# EVALUATION OF A SLIDE IN UNWEATHERED SHALE

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by

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# EVALUATION OF A SLIDE IN UNWEATHERED SHALE


## Abstract

During construction of a predominately 1-horizontal to 1-vertical excavation in the Eagle Ford Geologic formation, a 39-foot high block of unweathered shale, measuring 56 by 250 feet, moved laterally 29 feet over a 48 hour period. The excavation had been completed to approximate Elev. 380, with the top of bank at Elev. 434. The bottom of the slide occurred along a bedding plane at Elev. 386.7.

The failure consisted of an active block, weighing an estimated 6110 tons, and a passive block weighing 717 tons. The active block measured 150 feet by 56 feet, and was defined by two joints intersecting at an angle of 98$^{o}$ some 56 feet into the slope.

Failure is attributed to hydrostatic pressure within the pre-existing joint system. The joints are believed to have opened due to relief of stresses associated with the excavation. Statics were used to analyze the slide. Based on this analysis, the available shear along the failure plane equates to an angle of friction of 19.2$^{o}$.

The method of stereographic projection was then used to evaluate the potential for additional slides. The slide, the method of analysis, laboratory shear strength data, and remedial repair are described.


## INTRODUCTION

Over a two-day weekend in mid-October, 1985, a slope failure encompassing some 4180 cubic yards of soil and unweathered shale occurred during the final stages of the excavation of a planned lake. The failure consisted of three distinct units; horizontal movement of a large active block, and a passive block of unweathered shale, and a subsequent rotational slide within the overburden soils. The volume of the active and passive blocks was some 3,700 cubic yards, weighing an estimated 6,830 tons. A photograph of the slide is provided in **Figure 1.**

Due to the size and nature of the failure, an investigation was initiated to evaluate the cause, remedial repairs, and potential for additional slides of a similar nature. The findings of that investigation are provided herein.

## Project Description

The project consisted of construction of an interior drainage lake in conjunction with reclamation of flood plain land. The lake at normal pool (Elev. 418) has an average depth of about 40 feet. The lake measures roughly 740 feet by 1400 feet in plan view.

Original ground surface was at approximate Elev. 425. Project plans called for finished grade at Elev. 434. From Elev. 434 to 419, 3 horizontal to 1 vertical (3:1) slopes were used, with 1:1 slopes from Elev. 419 to 389, and 3:1 slopes to the bottom of the excavation at about Elev. 380. All slopes below approximate Elev. 412 were excavated into unweathered shale.

The site is underlain by alluvial terrace deposits overlying clay shales of the Eagle Ford group, Cretaceous System. The terrace deposits extend from original ground surface (Elev. 425) to Elev. 412, and in descending sequence consist of silty and sandy clays and medium to fine-grained sands. A perched water table was present within the sands. The phreatic surface fluctuates slightly with seasonal rainfall, however, based on piezometer readings and experience, it is generally at approximate Elev. 420. The subsurface conditions and initial cross-section of the excavation are illustrated in **Figure 2.**

The Project excavation began in the summer of 1984. During initial excavation, the Contractor experienced difficulties associated with the groundwater. In order to reduce groundwater effects on construction and the potential for water loss into the alluvials after filling of the lake, a clay and shale "plug" was constructed from the top of shale (Elev. 412) to finished grade. This plug of recompacted clay and shale is shown in **Figure 2.**

After completion of the plug, construction proceeded normally to approximate Elev. 380. In late September, 1985, a relatively small block of shale measuring about 20 x 50 feet in the east side of the excavation moved into the excavation. The failure was inspected and determined to be a result of intersecting joints daylighting into the 1:1 excavated slope. Since the size of the failure was considered to be insignificant to the overall excavation, a 3:1 berm was constructed over the failed block. No additional movement was observed.

Approximately three weeks later, movement of the larger block occurred in the west side of the excavation. A field investigation was initiated to evaluate the cause of failure, and the potential for additional failures of a similar nature. The field investigation consisted of a geologic survey, installation of 12 piezometers within the alluvium, and three vertical and one inclined borings. The purpose of the vertical borings was to obtain samples of the shale for analysis. The inclined boring was performed for the purpose of detecting the presence and spacing of open joints. The results of the geologic survey and field and laboratory investigations are provided in subsequent sections.

## Description of the Slide

The slide consisted of three distinct units; an active block of unweathered clay shale, a passive block of unweathered clay shale, and a circular slide in the overburden soils. The circular slide was attributed to movement of the active block. **Figure 3** shows in plan dimension the post slide position of each of these units.

The active block measured at it's maximum points, 150 feet long, 56 feet wide at the base, and 39 feet high. The block weighed an estimated 6110 tons, and contained about 3280 cubic yards. It was defined by two vertical joints intersecting at a depth of 56 feet (measured horizontally) into the excavated slope with an included angle of $98^{o}$. These joints are shown in **Figure 3**. Iron staining was present on the joints of the active block and is shown in **Figure 4**.

The volume of the passive block was 385 cubic yards, weighing 717 tons. The circular slide encompassed 510 cubic yards of material weighing approximately 862 tons.

The bottom of the failure plane of both the active and passive blocks, as defined by survey at three locations, was at Elev. 386.7. The active block translated horizontally 29 feet, and rotated approximately two degrees counterclockwise. The rotation is attributed to the resistance provided by the passive block.

The failure plane, exposed within the stress fracture in the passive block and along the backside of the active block, exhibited very smooth surfaces being almost slick to the touch. No evidence of a thin bentonitic seam or weathered zone was observed. Exposure of the failure plane at the toe of the active block by excavation revealed a zone of crushed and pulverized, moist clay shale varying from one-half to one inch thick. The clay shale above and below the zone was competent and intact.

Although the actual time span of the failure was not observed, the site conditions after movement indicate a rapid failure, probably on the order of minutes.

## Material and Site Characterization

Site geology consists of a thin alluvial terrace deposit over clay shale of the Eagle Ford group. The alluvium consists of silty clays over sands to Elev. 412. Below this elevation, relatively unweathered gray shale extended to the full depth of excavation.

The shale is soft to moderately hard with an approximate two degree dip to the east. Distinct horizontal layers are evident. Bentonitic layers are common within shales of the Eagle Ford group; however, none were present within the depths investigated.

Both open and closed joints were observed during the geologic reconnaissance. In general, those joints which were open, were open about one inch as shown in **Figure 5**. The presence of joints is attributed to geologic stress relief. It is anticipated that the joints are closed under normal stress conditions, then opening as stresses are released during excavation and removal of overburden.

Perched groundwater was present within the alluvium at approximate Elev. 420, or about 14 feet below final ground surface.

Four consolidated-drained, direct shear tests were conducted on samples of the shale at the elevation of the base of the slide. Samples were sheared along bedding planes. Tests were conducted to evaluate the peak shear, residual shear, and residual multi-shear strengths. Test results are summarized in **Figure 6**.

The tests indicate the clay shale, along bedding planes, has a peak shear varying from 45.5° to 27°, with measured cohesion varying from 0 to 3.8 tons per square foot (tsf). The initial residual strengths were measured to be 9.5° to 21.5°. Multi-shear residual tests indicate shear strengths varying from 4° to 9°, with developed cohesion of 0.6 to 0.8 tons per square foot (tsf).

## ANALYSIS

Two types of analyses were performed; those associated with the cause and repair of the slide, and those associated with evaluating the excavation for the potential of additional slides. The slide was analyzed by statics, using the three dimensional physical properties of the active and passive blocks. The potential for additional block failures was evaluated by use of stereographic projection. Both types of analyses are discussed in the following sections.

### Slide Analysis

The size and dimensions of the various components of the slide were well defined by the use of field surveys immediately after the movement, and by aerial photography. By clear definition of the size of the components (i.e., the active and passive blocks and the circular slide), accurate determination of the weight of these components was possible.

The active block was considered to be acted upon by hydrostatic pressure within the two defined open joints. The location and orientation of these joints are shown in **Figure 3**. Neither cohesion, nor friction was considered to be applicable along either joint. An equivalent hydrostatic pressure equal to Elev. 420, decreasing with the shape of the excavation was used to compute the pressures on the

joints. Complete drainage of the joints was not considered to be applicable due to observed seeps in the excavation sidewalls at approximate Elev. 420 prior to the failure.

Resistance provided by the passive block was calculated by considering a frictional component along the base, but no friction or cohesion along the vertical failure plane (stress plane). Justification for not using a component of cohesion was considered because of the abundance of closely spaced, clean, well defined joints between shale blocks.

Pressures exerted by the rotational slide were also not considered in the analysis. An equal angle of friction along the bedding plane for both the active and passive blocks were used.

By solving for equilibrium, using a factor of safety of 1.0, the angle of friction along the bedding plane was calculated to be $19.2^{\circ}$. This compares with a measured laboratory residual shear strength of $9.5^{\circ}$ to $21.5^{\circ}$.

Additional analysis could have been performed considering various cohesive components along the vertical failure plane of the passive block. This approach was not adopted, however, due to the resulting line of action of the cohesion component. Due to the limited rotation of the active block ($2^{\circ}$) little, if any, cohesion is considered to be applicable along the stress plane. In addition, the propagation of the vertical stress failure after initiation of movement would negate, in the writers opinion, any effective cohesion.

**Additional Slide Analysis**

The potential for additional movement of shale blocks was analyzed by the use of stereographic projection. The complete description of the method is beyond the scope of this paper. An excellent treatment of the subject is provided by Goodman (1976).

In general, stereographic projection is a method of analysis whereby three dimensional rock blocks, defined by a system of joints and excavation, can be analyzed on a two dimensional diagram. The two dimensional diagram is called a stereonet.

The stereonet is a two dimensional representation of the arc traced by a plane passing through the center and bottom half of a sphere, projected into a horizontal plane passing through the center of the same sphere.

Initially, the site is surveyed to evaluate the trend, if one exists, of joints and/or sets of joints. This information would include the strike and dip of the joints or sets.

The stereonet can then be used to define the location of intersecting joints or groups of joints, and if the joint intersection daylights into the excavation. If a joint intersection daylights, the stability of the block defined can then be evaluated by applying the available frictional component along the joints. **Figures 7 and 8** illustrate this concept.

Three joints labeled J-1, J-2, and J-3, are shown in **Figure 7** along with the proposed cut, C-1. The portion of the circle to the north side of the arc defined as C-1, is the daylighted area of the excavation, i.e. the portion of the sphere where joints intersecting would daylight. As can be seen, the intersection of joints 2 and 3, (I2-3), and joints 1 and 3, (I1-3), both daylight into the excavation. The intersection of joints 1 and 2, (I1-2), does not. This indicates that the blocks defined by the excavation sidewall and joints 1 and 3, and joints 2 and 3, are potential slides if the available frictions are less than required, or hydrostatic forces exist.

The second part of the construction consists of plotting the poles and available friction of each joint on the stereonet. The pole is defined as the projection of the point of intersection of a line which passes through the center of the sphere, perpendicular to the plane (joint) and the lower half of the sphere.

The pole then is directly related to each joint. The circle around the pole is then defined by the available friction. The next step requires connection of the outside of the friction circles by rotation of the stereonet. If the center of the circle, the weight vector, W, falls within the area so defined, the block is stable. If the weight vector, W, falls outside the defined area, the block is unstable.

**Figure 8** illustrates the intersection of joints 2 and 3 where the available friction is $5^{\circ}$, compared to $20^{\circ}$ used in **Figure 7**. Neither figure considers the presence of hydrostatic or pore pressure in the analysis, although these forces can be included in the analysis where applicable.

In this manner, the shale blocks defined by the various joints were evaluated for stability. General trends in the joint system could be defined. A conclusion was reached, based on the general trend, that the potential for small slides was present but the probability of a second major slide of comparable magnitude was remote, provided a means of relieving the imbalanced hydrostatic pressure could be achieved. It was therefore, decided to repair the slide and evaluate the alternatives to relieve any imbalanced hydrostatic pressure.

## Slide Repair and Slope Stabilization

Since movement of the active block relieved the hydrostatic pressure, the various components of the slide were considered to be relatively stable. A 3:1 berm was, therefore, constructed on and around the larger blocks. A slight amount of settlement was observed

immediately after construction of the berm, however no additional movement has been observed.

Several alternatives were considered for the purpose of relieving the hydrostatic pressure within any open joints, to include construction of drains at various elevations, removal of the perimeter fill "plug", flattening the slopes, and rapid filling of the lake. The search for alternatives proved to be academic in that heavy rains provided sufficient water to fill the excavation to Elev. 418 early in 1986. This elevation will be maintained. No additional movement of the slope has been experienced to date.

## CONCLUSIONS

1. Information is presented which strongly indicates that the slide was initiated by hydrostatic pressures acting within pre-existing joints.

2. The available angle of friction along the base of the slide (i.e., bedding planes), equates to 19.2°.

3. Measured laboratory residual shear along the bedding plane determined angle of frictions varying from 9.5° to 21.5°.

4. The use of stereographic projection can be a valuable tool in evaluating slides of a similar nature. Judgment is required in assessing the applicable angle of friction and the potential for hydrostatic pressure.

## REFERENCES

Goodman, Richard E., **Methods of Geological Engineering**, West Publishing Co., 1976.

Figure 1. Block failure looking west. Note relative size of failure by size of individual standing to the right of center.

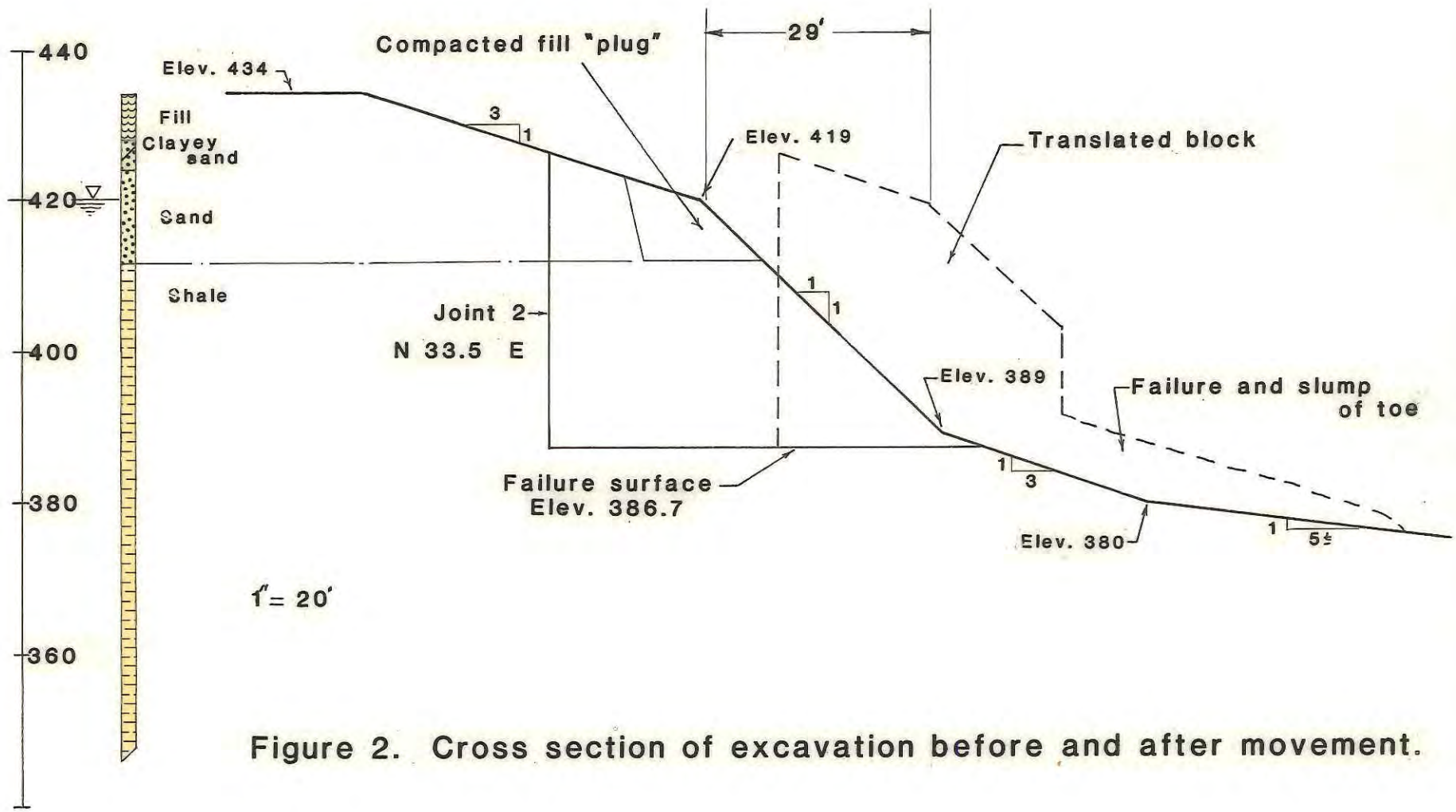


Figure 2. Cross section of excavation before and after movement.

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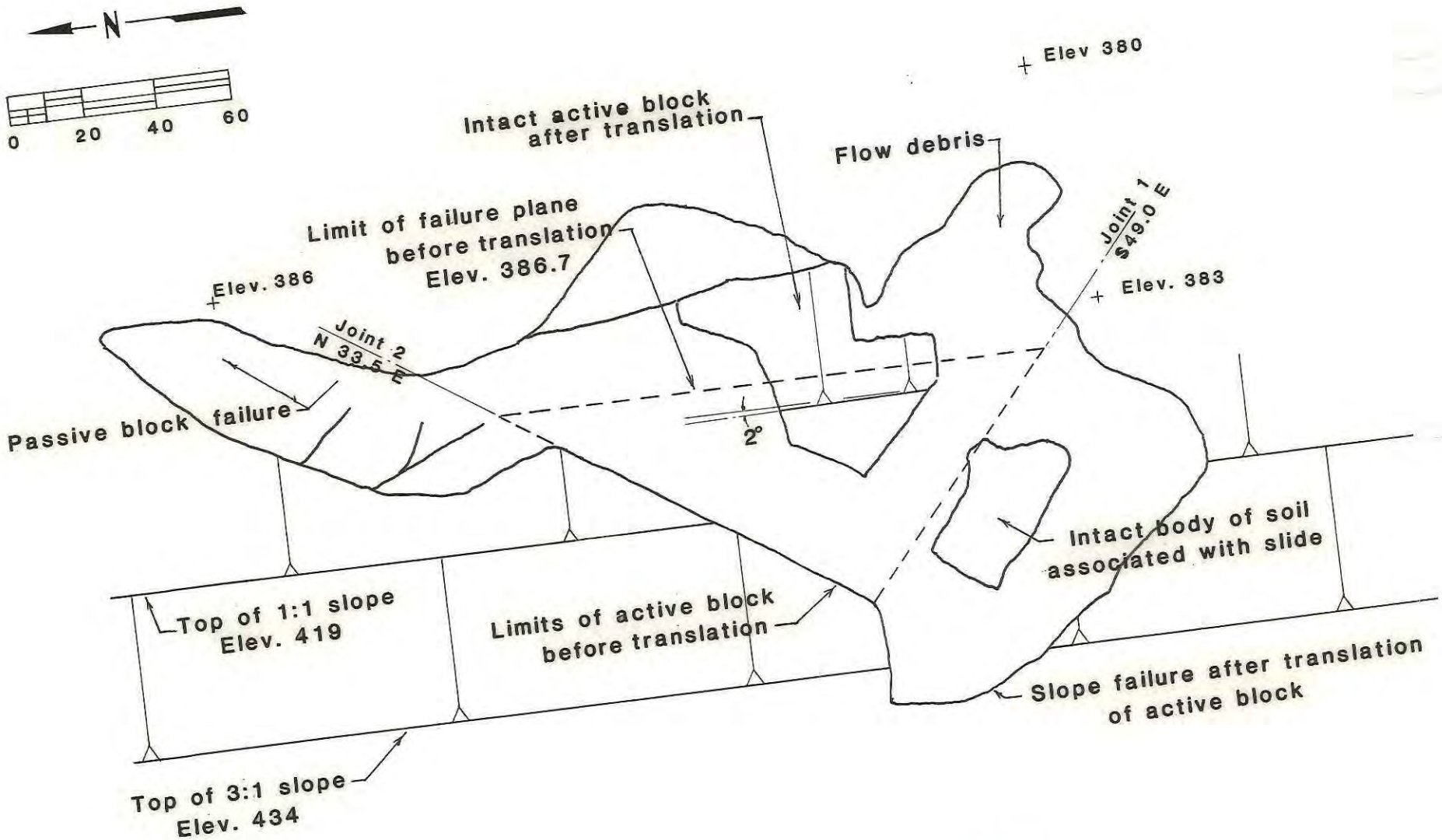


Figure 3 Plan view of excavation after slide

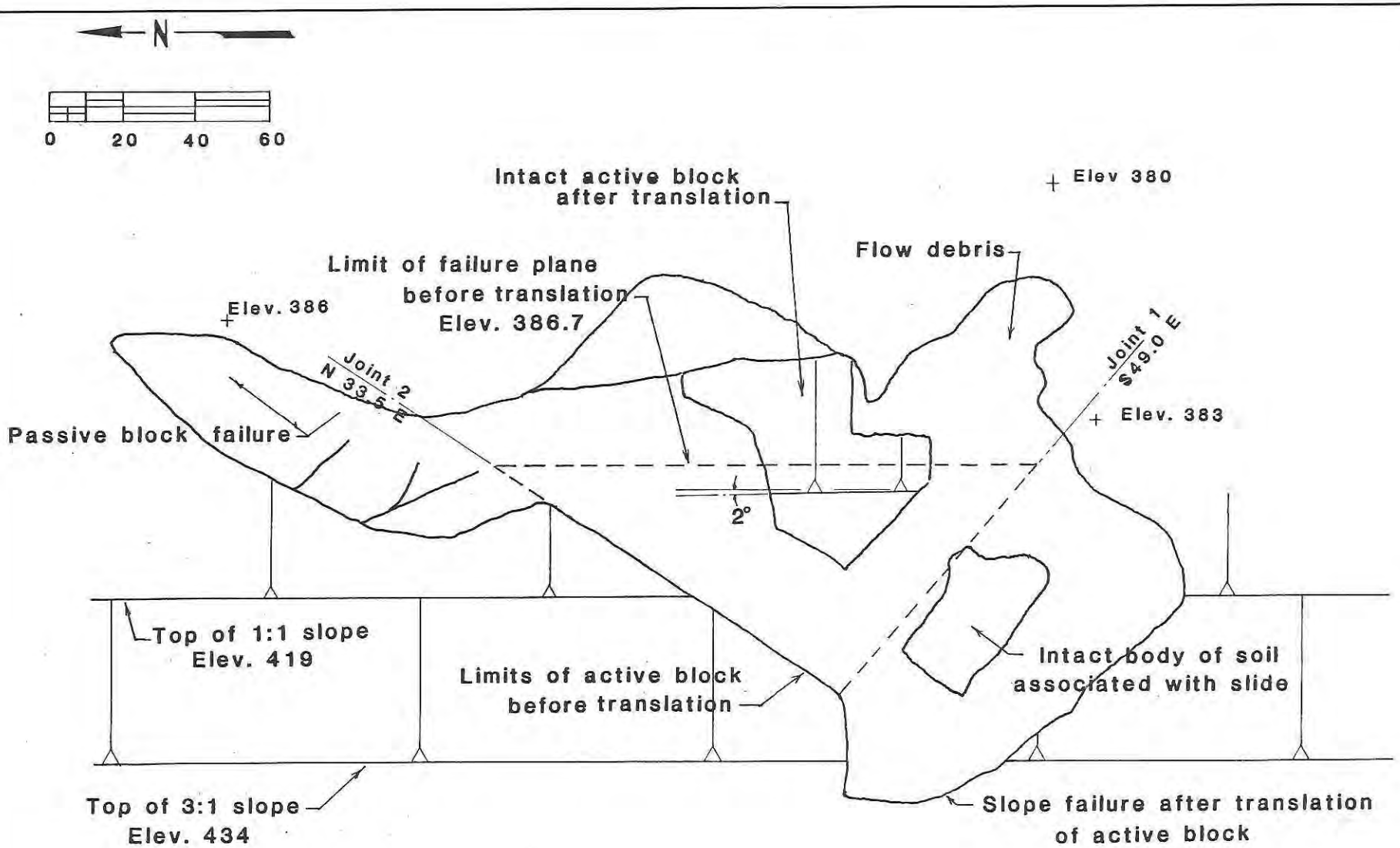


Figure 3. Plan view of excavation after slide

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Figure 4. Backside of failed wedge. Note visible iron stains along failed surface.

Figure 5. Exposed open joint approximately 200 feet south of failure. Note additional shear plane approximately two feet above the open joint.

| Moisture Content Range | | Sample (Elev) | Liquid Limit | Plastic Limit | Plasticity Index |
|---|---|---|---|---|---|
| Before | After | 485 | 53 | 22 | 31 |
| 10-16% | 14-16% | 486 | 49 | 21 | 28 |
| Material: Dark Gray Shale | | 487 | 52 | 23 | 29 |

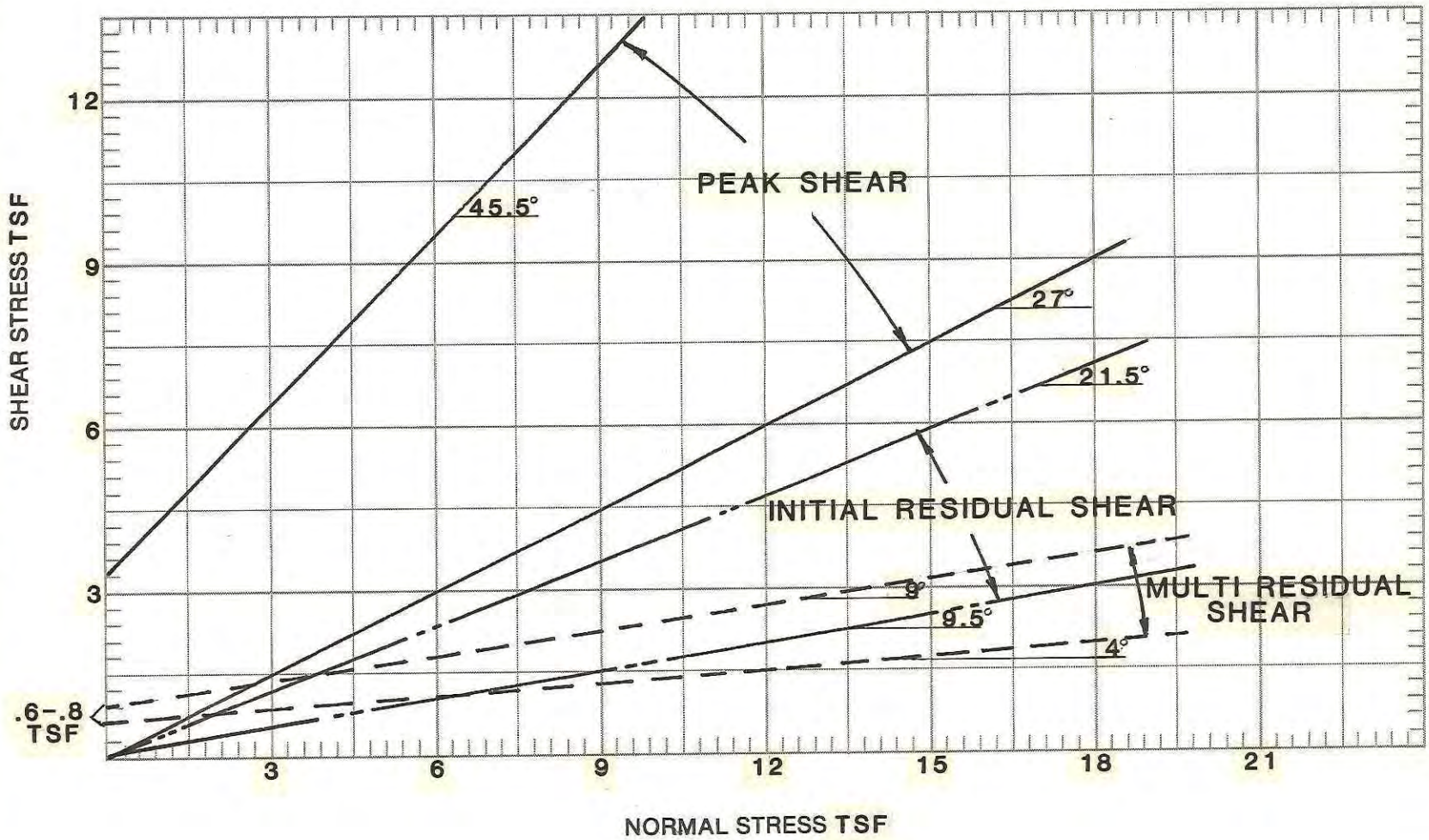


Figure 6. Summary of Consolidated-Drained Direct Shear tests

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Joint 2, J-2, N30°E, 50°S
Joint 3, J-3, S49°W, 70°N
Cut, C-1, E-W, 45°N
Angle of Friction, $\phi$, 5°

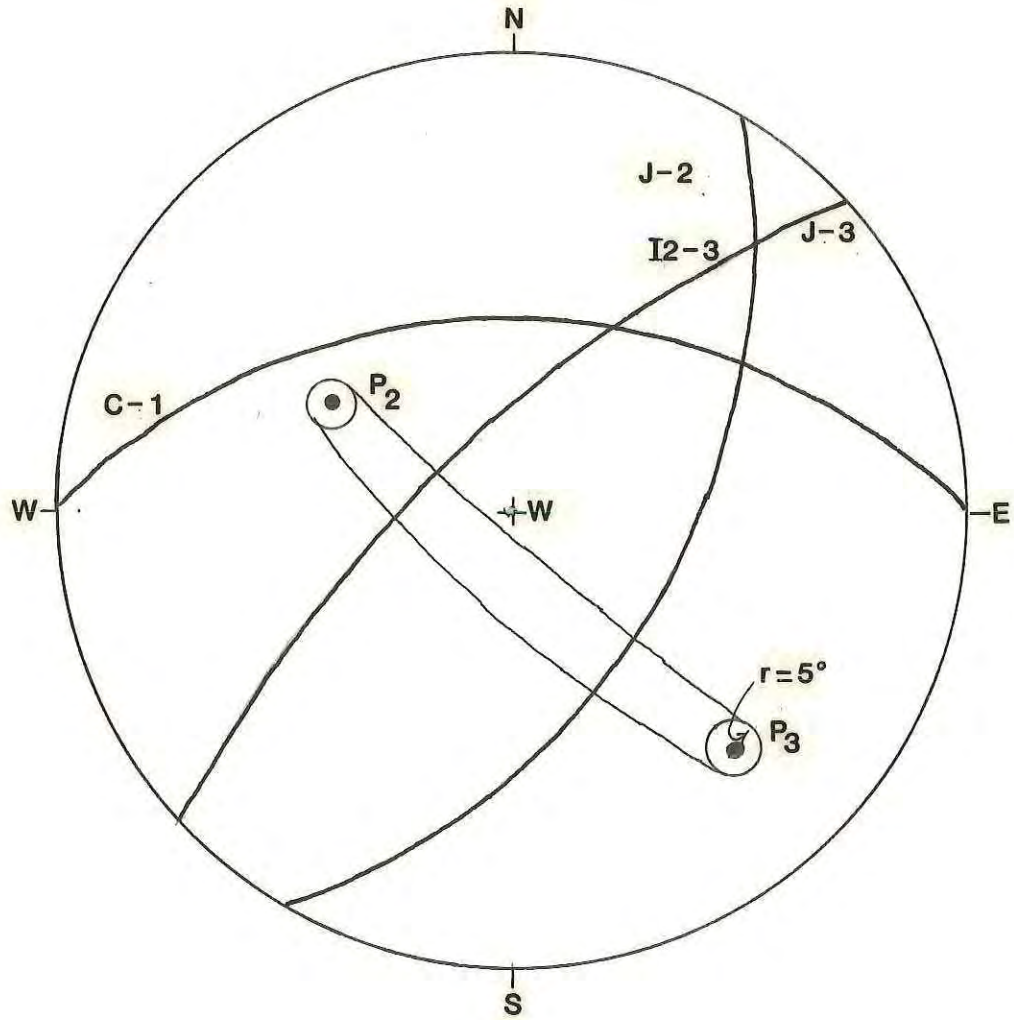


Figure 8. Instability of block defined by daylighting of joints 2 & 3 into excavation by reduction of angle of friction to 5°.

Joint 1, J-1, N-39°E, 38°S
Joint 2, J-2, N-30°E, 50° S
Joint 3, J-3, S49°W, 70°N
Cut, C-1, E-W, 45°N
Angle of Friction, 20°

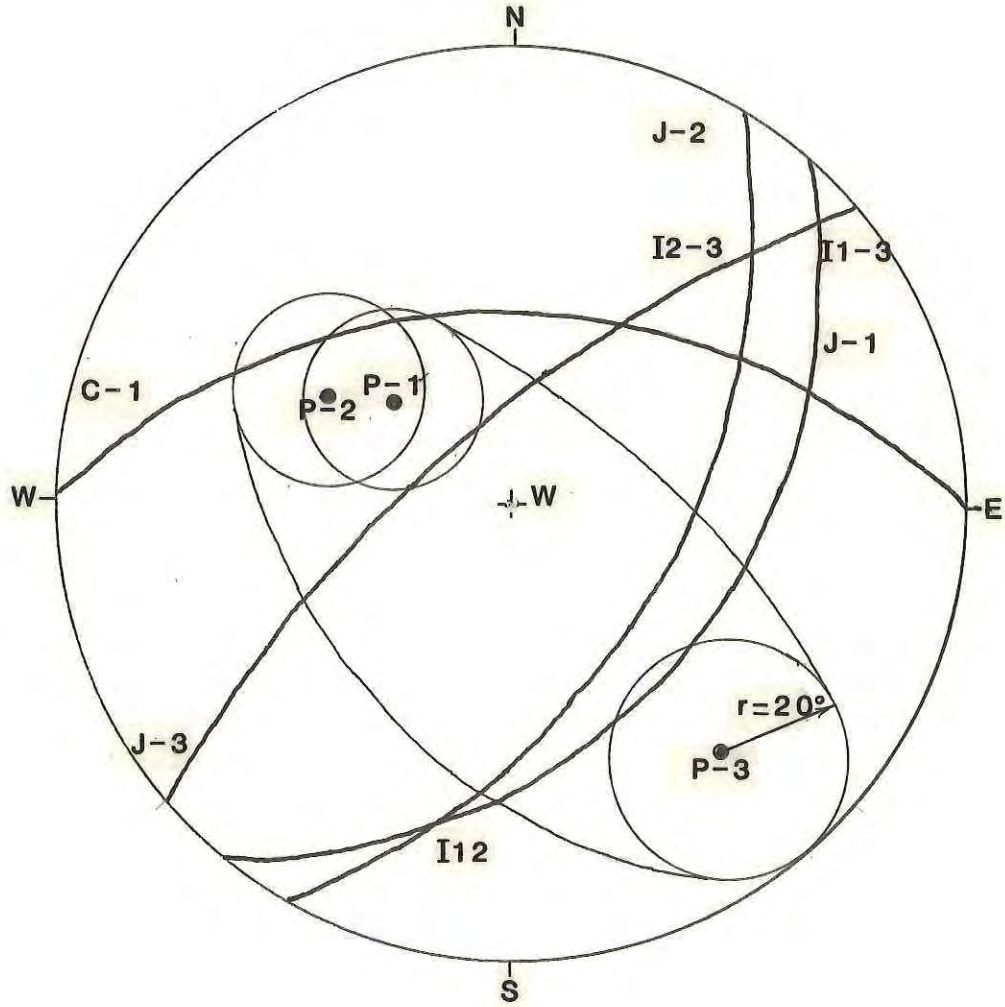


Figure 7. Example of use of sterographic projection for evaluation of potential sliding. All blocks as defined by joints daylighting into the excavation are stable, $\phi$=20°.

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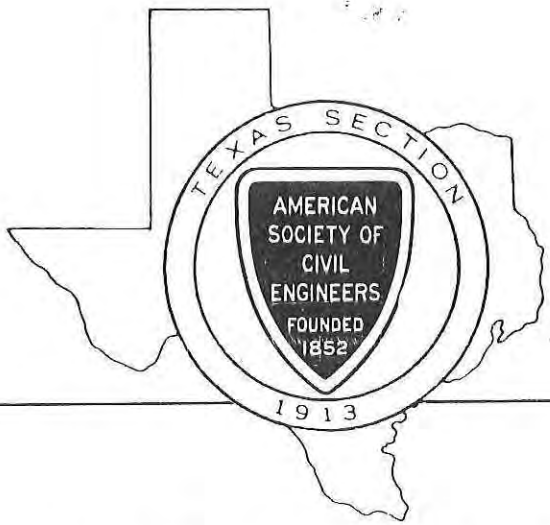
## Texas Section

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