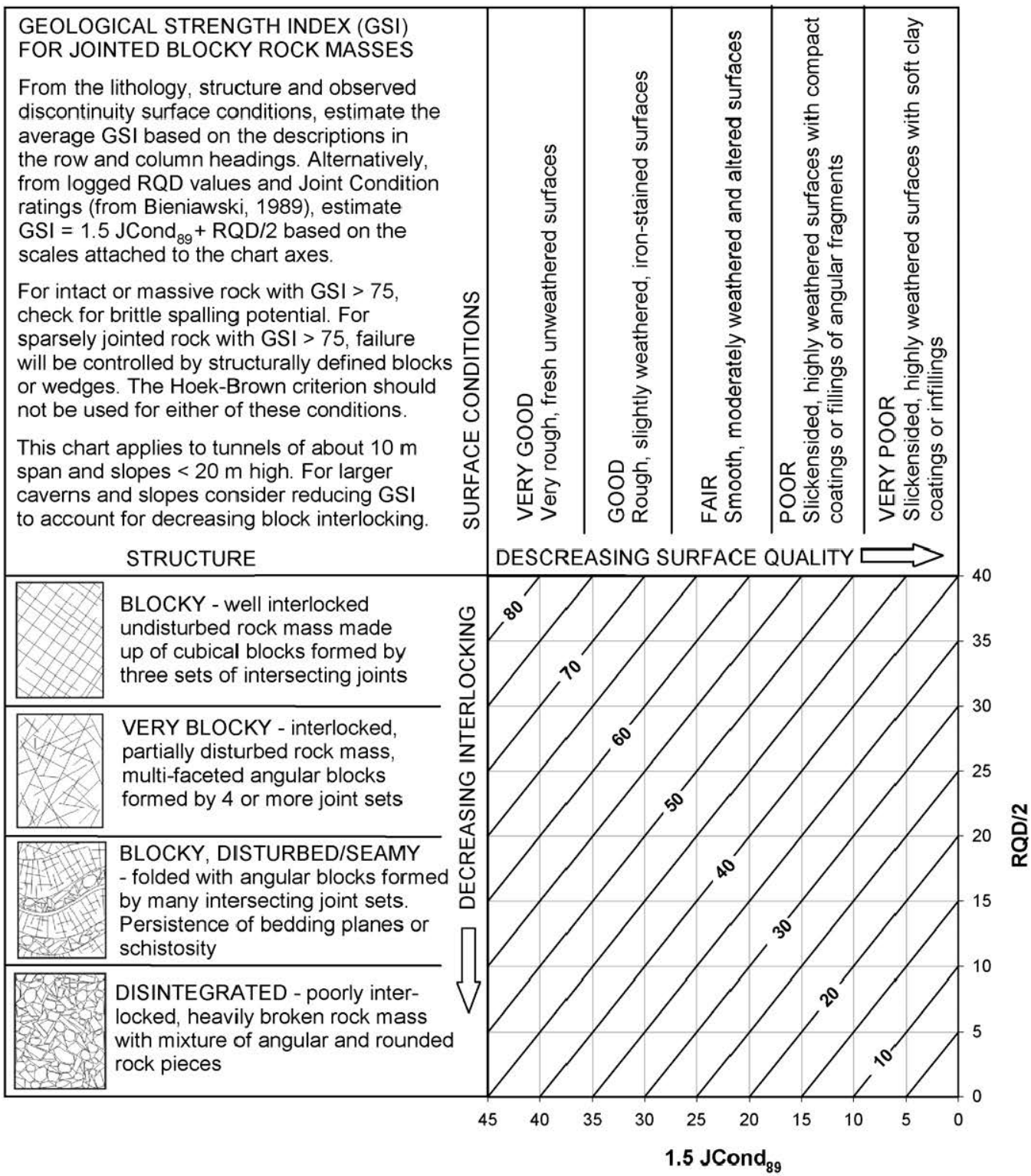


Figure 5: Quantification of GSI by Joint Condition and RQD.

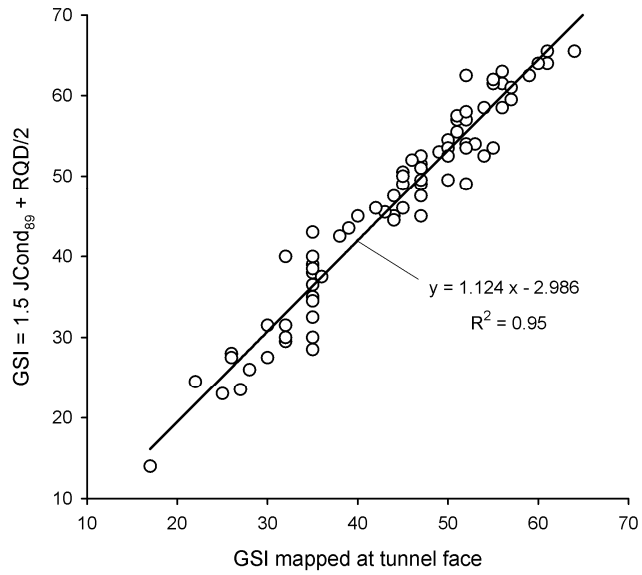


Figure 6: Comparison between mapped GSI and GSI predicted from $JCond_{89}$ and RQD.

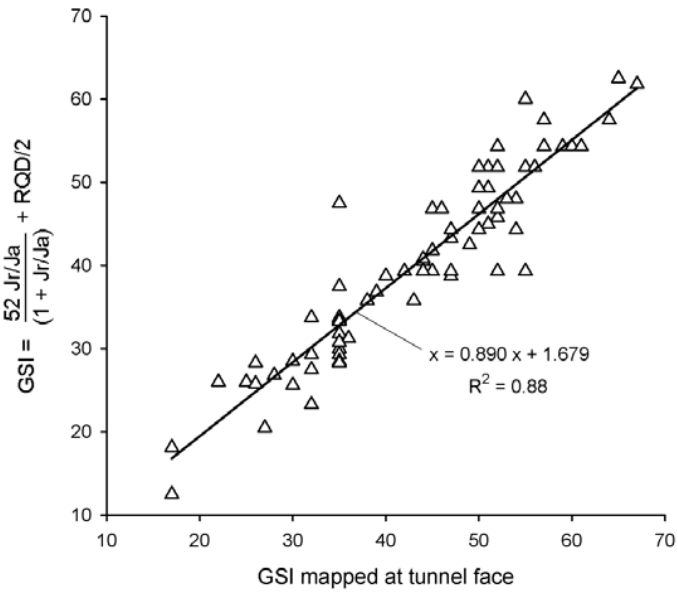


Figure 7: Comparison between mapped GSI and GSI predicted from Jr/Ja and RQD.

5. ALTERNATIVE JOINT CONDITION SCALE

In recognition of the fact that values of $JCond_{89}$ are not always available in data from field mapping, the authors have examined two options for alternative scales for the surface quality axis in Figure 5.

The first candidate is the version of Joint Condition rating ($JCond_{76}$) included in the paper by Bieniawski (1976) [7] (see Appendix 1). Regression analysis of a plot of individual values assigned to $JCond_{76}$ and $JCond_{89}$ gives $JCond_{89} = 1.3\ JCond_{76}$ which, when substituted into equation 1, gives

$$GSI = 2\ JCond_{76} + RQD/2 \qquad (2)$$

A second candidate is the quotient Jr/Ja, included in the Tunnelling Quality Index (Q) of Barton et al (1974) [8]. This quotient (Jr/Ja) represents the roughness and frictional characteristics of the joint walls or fillings.

Comparing the ratings for $JCond_{89}$ with those allocated to Jr and Ja by Barton et al (1974) [7] (see Appendix 1) gives the relationship $JCond_{89} = 35\ Jr/Ja/(1 + Jr/Ja)$. Substitution of this relationship into equation 1 yields:

$$GSI = \frac{52\ Jr/Ja}{(1 + Jr/Ja)} + RQD/2 \qquad (3)$$

For the same data set used in the preparation of Figure 6, the predicted values of GSI are plotted against field mapped values of GSI in Figure 7. While the results for a linear regression analysis are not as good as those obtained for equation 1, the fit is an acceptable approximation for engineering applications.

6. RQD DETERMINED FROM FACE MAPS

When no core is available and RQD has to be determined from the mapping of tunnel faces, tunnel walls or slope faces, three methods are available.

The first involves a simple physical measuring rod or tape held against or in front of the face. The length of intact rock segments greater than 10cm falling between natural fractures intersecting the rod or tape are summed in a fashion similar to core-based RQD. This procedure is described in Hutchinson and Diederichs (1996) [9]. A virtual version of this approach can be carried out on high quality face photos or Lidar scans.

Priest and Hudson (1976) [10] found that a reasonable estimate of RQD could be obtained from discontinuity spacing measurements made on core or from an exposure by use of the equation:

$$RQD = 100\ e^{0.1\lambda}\ (0.1\lambda + 1) \qquad (4)$$

where λ is the average number of discontinuities per meter.

Palmström (1982) [11], also studied RQD but in relation to the Volumetric Joint Count, Jv, a measure of the number of joints crossing a cubic meter of rock. Based on mapping of exposures or on orthogonal scanline mapping underground, the following expression was derived:

$$RQD = 115 - 3.3\ Jv \qquad (5)$$

More recently, Palmström (2005) [12] extended his analysis by including computer generated blocks of different sizes and shapes. A new correlation between RQD and Jv was found to give somewhat better results that the commonly used RQD = 115 – 3.3Jv. He suggested that this relationship (equation 5) given in his 1982 paper should be modified to:

$$RQD = 110 - 2.5\ Jv \qquad (6)$$

7. CONCLUSION AND RECOMMENDATIONS

With some minor modifications to the GSI chart published by Hoek and Marinos (2000) [1] it has been found that two simple linear scales, $JCond_{89}$ and RQD, can be used to represent the discontinuity surface conditions and the blockiness of the rock mass. These ratings are well established in engineering geology practice, are simple to measure or estimate in the field and are possibly the ratings that give the highest degree of consistency between different geologists working on a single project. Most importantly, in a direct check between GSI estimated from the sum of these ratings and GSI obtained by direct tunnel face mapping, the agreement is acceptable for the characterization of jointed rock masses in order to obtain properties for input for numerical models.

In recognition of the fact that values of $JCond_{89}$ are not always available in data from field mapping, two alternative scales for the surface quality axis have been investigated. One of these is a relationship between $JCond_{89}$ and the $JCond_{76}$ version of this parameter, used in older data sets, which can be used as a direct replacement of $JCond_{89}$. The second alternative is the quotient Jr/Ja that gives a relationship to $JCond_{89}$ which provides an acceptable approximation for engineering applications.

The goal of this paper was to construct a practical set of scales for the GSI chart, based on existing and well established scales used in either the RMR or Q classifications. Cai et al (2004) [13], Somnez and Ulusay (1999) [14] and Russo (2007, 2009) [15, 16] have published quantified GSI charts which incorporate joint surface and rock structure scales based on parameters related to those used by the authors in constructing Figure 5. All of these quantified GSI charts, including that proposed in Figure 5 of this paper, have advantages and disadvantages. However, they all suffer from two significant shortcomings.

Firstly, the parameters used to specify the joint surface conditions (the equivalent of Scale A in Figure 5) are all based on ratings of joint roughness, joint alteration and joint waviness. These ratings, with the exception of joint waviness, are based upon assessment of the degree of surface roughness and alteration rather than on any physical measurements of the shear strength of the surfaces themselves. It is this shear strength that is a controlling parameter in the behavior of the jointed rock mass and it is questionable whether the somewhat arbitrary nature of the roughness and alteration ratings can provide a reliable assessment of this shear strength.

Secondly, the use of RQD by the authors or some variation of the volumetric joint count Jv or the block volume Vb, by the other authors, limits the definition of rock structure to the dimension of the blocks. This takes no account of the ratio of block size to the size of the tunnel or slope which, as shown in Figure 4, has a significant influence on the application of the GSI chart for characterizing the rock mass.

Direct measurement of physical properties and numerical modeling of the progressive failure and deformation of the rock mass, while not devoid of challenges and abuses by over-enthusiastic users, offer the potential for resolving some of these deficiencies.

Measurement of the frictional strength of sawn or ground surfaces of small specimens is simple enough in a field laboratory with basic equipment. Similarly, measurement of small and large scale surface undulations, at a scale relevant to the problem under consideration, and combining these measurements with the basic friction angle of the rock surface is a well-established procedure described by Barton and Choubey (1977) [17].

Numerical techniques such as the Synthetic Rock Mass model (Mas Ivars et al. (2011) [18]) provide the means of incorporating the joint fabric of a rock mass at different scales. In the long run these methods have the potential to allow direct three-dimensional modeling of all of the physical components of a rock mass and provide a much more rigorous alternative to the empirical characterization and rockmass parameter estimation approach using the GSI chart. In the short term, numerical modeling techniques can be used to develop rock structure scales which incorporate both the scale of the rock blocks and the scale of the engineering structure in which they exist.

Rating-based rock mass characterization scales, such as those used in this paper, have played a critical role in the development of practical design tools for rock engineering. However, while practitioners may continue to apply these methods for some time, researchers should turn their attention to the actual physical properties of rock joints and numerical modeling of rock fracture networks to develop and apply a better understanding of jointed rock mass behavior.

8. REFERENCES

1. Hoek, E. and Marinos, P. 2000. Predicting Tunnel Squeezing. *Tunnels and Tunnelling International*. Part 1 – November 2000, Part 2 – December, 2000.
2. Marinos. P, and Hoek, E. 2002. Estimating the geotechnical properties of heterogeneous rock masses such as flysch. *Bulletin of the Engineering Geology & the Environment* (IAEG). 60: 85-92.
3. Marinos P., Marinos V., Hoek E. 2007. Geological Strength Index (GSI). A characterization tool for assessing engineering properties for rock masses. Published in: *Underground works under special conditions*, eds. Romana, Perucho & Olalla, 13-21. Lisbon: Taylor and Francis.
4. Carter, T.G., Diederichs, M.S., and J.L.Carvalho, J.L., 2008 Application of modified Hoek-Brown Transition Relationships for assessing Strength and Post-yield Behaviour at both ends of the rock competence scale. *Proc. SAIMM v108 No6 pp325-338*.
5. Bieniawski, Z.T. 1989. *Engineering rock mass classification*. New York: Wiley Interscience.
6. Deere, D.U. 1963. Technical description of rock cores for engineering purposes. *Felsmechanik und Ingenieurgeologie* (Rock Mechanics and Engineering Geology), 1 (1), 16-22.
7. Bieniawski, Z.T. 1976. Rock mass classification in rock engineering. In *Exploration for rock engineering, proc. of the symp*., ed. Z.T. Bieniawski, 1, 97-106. Cape Town: Balkema
8. Barton, N.R., Lien, R. and Lunde, J. 1974. Engineering classification of rock masses for the design of tunnel support. *Rock Mech*. 6(4), 189-239.
9. Hutchinson, D.J. and Diederichs, M. 1996. *Cable Bolting in Underground Mines*. 406p. Vancouver. Bitech.
10. Priest, S.D. and Hudson, J.A. 1976. Discontinuity spacings in rock. *Int. J. Rock Mech. Min. Sci. & Geomech. Abstr*. Vol. 13, pp. 135-148.
11. Palmström, A. 1982. The volumetric joint count—a useful and simple measure of the degree of jointing. In: *Proceedings of the Fourth International Congress IAEG*, New Delhi, vol. V, 221–228.
12. Palmström, A. 2005: Measurements of and correlations between block size and rock quality designation (RQD). *Tunnels and Underground Space Technology*, 20, 326-377.
13. Cai, M., Kaiser, P.K., Uno, H., Tasaka, Y., Minami, M. 2004. Estimation of rock mass strength and deformation modulus of jointed hard rock masses using the GSI system. *Rock Mech Min Sci* 41(1):3–19.
14. Sonmez, H. and Ulusay, R. 1999. Modifications to the geological strength index (GSI) and their applicability to the stability of slopes. *Int J Rock Mech Min Sci*. 36, 743–760
15. Russo, G. 2007. Improving the reliability of GSI estimation: the integrated GSI-RMi system. *ISRM Workshop Underground Works under Special Conditions*, Madrid.
16. Russo, G. 2009. A new rational method for calculating the GSI. *Tunnelling and Underground Space Technology*. 24, 103-111.
17. Barton, N. and Choubey, V. 1977. The shear strength of rock joints in theory and practice. *Rock Mechanics*. 12(1), 1-54.
18. Mas Ivars, D., Pierce, M.E., Darcel, D., Montes, J.R., Potyondy, D.O., Young, R.P. and Cundall, P.A. 2011. The synthetic rock mass approach for jointed rock mass modeling. *Int. J. Rock Mech. Min. Sci.* 48(2), 219-244.

9. ACKNOWLEDGMENTS

The authors acknowledge contributions to the preparation of this paper from Professor Ted Brown, Professor Paul Marinos, Professor Peter Kaiser, Dr. Vassilis Marinos, Felipé Duran Del Valle, Jennifer Day, Nicole Boultbee and David Wood, who participated in the construction of the original GSI chart.

10. APPENDIX 1 –PARAMETER DEFINITION

The Rock Quality Designation (RQD) was developed by Deere (1963) [6]. The index was developed to provide a quantitative estimate of rock mass quality from drill core logs. RQD is defined as the percentage of intact core pieces longer than 100 mm (4 inches) in the total length of core. The core should be at least NW size (54.7 mm or 2.15 inches in diameter) and should be drilled with at least a double-tube core barrel. The correct procedures for measurement of the length of core pieces and the calculation of RQD are summarized in Figure 8.

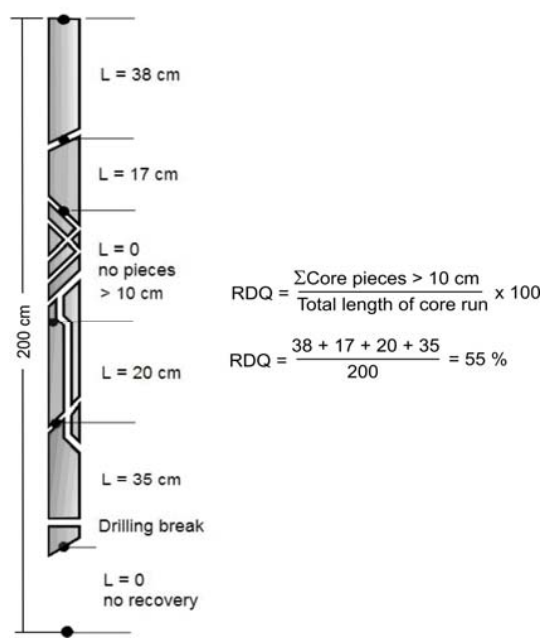


Figure 8: Definition of RQD, after Deere (1963) [6].

The definition of $JCond_{89}$ in Table 1 is reproduced directly from Bieniawski (1989) [5] while $JCond_{76}$, from Bieniawski (1976) [7], is defined in Table 2.

The parameters Jr and Ja, for rock wall contact, from Barton et al (1974) [8], are defined in Table 3

Table 1: Definition of $JCond_{89}$, after Bieniawski (1989) [5].

Condition of discontinuities	Very rough surfaces Not continuous No separation Unweathered wall rock	Slightly rough surfaces Separation < 1 mm Slightly weathered walls	Slightly rough surfaces Separation < 1 mm Highly weathered walls	Slickensided surfaces or Gouge < 5 mm thick or Separation 1 – 5 mm Continuous	Soft gouge > 5 mm thick or Separation > 5 mm Continuous
Rating	30	25	20	10	0

Guidelines for classification of discontinuity conditions

Discontinuity length (persistence) Rating	< 1 m 6	1 to 3 m 4	3 to10 m 2	10 to 20 m 1	More than 20 m 0
Separation (aperture) Rating	None 6	< 0.1 mm 5	0.1 – 1.0 mm 4	1 – 5 mm 1	More than 5 mm 0
Roughness Rating	Very rough 6	Rough 5	Slightly rough 3	Smooth 1	Slickensided 0
Infilling (gouge) Rating	None 6	Hard infilling < 5 mm 4	Hard filling > 5 mm 2	Soft infilling < 5 mm 2	Soft infilling > 5 mm 0
Weathering Rating	Unweathered 6	Slightly weathered 5	Moderate weathering 3	Highly weathered 1	Decomposed 0

Table 2: Definition of $JCond_{76}$, after Bieniawski (1976) [7]

Condition of discontinuities	Very rough surfaces Not continuous No separation Hard joint wall rock	Slightly rough surfaces Separation < 1 mm Hard joint wall rock	Slightly rough surfaces Separation < 1 mm Soft joint wall rock	Slickensided surfaces or Gouge < 5 mm thick or Joints open 1 – 5 mm Continuous joints	Soft gouge > 5 mm thick or Joints open > 5 mm Continuous joints
Rating	25	20	12	6	0

Table 3: Definition of Jr and Ja for rock wall contact (no pre-shearing), after Barton et al (1974) [8].

JOINT ROUGHNESS NUMBER Jr	Rating	JOINT ALTERATION NUMBER Ja	Rating
Discontinuous joints	4	Tightly healed, hard, non-softening, impermeable filling	0.75
Rough and irregular, undulating	3	Unaltered joint walls, surface staining only	1.0
Smooth, undulating	2	Slightly altered joint walls, non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc	2.0
Slickensided, undulating	1.5	Silty-, or sandy-clay coatings, small clay fraction (non-softening)	3.0
Rough or irregular planar	1.5	Softening or low friction clay, mineral coatings, i.e. kaolinite, mica. Also chlorite, talc, gypsum and graphite etc., and small quantities of swelling clays. (Discontinuous coatings, 1 – 2 mm or less in thickness)	4.0
Smooth, planar	1.0		
Slickensided, planar	0.5		

Estimating the geotechnical properties of heterogeneous rock masses such as flysch

Paul Marinos · Evert Hoek

Abstract The design of tunnels and slopes in heterogeneous rock masses such as flysch presents a major challenge to geologists and engineers. The complex structure of these materials, resulting from their depositional and tectonic history, means that they cannot easily be classified in terms of widely used rock mass classification systems. A methodology for estimating the Geological Strength Index and the rock mass properties for these geological formations is presented in this paper.

Résumé L'étude des tunnels et des talus dans des masses rocheuses hétérogènes, telles que le flysch, représente un défi majeur pour les géologues et les ingénieurs. La complexité de ces formations, résultat de leur histoire de sédimentation et de leur mise en place tectonique, pose des problèmes pour leur classification par des systèmes reconnus de classifications géotechniques. Dans ce travail une méthodologie pour l'estimation du GSI (Geological Strength Index) et l'évaluation des propriétés des masses rocheuses de flysch est présentée.

Keywords Rock mass classification · GSI · Geotechnical properties · Flysch

Mots clés Classifications géotechniques · Masses rocheuses · GSI · Propriétés géotechniques · Flysch

Received: 30 July 2000 / Accepted: 13 November 2000
Published online: 24 April 2001

P. Marinos (✉)
Civil Engineering Department,
National Technical University of Athens,
Athens, Greece
E-mail: marinos@central.ntua.gr

E. Hoek
Consulting Engineer,
Vancouver, Canada

Introduction

Many large civil engineering projects are currently under construction in countries where flysch is a very common geological formation. The design of surface and underground excavations in these materials requires knowledge of the mechanical properties of the rock masses in which these excavations are carried out. The paper presents a methodology for estimating these properties.

Estimation of rock mass properties

One of the most widely used criteria for estimating rock mass properties is that proposed by Hoek and Brown (1997) and this criterion, with specific adaptations to heterogeneous rock masses such as flysch, is briefly summarised. This failure criterion should not be used when the rock mass consists of a strong blocky rock such as sandstone, separated by clay-coated and slicken-sided bedding surfaces. The behaviour of such rock masses will be strongly anisotropic and will be controlled by the fact that the bedding planes are an order of magnitude weaker than any other features. In such rock masses the predominant failure mode will be gravitational falls of wedges or blocks of rock defined by the intersection of the weak bedding planes with other features which act as release surfaces. However, if the rock mass is heavily fractured, the continuity of the bedding surfaces will have been disrupted and the rock may behave as an isotropic mass. In applying the Hoek and Brown criterion to "isotropic" rock masses, three parameters are required for estimating the strength and deformation properties. These are:

1. The uniaxial compressive strength σ_{ci} of the "intact" rock elements that make up the rock mass (as described below, this value may not be the same as that obtained from a laboratory uniaxial compressive strength or UCS test).
2. A constant m_i that defines the frictional characteristics of the component minerals in these rock elements.
3. The Geological Strength Index (GSI) which relates the properties of the intact rock elements to those of the overall rock mass.

These parameters are discussed below.

Uniaxial compressive strength σ_{ci} of intact rock

In dealing with heterogeneous rock masses such as flysch, it is extremely difficult to obtain a sample of "intact" core for uniaxial compressive testing in the laboratory. The typical appearance of such material in an outcrop is illustrated in Fig. 1. Practically every sample obtained from a rock mass such as that illustrated in Fig. 1 will contain discontinuities in the form of bedding and schistosity planes or joints. Consequently, any laboratory tests carried out on core samples will result in a strength value that is lower than the uniaxial compressive strength σ_{ci} required for input into the Hoek–Brown criterion. Using the results of such tests in the Hoek–Brown criterion will impose a double penalty on the strength (in addition to that imposed by GSI) and will give unrealistically low values for the rock mass strength.

In some special cases, where the rock mass is very closely jointed and where it has been possible to obtain undisturbed core samples, uniaxial compressive strength tests have been carried out directly on the "rock mass" (Jaeger 1971). These tests require an extremely high level of skill on the part of the driller and the laboratory technician. The large-scale triaxial test facilities required for such testing are only available in a few laboratories in the world and it is generally not economical or practical to consider such tests for routine engineering projects.

One of the few courses of action that can be taken to resolve this dilemma is to use the point load test on samples in which the load can be applied normal to the bedding or schistosity of block samples. The specimens used for such testing can be either irregular pieces or pieces broken from the core, as illustrated in Fig. 2. The direction of loading should be as perpendicular to any weakness planes as possible and the fracture created by the test should not show any signs of having followed an existing discontinuity. It is strongly recommended that photographs of the specimens, both before and after testing, should accompany the laboratory report as these enable the user to judge the validity of the test results. The uniaxial compressive strength of the intact rock samples can be estimated with a reasonable level of accuracy by multiplying the point load index I_s by 24[1], where $I_s = P/D^2$. P is the load on the points and D is the distance between the points.

In the case of very weak and/or fissile rocks such as clayey shales or sheared siltstones, the indentation of the loading points may cause plastic deformation rather than fracture of the specimen. In such cases the point load test does not give reliable results. Where it is not possible to obtain samples for point load testing, the only remaining alternative is to turn to a qualitative description of the rock material in order to estimate the uniaxial compressive strength of the intact rock. A table listing such descriptions is given in Table 1, based on Hoek and Brown (1997).

Constant m_i

The Hoek–Brown constant m_i can only be determined by triaxial testing on core samples or estimated from a qualitative description of the rock material as described by Hoek and Brown (1997). This parameter depends upon the frictional characteristics of the component minerals in the intact rock sample and has a significant influence on the strength characteristics of rock.

When it is not possible to carry out triaxial tests (for the reasons discussed above), an estimate of m_i can be obtained from Table 2. Most of the values quoted have been derived from triaxial tests on intact core samples and the range of values shown is dependent upon the accuracy of the geological description of each rock type. For example, the term "granite" describes a clearly defined rock type and all granites exhibit very similar mechanical characteristics. Hence the value of m_i is defined as 32±3. On the other hand, the term "volcanic breccia" is not very precise in terms of mineral composition and hence the value of m_i is shown as 19±5, denoting a higher level of uncertainty.

Fortunately, in terms of the estimation of rock mass strength, the value of the constant m_i is the least sensitive of the three parameters required. Consequently, the average values given in Table 2 are sufficiently accurate for most practical applications.

Fig. 1
Appearance of sheared siltstone flysch in an outcrop

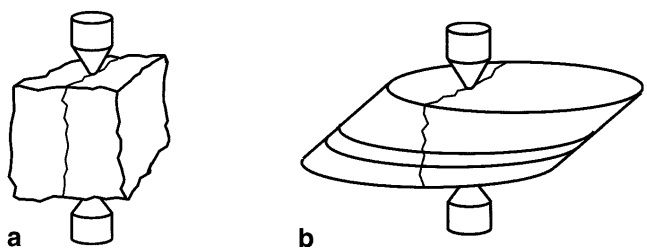


Fig. 2
Point load test options for intact rock samples from heterogeneous rock masses. **a** Test on sample chosen from surface exposure. **b** Test on sample broken from diamond drill core

[1]The constant of 24 is for a 54 mm core sample (Bieniawski 1974)

Table 1
Field estimates of uniaxial compressive strength of intact rock. (After Hoek and Brown 1997 with some changes in the examples)

Grade[a]	Term	Uniaxial comp. strength (MPa)	Point load index (MPa)	Field estimate of strength	Examples
R6	Extremely strong	>250	>10	Specimen can only be chipped with a geological hammer	Fresh basalt, chert, diabase, gneiss, granite, quartzite
R5	Very strong	100–250	4–10	Specimen requires many blows of a geological hammer to fracture it	Amphibolite, sandstone, basalt, gabbro, gneiss, granodiorite, peridotite, rhyolite, tuff
R4	Strong	50–100	2–4	Specimen requires more than one blow of a geological hammer to fracture it	Limestone, marble, sandstone, schist
R3	Medium strong	25–50	1–2	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with a single blow from a geological hammer	Concrete, phyllite, schist, siltstone
R2	Weak	5–25	[b]	Can be peeled with a pocket knife with difficulty, shallow indentation made by firm blow with point of a geological hammer	Chalk, claystone, potash, marl, siltstone, shale, rock salt
R1	Very weak	1–5	[b]	Crumbles under firm blows with point of a geological hammer, can be peeled by a pocket knife	Highly weathered or altered rock, shale
R0	Extremely weak	0.25–1	[b]	Indented by thumbnail	Stiff fault gouge

[a]According to Brown (1981)
[b]Point load tests on rocks with a uniaxial compressive strength below 25 MPa are likely to yield highly ambiguous results

Table 2
Values of constant m_i for intact rock, by rock group. Values in *parentheses* are estimates. Range of values quoted for each material depends upon granularity and interlocking of crystal structure, higher values being associated with tightly interlocked and more frictional characteristics. (This table contains several changes from previously published versions. These changes have been made to reflect data that have been accumulated from laboratory tests and experience gained from discussions with geologists and engineering geologists.)

Sedimentary	Clastic		Conglomerates[a] Breccias[a]	Sandstones 17±4	Siltstones 7±2 Greywackes (18±3)	Claystones 4±2 Shales (6±2) Marls (7±2)
	Non-clastic	Carbonates	Crystalline limestone (12±3)	Sparitic limestones (10±2)	Micritic limestones (9±2)	Dolomites (9±3)
		Evaporites		Gypsum 8±2	Anhydrite 12±2	
		Organic				Chalk 7±2
Metamorphic	Non-foliated		Marble 9±3	Hornfels (19±4) Metasandstone (19±3)	Quartzites 20±3	
	Slightly foliated		Migmatite (29±3)	Amphibolites 26±6	Gneiss 28±5	
	Foliated[b]			Schists 12±3	Phyllites (7±3)	Slates 7±4
Igneous	Plutonic	Light	Granite 32±3 Granodiorite (29±3)	Diorite 25±5		
		Dark	Gabbro 27±3 Norite 20±5	Dolerite (16±5)		
	Hypabyssal		Porphyries (20±5)		Diabase (15±5)	Peridotite (25±5)
	Volcanic	Lava		Rhyolite (25±5) Andesite 25±5	Dacite (25±3) Basalt (25±5)	
		Pyroclastic	Agglomerate (19±3)	Breccia (19±5)	Tuff (13±5)	

[a]Conglomerates and breccias may have a wide range of values, depending on the nature of cementing material and degree of cementation. Values may range between those of sandstones to those of fine-grained sediments
[b]These values are for intact rock specimens tested normal to bedding or foliation. Values of m_i will be significantly different if failure occurs along a weakness plane

Geological Strength Index (GSI)

The Geological Strength Index (GSI) was introduced by Hoek (1994), Hoek, Kaiser and Bawden (1995), Hoek and Brown (1997) and extended by Hoek, Marinos and Benissi (1998). A chart for estimating the GSI for flysch is presented in Table 3.

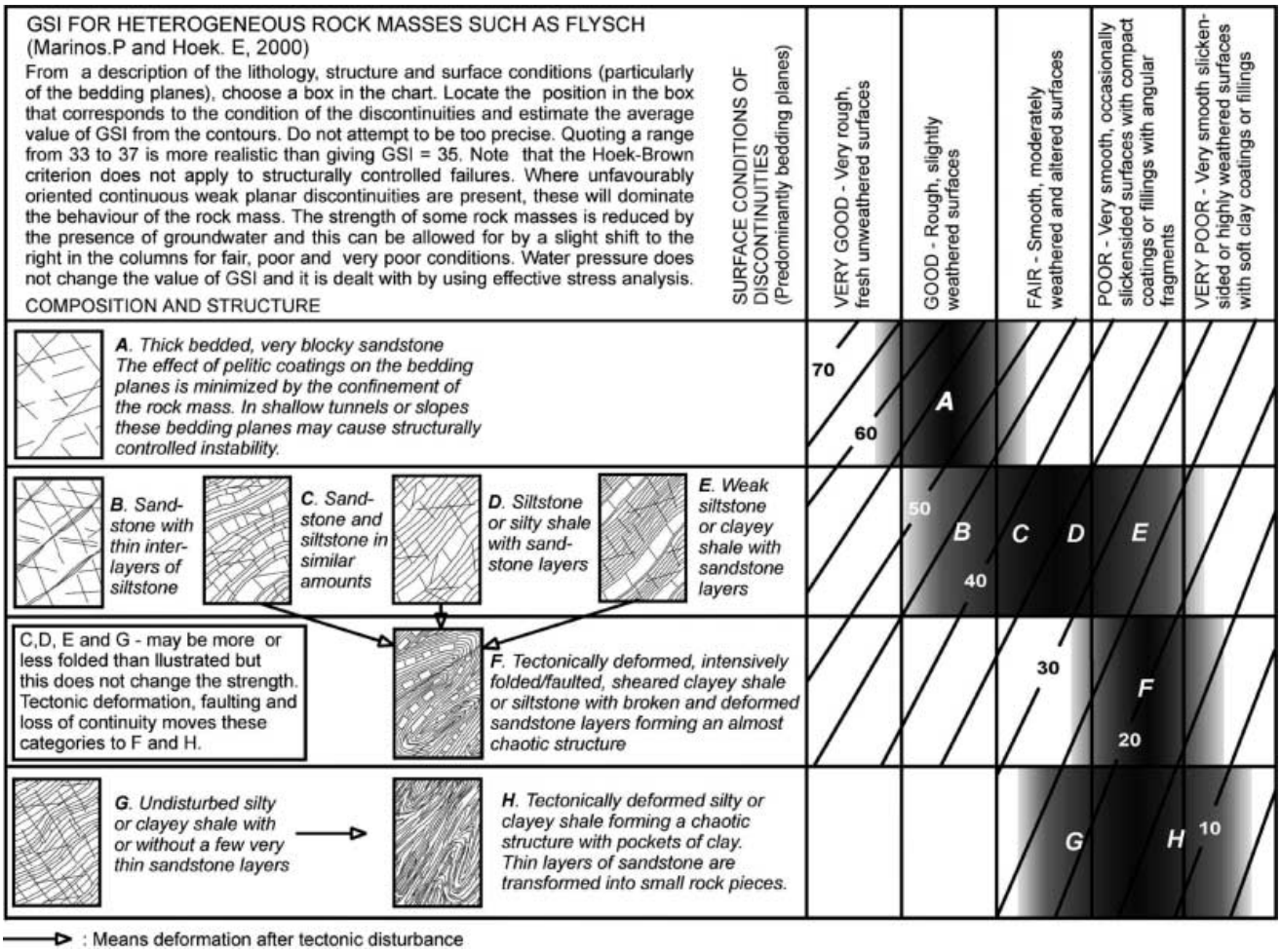
Mechanical properties of flysch

The term flysch is attributed to the geologist B. Studer and it comes from the German word "*fliessen*" meaning flow, probably denoting the frequent landslides in areas consisting of these formations. Flysch consists of varying alternations of clastic sediments that are associated with orogenesis. It closes the cycle of sedimentation of a basin before the "arrival" of the paroxysm folding process. The clastic material derives from erosion of the previously formed neighbouring mountain ridge.

Flysch is characterised by rhythmic alternations of sandstone and fine-grained (pelitic) layers. The sandstone may also include conglomerate beds. The fine-grained layers contain siltstones, silty shales and clayey shales. Rarely, limestone beds or ophiolitic masses may be found close to its margins. The thickness of the sandstone beds range

Fig. 3
"Portable" point load test device for use in the field

Table 3
GSI estimates for heterogeneous rock masses such as flysch



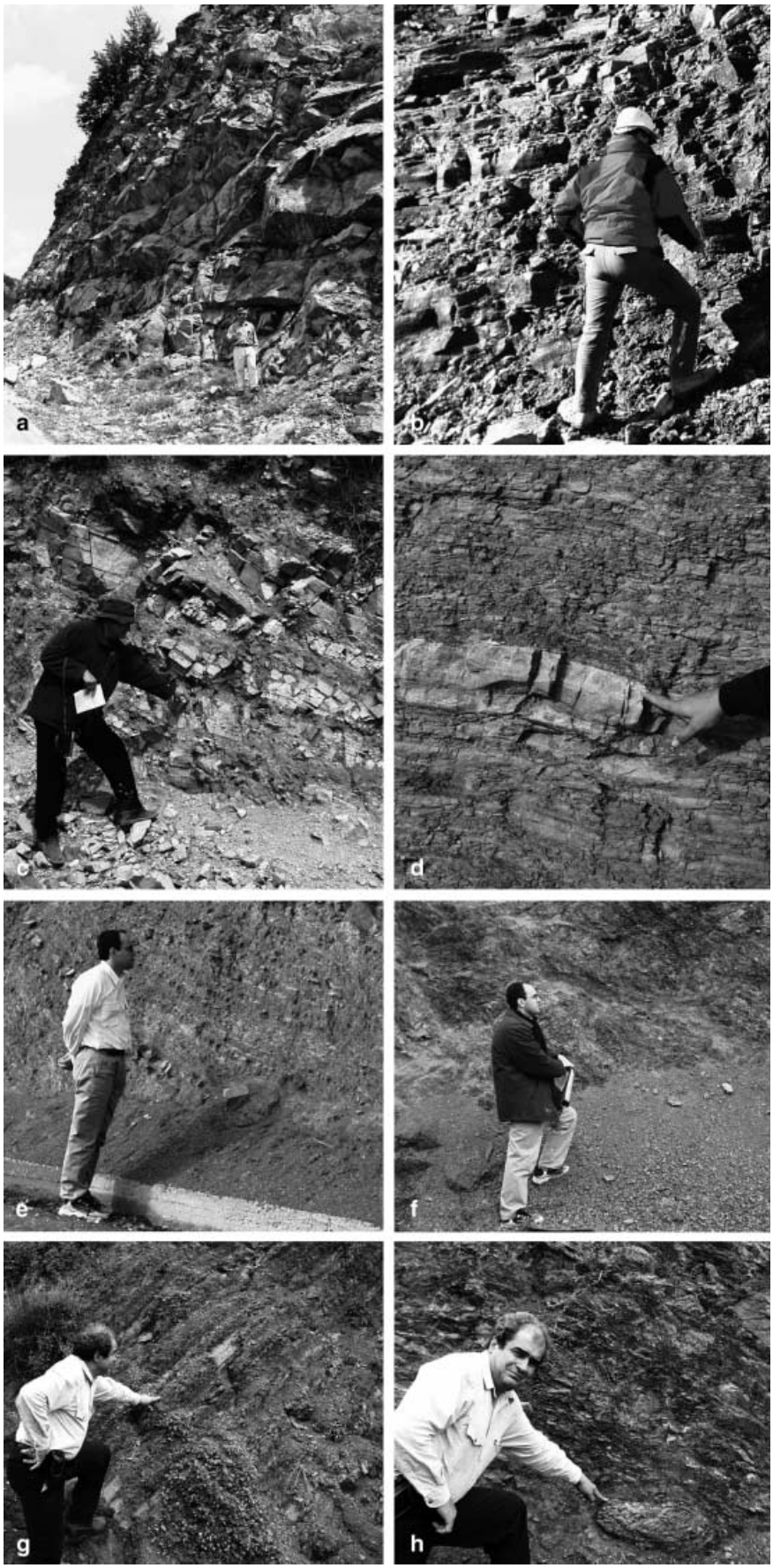


Fig. 4
Examples of flysch corresponding to descriptions in Table 3. **A** Thick-bedded blocky sandstone. Note that structural failure can occur when the dip of bedding planes is unfavourable. **B** Sandstone with thin siltstone layers. Small-scale structural failures can occur when bedding dip is unfavourable. **C** Sandstone and siltstone in equal proportions. **D** Siltstone or silty shale with sandstone. **E** Weak siltstone or clayey shale with sandstone layers. **F** Tectonically deformed clayey shale or siltstone with broken sandstone. **G** Undisturbed silty or clayey shale with a few thin sandstone layers. **H** Tectonically deformed clayey shale

from centimetres to metres. The siltstones and schists form layers of the same order but bedding discontinuities may be more frequent, depending upon the fissility of the sediments.

The overall thickness of the flysch is often very large (hundreds to a few thousand metres), albeit it may have been reduced considerably by erosion or by thrusting. The formation may contain different types of alternations and is often affected by reverse faults and thrusts. This, together with consequent normal faulting, results in a degradation of the geotechnical quality of the flysch rock mass. Thus, sheared or even chaotic rock masses can be found at the scale of a typical engineering design.

Geotechnically, a flysch rock mass has the following characteristics:

1. Heterogeneity: alternation of competent and incompetent members.
2. Presence of clay minerals.
3. Tectonic fatigue and sheared discontinuities, often resulting in a soil-like material.
4. The permeability of flysch rock masses is generally low and, because of the presence of clay minerals, the rock mass may be weakened to a significant degree where free drainage is not present.

Molasse is a term used to define a rock mass of similar composition but of post-orogenic origin associated with newly formed mountain ridges. It has the same alternations of strong (sandstones and conglomerates) and weak (marls, siltstones and claystones) materials, but there is not the same compressional disturbance.

Determination of the GSI for these rock masses composed of frequently tectonically disturbed alternations of strong and weak rocks presents some special challenges. However, because of the large number of engineering projects under construction in these rock masses, some attempt has to be made to provide better engineering geological tools than those currently available. Hence, in order to accommodate this group of materials in the GSI system, a chart for estimating this parameter has been developed and is presented in Table 3.

Influence of groundwater

The most basic impact of groundwater is upon the mechanical properties of the intact rock components of the rock mass. This is particularly important when dealing with shales, siltstones and similar rocks that are susceptible to changes in moisture content. Many of these maerials will disintegrate very quickly if they are allowed to dry out after removal from the core barrel. For this reason, testing of the "intact" rock to determine the uniaxial compressive strength σ_{ci} (see above) and the constant m_i must be carried out under conditions that are as close to the in-situ moisture conditions as possible. Ideally, a field laboratory should be set up very close to the drill rig and the core prepared and tested immediately after recovery.

In one example in which a siltstone was being investigated for the construction of a power tunnel for a hydro-electric project, cores were carefully sealed in aluminium foil and wax and then transported to a laboratory in which very high-quality testing could be carried out. In spite of these precautions, the deterioration of the specimens was such that the test results were meaningless. Consequently, a second investigation programme was carried out in which the specimens were transported to a small laboratory about 5 km from the exploration site and the samples were tested within an hour of having been removed from the core barrel. The results of this second series of tests were very consistent and the values of uniaxial compressive strength σ_{ci} and constant m_i were considered reliable.

When laboratory testing is not possible, point load tests, using equipment similar to that illustrated in Fig. 3, should be carried out as soon after core recovery as possible in order to ensure that the moisture content of the sample is close to the in-situ conditions.

Examples of typical flysch

In order to assist the reader in using Table 3, examples of typical flysch outcrops are given in the photographs reproduced in Fig. 4.

Selection of σ_{ci} and m_i for flysch

In addition to the GSI values presented in Table 3, it is necessary to consider the selection of the "intact" rock properties σ_{ci} and m_i for heterogeneous rock masses such as flysch. As the sandstone layers are usually separated from each other by weaker layers of siltstone or shales, rock-to-rock contact between blocks of sandstone may be limited. Consequently, it is not appropriate to use the properties of the sandstone to determine the overall strength of the rock mass. On the other hand, using the "intact" properties of the siltstone or shale only is too

Table 4
Suggested proportions of parameters σ_{ci} and m_i for estimating rock mass properties for flysch

Flysch type (see Table 3)	Proportions of values for each rock type to be included in rock mass property determination
A and B	Use values for sandstone beds
C	Reduce sandstone values by 20% and use full values for siltstone
D	Reduce sandstone values by 40% and use full values for siltstone
E	Reduce sandstone values by 40% and use full values for siltstone
F	Reduce sandstone values by 60% and use full values for siltstone
G	Use values for siltstone or shale
H	Use values for siltstone or shale

Table 5
Spreadsheet for calculation of rock mass properties. (After Hoek and Brown 1997)

Input:	sigci=10 MPa			mi=10			GSI=30		
	Depth of failure surface or tunnel below slope[a]=25 m						Unit wt.=0.027 MN/m^3		
Output:	stress=0.68 MPa			mb=0.82			s=0.0004		
	a=0.5			sigtm=−0.0051 MPa			A=0.4516		
	B=0.7104			k=3.95			phi=36.58 degrees		
	coh=0.136 MPa			sigcm=0.54 MPa			E=1,000 MPa		
Calculation:									Sums
sig3	1E-10	0.10	0.19	0.29	0.39	0.48	0.58	0.68	2.70
sig1	0.20	1.01	1.47	1.84	2.18	2.48	2.77	3.04	14.99
ds1ds3	21.05	5.50	4.22	3.64	3.29	3.05	2.88	2.74	46.36
sign	0.01	0.24	0.44	0.62	0.80	0.98	1.14	1.31	5.54
tau	0.04	0.33	0.50	0.64	0.76	0.86	0.96	1.05	5.14
x	−2.84	−1.62	−1.35	−1.20	−1.09	−1.01	−0.94	−0.88	−10.94
y	−2.37	−1.48	−1.30	−1.19	−1.12	−1.06	−1.02	−0.98	−10.53
xy	6.74	2.40	1.76	1.43	1.22	1.07	0.96	0.86	16.45
xsq	8.08	2.61	1.83	1.44	1.19	1.02	0.88	0.78	17.84
sig3sig1	0.00	0.10	0.28	0.53	0.84	1.20	1.60	2.05	7
sig3sq	0.00	0.01	0.04	0.08	0.15	0.23	0.33	0.46	1
taucalc	0.04	0.32	0.49	0.63	0.76	0.87	0.97	1.07	
sig1sig3fit	0.54	0.92	1.30	1.68	2.06	2.45	2.83	3.21	
signtaufit	0.14	0.31	0.46	0.60	0.73	0.86	0.98	1.11	

Cell formulae:
(σ_n) stress=if(depth>30, sigci×0.25,depth×unit wt×0.25)
(m_b) mb=mi×EXP((GSI−100)/28)
s=IF(GSI>25,EXP((GSI−100)/9),0)
a=IF(GSI>25,0.5,0.65−GSI/200)
(σ_{tm}) sigtm=0.5×sigci×(mb−SQRT(mb^2+4×s))
(σ_3) sig3=start at 1E-10 (to avoid zero errors) and increment in seven steps of stress/28 to stress/4
(σ_1) sig1=sig3+sigci×(((mb×sig3)/sigci)+s)^a
($\delta\sigma_1/\delta\sigma_3$) ds1ds3=IF(GSI>25,(1+(mb×sigci)/(2×(sig1−sig3))),1+(a×mb^a)×(sig3/sigci)^(a−1))
(σ_n) sign=sig3+(sig1−sig3)/(1+ds1ds3)
(τ)tau=(sign−sig3)×SQRT(ds1ds3)
x=LOG((sign−sigtm)/sigci)
y=LOG(tau/sigci)
xy=x×y
xsq=x^2
A=acalc=10^(sumy/8−bcalc×sumx/8)
B=bcalc=(sumxy−(sumx×sumy)/8)/(sumxsq−(sumx^2)/8)
k=(sumsig3sig1−(sumsig3×sumsig1)/8)/(sumsig3sq−(sumsig3^2)/8)
(ϕ) phi=ASIN((k−1)/(k+1))×180/PI()
(c) coh=sigcm/(2×SQRT(k))
σ_{cm} sigcm=sumsig1/8−k×sumsig3/8
E=IF(sigci>100,1000×10^((GSI−10)/40),SQRT(sigci/100)×1000×10^((GSI−10)/40))
phit=(ATAN(acalc×bcalc×((signt−sigtm)/sigci)^(bcalc−1)))×180/PI()
coht=acalc×sigci×((signt−sigtm)/sigci)^bcalc−signt×TAN(phit×PI()/180)
sig3sig1=sig3×sig1
sig3sq=sig3^2
taucalc=acalc×sigci×((sign−sigtm)/sigci)^bcalc
s3sifit=sigcm+k×sig3
sntaufit=coh+sign×TAN(phi×PI()/180)
tangent = coht+signt×TAN(phit×PI()/180)
[a]For depths below surface of less than 30 m, average stress on the failure surface is calculated by the spreadsheet. For depths greater than 30 m average stress level is kept constant at the value for 30 m depth
The example included in Table 5 is for evaluation of the mechanical properties of a rock mass of flysch consisting of weak siltstone and sandstone layers. Tables 1, 2, 3 and 4 are used to obtain the following input parameters: (1) Equivalent intact rock strength σ_{ci}=10 MPa (weighted from values of σ_{ci} of sandstone and siltstone from Table 1 and Table 4); (2) equivalent constant m_i = 10 (weighted from values of m_i of sandstone and siltstone from Table 2 and Table 4); (3) geological Strength Index GSI = 30 (flysch type E in Table 3)

conservative as the sandstone skeleton certainly contributes to the rock mass strength. It is proposed that a "weighted average" of the intact strength properties of the strong and weak layers should be used. Suggested values for the components of this weighted average are given in Table 4.

Estimating rock mass properties

Having defined the parameters σ_{ci}, m_i and GSI as described above, the next step is to estimate the mechanical properties of the rock mass. As the procedure for making these estimates has been described in detail by Hoek

and Brown (1997), it will not be repeated here. A spreadsheet for carrying out these calculations is given in Table 5.

Deep tunnels

For tunnels at depths of greater than 30 m, the rock mass surrounding the tunnel is confined and its properties are calculated on the basis of a minor principal stress or confining pressure σ_3 up to 0.25 σ_{ci}, in accordance with the procedure defined by Hoek and Brown (1997). In the case of "deep" tunnels, equivalent Mohr Coulomb cohesive strengths and friction angles together with the uniaxial compressive strength σ_{cm} and the deformation modulus E of the rock mass can be estimated by means of the spreadsheet given in Table 5 by entering any depth greater than 30 m.

Shallow tunnels and slopes

For shallow tunnels and slopes in which the degree of confinement is reduced, a minor principal stress range of $0<\sigma_3<\sigma v$ is used, where σv=depth × unit weight of the rock mass. In this case, depth is defined as the depth below the surface of the tunnel crown or the average depth of a failure surface in a slope in which a circular failure type can be assumed, i.e. where the failure is not structurally controlled.

In the case of shallow tunnels or slopes, the spreadsheet presented in Table 5 allows the user to enter the depth below surface and the unit weight of the rock mass. The vertical stress σv calculated from the product of these two quantities is then used to calculate the rock mass properties. An example is given for a tunnel or a failure surface at a depth of 25 m below the surface. The estimated properties of this heterogeneous rock mass, from the Hoek–Brown criterion, are:

1. Cohesive strength c=0.136 MPa
2. Friction angle ϕ=36.6°
3. Rock mass compressive strength σ_{cm}=0.54 MPa
4. Deformation modulus E=1,000 MPa

Acknowledgements The development of the methodology presented in this paper was carried out by the authors as part of their consulting services to Egnatia Odos S.A., an organisation responsible for the design and construction of a 680 km highway across northern Greece. Particular acknowledgements are due to Mr. Nikos Kazilis, head of the tunnelling department of Egnatia Odos S.A., Mr. George Agistalis, Mr. Nikos Syrtariotis and Miss Maria Benissi for their assistance in the preparation of this paper.

References

Bieniawski ZT (1974) Estimating the strength of rock materials. J SAfr Inst Min Metall 74:312–320

Brown ET (ed) (1981) Rock characterization, testing and monitoring – ISRM suggested methods. Pergamon, Oxford, pp 171–183

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

Hoek E, Brown ET (1997) Practical estimates of rock mass strength. Int J Rock Mech Min Sci Geomech Abstr 34(8):1165–1186

Hoek E, Kaiser PK, Bawden WF (1995) Support of underground excavations in hard rock. AA Balkema, Rotterdam

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

Jaeger JC (1971) Friction of rocks and stability of rock slopes. The 11th Rankine Lecture. Géotechnique 21(2):97–134

Proceedings of the 15th European Conference on Soil Mechanics and Geotechnical Engineering
A. Anagnostopoulos et al. (Eds.), Part 1, pp. 435-440, Athens 2011.

JY-4

Estimation of geotechnical properties and classification of geotechnical behaviour in tunnelling for flysch rock masses

Estimation des propriétés géotechniques et classification du comportement des massifs rocheux du flysch aux tunnels

V. Marinos[1]
Aristotle University of Thessaloniki

P. Fortsakis & G. Prountzopoulos
National Technical University of Athens

ABSTRACT

Flysch formations are generally characterised by diverse heterogeneity, presence of members with low strength and tectonically disturbed structures. The complexity of these geological materials demands a more specialized research and geological characterization, due to the special features of their rock mass types regarding both their structure and their lithological characteristics. The present paper proposes a standardization of the qualitative engineering geological characteristics, the geotechnical classification, the assessment of the behaviour in underground excavations and the qualitative instructions-guidelines for the primary support measures for flysch rock masses. In order to investigate flysch rock mass properties, 12 tunnels of Egnatia Highway, in Northern Greece, were examined taking into account data from their design and construction records. More specifically, flysch formations are classified here in 11 rock mass types (I to XI), according to the siltstone-sandstone participation and their tectonic disturbance. A modified GSI chart for heterogeneous rock masses such as flysch is presented, where a certain range of GSI values is proposed for every rock mass type. The engineering geological behaviour of flysch in tunnel excavation lies within a large spectrum. A further classification of every flysch rock mass type based on their geotechnical behaviour during tunnel excavation, in relation to categorised failure mechanisms (deformation due to overstressing, overbreaks or wedge failure, “chimney” type failure, ravelling ground) and in situ stresses, is presented. Finally, general principles and guidelines for the selection of the immediate support measures are proposed.

RÉSUMÉ

Les formations de flysch sont généralement caractérisées par une hétérogénéité diverse et la présence de membres de faible résistance avec une structure tectonique perturbée. Ce développement de caractéristiques spéciales de massif de flysch au niveau de sa structure et sa lithologie exige une caractérisation géologique plus délicate. Dans un premier temps ce document propose une normalisation des caractéristiques géologiques qualitatives et une classification géotechnique. Or, dans un deuxième temps, il présente une méthode d’évaluation de leur comportement pendant les excavations souterraines et une approche qualitative de direction concernant les mesures de support principal pour les masses de flysch rocheux. Afin d'étudier les propriétés de flysch, 12 tunnels de l'autoroute d’Egnatia, dans le nord de la Grèce, ont été examinés prenant en compte les données de leur conception et leur construction. Plus précisément, les formations de flysch sont ici classées dans 11 différents types des roches (I à XI) en fonction de la participation de siltstone-grès et de leurs perturbations tectoniques. Un nouveau diagramme pour le GSI des roches hétérogènes telles que le flysch est présentée, où un certain éventail de valeurs est proposé pour chaque type. Le comportement de flysch en cours d’excavation du tunnel présente une large gamme, principalement en raison de l'hétérogénéité de la masse rocheuse et ses perturbations tectoniques. En outre, pour chaque type du massif de flysch rocheux, un classement additionnel est proposé par rapport à son comportement géotechnique, au cours d’excavation de tunnel, lié aux mécanismes de rupture (déformation due à la surcharge, hors profils, chute de dièdres ou de type cheminée, on éboulement) et aux contraintes développées in situ dans chaque cas. L’article conche avec des principes généraux et des recommandations pour le soutènement immédiat.

Keywords: Flysch, tunnel, GSI, tunnel behaviour, temporary support

[1] Corresponding Author.

1 INTRODUCTION

In the last decades there has been a rapid development in almost all stages of geotechnical design. Analysis and computational methods are fields where great progress has been made. However, regardless the great capabilities offered by the state of the art computational tools, the results still involve uncertainties due to the difficulties in defining design parameters. Hence, attention should be given to the definition of the geotechnical parameters and behaviour of the rock mass in engineering works.

Flysch formations are generally characterized by diverse heterogeneity, presence of members with low strength and tectonically disturbed structures. Such formations are classified here into certain rock mass types according to the siltstone-sandstone participation and their tectonic disturbance and a new GSI chart for heterogeneous rock masses such as flysch is presented.

A further classification of flysch rock masses based on their geotechnical behaviour (deformation due to overstressing, overbreakes or wedge failure, “chimney” type failure, raveling ground and their equivalent scale) is presented hereafter. Finally, temporary support measures concept and principles for every rock mass type are presented based on the available tunnelling experience.

A fundamental tool of this research was the TIAS database (Tunnel Information and Analysis System – [1], [2]), with a great number of geological and geotechnical data from the design and the construction of 12 tunnels of Egnatia Highway in Northern Greece, driven in flysch environment.

2 GEOTECHNICAL CLASSIFICATION

Flysch is composed of varying alternations of clastic sediments that are associated with orogenesis, since it ends the cycle of sedimentation before the paroxysm folding process. It is characterized mainly by rhythmic alternations of sandstone and pelitic layers (siltstones, silty or clayey shales). The thickness of sandstone or siltstone beds ranges from centimeters to meters.

The variety of geological conditions under different in situ stresses, in mild and heavy tectonism, provided significant amount of information regarding the engineering geological conditions and geotechnical behaviour of several flysch rock mass types. These behaviours were analysed and evaluated so as to define the geotechnical characteristics for every flysch type.

In order to investigate the rock mass properties of flysch, 12 tunnels driven in various geological environments were examined. The 2001 chart [3] is being revised here with adjustments in values and additions of new rock mass types. Flysch formations are classified into 11 rock mass types (I to XI) according to the siltstone-sandstone participation and their tectonic disturbance. Hence, a new GSI diagram for heterogeneous rock masses such as flysch is presented, where a certain range of GSI values for every rock mass type is proposed (Figure 1). In the new diagram, GSI values are increased from 10 to 35 units for the “Blocky” to “Undisturbed” structures, respectively, particularly for the siltstone type. The high presence of siltstone beds does not decrease the GSI value, but only in the highly disturbed forms. When a rock mass is undisturbed or slightly disturbed, independently of siltstone or sandstone predominance, high GSI ratings have to be considered. This was confirmed from tunnel construction, where lighter temporary support categories (correlated with high GSI values) were implemented and marginal measured deformations were observed.

Another addition to the GSI chart is the bedding thickness consideration of the competent sandstone beds. In types IV and V (slightly disturbed structures) when the thickness of sandstone beds exceeds 50cm, an increase of the GSI value by 5 is suggested.

In addition, it is necessary to consider values of the “intact” rock properties σ_{ci}, m_i and E_i for the heterogeneous rock mass as a unit. Some quantitative estimates of heterogeneous intact rock properties via laboratory tests [4], [5] have already been presented. For cases when laboratory tests are not feasible, a “weighted average” of the intact strength properties of the strong and weak layers is proposed in Table 1.

Proceedings of the 15th European Conference on Soil Mechanics and Geotechnical Engineering
A. Anagnostopoulos et al. (Eds.), Part 1, pp. 435-440, Athens 2011.

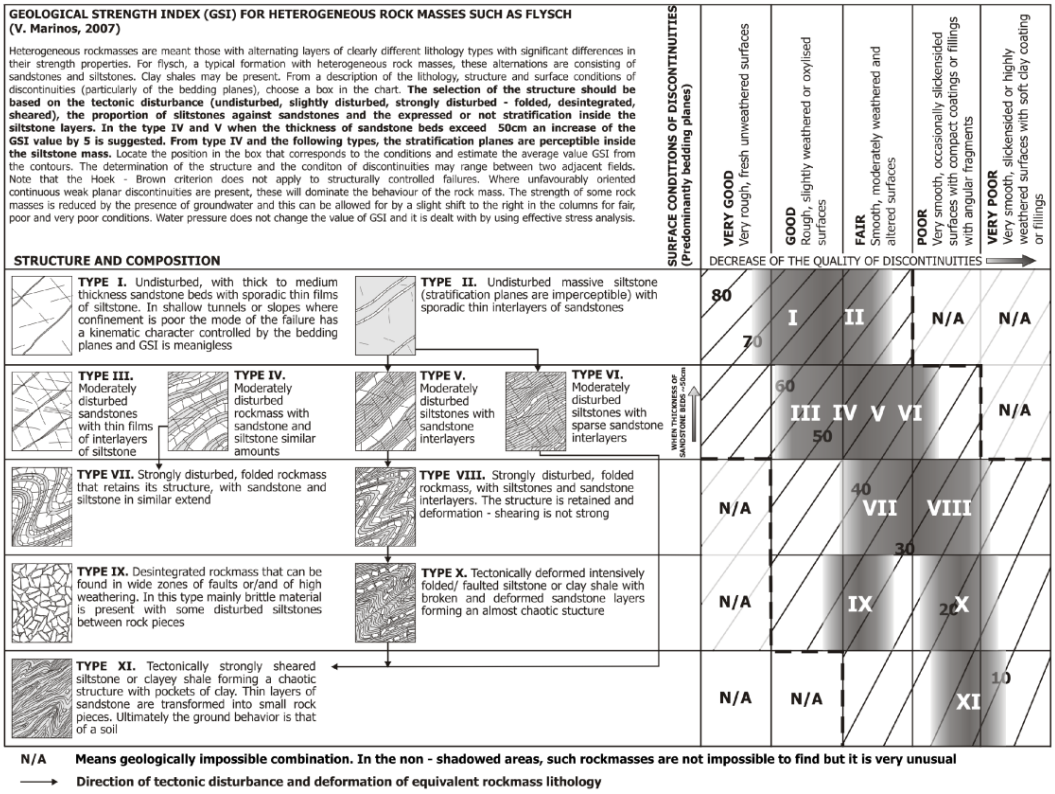


Figure 1. A new GSI classification chart for heterogeneous rock masses such as Flysch

Table 1. Suggested proportions of values for the "intact rock" properties estimation of each flysch rock type

Flysch type	Proportions of values for the weighted "intact rock" property estimation
I, III	Use values for sandstone beds
II, VI, XI	Use values for siltstone or shale
IV	Thin beds: Reduce sandstone values by 10% and use full values for siltstone Thick beds: Use equivalent values for siltstone and sandstone beds
V, VII, VIII	Reduce sandstone values by 20% and use full values for siltstone
IX	Use initial values for siltstone and sandstone beds without reduction according to their participation
X	Reduce sandstone values by 40% and use full values for siltstone

Note: if adjusted values are found lower than the value of the weak layer, use that value.

3 ENGINEERING GEOLOGICAL BEHAVIOUR DURING TUNNELLING

Flysch can be stable, even under high overburden, exhibit wedge sliding and wider chimney type failures, or cause serious deformation even under low to medium overburden. Its behaviour is controlled by its main geotechnical characteristics, as well as by the in-situ stress and groundwater conditions. The study on the behaviour of various flysch types was based on a large set of data (TIAS database) from excavation face mapping, rock fall records, convergence measurements and in-situ observation of many excavation faces. A range of geotechnical behaviour during tunnelling for a number of flysch rock mass types (I to IX) is presented hereafter.

Types I and III. The rock mass behaviour is purely anisotropic and is controlled by the orientation of discontinuities, mainly the bedding, in

relation to the orientation of the tunnel. As a result, there is a possibility of wedge detachment and sliding. Sliding can occur along thin siltstone layers with low shear strength that are often present on bedding planes, especially in Type III.

Type II. The behaviour of the rock mass is controlled by the low strength of the siltstone and the excavation depth. In great depths limited deformation can develop, whereas in small depths the tunnel is generally stable and, depending on the orientation of the tunnel and the discontinuities, sliding and fall of wedges can occur.

Type IV. The behaviour of the rock mass is anisotropic. In case of unfavourable discontinuity orientation, mainly of the bedding, in relation to the tunnel orientation, detachment and sliding of blocks can be observed, possibly along siltstone layers. When the layers are close to horizontal and especially when the rock mass is thin-bedded, overexcavation problems can appear. In places where the rock mass is locally more loose and weathered with no significant confinement, limited chimney type failures can occur.

Types V and VI. The rock mass behaviour is close to isotropic concerning deformation. Behaviour is controlled by the low strength of siltstone and limited deformation can develop under medium overburden. In small depths the tunnel is generally stable, but depending on the orientation of discontinuities, sliding and fall of wedges can occur. Close to the surface extended overexcavation and chimney type failures can appear, due to weathering and foliation, especially in Type VI (reduced sandstone presence).

Type VII. The behaviour of the rock mass can be well considered as isotropic. It is controlled by the low strength of the intact rock and limited deformation starts to develop under medium overburden. Yet, there is a possibility of local wedge detachment and sliding, enhanced by the siltstone layers, if the geometry of joints favours it. As a result of the relatively good "interlocking" of the rock mass due to its folded structure, no extended falls are expected, except only in weathered zones close to the surface.

Type VIII. The rock mass behaviour is clearly isotropic. Due to the low strength of the siltstone, deformation starts to develop under medium overburden. Detachments and slides of blocks may locally occur. As a result of the relatively good "interlocking" of the rock mass due to its folded structure, extended falls are only expected in weathered parts in very small depths. Such cases need great care, as extended overexcavation and chimney failures can be observed, due to weathering and foliated structure.

Type IX. This rock mass type is not characteristic of a typical flysch, as siltstone formations do not usually exhibit brittle behaviour. It is however often encountered in meta-flysch like series, such as the "Athenian Schist" excavated for the Athens metro. The behaviour of the rock mass is isotropic, governed by the disintegrated structure, and after excavation it can start to collapse. Although the equivalent friction angle is high, the equivalent cohesion of the disintegrated mass is practically negligible, except if some secondary fine binding material gives a small cohesion to the rock mass. In cases of open structure and strong presence of water, raveling is immediate and extensive and can not be easily limited until the induced void creates a ground arch or reaches the ground surface. In great depths, as the intact rock has a considerable strength, no significant deformation is expected.

Types X and XI. The behaviour of the rock mass is clearly isotropic, controlled by its low strength and high deformability that are responsible for the development of important deformation, even under low to medium overburden. In greater depths, squeezing conditions can be adverse [6], [7], [8], causing sometimes failure of rigid support sections due to overloading of the shell, especially in Type XI. This can lead to adoption of a yielding support that can undertake the high loads without failing. Additionally, particular care is needed close to the surface, where important overexcavation can occur, due to weathering and the foliated, fragile structure. Finally, an additional problem arises, regarding estimation of intact rock parameters, as it is difficult to take representative intact samples. Marinos et al. [6] and Grasso et al. [7] propose geotechnical parameters values for flysch rock masses of Type XI through back analyses.

Proceedings of the 15th European Conference on Soil Mechanics and Geotechnical Engineering
A. Anagnostopoulos et al. (Eds.), Part 1, pp. 435-440, Athens 2011.

ROCKMASS TYPE	STRUCTURE	TEMPORARY SUPPORT RECOMMENDATIONS
Type I. Undisturbed, with thick to medium thickness sandstone beds with sporadic thin films of siltstone.		• Excavation step: ≥3.0m • Installation of split-set bolts (e.g. Swellex) to support the unstable wedges (Sparse installation is not recommended due to the large dimensions of typical transportation tunnels)
Type II. Undisturbed massive siltstone with sporadic thin interlayers of sandstones.		• Excavation step: 2-3m • Bolts installation to support the unstable wedges and control the deformation in case of high overburden • Light steel sets in case of weathered rockmass, depending on excavation depth
Type III. Moderately disturbed sandstones with thin of siltstone interlayers.		• Excavation step: 1.5-2m • Installation of split-set bolts (e.g. Swellex type) for the support of unstable wedges • Light steel sets in case of loose structure
Type IV. Moderetaly disturbed rock mass with sandstone and siltstone similar amounts.		• Excavation step: 1.5-2m • Systematic bolt installation to support the unstable wedges, prevent the rockmass loosening and control the deformation in case of high overburden • Spiles and light steel sets in case of loose structure and weathered rockmass to avoid local chimney type failures
Type V. Moderately disturbed siltstones with thin sandstone interlayers.		• Excavation step: 1.5-2m • Systematic bolt installation to support the unstable wedges, prevent rockmass loosening and control the deformation under high overburden • Light steel sets to increase the rigidity and strength of the support shell • Spiles in case of loose and weathered structures to avoid chimney type failures • Face retaining measures: Depending on excavation depth (fibreglass nails)
Type VI. Moderately disturbed siltstones with sparse sandstone interlayers.		• Excavation step: 1.5-2m • Dense bolt pattern to control the deformation and prevent rockmass loosening • Steel sets to increase the rigidity and strength of the support shell • Spiles to stabilise loose and weathered structures and avoid chimney type failures • Face retaining measures: Depending on excavation depth (fibreglass nails) • Depending on bedding orientation, anisotropic stress induced deformations may be observed
Type VII. Strongly disturbed, folded rock mass that retains its structure, with sandstone and siltstone in similar extent.		• Excavation step: 1.5-2m • Dense bolt pattern to control of deformation and rockmass loosening prevention • Steel sets to increase the rigidity and strength of the support shell • Face retaining measures: Depending on excavation depth (fibreglass nails or/and forepolling)
Type VIII. Strongly disturbed, folded rock mass with siltstones and sandstone interlayers. The structure is retained and deformation – shearing is not strong.		• Excavation step usually small: 1-1.5m • Dense bolt pattern to control the deformation • Steel sets to increase the rigidity and strength of the support shell • Face retaining measures: Depending on excavation depth (fibreglass nails or/and forepolling) • Permanent and probably temporary invert to improve the shell rigidity.
Type IX. Disintegrated rockmass that can be found in wide zones of faults or/and of high weathering.		• Excavation step usually small (~1m) • Face buttress • Dense pattern of self-drilling anchors. Grouting to locally increase the rockmass cohesion • Steel sets to increase the rigidity and strength of the support shell • Spiles to presupport tunnel roof and prevent the development of chimney type failure • Alternatively in case of completely cohesionless rockmass grouting around tunnel section is proposed (e.g. through perforated forepolles)
Type X. Tectonically deformed intensively folded/faulted siltstone or clay shale with broken and deformed sandstone layers forming an almost chaotic structure.		• Small excavation step (~1m) • Dense bolt pattern to control the deformation • Steel sets in order to increase the rigidity and strength of the support shell • Face retaining measures: Depending on excavation depth (fibreglass nails or/and forepolling) • Permanent and temporary invert to improve the shell rigidity
Type XI. Tectonically strongly sheared siltstone or clayey shale forming a chaotic structure with pockets of clay.		• Small excavation step (~1m) • Dense bolt pattern and steel sets to increase the rigidity and strength of the support shell • Face retaining measures: Depending on excavation depth (fibreglass nails or/and forepolling) • Permanent and temporary invert to improve the shell rigidity • In case of very high overburden (>100-150m) the construction of a flexible support system using yielding elements may be required.

Remarks:

- The excavation is referred to Top heading and Bench method. Full face excavation in weak rockmasses imposes strong face retaining measures and small distance between temporary support and final lining.
- Shotcrete is not referred in the recommendations due to its wide application. More specifically, when shotcrete is used to avoid rockmass loosening and to ensure the personnel safety, its thickness is generally small and it is determined according to experience and evaluation of the magnitude of possible wedge failure. In stress induced phenomena due to the combination of weak rockmass and high excavation depth or/and swelling phenomena, shotcrete should be analysed as a structural element and the requisite thickness and reinforcement is determined through numerical analyses.
- The excavation step will be determined according to: (a) the anticipated size of wedges in the case of competent undisturbed rockmasses (b) the size of the wedges and the structure loosening prevention, in the case of disturbed rockmasses with no deformation problems (c) the prevention of structure loosening and decrease of deformation, in the case of weak rock masses where significant deformation is anticipated. However, the installation of spiles allows the increase of the excavation step.
- Drainage holes are proposed in case of permeable sandstone beds and relief holes in case of trapped, low permeable, groundwater zones under the water table.
- Special support requirements should be considered in case of swelling rockmasses (e.g. possible in type VI, VIII, X, XI).

Figure 2. General directions for the immediate support measures for every flysch type

4 CONCEPT AND RECOMMENDATIONS OF TEMPORARY SUPPORT

The wide range of engineering geological behaviour leads to a corresponding range of temporary support measures that can be applied in flysch rock masses. The temporary support proposed in the specific tunnels examined in the present paper varies from very light to very rigid or yielding for severe squeezing conditions. Some indicative suggestions for the philosophy of temporary support in tunnel excavation through each flysch type are presented in Figure 2. These proposals take into account both the rock mass behaviour and the critical failure mechanism, but do not replace the detailed design analysis of the tunnel support, adjusted to the in situ conditions and particularities of each project.

5 CONCLUSIONS

The present paper presents a new GSI chart for heterogeneous rock masses such as flysch, describes the critical failure mechanisms in tunnels and proposes guidelines for the immediate support for each flysch rock mass type.

New rock mass types are included in the new GSI chart, compared to the initial proposed for flysch-like rock masses by Marinos & Hoek in 2001 [3], whereas increased GSI values are generally assigned to undisturbed rock masses, even if they are dominated by siltstone.

Concerning its geotechnical behaviour, flysch can be stable, exhibit wedge sliding and wider chimney type failures or raveling, or develop serious deformation even under low overburden. Its behaviour is controlled by the siltstone-sandstone participation and the tectonic disturbance, as well as by the in-situ stress and groundwater conditions.

Finally, conceptual temporary support measures are proposed based on both the expected failure mechanism and some indicative geotechnical parameters values for intact sandstone and siltstone. Yet, they cannot replace the detailed analysis and the application of engineering judgement adjusted for each particular project separately.

ACKNOWLEDGEMENT

The authors would like to acknowledge Ms. D. Papouli, Geologist, MSc for her assistance in the editing of the paper.

REFERENCES

[1] V. Marinos, *Geotechnical classification and engineering geological behaviour of weak and complex rock masses in tunneling*, Doctoral thesis, School of Civil Engineering, Geotechnical Engineering Department, National Technical University of Athens (NTUA), Athens, 2007. (In greek)

[2] V. Marinos, P. Fortsakis, G. Prountzopoulos, P. Marinos, K. Korkaris, K. Mirmiris & D. Papouli. TIAS database: A tunnel information and analysis system. *In Proceedings of the 11th IAEG Congress, Auckland, New Zealand*, 2010.

[3] P. Marinos and E. Hoek, Estimating the geotechnical properties of a heterogeneous rock mass such as flysch, *Bulleting of Engineering Geology and the Environment*, **60** (2001), 82-92.

[4] I. Mihalis, A. Parathyras, G. Prountzopoulos, A. Papazachariou and L. Schwarz, 2010. "The Tunnels of Panagia-Anilio Section of the New Kalambaka-Ioannina-Igoumenitsa Railway. Evaluation of Geotechnical Investigation". *Proceedings of the 6th Hellenic Conference on Geotechnical and Geoenvironmental Engineering, Volos 29 September – 1 October 2010*, **2**, 343-350. (In Greek)

[5] H. Saroglou and H. Steiakakis, Prediction of strength of anisotropic and layered flysch-type rocks, *Proceedings of the 6th Hellenic Conference on Geotechnical and Geoenvironmental Engineering, Volos 29 September – 1 October 2010*, **2**, 243-249. (In Greek)

[6] V. Marinos, P. Fortsakis and G. Prountzopoulos. Estimation of rock mass properties of heavily sheared flysch using data from tunnelling construction. *Proceedings of the 10th IAEG International Congress, Nottingham, United Kingdom*, 2006.

[7] P. Grasso, G. Scotti, G. Blasini, M. Pescara, V. Floria, and N. Kazilis, Successful application of the observation design method to difficult tunnel conditions - Driskos Tunnel, *Underground Space Use: Analysis of the Past and Lessons for the Future, London, Erdem & Solak (eds)*, 463–470, Taylor & Francis Group, 2005.

[8] C.P. Tsatsanifos, P.M. Mantziaras and D. Georgiou Squeezing rock response to NATM tunneling: A case study, *Proceedings of the International Symposium on Geotechnical Aspects of Underground Construction in Soft Ground, Japan, Kusakabe, Fujita & Miyazaki (eds)*, 167-172, Balkema, Rotterdam, 2000.