Geotechnical Engineering Office, Civil Engineering and Development Department
The Government of the Hong Kong Special Administrative Region

GEO Technical Guidance Note No. 53 (TGN 53)
Supplementary Guidelines for Foundation Design and Construction

1. SCOPE

1.1 This Technical Guidance Note (TGN) updates the technical guidelines for foundation design given in the GEO Publication No. 1/2006 (GEO, 2006).

1.2 Any feedback on this TGN should be directed to the Technical Secretary of the GEO.

2. TECHNICAL POLICY

2.1 The guidelines promulgated in this TGN were agreed by the GEO's Geotechnical Control Conference (GCC).

3. RELATED DOCUMENTS

3.1 BD (2024). *Code of Practice for Foundations* 2017 (2024 Edition). Buildings Department, Hong Kong, 111 p.

3.2 GEO (2006). *Foundation Design and Construction (GEO Publication No. 1/2006)*. Geotechnical Engineering Office, Hong Kong, 376 p.

3.3 GEO (2025). *Supplementary Guidelines for Foundation Design in Areas underlain by Marble and Marble-bearing Rocks (Technical Guidance Note No. 26)*. Geotechnical Engineering Office, Hong Kong, 5 p.

3.4 Hoek, E. and Brown, E.T. (2019). The Hoek-Brown failure criterion and GSI – 2018 edition. *Journal of Rock Mechanics and Geotechnical Engineering*, vol. 11, pp 445-463.

3.5 Bieniawski, Z.T. (1989). *Engineering Rock Mass Classification*. John Wiley, New York, 251 p.

4. BACKGROUND

4.1 GEO Publication No. 1/2006 provides technical guidelines for the design and construction of foundations, prepared in alignment with the Code of Practice for Foundations (CoPF) issued by the Buildings Department in 2004. In 2017, the Buildings Department released the second edition of the CoPF (CoPF 2017). Consequently, a revision of GEO Publication No. 1/2006 is in progress to enhance the guidelines in line with advancements in foundation practice and to maintain consistency with the updated technical standards. However, it is considered desirable to publish supplementary guidelines on foundation design, whenever they are ready, that could improve practice and result in substantial saving in terms of construction cost and programme.

5. SUPPLEMENTARY TECHNICAL GUIDELINES

5.1 SHALLOW FOUNDATIONS FOUNDED ON GRANULAR SOILS

5.1.1 Paragraph 3.2.1.2 of the GEO Publication No. 1/2006 discusses the empirical method for determining the allowable bearing pressure obtained from SPT N values for non-cohesive soils. It cites the correlations by Terzaghi and Peck (1967) using 10 N and 5 N for dry and submerged conditions, respectively, which are conservative. The presumed allowable bearing pressure for shallow foundations on granular soils should be based on the values as given in Table 1, instead of the correlations by Terzaghi and Peck (1967).

5.1.2 Allowable bearing pressures for shallow foundations on soils can also be assessed using bearing capacity theory, provided that a sufficient factor of safety is applied and the predicted total and differential settlements remain within acceptable limits. Such assessments can justify adopting bearing pressures exceeding the values prescribed in Table 1.

5.1.3 For estimating settlement of foundations bearing on granular soils, the drained soil modulus E´ can be correlated with SPT N values, with E´ = 1.5 N for fill, alluvium and colluvium and E´ = 2 N for other insitu soils.

5.2 FOUNDATIONS FOUNDED ON IGNEOUS BEDROCK

5.2.1 Over the years, instrumented pile loading tests have been carried out for piles founded on various rock formations, including the comprehensive pile testing programme for the Airport Railway, West Rail, the West Kowloon District Development and private development at Sai Sha. GEO has collected and analysed these pile test data to improve knowledge and practice in foundation engineering. The instrumented pile loading test results indicate that there are substantial safety margins of the proven bearing capacity and shaft friction when compared with the recommendations given in the publication. Hence, updated guidelines on the presumed allowable bearing capacity and bond friction for igneous rocks have been derived.

5.2.2 The presumed allowable bearing pressures given in Table 2 should supersede Table 6.6 of the GEO Publication No. 1/2006. The use of the presumed values should be subject to the conditions given in the notes to Table 2. In addition, the presumed allowable bond/friction for piles socketed into igneous rocks of different weathering grades is also given in the same table.

5.3 FOUNDATIONS FOUNDED ON SEDIMENTARY AND META-SEDIMENTARY ROCKS

5.3.1 GEO has conducted a comprehensive instrumented pile loading test programme in 2024 - 2025, which supplements the understanding of foundations bearing on jointed rock masses. Table 3 serves as an additional recommendation to GEO Publication No. 1/2006, which provides presumed allowable bearing pressures and bond/friction for foundations bearing on Category 2 rocks applicable to sedimentary and meta-sedimentary rocks. The use of Table 3 should also be subject to the conditions given in the notes to Table 2. Where foundation design adopts a presumed allowable bearing pressure value exceeding 7500 kPa, pile loading test should be conducted to confirm the design bearing capacity of the rock mass.

5.3.2 The presumed allowable bearing pressures for Category 2 rocks are derived based on rock mass characterization using the Rock Mass Rating (RMR) system, with correlations established between RMR and the corresponding deformation modulus. The Hoek-Brown failure criterion (Hoek and Brown, 2019) is adopted to evaluate the rock mass strength parameters. For Category 2(a) rocks, an RMR range of 40 to 70 has been assumed for rock mass to demonstrate satisfactory performance in terms of both settlement and bearing stability under the presumed bearing pressures. For Category 2(b) rocks, the deformation modulus comparable to highly decomposed rocks (assumed to be equivalent to, at least, an RMR of 40) is adopted for settlement assessment. As Category 2(b) rocks should have a total core recovery of the specified grade strength exceeding 85%, normally the assumed RMR value of 40 could be achieved for Category 2(b) rocks. For Category 2(c) rocks, the presumed allowable bearing pressure is assessed by assuming the rock material equivalent to highly decomposed rocks for simplification. Nevertheless, where the assumptions for Category 2 rocks are deviated, settlement assessment of the rock mass under the presumed bearing pressures should be conducted. When assessing the settlement, including differential settlement, of structures, due consideration should be given to the basis for establishing the presumed allowable bearing pressures as detailed in Sections 5.4.2. A bearing pressure higher than that given in Table 3 may be determined by rational design approach, using the RMR method or other recognized engineering principles, and verified by pile loading tests.

5.3.3 Shale, mudstone and meta-mudstone can exhibit significant strength loss when in contact with water, whereas phyllite and graphitic schists may decompose into weak clayey silts. The presumed allowable bearing pressure for Category 2 rocks is not applicable to foundations bearing on these types of rock. Where these types of rock are present within the load influence zone (including rock bond/friction and end-bearing), separate assessment should be conducted and the design assumptions should be verified by pile loading test.

5.3.4 Uniaxial compressive strength (UCS) and point load strength index (PLSI) tests are typically used to determine intact rock strength. Preference should be given to UCS tests for establishing the intact rock strength. Where core specimens of sufficient length cannot be obtained for UCS testing in accordance with the relevant testing standard (e.g. ASTM), PLSI testing may be adopted as an alternative to obtain strength index values, PLI_{50}. For sedimentary and meta-sedimentary rocks, a correlation factor of 16 should be used to convert PLI_{50} to equivalent UCS. Rock specimens exhibiting prominent foliation or schistosity planes inclined at approximately 30° - 45° to the foundation loading direction should be selected, as this orientation typically corresponds to the critical minimum strength of anisotropic rocks.

5.3.5 Category 2 rock should exclude marble and marble-bearing rocks. TGN 26 provides supplementary guidelines for foundations bearing on marble and marble-bearing rocks.

5.4 DETERMINATION OF BEARING CAPACITY OF JOINTED ROCK MASS BASED ON RMR

5.4.1 GEO Publication No. 1/2006 provides a rational design method for determining the allowable bearing pressure for jointed rock masses using the RMR method. The findings from the GEO instrumented pile loading tests have supplemented the deformation characteristics of rock masses with RMR ranging between 36 and 60. Accordingly, a revised correlation between the deformation modulus and RMR has been established, with Figure 2 superseding Figure 6.7 of

GEO Publication No. 1/2006.

5.4.2 When deriving the allowable bearing pressure of the rock mass, it is not required that the RMR of all rocks within the zone of influence should be of the same or better rating. The allowable bearing pressure can be determined by specifying an acceptable settlement and should not exceed the uniaxial compressive strength of the intact rock. For structures that are not particularly sensitive to movement, the acceptable settlement can be taken as 30 mm. In determining the presumed allowable bearing pressures for Category 1 and 2 rocks, the settlement is also subject to an upper limit equal to 1% of the pile base diameter. The effect of foundation load on the rock mass is significantly reduced at depths greater than twice the pile base diameter. Hence, for the purpose of assessing the allowable bearing pressure, the minimum depth for computing RMR should be at least twice the pile base diameter or 5 m, whichever is greater. Table 6.5 and the allowable bearing pressures indicated in Figure 6.8 of the GEO Publication 1/2006 are deleted.

5.4.3 Where foundations bearing on rock masses with pervasive discontinuities are anticipated, based on ground investigations conducted in previous investigation stages, and where the RMR method is adopted for determining bearing capacity, ground investigation for foundation works (e.g. predrilling and post-construction post-drilling boreholes) should employ triple-tube core barrels with air foam as the flushing medium to recover rock cores at relevant depths. This drilling technique is effective in preserving salient geological features and maximizing core recovery for accurate RMR assessment.

5.4.4 When preparing logs for ground investigation boreholes that will subsequently be used for RMR assessment, the following enhanced logging practice should be adopted:

(a) Core logging should be carried out as promptly as practicable following completion of drilling to preserve the in-situ characteristics of rock specimens and to minimize post-extraction alteration. Photographs of rock cores should be taken during the logging process to record core conditions at the time of examination.

(b) Where joint conditions (including aperture, roughness and infilling) vary within intervals that result in different rock mass characteristics, a quantitative description should be provided to reflect such lithological and structural variations, including the orientation of any foliation and bedding planes. Textural descriptions covering a spectrum of joint conditions (e.g. smooth to very rough) should be avoided.

(c) Joint separation or aperture size should be documented according to the RMR rating on separation, ranging from none, < 0.1 mm, 0.1 - 1 mm, 1 - 5 mm and > 5 mm.

5.4.5 Table 4 updates the guidance on assigning individual ratings for RMR assessment and should replace Table 6.4 of the GEO Publication No. 1/2006. It provides a consistent approach for interpreting ground investigation records for the purposes of RMR assessment.

5.5 INTERACTION BETWEEN ADJACENT PILES AND OVERLAPPING OF END-BEARING STRESSES FOR LARGE DIAMETER BORED PILES BEARING ON ROCKS

5.5.1 When large-diameter bored piles bearing on rock are placed in close proximity, there is often a concern over whether overlapping bearing stresses beneath the piles may exceed the presumed allowable bearing pressure. A common solution is to install piles to greater depths below a hypothetical 45° envelope projected upward from the deepest pile base to avoid such stress interaction. Recent studies have investigated this interaction for piles at different spacings and founding levels using elastic theory and numerical modelling. The findings confirm that the increase in vertical stress due to foundation load disperses at a ratio of 1:2 (horizontal : vertical), with the extent confined to a zone extending radially from the edge of the pile base to a distance equal to the pile base diameter (B), and vertically downward to a depth of approximately 2.5B below the base. Hence, there is no need to restrict the founding level of adjacent piles to below a 45° envelope projected upward from the founding level of the deepest pile.

5.5.2 The minimum clear spacing between large-diameter bored piles is typically 0.5 m as stipulated in GEO Publication No. 1/2006. Provided that piles are spaced at distances greater than this minimum clear spacing, the overlapping of vertical stress will not exceed the applied bearing pressure in the bearing rock stratum. The founding level of piles need not be lowered solely to avoid overlapping of bearing stress.

5.5.3 Where the clear spacing between piles is less than the largest pile base diameter and the elevation difference between pile bases exceeds this clear spacing, an assessment should be conducted. This assessment should ensure that piles are not founded on corestones or affected by steeply inclined bedrock profiles, and that the structural capacity of the pile is adequate, taking into account any stress increase in the pile shaft caused by adjacent piles founded at higher levels.

5.5.4 The above principles are also applicable to rectangular loaded areas, including barrettes and footings. For simplicity, the largest dimension of the rectangular loaded area of barrettes and footings should be used when comparing the clear spacing between them.

(Raymond WM Cheung)
Head, Geotechnical Engineering Office

Table 1 – Presumed Allowable Vertical Bearing Pressure under Foundations in Non-cohesive Soils (Sands and Gravels) on Horizontal Ground

Category	Descriptions for non-cohesive soils (sands and gravels)	Presumed allowable bearing pressure (kPa)
4(a)	Very dense, SPT N value > 50	500
4(b)	Dense, SPT N value 30 - 50; requires pick for excavation; 50 mm peg hard to drive	300
4(c)	Medium dense, SPT N value 10 - 30	100
4(d)	Loose, SPT N value 4 - 10; can be excavated with spade; 50 mm peg easily driven	< 100

Note:

1. Foundations should be founded at a depth of at least 1 m below ground surface.
2. The groundwater level should not be higher than 1 m below ground surface.
3. The presumed allowable bearing pressures are established based on a maximum total settlement of 30 mm. Serviceability of structures should be assessed.
4. For foundations exceeding 6 m in dimension or where conditions differ from Notes 1 to 2 above, a separate assessment of allowable bearing capacity and settlement should be conducted based on the settlement tolerance of the structure.
5. Sands and gravels should be of silica in nature and non-crushable.
6. Soil category is defined in accordance with the Code of Practice for Foundations, 2017 (BD, 2024).

Table 2 – Presumed Allowable Bearing Pressure and Bond/Friction for Foundations bearing on Igneous Rocks

Category	Description of Igneous Rocks	Presumed Allowable Bearing Pressure (kPa)	Presumed Allowable Bond or Friction (kPa)
1(a)	Fresh to slightly decomposed strong to very strong granite or volcanic rock of material weathering grade II or better, with 100% TCR of the designated grade which has a minimum UCS of rock material not less than 75 MPa (or an equivalent point load index strength PLI_{50} not less than 3 MPa)	12500	1000 (under compression or transient tension) 800 (under permanent tension)
1(b)	Fresh to slightly decomposed strong granite or volcanic rock of material weathering grade II or better, and with not less than 95% TCR of the designated grade, which has a minimum UCS of rock material not less than 50 MPa (or an equivalent point load index strength PLI_{50} not less than 2 MPa).	10000	
1(c)	Slightly to moderately decomposed moderately strong granite or volcanic rock of material weathering grade III or better, and with not less than 85% TCR of the designated grade, which has a minimum UCS of rock material not less than 25 MPa (or an equivalent point load index strength PLI_{50} not less than 1 MPa).	7500	700 (under compression or transient tension) 560 (under permanent tension)
1(d)	Moderately decomposed, moderately strong to moderately weak granite or volcanic rock of material weathering grade III or better, and with not less than 50% TCR of the designated grade.	3000	300 (under compression or transient tension) 150 (under permanent tension)

Notes to Table 2:

(1) The presumed values for allowable bearing pressure given are for foundations with negligible lateral loads at bearing level.
(2) The self-weight of the length of pile embedded in soil or rock does not need to be included into the calculation of bearing stresses.
(3) Minimum socket depth along the pile perimeter is 500 mm for categories 1(a) and 1(b), and 300 mm for categories 1(c) and 1(d).
(4) TCR of the designated grade is defined in Figure 1.
(5) The TCR of the designated grade should be verified to a depth at least 5 m into the specified category of rock, or the rock socket length, whichever is greater. Where the foundations also rely on end-bearing resistance, the rock within the depth of 5 m below the foundation base should be verified. This requirement is deemed to be complied with if the rock core within the above verified depth can be divided into a number of segments (in consecutive manner) such that (a) each segment is 1 m; and (b) the calculated TCR in accordance with Figure 1 of each segment should satisfy the required percentage of TCR of the designated grade.
(6) The bearing surface of rock on which the foundation will be rested should be of the designated category and no rock core within 600 mm depth from the pile base is logged as “non-intact” in accordance with GEOGUIDE 3.
(7) Weathering grades and non-intact condition are defined in GEOGUIDE 3.
(8) The point load index strength of rock quoted in the table is the equivalent value for 50 mm diameter cores.

(9) Concrete or grout should have a minimum characteristic compressive strength of 30 MPa.

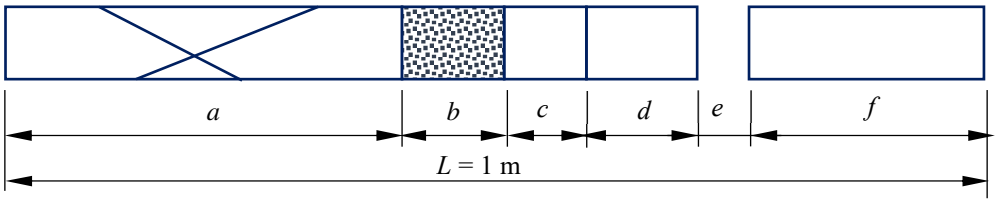

Figure 1 – Determination of TCR for the Designated Grade of Rock

$$\text{TCR of the designated grade} = \frac{a+c+d+f}{L}$$

1. a, c, d and f are materials of the designated grade or better.
2. b are materials inferior than the designated grade.
3. e are materials washed away during drilling.
4. The maximum continuous length of materials washed away/inferior to the designated grade, $b+e$, should not be greater than 300 mm.
5. TCR of the designated grade should not be confused with TCR of the core run shown in the site investigation report, which is equal to $\frac{a+b+c+d+f}{L}$.

Table 3 – Presumed Allowable Bearing Pressure and Bond/Friction for Foundations bearing on Sedimentary and Meta-sedimentary Rocks

Category	Description of Sedimentary and Meta-sedimentary Rocks	Presumed Allowable Bearing Pressure (kPa)	Presumed Allowable Bond or Friction (kPa)
2(a)	Fresh to moderately decomposed, strong to moderately strong sedimentary or meta-sedimentary rocks of material weathering grade III or better, with not less than 85% of TCR of the designated grade, which has a minimum UCS of rock material not less than 25 MPa (or an equivalent point load index strength PLI_{50} not less than 1.6 MPa), with an RMR at least 40.	5000 - 12500	1000 (under compression or transient tension) 800 (under permanent tension)
2(b)	Moderately decomposed, moderately strong sedimentary or meta-sedimentary rock of material weathering grade III or better, and with not less than 85% of TCR of the designated grade, which has a minimum UCS of rock material not less than 20 MPa (or an equivalent point load index strength PLI_{50} not less than 1.25 MPa).	5000	700 (under compression or transient tension) 560 (under permanent tension)
2(c)	Moderately decomposed, moderately strong to moderately weak meta-sedimentary or sedimentary rock of material weathering grade III or better, and with not less than 50 % of TCR of the designated grade.	3000	300 (under compression or transient tension) 150 (under permanent tension)

Notes to Table 3:

(1) For end-bearing foundations, TCR and/or RMR should be assessed within the load influence zone beneath the foundation base, which is twice the diameter of the foundation base, or 5 m, whichever is deeper.

(2) The presumed allowable bearing pressure for Category 2(a) rock should be linearly interpolated between 5000 kPa and 12500 kPa, corresponding to RMR of 40 to 70.

(3) When assessing the RMR, rock cores should be divided into a number of segments (in consecutive sequence) such that (a) each segment is 1 m in length; and (b) the average RMR of each segment satisfies the RMR corresponding to the adopted presumed bearing pressure.

(4) Where presumed allowable bearing pressure of more than 7500 kPa is adopted, pile loading test should be conducted to confirm the design bearing pressure.

(5) Minimum socket depth along the pile perimeter is 500 mm for Categories 2(a) and (b); and 300 mm for Category 2(c) rocks.

(6) For shallow foundations, the founding level should be located at least 1.5 m below the ground surface.

(7) Rock core specimens should be selected for UCS testing. Where length of core specimens is not sufficient for conducting UCS testing, PLSI testing may be adopted as an alternative to obtain strength index values. Specimens exhibiting prominent foliation or schistosity planes inclined at approximately 30° - 45° to the foundation loading direction should be selected, as this orientation typically corresponds to the critical minimum strength of anisotropic rocks.

(8) The presumed allowable bearing pressures for Category 2 rocks should not be applied to shale, mudstone, meta-mudstone, phyllite and graphite schist. Allowable bearing pressures for foundations bearing on these rock types should be determined by separate assessment and validated by pile loading tests.

(9) Other conditions listed in CoPF 2017 and Table 2 should be followed.

(10) Assumptions for establishing the presumed allowable bearing pressures are given in Sections 5.3.2 and 5.4.2. Where the assumptions are deviated, a settlement assessment of the rock mass should be conducted.

Table 4 – Rating Assigned to Individual Parameters using RMR Classification System (Based on Bieniawski, 1989)

(A) Strength of Intact Rock						
Uniaxial compressive strength, σ_c (MPa)	> 250	250 - 100	100 - 50	50 - 25	25 - 5	5 – 1	< 1
Point load strength index, PLI_{50} (MPa)	See Note (1) for converting PLI_{50} to UCS, σ_c						
Rating	15	12	7	4	2	1	0
(B) Rock Quality Designation (RQD)						
RQD (%)	100 - 90	90 - 75	75 - 50	50 - 25	< 25		
Rating	20	17	13	8	3		
(C) Spacing of Joints[2],[3]						
Spacing	> 2 m	2 m - 0.6 m	0.6 m - 0.2 m	200 - 60 mm	< 60 mm		
Rating	20	15	10	8	5		
(D) Conditions of Joints[4]						
Discontinuity length[8]						
Rating	2						
Separation[5]	None	< 0.1 mm	0.1 - 1 mm	1 - 5 mm	> 5 mm		
Rating	6	5	4	1	0		
Roughness[6]	Very rough	Rough	Slightly rough	Smooth	Slickenside		
Rating	6	5	3	1	0		
Infilling (gouge)[7]	None	Hard filling < 5 mm	Hard filling > 5 mm	Soft filling < 5 mm	Soft filling > 5 mm		
Rating	6	4	2	2	0		
Weathering	Unweathered	Slightly weathered	Moderately weathered	Highly weathered	Decomposed		
Rating	6	5	3	1	0		
(E) Groundwater						
Rating[8]	7						

Notes :

(1) Where point load strength index values are determined from rock specimens, the equivalent uniaxial compressive strength (UCS), σ_c, can be derived using correlation factors of 25 and 16 with PLI_{50}, for Category 1 and 2 rocks, respectively. Where rock strength is less than 20 MPa, UCS testing is preferred.

(2) Joint spacing should be calculated by taking the weighted average of the reciprocal of the fracture index (FI) applied to intact core recovery segments of 1 m or less in vertical extent.

(3) For rock core intervals with FI > 20, or designated as “no recovery” or “non-intact”, a rating of 5 should be assigned.

(4) Where no specific depth is given for the joint conditions recorded in the borehole logs, those conditions should be applied to the entire core interval.

(5) Where aperture size is recorded in accordance with Table 9 of Geoguide 3, the upper bound value of the aperture size should be adopted for assigning the rating on separation.

(6) Where joint roughness is described in accordance with Geoguide 3, the following references should be adopted for deriving the roughness rating for RMR assessment:

Rough stepped or rough undulating: Very rough
Rough planar: Rough
Smooth stepped, or smooth undulating: Slightly rough
Smooth planar: Smooth

Slickensided stepped, slickensided undulating, slickensided planar: Slickensided

(7) For rating on infilling (gouge), where descriptions of hardness and thickness of infilling material are not given, the following reference should be adopted for deriving the RMR:

Cohesive and non-cohesive soils, kaolin, chlorite: Soft filling
Calcite, Manganese and Quartz: Hard filling
Clean, stained, no infill: A rating of 6 should be assigned
Coating: Thickness < 5 mm
Infilling: Thickness ≥ 5 mm

(8) The ratings are fixed as the parameter is considered not relevant to the evaluation of allowable bearing pressure of rock mass.

(9) RMR is calculated as the sum of individual ratings assigned to parameters (A) through (E).

(10) Separate RMR should be assigned for each core segment exhibiting changes in any individual RMR parameter (including RQD, fracture index, joint aperture, roughness, or infilling). The overall RMR for the assessed length of rock cores should be computed as the thickness-weighted average of the individual segment ratings. For existing boreholes containing only generic descriptions of individual intervals, individual ratings should be conservatively interpreted for RMR assessment purposes.

(11) RMR assessment should be performed for all rock mass intervals, commencing from initial interception of Category 2(b) or better rocks to the terminal depth of the borehole.

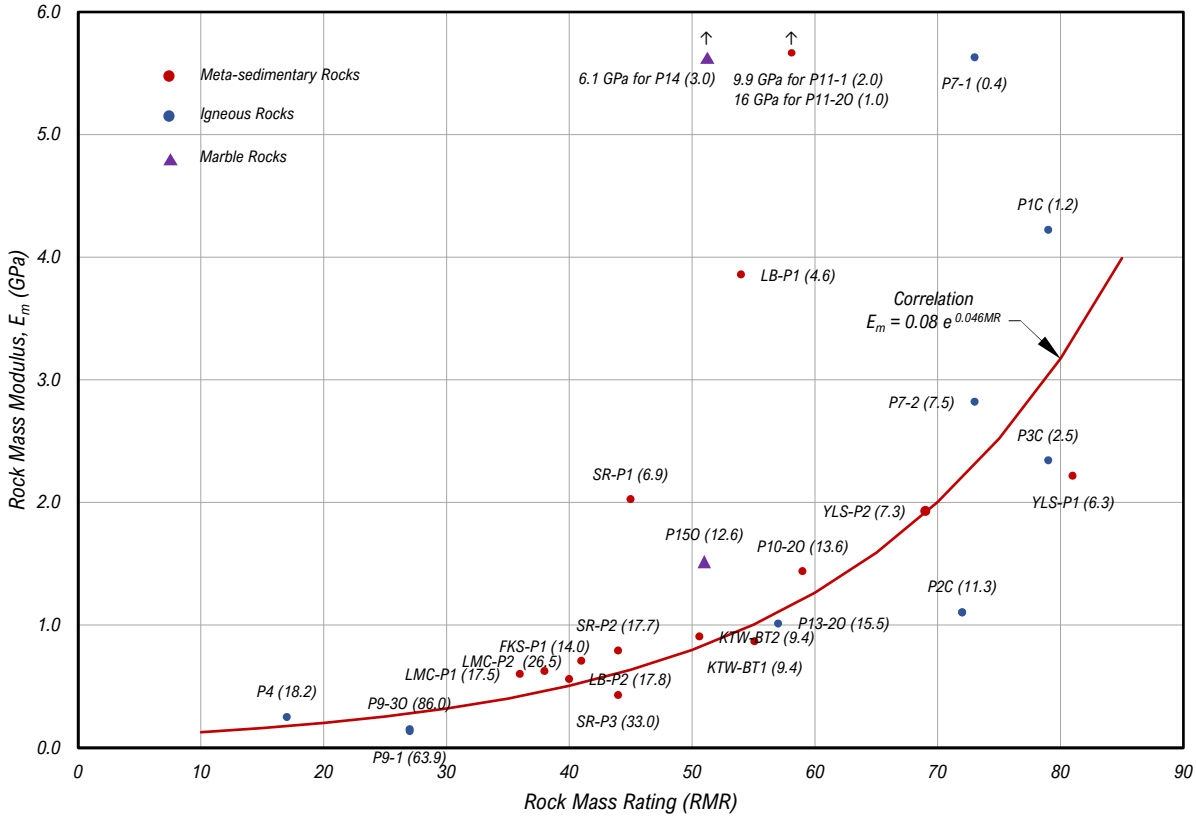

Figure 2 – Relationship between Deformation Modulus and RMR for a Jointed Rock Mass