150259 Ott Water Engineers
Jim Ford Creek Hydro
General

Oxford Energy
617/367-0032
Harry Wolf

KIM DE RUBERTIS
CONSULTING ENGINEER

June 7, 1989

Mr. Robert D. King
Ott Engineering, Inc.
1412 140th Place, NE
Bellevue, WA 98007

Dear Bob,

In my letter report dated May 23, 1989 on the inspection of the Jim Ford Creek Hydroelectric Project, I mentioned that I had taken a number of photographs of the site and the work performed. Enclosed are 50 color slides. Please make whatever copies you require, and return the slides to me at your convenience.

Yours very truly,

Kim de Rubertis

enclosures

P. O. BOX 506 - CASHMERE, WASHINGTON 98815 - (509) 782-3434

KIM DE RUBERTIS
CONSULTING ENGINEER

May 23, 1989

Mr. Robert D. King
Ott Engineering, Inc.
1412 140th Place, NE
Bellevue, WA 98007

Dear Bob,

You asked me to perform an inspection of this project, specifically, to complete tasks 1, 2, and 3 of the attached work plan. I performed the inspection yesterday in the company of

Mr. Bruce Hampton	OXESS
Mr. Luke Aldrich	Idaho Department of Lands
Mr. Joe Eichert	Idaho Department of Lands
Mr. John Elsbury	Idaho Department of Lands
Mr. Verl King	Idaho Department of Water Resources

and the purpose of this letter is to report the results of that inspection.

1. Slope Stability.

I inspected the entire penstock alignment for areas of potential instability. In several areas, excavated soil slopes have failed. The failures are shallow and do not involve large quanitities of materials. Shallow failures in the excavated soil slopes are not a threat to the integrity of the penstock. Similar failures are likely to occur for the life of the project, and the plan for maintenance should anticipate them.

Near Station 56+00, a lake bed is exposed in the excavated soil slope. The lake bed is about 50 ft thick, sandwiched between two layers of hard basalt. It consists of gray, highly plastic clay which is topped with 2-4 ft of clean, gray, medium to coarse sand. A slide is active in this lake bed at and adjacent to Station 56+00. The penstock is buried in the clay, which extends roughly 15 ft below the invert of the penstock. The slide appears to have involved only materials above the road (and penstock), some of which have slid into the road and have subsequently been cleared away. The failed area is not large, perhaps 100 ft long by a maximum of 20 ft wide. Older trees growing on the slope exhibit "pistol-grip" shape suggesting that the slope has been failing for many years. A "nest" of boulders on the slide surface seems to confirm this. Each time the slope fails incrementally, several of the columns of the overlying hard basalt lose support and fail, thereby creating the "nest". Further slope failure in the lake bed is possible. Should such a failure occur, the penstock could become involved. Providing positive drainage is important in reducing the risk of further sliding which could involve the penstock.

P. O. BOX 506 - CASHMERE, WASHINGTON 98815 - (509) 782-3434

In my report of March 20, 1989, I suggested buttressing the slope below the failed section of CMP, especially in the area of the slough below Station 80+00. This will promote slope stability.

2. Surface Drainage.

I inspected the entire penstock alignment to identify drainage courses which could cause saturation of the penstock trench or adjacent soils. Between Stations 45+01 and 50+77 are two small drainage courses. A test hole dug near Station 50+47 revealed that the soils in the penstock trench are saturated. I believe that the saturation occurs because of inadequate drainage. At the two drainage courses, the penstock is buried more than 40 in, and small culverts could be installed to provide cross drainage above the penstock.

In general, positive drainage can be provided throughout the length of the penstock by simply grading the road to drain uniformly to the valley. Concentrated flows should be avoided to the extent possible because concentrated flows may saturate steep slopes and weaken them.

3. Construction Quality and Trench Seepage.

Twelve test holes were set out, eleven of which were dug. Wes Albright dug the holes with a JD500 backhoe. The holes were backfilled after inspection. Findings are described below.

Test Hole	Station	Backfill Depth	Backfill Condition	Moisture Condition	Remarks
1	81+30	32 in	Very rocky	Moist	Large rock within few inches of pipe
2	80+10	43 in	Rocky	Very moist	Less rock
3	78+90	23 in	Dense, fine	Very moist	No rock
4	77+70	18 in	Dense, fine	Wet	Scant cover
5	76+50	46 in	Dense, fine	Damp	Nicked pipe with backhoe
6	75+30	66 in	Dense, fine	Damp	
7	65+05	52 in	Dense, fine	Wet	Saturated at bottom
8	55+82				Not dug, too close to bank
9	50+47	43 in	Dense, fine	Saturated	Caving, nicked pipe with backhoe
10	41+51	48 in	Dense, fine	Wet	Broke control line
11	31+32	37 in	Dense, fine	Wet	In lakebed clay
12	15+15	48 in	Dense, fine	Damp	

Locations are approximate. With the exception of Holes #1 and #2, the backfill is of excellent quality. The backfill is clayey and poorly drained. There is no evidence of subsurface drainage along the line of the penstock. Seepage collars or cutoffs are not required. Except for the area including Holes #3 and #4, the penstock is adequately covered for normal traffic. Representatives of the Department of Lands expressed concern over the CMP to bear the loads of typical logging traffic. Perhaps you could give some consideration to this point.

4. Other Observations.

The replacement pipe on site is 27 ft 9 in long, 42-in ID, 0.25-in wall thickness steel pipe.

The forebay water surface recorder is located upstream from the trashrack. It should be located downstream from the trashrack. The recorder cable is exposed and unprotected on the ground. It should be place in buried conduit.

In a few locations along the penstock route, cable is exposed on the ground. If the exposed cable is to be used (i. e. not waste cable), then it should be buried for protection.

The three penstock anchor blocks resting on grade above the thrust block near the bottom of the "notch" show separation of the ring beam sole plates as much as 0.5 in from the anchor blocks, and some of the anchor bolts are bent in a downstream direction. I saw no evidence of distress in the thrust block concrete or in the steel of the embedded penstock. The serviceability of the exposed section of steel penstock should be re-evaluated based on its present condition.

I took photographs of all of the features examined in the field, and I will append a set of them to this letter as soon as I receive them from the printer. If you have questions about these observations or recommendations, or if you require further information, please let me know.

Yours very truly,

Kim de Rubertis

enclosures

May 3, 1989

MEMORANDUM

TO:	Chuck Cuddy Archie Ford John Elsbury Bob King
FROM:	Kim de Rubertis
SUBJECT:	Jim Ford Creek Hydroelectric Project

Please add the enclosed photographs to my letter dated March 27, 1989. These are the photographs referred to in the second sentence of the paragraph entitled "**Observations**".

Photo 1 -- General View of Area Where Penstock Broke

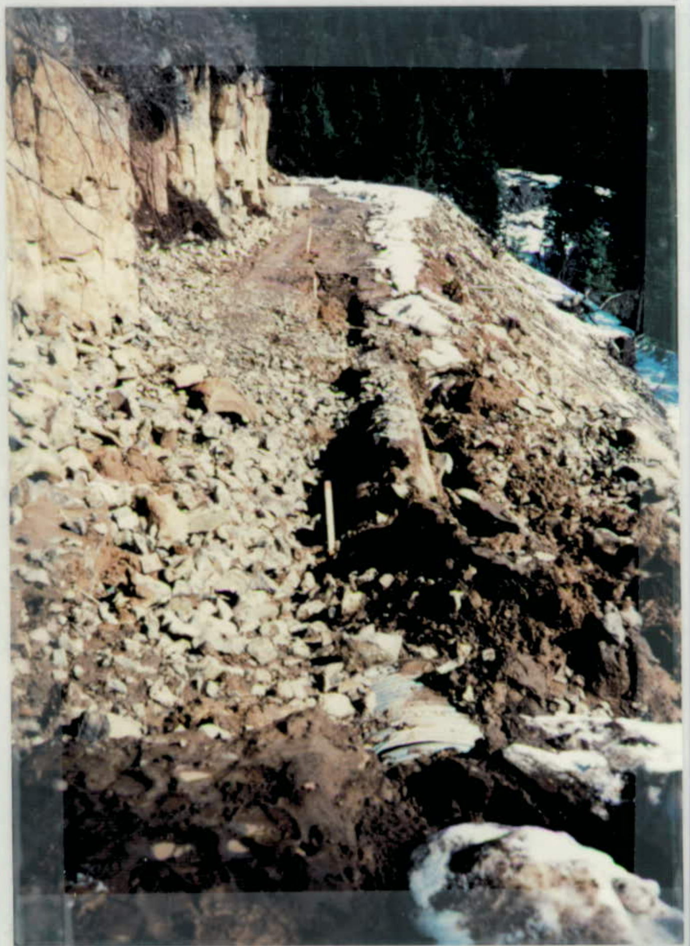


Photo 2 -- CMP and Steel Pipe

Photo 3 -- Break in CMP

Photo 4 -- Slope Below Break in CMP

KIM DE RUBERTIS
CONSULTING ENGINEER

March 27, 1989

Mr. Charles D. Cuddy
Ford Hydro Limited Partnership
P. O. Box 1940
Orofino, ID 83544

Dear Chuck,

On March 17th, you asked me to visit the Jim Ford Creek Hydroelectric Project site to inspect a slide which broke the penstock and to make recommendations for repair of the damage. On March 20th, I visited the site with you and Archie Ford of Ford Hydro and with John Elsbury of Idaho Department of Lands. The purpose of this letter is to present my observations, conclusion, and recommendations.

Observations. Figures 1 and 2 show the approximate location and condition of the slide. [Several photographs will be added showing the conditions observed.] The slide occurred roughly between stations 81+25 and 82+75 where the penstock is buried beneath the road and at the transition from 48-in dia CMP to 42-in dia steel pipe. The slide exposed about 60 ft of pipe. The CMP broke a few feet upstream from the transition. I could not see all of the CMP because it was partly buried in earth, but it appeared to me that the pipe split circumferentially in a groove between two corrugations. You had cut a hole into the CMP and reported that inspection revealed the CMP to be bowed for a length of about 15 ft upstream from the break. You said that the steel pipe was too slick to inspect. I did not observe any distortion in the exposed steel pipe.

Movement of the slide appears to parallel the slope below the road, with maximum horizontal and vertical movements of about 1.6 ft and 2 ft, respectively. The length of the slide is about 100 ft. A crack in the earth uphill and parallel to the pipe marks the limit of movement near the center of the road.

The steel pipe does not appear to have moved the same amount as the earth around it. Some road fill was perched on top of the pipe, so it appears that the earth slumped, exposing the pipe and exerting enough force to break the CMP.

Uphill of the slide, columnar basalt is exposed in the road excavation and in outcrops. It is strong and durable rock because it stands on steep slopes and because it required blasting to excavate for the road. Columns are nearly vertical. Cooling joints are spaced about 2 ft, with random horizontal joints not closer than 5 ft apart. You told me that you expect rock at the slide area to be about 2 ft below the excavated (left) shoulder of the road. Downhill of the slide, the slope (roughly 1.5H:1V) is covered with angular rock derived from the road excavation. The as-built drawing suggests that as much as 6 ft of fill was placed on the natural slope to build the road. You also told me that the fill was placed on colluvium which overlies rock. A road was cut at the toe of the slope resulting in some steepening at the toe with the excavated material cast downhill to form the road. Near the toe of the slope approximately below station 80+00 , a slump is evident in a clump of trees, roughly 30 ft x 30 ft exposing a 5-ft high scarp. Below about station 79+00, a debris barrier on the slope may act as a dam, increasing saturation in the lower part of the slope.

When I inspected the slope below the slide, it appeared to me to be saturated. Water was flowing down the road in some places.

There is no discernable evidence of the slide in the slope below the road. There is no bulge in the slope, the toe, or in the woods below. There are no trees showing tilt reflective of recent movement.

Conclusion. Settlement occurred in the saturated, uncompacted fill and in the natural soil underlying the fill. Settlement of the steel pipe redistributed the stresses about the CMP causing it to fail, thereby increasing the water percolating through the slope as the pipe emptied. The added water exacerbated settlement.

Recommendations. I discussed my recommendations with you and Archie Ford in the field, and I summarize them below in terms of remedy, work required, materials required, precautions to be observed, and details.

Remedy. Realign the penstock to take advantage of being able to support it with rock. Anchor the penstock to rock. Improve drainage. Buttress the toe of the slope, especially the slump below station 79+00, with free-draining material.

Work Required. Place temporary fill for access. Remove the upstream deadman for the highline. Perform excavation -- common, with some rock. Remove and store pipe. Install anchors. Reinstall and bed the pipe, realigned. Anchor the pipe. Regrade the road. Place free draining material between the rock and penstock. Buttress the toe of the slope with free-draining material. Remove and replace the debris barrier below station 76+00. Provide positive drainage along the road and slope.

Materials Required. Pipe will be required to replace the damaged section of CMP. I understand that you intend to use replacement steel pipe, thereby moving upstream the transition from CMP to steel pipe. A coupling will be needed to add the steel pipe. Anchors, with appropriate straps, will be needed to anchor the pipe to rock. Concrete to bed the pipe will be needed. Filter cloth and free draining backfill are required between the pipe and the rock. Free draining buttress material is needed at the toe of the slope. Random fill to regrade the road also will be required.

Precautions to be Observed. Cover over the penstock is only about 1 ft at the slide area. If this cover is typical of the cover over the remainder of the penstock, then caution must be exercised in moving equipment on top of the penstock so that it is not broken by traffic. If you reuse damaged pipe, you need to recognize that it is likely that the pipe will be weaker than new pipe, and you need to accept the risk. As long as the slope remains saturated, heavy blasting may induce more sliding. My advice is to use light blasting, if blasting is required. The control lines are stretched tight by the slide. You may wish to consider relocating them with the penstock. If you realign the pipe in such a way as to produce a significant bend above grade, the bend must be suitably restrained.

Details. I expect the excavated rock surface to be rough, both under and to the left of the penstock. If the rough surface is not smoothed, the penstock will be point loaded wherever rock is at grade. This is undesirable. My advice is to bed the penstock on a leveling course of lean concrete which can be placed before or after the penstock is installed. I would leave short (maximum 10 ft), unsupported openings at the joints to drain behind the pipe and to permit disassembly of the pipe. I would backfill the space between the rock and the pipe to the top of the pipe with free draining material. I suggest using filter cloth to retain the backfill at pipe joints. If you perform the work to place the concrete-bedded penstock on rock and if no more ground is lost, then the penstock should be permanently serviceable, subject to routine maintenance. Anchors can improve the odds that you will not suffer another break in the pipe. Anchoring straight down is risky because an anchor may find purchase only in one column of the columnar basalt. Anchoring up is best but may be difficult because of the limited space for the track drill to work. Anchoring down into the basalt at an angle of 30º to 45º may be the best practical choice; however, it will yield a pattern of forces which does not fully take advantage of the tensile strength of the bolts. I consider anchoring the penstock to be reinforcement, rather than support, and I do not believe the anchor design is critical. I suggest you use fully resin-encapsulated #6 to #8 rock bolts set about 10 ft into the rock. Drilling the anchor holes will reveal whether this is deep enough. Two anchors per 40-ft length of pipe should be adequate. Wide bands to contain the pipe within the anchors are recommended. The exposed pipe will be subjected to temperature stresses, and you may wish to provide some restraint to reduce the tendency of the pipe to "walk" or creep. Possible anchor details are shown on Figure 3.

Restoring the buttress at the toe of the slope will promote stability. Excavated material from the realignment of the penstock will be suitably free draining; however, you may need to borrow some additional material to make a proper job of dressing the toe of the slope.

I don't see much prospect to drain the slope above the penstock so that it drains away from the area of concern (stations 76+00 to 83+00) because the terrain is too rough and rocky. Positive drainage at and below road level will promote stability. You may wish to consider a lined ditch to train drainage to locations where it can be discharged without saturating potentially unstable soils. Removing the debris barrier also will be a step toward positive drainage.

If you have any questions concerning this letter, please let me know. Good luck with your repairs!

Yours very truly,

Kim de Rubertis

cc: Bob King
Archie Ford
John Elsbury

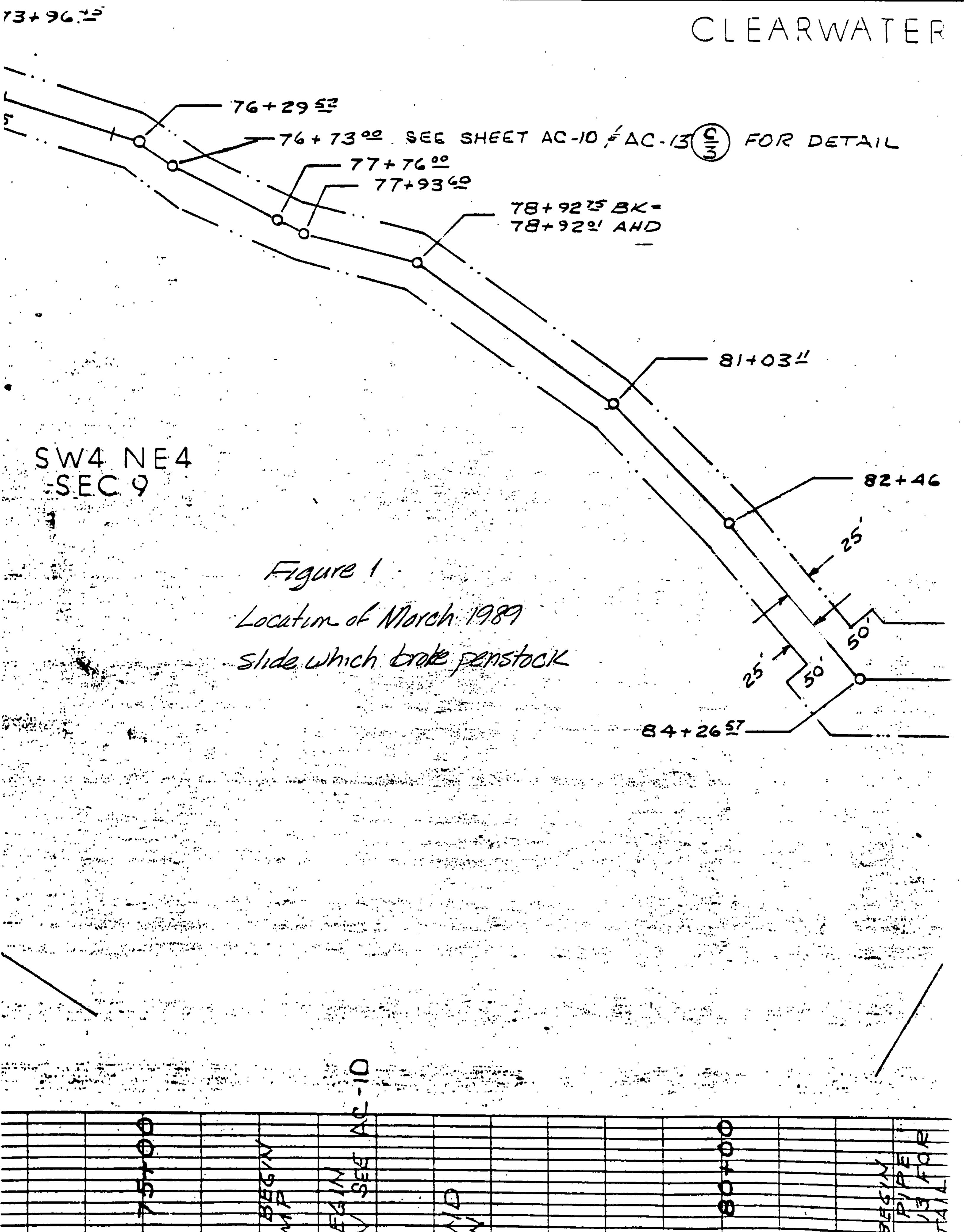
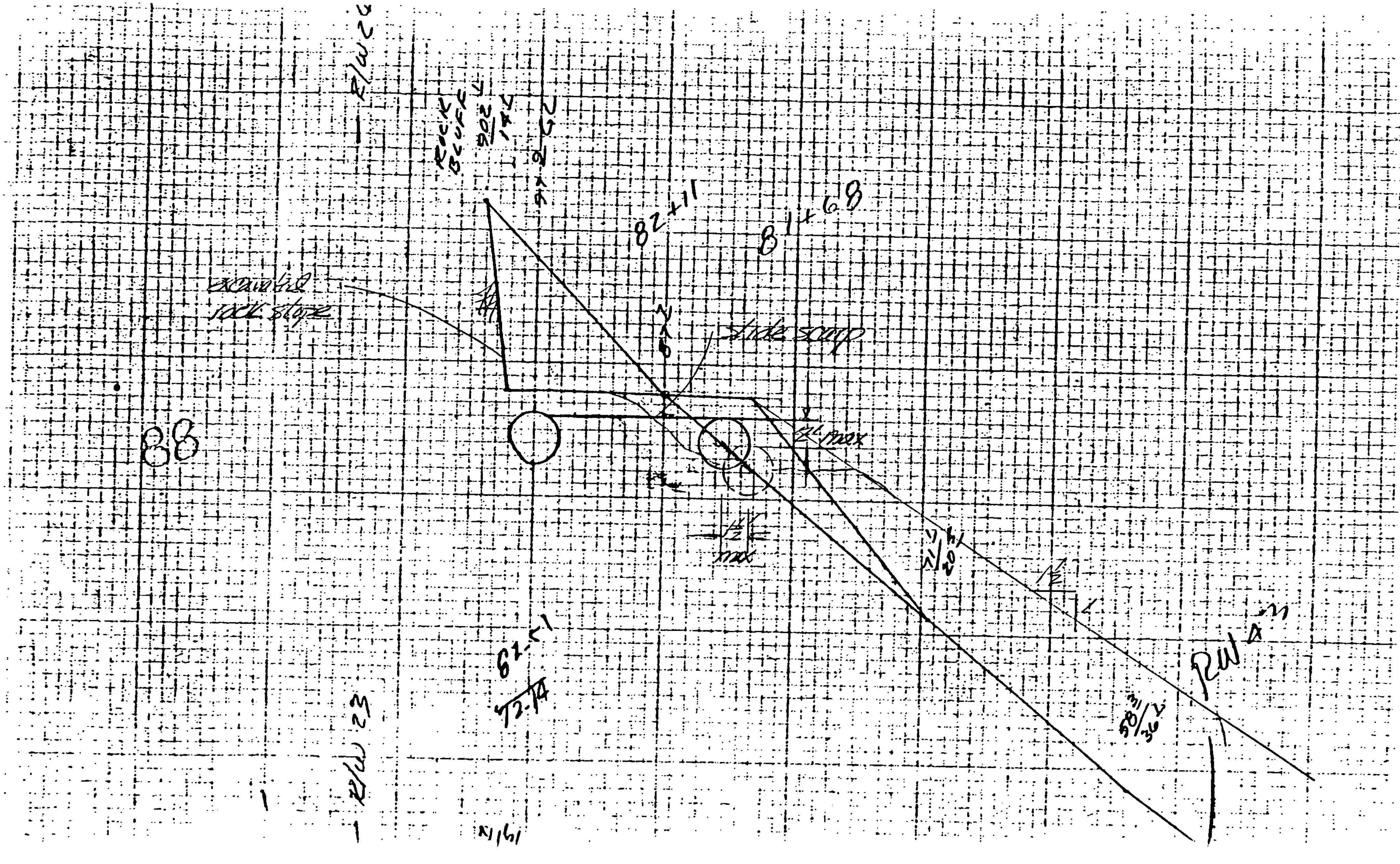


Figure 1
Location of March 1989 slide which broke penstock

Figure 2
Cross Section Through Slide of March, 1989



JOB Jim Ford Ck.

SHEET NO. 1 OF 3

KIM DE RUBERTIS
CONSULTING ENGINEER
CASHMERE, WASHINGTON

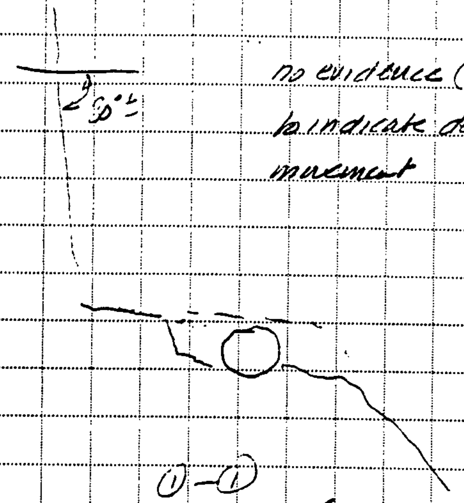
CALCULATED BY dcR DATE 3/20/89

CHECKED BY DATE

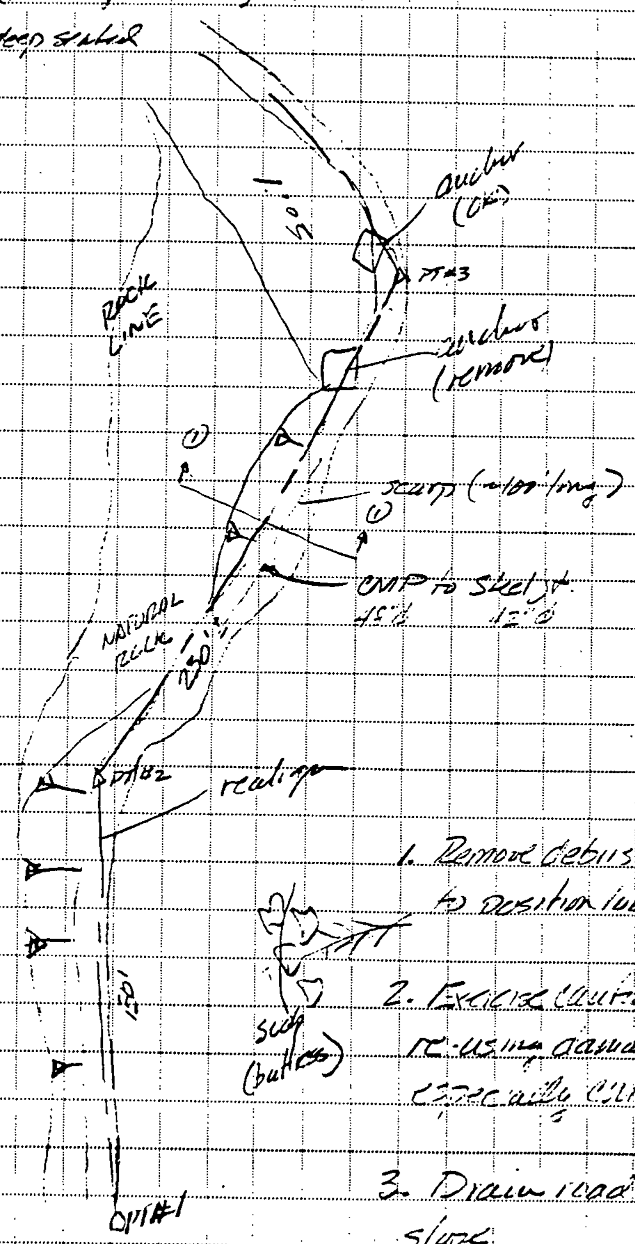
SCALE NTS

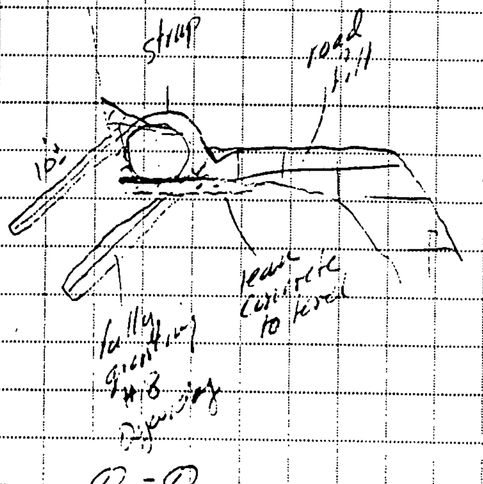
failure due to poor drainage and consolidation of fill – does not appear to involve entire slope yet

no evidence (bulge or trees) to indicate deep seated movement



①–① Conditions as found





①–① Remedy

channel bedding to permit drainage

1. Remove debris clear to position lower on slope
2. Exercise caution in re-using damaged pipe, especially CMP
3. Drain road + toe of slope
4. Restore buttress
5. Inspect top of slope

Problems: 1. moving equipment over penstock with [illegible] 2. pipe supply 3. blasting 4. control line

KIM DE RUBERTIS
CONSULTING ENGINEER
CASHMERE, WASHINGTON

CALCULATED BY DATE
CHECKED BY DATE
SCALE

consider relieving slide at ashy material (trees show movement) back to scarp

---

slump is probably the proper term for the movement

scarp (crack) is exposed about 100' long
fill settled 2' ± exposing pipe
CMP broke at joint with steel pipe
inspection (Westford) showed deformation about 15' u/s from break in CMP
steel pipe too slick to inspect

exposed fill slope (~1.5H:1V) lies on some natural material (colluvium)

Westford believes bedrock is about 2' below ℄ road at slump

excavated rock slope uphill from slump in strong basalt, outcrops above cut

slope is saturated

road was cut at toe of slope, cut and fill, some steepening at toe

there is a slump at clump of trees, roughly 30'x30' exposing about 5' backwall

there is no discernable evidence of slide in slope. there is no bulge at toe or below in woods. there are no trees showing tilt reflective of recent movement.

the conclusion is that saturation of fill caused settlement which caused CMP to crack, adding more water and accelerating movement

KIM DE RUBERTIS
CONSULTING ENGINEER
CASHMERE, WASHINGTON

JOB
SHEET NO. 3 OF 3
CALCULATED BY DATE
CHECKED BY DATE
SCALE

why it happened. settlement of fill, poor drainage

what will fix it

anchor penstock to rock, realigned
improve drainage
buttress toe

work required.

remove anchor block
excavation, common, maybe some rock
install anchors
place bedding, drained
realign pipe, anchor
regrade road
drain penstock
buttress toe
replace silt trap
drain road and slope

materials required.

pipe
anchors, say 15' OC 2 reqd say 40 ea #8 Dywidag X10'
bedding, lean concrete would be best if rough
free draining backfill
random fill

CELTITE, INC.

SUGGESTED SPECIFICATIONS

SPECIFICATION GUIDELINES FOR CELTITE POLYESTER RESIN CARTRIDGES FOR ROCK ANCHORAGES

CELTITE, INC.
150 Carley Court
Georgetown, KY 40324
(502) 863-6800
(800) 626-2948
Grand Junction, CO
Princeton, W VA
Cleveland, OH

The following guidelines are provided as an aid to specification writers and contractors in preparing project documents to illustrate basic installation practices for Celtite polyester resin anchored and grouted rockbolts or dowels.

The Celtite polyester cartridges resin system has been used for both rock and concrete structures. Please write or call Celtite, Inc. for specific information or application assistance.

MATERIALS

Rock bolt assemblies shall be a standard product of a company regularly engaged in their manufacture. The rock bolts shall be furnished complete with all accessories, including steel bearing plate, hex nuts, and when required, beveled washers and flat-hardened washers.

A. Rock Bolts

Designed rock bolts shall be of the following type:

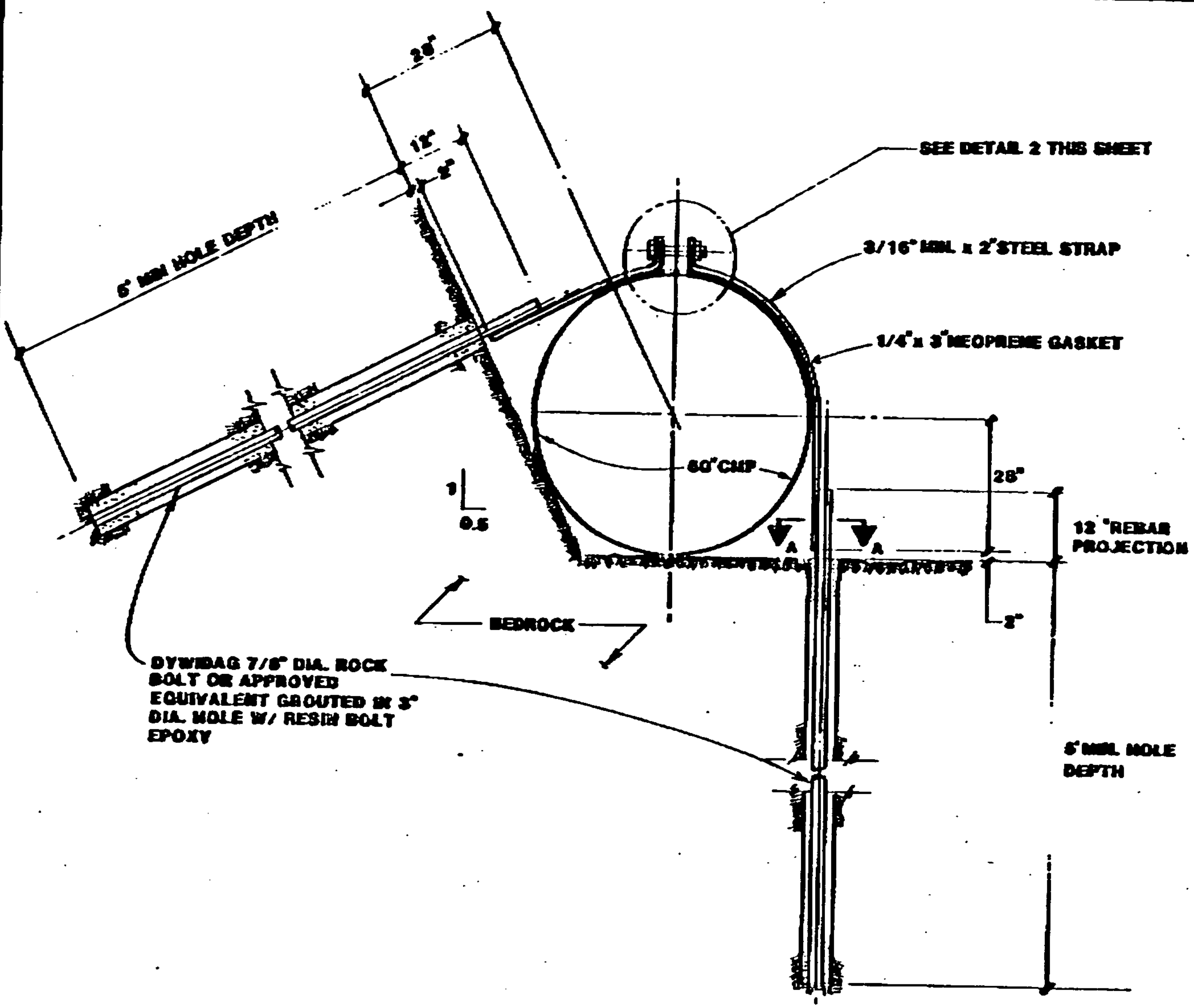
Rock bolts shall conform to ASTM Designation A-615, Grade 60, "Standard Specifications for Deformed Billet-Steel Bars for Concrete Reinforcement". The rock bolts shall have a minimum of 5-1/ 2" of thread on the outer end, or be of the threadbar type, offering a thread pattern along its entire length.

Both the threadbar and threaded (rolled or cut) rock bolts shall be supplied with spin adaptors to facilitate installation by rotating without damaging the threads, using standard tunnel or construction tools and equipment.

When mechanical couplers are used to assemble individual lengths of rock bolts, they shall be capable of developing 125% of the minimum yield strength of the bolt.

B. Bearing Plates

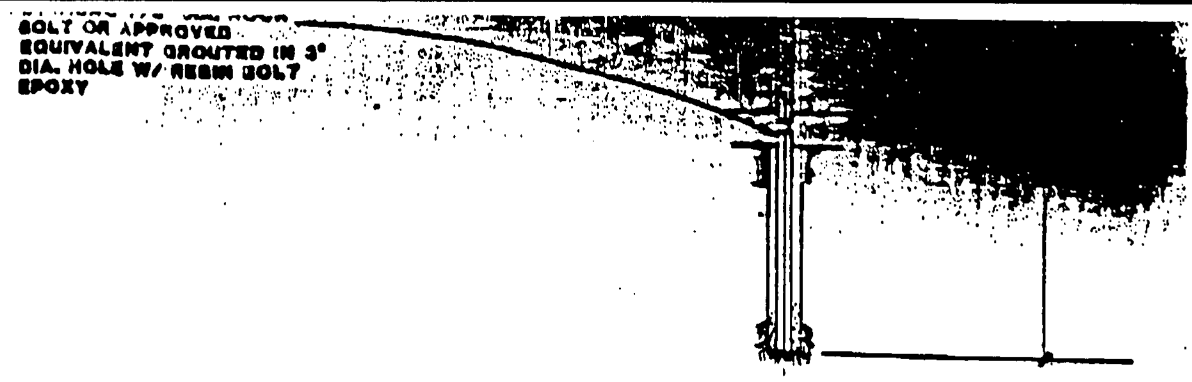
Steel bearing plates shall conform to ASTM Designation A-36, "Standard Specifications for Structural Steel". The bearing plates shall be 3/8" flat steel or equal, providing not less than 36 sq. in. area for each bolt.



SUGGESTED ANCHOR FOR SPILLWAY PIPE

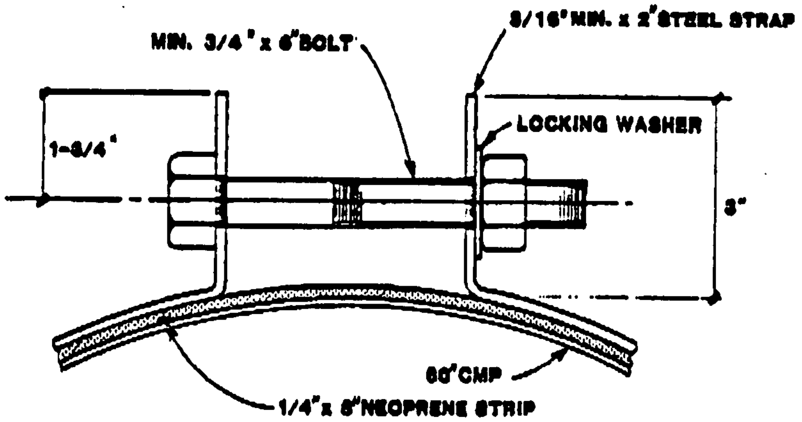
NOT TO SCALE

TYP



SUGGESTED ANCHOR FOR SPILLWAY PIPE

NOT TO SCALE



DETAIL 2 - STRAP CONNECTION

NOT TO SCALE

No	DESCRIPTION	BY	CKD	APPROV	DATE	REVISIONS No	DESCRIPT
0	ISSUED FOR REVIEW				6/88		
1	ADDED HEADWALL DETAILS AND REVISED NOTES			DAP	10/88		

REVISIONS

Golder Associates Inc.
CONSULTING ENGINEERS

March 19, 1990 Our ref: 903-1041

Ott Engineering
1412 140th Place N.E.
Bellevue, Washington 98007

ATTENTION: Mr. Robert King

RE: JIM FORD CREEK HYDROELECTRIC PROJECT
GEOTECHNICAL EVALUATION AND RECOMMENDATIONS
WEIPPE, IDAHO

Dear Bob:

Golder Associates Inc. is pleased to present the findings of our study of the Jim Ford Creek hydroelectric project, located near Weippe, Idaho. The purpose of this study was to assess the cause of an observed slope failure adjacent to the penstock alignment and prepare geotechnical recommendations to repair the slope failure and improve the overall stability of the penstock alignment in this area. We performed this work based on your March 14, 1990 authorization of our confirming proposal dated March 14, 1990.

PROJECT DESCRIPTION

The Jim Ford Creek hydroelectric project consists of a small hydroelectric development on Jim Ford Creek, located northwest of Weippe, Idaho. The project consists of a run-of-the-river diversion structure, an underground CMP penstock, an underground section of one-quarter inch thick wall, steel pipe penstock, an above ground one-quarter inch plus thick wall, steel pipe penstock, and a powerhouse. The below ground portion of the penstock has an average gradient of approximately 0.5 to 2 percent, except for the last 550 feet of the below ground penstock where the gradient increases to approximately 8 percent.

SITE CONDITIONS

The field observations described herein were made on March 15, 1990. The observations were made by a Golder Associates Senior Engineering Geologist. He was accompanied in the field by Mr. Jerry Micka, the hydroelectric project operator. At the time of our site visit the project was shut down, and the penstock had been drained at the request of the Idaho State Department of Lands.

Where the gradient of the penstock increases from 2 percent to 8 percent (Stations 76+96 to 82+46), the penstock traverses a sideslope with an average slope of approximately 35 to 40 degrees below the penstock. Above the penstock in this area, is a near vertical rock wall composed of massive, aphanitic basalt with numerous, near vertical cooling joints. The joint spacing was approximately one to two feet. The rock wall was approximately 10 to 30 feet high. Below the penstock the slope is underlain by soil and has an average slope of approximately 35 to 40 degrees for approximately 100 feet. Below this the slope angle decreases to approximately 15 to 25 degrees (Figure 1).

Slope Failures

Within the area where the below ground penstock has a gradient of eight percent, there have been three previous slope failures which have resulted in damage to the penstock. The most recent failure occurred in 1989. The penstock has been repaired in the three previous locations of the slope failures and the slope below the penstock has been regraded. The regraded slopes are underlain by silty, sandy gravel and cobbles with occasional boulders (up to approximately 24 inches). We understand that this material was obtained locally and placed using a dozer.

The area of the recent failure is in the only remaining portion of the slope that has not been regraded. The failure is located between approximately Stations 80+70 and 81+00. The slope failure is approximately 30 feet wide and has an upper and lower scarp. The upper scarp is located adjacent to the road along the penstock alignment. We understand that this scarp formed last fall. Total vertical displacement on this scarp was approximately five feet. Based on a paint line made last fall shortly after the scarp formed, approximately three feet of vertical displacement has occurred this winter.

Approximately 10 to 15 feet below the upper scarp is the lower scarp. This scarp was approximately 10 feet high with a slope of approximately 65 to 70 degrees (Figure 1). We understand that this scarp formed over the weekend of March 10 and 11, 1990. Downslope of the lower scarp, the failed material had moved up to about 100 feet and had knocked over trees and buried a lower access road. The slide debris was up to approximately 10 feet in thickness and consisted predominantly of clayey silt, with little to trace sand and gravel. The slide debris was very soft and wet and at the time of our site visit, it would not support the weight of a human being. Exposed immediately below the lower scarp was stiff clayey silt. This material is interpreted to be colluvium composed predominantly of the loess soils which blanket much of the upland area in the vicinity of the project.

Two test pits were excavated into the upper scarp using a backhoe. Exposed in the test pits was approximately seven feet of loose/soft fill overlying approximately one foot of organic-rich topsoil. Underlying the topsoil was stiff, clayey silt. The thickness of the soft/loose fill decreased inward from the edge of the road to the vicinity of the penstock. The penstock was exposed in the two test pits and was bedded in stiff clayey silt that was well compacted around the pipe. The pipe appeared to be placed in the native soils.

It was not possible to extend the test pits to the lower scarp. Exposed in the headscarp, however, was soft, clayey silt, with little sand and gravel fill. This material was approximately eight to ten feet thick and overlied a topsoil horizon approximately one foot thick. At the base of the scarp was stiff clayey silt. These stiff soils dipped downward at approximately 25 to 30 degrees and appeared to form the slide plane for the slope failure. Based on the topography in the area of the slide, the fill extended well beyond the slide zone and appeared to range up to about ten feet in thickness.

Water was seeping out of both scarps at the time of our site visit. Below the lower scarp, slight water flow was observed on the top of the stiff clayey silt where it was exposed between the scarp and slide debris. Numerous water seeps were observed in the upper loose soils exposed in the test pits excavated into the upper scarp. No water seeps were observed in the underlying stiff clayey silt. Where exposed in the test pits, the penstock was not damaged and there was no evidence to suggest that water had been leaking from the penstock.

CONCLUSIONS AND RECOMMENDATIONS

The slope failures between Stations 80+70 and 81+00 appear to have been due to sliding of the saturated, soft fill soils at the contact between the topsoil/fill and the underlying stiff, clayey silt colluvium. The failures were probably initiated due to infiltrating water from snow-melt and rain fall into the soft, relatively permeable, fill resulting in a loss of strength and due to high seepage forces flow at the contact between the fill/topsoil and the stiff colluvium. The failures did not appear to result in damage to the penstock at the time of our site visit. The penstock appeared to have been placed in the stiff colluvial soils and backfilled with compacted clayey silt. These materials appeared to be relatively stable; however, continued upslope failures could jeopardize the penstock. For this reason we recommend that the following measures be implemented to stabilize the area.

It may be possible to operate the project if it is carefully monitored and the drainage control measures as described below are implemented prior to completion of the permanent slope repairs. If the project is operated prior to implementation of the permanent repairs, the project should be prepared to be shut-down rapidly if continued failure approaches the penstock or if an increased amount of water seepage is observed. In no way should the project operate prior to making the initial immediate repairs as described below.

Immediate Slope Improvement Measures

The following measures should be implemented as soon as possible in the area of the slides:

- Improve Roadway Drainage - The penstock alignment roadway between Stations 78+50 and 84+00 should be regraded to remove all depressions which could pond water and to allow for positive drainage away from the top of the slope. The road should be sloped inward to prevent water discharge and infiltration on the loose fill soils on the outboard side of the penstock road. In addition, a shallow ditch should be excavated on the inboard side of the road from approximately Stations 78+50 to 86+00. The ditch should discharge onto the natural bench in the vicinity of Station 86+00

- Cover Slide Area with Plastic - The slide area should be covered with plastic sheets to prevent additional water infiltration due to snow-melt or rain. The plastic sheets should extend from the road to well below the lower scarp. The plastic sheets should be weighted down with sand bags or other suitable means. Large rocks should not be used because they tend to damage the sheets. It may be necessary to tie the sand bags to a rope which is anchored on the upslope side of the slide in order to prevent the sand bags from sliding down the plastic. A sufficient number of sand bags should be used to prevent the plastic from lifting due to wind.

- Installation of Monitoring Stakes - A series of surveyed monitoring stakes should be installed in and adjacent to the slide area as shown in Figure 2. The stakes should be located horizontally and vertically by survey and monitored at least initially on a daily basis and then weekly during operation of the project prior to completion of the permanent repairs. If the project is not operated, the monitoring stakes should monitored weekly. Vertical and horizontal movement should be recorded. The project should be able to shut-down rapidly and the penstock dewatered if movement is detected in the vicinity of penstock.

- Visual Slope Monitoring - The slide and the slope in the vicinity of the slides should be monitored on a daily basis to observe for any additional movement or any increase in water seepage. This will require lifting the plastic sheets to observe the conditions underneath them. If an increased amount of water seepage is observed, the project should be shut down immediately and the penstock drained, because the increased seepage may be indicative of water leaking from the damaged penstock.

Permanent Repairs

The slope in the vicinity of the failures needs to be further stabilized. At the three previous locations of slope failures, the slopes have been regraded to a more stabile configuration by removing the loose fill at the top of the slope and decreasing the slope angle by placing gravelly/cobbly fill at the bottom of the slope. Because of the previous success with this method in the area, it is feasible to do similar at the location of the recent slides. However, due to soft, saturated condition of the fill, it would be difficult to perform any mass grading in this area until the soils dry out, which could be in several months.

An alternative to mass grading of the slope is to construct a reinforced fill outboard of the penstock. A reinforced fill consists of compacted soil with several layers of geogrid reinforcement that extend into the fill. The geogrid and soil interact to provide a self-supporting structure. Geotextile is used on the outside of the structure to prevent ravelling of the soil fill. Due to the interaction of the geogrid reinforcement and the fill it is possible to construct walls with batters up to 1H:6V (horizontal to vertical). Typically, specialized construction equipment is not required to construct these structures.

For the proposed reinforced fill, we recommend using the reinforcement system illustrated in Figure 3. This section is based on a maximum fill height of 10 feet and a maximum surcharge load of 250 psf.

All existing fill should be removed in the area of reinforced fill. A horizontal bench should be excavated into the stiff colluvium soil. The subgrade should be over-excavated one foot, proof-rolled, and backfilled with one foot of fill compacted to at least 90 percent maximum dry density per ASTM D-1557.

We recommend using a clean, free draining, granular material for structural fill within the geogrid reinforced earthwall. Structural fill should be placed in maximum eight-inch loose lifts and compacted with a large steel-wheeled vibratory roller to at least 90 percent of the maximum Modified Proctor density (ASTM D-1557). Large construction equipment should remain at least three feet from the face of the reinforced fill. Structural fill paced within three feet of the face of the wall should be placed in eight-inch loose lifts and compacted firmly with a minimum of two phases using light, hand operated mechanical compaction equipment.

Construction equipment should not be allowed to operate directly on the reinforcement. A minimum of six-inches of fill should be placed over the reinforcement before equipment may operate over the reinforcement.

The performance of reinforced fill greatly depend on the use of good construction practices. Due to the general absence of local experience with these structures, we strongly recommend that an engineer from our firm be present to observe construction of the proposed reinforced fill.

We recommend that the reinforced fill be constructed between Stations 79+00 and 81+30. In addition, it is our opinion that it is possible to construct the reinforced fill almost immediately. The onsite fill from the vicinity of Station 86+00 appeared to be suitable for use, provided that the rock larger than six inches in longest dimension is removed.

CLOSING

This letter has been prepared for the exclusive use of Ott Engineering and Oxess Hydro for the specific application to this project. The geotechnical recommendations included herein are in general accordance with locally accepted geotechnical engineering practice. There are possible variations in the subsurface conditions with time and between explorations. We recommend that a contingency be planned for unanticipated conditions. In addition, we recommend that an engineer from our firm be onsite during the construction of the reinforced fill (if that option is chosen) to observe subgrade preparation, fill placement and compaction, and the placement of the geogrid.

It has been a pleasure to work with you on this interesting and challenging project. We are available to answer any question you may have on the recommendations included herein or other project issues.

Sincerely,

GOLDER ASSOCIATES INC.

Clifford C. Knitter
Senior Engineering Geologist

Robert L. Plum, P.E.
Associate

cc: Mr. Harry Wolf - Oxess Hydro
Mr. Alvin Carr - Idaho Department of Lands

Attachments

Golder Associates

SUBJECT JIM FORD CREEK

Job No. 903-1041
Ref.

Made by CEK
Checked
Reviewed

Date 19 MARCH 90
Sheet of

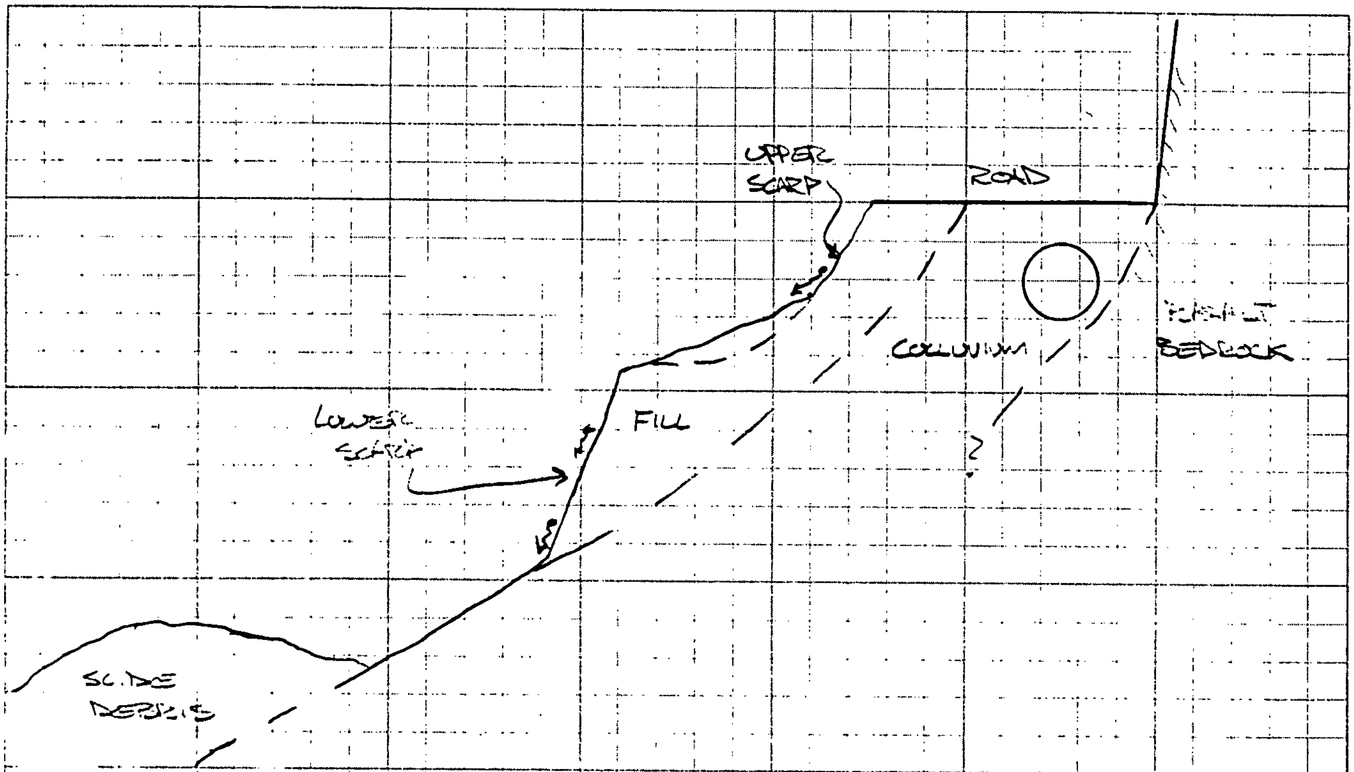


FIGURE 1.

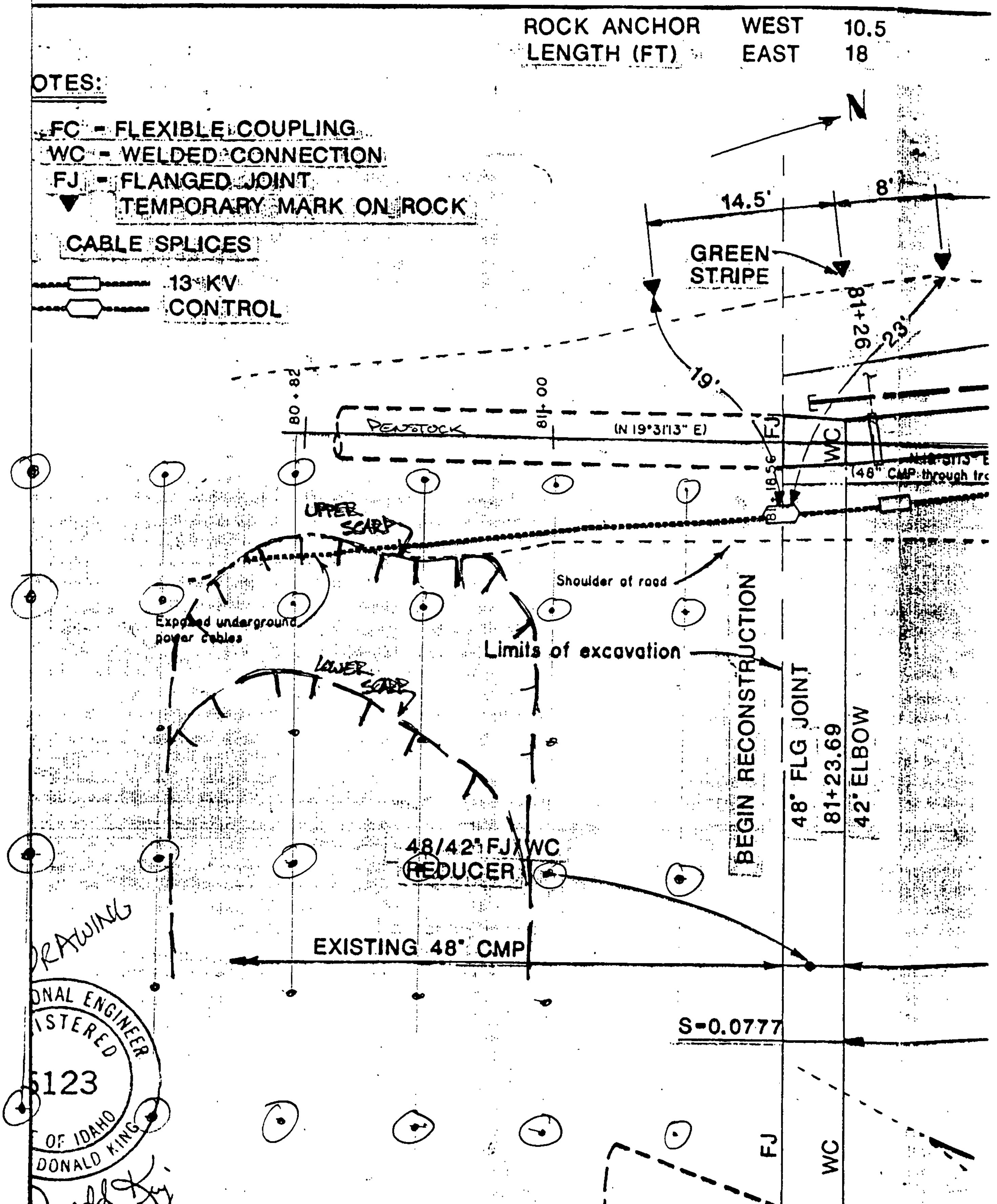
SCHMATIC SECTION THROUGH AREA OF SLOPE FAILURE

From: OH Water Engineers Drawing "Reconstruction of Penstock Sta. 81-18.56 to 83+44.47 Plan and Profile" dated July 11, 1989

FIGURE 2. SITE PLAN

⊙ Approximate Monitoring Stake Location

Scale 1 inch = 10 feet



Golder Associates

SUBJECT JIM FORD CREEK		
Job No. 903-1041	Made by CCK	Date 19 MARCH 90
Ref.	Checked	Sheet 1 of
	Reviewed	

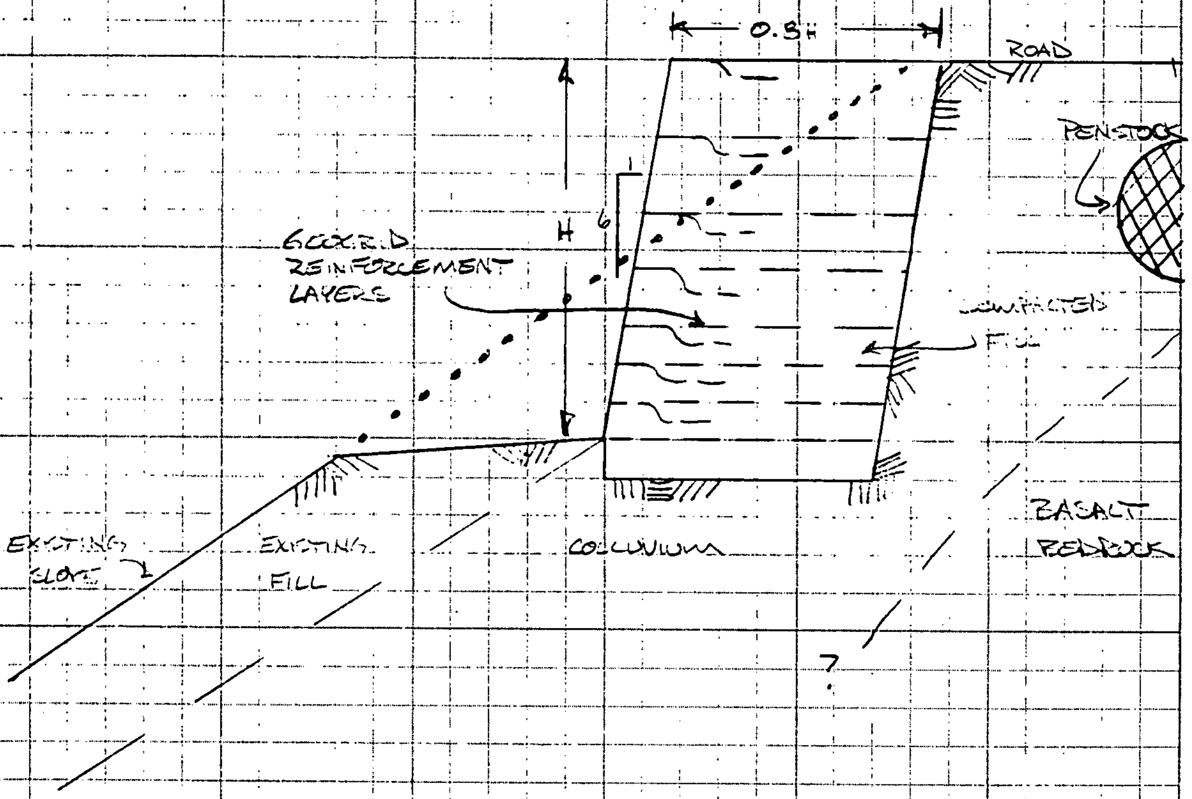


FIGURE 3.

SCHEMATIC SKETCH OF REINFORCED FILL

0 FEET 5

SCALE

Golder Associates

SUBJECT JIM FORD CREEK		
Job No. 903-1041	Made by CCK	Date 19 MAR 90
Ref.	Checked	Sheet 1 of
	Reviewed	

Cliff,

There are two problems here:

1. On page 5 you call for fill to be compacted to 90% D-1557 with a large steel wheel vibratory roller. But you only have 7' of width, which is insufficient for such equipment. Better use hand compactors or something to be pulled by a JD350.

2. How will you get the backslope of the excavation to stand at 1H:6V while you bring up the reinforced fill? Better think about this some more.

Kim
3/24/90

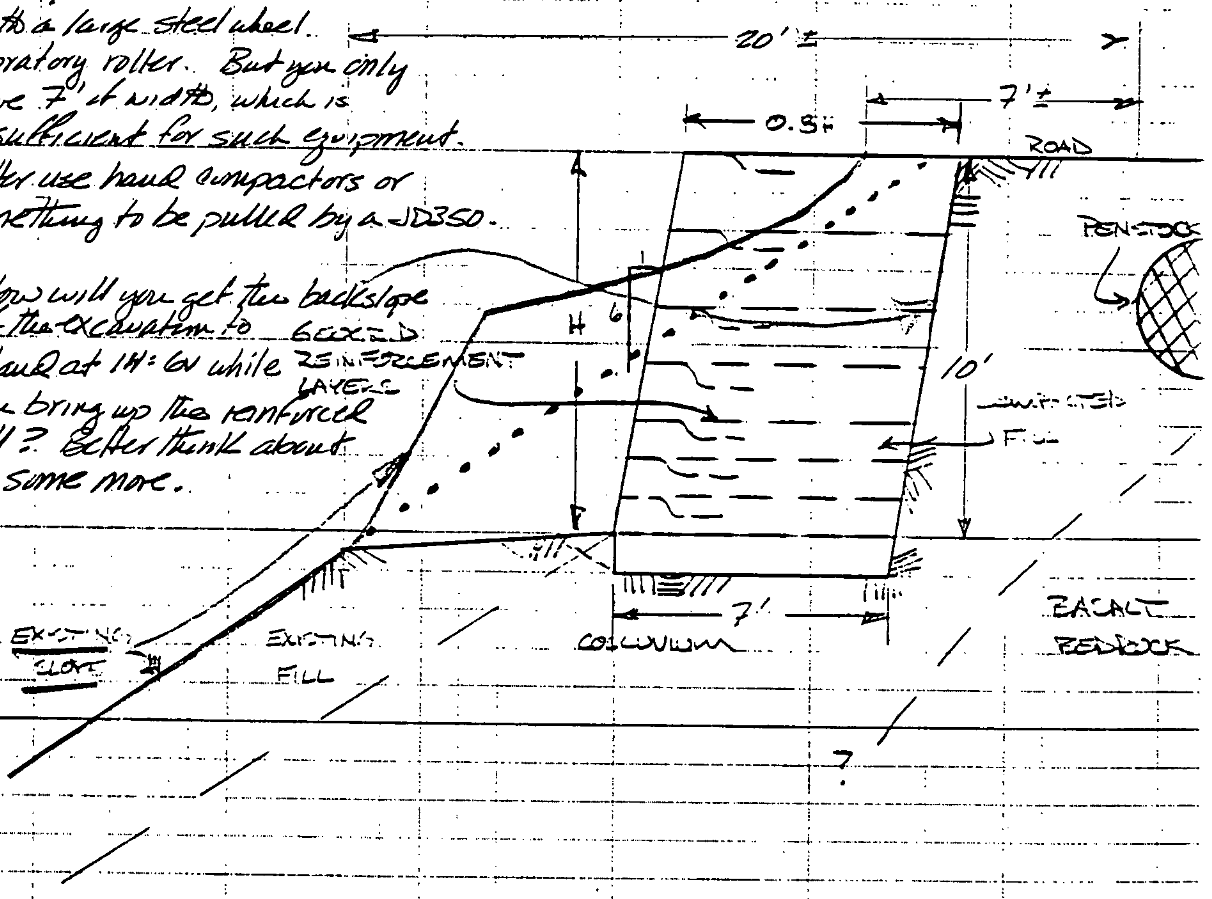


FIGURE 3.

SCHEMATIC SKETCH OF REINFORCED FILL

0 — 5 FEET

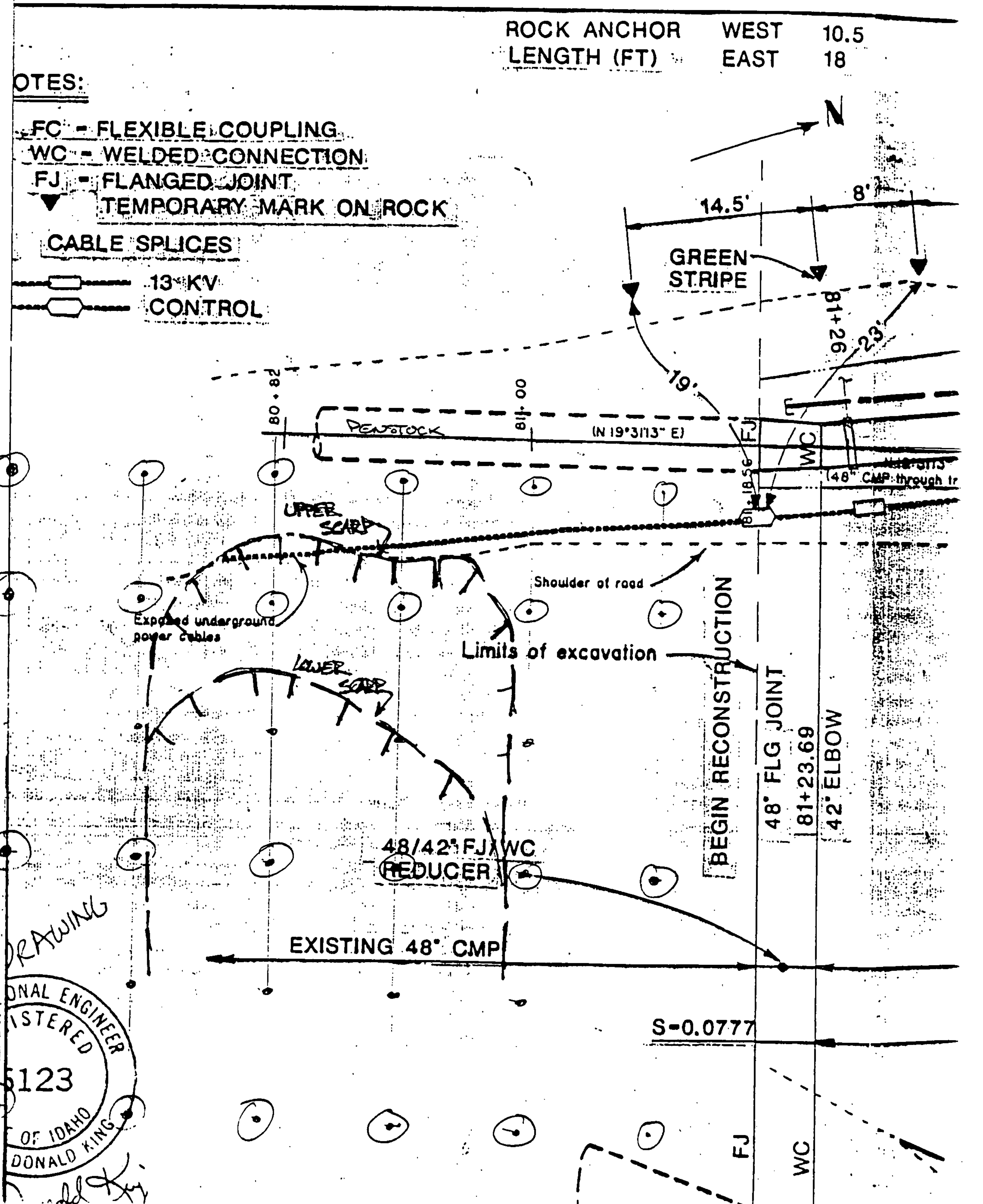
SCALE

From: CH Water Engineers Drawing "Reconstruction of Penstock Sta. 81-18.56 to 82+44.47 Plan and Profile" dated July 11, 1959

FIGURE 2. SITE PLAN

(·) Approximate Monitoring Location

Scale 1 inch = 10 feet



Golder Associates

SUBJECT JIM FORD CREEK		
Job No. 903-1041 Ref.	Made by CCK Checked Reviewed	Date 19 MARCH 90 Sheet of

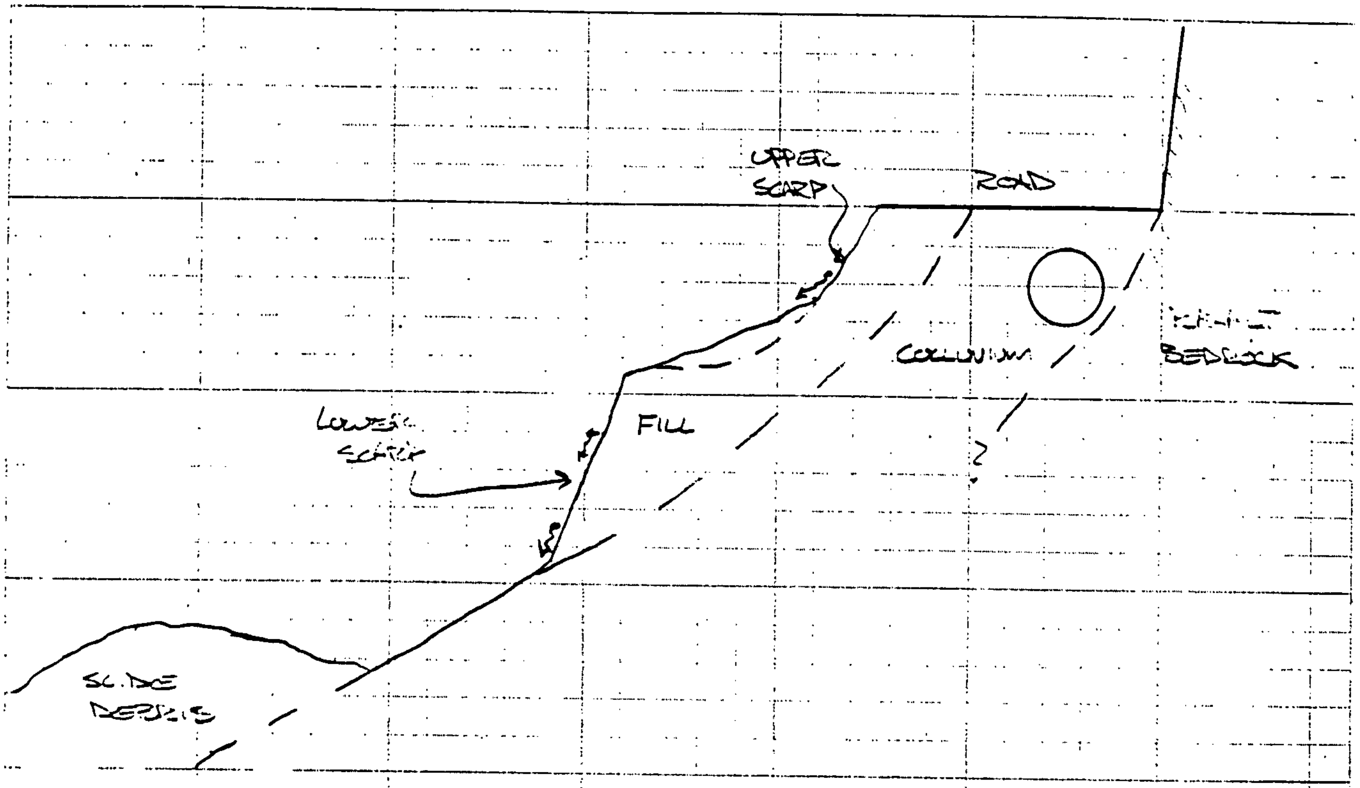


FIGURE 1.

SCHMATIC SECTION THROUGH AREA OF SLOPE FAILURE