Use of Nonlinear Strength Criteria in Stability Analyses of Bridge Foundation on Jointed Rock

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During construction of a bridge project, it became important to check the stability of one of the bridge pier foundations. The spread-footing foundation was located at the crest of a 60-ft-deep bedrock channel formed by a series of steeply dipping joints. The overall slope of the channel face was about 50 degrees, and rock mass conditions exposed during construction prompted concerns about potential instability of the channel wall beneath the footing. As a consequence, the foundation was modified: a keyway was excavated beneath the footing and a series of long rock bolts designed to guard against possible sliding of the block of rock was installed beneath the footing. At the request of the owner, independent investigations and analyses were undertaken to evaluate the stability of the modified foundation. The approach used for the stability evaluation was first to assess which mechanisms of failure could be considered kinematically feasible on the basis of field investigations of geological conditions. Then, for each potential failure mechanism, appropriate strength parameters were assigned to the sliding surfaces involved. Finally, the stability of the system against each postulated failure mechanism was evaluated. In this process, the nonlinear Barton criterion for shear strength of a rough discontinuity was applied to a potential block sliding mechanism, and the nonlinear Hoek-Brown criterion for the strength of a pervasively jointed rock mass was applied to a potential circular failure surface passing beneath the footing. In the latter case, stability analysis methods developed by Bishop and Sarma were used, because the latter method allows specific geological structural features to be incorporated in the body of the slide mass with shear strength parameters that differ from those along the basal plane of sliding. This approach is considered to more closely reflect the actual conditions often present in a jointed rock mass. The methods used and the results obtained from the stability evaluation are summarized, with particular reference to the selection of Barton and Hoek-Brown shear strength parameters.

The structure considered in this paper is a bridge crossing of a 200-ft wide navigation channel. The bridge crosses the east-west channel in a north-south direction and has three interior piers in addition to its two end abutments (Figure 1). The navigation channel passes between the two most southerly interior piers, which are separated by a 390-ft span. The southern bank of the channel is formed by a steep bedrock face that rises to about 60 ft above the base of the channel (Figure 2). The lower 30 ft of this face are nearly vertical; it is separated by a 20-ft-wide bench from the upper 30 ft, which slope at 65 degrees to the north. Overall, the channel bank slopes at about 50 degrees to the north.

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The pier resting above the southern bank of the channel was designed to carry a total dead-plus-live load of 11,300 kips. The earthquake conditions assumed by the designers resulted in a lateral seismic loading equivalent to 0.3 times gravity. The foundation for this bridge pier was originally designed as a 34- × 46-ft spread footing bearing on a prepared horizontal rock surface. To develop the bearing surface, a rectangular box cut or sinking cut was planned; the depth of the cut varied from 12 ft at the northwest corner of the footing to 32 ft at the southeast corner because of the ground surface topography. During excavation of the cut, the contractor had some difficulties in maintaining this enclosed shape, and the front or northern face of the cut was lost. The resulting excavation was a bench with a flat floor at elevation 18 [elevations are given as feet above mean sea level (MSL)]. Inspection of the rock conditions exposed during this early stage of construction apparently aroused concern about the stability of the foundation. As a result, it was decided to strengthen the foundation area by cutting an 8-ft-deep "keyway" into the base of the excavation, installing thirty-one 150-kip-capacity rock bolts of various lengths and orientations and ultimately backfilling with reinforced concrete to the top of the footing at elevation 28. The final foundation geometry is shown in Figure 2.

In light of these experiences during construction and the various modifications made as a result, the owner wished to verify that the stability of the foundation was acceptable, and independent consultants were retained to do so. The geological conditions beneath the footing were investigated to identify what potential failure mechanisms, if any, could be considered kinematically feasible. On the basis of the investigation results, the relevant engineering properties of the foundation rock mass were assessed with particular regard to the shear strength available along significant preexisting discontinuities and the shear strength parameters governing the overall behavior of the jointed rock mass. These data were used as input to a series of analyses to evaluate the stability of the foundation under static and earthquake conditions, both with and without consideration of the rock bolts that had been installed.

SUMMARY OF POTENTIAL FAILURE MECHANISMS

For failure to occur through the rock mass underlying the footing, a failure mechanism that is kinematically possible

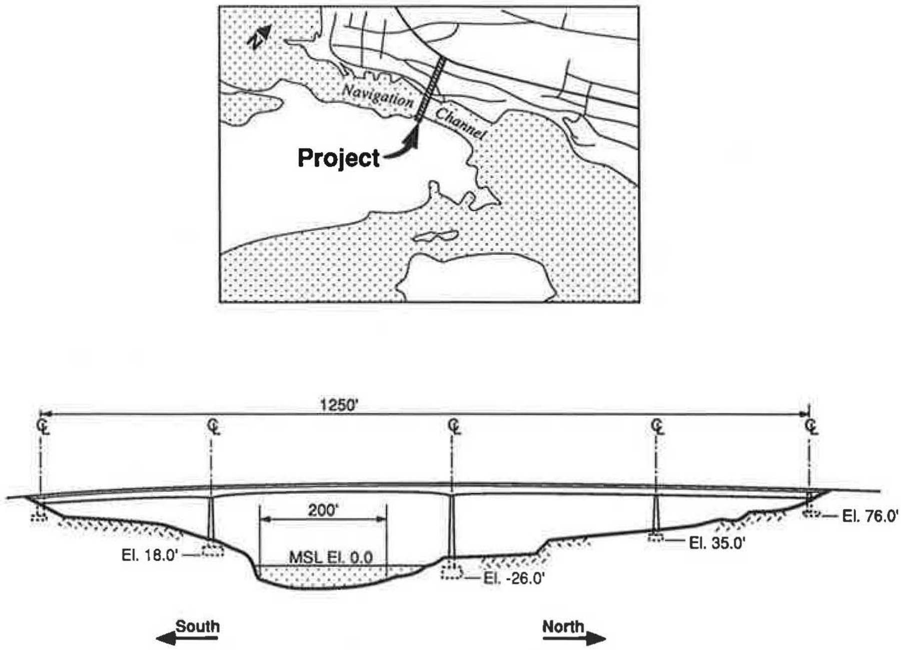

FIGURE 1 Project layout.

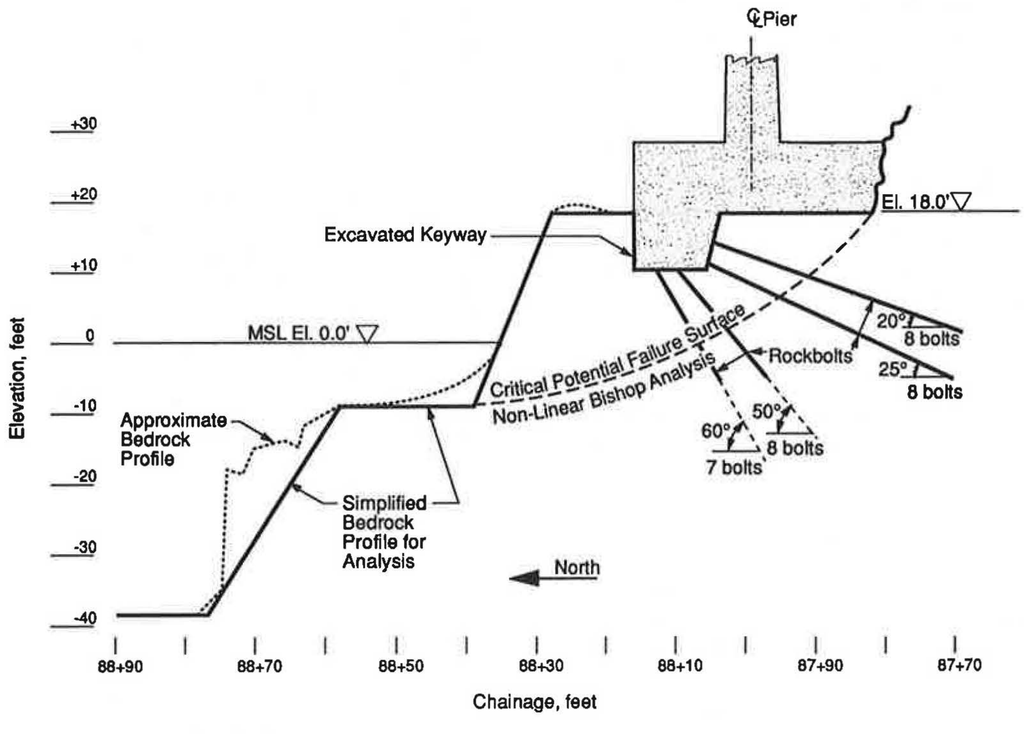

FIGURE 2 Pier foundation.

must be formed or it must preexist. Typical potential failure mechanisms have been presented by Hoek and Bray (*1*, p. 358), and the mechanisms relevant to the footing under consideration are shown in Figure 3. In general, there are two classes of such mechanisms: those that are controlled by preexisting discontinuities within the rock mass, or so-called structurally controlled failures, and those that require a new failure surface to form at some critical location, passing through the assemblage of jointed, blocky material that constitutes the overall rock mass; these are called general or overall failure mechanisms.

Structurally controlled mechanisms are governed by preexisting discontinuities within the rock mass, such as joints, faults, shear planes, or bedding planes, and the geometry of the potential sliding mass will be defined by these bounding planes. The stability of the mass will then be controlled by

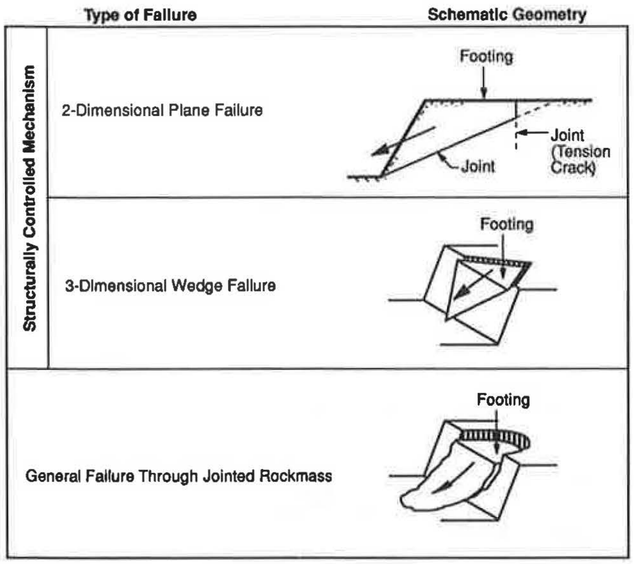

FIGURE 3 Potential failure mechanisms.

the shear strength that can be mobilized along these planes in comparison with the driving forces involved. For the foundation in question, two types of structurally controlled failure mechanism were potentially significant to the overall stability of the footing, these being a simple two-dimensional block sliding or planar failure mechanism and a three-dimensional wedge failure mechanism. In both cases, the potential for a kinematically feasible failure mechanism to form depends on whether the necessary bounding planes exist in the rock mass, that is, whether a failure geometry can be formed.

Thus, a primary objective of the geological investigation was to assess whether subsurface conditions existed giving rise to kinematically feasible, structurally controlled failure mechanisms, of either the planar or the wedge type, that might jeopardize the safety of the foundation. Hoek and Bray (*1*) have summarized the geometric conditions that must be met for these failure mechanisms to be kinematically possible, and these conditions were used as guidelines in assessing the potential for planar- or wedge-type failures to develop. For plane failure,

- The strike of the basal sliding plane must be within about ±20 degrees of the strike of the slope face;
- The dip of the base plane must be less than the dip of the slope face for the potential failure plane to daylight;
- Release planes must be present at the two ends of the potential failure mass;
- The dip of the base plane generally must be greater than the friction angle along the plane, in the absence of other disturbing forces; and
- The existence of a water-filled tension crack or joint forming a backscarp may significantly reduce stability.

For wedge failure,

- Two intersecting joint planes must exist, having a line of intersection plunging at less than the dip of the slope face in order to daylight;
- The direction of the line of intersection must be such that there is a plunge component out of the slope that meets the above criterion; and
- Truncation of the apex of the wedge by a water-filled joint or tension crack will reduce stability.

If the geometric conditions for planar sliding or wedge mechanisms are satisfied, then the potential for failure depends on the shear strength available along the bounding planes compared with the driving forces involved. Many authors have discussed the determination of reasonable shear strength parameters for discontinuities in rock, and these discussions are summarized in Hoek and Bray (*1*). For practical field applications, the relationship proposed by Barton (*2*) between the shear strength of a rough joint and the normal stress acting across the joint is particularly useful, and this approach was used for these investigations. Further detail is given later, in the discussion of engineering properties of the rock mass.

Besides the potential for structurally controlled failure mechanisms as noted, a general rotational failure surface may be formed through a rock mass that is closely or ubiquitously jointed relative to the scale of the foundation (Figure 3). In this situation, a failure surface may form anywhere within the rock mass, constrained only by the overall shear strength properties of the mass and the loads applied to it. In general the rock mass will comprise more or less interlocking blocks of essentially intact rock material separated by discontinuities, and the shear strength envelope of such a rock mass will generally be nonlinear because of the effects of dilation at the low normal stresses that commonly prevail in slope stability problems. For this reason, it was decided that the nonlinear strength envelope or failure criterion proposed by Hoek and Brown (*3*, p. 527) was most appropriate for application to any area of the foundation rock mass through which a general failure surface might develop because of its blocky, jointed nature. However, the problems of assessing the appropriate values of the strength parameters for such rock masses on the basis of theoretical or laboratory work are formidable. In recognition of this problem, a suggested method by which reasonable estimates of the strength of jointed rock masses can be made was the subject of Hoek's 1983 Rankine lecture (*4*). The method relies on characterizing the rock mass according to its lithology (i.e., the rock type) and its overall quality using the well-known Bieniawski (*5*) or Barton (*6*) classification systems. This characterization of the rock mass is then used to evaluate the necessary Hoek-Brown strength parameters. Further details are noted in the discussion of the engineering properties of the rock mass.

- Is it reasonable to assume that there are existing discontinuities in the foundation rock mass that could, on the scale of the footing, form the base plane for a two-dimensional planar sliding failure?
- Are there existing discontinuities in the rock mass that could, on the scale of the footing, fulfill the geometric requirements for a potential wedge failure?
- For either of those two cases, what geometric and strength parameters are reasonable to assign to the planes?

- What is the lithology and quality of the rock mass in the foundation, and do these factors vary for different domains across the footing area?
- What strength parameters are reasonable to assign to the jointed rock mass?

SUMMARY OF GEOLOGICAL CONDITIONS

When this stability evaluation was requested, construction had already progressed to the point that the footing concrete had been poured and crushed rock backfill placed over much of the site (Figure 4). Therefore, direct mapping of the rock mass immediately beneath the footing was no longer possible, and geologic description of the site conditions was accomplished through literature review, analysis of air photographs, detailed geologic mapping of adjacent areas, and the drilling of three coreholes through the footing into the underlying foundation rock mass. The relevant findings are briefly summarized in the following.

Geologic Units

The bridge pier under consideration is founded on an assemblage of sedimentary rocks thought to be weakly metamorphosed. On the basis of visual examination, these rocks were classified as interbedded metagreywackes and argillites. The metagreywacke is dark gray, with a grain size varying from fine sand to silt. It is classified mechanically as weak to medium-strong rock, according to International Society for Rock Mechanics standards. The grayish-black argillite usually occurs as interbeds of less than 4 in. thick. Slaty cleavage is developed locally, and in some cases "slate" may be a more appropriate rock name. The argillite is generally weaker than the metagreywacke, so joints, healed fractures, and drill breaks are frequently associated with the argillite interbeds.

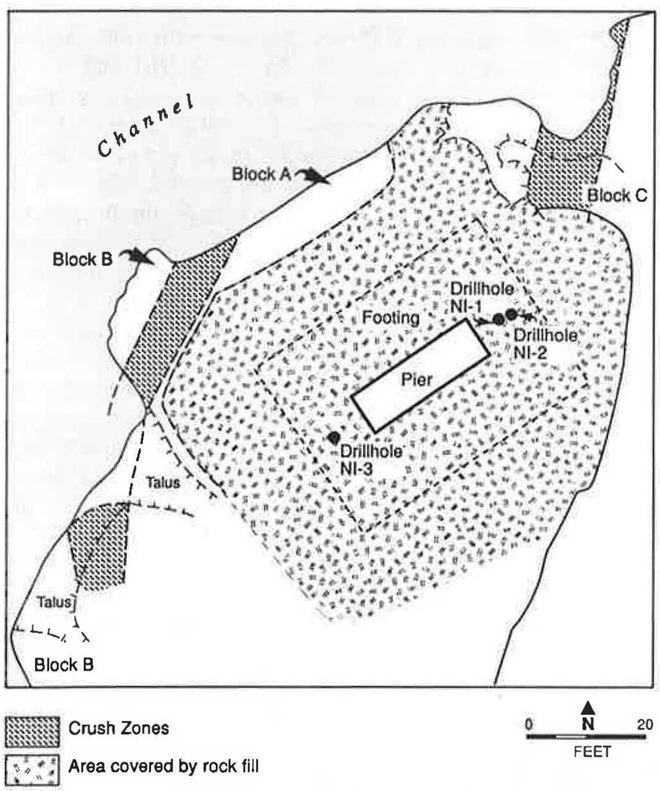

FIGURE 4 Rock mass domains in foundation area.

Rock Mass Domains

The rock mass near the pier footing was divided into three domains, designated Blocks A, B, and C, separated by crush zones or shear zones (Figure 4). The largest shear zone at the site lies to the east of Block A, separating it from Block C. As shown in Figure 4, this major crush zone appears in outcrop near the northeast corner of the footing and trends toward the pier location. On the basis of inference from the available outcrops and data from corehole drilling, it is probable that Block A rocks underlie approximately the western half of the footing area and shear zone or crush zone rocks underlie the eastern half. Projections of the trend of the major shear zone suggest that this proportion is likely, but the extent of the shear zone beneath the footing could not be located precisely because of masking by the coarse rockfill that had been placed over the footing. However, the overall width of the shear zone beneath the footing is constrained by the evidence from Borehole NI-3 to the west of the pier, which penetrates rocks interpreted as belonging to Block A and is clearly not within the much more fractured rocks of the shear zone.

The shear zone trends at an angle of about 45 degrees across the foundation and, within the shear zone itself, closely spaced vertical joints strike in a direction parallel to the overall trend of the zone. The rock within the shear zone comprises closely interlocked, vertically oriented slabs of hard, intact material, such that a significant amount of breakage of intact rock material would be required for any failure plane to cut across the grain of this shear zone. On the basis of geological evidence, the shear zone was not considered to represent a currently active fault.

The domain designated as Block A underlies at least the western part of the foundation. This block has consistent bedding with a strike of N23°E and a dip of 65 degrees south. The major slope face behind the footing and the channel face itself both have a strike of approximately N55°E and a dip of 55 degrees north, reflecting the presence of a general set of throughgoing, planar joints. Cross joints appear at intervals from 6 in. to 3 ft and are also planar but have a relatively rough surface. Persistence of the cross joints varies from 1 to 8 ft, and they tend to die out or be offset as they cross other joints or interbeds. The rock materials within Block A have considerable mechanical strength, as indicated by point-load tests that give strengths of about 20,000 psi. Jointing frequency in Block A is relatively low and rock quality designations (RQDs) of 100 percent were common in many of the core runs, with an average RQD value of 89 percent in Hole NI-3.

Structurally Controlled Potential Failure Mechanisms

Discontinuity orientation data were collected in areas adjacent to the bridge pier, and it was determined that these data were fully consistent with information on the regional structural geology. These data indicated that four discontinuity sets besides the previously noted shear zones were present in the general vicinity of the pier, as summarized below:

Discontinuity Set	*Strike/Dip*	*Description*
1	N55°E/55°N	Major joint set at site; forms channel wall
2	N30°W/75°N	Small number of joints in Block C
3	N20°E/65°S	Bedding in Block A
4	N65°E/70°S	Minor Block C joints
Crush (shear) zones	N15°E/90°	From few inches to several feet wide

These data were analyzed using stereographic projection to determine which failure mechanisms were kinematically feasible, as shown in Figure 5. This analysis indicated that although the steeply dipping Set 1 joints, lying parallel to the channel wall, were available to form the tension crack or backplane needed to release a planar failure, the necessary base sliding plane was absent from the data. In general, significant discontinuities with a northerly dip that could daylight out of the channel face were completely absent, not only from the immediate vicinity of the bridge pier but also from a much wider surrounding area. These surface observations were corroborated by the data from the carefully conducted drilling, which used triple-tube core barrel techniques to ensure full core recovery. No evidence was found of continuous planes that intersected the vertical coreholes at dip angles that could allow them to act as potential basal slide planes. The lack of any evidence of the existence of a potential basal slide plane effectively ruled out plane failure or two-dimensional block sliding as a mechanism that could jeopardize overall foundation stability. However, small, localized failures could still occur in the thin zone of rock lying between the north face of the footing and the channel wall, because of local variations in the dip of the Set 1 joints that directly control the shape of the channel wall.

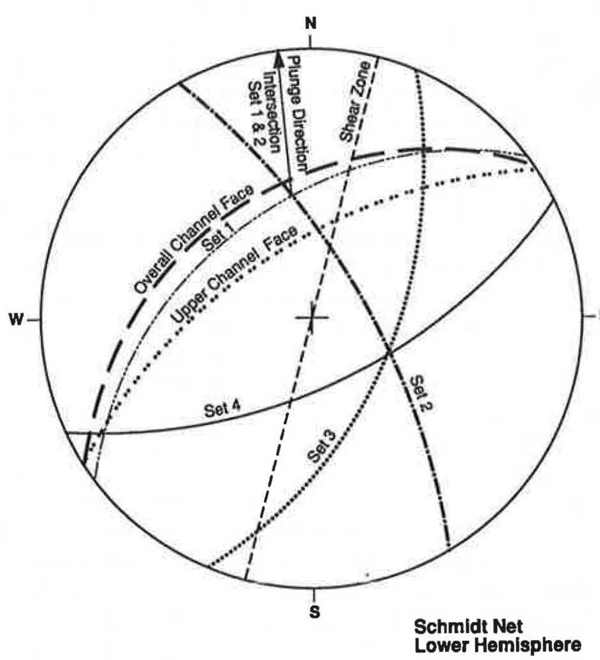

FIGURE 5 Summary of discontinuity orientations.

For wedge failures to be kinematically feasible and of significant concern, appropriate joint orientations must exist with respect to the free face (channel face), and the joints must be of sufficient persistence to allow sliding. As shown in Figure 5, only Joint Sets 1 and 2 could combine to form wedges that meet the necessary geometric criteria for potential failure. However, the Set 2 joints were mapped only within Block C, to the east of the actual footing area; some wedge failures were in fact evident in this Block C area, but they were of very minor extent, because the plunge of the line of intersection of the wedges is within a few degrees of the dip angle of the free face (Figure 5). On the basis of these factors, wedge failure was not considered to be a credible failure mode of significance with respect to the stability of the pier foundation.

General Rotational Failure

The development of a general slide plane through the foundation rock mass, along which an overall rotational failure could occur, depends on the presence of materials that are weak compared with the loads applied to them. In this regard, intensely jointed rocks are conceptually similar to granular soils with a very large grain size. Rock in the shear zone that underlies about half of the footing is the weakest material in the area of interest because of the intense jointing associated with the shearing to which the rock has been subjected. Within the major shear zone penetrated by Holes NI-1 and NI-2, fractures were so numerous that it was not possible to separate drill breaks from natural fractures. Naturally, the vertical orientation of the drillholes combined with the vertical attitude of the jointing within the shear zone, and therefore of the slabs or pods of intact rock, tended to exaggerate this effect, giving rise to particularly low RQD values. Nevertheless, the formation of a general slide surface in this material, not constrained by specific preexisting discontinuities, must be evaluated as a potentially credible failure mechanism. In undertaking this evaluation, it is important to keep in mind that the relative attitudes of the channel face, the shear zone, and the rock fabric within the shear zone are such that any new rotational sliding plane of failure that formed beneath the footing would be forced to cut across the grain of the intact rock fragments within the shear zone, requiring fracture of these intact rock materials.

In summary

- About half of the footing area (the western half) is founded on a block of relatively fresh, strong, interbedded greywackes and argillites (Block A) that has a fairly low joint frequency indicated by an average RQD of about 90 percent.
- Within Block A rocks, the steep channel face is controlled by a major joint set (Set 1) that could give rise to localized plane failures in front of the footing because of local variations in dip of the joints. No other planar failure or wedge failure mechanisms of significance were identified in Block A rocks.

- About half the footing area (the eastern half) is underlain by a major vertically oriented shear zone that strikes across the foundation at an angle of about 45 degrees.
- Within the shear zone, the closely spaced vertical jointing results in a fabric of vertically oriented interlocking pods or slivers of hard, intact rock material.
- Although development of a general surface of sliding across the shear zone rocks would require some fracturing of intact rock material, this potential failure mechanism must be further evaluated to determine if it threatens the stability of the foundation.

ENGINEERING PROPERTIES OF ROCK MASS

As noted, there are two geologically credible failure mechanisms that could affect the pier foundation. First, there is the possibility of local planar or block sliding occurring in front of the footing, along one of the preexisting Set 1 joint planes. Second, there is the potential for a general failure surface to develop through the rocks of the major shear zone that cuts across the foundation. In the first case, it is necessary to assign some appropriate shear strength parameters to the specific discontinuities on which sliding could occur. In the second case, it is necessary to evaluate the shear strength parameters for the overall rock mass material within the broad shear zone or crush zone beneath the footing.

In both cases, the shear strength envelopes of the rock materials are known to be nonlinear as a function of normal stress due to the effects of dilation, or volume increase that occurs as two rough surfaces are sheared past each other. Various investigators have proposed nonlinear strength envelopes for rock material in order to capture this effect (*1*–*3*), because ignoring this nonlinearity can result in seriously underestimating the shear strength of the material, particularly at the fairly low normal stress levels common in slope stability problems. However, many of the widely available slope stability analysis programs have been based on the use of linear strength envelopes for the materials, and it is therefore sometimes necessary or desirable to use equivalent linear strength parameters for these decidedly nonlinear materials. One way to do this is to first develop the nonlinear strength envelope that appears most appropriate for the material and then evaluate the approximate normal stress level that will be acting in the material and use this information to determine the slope angle (ϕ) and the cohesion intercept (c) of the tangent to the failure envelope at this specified level of normal stress. These parameters can be used as equivalent linear shear strength parameters, applicable to cases in which the normal stresses do not deviate too markedly from those assumed. Sometimes it may be more appropriate to use the slope of the secant to the nonlinear strength envelope at the normal stress level of interest. In this case the cohesion intercept would be zero and the equivalent linear shear strength would be defined by the single parameter of the overall friction angle (ϕ).

Shear Strength of Discontinuities

As previously summarized, the only significant discontinuities that are present in the rock mass and that could contribute to the formation of a structurally controlled failure in the foundation area are the joints that lie subparallel to the face of the channel and actually control the geometry of this face. These joints, referred to as Set 1, strike at about N55°E and dip generally between 50 and 70 degrees to the north. Such joints could form the base plane for a localized failure near the face of the channel; they could also form a tension crack or backscarp for a slide mass. In addition, these joints could form the interior slice boundaries within a general rotational slide mass. In each case, the shear strength available along these joints must be evaluated before meaningful stability analyses can be conducted.

To estimate the shear strength of cohesionless joint surfaces, the following relationship proposed by Barton (*2*) is particularly useful.

$$\tau = \sigma' \tan[\phi_b + \mathrm{JRC} \log_{10}(\mathrm{JCS}/\sigma')] \tag{1}$$

where

τ = shear strength,
σ' = effective normal stress across joint,
ϕ_b = basic friction angle of a planar discontinuity in the type of rock under consideration,
JRC = joint roughness coefficient, measured against standard profiles published by Barton and ranging in value from 5 for a smooth surface to 20 for a rough, undulating surface, and
JCS = joint wall compressive strength, which equals the uniaxial strength of the intact rock material for clean, unweathered joints.

On the basis of field investigations and index testing, such as point-load testing to assess comprehensive strength, the following parameters were selected for use in subsequent stability analyses:

- ϕ_b: The rock material is predominantly greywacke. On the basis of data published by Barton (*2*) and by Martin and Miller (*7*) on joint surfaces in moderately weathered greywacke, a value of ϕ_b between 25 and 30 degrees was considered appropriate.
- JRC: On the scale of the exposed joint faces visible in the field, to a maximum of 10 ft of continuity, values of JRC varied from 4 to 10 in a comparison with standard roughness profiles (*1*). However, on the scale of any failure mass large enough to significantly affect the integrity of the footing, the effect of undulations along the plane of the joint face must also be considered. Thus, a minimum value of 7 was considered to be conservative but reasonable.
- JCS: For the discontinuities under consideration, the wall rocks of the joints consist of unweathered to slightly weathered materials ranging in composition from siltstone to fine-grained sandstone. Point-load test results indicated uniaxial compressive strengths of the rock material in the order of 20,000 psi. For stability analyses a value of 7,500 psi was used to ensure that the results were conservative but realistic.

The normal stress acting across any specific joint plane (σ') will depend on the particular geometry involved, the loading conditions, and the joint water conditions or degree of saturation of the rock mass. For the conditions being considered, the effective normal stresses lay in the range from 0 to 3,000 pounds/ft^2 (psf). On the basis of the parameters for Barton's Equation 1, Figure 6 shows a shear strength plot or Mohr envelope for these Set 1 joints. Note that this curvilinear envelope is only valid for cohesionless joints and does not apply to those conditions in which failure is forced to occur through bridges of intact rock. For such situations an additional allowance must be made for a cohesive component of the joint shear strength. As shown in Figure 6, the shear strength envelope is not very strongly curved in the region from $\sigma' = 0$ to 3,000 psf. At an effective normal stress level of $\sigma' = 1{,}000$ psf, the shear strength may be reasonably represented by an equivalent overall friction angle of $\phi = 50$ degrees and a cohesion of $c = 0$, which is the slope of the secant to the envelope at this normal stress level. This overall equivalent friction angle can then be used directly in stability analyses based on the well-known linear Mohr-Coulomb relationship

$$\tau = c + \sigma' \tan \phi \tag{2}$$

Shear Strength of Jointed Rock

As noted previously, there is a major shear zone or crush zone passing beneath the eastern half of the footing. Within this zone is a structure of vertically oriented interlocking angular fragments of generally unweathered greywacke and argillite. The overall attitude of the zone and of the long axes of the blocks of intact rock within the zone is close to vertical in dip, with the zone striking obliquely across the footing at an angle of about 45 degrees.

To provide a conservative but reasonable assessment of the stability of the foundation, it was assumed for analysis purposes that the entire rock mass beneath the footing consisted of one large shear zone. The logic behind this approach was that such a hypothesis would reflect the worst-case scenario that could reasonably be postulated within the constraints imposed by the geological framework of the site. If the calculated foundation stability were found to be acceptable under these postulated conditions, then there would be considerable confidence in the satisfactory performance of the actual foundation rock mass, which was predicted to comprise about equal proportions of Block A rock and shear zone rock.

To proceed with stability analyses incorporating general surfaces sliding through the crush zone or shear zone rocks underlying the footing, estimates were required for the shear strength of this jointed mass. Methods by which such estimates can be made have been outlined by Hoek (*4*). For this purpose, Hoek and Brown's empirical relationship between the major and minor principal stresses acting on an element of rock at failure is used (*3*):

$$\sigma_1 = \sigma_3 + (m\sigma_c\sigma_3 + s\sigma_c^2)^{1/2} \tag{3}$$

where

σ_1 = major principal stress at failure,
σ_3 = minor principal stress at failure,
σ_c = uniaxial compressive strength of intact rock particles within the jointed mass, and
m, s = empirical constants.

To evaluate the empirical constants m and s, the material is first classified according to its lithologic origin and then according to the overall quality of the rock mass (*4*). To characterize the overall quality of the rock mass, the widely known rock mass classification systems proposed by Barton (*6*) and Bieniawski (*5*) are used. In addition, in later publications Hoek (*8*) has considered whether or not the interlock of the blocks within the rock mass has been retained ("undisturbed rock mass") or lost ("disturbed rock mass"). Following these procedures in a conservative manner, the rock

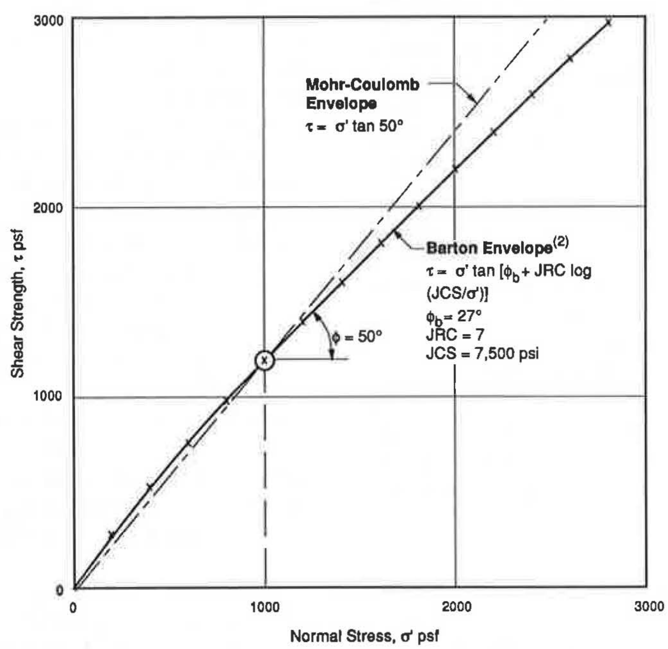

FIGURE 6 Estimated shear strength of Set 1 joints.

mass within the crush zone was considered to be disturbed, characterized as lithified argillaceous to arenaceous rocks of generally fair quality, locally ranging from good to poor quality, equivalent to a Rock Mass Rating of 25 to 50 (*5*). The unconfined strength of intact rock fragments is 7,500 psi.

For a rock mass so characterized, an appropriately conservative value of the empirical Hoek-Brown parameter m was selected as $m = 0.13$. The parameter s is generally considered to represent the degree of brokenness of the rock mass. The degree to which the shear zone rock mass should be considered as broken depends on the amount of intact material that would have to be sheared through in developing a sliding surface that cuts across the grain of the shear zone, as previously discussed. Therefore, it was decided to investigate the effect on stability of a range of values for the parameter s, from $s = 0.00005$ (more broken) to $s = 0.002$ (less broken).

On the basis of mathematical relationships (*4*), the Hoek-Brown failure criterion, which is expressed in terms of principal stresses, can be used to calculate envelopes of available shear strength (Mohr envelopes). Figure 7 shows the rock mass shear strength envelopes for the selected values of the Hoek-Brown parameters noted earlier. It is evident from the envelopes plotted on Figure 7 that the degree of brokenness of the rock mass, as expressed by the value of the parameter s, has a significant effect on the shear strength that can be mobilized along a potential general failure plane that passes through the rock mass. For the rock mass beneath the bridge pier, the fact that any general failure beneath the footing would be forced to cut across the structure of the rock within the shear zone is significant in this regard.

As shown schematically in Figure 7, at any selected point on the strength envelope defined by a specific normal stress the available shear strength may be defined by the values of instantaneous cohesion (c_i) and instantaneous friction (ϕ_i) determined from the cohesion intercept and slope of the tangent to the envelope at that value of normal stress.

PIER FOUNDATION STABILITY ANALYSES

The geometry of the rock profile beneath the bridge pier is shown in Figure 2. As indicated, a slightly simplified straight-line bedrock profile was used for analysis. A bearing pressure of 8 kips/ft^2 was assumed for the foundation loading, equivalent to a total vertical load of 12,500 kips on the 34- × 46-ft footing. This is slightly conservative, because the actual design

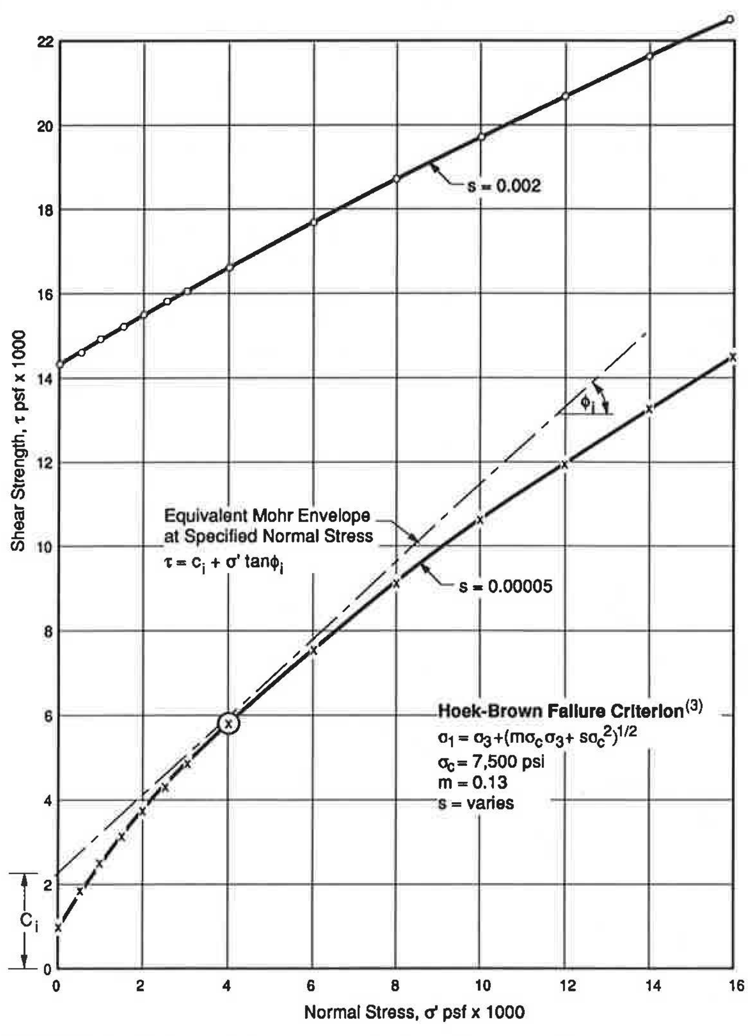

FIGURE 7 Estimated shear strength of crush zone rocks.

load is 11,300 kips. For earthquake conditions, a lateral acceleration of 0.3 times gravity was applied to the total vertical load on the foundation (12,500 kips), giving a lateral seismic load of 3,750 kips. For the rock itself, a lateral pseudostatic earthquake load was applied to the potential slide mass under consideration in each analysis, equal to 0.3 times the weight of the slide mass.

It is important to note that the simple pseudostatic approach that has been followed to assess the stability of the foundation under earthquake conditions is generally considered to be conservative (*9,10*). This method imposes a horizontal force of constant magnitude and direction on the foundation, whereas in reality this peak force is imposed only momentarily before decreasing and then reversing in direction. Although the method is conservative, it is nevertheless useful—if slopes can be shown to be stable under these assumed pseudostatic forces, then considerable confidence can be expressed in their actual performance under earthquake conditions. Seed (*9*) has suggested that if a factor of safety of about 1.15 against slope failure is obtained when using seismic acceleration coefficients of 0.1 *g* for magnitude $6^{1/2}$ earthquakes or 0.15 *g* for magnitude $8^{1/4}$ earthquakes, this should be sufficient "to ensure that displacements will be acceptably small." To be conservative, it was decided that this minimum factor of safety of 1.15 would be desirable, even though the seismic acceleration coefficient of 0.3 *g* that was used for analysis represented an acceleration of two to three times the values discussed by Seed.

As indicated in Figure 2, thirty-one 150-kip-capacity rock bolts had been installed in a staggered pattern from within the keyway excavation beneath the footing. Because these bolts were grouted, they act as a stiff, fully bonded system and the full yield capacity of the bolts will be mobilized with very small lateral displacement of any potential slide mass within the foundation, perhaps on the order of a few tenths of an inch. At yield, the bolts represent a total load capacity of 4,650 kips, or 101 kips/longitudinal-ft of the foundation, angled into the rock mass at inclinations varying from 20 to 60 degrees below the horizontal.

In general, all analyses were conducted assuming that the rock mass is fully saturated, clearly a conservative assumption under normal conditions. Unfortunately, however, a vertical grout curtain was apparently installed between the footing and the channel wall, and this curtain will tend to inhibit drainage and dissipation of joint water pressures within the foundation rock mass. Nevertheless, it is judged that the presence of the intense vertical jointing within the shear zone beneath the footing will permit some drainage of the rock mass, and any degree of drainage will make the foundation more stable than the analyses' assumption of full saturation.

Plane Failure

As previously noted, no geometric conditions were found that could give rise to a large-scale structurally controlled plane failure within the foundation rock beneath the footing. However, conditions may exist for a more localized potential failure to develop in front of the footing, as shown in Figure 8. The backscarp of such a failure would be defined by an existing Set 1 joint located along the face of the footing, with the base plane formed by a stepped series of Set 1 joints. However, because the dip of the Set 1 joints is generally greater than 50 degrees, formation of the flatter-lying stepped base plane would require some fracturing of bridges of intact material between the individual en echelon joints. Thus, the base plane could not be considered cohesionless and, in recognition of this, a modest cohesion value of 1,000 psf was assumed to exist along any potential base plane of sliding. An equivalent linear angle of friction for the Set 1 joints of 50 degrees was used for analysis, which was based on interpretation of Barton's nonlinear shear strength criterion.

Using these parameters, a series of simple block sliding analyses was done to determine the geometry of the most critical potential slide mass. For the fully saturated condition it was determined that the critical depth of a water-filled backscarp or tension crack was 11 ft and that the critical angle

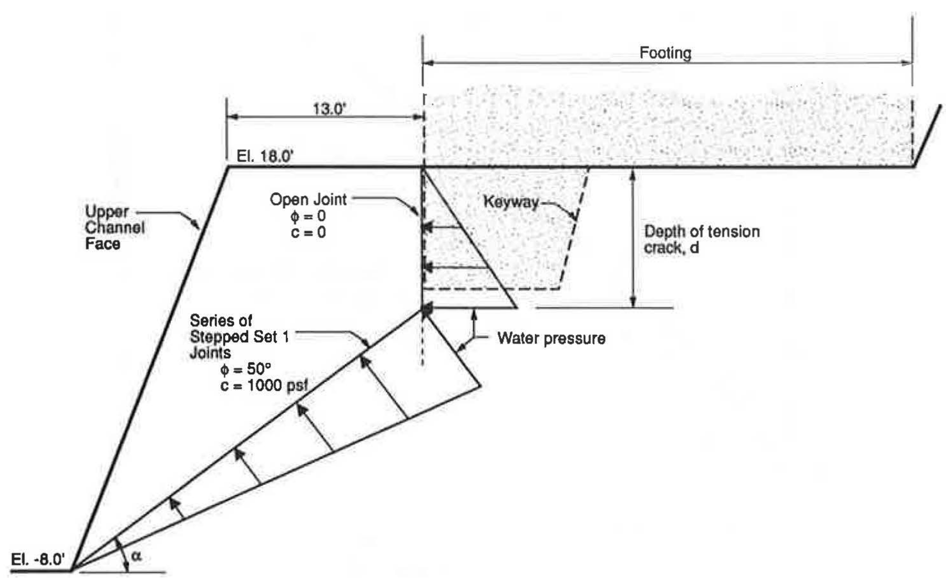

FIGURE 8 Potential plane failure in front of footing, for saturated conditions (α_{crit} = 35 degrees, d_{crit} = 11.0 ft).

of the base plane was a dip of 35 degrees, as shown in Figure 8. The computed factors of safety (FS) for this potential plane failure mechanism under fully saturated and fully drained conditions were as follows:

- Saturated conditions: $(FS)_{static} = 2.08$, and $(FS)_{seismic} = 1.39$.
- Drained conditions: $(FS)_{static} = 2.85$, and $(FS)_{seismic} = 1.90$.

A minimum factor of safety of approximately 1.4 is reached under conditions of full saturation and seismic (pseudostatic) loading, indicating that this postulated plane failure mechanism is not likely to occur. It was concluded that there were no significant plane failure mechanisms which could be realistically postulated in the vicinity of the pier footing.

General Rotational Failure

The Hoek-Brown strength parameters for the overall rock mass were based on the conservative assumption that the entire mass below the footing composes a major crush zone, as previously discussed. Using the nonlinear strength envelopes resulting from the Hoek-Brown criterion (Figure 7), a series of analyses was conducted to assess the stability of the footing against formation of a general rotational failure surface through the foundation.

Initial analyses were undertaken using the simplified Bishop method of slices, incorporating a nonlinear material strength criterion. The widely used Bishop limiting equilibrium method of slices assumes a circular failure surface, vertical boundaries for the interior slices, and interslice boundary forces equal to zero. Although these assumptions certainly are simplifications, the method is nevertheless useful for conducting initial stability analyses and searches to locate the critical potential failure surface. Resulting from these analyses, Figure 2 shows the location of the critical failure surface for the following assumptions:

- Parameters for Hoek-Brown failure criterion—$m = 0.13$, $s = 0.00005$, and $\sigma_c = 7{,}500$ psi;
- Fully saturated rock mass;
- Static analysis (no earthquake loading); and
- Rockbolt forces not included.

For these initial assumed conditions, the calculated factor of safety under static conditions was $(FS)_{static} = 2.17$.

For an actual potential slide mass, the boundaries of the interior slices would most likely be formed by preexisting joints belonging to Set 1, and in undertaking additional analyses it was desirable to model the fact that the shear strength available along these preexisting interslice boundaries would be different from that available within the main body of the rock mass, that is, along the basal planes of sliding. In addition, further parametric analyses were required to look into the effects of the 31 rock bolts that had been installed, the effects of earthquake loading, the effects of rock mass drainage, and the effects of variations in the estimated rock mass strength parameters—notably the parameter s, which reflects the relative degree of brokenness of the rock mass. For these purposes, the powerful limit equilibrium method of slices developed by Sarma (*4,11*) was used. This method allows consideration of a noncircular failure surface and incorporation of specific structural features as part of the potential slide mass, as well as including interslice forces, different strength parameters on different surfaces, and application of additional forces such as bolt forces or pseudostatic earthquake loading forces. However, Sarma analyses that used nonlinear shear strength criteria along the various surfaces of sliding were not widely available at the time, so it was decided to use an equivalent linear approach based on the instantaneous friction and the instantaneous cohesion values for the normal stress levels acting across the various surfaces. The effective normal stresses across each surface of sliding are calculated during the course of the Sarma analysis, and these values have been used to determine the appropriate values of instantaneous friction angle (ϕ_i) and instantaneous cohesion (c_i) along the basal surface of each slice.

On the basis of the initial Bishop analyses, a failure surface having the geometry shown in Figure 9 was then used for further analysis using the Sarma method. Because this geometry was based generally on the critical failure surface location as identified by the initial Bishop analyses, then for analyses including somewhat modified loading or strength assumptions, the location of the critical surface would be expected to change slightly. However, the changes would not be significant, except possibly in a case that indicated that stability was marginal. In such a case, additional searches were performed to confirm the location of the critical failure surface. As shown in Figure 10, equivalent linear or instantaneous values for the Mohr-Coulomb parameters of friction angle and cohesion were determined for the base of each of the three slices, on the basis of the nonlinear Hoek-Brown envelope and depending on the normal stress acting across each base plane respectively. The effects of the nonlinear shear strength behavior of the material, in terms of the changes in the values of friction angle and cohesion mobilized at different normal stress levels, are evident from the strength parameter values for the base of each slice as noted.

Base of Slice Number	*Normal Stress Across Base (psf)*	*ϕ_i (degrees)*	*c_i (psf)*
1	2,500	47	1,700
2	7,500	37	3,000
3	5,000	41	2,400

For the boundaries of the interior slices, formed by Set 1 joints, it was assumed that these joints were cohesionless with an instantaneous friction angle of $\phi_i = 50$ degrees, as previously discussed for Set 1 joints.

Using the same assumptions as those used in the Bishop analysis as noted above, the Sarma analysis gave a factor of safety under static conditions of $(FS)_{static} = 2.26$, which compares well with the computed Bishop value of 2.17.

Adding peak earthquake loading which is equivalent to a pseudostatic lateral load of 0.3 g, as previously discussed, gave $(FS)_{seismic} = 1.23$.

Because of the conservative nature of the assumptions incorporated in these analyses regarding rock mass strength, lack of drainage, no rock bolts, and a pseudostatic earthquake loading, these computed factors of safety indicated that the foundation stability was fully adequate.

The following computed factors of safety provide some measure of the sensitivity of these analysis results to the values of the input parameters assumed.

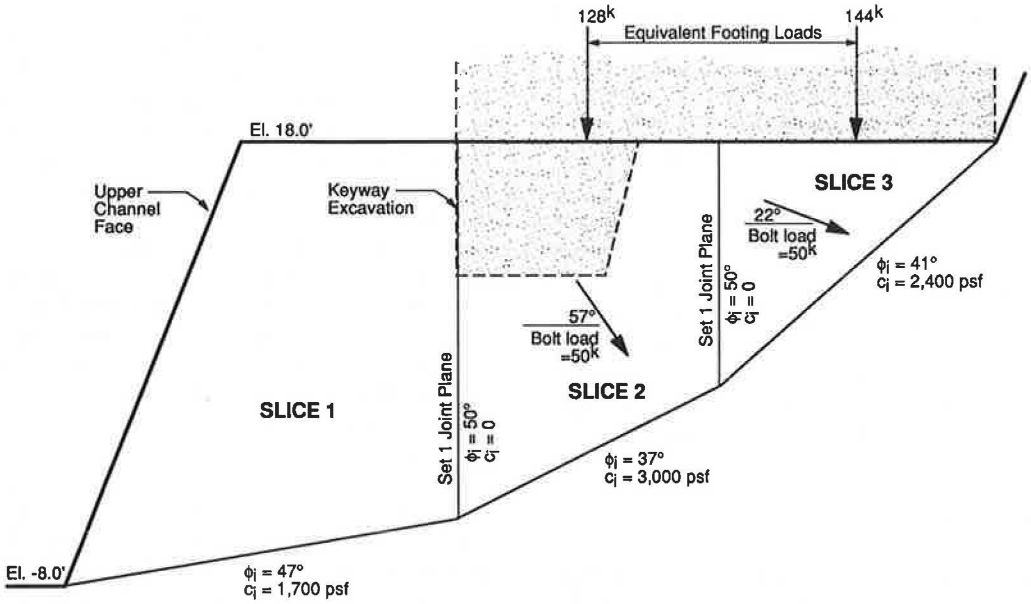

FIGURE 9 Conditions for Sarma analyses of potential general failure beneath footing (loads shown are for 1-ft-thick slice; Set 1 joint planes assumed as interior slip surfaces at 90-degree dip).

- For the assumed conditions as stated above, $(FS)_{static} = 2.26$ and $(FS)_{seismic} = 1.23$.
- For an increase in the Hoek-Brown strength parameter s from $s = 0.00005$ (equivalent to fair- to poor-quality rock) to $s = 0.002$ (equivalent to fair- to good-quality rock), $(FS)_{static} > 5.0$ and $(FS)_{seismic} = 3.40$.
- For initial assumed conditions plus the effect of the thirty-one 150-kip rock bolts installed during construction, $(FS)_{static} = 3.03$ and $(FS)_{seismic} = 1.94$.
- For initial assumed conditions but with the rock mass fully drained, $(FS)_{static} = 2.61$ and $(FS)_{seismic} = 1.48$.

CONCLUSIONS

During excavation of a bridge pier foundation and construction of the associated spread footing on rock, it had been postulated that the jointed and broken nature of the rock mass that was actually exposed could jeopardize the stability of the foundation. As a consequence, the footing design was modified to incorporate an excavated keyway, backfilled with reinforced concrete, from which long, grouted rockbolts were placed. Careful investigation of the site was later undertaken by independent consultants to confirm that the bridge pier foundation was indeed stable. The potential failure mechanisms identified as being kinematically possible included small-scale planar sliding of localized blocks formed in front of the footing by preexisting joint sets in the rock, and the development of a more generalized surface of rotation through the slabby, vertically oriented shear zone rocks found beneath a portion of the foundation. For analysis, it was assumed that the complete foundation was underlain by these shear zone rocks. Nonlinear strength criteria were used in the stability analyses, for the preexisting joints and for the shear zone rocks, because these criteria incorporate the important contribution to strength of the dilation that occurs when interlocking surfaces are sheared. This contribution is evident in

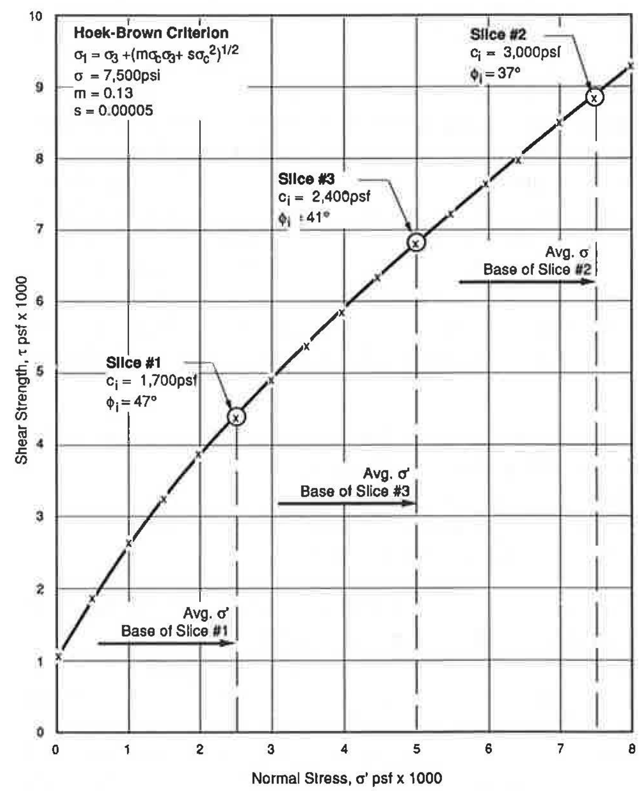

FIGURE 10 Equivalent linear Mohr-Coulomb parameters for Sarma analyses.

the high value of the instantaneous friction angle (ϕ_i) mobilized for such materials when sheared under low normal stresses. Where field investigation shows that there is a substantial degree of particle interlock in the fabric of the rock mass, along the direction of shearing necessary to cause failure, then ignoring this factor can result in significantly underestimating the available shear strength along potential surfaces of sliding. The results of the investigations and analyses showed that the stability of the bridge foundation was fully acceptable. Although the addition of rock bolts increased the computed factors of safety, these factors were already well within the range of normally accepted design values.

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