NOTICE

All drawings located at the end of the document.

FRENCH DRAIN

GEOTECHNICAL INVESTIGATION

U.S. DEPARTMENT OF ENERGY
Rocky Flats Plant
Golden, Colorado

EG&G ROCKY FLATS
EG&G ROCKY FLATS, INC.

5 OCTOBER 1990

VOLUME 1 - TEXT

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REVIEWED FOR CLASSIFICATION/UCNI
By F. J. Curran U-NU
Date 9-9-94

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EXECUTIVE SUMMARY

This document presents the results of the geotechnical investigation for the construction of an interception trench (french drain) performed as part of the Interim Remedial Action at the 881 Hillside Area, Operable Unit No. 1 (OU1) at the Rocky Flats Plant. This geotechnical investigation is pursuant to the Interim Measures/Interim Remedial Action Plan (IM/IRAP; EG&G, 1990b). The IM/IRAP proposes an interim remedial action to mitigate the release and migration of alluvial ground-water contaminants from the 881 Hillside Area. The proposed remedial action is to collect the contaminated alluvial groundwater from a withdrawal well in an area showing the highest level of ground-water contamination, an outfall sump for the Building 881 footing drain, and a subgrade interception trench (french drain) to be constructed across the base of 881 Hillside. The ground water will be pumped to a new treatment facility and subsequently released to the South Interceptor Ditch.

Sites at the 881 Hillside Area were selected as High Priority sites for investigation and cleanup as a result of Plant-wide characterization activities which showed elevated concentrations of volatile organic compounds in ground water upgradient from Woman Creek (U.S. DOE, 1987a). The Phase I and Phase II RIs indicated that the unconfined ground-water flow system is contaminated. The most pronounced organic contamination is in the eastern portion of the 881 Hillside Area, with tetrachloroethene, trichloroethene, 1,1-dichloroethene, 1,1-dichlorcethane, 1,1,1-trichloroethane, 1,1,2-trichloroethane, and carbon tetrachloride reaching several thousand micrograms per liter in many ground-water samples.

The proposed french drain will be constructed by excavating a trench approximately 2,100 feet along the 881 Hillside, downgradient of the contaminated alluvial ground-water plume. The trench will extend from the surface and be keyed into 2 feet of bedrock exhibiting a permeability of 1×10^{-6} centimeter per second (cm/sec). The geotechnical data presented in this document will be incorporated into the project design plans and specifications.

The geotechnical results of this investigation indicate that relatively impermeable bedrock units, composed of primarily claystone and siltstone, occur beneath the proposed french drain alignment. Occasional thin (less than five-feet thick) "dirty" sandstone units were encountered. However, back pressure permeability analysis of all bedrock units indicated permeability values of less than 1×10^{-6} cm/sec. In-situ permeability values, determined by Packer Testing, indicates several areas along the proposed alignment have horizontal permeabilities, most likely due to fracture or bedding orientation, of greater than 1×10^{-6} cm/sec.

Slope stability analysis, based upon the geotechnical properties of the soil and bedrock units, indicate that short-term slopes of 40° (as measured from the horizontal) can be utilized for construction. Geochemical analysis of soil encountered during the french drain study identified elevated levels of volatile organic compounds, primarily toluene in the majority of the samples selected for analysis. The occurrence of toluene does not appear to be a result of sample or laboratory contamination, and the source of toluene is currently not known. The presence of toluene in the soils to be excavated for the french drain will need to be addressed as a health and safety consideration during construction and as a soil treatment/disposal issue.

Recommendations regarding alignment modifications are presented in Section 5. The modifications include the extension of the western portion of the french drain. This extension, which will require additional investigation, may reduce excavation depth by 15 to 20 feet. An additional recommended alignment change is made to avoid a suspected "healed" slump. This alteration of alignment would occur on the eastern portions of the french drain.

SECTION 1

INTRODUCTION

1.1 BACKGROUND

This report presents results of a geotechnical investigation performed as part of an interim remedial action at the 881 Hillside Area, Operable Unit No. 1 (OU 1) at the Rocky Flats Plant (RFP). The interim remedial action consists of collection of contaminated alluvial ground water in a french drain, transfer via underground pipeline of collected water to a treatment facility, treatment of the water, and transfer via underground pipeline of the effluent to a surface water discharge point. This geotechnical investigation was performed to support the production of plans and specifications for the french drain and associated influent and effluent lines.

Rocky Flats Plant (RFP) is located in northwestern Jefferson County, Colorado, approximately 16 miles northwest of downtown Denver, Colorado. The Plant site consists of 6,550 acres of federally-owned land. Plant facilities are located within a security-fenced area of approximately 380 acres and are surrounded by a buffer zone of approximately 6,170 acres. The RFP is a government-owned, contractor-operated facility, which is part of the nationwide nuclear weapons production complex.

Both radioactive and nonradioactive wastes are generated at the RFP. Current waste handling practices involve on-site and off-site recycling of hazardous materials at another DOE facility. However, both storage and disposal of hazardous, radioactive and mixed radioactive wastes occurred on-site in the past. Preliminary assessments under the Environmental Restoration (ER) Program identified some of the past on-site storage and disposal locations as potential sources of environmental contamination. One of these locations is the 881 Hillside Area.

According to the Federal Facility Agreement and Consent Order [Inter-Agency Agreement (IAG)] between DOE, the Environmental Protection Agency (EPA), and the Colorado Department of Health (CDH), the 881 Hillside Area has the highest priority for remedial action and is designated as OU 1. Elevated concentrations of volatile organic compounds (VOCs) exist in the alluvial ground water and the area is adjacent to a surface water drainage. Twelve Individual Hazardous Substances Sites (IHSS) are included in OU 1 due to their proximity to each other (Figure 1-1).

ER Program activities for OU 1 to date include two phases of remedial investigation (RI), preparation of a draft feasibility study (FS), and preparation of an Interim Measures/Interim Remedial Action Plan (IM/IRAP). An initial (Phase I) RI was completed in 1987, and a draft Phase I RI report was submitted to EPA and CDH in July 1987 (Rockwell International, 1987). Based on results of that investigation, a second phase of field work was conducted at OU 1 in the fall of 1987. In March 1988, a draft Phase II RI report (Rockwell International, 1988a) and draft FS report were submitted to EPA and CDH (Rockwell International, 1988b). A Phase III RI Work Plan is currently being finalized for OU 1.

This geotechnical investigation is pursuant to the IM/IRAP (EG&G,1990b). The IM/IRAP proposes an interim remedial action to mitigate the release and migration of alluvial ground-water contaminants from the 881 Hillside Area. The proposed remedial action is to collect the contaminated alluvial ground water from a withdrawal well in an area showing the highest level of ground water contamination, an outfall sump for the Building 881 footing drain, and a subgrade interception trench (french drain) to be constructed across the base of 881 Hillside. The ground water will be pumped to a new treatment facility and subsequently released to the South Interceptor Ditch (Figure 1-1).

The proposed french drain will be constructed by excavating a trench approximately 2,100 feet along the 881 Hillside, downgradient of the contaminated alluvial ground-water plume. The trench will extend from the surface and be keyed into bedrock. The geotechnical data presented in this document will be incorporated into the project design plans and specifications.

1.2 PURPOSE AND SCOPE

During 1990, the geotechnical investigation was conducted at the 881 Hillside Area to define the geotechnical characteristics along the proposed alignment of the french drain (and any currently proposed extensions) as well as along associated influent and effluent lines to and from the treatment facility.

Additionally, geochemical analyses of soilswere performed to evaluate excavation and backfill health and safety issues. The purpose of this document is to report the findings of this geotechnical investigation.

Specific objectives of the investigation along the proposed french drain alignment are to determine:

- bedrock lithology including identification of sandstone units;
- depth to bedrock;
- appropriate level of personal protective equipment that would be required during construction;
- geotechnical characteristics of area soils and bedrock;
- in situ hydraulic conductivities of bedrock;
- specific in situ hydraulic conductivity of each encountered bedrock sandstone unit greater than three feet in thickness;
- appropriateness of proposed french drain location;
- chemical characteristics of soils along the alignment; and
- slope stability of the entire 881 Hillside area

Objectives of the investigation along the proposed influent/effluent lines are to determine:

- bedrock lithologies including identification of sandstone units;
- depth to bedrock;
- geotechnical characteristics of area soils;
- appropriate level of personal protective equipment that would be required during construction; and,
- appropriate disposal options for excavated soils.

During the geotechnical investigation, a series of 42 borings on approximately 100-foot centers were advanced along the entire length of the proposed french drain, influent/effluent lines, and the french drain extension alignment. Geotechnical testing was conducted on soil and bedrock samples taken from selected borings to assist in designing the french drain collection system. Data collected from the borings also allowed preparation of lithologic logs and geologic cross sections. The latter includes a set of north-south and east-west cross-sections that present the geological and geotechnical conditions along the alignment of the french drain system. Chemical analyses of selected soil and bedrock samples were conducted to determine health and safety requirements for construction, to determine disposal requirements for excavated soil, and to support the remedial investigation of the 881 Hillside Area. The coordinates of the proposed location of the french drain and the influent and effluent pipelines are based on Drawing No. 38548-127, titled General Site Plan and French Drain Re-Survey (Engineering Science, 1990).

SECTION 2

881 HILLSIDE HYDROGEOLOGIC SETTING

2.1 GEOLOGY

2.1.1 Surficial Geology

Surficial materials at the 881 Hillside Area consist of the Rocky Flats Alluvium, colluvium, valley fill alluvium, and artificial fill uncomformably overlying bedrock. In addition, there are a few isolated exposures of claystone bedrock. Figure 2-1 presents the distribution of surficial materials. The study area is located on the south-facing hillside which slopes down from the Rocky Flats terrace toward Woman Creek on the south side of the Plant. Rocky Flats Alluvium caps the top of the slope, and colluvium covers the hillside. Artificial fill and disturbed surficial materials are present around Building 881 and south and east of the building to the South Interceptor Ditch. Artificial fill overlies colluvium at IHSS 130, and surficial materials are disturbed in the vicinity of IHSSs 119.1 and 119.2 Valley fill alluvium is present along the drainage Woman Creek south of the 881 Hillside Area, and terrace alluvium occurs on the north side of the Woman Creek valley fill alluvium.

2.1.1.1 Rocky Flats Alluvium

The Rocky Flats Alluvium is a series of coalescing alluvial fans deposited by braided streams (Hurr, 1976). The erosional surface (pediment) on which the alluvium was deposited slopes gently eastward truncating the Fox Hills Sandstone, the Laramie Formation, and the Arapahoe Formation at the Rocky Flats Plant.

All of the alluvium was removed by erosion in the Woman Creek drainage south of the 881 Hillside Area and in the South Walnut Creek drainage to the north. The result is a terrace of Rocky Flats Alluvium extending eastward from the Plant between the two drainages. This terrace forms the crest of the 881 Hillside Area. Rocky Flats Alluvium occurs north of the study area and east of Building 881 in boreholes P302390, P302490, and P302590. The alluvium ranged from seven feet (P302590) to 19.1 feet (P302490) in thickness. Cross section A-A' (Plate 1) exhibits the alluvial thickness in the vicinity of these boreholes.

2.1.1.2 Colluvium

Colluvium is present on the hillside below the Rocky Flats terrace west and east of Building 881 and extends south to the Woman Creek drainage (Figure 2-1). These materials are deposited by slope wash and downslope creep of Rocky Flats Alluvium and bedrock. Along an east-west trending line, colluvium ranges from four feet (B300290) to twenty feet (B301890) in thickness as shown in cross section A-A' (Plate 1). Colluvial materials along the north-south line shown in cross section B-B' (Plate 1); range from 7.4 feet (B302790) to 28.9 feet (B303290) in thickness.

Colluvial materials on the 881 Hillside have been disturbed by construction of Building 881, various excavation activities associated with the IHSSs, and construction of the South Interceptor Ditch. These areas are shown as disturbed ground on Figure 2-1. Within IHSSs 119.1 and 119.2, shallow excavation took place to construct roadways and to provide level surficial drum storage areas. Colluvium is also disturbed south of Building 881 in the vicinity of IHSSs 106 and 107. This area was excavated during construction of the skimming pond in 1972 (IHSS 107). Finally, colluvium was excavated along the South Interceptor Ditch during its construction from 1979 to 1981.

Colluvium is undisturbed on the eastern part of the hillside south of IHSSs 130, 119.1 and 119.2. The colluvium is thickest in the north-south trending swales draining the 881 Hillside (B303190 and B303290) and thinnest over the intervening ridge (B300190, B300290, and B303490). Colluvium predominately consists of clay with common occurrences of sandy clay and gravel layers.

2.1.1.3 Artificial Fill

A 1937 aerial photograph was used to photogrametrically create a pre-Plant physiographic map. A comparison of current topographic mapping with the earlier 1937 map shows where artificial fill has been added to or soil removed from the 881 Hillside Area (Figure 2-2). The pre-plant physiographic map was of only sufficient quality to provide accurately 10 foot contours. Therefore, topographic disturbance of less than 10 feet cannot be identified. There are three areas of artificial fill on the 881 Hillside derived from separate sources (Figure 2-1 and 2-2). An area southwest of Building 881 had artificial fill added to the surficial deposits during excavation of the building. An area southeast of Building 881 (IHSS 130) was used to dispose of soil contaminated with low levels of plutonium between 1969 and 1972 (EG&G, 1990a). The artificial fill at IHSS 130 overlies natural colluvial materials. A considerable amount of fill was also added to the area directly east of Building 881 and south of the 904 Pad contractor trailer yard to extend the area above the slope.

Material excavated for the Building 881 foundation was spread over a large area generally south of the building. This very poorly sorted and unconsolidated artificial fill was derived from Rocky Flat Alluvium, colluvium, and silty claystone bedrock. The fill is predominantly composed of silty clay with some gravelly zones. Buried topsoil (in place) was encountered beneath the unconsolidated artificial fill in boreholes B302090, B304290, B302190 and B302290. Artificial fill ranged in thickness from 12.3 feet in B302290 to 20.3 feet in B302190.

Artificial fill encountered south and east of Building 881 associated with the western portion of IHSS 130 was encountered in boreholes P302590, P302690, P302790, P302890 and B302990, and ranged from two feet in thickness at borehole P302690 to 4.7 feet in thickness at P302890.

2.1.2 Bedrock Geology

The bedrock beneath the 881 Hillside consists of claystones with interbedded lenticular sandstones, siltstones, and occasional lignite deposits. The bedrock sediments were deposited by meandering streams flowing generally from west to east off the ancestral Front Range. Sandstones were deposited in stream channels and as overbank splays, and claystones were deposited in back swamp and floodplain areas. Leaf fossils, organic matter, and lignite beds were encountered within the claystones during previous drilling at the 881 Hillside. Contacts between various lithologies are both gradational and sharp. Based on preliminary results of the ongoing high resolution seismic reflection program at RFP, bedrock is dipping less than two degrees to the east.

2.2 HYDROLOGY

2.2.1 Ground-Water Hydrology

Unconfined ground-water flow occurs in surficial materials and subcropping sandstones on the 881 Hillside (EG&G, 1990a). Subcropping claystone appears to be unsaturated based on the evidence of numerous dry wells completed in weathered claystone just below the bedrock contact. Confined ground-water flow occurs in deeper sandstone units of the bedrock (EG&G, 1990a). These two ground-water systems are discussed in greater detail below.

2.2.1.1 Unconfined Flow System

Ground water at the 881 Hillside Area is present in the colluvium and subcropping sandstone under unconfined conditions (EG&G, 1990a). Recharge to the water table occurs as infiltration of incident precipitation and as seepage from the South Interceptor Ditch.

The shallow ground-water flow system is quite dynamic, with large water level changes occurring in response to precipitation events. Water levels are highest during the spring and early summer months due to heavier precipitation and snow melt. Water levels decline during late summer and fall, and many colluvial wells located near the eastern part of the french drain go dry during this time of year. Figures 2-3, 2-4 and 2-5 depict potentiometric conditions in surficial materials in January, April and July 1990, respectively. At the Rocky Flats Alluvium pediment edges, ground water emerges as seeps and springs at the contact between the alluvium and claystone bedrock (contact seeps). The water is consumed by evapotranspiration, or flows southeast through colluvial materials following topography toward the valley fill and terrace alluviums of Woman Creek. During the driest portions of the year, evapotranspiration can result in no flow in either the colluvium or the valley fill alluvium.

2.2.1.2 Confined Flow System

Confined ground water is present in deep sandstones in the unweathered bedrock at the 881 Hillside area. However, confined ground water was not encountered during this french drain geotechnical study due to the limited drilling beneath the bedrock contact.

2.2.2 Surface Water Hydrology

At the Rocky Flats Alluvium pediment edges, ground water emerges as seeps and springs where the contact between alluvium and bedrock claystone outcrops near the ground surface. This water flows as surface water for short distances where it is consumed by evapotranspiration or it enters the colluvial unconfined flow system.

The surface water on the 881 Hillside is ephemeral due to response of spring runoff and storm events. During these events, seeps and springs flow on the far eastern part of the 881 Hillside. Also, a small drainage ditch near IHSS 102 has intermittent flow into the South Interceptor Ditch.

SECTION 3

881 HILLSIDE GEOTECHNICAL INVESTIGATION RESULTS

3.1 GEOTECHNICAL PROPERTIES

Samples were collected for laboratory geotechnical testing including grain size/hydrometer analysis, moisture-density, direct shear strength, triaxial compressive strength, unconfined compressive strength, back pressure permeability, Atterberg limits, and consolidation/swell analysis. In situ packer tests for determination of permeability are also presented in this section.

Cross section A-A' (Plate 1) exhibits the borehole along the east-west trending line. Colluvial materials along the north-south line shown as cross section B-B' (Plate 1); range from 7.4 feet (B302790) to 28.9 feet (B303290) in thickness. Borehole logs are presented in Appendix A. The results of the geotechnical tests are presented in Appendix B. A summary table of geotechnical test results for surficial materials is provided in Table 3-1 and for bedrock materials in Table 3-2.

3.1.1 Grain Size/Hydrometer Analysis

Grain size analysis was performed to properly classify the specific geologic units encountered during the geotechnical field investigation. Gradation is a descriptive term which refers to the distribution and size of grains in a soil matrix (U.S. Department of Interior, 1971). The amounts of each particle size group are determined by laboratory tests usually referred to as the mechanical analysis of a soil. The amounts of the gravel and sand fractions are determined by sieving, using ASTM D-1140; silt, clay and colloid contents are determined by sedimentation (hydrometer analysis) using ASTM D-422 (ASTM, 1989). The results of these tests are presented in the form of a cumulative grain-size curve in which particle sizes are plotted on a logarithmic scale with respect to percentage, by weight, of the total sample plotted on a linear scale (Appendix B-1). For soil consisting mainly of coarse grains, the grain-size distribution reveals physical properties of the material. As an example, gradation of coarse-grained soils are described as 1) well-graded (poorly sorted) if there is a good representative of all particle sizes, and 2) poorly-graded (well sorted) if most particles are the same size.

TABLE 3-1

COLLUVIAL\SURFICIAL MATERIALS GEOTECHNICAL RESULTS

Borehole Number	Test Interval (ft)	GRAIN SIZE (%) Sand	Silt	Clay	Lithologic Name	MOISTURE-DENSITY Moisture % dry wt	Dry Density (lb/ft^3)	Wet Density (lb/ft^3)	DIRECT SHEAR STRENGTH (lb/ft^2) Normal/Peak	ϕ Angle (degrees)	UNCONFINED COMPRESSIVE STRENGTH (lb/ft^2)	ATTERBERG LIMITS liquid/plastic/plastic limit / limit / index	ID
B300590	10.6-11.3	25	20	55	Silty sandy clay	16.9	112.6	131.6					
B300790	6.0-6.3	9	29.5	61.5	Silty clay	14.3	119.3	136.3					
P302390	8.45-8.7					5.9							
P302390R	8.5-9.7B					7.4	120.2	129.1			2848		
	8.5-7.7B					6.8							
P302490	8.23-8.57					15.7	96.4	111.5					
P302490R	4.0-4.8M					20.0	105.9	127.1			4207		
	4.0-4.8M					14.4	113.5	129.8					
P302590	6.0-7.0B					9.9							
P302690	9.7-10.7B					20.3	108.1	130.0			6537		
	9.7-10.7M					22.8	100.1	122.9					
P302790	3.7-4.6B					25.0	100.9	126.1			2437		
	3.7-4.6M					23.6	102.5	126.6					
	4.7-6.2B					23.0	104.3	128.3			4833		
	4.7-6.2M					28.3	96.8	124.2					
P302890	4.0-4.7B					25.6	90.9	114.1			1016		
	4.0-4.7M					27.8	97.7	124.8					
B302990	4.0-5.5B					23.9	99.5	123.3			3843		
	4.0-5.5M					22.5	95.7	117.2					
B303090	4.2-5.2B					21.0	102.4	123.9					
	4.2-5.2M					18.9	108.3	128.8			6586		
B303190	4.85-5.85					26.1	100.8	127.2			1950		
B303290	4.0-4.9B					17.1	108.6	127.2			6304		
	4.0-4.9M					17.1	105.8	123.8					
B303790	5.3-6.3	41	15	44	Sandy clay	14.3	94.7	108.5	1000 / 1720	40		53/15.4/37.6	CH

TABLE 3-1 (continued)

COLLUVIAL/SURFICIAL MATERIALS GEOTECHNICAL RESULTS

Borehole Number	Test Interval (ft)	GRAIN SIZE (%) Sand	Silt	Clay	Lithologic Name	MOISTURE-DENSITY Moisture % dry wt	Dry Density (lb/ft^3)	Wet Density (lb/ft^3)	DIRECT SHEAR STRENGTH (lb/ft^2) Normal/Peak	ϕ Angle (degrees)	UNCONFINED COMPRESSIVE STRENGTH (lb/ft^2)	ATTERBERG LIMITS liquid/plastic/plastic limit / limit / index	ID
B303990	3. 7-5.2					18.5	109.8	130.1	1000 / 4963	39.5			
	10.7-11.2	19	56	25	Clayey silt	12.7	disturbed sample						
	11.2-12.1					15.5	109.2	126.1	1000 / 1660	41		28.6/17.9/10.7	CL
B304090	4.5-5.5	3	28	69	Silty clay	19.9	106.5	127.6	1000 / 3000	33		71.0/20.0/51.0	CH
B304190	4.0-4.5	21	30.5	48.5	Silty sandy clay	17.3	108.8	127.6				53.5/16.5/37.0	CH
	8.0-9.0					22.5	104.0	127.4	1000 / 1620	10			
	12.7-13.5					21.7	103.9	126.5	1000 / 2950	11			
B304290	8.8-9.5	7	36	57	Silty clay	15.5	95.9	110.8				57.5/18.5/39.0	CH
	12.7-13.4					22.4	95.5	116.9	1000 / 1630	21.5			
	16.7-17.6	32	19.5	48.5	Sandy clay	27.0	90.7	115.1	1000 / 2550	45			
	20.8-21.7	9	42	49	Silty clay	17.3	110.9	130.1					

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TABLE 3-2

BEDROCK GEOTECHNICAL RESULTS

Borehole Number	Test Interval (ft)	Grain Size (%) Sand	Grain Size (%) Silt	Grain Size (%) Clay	Lithologic Name	Moisture-Density Moisture % dry wt	Moisture-Density Dry Density (lb/ft³)	Moisture-Density Wet Density (lb/ft³)	Direct Shear Strength (lb/ft²) Normal/Peak	ϕ Angle (degrees)	Triaxial Compressive Strength (lb/ft²) Normal/Peak	Unconfined Compressive Strength (lb/ft²)	Back Pressure Permeability (cm/sec)
B300190	10.5-10.8	46	29	25	Silty clayey sandstone	14.7	114.9	131.8					
	16.8-17.3					16.6	114.6	133.6					
	23.0-23.4					15.2	116.2	133.8				5387	
	26.1-26.7					14.4	120.5	137.9				10497	
B300290	16.9-17.2					15.8	115.1	133.3					
	19.9-20.5	12	43	45	Silty claystone	16.0	113.4	131.5				4341	
	24.5-25.0					15.5	116.0	134.0					
	26.7-27.0					16.2	116.5	135.3				4943	
B300390	15.2-15.8					17.4	113.2	133.0					
	29.5-30.0					19.7	107.9	129.1					
B300490	12.6-13.1					14.7	122.0	139.9				10481	
	18.2-19.1					20.3	110.2	132.6				6510	
	28.0-28.6					20.7	107.6	129.8				5145	
B300590	10.6-11.3	25	20	55	Silty sandy claystone	16.9	112.6	131.6					
	14.5-14.8					16.6	111.3	129.7					
	21.2-21.9	1	34	65	Silty claystone	18.6	111.3	132.0				15271	
B300690	12.5-13.2					14.0	120.6	137.5					
	18.5-19.0					19.3	114.5	136.5				14969	
	19.0-19.5	16	59	25	Clayey siltstone		21.9						1.6×10^{-7}
	19.5-19.9					22.7	111.6	136.8					
B300790	10.0-10.4					19.1	113.0	134.6					
	16.0-16.8	0	45	55	Silty claystone	17.8	117.1	137.9				3854	
	18.6-18.8					14.6	117.5	134.7				25132	
	20.8-21.5	1	56	43	Clayey siltstone	15.3	116.2	133.9				16596	6.0×10^{-9}
B300890	15.1-15.4					19.3	108.8	129.9	1700 / 3460	44			
	26.1-27.7					11.3	124.5	138.6				132754	
B300990	19.3-20.1					16.4	112.9	131.5	2000 / 3220	40	2000 / 12630		
	24.3-25.1					20.3	109.0	131.1	1500 / 3820	40			

TABLE 3-2 (continued)

BEDROCK GEOTECHNICAL RESULTS

Borehole Number	Test Interval (ft)	Grain Size (%) Sand	Silt	Clay	Lithologic Name	Moisture-Density Moisture % dry wt	Dry Density (lb/ft^3)	Wet Density (lb/ft^3)	Direct Shear Strength (lb/ft^2) Normal/Peak	ϕ Angle (degrees)	Triaxial Compressive Strength (lb/ft^2) Normal/Peak	Unconfined Compressive Strength (lb/ft^2)	Back Pressure Permeability (cm/sec)
B301090	25.5-26.0					23.2	101.7	125.3					
	29.4-30.0					20.1	108.8	130.7					3.1×10^{-8}
	31.1-31.5	0	20	80	Silty claystone							3439	
B301190	23.6-24.0	41	44	15	Sandy siltstone	20.2	105.9	127.4					
	23.6-24.0	16	43	41	Clayey siltstone								2.4×10^{-8}
	31.1-32.0	32	38	30	Sandy clayey siltstone	14.5	116.8	135.8				10408	
	35.0-35.5	21	50	29	Sandy clayey siltstone	15.7	120.1	138.9					1.1×10^{-8}
B301290	9.7-9.9					19.4	108.0	129.0	900 / 2650	44			
	18.0-18.5					17.1	113.7	133.1					
B301390	23.8-24.4	13	62	25		16.5	111.4	129.8			1000 / 9506		
	28.8-29.4					17.9	115.4	136.1					
	30.8-31.2					13.2	119.3	135.1					1.9×10^{-7}
B301490	20.7-20.9					15.6	113.9	131.7				15565	
	23.7-23.9					16.1	110.6	128.4					
	28.3-28.6					16.2	114.1	132.6					
	30.3-31.3					19.4	109.0	130.2			3100 / 12963		3.1×10^{-8}
B301590	26.75-27.10	42	40.5	17.5	Silty sandstone								
B301690	29.6-30.0	10	58	32	Clayey siltstone								1.5×10^{-8}
	32.4-32.7	36	47	17	Sandy siltstone								2.4×10^{-8}
	37.0-37.3	2	42	56	Silty claystone								7.1×10^{-8}
B301790	20.2-20.9					20.0	110.9	133.2			2000 / 10356		
	29.6-29.9					17.5	114.2	134.2				6666	
B301890	27.1-27.9					18.9	111.7	132.8					
	28.9-29.3	19.5	55.5	25	Clayey siltstone		16.8	115.2	134.5			12063	
	33.3-33.7	12	53	35	Clayey siltstone		13.7	116.5	132.5				
B301990	26.8-27.7	40	41	19	Sandy siltstone	9.7	112.8	123.8	1000 / 2760	57			
	29.6-30.3	28	51	21	Sandy clayey siltstone								
	36.6-37.5					10.8	127.0	140.7					7.0×10^{-8}
B302090	40.2-41.5					21.8	105.5	128.5			1000 / 86771	6799	

TABLE 3-2 (continued)

BEDROCK GEOTECHNICAL RESULTS

Borehole Number	Test Interval (ft)	GRAIN SIZE (%) Sand	Silt	Clay	Lithologic Name	MOISTURE-DENSITY Moisture % dry wt	Dry Density (lb/ft^3)	Wet Density (lb/ft^3)	DIRECT SHEAR STRENGTH (lb/ft^2) Normal/Peak	ϕ Angle (degrees)	TRIAXIAL COMPRESSIVE STRENGTH (lb/ft^2) Normal/Peak	UNCONFINED COMPRESSIVE STRENGTH (lb/ft^2)	BACK PRESSURE PERMEABILITY (cm/sec)
B302190	40.0-40.3	50	37	13	Silty sandstone								
B302290	29.3-30.4					17.9	114.7	135.2			1000 / 15368		
B303790	30.8-31.5					14.2	124.6	142.2	1000 / 3780	42.5			
	30.8-31.5	19	50	31	Clayey siltstone		15.9	109.6	127.0				$4.0x10^{8}$
B303890	26.5-27.7					12.6	125.6	141.5				24416	
B303990	13.9-14.4					18.6	104.8	124.3	1000 / 2050	30			
B304090	9.4-10.0					20.0	102.6	123.1	1000 / 2580	32			
B304190	16.4-17.1					23.1	102.4	126.0	1000 / 2500	13.5			
B304290	24.9-25.6					20.4	99.6	119.9	1000 / 2040	47.5			

The Wentworth scale for grain-size classification was used for the bedrock textural classification; sieve mesh #230 (0.0625 mm) was the lower limit of sand size, and sieve mesh #10 (2.0 mm) was the upper limit of sand size. The clay fraction is considered to be less than 0.004 mm on the hydrometer test.

Surficial materials observed on the 881 Hillside generally consist of clay with occasional sand and gravel lenses. Geotechnical grain size analysis of these materials reflects the field log observation (Figure 3-1).

The predominant bedrock lithologies encountered during drilling were claystones and siltstones. It should be noted that permeable, well-sorted sandstones were not observed. Grain size/hydrometer sample analysis was performed to confirm the presence or absence of sandstone. This selective sampling thus skewed the results toward the more coarse-sized fraction (Figure 3-2). Actual sandstone to claystone/siltstone ratios could be assumed to be substantially less than the 13 percent (3 sandstones out of 24 claystones/ siltstones) presented on Figure 3-2.

It should also be noted that the field logs have been changed based on the laboratory grain size/hydrometer analysis. However, the original field notebooks do not reflect this change. Only the lithology that was tested for grain size analysis was changed on the borehole log; similar field lithology descriptions in the same borehole were not altered.

3.1.2 Moisture - Density Analysis

To evaluate in situ material density and moisture saturation, moisture/density tests were performed on most of the geotechnical samples collected for the 881 Hillside project. The wet density of the material (if in situ moisture density) is necessary for computation of rock mass stress. Moisture-density values are determined using ASTM methods D-2216 and D-2937, respectively (ASTM, 1989). Moisture is the most influential factor affecting the properties of soil and rock, and it is also the principal factor subject to change either from natural or intentional causes. The amount of moisture (water) is always given as a percentage of the dry weight of the soil.

Density is the weight of a unit volume of a rock mass. It can be expressed as either a wet or dry density. Wet density is the unit weight of the solid particles and the contained moisture expressed in pounds per cubic foot. Wet density is generally close to the actual natural (in-place) density of the rock and is close to the density expected to be encountered during french drain construction. Dry density is normally used for expressing the unit weight of rock; it is a fixed quantity independent of moisture change.

The results of the moisture-density tests are given in Appendix B-2. The average moisture content observed for surficial materials was 18.4 percent, and for bedrock was 17.2 percent. The average wet density for surficial material was 124.2 lb/ft^3 and for bedrock was 132.3 lb/ft^3.

3.1.3 Direct Shear Strength

To evaluate, the potential for movement of rock and soil scopes the natural cohesive strength and internal friction angles of the material must be evaluated. The engineering computations for the strength of a soil or rock deal primarily with its shear strength; that is, the resistance to sliding of one mass of soil or rock against another, and rarely with the compressive or tensile strength. Shear strength is a function of both internal friction and cohesion. Internal friction is the resistance to sliding within a soil or rock mass. Internal friction is that part of the shear strength of a rock that depends on the magnitude of the normal stress on a potential shear fracture. Generally,internal friction increases with sand and gravel content and decreases with increasing moisture content. Cohesion is the attraction between individual particles due to molecular forces and the presence of moisture films. Cohesion is very high in clay, but of little or no significance in silt and sand. The stability and hence the structural properties of soil are affected to a large extent by internal friction and cohesion. These combine to make up the shearing resistance. ASTM Method D-3080 was used to perform this test (ASTM, 1989). These tests were performed using various confining pressures in order to evaluate the shear strength failure envelope.

The results of the direct shear strength tests are given in Appendix B-3. The average direct shear strength for surficial materials was 1622 lb/ft^2, and for bedrock materials was 1705 lb/ft^2.

3.1.4 Triaxial Compression Strength Test

The triaxial test is considered to provide the best soil parameters and stress-strain data. It is necessary to test the sample using in situ pore pressure if correct soil parameters are to be obtained. A soil or rock specimen is subject to a constant lateral pressure while a vertical axial load is applied. The vertical load is increased to failure of the specimen. The test data are analyzed graphically by use of Mohr circles to determine the cohesion and internal friction of the specimen. The results of the triaxial test are used in various formulas to determine the load-carrying capacity of the soil. The test is further described in ASTM D-2850 (ASTM, 1989).

The average of the triaxial compressive strength test for bedrock was 24597 lb/ft^2. Triaxial tests were not performed on surficial material.

3.1.5 Unconfined Compressive Strength Test

The unconfined compressive strength test is performed to determine the in situ strength of a cohesive soil at natural moisture conditions. The test is similar to the triaxial compression test except no lateral pressure is applied. A vertical axial force is applied until the sample fails along a shear plane or bulges. The failure will tend to develop along the weakest portion of the sample in contrast to direct shear which impose a shear plane. The vertical strains or deformations are measured as the applied load is increased. The results are presented in a summary table or by a stress-strain curve along with sketches or photos of the failed sample. The shear strength is assumed to equal half the compressive strength. The ASTM Method D-2166 was used for soils and D-2938 was used for bedrock (ASTM, 1989). The average of the unconfined compressive strength test for surficial material was 4056 lb/ft^2 and for bedrock was 11507 lb/ft^2.

3.1.6 Back Pressure Permeability

Back pressure permeability is used to determine the saturated hydraulic conductivities of soils or rocks. An approximate value for hydraulic conductivity may be obtained in the laboratory using a constant-head permeability test. This laboratory test usually gives only vertical hydraulic conductivity values.

The back pressure permeability test used was EPA Method 9100 (U.S. EPA, 1986). The results of the test data are presented in Appendix B-6. The bedrock permeabilities ranged between 1.5 x10^{-6} cm/sec on depth interval 26.75-27.10 feet for borehole B301590 down to 6.0 x10^{-9} cm/sec in depth interval 18.6-18.8 feet in borehole B300790. The average of these values was 1.80 x 10^{-7} cm/sec. The Interim Remedial Action Plan specifies the french drain is to be keyed into 2 feet of bedrock with a hydraulic conductivity of 10^{-6} cm/sec or less. These hydraulic conductivity values of discrete samples usually fall below the 10^{-6} cm/sec criterion. No permeability tests were performed on surficial material.

3.1.7 Atterberg Limits

Atterberg limits are used to define the boundaries between the four states of fine-grained material as follows: 1) liquid limit is the liquid/plastic boundary; 2) plastic limit is the plastic/semi-solid boundary; 3) shrinkage limit is the semi-solid/solid boundary. The testing was in accordance with ASTM Method D-4318 (ASTM, 1989).

The values obtained from the liquid and plastic limits furnish a basis for soil classification when placed on an Atterberg limits cross plot. The water-plasticity ratio determines the strength of a material. A material with a high water-plasticity ratio has a low strength. Semi-empirical relationships between material properties and the limits can be used to make predictions on the behavior and properties of a soil that has similar limits without performing difficult and lengthy tests of compressibility, permeability, and strength.

Atterberg limits were only performed on five alluvial samples taken along the french drain alignment and the results of the tests are provided in Appendix B-7. Of the five tests, four fell into the CH category of the Unified Soil Classification System (ASTM 2487) and one was a CL. The four CH type soils all had a high plasticity index from 37.0 to 51.0, high liquid limits of 51.0 to 71.0 and plasticity limits of 16.5 to 20.0 percent moisture. The one CL type soil had a relatively low plasticity index of 10.7, a low liquid limit of 28.6, and a plasticity index of 17.9.

3.1.8 Consolidation/Swell Test

The consolidation/swell tests are performed to determine the compressibility and elasticity or expansion of surficial and bedrock material in accordance with ASTM method D-2435. The testing is performed to evaluate the effects of the material properties on structures in contact with the surficial or bedrock materials.

The testing is performed by calculating a minimal vertical axial load, then the sample is laterally confined with vertical porous membranes to allow removal of hydrostatic pressure, and the vertical axial load is applied until consolidation reaches stasis. The sample is then re-wetted and expansion of the sample is measured by restraining gauges. Additional vertical axial load is added until the original size of the sample is attained. Two consolidation/swell tests were performed and the results of the tests are presented in Appendix B-8. Results indicate that bedrock and colluvium is comprised of expansive material.

3.1.9 In Situ Packer Testing

In situ packer tests were performed in boreholes drilled along the proposed french drain alignment to characterize the hydraulic conductivity of weathered bedrock at the 881 Hillside Area. The results of the packer tests are used to determine the macro hydraulic conductivity as a result of rock discontinuities. This data will then be used to evaluate the IRAPs requirement for bedrock hydraulic conductivities no greater than 10^{-6} cm/sec. Test procedures and results are presented below.

3.1.9.1 Packer Test Procedures

The tests were conducted in cored sections of an open bedrock borehole to isolate and determine hydraulic conductivity at depths of approximately five, ten, and fifteen feet below the alluvium/bedrock contact. Increments of the borehole were isolated by means of inflatable rubber packers. Water was then pumped into the section of borehole between the packer seals under pressure to determine values for permeability of the rock matrices encountered. Thick sandstone units (greater than five feet) were not observed during the bedrock coring, therefore discrete packer tests were not conducted exclusively in sandstone. Only thin, clayey silty sandstone interbedded with claystone was encountered during the french drain field investigation.

Double packer tests were accomplished by rotary coring the bedrock to the total depth and then packer testing the three five foot sections from the contact down. In theory, this practice seemed to be the most time efficient approach. However, following cave in of boreholes B300390 and B300490 caused by the recurrent damage of borehole side walls during packer inflation and deflation, it was decided that single packer tests would provide equivalent data and reduce the potential for borehole collapse. Single packer tests were achieved by coring the bedrock to the desired depth and then packer testing the interval using a single packer at the top of the test interval.

A rule of thumb for maximum allowable test pressure to the test zone is 1 psi per foot of overburden when testing above the water level, and 0.57 psi per foot of depth when testing below the water level. These limits assume an overburden unit weight of 144 pounds per cubic foot (pcf) and that there are no naturally occurring excess water pressures.

The maximum pressure is obtained using the overburden pressure of 1 psi per foot of overburden as P_{inj} (100 percent).

$$P_{ovb} = P_{wc} + P_g + P_{gh} = P_{inj}\ (100\%)$$

To calculate P_g for packer testing

$$P_g = P_{ovb} - P_{wc} - P_{gh}$$

Where:

P_{inj} = injection pressure
P_{wc} = Pressure (water column) in psi
P_{gh} = Pressure (gauge ht) in psi
P_{ovb} = Pressure (overburden)
P_g = Pressure (gauge)

and where P_{wc} = (depth to top of test interval in ft) (0.433 psi/ft)

and P_{gh} = (height of gauge above ground level in ft) (0.433 psi/ft)

Generally tests were performed at P_g's of 12 percent, 50 percent, and 100 percent. If flow occurred at a particular P_g then the previous P_g was retested to observe any flow increases at the lower pressure.

Hydraulic conductivity values are calculated from the results of the packer tests and field data as follows (U.S. Department of the Interior, 1974):

$$K = \frac{Q}{2\pi LH} \ell n \frac{L}{r} \qquad = \frac{Cp\,Q}{H}$$

Where:

K	=	Hydraulic conductivity in ft/yr
Q	=	Constant rate of water loss in gal/min
L	=	Length of test interval in ft
H	=	Total head acting on the test section in ft
r	=	Radius of the borehole in ft
ℓn	=	Natural log
Cp	=	a constant based upon the diameter of the borehole and length of the test interval.

Values of Cp are given in the following table for Nx core diameter and various lengths of test intervals. (U.S. Department of the Interior, 1974).

Length of test interval L (Feet)	Cp
1	23,300
2	15,500
3	11,800
4	9,700
5	8,200
6	7,200
7	6,400

The limit of the calculated hydraulic conductivity depends on the duration of the test and the amount of pressure being applied. The limits were calculated by using the minimum accurate detectable gauge increment of water loss of 0.05 gallons. If no water loss occurred during packer testing, the final result of hydraulic conductivity was given as being "less than" the computed value. It was determined that for two-minute tests using excessive overburden pressures, a hydraulic conductivity of 10^{-6} cm/sec was the test limit. For 12-minute tests at normal overburden pressure, 10^{-6} to 10^{-7} cm/sec was attainable. As necessary, 12-minute tests were run. As described in the Interim Remedial Action Plan, it is only necessary for design of the french drain to know if the hydraulic conductivity is greater or less than 10^{-6} cm/sec.

On occasion, the inflatable packer could not be tightly sealed against the rock due to irregularities in the borehole cross section or insufficient packer inflation pressure. The borehole could have been damaged due to water circulation while coring and/or borehole wall failure of incompetent rock. In these instances, water leaked between the packer and the borehole wall. During double packer testing intervals 13.99 to 20.25 feet and 16.11 to 22.37 feet in borehole B300190, and intervals 22.07 to 27.34 feet and 25.03 to 30.30 feet in borehole B301090 indicated breach of the packer seals. Some of the packer seals were also broken during single packer testing.

3.1.9.2 Results of Packer Tests

In situ packer testing evaluates horizontal hydraulic conductivity. Water loss occurs through dilation of bedding planes, fractures, and faults (if present), in addition to migration through permeable units (i.e., sandstones). Due to site specific geologic conditions, both single and double packer tests were performed.

Double packer tests were performed on boreholes B300190, B300290, B300590, B300690, B301090 and B301490. In theory, this practice seemed to be the most time efficient approach. However, due to the cave in of boreholes B300390 and B300490, it was determined that single packer tests would provide acceptable data and reduce the potential for borehole collapse. Results of the packer testing are shown in Table 3-3. Most of the packer tests performed showed no water loss. In these cases, a hydraulic conductivity result of less than a value was assigned to these intervals.

Several packer test intervals did show water loss (Table 3-4). Hydraulic conductivity values for test intervals 14.95 to 21.25 feet and 18.09 to 24.35 feet in borehole B300290 were calculated at 2.2×10^{-3} cm/sec and 2.3×10^{-3} cm/sec, respectively. Hydraulic conductivity values for test intervals 16.5 to 20.3 feet and 20.3 to 25.3 feet for borehole B301590 were determined at 8.5×10^{-4} cm/sec and 4.2×10^{-5} cm/sec, respectively. Other water loss intervals include 19.5 to 24.5 feet in borehole B301290 and 22.07 to 27.35 feet in borehole B301090, with hydraulic conductivity values of 1.1×10^{-4} cm/sec and 6.4×10^{-5} cm/sec, respectively. Several other test intervals showed minor water loss (Table 3-4).

As is apparent from the previous discussion, several boreholes (B300290, B300890, B301090, B301290 and B301590) contain intervals that do not meet the IRAP's requirement of a hydraulic conductivity less than 10^{-6} cm/sec. However, it must be emphasized that the discrete bedrock units individually do meet the IRAP requirements based upon back pressure permeability data.

TABLE 3-3

PACKER TEST RESULTS
881 HILLSIDE
FRENCH DRAIN GEOTECHNICAL STUDY

Borehole	Depth to Bedrock (ft)	Test Interval (ft)	Length of Test Interval (ft)	Permeability (ft/yr)	Permeability (cm/sec)	Lithology	Comment	Single or Double Packer
B300190	6.0	13.99 - 20.25	6.26			claystone, siltstone + sandstone	lost packer seal	D
		16.11 - 22.37	6.26			claystone, siltstone + sandstone	lost packer seal	D
		18.82 - 25.08	6.26	<1.6	$<1.5 \times 10^{-6}$	claystone + siltstone		D
B300290	6.0	14.95 - 21.25	6.26	2228	2.2×10^{-3}	claystone + siltstone		D
		18.09 - 24.35	6.26	2399	2.3×10^{-3}	silty claystone		D
B300390	10.5	N/A	N/A	N/A	N/A		borehole caved in	N/A
B300490	7.7	N/A	N/A	N/A	N/A		borehole caved in	N/A
B300590	11.5	9.71 - 14.98	5.27	< 2.9	$<2.8 \times 10^{-6}$	clay + claystone	no water loss	D
		14.76 - 20.03	5.27	< 3.8	$<3.7 \times 10^{-6}$	silty claystone	no water loss	D
		17.13 - 22.4	5.27	< 4.7	$<4.5 \times 10^{-6}$	silty claystone	no water loss	D
B300690	12.5	10.1 - 15.37	5.27	< 3.4	$<3.3 \times 10^{-6}$	clay, claystone + siltstone	no water loss	D
		14.98 - 20.25	5.27	< 3.8	$<3.7 \times 10^{-6}$	claystone + silty sandstone	no water loss	D
		20.1 - 25.37	5.27	0.9	8.4×10^{-7}	claystone, siltstone + sandstone		D
B300790	3.7	10.12 - 13.0	2.88	<5.2	$<5.0 \times 10^{-6}$	claystone + sandstone	no water loss	S
		13.0 - 17.0	4	< 8.7	$<8.4 \times 10^{-6}$	silty claystone	no water loss	S
		17.0 - 23.0	6	< 4.9	$<4.8 \times 10^{-6}$	claystone	no water loss	S
B300890	12.0	16.26 - 19.5	3.24	< 6.1	$<5.9 \times 10^{-6}$	claystone	no water loss	S
		19.5 - 25.0	5.5	< 3.5	$<3.4 \times 10^{-6}$	silty claystone	no water loss	S
		24.5 - 29.5	5.0	20.7	2.0×10^{-5}	silty claystone + claystone		S
B300990	12.1	14.3 - 19.3	5.0	< 2.7	$<2.6 \times 10^{-6}$	claystone	no water loss	S
		19.3 - 24.3	5.0	< 1.3	$<1.3 \times 10^{-6}$	claystone	no water loss	S
		24.3 - 29.3	5.0	< 2.0	$<2.0 \times 10^{-6}$	claystone	no water loss	S
B301090	21.0	22.07 - 27.34	5.27	65.7	6.4×10^{-6}	claystone		D
		25.03 - 30.30	5.27	N/A	N/A	claystone	lost packer seal	D
B301190	18.6	23.0 - 28.0	5.0	< 6.9	$<6.6 \times 10^{-6}$	claystone + clayey silstone	no water loss	S
		28.0 - 33.0	5.0	< 5.5	$<5.3 \times 10^{-6}$	claystone + siltstone	no water loss	S
		34.0 - 39.0	5.0	< 3.6	$<3.5 \times 10^{-6}$	claystone + siltstone	no water loss	S

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TABLE 3-3 (Continued)

PACKER TEST RESULTS
881 HILLSIDE
FRENCH DRAIN GEOTECHNICAL STUDY

Borehole	Depth to Bedrock (ft)	Test Interval (ft)	Length of Test Interval (ft)	Permeability (ft/yr)	Permeability (cm/sec)	Lithology	Comment	Single or Double Packer
B301290	9.5	9.5 - 14.5	5.0	< 1.6	$<1.5 \times 10^{-6}$	claystone	no water loss	S
		14.5 - 19.5	5.0	< 1.0	$<1.0 \times 10^{-6}$	claystone	no water loss	S
		19.5 - 24.5	5.0	112.3	1.1×10^{-4}	sandy claystone		S
B301390	13.0	14.8 - 18.8	4	3.6	3.5×10^{-6}	claystone		S
		18.8 - 23.8	5	1.8	1.7×10^{-6}	claystone + silty sandstone		S
		24.8 - 28.8	4	3.6	3.5×10^{-6}	claystone + silty sandstone		S
		28.8 - 33.8	5	< 0.5	$<5.0 \times 10^{-7}$	claystone	no water loss	S
B301490	16.1	18.0 - 23.27	5.27	< 5.1	$<4.9 \times 10^{-6}$	claystone	no water loss	D
		22.7 - 27.97	5.27	< 6.2	$<6.0 \times 10^{-6}$	claystone, siltstone + sandstone	no water loss	D
		27.5 - 32.77	5.27	< 2.4	$<2.3 \times 10^{-6}$	claystone	no water loss	D
B301590	13.5	16.5 - 20.3	3.8	878.8	8.5×10^{-4}	claystone		S
		20.3 - 25.3	5.0	43.7	4.2×10^{-5}	claystone + siltstone		S
		25.3 - 30.3	5.0	2.4	2.3×10^{-6}	claystone + sandstone		S
		26.3 - 30.3	4.0	8.2	8.0×10^{-6}	claystone + sandstone		S
B301690	20.5	22.8 - 27.6	4.8	< 0.7	$<6.6 \times 10^{-7}$	claystone + siltstone	no water loss	S
		24.75 - 32.9	8.15	< 0.4	$<4.3 \times 10^{-7}$	claystone, siltstone + sandstone	no water loss	S
		32.9 - 37.8	4.9	0.5	4.4×10^{-7}	claystone, sandstone + siltstone		S
B301790	17.0	22.76 - 24.6	1.84	< 1.3	$<1.3 \times 10^{-6}$	claystone	no water loss	S
		24.6 - 29.6	5	< 0.6	$<5.8 \times 10^{-7}$	sandy claystone	no water loss	S
		29.6 - 34.6	5	< 0.5	$<4.8 \times 10^{-7}$	claystone	no water loss	S
		34.6 - 39.6	5	< 0.4	$<4.1 \times 10^{-7}$	claystone	no water loss	S
B301890	20.0	23.5 - 26.9	3.4	< 0.9	$<8.3 \times 10^{-7}$	claystone	no water loss	S
		26.9 - 31.9	5.0	4.0	3.9×10^{-6}	claystone + siltstone		S
		31.9 - 36.9	5.0	< 0.5	$<4.5 \times 10^{-7}$	claystone + siltstone	no water loss	S
		37.9 - 41.9	4.0	< 0.5	$<4.5 \times 10^{-7}$	claystone	no water loss	S
B301990	19.6	22.6 - 26.6	4.0	< 0.8	$<7.4 \times 10^{-7}$	silty claystone	no water loss	S
		26.6 - 31.6	5.0	< 0.6	$<5.4 \times 10^{-7}$	sandy claystone	no water loss	S
		31.6 - 36.6	5.0	< 0.6	$<5.6 \times 10^{-7}$	silty claystone	no water loss	S
		38.6 - 41.6	3.0	< 0.6	$<5.3 \times 10^{-7}$	silty claystone	no water loss	S
B302090	25.0	29.0 - 32.0	3.0	< 0.8	$<7.2 \times 10^{-7}$	silty claystone	no water loss	S
		32.0 - 37.0	5.0	< 0.6	$<5.5 \times 10^{-7}$	silty claystone	no water loss	S
		33.0 - 37.0	4.0	< 0.6	$<5.2 \times 10^{-7}$	silty claystone	no water loss	S
		38.0 - 42.0	4.0	< 0.5	$<4.5 \times 10^{-7}$	silty claystone	no water loss	S

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TABLE 3-3 (Continued)

PACKER TEST RESULTS
881 HILLSIDE
FRENCH DRAIN GEOTECHNICAL STUDY

Borehole	Depth to Bedrock (ft)	Test Interval (ft)	Length of Test Interval (ft)	Permeability (ft/yr)	Permeability (cm/sec)	Lithology	Comment	Single or Double Packer
B302190	27.5	32.0 - 35.2	3.2	< 0.6	$<6.2 \times 10^{-7}$	claystone	no water loss	S
		35.2 - 40.0	4.8	< 0.4	$<4.2 \times 10^{-7}$	claystone	no water loss	S
		40.0 - 45.0	5	< 0.4	$<3.6 \times 10^{-7}$	claystone + siltstone	no water loss	S
B302290	17.5	23.0 - 27.0	4	< 0.8	$<7.4 \times 10^{-7}$	claystone	no water loss	S
		27.0 - 32.0	5	< 0.6	$<5.4 \times 10^{-7}$	claystone	no water loss	S
		32.0 - 37.0	5	< 0.5	$<4.5 \times 10^{-7}$	claystone	no water loss	S
B303790	17.0	17.0 - 22.0	5	< 0.9	$<8.2 \times 10^{-7}$	silty claystone	no water loss	S
		22.0 - 27.0	5	< 0.7	$<6.6 \times 10^{-7}$	claystone	no water loss	S
		27.0 - 32.0	5	< 0.6	$<5.4 \times 10^{-7}$	silty claystone	no water loss	S
B303890	10.4	14.0 - 18.0	4	< 1.2	$<1.2 \times 10^{-6}$	claystone	no water loss	S
		20.0 - 23.0	3	< 1.1	$<1.0 \times 10^{-6}$	claystone	no water loss	S
		23.0 - 28.0	5	< 6.2	$<6.2 \times 10^{-7}$	claystone	no water loss	S

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TABLE 3-4

Test Intervals With Water Loss

Well No.	Test Interval in ft	Hydraulic Conductivity ft/yr	cm/sec	Comments
B300290	14.95 - 21.25	2228	2.2×10^{-3}	
	18.09 - 24.35	2399	2.3×10^{-3}	
B300690	20.1 - 25.37	0.9	8.4×10^{-7}	
B300890	24.5 - 29.5	20.7	2.0×10^{-5}	
B301090	22.07 - 27.34	65.7	6.4×10^{-5}	
B301290	19.5 - 24.5	112.3	1.1×10^{-4}	
B301390	14.8 - 18.8	3.6	3.5×10^{-6}	
	18.8 - 23.8	1.8	1.7×10^{-6}	
	24.8 - 28.8	3.6	3.5×10^{-6}	
B301590	16.5 - 20.3	878.8	8.5×10^{-4}	
	20.3 - 25.3	43.7	4.2×10^{-5}	
	26.3 - 30.3	8.2	8.0×10^{-6}	
B301690	32.9 - 37.8	0.5	4.4×10^{-7}	indistinct water loss
B301890	26.9 - 31.9	4.0	3.9×10^{-6}	

3.1.9.3 Comparison of In Situ Packer Test Results with Laboratory Back Pressure Permeabilities

Table 3-5 compares the results of the packer tests with the laboratory back pressure permeability data. The packer test results are generally one to two orders of magnitude more permeable. These results occur in part because back pressure permeabilities measure the vertical hydraulic conductivity which is generally less than horizontal hydraulic conductivity. This condition, under which one or more of the hydraulic properties of an aquifer vary according to the direction of flow, is caused by the orientation of clay minerals in sedimentary rock (Freeze & Cherry, 1979). Additionally, packer tests evaluate rock discontinuities. These discontinuities or zones of weakness would naturally not be selected for discrete permeability analysis.

3.1.10 Summary

An objective of the geotechnical investigation for the french drain is to better characterize geologic units using extensive laboratory analysis. These analyses were used to provide textural data, in particular grain size analysis, to determine rock and soil classifications. In addition, moisture, density and permeability tests of bedrock will aid in geologic classification and interpretation. The following paragraphs briefly describe the classification of the three prominent units encountered during the field program.

The alluvial soils that were encountered only along the northern portions of the influent/effluent line, are indistinguishable from the colluvial soil present in the remainder of the borings. Geotechnical analysis classifies this material as a CH/CL inorganic clay with varying amounts of sand, silt and occasional gravel. This material did not exceed 10 feet in depth in the three borings in which it was encountered.

Colluvial soils developed on slopes will make up the majority of the material that will be excavated for construction of the french drain. This material, as is the alluvial soil, is classified as a CH/CL inorganic clay of moderate plasticity with varying amounts of coarse fraction sand, silt and occasional gravel. The wet density of the soil averages 124 pcf with a moisture content of approximately 19 percent. The average unconfined compressive strength of the soil is approximately 4000 psf. However, the individual tests vary from 1000 psf to over 6000 psf. This variability is expected considering the mixed composition of the soil. The material averages 15 feet thick over the french drain alignment. The thickest soil profile occurred on the

TABLE 3-5

COMPARISON OF IN SITU PACKER TESTS WITH LABORATORY BACK PRESSURE PERMEABILITIES PERFORMED ON BEDROCK

Borehole Number	Interval of Packer Test (ft)	Permeability of Packer Test (cm/sec)	Field Lithology Description	Interval of BPP Test (ft)	Permeabilities of BPP Test (cm/sec)	Grain Size Analysis Results
B300690	14.98-20.25	$<3.7x10^{-6}$	claystone & silty sandstone	18.5-19.0	$1.6x10^{-7}$	N/A
B300709	17.0-23.0	$<4.8x10^{-6}$	claystone	18.6-18.8	$6.0x10^{-8}$	N/A
B301090	25.03-30.3	N/A	claystone	25.5-26.0	$3.1x10^{-8}$	N/A
B301190	23.0-28.0	$<6.6x10^{-5}$	claystone & clayey siltstone	23.6-24.0	$2.4x10^{-8}$	sandy siltstone
B301190	34.0-39.0	$<3.5x10^{-6}$	claystone & siltstone	35.0-35.5	$1.1x10^{-8}$	sandy clayey siltstone
B301390	N/A	N/A	sandy claystone & silty sandstone	23.8-24.4	$1.9x10^{-7}$	clayey siltstone
B301490	27.5-32.77	$<2.3x10^{-6}$	claystone	28.3-28.6	$3.1x10^{-8}$	N/A
B301590	26.3-30.3	$8.0x10^{-6}$	claystone & sandstone	26.75-27.1	$1.5x10^{-6}$	silty sandstone
B301690	24.75-32.9	$<4.3x10^{-7}$	claystone, siltstone & sandstone	29.6-30.0	$2.4x10^{-8}$	clayey siltstone
B301690	32.9-37.8	$<4.4x10^{-7}$	claystone, sandstone & siltstone	32.4-32.7	$7.1x10^{-8}$	sandy siltstone
B301990	26.6-31.6	$<5.4x10^{-7}$	sandy claystone	29.6-30.3	$7.0x10^{-8}$	sandy clayey siltstone
B303790	27.0-32.0	$<5.4x10^{-7}$	silty claystone	30.8-31.5	$4.0x10^{-8}$	clayey siltstone

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western portion of the drain alignment where it is composed of inplace and disturbed fill material. Here the depth to bedrock exceeds 28 feet of in-place and disturbed CH/CL soil including some bedrock fragments. Due to the cohesive characteristics of these materials, shallow (<5') near vertical slopes can be maintained for short periods. However, desiccation of clay will cause eventual failure of steep slopes.

Bedrock, comprised of claystone, siltstone and minor sandstones is discussed as a single unit. This combination was done to facilitate slope construction. However, the units, although of multiple classifications, are so closely related in both textural and particle size distribution that geotechnically these units will act as a single rock mass. The average wet density of the bedrock is approximately 132 pcf, with a moisture content of approximately 17.2 percent. The average unconfined compressive strength of the rock exceeds 11,000 psf. However, the direct shear test cohesion averaged less than 2000 psf. This variability reflects the lenticular horizontal bedding of the rock mass. In situ permeability testing (packer tests) of the bedrock identified zones of permeability in excess of 10^{-6} cm/sec. However, back pressure permeabilities were never greater than 1.0 x 10^{-6} cm/sec. The comparison of these two results indicates that fractures and other rock discontinuities are the primary source of rock mass permeability, not saturated flow through the units. This fracture permeability is particularly well displayed in boring B300290 where in situ water loss indicates a permeability of 1.0 x 10^{-3} cm/sec. Review of the boring logs indicate that a potential "healed" slump occurs in the vicinity of borings B300290 through B300590. Water loss and the caving of the boreholes correlate to a gravel zone within the bedrock. This positioning of gravels within bedrock was observed numerous times during slump mapping (Section 4.1.1), which lead to the conclusion that this feature might be a "healed" slump. Excavation into bedrock will seldom exceed 4 to 5 feet. However, some cuts will be approximately 10 feet deep into bedrock.

3.2 <u>GEOCHEMISTRY OF SURFICIAL MATERIALS</u>

The rationale and design for the proposed french drain is based on the results of previous geochemical investigations of the 881 Hillside Area. These results are summarized briefly below. Analyses of samples collected from the boreholes drilled along the french drain and influent and effluent lines provide additional data for evaluating the proposed french drain location. These results are presented in Appendix C. The presence in soil of toluene in samples from numerous intervals of the french drain boreholes represents the most significant indication of VOCs in the immediate vicinity of the french drain.

3.2.1 Previous Investigations

Prior analyses of organic constituents in soils of the 881 Hillside have provided qualitative data indicating the presence of methylene chloride, acetone, and phthalates in most samples, and tetrachloroethylene (PCE), trichloroethene (TCE) 1,1,1-trichloromethane (1,1,1-TCA) at a few boreholes (Rockwell International, 1988). Toluene was not detected in previous investigations . These data were rejected during data validation primarily due to small sample size, and there are remaining uncertainties regarding laboratory contamination of methylene chloride, acetone and phthalates, so the previous investigations cannot provide quantitative definition of organic contaminants in the soils. VOCs in the ground water of the 881 Hillside are concentrated in two areas which are close to IHSSs and upgradient of the french drain. The principal ground-water contaminants are PCE, TCE, carbon tetrachloride (CCl_4), and their degradation products. Toluene is not a characteristic ground-water contaminant at the 881 Hillside Area; the highest of the three "detectable" toluene results in the 1988 Rockwell International RI/FS study was 2J $\mu g/l$ ("J" signifies that the analyte was estimated below detection limit).

Several metals and radionuclides in soils were reported to be present at concentrations above the preliminary background levels established by the draft Background Geochemical Characterization Report (Rockwell International, 1989b). However these analytes do not exhibit recognizable gradients or strongly elevated concentrations (EG&G, 1990a), and therefore did not govern the location of the french drain. Similarly, several metals and radionuclides exceed preliminary background levels in 881 Hillside ground water, but do not display patterns or pose hazards which govern the french drain location.

3.2.2 Geochemistry of Soils at the Proposed French Drain Location

Boreholes drilled along the proposed french drain and influent and effluent lines during this investigation were sampled for analysis of volatile organic compounds using stainless steel cylinders inserted at the bottom of each two foot core barrel run. In addition, approximately four-foot interval composite samples were made over the remaining recovered core for metal, radionuclide, semivolatile, and pesticide/PCB analyses. Larger intervals were composited when core recovery was poor.

Toluene, methylene chloride, and acetone were the only VOCs detected in the boreholes drilled during the french drain investigation. Toluene is the most significant of this group because it occurs at multiple intervals in the majority of boreholes at concentrations ranging up to 1200 $\mu g/kg$ (at B303390). Methylene chloride and acetone, in contrast, typically occur at relatively low concentrations (less than five times the detection limit) and are present in laboratory blanks, and therefore may not indicate actual soil contamination, but a laboratory containment. Higher concentrations in boreholes B3000190 and B300290 may represent true contamination according to contract laboratory program (CLP) criteria for evaluating low levels of these compounds. Trichloroethane was reported at 1J $\mu g/kg$ for only one sample in one borehole (B301090) and is therefore not considered a contaminant.

The distribution of toluene in the french drain boreholes does not show horizontal or vertical gradients over the depths sampled (up to approximately 20 feet). However, several of the samples with the highest concentrations (>400 $\mu g/l$) were collected within the top four feet of soil, along most of the proposed french drain alignment. High within-hole fluctuations generally indicate a random vertical distribution of toluene based on these data. Field, trip, and laboratory blanks did not contain toluene, nor did rinsates of the hollow stem auger used in drilling and of the stainless steel cylinders used for sampling inside the coring device (field and trip blank data are provided in Appendix C).

Two semi-volatile compounds were detected in french drain samples, although only bis(2-ethylhexyl)phthalate was a common constituent. Bis(2-ethylhexyl)phthalate typically occurred at concentrations between 40-100 $\mu g/kg$, was estimated below the detection limit (flagged "J") and was present in the blank (flagged "B"). The maximum concentration was 1400 B $\mu g/kg$ in B301590. The presence of this compound is consistent with previous investigations of the 881 Hillside, and similarly involves some uncertainty as to whether there is actual phthalate contamination or that the contamination is due to sample handling with surgical gloves. This latter possibility will be tested in the Phase III remedial investigation of the 881 Hillside (EG&G, 1990a), but does not directly influence plans for the interim action. Phenanthrene was detected in two samples (38J $\mu g/kg$ in the 0-8 foot composite from B303390; and 42J $\mu g/kg$ in the 0-9.7 foot composite from B303590). Volatile and semi-volatile results are not yet available for boreholes B303990 through B304290.

Metals concentrations (presented in Appendix C) are consistent with the ranges reported in previous investigations, and do not influence the positioning of the french drain. Radionuclide data have not yet been received from the laboratory.

3.2.3 Geochemical Considerations for Interim Action Design

The volatile organic compounds which were detected in previous investigations of soils and ground water at the 881 Hillside Area have not migrated to the proposed drain location. However, toluene, which was not detected in soils during previous investigations, is present in soils along the proposed drain location. In conclusion, the presence of toluene has implications pertaining to worker safety during french drain construction, and final disposition of excavated materials. However, the presence of toluene does not alter the proposed location of the french drain.

Although the concentrations of toluene are low, the presence of the compound in some of the soils that will be excavated during french drain construction may require that the soils be handled as potentially hazardous wastes. Thus disposal of excavated soils at the 881 Hillside Area, is not anticipated to be an acceptable plan.

Two attributes of the toluene distribution in the soils sampled during this drilling program are particularly important for evaluating the proposed french drain location:

- The toluene concentrations are typically on the order of a few hundred $\mu g/l$. The absence of corresponding toluene concentrations in ground-water monitoring wells in the vicinity confirms that these soil concentrations do not pose a strong risk of potential desorption to ground water exiting the 881 Hillside Area. The soils' high clay content provides further assurance that the sorption capacity for toluene is strong (Dragun, 1988).
- The absence of any clear gradient in the toluene distribution in soil and the absence of toluene in local ground water suggests that it is not presently migrating from a large-volume, high concentration, upgradient source in the vicinity. More importantly there is no indication from these data or from previous investigations that a high concentration pulse of toluene or other VOCs has already bypassed the french drain location.

In conclusion, the geochemical evidence discussed above indicates that the proposed french drain location should not be altered. Such a change could cause major delays which would preclude the primary goal of this interim action--to rapidly institute a mechanism for mitigating migration of contaminants in alluvial ground water.

SECTION 4

SLOPE STABILITY EVALUATION

4.1 LANDSLIDE (SLUMP) GEOMORPHOLOGY

Landslide (slump) features were mapped at the Rocky Flats Plant to investigate local slope characteristics which may influence construction activities associated with the french drain installation. Slump features in general are characterized by evidence of downward and backward curvilinear movement of a soil or rock mass on a slope (Figure 4-1). Typically there is intermittent movement of the primary block and secondary slumping resulting in terraced or nested slumps. The base of slump features may exhibit soil liquidfication, particularly in clayey materials saturated with water.

Slumping is a natural result of valley widening by lateral erosion where oversteepening and undercutting of slopes by stream activity occurs. Slumping may be accelerated by seeps and springs that commonly occur at the outcrop of the contact between bedrock and surficial materials at pediment edges; by construction activities that oversteepen or undercut slopes with drainage ditches and/or roadcuts; by overloading of slopes with artificial fill or water saturation; and/or by fluctuation of water levels at the base of slopes due to ponding or changes in stream discharge.

4.1.1 Mapping Procedures

Slumps were mapped in the Rock Creek, Walnut Creek, and Woman Creek drainages (Plate 2). This mapping was restricted to active or recent slump features and distinct, older, weathered slumps where the headwall, base or toe, and lateral boundary could be easily distinguished. Such conditions were most common in the upper narrow valleys. Older "healed" slumps could also be inferred in the broader valleys based on hummocky topography, but these were generally not mapped during this study.

The mapping was conducted using a Brunton compass, steel tape and topographic maps (1:500 scale). Slope strike was referenced to true north, and slope dip was measured adjacent to the limbs of the

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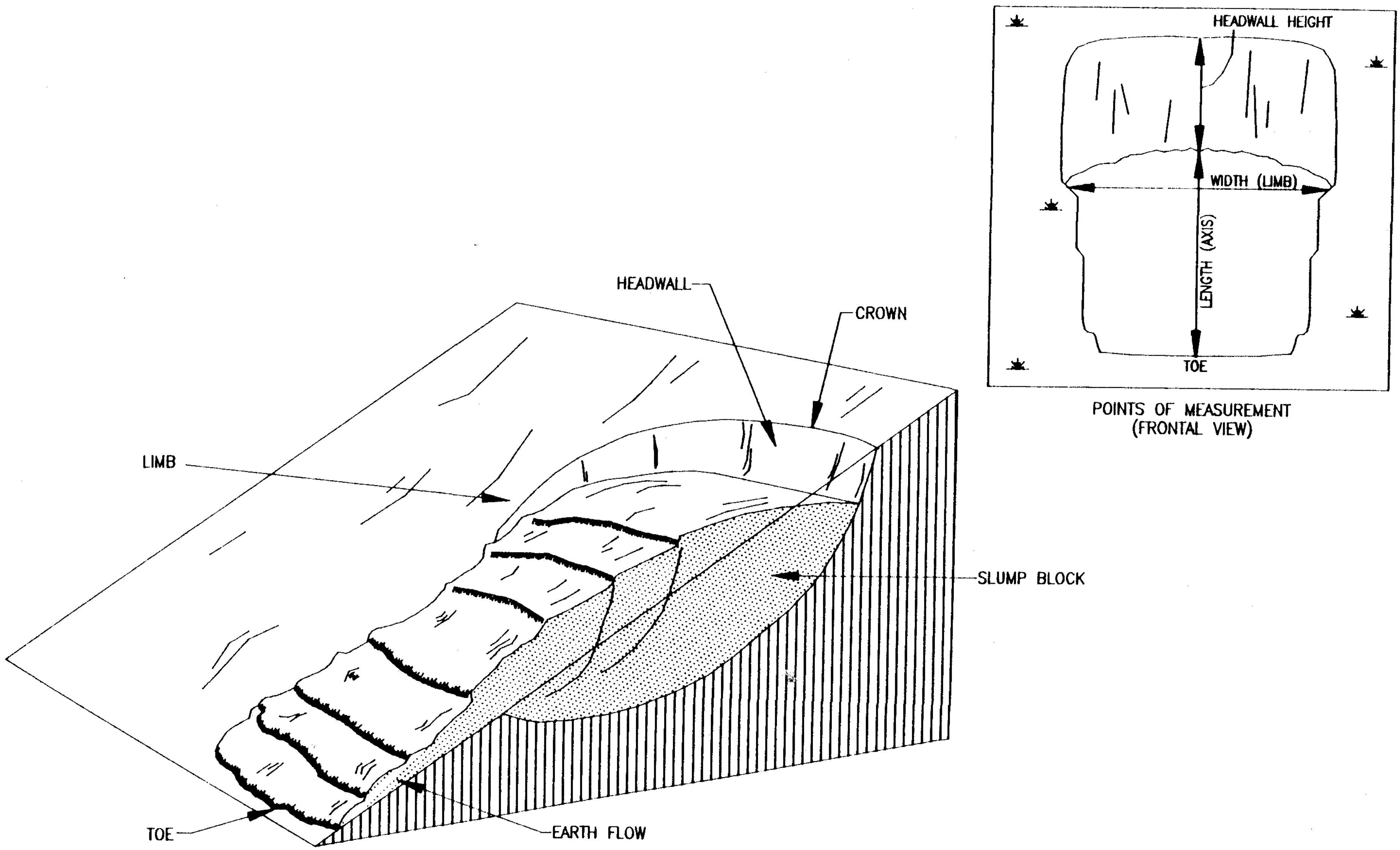

POINTS OF MEASUREMENT
(FRONTAL VIEW)

FIGURE 4-1
SLUMP FEATURES

(NOT TO SCALE)

slump that represented the original slope. The apparent critical slope angle, (i.e., the local maximum slope inclination which the soil and rock materials underlying the slope can support without failure), was determined. Volume calculations were based on measurements of the length, width, and headwall height using a steel tape. The slump length is the distance from the base of the headwall to the toe of the slump. The width is the distance between the limbs of the scarp at the points where they were parallel to slump movement direction. The headwall height was measured from the base to the crown of the headwall scarp at the central axis of the slump. Only one slump feature observed in the Woman Creek area was directly measured (WMCN5), because access to other slumps in that drainage was restricted. However, qualitative observations and estimates of slump dimensions were possible from outside the restricted area.

4.1.2 Slump Distribution and Characteristics

Natural slumping and slumping induced by RFP activities have occurred in both the northern and southern portions of the steam drainages. Four main types of slumps (A,B,C, and D), based upon genesis and form were identified from the field mapping results (Plate 2). Thirty individual slumps were identified during the mapping exercise (Tables 4-1 through 4-4).

Type A slumps, the simplest form, involve slopes which terminate in a drainage area without seeps or springs and without construction activities (Figure 4-2). Of the ten type A slumps that were identified at RFP, five were in the North Walnut Creek drainage (WCN3, WCN4, WCN5, WCN8, and WCN9), three were in the Rock Creek drainage (RCN1, RCN2, and RCS3), and two were in the Woman Creek drainage (WMCN2 and WMCN4). The apparent critical slope dip angle, ranged from 17 degrees for RCS3 to 25 degrees for WCN4 (Table 4-1). Volumes ranged from 11,160 cubic feet for WCN5 to 1,030,000 cubic feet for WCN9.

Type B slumps involve slopes with seeps or springs which terminate in a drainage area and which are not influenced by construction activities (Figure 4-3). Seeps indicate a potential for slope overloading due to saturation of permeable colluvium and creation of a slip surface at the colluvium/bedrock contact. In all cases of type B slumps, the location of the initial headwall coincided with the seep. Ponding of seepage water between the headwall and the top of the slump block was common. Nine type B slumps were identified in the

Table 4-1
Summary of Slump Characteristics
Type A Slumps

Slump I.D.	Slope Strike	Slope Dip	Length of Feature (ft)	Width of Feature (ft)	Approx. Volume (ft^3)	Head Wall Height (ft)	Condition
RCN1	N38°E	23°NW	100	92	27600	3	Fresh to Slightly Weathered.
RCN2	N38°E	19°SE	125	220	275000	10	Moderate to Fresh, SW - Moderately Fresh, NE - Old Weathered.
RCS3	N22°E	17°SE	190	307	292000	5	Healed, Weathered.
WCN3	N80°E	25°-21°SE	150	298	536000	12	Fairly Recent.
WCN4	N90°E	25°S	120	286	275000	8	Active.
WCN5	N75°W	23°SW	90	31	11000	4	Recent, Active.
WCN8	N75°E	19°SE	152	220	100000	3	Active, Recent.
WCN9	N75°E	18°SE	192	447	1030000	12	Old, Healed.
WMCN2							
WMCN4							

RCN=Rock Creek north; RCS=Rock Creek south; WCN=North Walnut Creek; SWCN=South Walnut Creek north; SWCS=South Walnut Creek south; WMNC=Woman Creek north

Table 4-2
Summary of Slump Characteristics
Type B Slumps

Slump I.D.	Slope Strike	Slope Dip	Length of Feature	Width of Feature	Approx. Volume	Head Wall Height	Condition
RCN3	N60°E	24°SE	138	172	593000	25	Old, Weathered, Healed.
RCN4	N90°E	18°S	151	303	458000	10	Old, Healed, Active On Undercut Headwall Up to 30'.
RCN5	N60°W	12°NE	84	96	32000	4	Moderately Active, Not Healed, Moderately Recent.
RCN6	N70°E	11°SE	503	430	1300000	6	Top 265' - Old and Healed, Bottom 238' - Active.
RCN7	N85°W	14°SW	262	400	838000	8	Old, Healed.
RCN8	N80°E	17°SE	197	430	424000	5	Very Old and Healed.
RCS2	N18°E	22°NW	150	287	215000	5	Active.
RCS4	N50°E	28°NW	430	410	3530000	20	Old, Healed.
WMCN3							

RCN=Rock Creek north; RCS=Rock Creek south; WCN=North Walnut Creek; SWCN=South Walnut Creek north; SWCS=South Walnut Creek south; WMNC=Woman Creek north

Table 4-3
Summary of Slump Characteristics
Type C Slumps

Slump I.D.	Slope Strike	Slope Dip	Length of Feature	Width of Feature	Approx. Volume	Head Wall Height	Condition
SWCS1	N80°E	9°NW	80	77	30800	5	Fresh, Active.
SWCS5	N80°E	11°NW	75	145	21750	2	Fresh to Slightly Weathered.
SWCS6	N90°E	11°N	25	65	1625	1	Fresh to Slightly Weathered.

RCN=Rock Creek north; RCS=Rock Creek south; WCN=North Walnut Creek; SWCN=South Walnut Creek north; SWCS=South Walnut Creek south; WMNC=Woman Creek north

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Table 4-4
Summary of Slump Characteristics
Type D Slumps

Slump I.D.	Slope Strike	Slope Dip	Length of Feature	Width of Feature	Approx. Volume	Head Wall Height	Condition
SWCN1	N80°E	32°SE	33	62	2046	1	Fresh, Active.
SWCN2	N45°W	30°NE	23	43	7912	8	Fresh.
SWCN3	N24°W	32°SW	23	21	3622.5	7.5	Fresh.
SWCS2	N70°W	17°NE	245	394	965300	10	Fresh, Active.
SWCS3	N90°E	24°N	25	50	1250	1	Fresh, Active.
SWCS4	N12°W	24°SW	49	154	11319	1.5	Fresh, Active.
WCN1	N85°E	20°SE	160	717	918000	8 Average	Fairly Fresh.
WCN2	N60°W	15°NE	80	100	20000	2	Recent, Active.
WCN6	N70°E	15°SE	46	105	9660	2	Fresh.
WCN7	N75°E	19°SE	100	98	29000	3	Recent, Active
WMCN1							
WMCN5	N80°W	28°SW	15	21	1102.5	3.5	Fresh.

RCN=Rock Creek north; RCS=Rock Creek south; WCN=North Walnut Creek; SWCN=South Walnut Creek north; SWCS=South Walnut Creek south; WMNC=Woman Creek north

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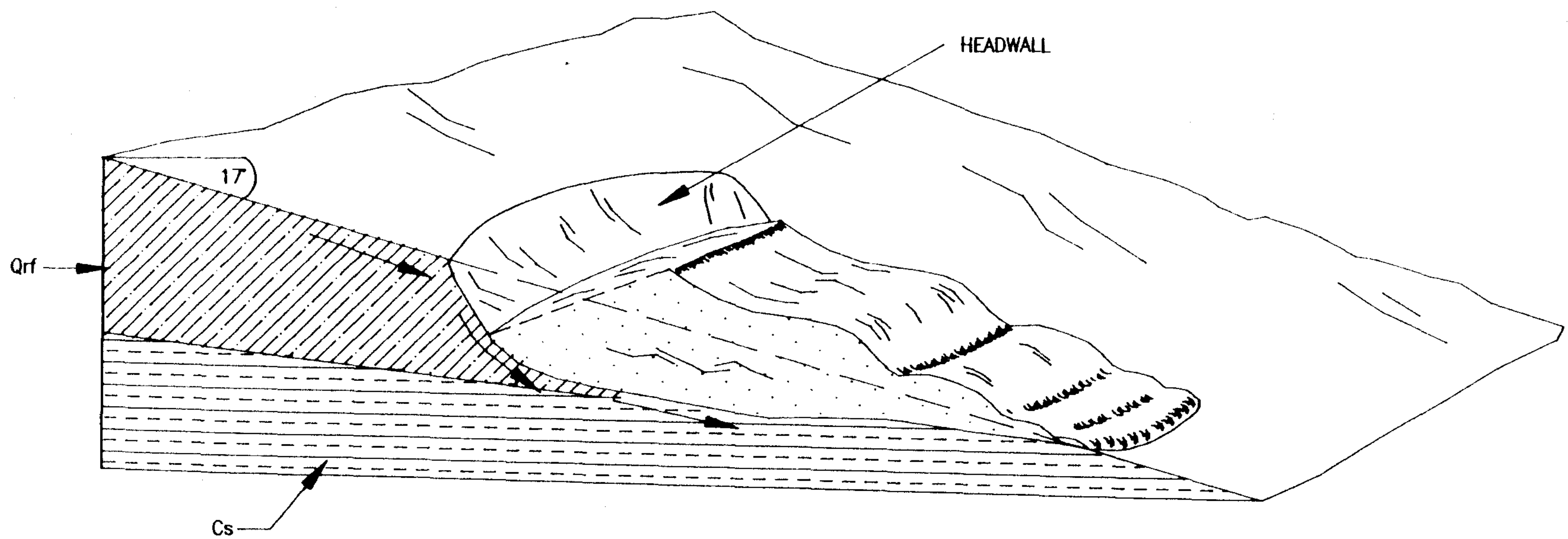

FIGURE 4-2
TYPE A SLUMP - NATURAL SLUMP

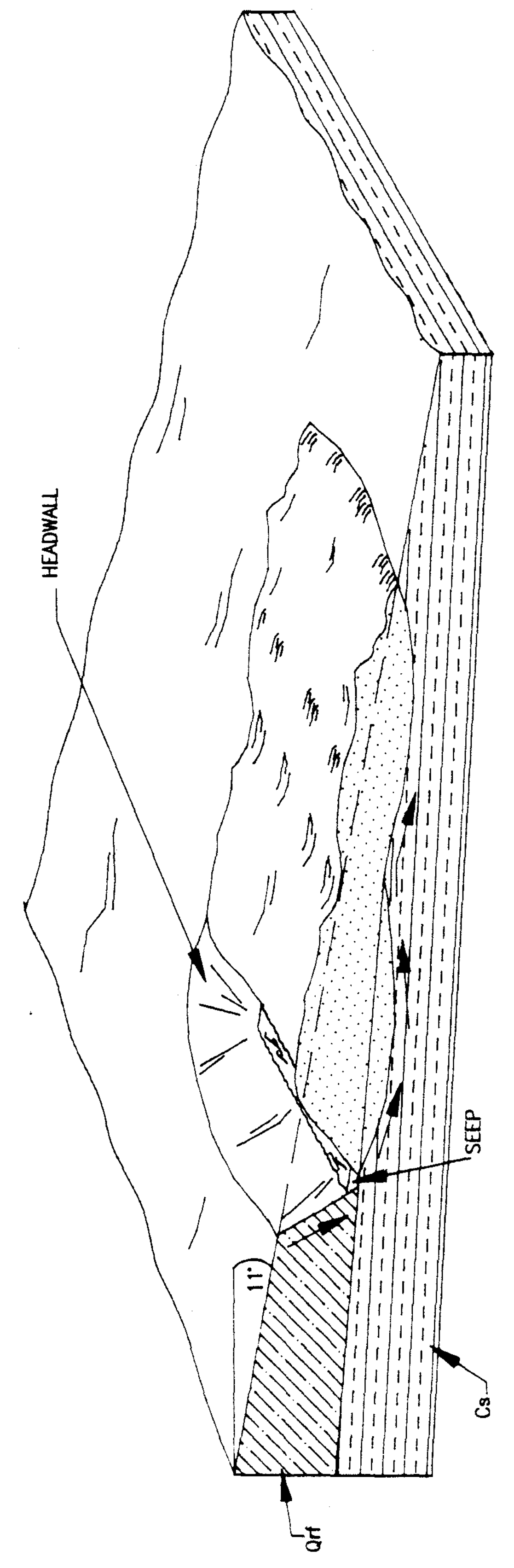

FIGURE 4-3
TYPE B SLUMP
SEEP AT HEADWALL BASE

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Rock Creek drainage (RCN3-RCN8 and RCS1, RCS2, and RCS4), and one was mapped in the Woman Creek drainage (WMCN3). The apparent critical slope dip angle for type B slumps ranged from 11 degrees for RCN6 to 28 degrees for RCS4 (Table 4-2). Volumes ranged from 19,200 cubic feet for RCS1 to 3,530,000 cubic feet for RCS4. The lower slope dip angle and larger slump volume indicate that saturation of the slope facilitates slumping.

Type C slumps involve slopes which terminate in a pond or other features which indicate possible saturation at the toe or base of the slump (Figure 4-4). Other characteristics were the same as that of type A and B slumps. All three type C slumps were in the South Walnut Creek drainage (SWCS1, SWCS5, and SWCS6). The apparent critical slope dip angle ranged from 9 degrees, measured at SWCS1, to 11 degrees measured at SWCS5 and SWCS6 (Table 4-3). Volumes range from 1625 cubic feet at SWCS6 to 30,800 cubic feet at SWCS1. The low volumes are attributed to the localized nature of conditions which define this slump type. The saturation of the slope toe is responsible for slope destabilization, resulting in low critical angles for type C slumps.

Type D slumps involve slopes which were activated by the presence of road cuts and/or drainage ditches across or at the base of the slope, or by slope overloading due to fill placement and saturation of permeable fill materials (Figure 4-5). Construction activities commonly undercut the base of slopes and thereby remove the support that the toe provides for upslope mass (Figure 4-6). Associated vegetation removal can also enhance erosion, facilitating slope saturation and/or causing oversteepening. Excess mass of artificial fill, and the relatively rapid saturation of permeable fill materials, can further destabilize slopes that have been subject to construction activity. Of the twelve type D slumps which were identified, six were in the South Walnut Creek drainage (SWCN1-SWCN3 and SWCS2-SWCS4), four were in the North Walnut Creek drainage (WCN1, WCN2, WCN6 and WCN7), and two were in the Woman Creek drainage (WMCN1 and WMCN5). The apparent critical slope dip angle ranged from 15 degrees, measured at slumps WCN2 and WCN6 to 32 degrees measured at slumps SWCN1 and SWCN3 (Table 4-4). Volumes ranged from 110 cubic feet at WMCN5 to 965,000 cubic feet at SWCS2.

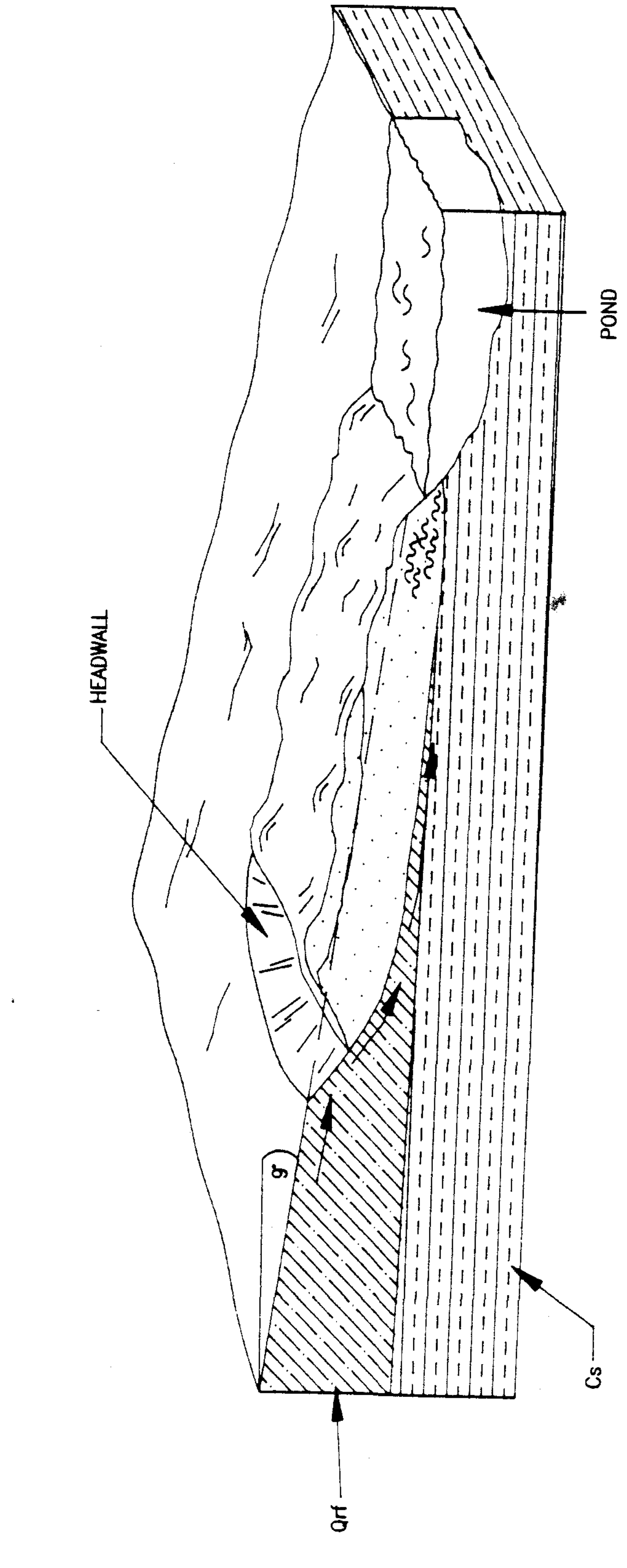


FIGURE 4-4
TYPE C SLUMP
SATURATION AT BASE OF TOE

FD014.CWpj-100190

FD015.CWpj-100190

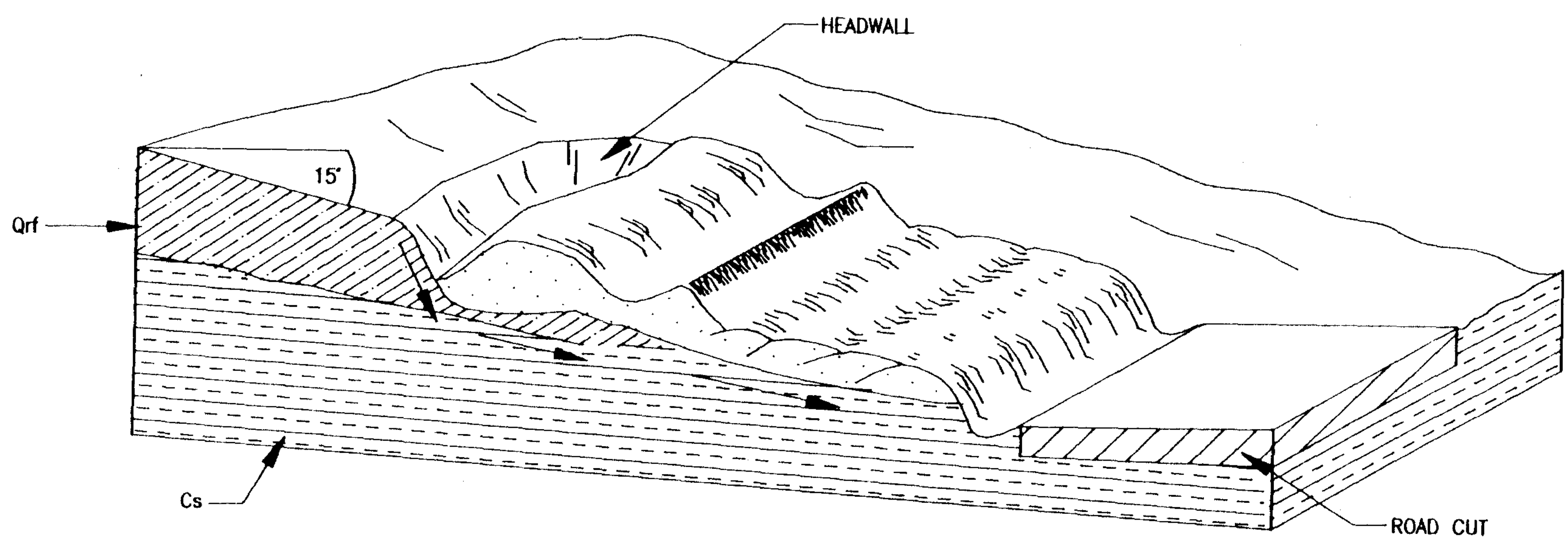


FIGURE 4-5
TYPE D SLUMP
ROAD CUT

FD016.CWpi-100190

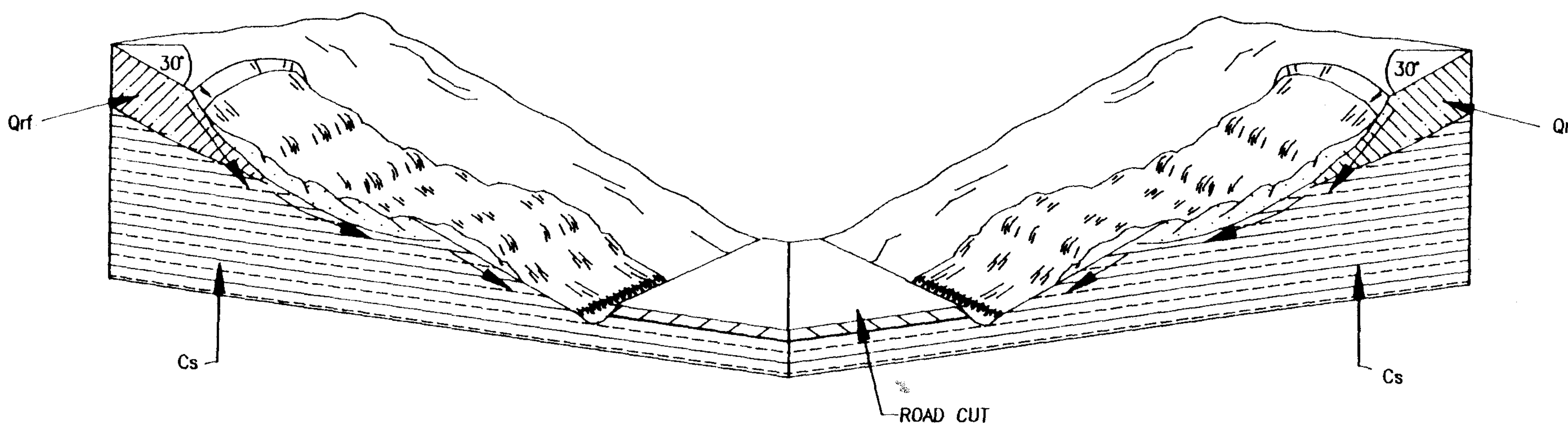


FIGURE 4-6
TYPE D SLUMP
ROAD CUT EMBANKMENTS

4.1.3 Evaluation of Sump Feature Characteristics and their Potential Impact on French Drain Construction

Type D sumps, which are defined as those features that are formed due to construction activities overstepping or overloading slopes, would most likely be the type of feature that would occur during the french drain installation. Review of Table 4-4, Summary of Sump Characteristics Type D Sumps, indicate a substantial range of feature sizes. However, with the exception of SWCS2 and WCN1 which are an order of magnitude larger than the remaining type D features, the average size of the nine remaining sumps is fairly consistent. The following list displays the Type D averages by the excluding SWCS2 and WCN1 sumps.

Average Type D Sump
(Excluding SWC52 and WCW1)

Slope Dip	Length	Width	Volume
24°	44'	73'	9500 ft^3

The above referenced data indicate that the natural slopes present on the 881 Hillside are susceptacle to movement. If construction disturbances are superimposed upon the natural slope "theoretical" slump features approaching 10,000 ft^3 (370 cubic yards) are possible.

4.2 STABILITY ANALYSIS

Slope stability is an important consideration when dealing with both man-made or natural slopes. The overstressing of a slope or reduction in shear strength of a soil or rock may cause rapid or progressive displacement (failure). The principle modes of failure in soil or rock are rotations on a curved accordent slip surfaces, translation on a plane surface and displacement of a wedge-slope mass along planes of weakness. Other modes of failure include toppling of rock slopes, falls, block slides, lateral spreading, earth and mud flow in clayey soils, and debris flows in coarse-grained soils.

4.2.1 Effect of Soil Versus Rock Type

In homogeneous cohesive soils, the typical initial failure surface is usually deep; whereas, shallow surface sloughing and sliding is more typical in homogeneous cohesionless soils. For nonhomogeneous soils, the slope and location of the failure depends on the strength and stratification of the various soil types.

Failure planes in rock occur along zones of weakness or discontinuities (fissures, joints, faults) and bedding planes (strata). The orientation and strength of the discontinuities are the most important factors influencing the stability of rock slopes.

4.2.2 Factor of Safety

The normally accepted factor of safety against slope instability is 1.5 for static (long-term) conditions and >1.0 for seismic conditions. However, when dealing with slope stability, one must consider the effect (if any) of a slope failure and the mechanism for the failure. The factors of safety stated above are for water dams with a phreatic surface (water table) within the dam. When dealing with natural slopes such as the 881 Hillside, a different condition exists, as there is no water impounded but only seasonally fluctuating perched water along the alluvial/colluvial and bedrock interface. Based on these facts it would be acceptable to use a lower factor of safety; however, due to the high profile nature of the project and the variability of the soil properties, a factor of safety for static conditions on the 881 Hillside of 1.5 has been established.

The next factor of safety which must be considered is that which could occur during construction (short-term). This factor of safety would be applied to cuts in the slope that would only be open during construction. Normally, this factor of safety is set at 1.2, thus accepting a somewhat higher risk due to the short-term conditions.

4.2.3 Seismic Stability

Seismic stability must also be considered when evaluating the long-term stability of slopes. This evaluation can be done in many ways, ranging from an intensive geological evaluation of a site, all surrounding faults, the potential for movement, and the intensity of the movement, the use of an existing study to determine the "g" (horizontal acceleration of gravity) force that would occur at the site during a seismic event.

To perform seismic analysis of slopes, two parameters must be determined; "g", and the time period to be considered for which there is a 90 percent probability of the "g" force not being exceeded. The U.S. Geological Survey has produced maps of the U.S. for determining the "g" force for 50- and 250-year event intervals. Based on those maps, the "g" forces for the 881 Hillside slopes are:

50-year recurrence = .05 g

250-year recurrence = .10 g

4.2.4 Stability Analysis

Stability analysis of the 881 Hillside slopes were performed by two methods. The first method takes into account probability of failure of construction slopes and is done by hand calculation (Canmet, 1977) and the second method used for long term stability was the modified Bishop Method performed by a computer program, the methodology of which is widely accepted. These two analyses are presented in the following sections.

4.2.4.1 Canmet Method

To evaluate the stability of the excavated sections of the french drain, an empirical slope stability analyses was performed using the following slope design formula for each identified geologic unit.

$$ic = \frac{445c}{(y\ H+q)} + \Phi\ (1.2\text{-}0.3\frac{D}{H})\ \text{-}7 \quad \text{(Canmet, 1977)}$$

where:

ic	=	Critical slope angle (°) with a 50% probability of failure (P_f) safety factor of 1.0
c	=	Cohesion (PSF)
y	=	Density (lb/ft^3)
H	=	Maximum height of highwall (ft)
q	=	Surcharge stress (PSF) None assumed
Φ	=	Angle of internal fraction (°)
D	=	Height of ground water above toe of slope (ft) assumed to be 0.0

A stability section was then evaluated from available geologic data and the information obtained from the geotechnical boreholes. The material types and thickness encountered in the holes are considered representative of what would be exposed along the length of the construction slopes of the proposed french drain. The stability section has therefore been developed using the depths at which the principal units were encountered in the boreholes. Minor variations in depths of the principal units should not significantly influence the results of stability analyses of the overall slope.

For the purposes of analysis, the stratigraphic units have been assumed horizontal. Any small component of dip which might exist along the french drain would have negligible effect on the stability analyses.

4.2.4.2 Material Properties

Colluvial Material comprises, on a average, the upper 15 feet of the french drain excavation. This material is a mixture of sandy/silty clays (CH) with occasional gravel lens. Average geotechnical design parameters are:

y	Wet Density	124 PCF
c	Cohesion (Direct Shear)	1622 PSF
φ	Friction Angle	32°
H	Maximum height of high wall	28 feet

Bedrock Units, though comprised of claystone, siltstone, and occasional sandstones, are of such similar composition as to act as a single geotechnically related unit. Average depth of the base of the french drain excavation within bedrock is 19 feet. Average geotechnical design parameters are for the bedrock unit are:

y	Wet Density	132 PCF
c	Cohesion	1705 PSF
ϕ	Friction Angle	39°
H	Maximum height of high wall	32 feet

Groundwater Conditions based upon previous investigations performed on the 881 Hillside groundwater are best described as variable. Small gravel lens within the colluvial units produce seasonal "perched" flows from multiple levels. Groundwater within shallow bedrock units was not observed, therefore, for the slope design analysis it is assumed that to the base of the proposed french drain excavation both the colluvial and bedrock units are in an unsaturated condition. However, shallow fracture or lithologically controlled groundwater will be present during construction. This will occur during high precipitation events or periods and in the vicinity of surface water drainages particularly near Borings B301490 through B301790.

4.2.4.3 Results of Analysis by Canmet

Analyses were performed to evaluate the stability of slopes of uniform angle and composition under unsaturated conditions. The section used for the analyses are shown on Plate 1. The results of these stability analyses are presented on the following table.

RESULTS OF STABILITY ANALYSES

Slope Angle	Geologic Unit	Safety Factor
59°	Colluvial	1.0
49°	Bedrock	1.0
40°	Colluvial	1.2
40°	Bedrock	1.2

For fully unsaturated conditions, the safety factor of 1.00 indicates that 49° and 59° slopes, as measured from the horizontal, would be marginally stable for bedrock and colluvial material if it was completely unsaturated. However, even complete drainage would not result in a safety factor which would generally be considered acceptable for this type of slope design. The safety factor of 1.20 for fully drained slopes of 40° (as measured from the horizontal) for colluvial and bedrock material, indicates that this design would meet the stability criteria usually applied to this type of construction slope design.

4.2.4.4 Modified Bishop Method

For the overall long term stability of the slopes on the 881 Hillside, topography was evaluated to determine the areas where the critical slopes would be based on the angle of the slope and location of the french drain. Based on this evaluation, two slopes were chosen as shown on Figure 4-7. Of the two sections chosen, Section 1 is at the location of the inferred "healed" slump area. These sections were evaluated for the stability under both static and seismic conditions using the properties shown in Section 3.2 with the exception that no value was taken for cohesion. This was done because a potential "healed" slump feature has been identified and failure within an old slump would not be able to mobilize the cohesion which would have been destroyed during previous movement.

The results of the slope stability analyses are summarized on Table 4-5 and shown on Figure 4-7. All computer runs showing all failure plains are included in Appendix E. The results show that even using the most conservative assumptions, the factor of safety under all conditions exceeds the recommended factor of safety of 1.5 for long-term slope stability.

4.3 EFFECT OF FRACTURES ON SLOPE STABILITY

The effect of fractures on slope stability cannot be completely evaluated at this time. However, some general comments regarding the influence of fractures on slope stability can be applied to the French Drain project. Fractures in general, are most likely to cause severe slope stability problems when they parallel or are near to parallel to the slope and dips are in the same direction as, but less steep than the slope. The extent of potential instability due to the fractures alone decreases as the strike of the fractures diverge from the strike.

TABLE 4-5

RESULTS OF SLOPE STABILITY ANALYSIS

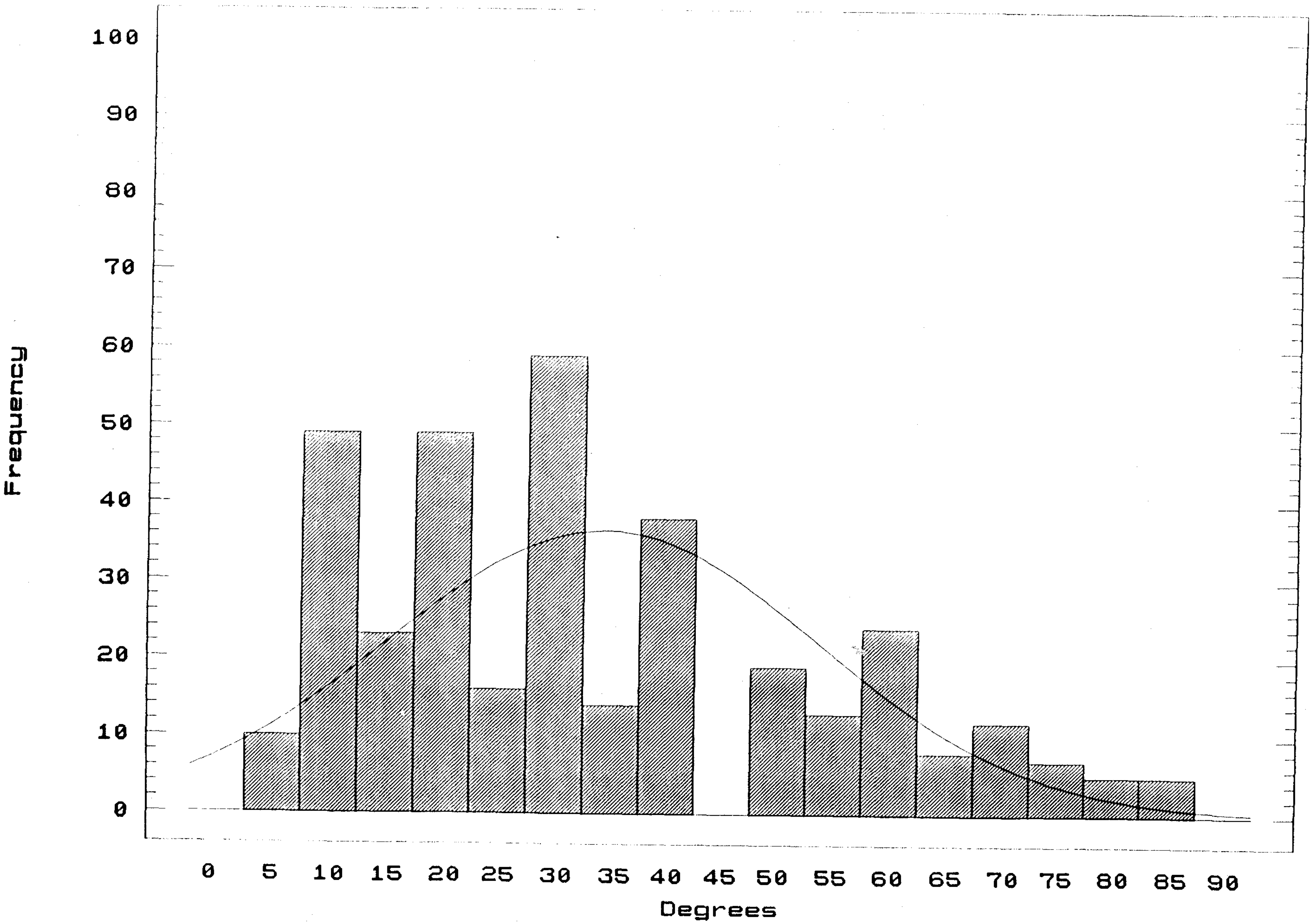
Section	Minimum Factor of Safety Static	Seismic
1	2.95	1.84
2	3.24	1.96

of the slope. The risk of instability due to fractures also increases as the shear strength of the fractured material decreases. If fracture zones are relatively impermeable, water associated with the fracture can also reduce stability in the area.

Fracture frequency distributions taken from the fracture intercepts noted on the boring logs are duplicated on Figure 4-8. It should be emphasized that the fracture orientation on the boring logs is in respect to vertical orientation (i.e., a horizontal fracture is 90 degree). Figure 4-8 is presented using a horizontal orientation as will be encountered in the field. The major fracture trend at the french drain site is currently unknown. However, it is assumed that fractures are primary a result of stress relief along drainage margins. This assumed orientation is relatively unfavorable for slope stability since proposed construction slopes will trend close to drainage orientation. Additionally, average fracture dip is relatively low (33 degree) which adds instability to slopes of higher angles. For these reasons, known fractures are expected to case some stability problems. However, the influence of fractures will be evaluated in more detail as development of the trench progresses, thus allowing a more precise interpretation of the nature, location and effect on construction of fractures.

FRACTURE FREQUENCY DISTRIBUTION

FIG 4-8



Angles are in respect to horizontal

0, 45, & 90 degree fractures were mechanical and omitted as data.

SECTION 5

CONCLUSIONS AND RECOMMENDATIONS

The results of the slope stability analysis have indicated that relatively steep slopes, greater than 50 degrees could be utilized during construction of the french drain. However, with the uncertainty of the effect of low angle fractures that could result in shear plain failure and the erratic and variable ground water conditions, high angle construction slopes are not recommended. The results of the long term stability analysis indicate that construction of the french drain should not affect the existing slope stability. However, review of the active landslide data presented in this report indicates that the natural slopes along the 881 Hillside are at or greater than the slopes upon which landslides have occurred. This condition of potential natural slope instability combined with extensive excavation must be incorporated into all construction related decisions.

In situ permeability results (packer testing) indicate that at several locations along the proposed french drain alignment the IRAP's requirement for bedrock permeability of less than 10^{-6} cm/sec cannot be met under current design concepts. To meet the IRAP requirements, excavations in excess of 50 feet will be necessary to maintain gravity flows. Additionally, excavations of this depth will result in significant fracture dilation due to vertical stress relief. Therefore, the intent to key the drain into deep low permeable bedrock will most likely not be achieved.

However, back pressure permeability tests meet or exceed all IRAP specifications for specific bedrock units. These data are interpreted as indicating that the bedrock units have essentially no vertical permeability. Using this observation, it can be concluded that upgradient contaminant sources, which occur in excess of 30 feet stratigraphically above the drain, will not result in contaminant penetration beneath the proposed drain.

Based upon the previous conclusions the following recommendations are presented.

- Slope angles for colluvial and bedrock material were combined to ease construction. It is recommended that construction slopes in bedrock and colluvial material be 40 degrees.

- The area adjacent to holes B300390 and B300490 is interpreted as containing a "healed" slump feature. It is recommended that the alignment be moved to avoid this feature or cross the Hillside at an oblique angle to reduce the potential for remobilization of the slump.

- The trench alignment along the extreme western portion of the excavation (holes B302090 through B302290) is proposed to be constructed within disturbed soil. The presence of the disturbed soil is verified in the borings and from review of the pre-plant 1937 topography. Additionally, to achieve the proper elevation within the bedrock, cuts in excess of 30 feet will be required. The alignment should be extended due west of boring B0301990 to avoid the deep cuts and disturbed soil.

- Construction dewatering of the excavation will be required due to surface runoff and seasonal groundwater flow. Particular emphasis should be placed in dewatering between Borings B301490 and B301790.

- In general, foundations and thrust blocks can be designed for bearing pressures of 2,500 PSF within colluvial units. However, if critical structures are to be constructed, specific geotechnical studies should be performed.

- Due to the proposed long-term functional life of the french drain, installation of slope indicators along the alignment is recommended. This would provide notice of movement prior to actual slope failure.

- Backcast soil should be placed on the downhill side of the excavation and heavy equipment should avoid travel along the uphill crest.

- The working face of the excavation should be inspected daily prior to the continuation of construction activities. This is a OSHA requirement for excavations (29 CFR Part 1926.651K).

The recommendations of this investigation provide a rational basis for slope design for the french drain project. However, it should be recognized that stability investigations are based on limited data, generalizations as to the nature of the slopes, and simplifications of failure mechanisms. For this reason, slope design should be considered a continuing process in which the actual performance of the slopes is compared to predicted behavior. These observations should be used to better define the areas where stability problems arise as well as the areas where steeper slopes might be possible, and to define more accurately the nature of any instability which does occur. In addition, any significant failures which develop should be carefully monitored and surveyed to enable accurate back analysis for the design of remedial measures or modifications to existing slope designs.

SECTION 6

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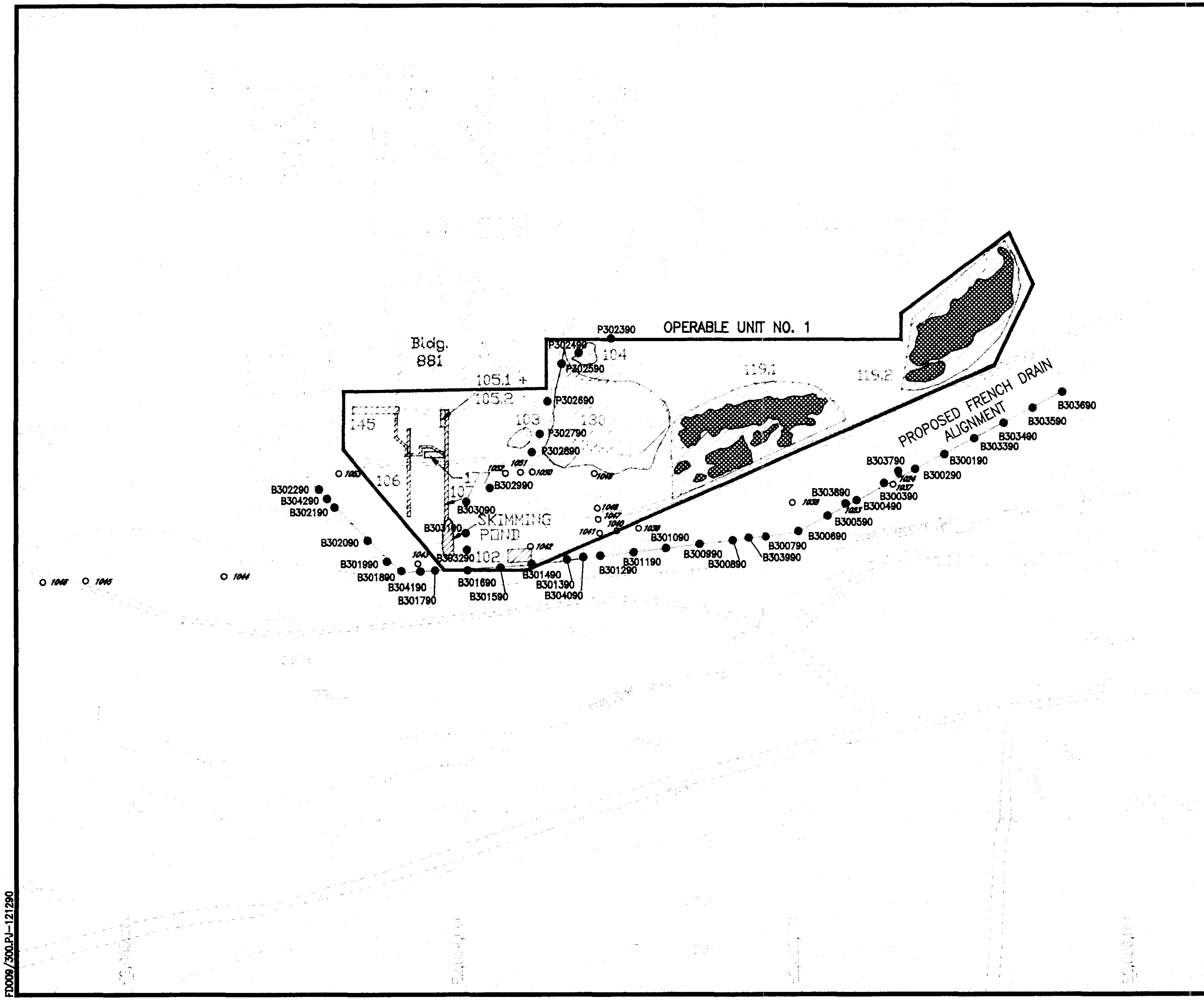

EXPLANATION

Individual Hazardous Substance Site (IHSS)

145 IHSS Designation

Maximum Extent of IHSS 119 Barrel Storage Based on Aerial Photographs dated 04/29/67, 04/10/68, 05/24/69, and 03/30/71.

B303690 Boring Location and Number French Drain Alignment

N

Scale: 1" = 300'

0' 150' 300'

CONTOUR INTERVAL = 20'

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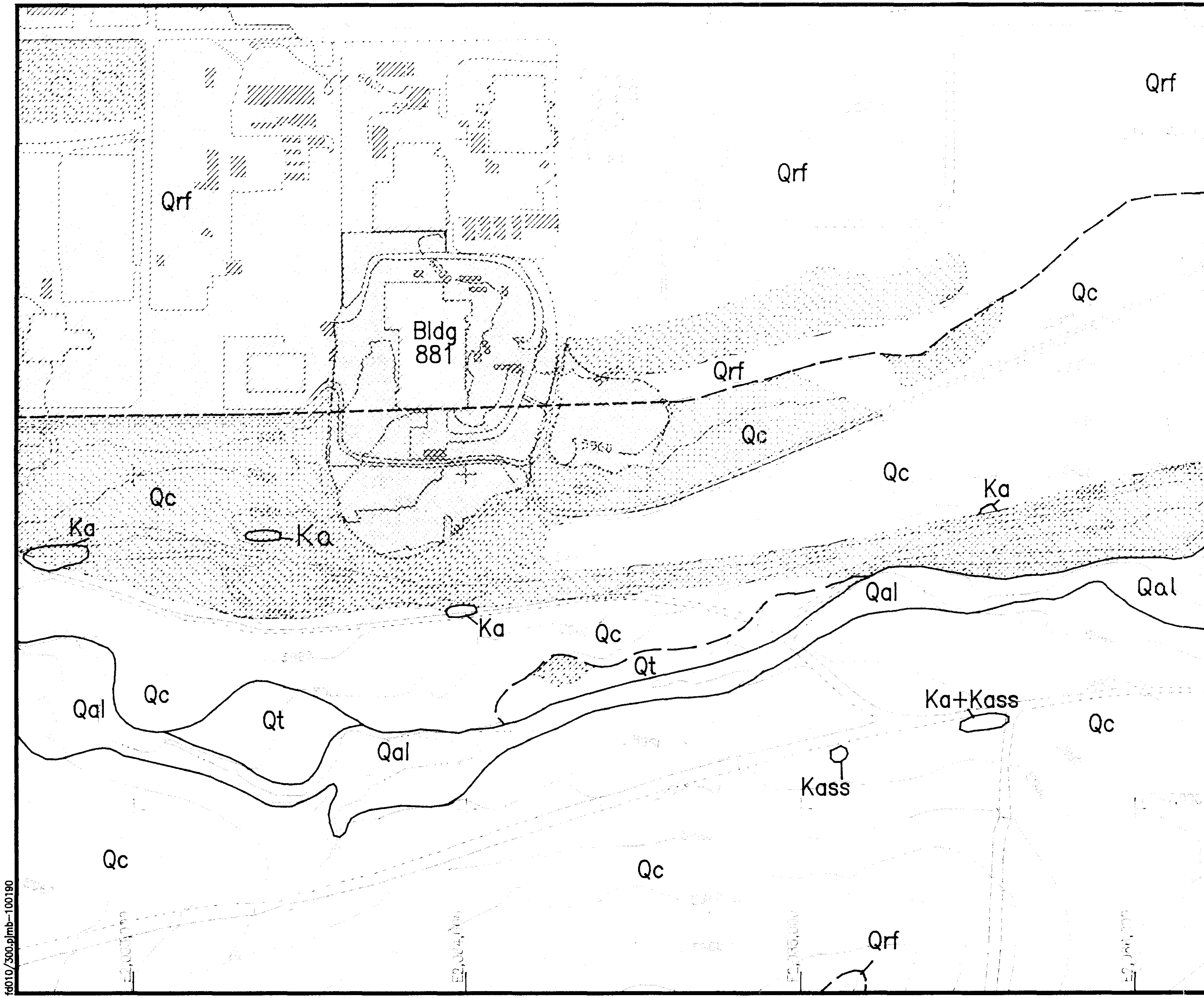
GEOTECHNICAL STUDY

FIGURE 1-1

INDIVIDUAL HAZARDOUS SUBSTANCE SITE LOCATIONS AND PROPOSED FRENCH DRAIN COLLECTION SYSTEM

December, 1990

FD009/300.PJ-121290



EXPLANATION

- Artificial Fill
- Pavement or Gravel
- Disturbed Ground
- Qal Recent Valley Fill
- Qc Colluvium
- Qt Terrace Alluvium
- Qrf Rocky Flats Alluvium
- Ka Arapahoe Formation, Claystone
- Kass Arapahoe Formation, Sandstone
- Geologic contact, dashed where approximately located

N

Scale: 1" = 300'

0' 150' 300'

CONTOUR INTERVAL = 20'

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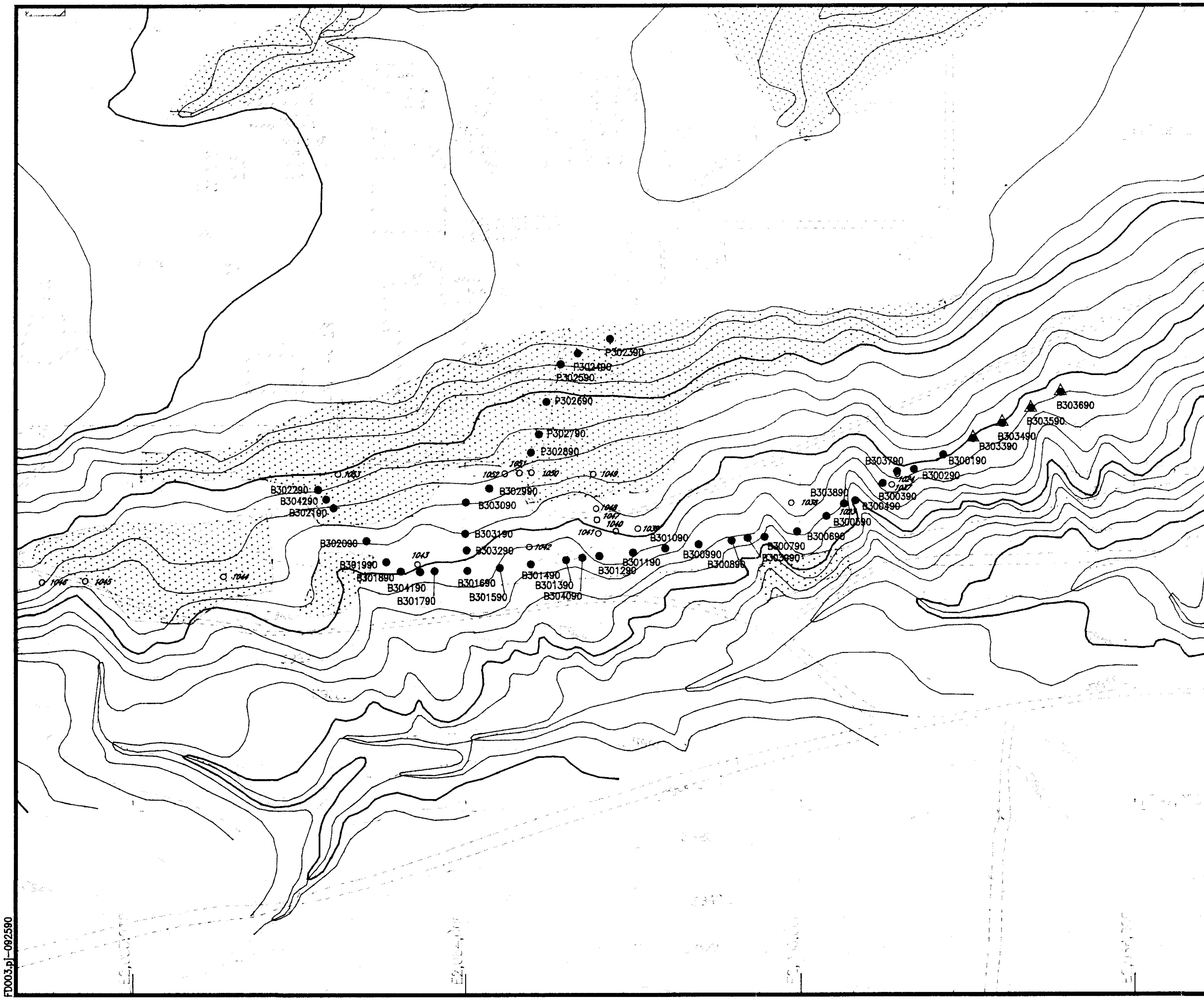
FIGURE 2-1

SURFICIAL GEOLOGY

October 1990



EXPLANATION

B300190 ● French Drain Boreholes

B303690 ▲ French Drain Piezometers

Pre-1937 Topographic Contours – Interval = 10 ft. Photography dated 7/23/37; on file with U.S. Forest Service. Photo-interpretation by: TANI PHOTOGRAMMETRIC SERVICE.

Artificial Fill Placements

N

Scale: 1" = 300'

0' 150' 300'

CONTOUR INTERVAL = 20'

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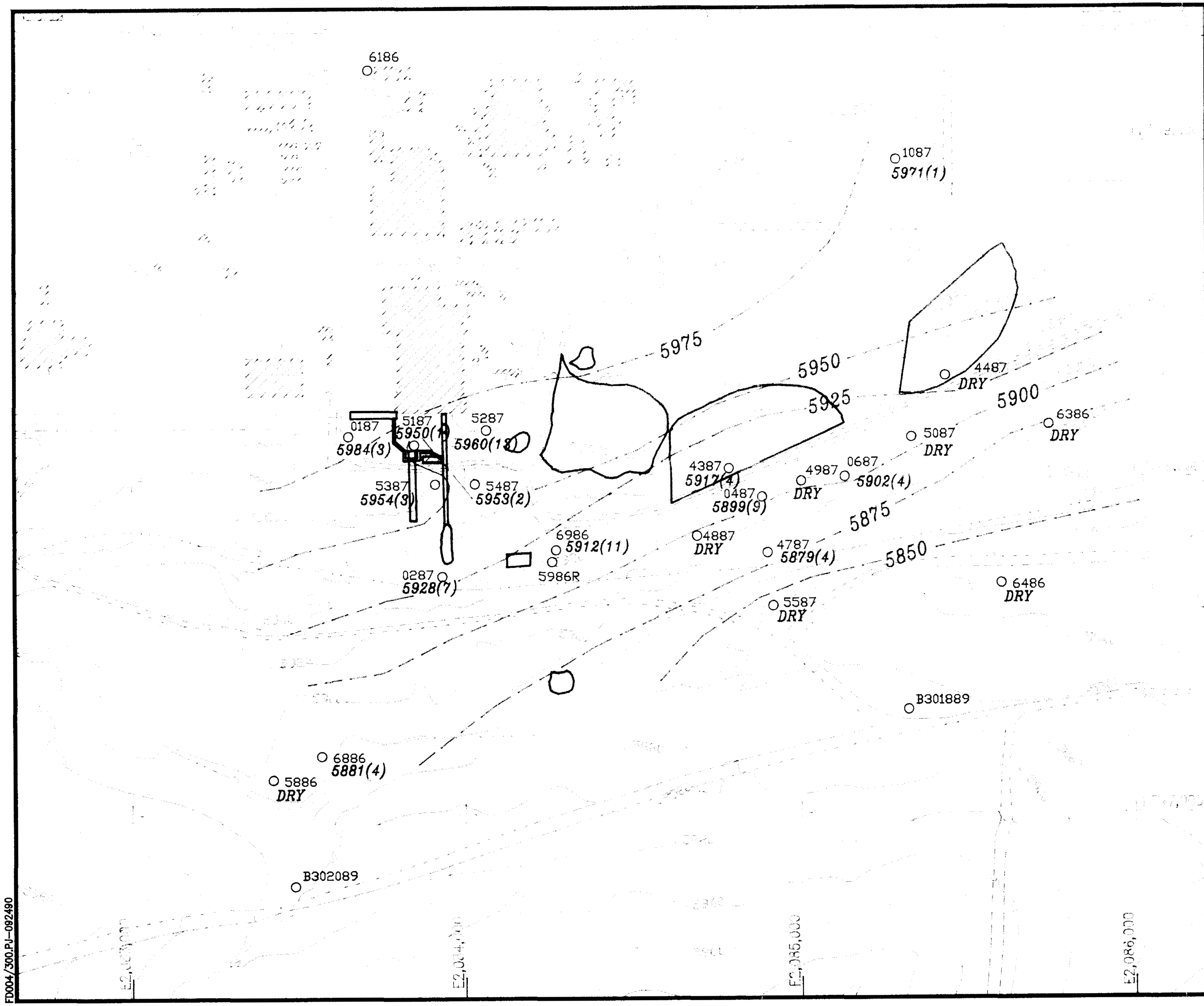
FIGURE 2-2

COMPARISON OF PRE-PLANT (1937) AND PRESENT TOPOGRAPHIC CONTOURS WITH NOTED FILL LOCATIONS

October 1990



EXPLANATION

6386 ○	Alluvial Monitoring Well
	Individual Hazardous Substance Site (IHSS)
(3)	Saturated Thickness (feet)
5902	Potentiometric Surface Elevation (feet above mean sea level)
5920	Line of Equal Potentiometric Surface Elevation (feet above sea level – dashed where approximately located)
ND	No Data – well not yet completed

N

Scale: 1" = 300'

0 150' 300'

CONTOUR INTERVAL = 20'

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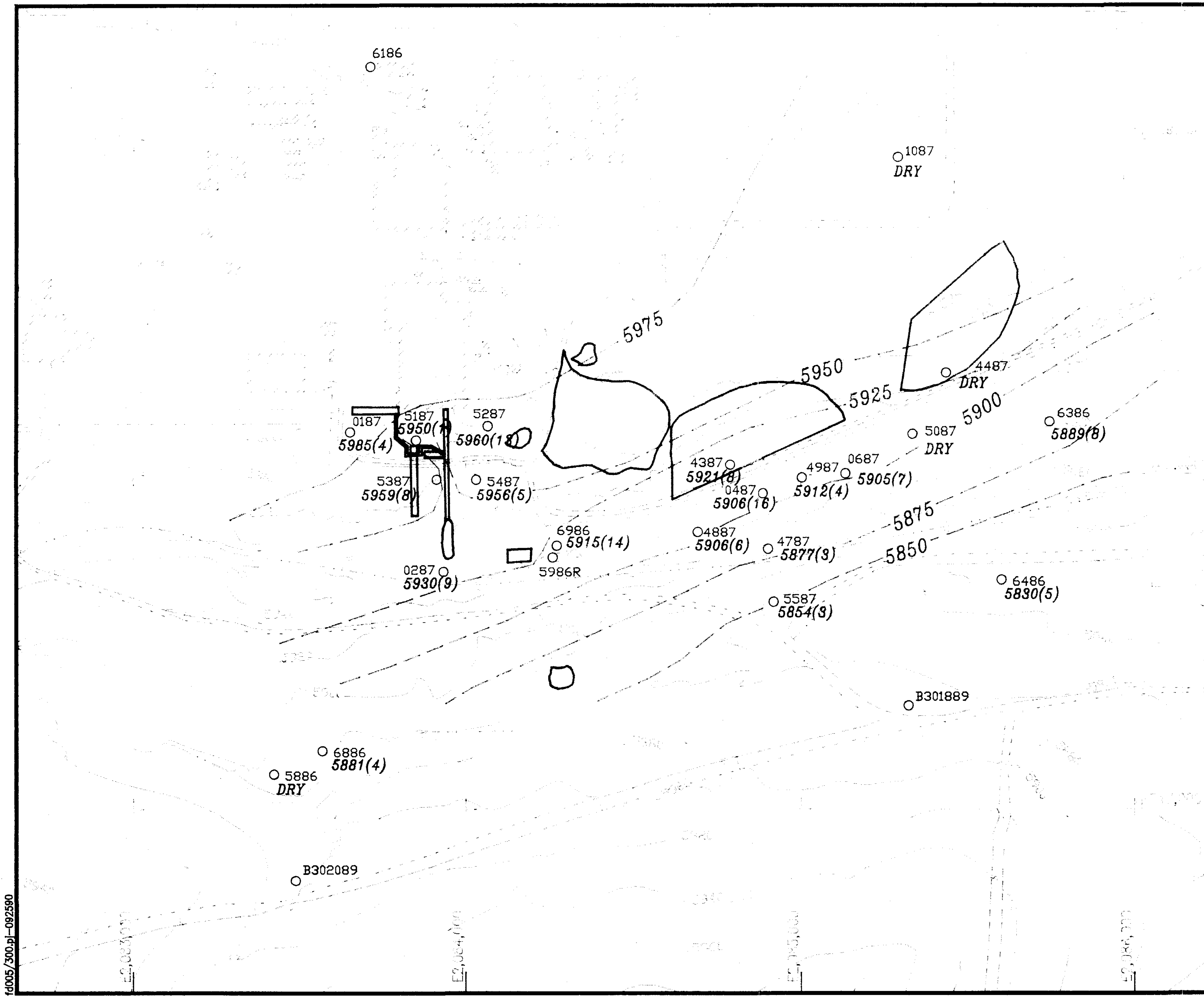
FIGURE 2–3

POTENTIOMETRIC SURFACE OF UNCONFINED GROUNDWATER AND ASSOCIATED SATURATED THICKNESS
January 1990

October 1990

FD004/300.PJ–092490



EXPLANATION

6386 ○	Alluvial Monitoring Well
	Individual Hazardous Substance Site (IHSS)
(3)	Saturated Thickness (feet)
5902	Potentiometric Surface Elevation (feet above mean sea level)
5920	Line of Equal Potentiometric Surface Elevation (feet above sea level – dashed where approximately located)
ND	No Data – well not yet completed

N

Scale: 1" = 300'

0 150' 300'

CONTOUR INTERVAL = 20'

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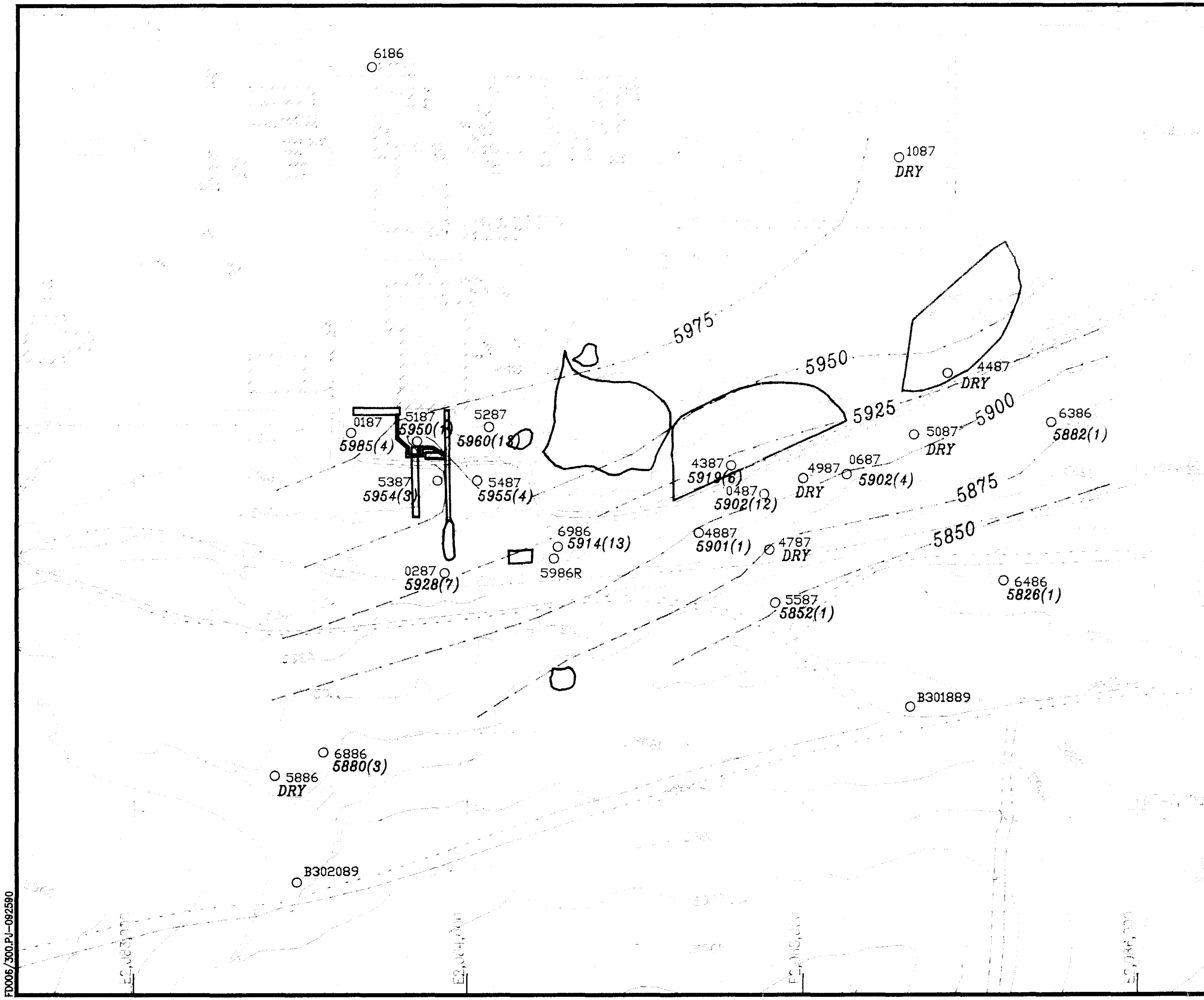
GEOTECHNICAL STUDY

FIGURE 2-4

POTENTIOMETRIC SURFACE OF UNCONFINED GROUNDWATER AND ASSOCIATED SATURATED THICKNESS
April 1990

October 1990

fd005/300.pl-092590



EXPLANATION

6386 ○	Alluvial Monitoring Well
	Individual Hazardous Substance Site (IHSS)
(3)	Saturated Thickness (feet)
5902	Potentiometric Surface Elevation (feet above mean sea level)
5920	Line of Equal Potentiometric Surface Elevation (feet above sea level – dashed where approximately located)
ND	No Data – well not yet completed

N

Scale: 1" = 300'

0 150' 300'

CONTOUR INTERVAL = 20'

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GEOTECHNICAL STUDY

FIGURE 2-5

POTENTIOMETRIC SURFACE OF UNCONFINED GROUNDWATER AND ASSOCIATED SATURATED THICKNESS
July 1990

October 1990

FD006/300.PJ-092590

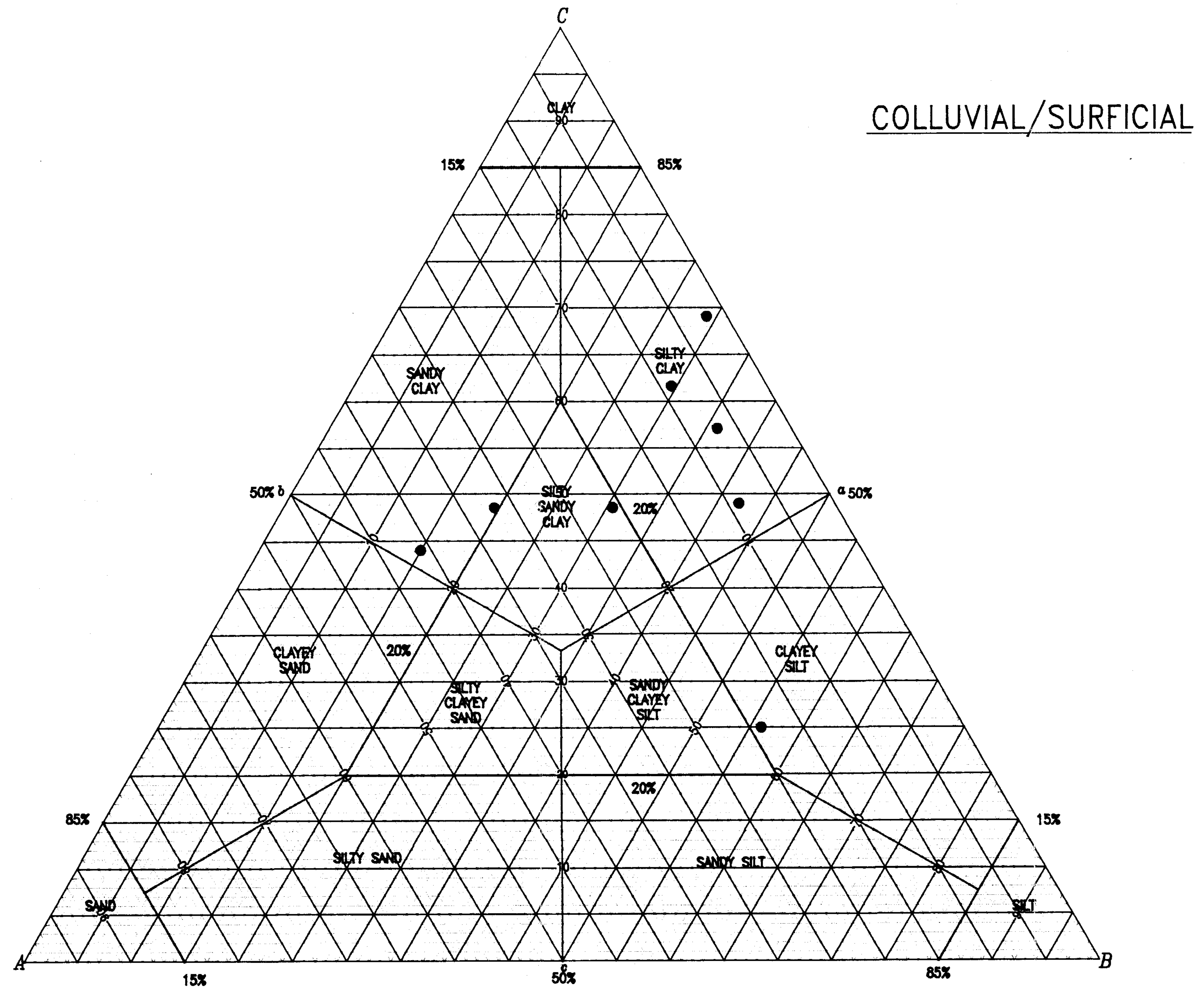


FIGURE 3-1
TEXTURAL CLASSIFICATION CHART OF THE
ALLUVIAL/COLLUVIAL GRAIN SIZE/HYDROMETER RESULTS

FD008.CWp|-121290

BEDROCK

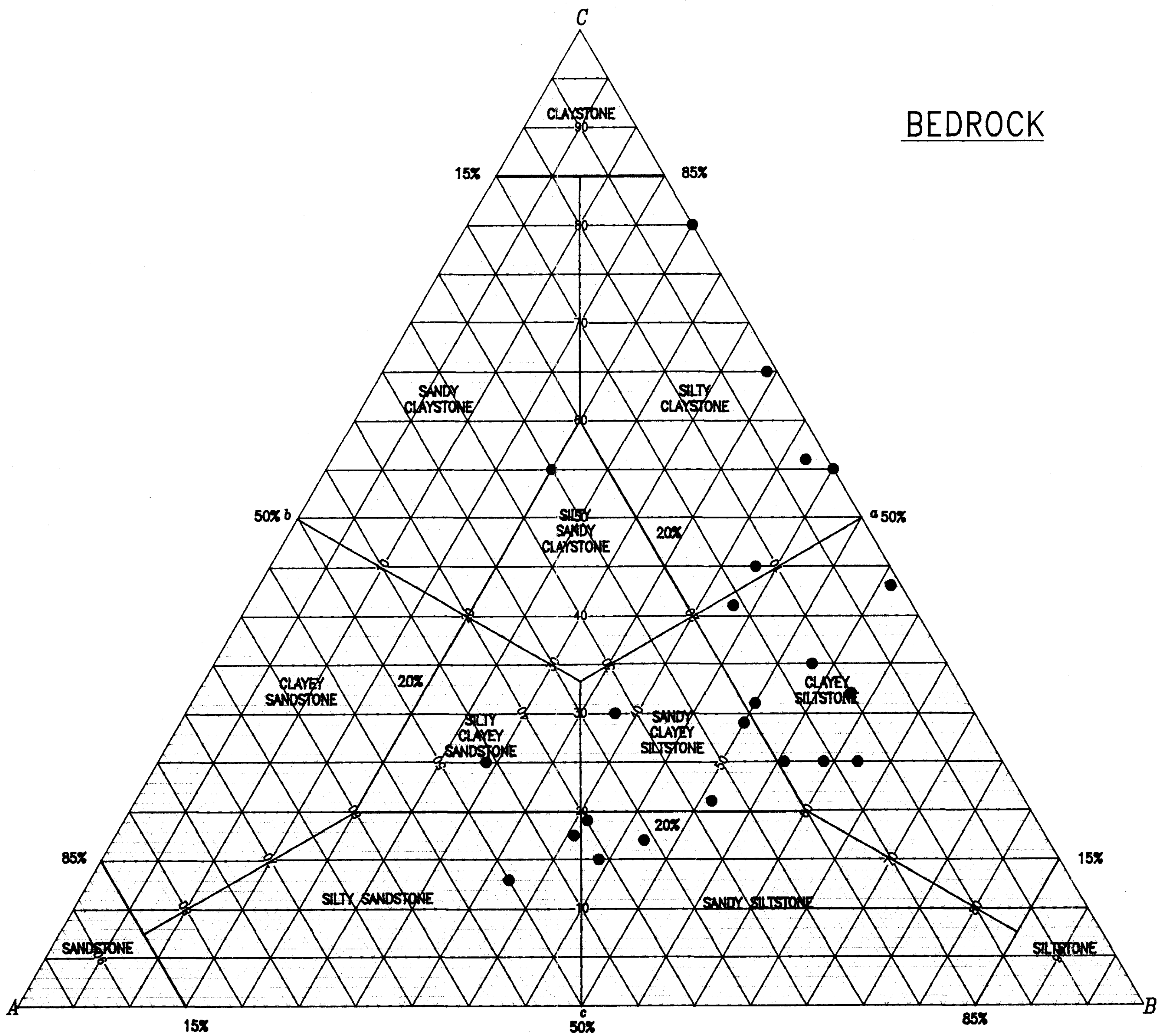
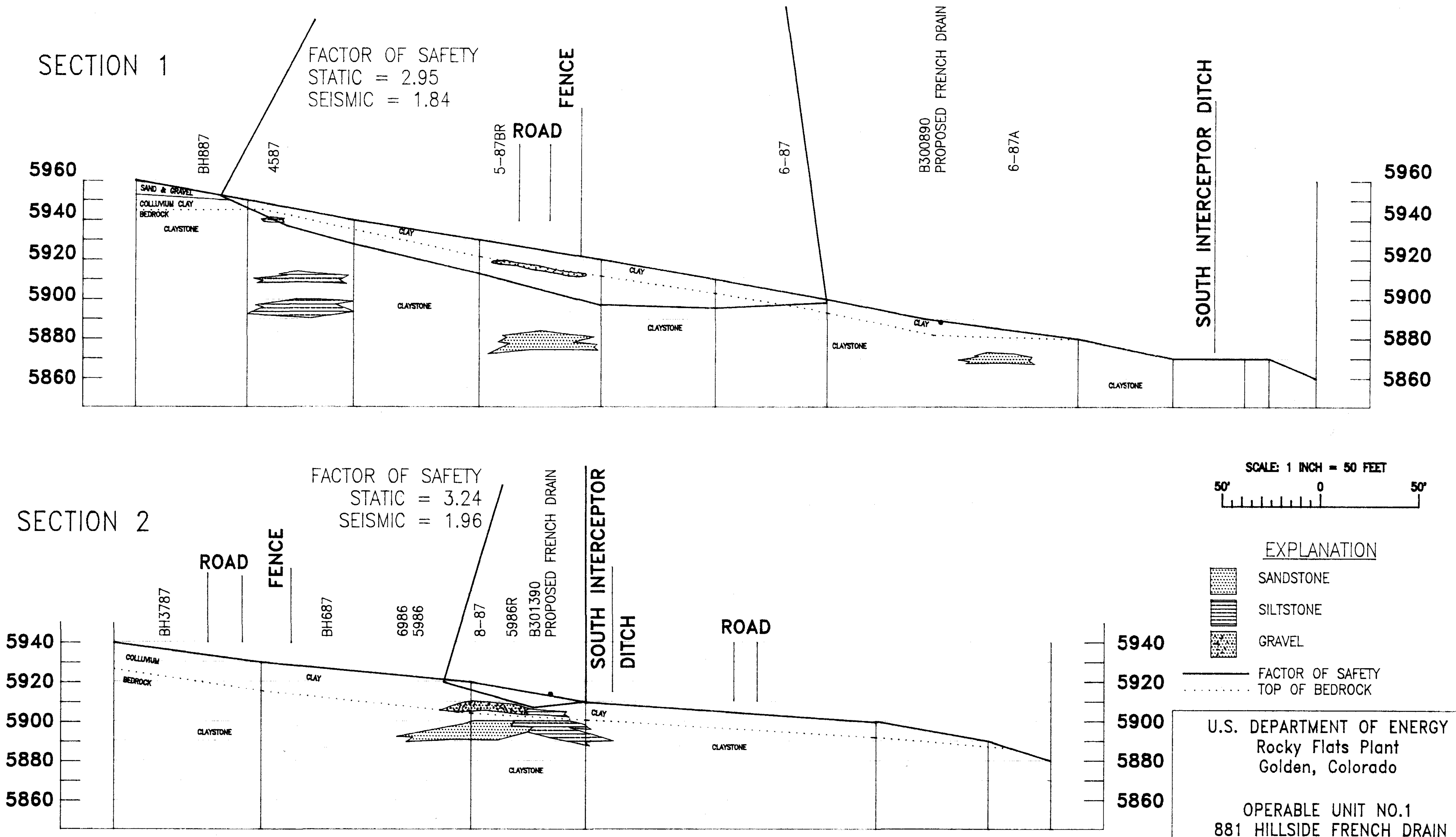


FIGURE 3-2
TEXTURAL CLASSIFICATION CHART OF THE
BEDROCK GRAIN SIZE/HYDROMETER RESULTS

FD007.CWp)-121290



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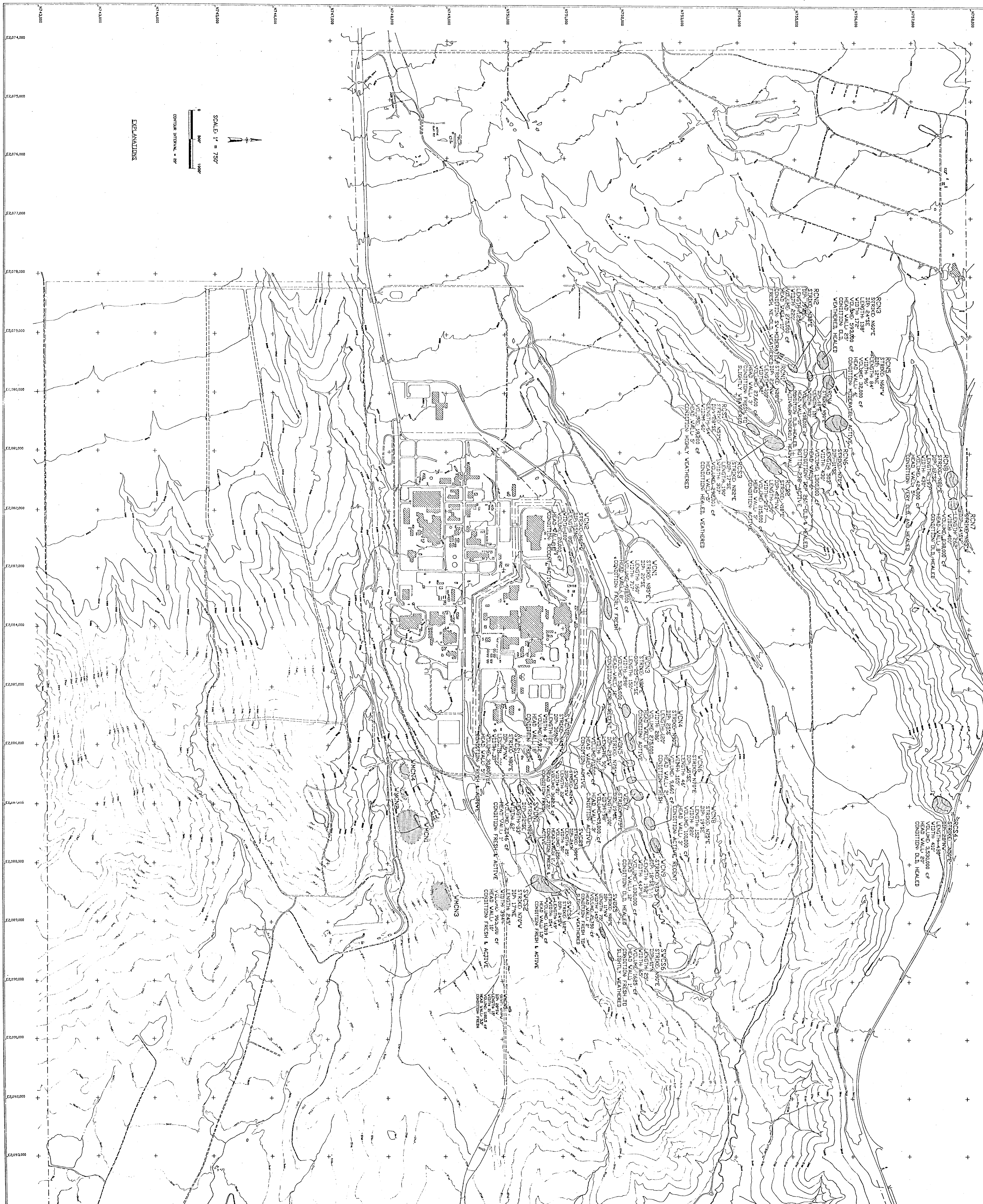
881 HILLSIDE FRENCH DRAIN
GEOTECHNICAL STUDY

FIGURE 4-7

STABILITY PROFILE

October 1990

FDCUTS.MB100290



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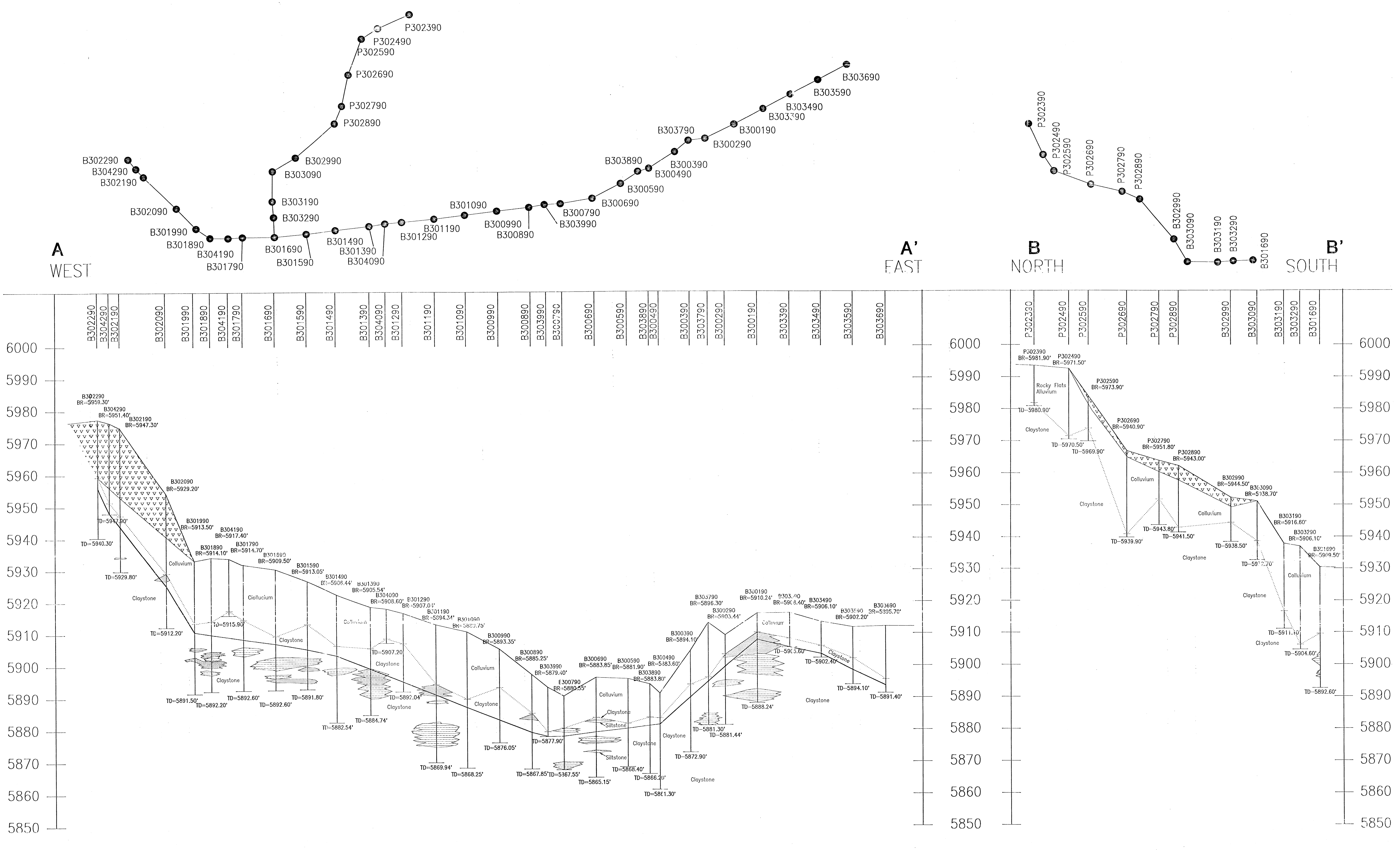
Golden, Colorado

PLATE 2

SLUMP FEATURE LOCATIONS

NO.	DATE	REVISION



EXPLANATION

TOP OF BEDROCK
PROJECTED FRENCH DRAIN INVERT
1024 GAS PIPELINE MARKERS
SILTSTONE
SANDSTONE
CLAYSTONE
AREAS OF ARTIFICIAL FILL

SCALE:
Vertical Scale: 1"=10'
Horizontal Scale: 1"=100'

ISSUE	DESCRIPTION	DATE	RFP	DOE	CLASS	JOB NO.
A	ORIGINAL ISSUE	9/18/90				2029-33-09-0020

	BY	DATE
DESIGNED		
DRAWN		
CHECKED		
APPROVED		
SUBMITTED		
APPROVED RFP		
APPROVED DOE		

U.S. DEPARTMENT OF ENERGY
ROCKY FLATS AREA OFFICE GOLDEN, COLORADO

ROCKY FLATS PLANT
GOLDEN, COLORADO 80402-0464

PLATE 1
GEOLOGIC CROSS SECTIONS OF THE PROPOSED FRENCH DRAIN COLLECTION SYSTEM
A-A' WEST TO EAST, B-B' NORTH TO SOUTH

SIZE	DRAWING NUMBER	ISSUE	SHEET
E			1 OF 1

FD018.PJMB100490