❖ BENCH FAILURE MODES

Before we enter this arena, let us first define our definition of bench scale.

First, a bench face is the vertical to intermediate dipping wall created in rock by excavation actions (Figure A1). This wall, or face, will also have the added component of a "bench". This bench, located at the base of the rock face, will be an area reserved for catching (restraining) rocks that detach from the excavated face. Thus the term bench face, or the face above the bench.

Bench scale is therefore any failure occurring within the confines of a bench face, i.e. having a height less than or equal to the excavated bench height. This is generally in the neighborhood of 3-30m.

Bench failure modes in rock are primarily, though not always, structurally controlled. This brief description will be restricted to such failures, ignoring rock mass, or "substance" failures.

A structurally controlled failure will here be defined as a rock block:

- defined on all sides with the exception of the bench face, by existing geologic structures;
- having at least one such geologic structure daylighting in the face such as to allow kinematic freedom under the action of gravity;
- sliding free of the slope on such a geologic structure allowing motion.

The bulk of most bench faces are controlled by two simple failure geometries. These are wedge and plane shear failure geometries. More complex failure modes include step-path and toppling. Only plane shear and wedge failures will be briefly described here, with step-path modes covered as an overview.

Plane shear geometries have been touted as being the simplest of the structurally controlled failure family. They are, in fact, a special case of the polyhedral wedge geometry family. In order for "plane shear" failure geometry to exist, and to have the potential for failure, it must have, at a minimum, one end release. If the plane shear plane is parallel to the slope then it requires, at a minimum, two end releases. Often, the "simple" plane shear as seen in the field will appear as in Figure A2. This failure geometry consists of the glide plane, two end releases, and a back release. While some assumptions can be made regarding the release mechanisms for plane shear failure, as will be shown later in this chapter, the prevailing assumption that such failures are two dimensional in nature is generally erroneous.

Wedge geometries, as mentioned above, are by geometric definition the simplest of the structurally controlled failure geometries. They can, and are generally assumed to, be composed of two planes (Figure A2). Wedge failures, too, have many variations. Wedge failure geometries

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exist that have a back release. Non-daylighting wedges that incorporate a plane shear glide plane are also found. The number of variations is for all practical purposes infinite.

One fact must be kept in mind however. The greater the number of planes involved in defining the failure geometry, the lesser the probability that the exact failure geometry will exist. Two plane and three plane geometries have a much greater likelihood of being created than one consisting of, for example, twenty four planes. Thus, the simpler the geometry, the greater the likelihood of existence.

- **Failure mode determination by structure orientation**

In order for one to analyze a specific failure mode, one must first define the geologic structures that contribute to the failure mode. For the purposes of this document, it will be assumed that structures striking within 20° of being parallel to the design bench face may contribute to plane shear failure (Figure A2).

Wedge failure analysis utilizes those structures found dipping out of the slope that are not included in plane shear analysis. In other words, structures at right angles to the slope to within 20° of being parallel to the slope (Figure A2).

- **Digital discontinuity length/density assignation**

Discontinuity generation and analysis can be conducted in closed form or utilizing Monte-Carlo techniques. Monte-Carlo simulation is the most intuitively obvious and will be described here.

For each joint set, a simulation, or generation, space is created for the bench. The shape most readily utilized is a rectangular solid enclosing the design bench (Figure A3). The dimensions of this generation space are a function of the bench size and the length (or diameter) of the discontinuity set to be modeled.

Joint centers are then randomly distributed within this generation space, governed by the joint center density value for the individual discontinuity set. A discontinuity diameter (assumed circular joints) and orientation is ascribed to each individual joint center.

Once this process is completed, the Monte-Carlo generated joints are passed through a filter. This filter determines whether the joint transects the bench (i.e. intersects the actual bench and the face) or not. If it does not, the probability of failure is assumed to be zero and it is discarded. Only those structures transecting the bench are saved. This "filter" is relatively simple to construct. It is a function of the bench height, the face angle, and the individual generated structure's characteristics.

This length/density assignation is repeated for each discontinuity set that has some member included in the plane shear "pie" on the stereonet (Figure A2).

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- **Failure probability**

We have numerically created a sample of structures that transect our design bench. Now, the task remains to determine the probability of failure for each of the structures. While it is possible to Monte-Carlo strengths for each of the bench intersecting structures generated in the previous sub-section, this is not the most efficient way to conduct the analysis.

First, we require a shear and normal stress on each of the discontinuity planes. For this, we require the unit weight of the lithologic unit in question (an average value is sufficient) and the geometry of the failure block.

As was discussed earlier, the effect of joint asperities is greatest at low normal loads. Normal loads in bench scale failures are very low. Thus, the apparent friction angle exhibited by bench face discontinuities will be higher than those determined from a normal lab testing program. This is especially true if a linear normal stress/shear stress diagram is utilized. Your bench angles may end up being too conservative unless the model is re-calibrated to work at extremely low normal stresses.

The author prefers to utilize a curvilinear model to represent normal and shear stress of tested and/or estimated discontinuity samples. A variety of curvilinear models are available. One of these is shown as Figure A6.

Parameters for this model can be easily developed from direct shear test data. Insure that testing is conducted in the stress range that is to be modeled.

- **Step-path failure**

A step-path, plane shear failure is a plane shear failure mechanism on a sliding plane defined by two or more discontinuities of differing orientations (primarily dip). This can be seen in Figure A4.

Generally, only two joint sets are analyzed in the context of step-path formation. However, this is not always the case as multiple joint sets can be combined to form a multi-set step-path. This determination depends on the designer, the evidence that exists in the field, and on stereonet interpretation, given the face orientation.

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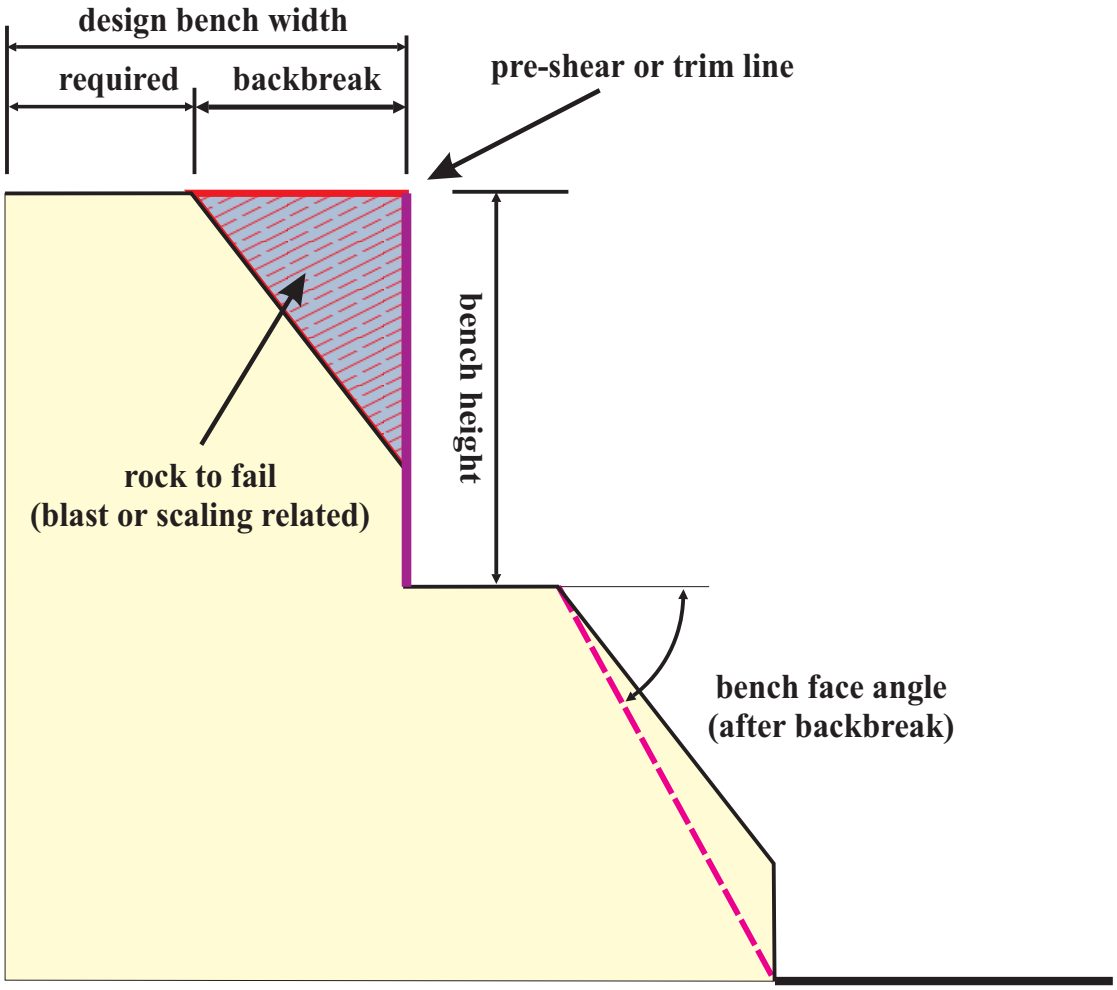


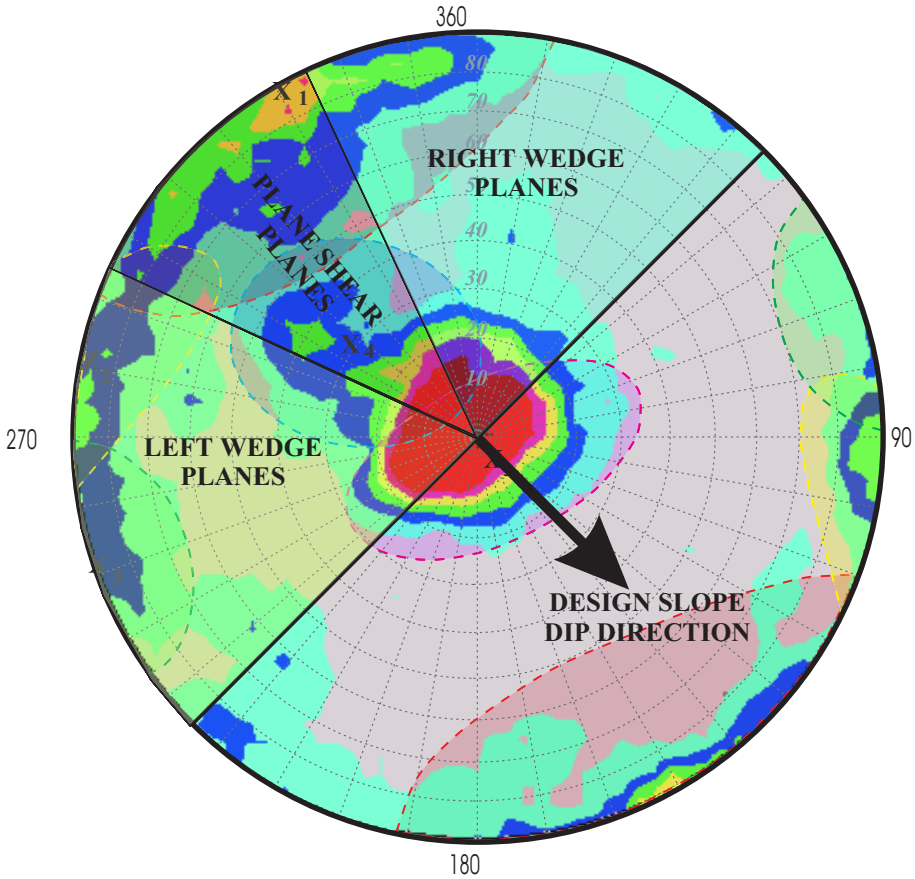
Catch bench

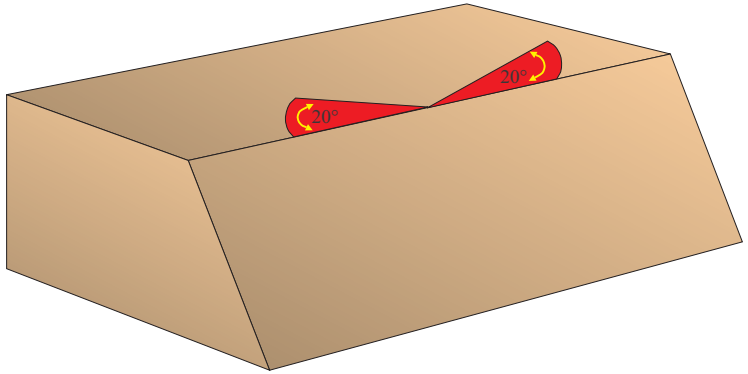
This is a cut bench on the rock slope. Its sole purpose is to "catch" rocks so that they do not continue unhindered to the toe of the slope. The bench is designed with a specific width relative to its height so that rocks will come to rest before falling off the next crest. In addition, a "backbreak" distance is incorporated into design as the bench crest will often fail from its (usual) vertical position. The design catch bench width is thus always wider than the required width for safety.

Bench face angle ( )

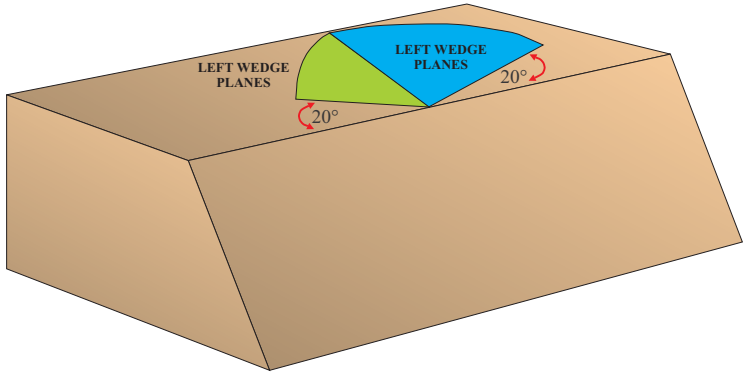
This is the angle from the toe of the bench to the crest. As backbreak often occurs, this angle will almost always be less than the blasted vertical bench face.

FIGURE A1





PLANE SHEAR FAILURE
dip direction within 20 degrees of face dip direction



WEDGE FAILURE
All structures dipping out of the face excluding those belonging to the plane shear set

SIMPLE FAILURE GEOMETRIES

FIGURE A2

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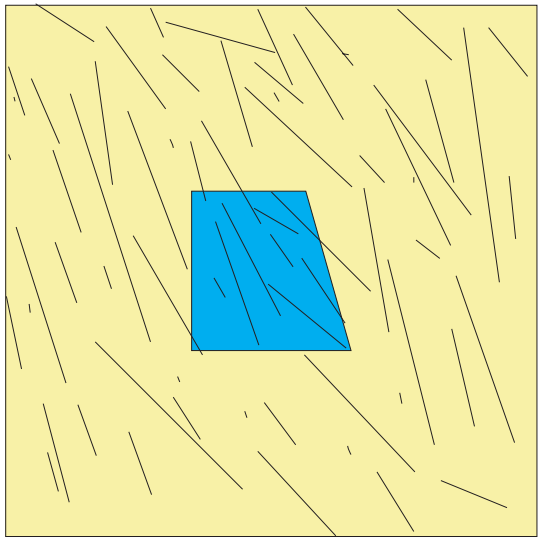
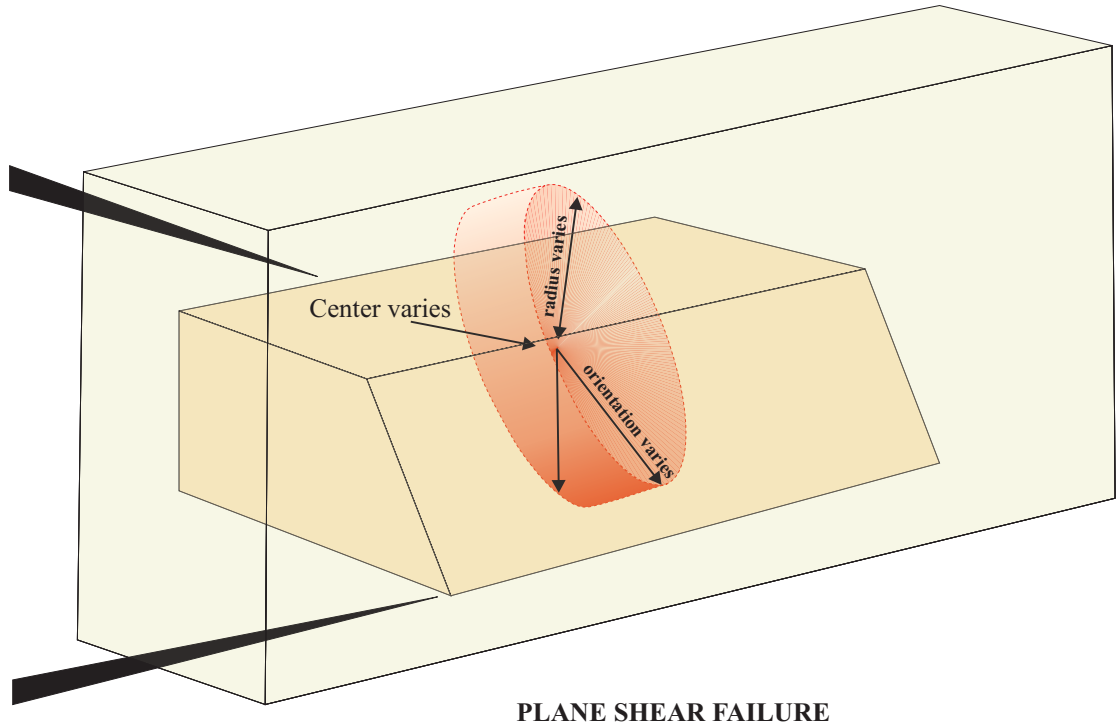


PLANE SHEAR FAILURE
Two dimensional section showing Monte-Carlo generated joint traces



PLANE SHEAR FAILURE
Three dimensional schematic of structure model

The minimum outer dimension of the bounding generation solid is:
Given P(D)>0.01,
X = 2*D+bench height
where: P(D) = probability of joint diameter exceedance,
X = dimension of the generation solid, height and width

Closed form, direct statistical, analysis can be conducted as an alternative to Monte-Carlo

SCHEMATIC MODELING PROCESS - PLANE SHEAR FAILURE

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FIGURE A3

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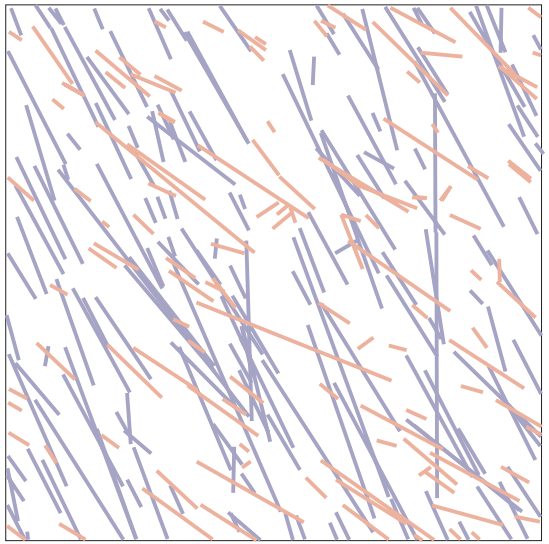
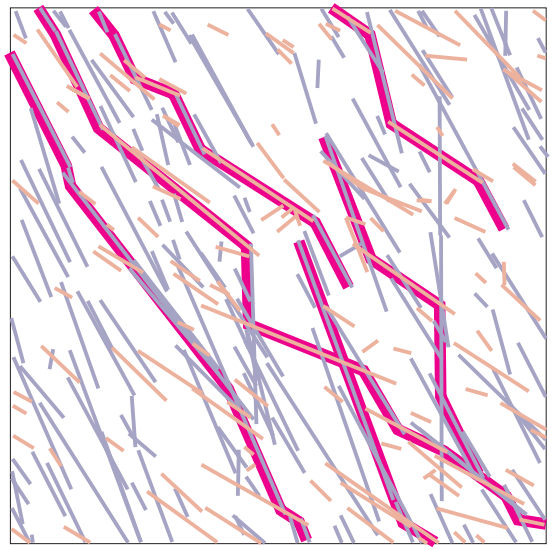
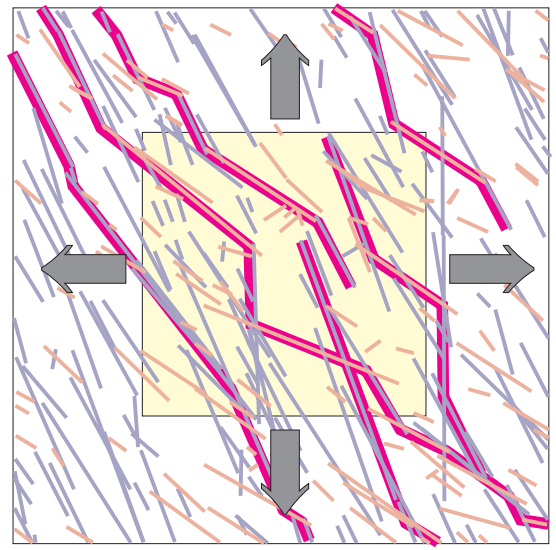


Generation of joint trace section



Identification of potential step paths



"Floating" bench for face angle design

This is the random endpoint sectional model of a rock mass (sometimes called the "pick-up-sticks model"). Discontinuities are generated in space based upon mapped spatial characteristics. Endpoints are allowed to fall wherever the statistical generation process allows. Step-paths are then identified through the process of searching for intersecting discontinuities. In one form of analysis, a sectional bench is "floated" across the step path model. If a step path intersects the crest and the face, then the stability of the step-path is evaluated. This is best done in a closed form, allowing a probability of the failure to be calculated.

Note: not all step-paths were identified on the shown sections. This is for illustration only.

MONTE-CARLO STEP-PATH CREATION

FIGURE A4

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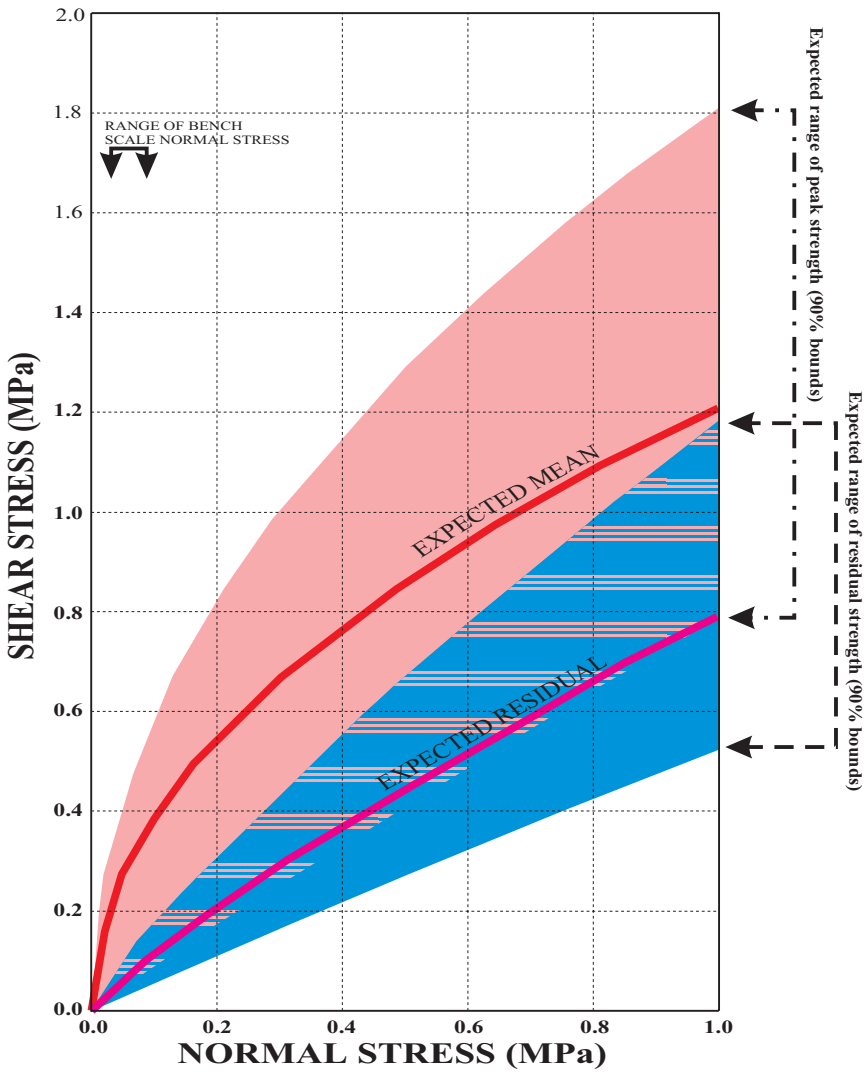

The adjacent diagram was determined from Barton and Choubery's relationship for the peak shear strength of natural joints. The estimated shear strength range has been established by a computer program evaluating the range of strength parameters for the equation below. Residual strength calculation assumed a reduction in roughness and contact area.

All of the values below have been estimated. They are, however, reasonable numbers for a rock mass of the type examined.

$$\tau=\sigma_n \tan\left(JRC \log_{10}\left(JCS/\sigma_n\right)+\phi_b\right)$$

σ_n = Normal stress (range from 0 to 2.0 MPa)

JRC = Joint roughness coefficient (10; standard deviation of 3)

JCS = Joint compressive strength (80 MPa; standard deviation of 30)

ϕ_b = Base friction angle (32 degrees; standard deviation of 3 degrees)

τ = Expected shear resistance (MPa)

A probabilistic model for discontinuity shear strengths is utilized for analysis. This can be utilized on an individual structure or in closed form on a family of structures. It incorporates the range of strength values to be encountered instead of utilizing a single, fixed value.

FIGURE A5