Phenomenological vs Mechanistic

Phenomenology: a 20th-century philosophical movement, the primary objective of which is the direct investigation and description of phenomena as consciously experienced, without theories about their causal explanation and as free as possible from unexamined preconceptions and presuppositions.

For example, one observes a phenomenon and asks how/why that specific phenomenon occurs (i.e. inductive reasoning).

Samples removed from a triaxial cell have inclined failure planes, therefore failure must have occurred through shear and the development of shear fractures.
Phenomenological -vs- Mechanistic

Such approaches are ‘holistic’ as they disregard details of the underlying mechanisms while concentrating on the overall performance of a system.

Mechanistic approaches on the other hand, try to break the problem/system down into its constituent parts to understand the cause and effect relationships (and their evolution), which govern the behaviour of the system.

Close examination of samples prior to failure (i.e. through thin sections, acoustic emission, etc.) show the development of tensile microfractures that propagate and coalesce into larger macroscopic fractures with increasing loads.

Phenomenological Approach: Case History

Kilchenstock, Switzerland (1930)

Klöpfl (1997)
**Phenomenological Approach: Case History**

Kilchenstock: Where were we 70 years ago?

Nov. 1st telegram to the Canton President from Prof. Albert Heim:

"The slide seems to be near, recommend an order to evacuate and flee" 

Löw (1997)

"Lack of experience at Kilchenstock has misled us"
Phenomenological Approach: Case History

Kilchenstock: Where were we 70 years ago?

"Whoops!!"

Phenomenological Approach: Limitations

Grimselstrasse, Switzerland (2000)
Phenomenological Approach: Limitations

Temporal Prediction of Failure:

- Road closed
- Water injection down tension crack (~ 9000 l/min)
- Blast - 19 tonnes of explosives (for 150,000 m² of rock)
Today:
In many ways, we are still using the same phenomenological approaches to assess rock slope hazards and the potential for catastrophic failure as practitioners were using several decades ago.

- Advancements have been made through these techniques, however success is still variable.
- How well do we understand the processes and mechanisms promoting instability?

Key Questions:
Processes and mechanisms driving instability and shear/failure plane evolution:
- Prediction & early warning capabilities
- Mechanisms, spatial & temporal variability
In the 1940's, Karl Terzaghi adapted the phenomenological approach to develop a systematic means to solve geotechnical problems. This has become known as the "observational method", the conceptualization behind which Terzaghi wrote (paraphrased here):

"In the engineering of large geotechnical works, a vast amount of effort goes towards securing only roughly approximate values for the physical constants that appear in the equations. In these equations many additional variables are not considered or remain unknown. Therefore, the results of computations are no more than working hypotheses, subject to confirmation or modification during construction."

"In the past, only two methods have been used for coping with inevitable uncertainties: either to adopt an excessive factor of safety, or else to make assumptions in accordance with general experience. The first of these is wasteful; the second is dangerous as most failures occur due to unanticipated or unknown geological factors/processes."

"A third method, the observational method, provides a 'learn as you go' alternative. The procedure for this is to base the design on whatever information can be secured, make note of all possible differences between reality and the assumptions, then compute, on the basis of the original assumptions, various quantities that can be measured in the field. Based on the results of these measurements, gradually close the gaps in knowledge and, if necessary, modify the design during construction."

Terzaghi & Peck (1948)
The Observational Method in Design

In brief, the complete application of the method embodies the following components:

a) Sufficient exploration to establish the general nature, pattern and properties of the soil deposits or rock mass;

b) Assessment of the most probable conditions and the most unfavourable conceivable deviations from these conditions;

c) Establishment of the design based on a working hypothesis of behaviour anticipated under the most probable conditions;

d) Selection of quantities to be observed as construction proceeds and calculation of their anticipated values on the basis of the working hypothesis;

e) Calculation of values of the same quantities under the most unfavourable conditions compatible with the available data concerning the subsurface conditions;

f) Selection in advance of a course of action or modification of design for every foreseeable significant deviation of the observational findings from those predicted on the basis of the working hypothesis;

g) Measurement of quantities to be observed and evaluation of actual conditions;

h) Modification of design to suit actual conditions.
The Jubilee Line Extension to the London Underground, started in 1994 and called for twin tunnels 11 km long, crossing the river in four places, with eleven new stations to be built, eight of which were to be underground. One of the more problematic of these was a station placed right opposite Big Ben.

The technical implications were immense. Built in 1858, Big Ben is known to be on a shallow foundation. It started to lean towards the North shortly after completion. Any ground movement in the vicinity would exaggerate this lean, and threaten the stability of the structure.
Observation Method Example – Jubilee Extension

To deal with excavation-induced settlements that may irreversibly damage historic buildings in the area, the design called for the use of compensation grouting during tunnelling. In this process, a network of horizontal tubes between the tunnels and the ground surface is introduced, from which a series of grout holes are drilled. From these, liquid cement can be injected into the ground from multiple points to control/prevent movement during excavation of the main tunnels.

- learn as you go
- base the design on information that can be secured, making note of all possible differences between reality and the assumptions
- compute, based on original assumptions, various quantities that can be measured in the field
- based on the results of these measurements, gradually close the gaps in knowledge and, if necessary, modify during construction.

The observational method:

- learn as you go
- base the design on information that can be secured, making note of all possible differences between reality and the assumptions
- compute, based on original assumptions, various quantities that can be measured in the field
- based on the results of these measurements, gradually close the gaps in knowledge and, if necessary, modify during construction.
Observation Method Example – Jubilee Extension

Instrumentation was attached to Big Ben and to the buildings in the vicinity to measure movement (with some 7000 monitoring points), and computers were used to analyze the data to calculate where and when the grout has to be injected.

For Big Ben, a movement of 15 mm at a height of 55m (approximately the height of the clock face above ground level) was taken to be the point at which movement had to be controlled. Throughout the 28 month construction period, experience had to be gained as to which tube to use for grouting, the volume of grout to be injected and at what rate.

It was calculated that without the grouting, the movement of Big Ben would have gone well over 100 mm, which would have caused unacceptable damage.

Following construction, the grouting pipes were left in place and monitoring continued. Thus, compensation grouting can be restarted if required. However, instrumentation is showing that no further grouting is necessary.
Classification Systems in Design

Even with many resources available for site investigation, there still can remain problems in applying theories in practical engineering circumstances. Considering the three main design approaches for engineering rock mechanics—analytical, observational and empirical, rock mass classifications today form an integral part of the most predominant design approach, the empirical design method.

Indeed, on many underground construction, tunnelling and mining projects, rock mass classifications have provided the only systematic design aid in an otherwise haphazard “trial-and-error” procedure.

Rock Mass Failure Mechanisms

The Stability of an underground opening is a function of:

- structurally-controlled failure
- stress: low or high
- structure: falling, sliding
- rock mass
The boundaries of the structural regions usually coincide with a major structural feature such as a fault or with a change in rock type. In some cases, significant changes in discontinuity spacing or characteristics, within the same rock type, may necessitate the division of the rock mass into a number of small structural regions.

The objectives of rock mass classifications are to:

- Identify the most important parameters influencing the rock mass.
- Divide a rock mass formation into groups of similar behaviour.
- Provide a basis for understanding the characteristics of each rock mass class.
- Relate experiences of rock conditions at one site to those at another.
- Derive quantitative data and guidelines for engineering design.
- Provide a common basis for communication between geologists and engineers.

These objectives suggest the three main benefits of rock mass classifications:

- Improving the quality of site investigations by calling for the minimum input data as classification parameters.
- Providing quantitative information for design purposes.
- Enabling better engineering judgment and more effective communication on a project.
The Rock Mass Rating (RMR) system was developed in 1973 in South Africa by Prof. Z.T. Bieniawski. The advantage of his system was that only a few basic parameters relating to the geometry and mechanical conditions of the rock mass were required.

Rating adjustments are included to account for the adverse nature discontinuity angles may have with respect to the excavation or slope direction.

The adjusted value gives the final RMR value for the rock mass, for which several rock mass classes are described.

For example:

A mudstone outcrop contains three fracture sets. Set '1' comprises bedding planes; these are highly weathered, slightly rough and continuous. The other two sets are jointing; both are slightly weathered and slightly rough. The strength of the intact rock is estimated to be 55 MPa with an RQD of 60% and a mean fracture spacing of 0.4 m. The fractures are observed to be damp.
Example:
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$$RMR = 6 + 12 + R_3 + R_4 + R_5$$

$$RMR^* = 53 \text{ to } 58$$

$$RMR = 6 + 12 + (15 \text{ to } 20) + 10$$
The Q-system of rock mass classification was developed in 1974 in Norway by Prof. N. Barton. The system was proposed on the basis of an analysis of 212 tunnel case histories from Scandinavia.

\[
Q = \frac{\text{RQD} \cdot J_a \cdot J_w}{I_a \cdot I, \text{SRF}}
\]

where
- RQD = rock quality designation,
- \(J_a\) = joint set number (related to the number of discontinuity sets),
- \(I_a\) = joint roughness number (related to the roughness of the discontinuity surfaces),
- \(J_w\) = joint alteration number (related to the degree of alteration or weathering of the discontinuity surfaces),
- \(J_w\) = joint water reduction number (relates to pressures and inflow rates of water within the discontinuities), and
- SRF = stress reduction factor (related to the presence of shear zones, stress concentrations and squeezing and swelling rocks).

The motivation of presenting the Q-value in this form is to provide some method of interpretation for the 3 constituent quotients.

The first quotient is related to the rock mass geometry. Since RQD generally increases with decreasing number of discontinuity sets, the numerator and denominator of the quotient mutually reinforce one another.

The second quotient relates to “inter-block shear strength” with high values representing better ‘mechanical quality’ of the rock mass.

The third quotient is an 'environment factor' incorporating water pressures and flows, the presence of shear zones, squeezing and swelling rock and the in situ stress state. The quotient increases with decreasing water pressure and favourable in situ stress ratios.
**Rock Mass Classification - Examples**

- Massive, strong rock
- Low stress regime
- Note lack of ground support
- RMR = 90 (very good rock)
- Q = 180 (extremely good rock)

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**Rock Mass Classification - Examples**

- Blocky rock
- Low stress regime
- Minimal but systematic ground support
- RMR = 70 (good rock)
- Q = 15 (good rock)
Rock Mass Classification - Examples

- Weak/foliated rock
- Low stress regime
- Note lack of ground support
- RMR = 40 (poor to fair rock)
- Q = 0.9 (v.poor to poor rock)

Massive, strong rock
- Extremely high stress regime
- Rockburst failure, complete closure of drift, extremely heavy support, screen retains failed rock
- RMR = 80 (good to v.good rock)
- Q = 0.5 (very poor rock)
Rock Mass Classification - Examples

- blocky rock
- high stress regime
- RMR = 40
  (poor to fair rock)
- Q = 0.8
  (very poor rock)

Application of Classification Systems

Both of the classification systems described were developed for estimating the support necessary for tunnels excavated for civil engineering schemes. For example, the database for the RMR has involved over 351 case histories throughout its development.

Bieniawski (1989)
38 different support categories have been suggested by Barton (1974) based on the relationship between the Q index and the equivalent dimension of the excavation.
Experience-Based Design: Empirical Approaches

It must be remembered though, that such guidelines are drawn from previous experiences (i.e. case histories) and are therefore limited by the range of conditions under which these experiences were generated.

Rock Mass Characterization vs. Classification

Alternatively, one can approach rock engineering design by trying to understand the failure process.

Martin (2002)
**Rock Mass Properties - Strength**

Remember! - we're talking about rock mass failure, not structurally controlled failures.

Classes of rock strength

- Sliding surface along discontinuity?
  - Yes
  - Joints parallel to face
  - Pair of intersecting joints
  - Use discontinuity shear strength
  - Closely fractured rock
  - Weak, massive rock

- No
  - Use rock mass shear strength

**Mohr-Coulomb Failure Criterion**

The Mohr-Coulomb failure criterion expresses the relationship between the shear stress and the normal stress at failure along a shear surface.

**BASIC EQUATIONS**

- Rock fails at a critical combination of normal and shear stresses:
  \[ t_{ltl} = c + \sigma_N \tan \phi \]
  - \( c \) = cohesion
  - \( \tan \phi \) = coeff. of friction
  \[ t_{ltl} = \frac{1}{2}(\sigma_N - \sigma_T) \sin 2\beta \]
  \[ \sigma_T = \frac{1}{2}(\sigma_N + \sigma_T) + \frac{1}{2}(\sigma_N - \sigma_T) \cos 2\beta \]

**FUNDAMENTAL GEOMETRY**

The equation for \( t_{ltl} \) and \( \sigma_N \) are the equations of a circle in \((\sigma_N, t_{ltl})\) space.

At failure, 
\[ 2\beta = 90 + \phi \]

\[ \Rightarrow \beta = 45 + \frac{\phi}{2} \]
Hoek-Brown Failure Criterion

Generalized Hoek-Brown failure criterion:

\[ \sigma'_1 = \sigma'_3 + \sigma_{cl} \left( m_b \frac{\sigma'_3}{\sigma_{cl}} + s \right)^a \]

- Intact rock strength: \( m \) = lab-determined
- Rock mass strength: \( s = 1 \)

\( m \) & \( s \) are derived from empirical charts that are related to rock mass quality:
- \( m \) = Friction
- \( s \) = Cohesion

Rock Mass Properties - Strength

Mohr-Coulomb

\[ \tau = c' + \sigma \tan \phi' \]

\[ \sigma'_1 = \frac{2c' \cos \phi'}{1 - \sin \phi'} + \frac{1 + \sin \phi'}{1 - \sin \phi'} \sigma'_3 \]

Generalized Hoek-Brown

\[ \sigma'_1 = \sigma'_3 + \sigma_{cl} \left( m_b \frac{\sigma'_3}{\sigma_{cl}} + s \right)^a \]
Hoek-Brown Failure Criterion & GSI

Hoek: "It had become increasingly obvious that Bieniawski's RMR is difficult to apply to very poor quality rock masses. It was also felt that a system based more heavily on fundamental geological observations and less on 'numbers' was needed."

Generalized Hoek-Brown:

\[ a' = a_0 + c_d \left( m_b \frac{c_d}{c_3} + e^a \right) \]

\[ m_b = m_1 \exp \left( \frac{GSI - 100}{28 - 14D} \right) \]

\[ e^a = \frac{1}{2} + \frac{1}{6} (e^{-GSI/15} - e^{-20/3}) \]

The GSI thus provides a system for estimating the reduction in rock mass strength for different geological conditions.

Geological Strength Index (GSI)

Values of GSI are related to both the degree of fracturing and the condition of the fracture surfaces.

mainly jointing

mainly faulting
**Hoek-Brown Failure Criterion & GSI**

- \( m_b = m_i \exp\left(\frac{GSI - 100}{28 - 14D}\right) \)

- \( s = \exp\left(\frac{GSI - 100}{9 - 2D}\right) \)

**Disturbance Factor**

- \( m_b \) is a reduced form of the rock mass constant \( m_b \), which is a function of intact rock type.

- \( s \) is a rock mass constant based on how fractured the rock mass is (where \( s = 1 \) for intact rock).

**Appearance of rock mass**

- **Description of rock mass**
  - Small-scale blasting in civil engineering slopes results in modest rock mass damage, particularly if controlled blasting is used as shown on the left hand side of the photograph. However, stress relief results in some disturbance.
  - Very large open pit mine slopes suffer significant disturbance due to heavy production blasting, and also due to stress relief from overburden removal.

**Suggested value of D**

- \( D = 0.7 \) Good blasting
- \( D = 1.0 \) Poor blasting
- \( D = 1.0 \) Production blasting
- \( D = 0.7 \) Mechanical excavation

**“D” is the disturbance factor, which depends upon the degree to which the rock mass has been subjected to blast damage and stress relaxation.**

Wyllie & Mah (2004)
GSI - Simplified Procedure

A simplified procedure to determine the Hoek-Brown rock mass strength parameters ignores the 'disturbance factor'. First, after determining the GSI, calculate \( m_b \):

\[
m_b = m_t \exp \left( \frac{GSI - 100}{28} \right)
\]

For \( GSI > 25 \):

\[
s = \exp \left( \frac{GSI - 100}{9} \right), \\
a = 0.5
\]

For \( GSI < 25 \):

\[
s = 0, \\
a = 0.65 - \frac{GSI}{200}
\]

For more information, check out the free copy of Evert Hoek’s notes and “H-B” software available on-line:

http://www.rocscience.com/hoek/Hoek.asp

Rock Mass Characterization - Classification

Classify the rock mass using:

- \( Q \)
- RMR

Describe the joints:
- Block Size
- Roughness/Strength
- Tunneling Factors

Rock Mass Classification

Geological Strength Index

Hoek-Brown Failure Envelope
- \( m \) (friction)
- \( s \) (cohesive)
- \( \sigma_{Qj} \)

Design

Support Requirements

Stability Analysis

Quantified Factor of Safety
Rock Mass Characterization & Design

- Generalized Hoek-Brown Criterion

\\[ q = \frac{k_s}{k} \left( \frac{q_{\text{ult}}}{q_{\text{ult}} + 3q_{\text{ult}}} \right) \]

- Hoek et al. (1995)

Structure:

- Blocks: Very well interlocked, undisturbed rock mass
- Very blocks: moderately disturbed rock mass
- Blocks: Interpreted with minor intersecting fractures forming angular blocks
- Blocks: Poorly interlocked, heavily jointed rock mass with a mixture of angular and rounded blocks

Empirical & Analytical Design

- Geological data collection
  - Laboratory tests and in situ tests
    - Rock mass classification and characterisation
      - \( Q \) and RMR values, Joint properties, in situ stress etc.
    - Selection of excavation alternatives
    - Analytical methods
      - Block failure
    - Empirical methods
      - Stress failure
  - Excavation method selection and support design
  - Support installation and quality control
  - Monitoring of excavation and support behaviour

Martin (2002)