APPENDIX C

SLOPE STABILITY EVALUATION

U. S. STEEL TAILINGS DIKES

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WELLINGTON, UTAH

MARCH 1978

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SLOPE STABILITY EVALUATION

U.S. STEEL TAILINGS DIKES

WELLINGTON, UTAH

I. INTRODUCTION

1. Purpose

A foundation investigation and stability analysis has been completed for four tailings dams located in the vicinity of the U.S. Steel Coal Cleaning facility near Wellington, Utah. The purpose of this investigation was to define the characteristics of the subsurface material within the existing dikes, to determine the factors of safety of the existing dikes with respect to failure, and to make recommendations relative to the improvement of the dikes if adequate factors of safety do not exist. The work performed at the site has been completed in a manner to achieve the basic objectives and the details of the investigation are outlined below.

2. Existing Site Conditions

The general location of the dike system currently in use by U.S. Steel is shown in Figure No. 1. It will be observed that the main dike, known as the Clearwater Dike is located immediately northeast from the Price River, which travels along the southerly side of the dike area. In addition to the Clearwater Dike, two other dikes are part of the tailings dike system. A fourth dike, consisting of soil excavated from a trench, constitutes the fourth part of the retaining facilities. This fourth dike is not used at the present time to store

WATER, HOWEVER, IT IS ANTICIPATED THAT IN THE FUTURE THIS FACILITY WILL BE MODIFIED AND RAISED AND BECOME A PART OF THE STORAGE FACILITIES.

THE CLEARWATER DIKE AND THE LOWER REFUSE DIKE ARE APPROXIMATELY 35 FEET HIGH, WHILE THE UPPER REFUSE DIKE AND THE NORTHERN DIKE ARE ABOUT 15 FEET AND 12 FEET HIGH RESPECTIVELY. THE DOWNSTREAM SIDE OF THE CLEARWATER DIKE ADJACENT TO THE PRICE RIVER IS EXPOSED AND THERE DOES NOT APPEAR TO BE ANY SEEPAGE EXITING ON THE FACE OF THE EMBANKMENT. IT DOES APPEAR, HOWEVER, THAT UNDERSEEPAGE IS TAKING PLACE BENEATH THE CLEARWATER DIKE. THE CREST ELEVATION OF THE CLEARWATER DIKE IS ABOUT 5370 AND WHILE THE WATER LEVEL UPSTREAM FROM THE DAM FLUCTUATES SOMEWHAT, THE APPROXIMATE WATER ELEVATION IS 5365. THE CREST ELEVATION OF THE LOWER REFUSE DIKE IS ALSO AT ABOUT 5370. THE ELEVATION OF THE TAILINGS AND THE WATER ABOVE THIS DIKE IS AT ABOUT 5367. IT IS APPARENT, THEREFORE, THAT THERE IS ONLY A SMALL DIFFERENCE IN ELEVATION BETWEEN THE WATER SURFACE DOWNSTREAM AND THE TAILINGS AND THE WATER UPSTREAM.

THE CREST ELEVATION OF THE UPPER REFUSE DIKE IS AT ELEVATION 5380. THE ELEVATION OF THE TAILINGS AND THE WATER IN THE UPPER REFUSE DIKE IS AT 5377. IT IS APPARENT, THEREFORE, THAT THERE IS ONLY AN ELEVATION DIFFERENCE OF 10 FEET BETWEEN THE TAILINGS IN THE WATER DOWNSTREAM AND THE TAILINGS IN THE WATER UPSTREAM. THE ELEVATION OF THE WATER AND THE TAILINGS IN THE VARIOUS STORAGE AREAS FLUCTUATES SOMEWHAT AND THE VALUES INDICATED ABOVE ARE APPROXIMATE VALUES ONLY.

SOME SEEPAGE APPEARS TO BE OCCURRING AROUND THE LEFT ABUTMENT OF THE UPPER TAILINGS DIKE. THE SEEPAGE IS OCCURRING IN NATURAL MATERIAL AND NOT IN THE FILL OF THE ABUTMENT. THE NORTHERN-MOST DIKE WAS FORMED BY EXCAVATING A TRENCH AND THE EXCAVATED MATERIAL FROM THE TRENCH WAS PILED UP TO FORM THE EMBANKMENT. IT APPEARS AS IF THIS EMBANKMENT WAS FORMED WITHOUT ANY SPECIAL

ATTENTION BEING GIVEN TO COMPACTING THE MATERIAL IN THE DIKE IN ACCORDANCE WITH STANDARDS USED IN ENGINEERED FILL.

3. Scope of Work

The scope of the work performed during this investigation included subsurface investigation along the axis of the dam to identify the nature of the subsurface material and to determine their permeability characteristics, laboratory investigations to determine the physical characteristics of the embankment material including strength characteristics, and a stability analysis to determine the factor of safety of the existing slopes.

II. SUBSURFACE INVESTIGATION

1. Procedures Used in Defining Subsurface Materials at the Site

The three tailings dams were constructed several years ago and no record is available of the chrracteristics of the material within the existing embankments. In order to provide a reasonable indication of the nature of the material within the embankments and within the foundation material at the various sites, it was our opinion that at least six test borings should be drilled ay each of the three locations. The exact depth of the test holes required to define the character of the materials could not be completely determined prior to the drilling and the exact depth that each test boring was extended below the crest was determined in the field. The location of the test borings along each of the dikes is presented in Figure No. 1. Drilling was performed using a rotary drill rig with water as the drilling fluid. Both disturbed samples and undisturbed samples were obtained during the drilling operations. Disturbed samples were obtained using a split spoon sampling tube while undisturbed samples were obtained using a 2.5 inch inside diameter thin-walled shelby tube. Field permeability tests were performed during the subsurface investigation in accordance with Designation E-18 of the U.S. Bureau of Reclamation's Earth Manual. Standard

PENETRATION TESTS WERE USED TO DETERMINE THE IN-PLACE DENSITY OF THE GRANULAR MATERIAL. THE STANDARD PENETRATION VALUE PROVIDES AN INDICATION OF THE SANDY MATERIAL, HOWEVER, IT DOES NOT PROVIDE A REASONABLE INDICATION OF THE IN-PLACE DENSITY OF COHESIVE SOILS AND CONSIDERABLE CARE SHOULD BE USED IN INTERPRETING THE STANDARD PENETRATION VALUE OBTAINED IN GRANULAR-TYPE SOILS. A LOG OF THE BORINGS WAS MAINTAINED IN THE FIELD AND A RECORD OF THE GROUNDWATER LEVEL IN THE DRILL HOLES WAS OBTAINED DURING DRILLING OPERATIONS.

2. Subsurface Soil Characteristics

As indicated above, samples were obtained by driving a two-inch split spoon sampler through a distance of 18 inches using a 140-pound weight dropped from a distance of 30 inches. The number of blows to drive the sampling spoon through each six inches of penetration is shown on the boring logs for all test holes. The sum of the last two blow counts, which represents the number of blows to drive the sampling spoon through 12 inches of penetration, is defined as the standard penetration value.

Each sample obtained in the field was classified in the laboratory according to the Unified Soil Classification System. The symbol designating the soil type according to this system is presented on the boring logs. A description of the Unified Soil Classification System is presented in Figure No. 2, and the full meaning of the various symbols shown on the boring logs can be obtained from this figure.

The characteristics of the subsurface materials at each of the dikes are discussed below as follows.

A. Upper Refuse Dike

Six test borings varying in depth from 15 to 55 feet were drilled along the crest of this facility in order to define the characteristics of the subsurface material. The boring logs for the six test holes are presented in Figure No. 3. It will be observed from the boring logs that the upper 15 to 25 feet of the material at this location consists of black coal refuse.

The results of the standard penetration tests performed in this material indicates that the coal refuse is in a medium to low density state. The coal refuse is underlain by a tan sandy silt which varies in thickness from about 5 to 20 feet. This material is generally in a soft condition as indicated by the low blow counts in test boring No. 1 and No. 3. It will be noted that standard penetration values of 2 were observed in several locations of these test holes. The brown silt was underlain by a granular zone which extended until bedrock was encountered in the lower portion of the profile. The granular material generally consists of sand and gravel, however, some silty sand lenses exist at various locations throughout the profile.

Gray weathered shale was encountered at a depth of between 45 and 46 feet in test holes 2 through 5. The upper portion of the shale was partially weathered and could be penetrated with a split spoon sampling tube. The shale became relatively hard, however, within a few feet of the depth at which it was encountered.

It should be noted that the coal refuse material, shown on the boring logs, is the primary material within the dikes and that the silt and granular material, shown on the boring logs, is the foundation material. It will be noted that the cohesive material underlying the coal refuse is in a relatively soft condition at a number of locations throughout the structure. Blow counts as low as 2 and 3 were observed in test borings 1 and 3.

Groundwater was encountered at a depth of about 10 feet below the existing ground surface at this dike. Since water exists on both sides of the dike, it can be expected that the subsurface material composing the dike will be essentially saturated.

B. Lower Refuse Dike

The characteristics in the subsurface material in the refuse dike was defined by drilling six test borings at locations as shown in Figure No. 1. The logs for the six test borings are presented in Figure No. 4, and it will be observed that the depth of the test borings varied from about 15 to 50 feet. The results of the subsurface investigation indicates that the subsurface material within the embankment consists predominantly of tan silty clays. Since the embankment at this location is approximately 35 feet high, the subsurface material below that elevation consists predominantly of the natural foundation materials. It will be observed that a silty clay layer composes the foundation material at the base of the structure. The silty clay layer at the base of the structure is underlain by a brown sandy gravel to gravelly sand in a medium to loose density state. This material is characteristic of the natural gravels which exist throughout the site.

Some irregularities were noted in the brown clay material at this site as the groundwater level varied in the dike from about 17 feet to 5 feet. The results of the standard penetration tests indicate that the clay material within both the dike and the foundation is in a medium-stiff condition. It should be observed that no bedrock was encountered at this location within the depth investigated. Since the subsurface materials below the base of the dam obtain appreciable quantities of granular type soils, it can be anticipated that some seepage will occur beneath the proposed structure at this site.

C. Clearwater Dike

The characteristics of the subsurface material at this dike were investigated by drilling 6 test borings to depths varying from 15 to 50 feet at locations as shown in Figure No. 1. The boring logs for the 6 test holes are presented in Figure No. 5, and it will be observed that the subsurface profile at this location is very similar to the material observed for the lower refuse dike.

The subsurface material within the embankment consists predominately of brown silty clay with some sand lenses. The results of the standard penetration tests indicate that the clay material is in a medium stiff condition.

The upper layer of the foundation material also consists of a gray silty clay in a medium stiff condition. This clay layer is approximately 10 feet thick and is underlain by a brown silty gravel to gravelly sand. The results of classification tests indicate that the cohesive material in this test boring consists predominately of low plasticity silts and clays.

Since the brown silty gravel has greater strength characteristics than the clay, it is presumed that any failure surface in this material will occur in the cohesive material above the brown silty gravel in the bottom of the profile.

Groundwater was encountered at about 8 to 9 feet in test borings No. 15, 16 and 17. This information appears to substantiate the proposition that a phreatic surface likely exists throughout the embankment material for this facility. No groundwater was observed on the downstream slope for this facility and hence it appears as if the phreatic surface exists within the embankment material.

3. Field Permeability Tests

Field permeability tests were performed at 5-foot intervals in each test boring in accordance with designation E-18 of the USBR Earth Manual. These test results are shown under the appropriate hole in Figures 3 to 5.

It will be observed from Figure No. 3 that the embankment material for the upper refuse dike has permeability rates varying from 9 feet per year to greater than 15,000 feet per year. The foundation material for this dike also has a wide range of permeability rates. The rates are generally greater than 5,000 feet per year.

Permeability rates for the lower refuse dike shown in Figure No. 4 vary from no measureable loss to 9500 feet per year. The embankment material is relatively semi-impervious. The permeability rate of the foundation material increases with depth.

The permeability data for the clearwater dike shown in Figure No. 5 indicates that the embankment material is semi-impervious with permeability rates less than 300 feet per year. The permeability rates of the foundation material for this dike do not exceed 3000 feet per year.

III. LABORATORY TESTS

1. Classification Tests

Classification Tests consisting of either Atterberg limits or mechanical analysis were performed on representative samples for each of

THE DIKES. A SUMMARY OF THE CLASSIFICATION TESTS PERFORMED DURING THIS INVESTIGATION ARE PRESENTED IN TABLE NO. 1.1 THROUGH NO. 1.3. THE RESULTS OF THE TESTS PERFORMED ON REPRESENTATIVE SAMPLES FROM HOLES NO. 3 THROUGH NO. 5 ARE CHARACTERISTIC OF THE MATERIAL IN THE UPPER REFUSE DAM; WHILE THE RESULTS OF TESTS PERFORMED ON SAMPLES FROM TEST HOLE NO. 9 DEFINE THE CHARACTERISTICS OF THE MATERIAL IN THE LOWER REFUSE DIKE. THE RESULTS OF THE TESTS SHOWN FOR HOLES NO. 16 AND 17 ARE CHARACTERISTIC OF THE SUBSURFACE MATERIAL IN THE CLEARWATER DIKE.

THE RESULTS OF THESE TESTS INDICATE THAT THE SUBSURFACE MATERIAL IN THE LOWER REFUSE DIKE AND IN THE CLEARWATER DIKE CONSIST PREDOMINATELY OF LOW PLASTICITY CLAYS, WHILE THE MATERIAL WITHIN THE UPPER REFUSE DIKE CONSISTS PREDOMINATELY OF GRANULAR-TYPE SOILS.

2. TRIAXIAL SHEAR TESTS

A. UPPER REFUSE DAM

AS INDICATED EARLIER IN THIS REPORT, THE EMBANKMENT OF THE UPPER REFUSE DAM CONSISTED PRIMARILY OF COAL REFUSE MATERIAL. THE EMBANKMENT IN THIS AREA WAS LOCATED ON A LOW PLASTICITY COHESIVE MATERIAL SANDWICHED BETWEEN COAL REFUSE MATERIAL IN THE EMBANKMENT AND A MEDIUM DENSE GRANULAR MATERIAL BELOW. THE STRENGTH CHARACTERISTICS OF THE COAL REFUSE MATERIAL WERE EVALUATED BY PERFORMING 3 CONSOLIDATED DRAINED TRIAXIAL TESTS TO GIVE THE MOHR ENVELOPE SHOWN IN FIGURE NO. 6.

IT WILL BE OBSERVED THAT THE FRICTION ANGLE OF THIS MATERIAL WAS ABOUT 28.5°. THE INTERCEPT OF THE MOHR ENVELOPE WITH THE SHEAR STRESS AXIS INDICATES THAT THE COAL REFUSE MATERIAL WOULD POSSESS SOME COHESION. SINCE THIS MATERIAL IS PREDOMINATELY GRANULAR IN NATURE, IT IS DOUBTFUL IF THIS COHESION ACTUALLY EXISTS; AND IN OUR OPINION, IT SHOULD NOT BE used in any stability computations. The cohesive material below the coal refuse material in the Upper Refuse Dike area was evaluated by performing 3 consolidated drained triaxial shear tests.

The Mohr Envelope resulting from the above tests is presented in Figure No. 7. It will be observed that the friction angle for this material was 35.1° with no cohesion intercept. It is our opinion that the samples used to obtain the shear strength characteristics of these materials are representative of the types of material likely to be encountered in the embankment and foundation material at this location.

B. Clearwater Dike and the Lower Refuse Dike

As indicated earlier in the report, the material composing each of these structures appeared to be relatively similar. Furthermore, the foundation material below the embankment also appeared to be similar at each location. In order to evaluate the strength characteristics of the embankment material for these 2 facilities, 5 triaxial shear tests were performed on representative samples obtained from test borings No. 9 and No. 17. The Mohr Envelope for consolidated drained shear tests performed on samples obtained from test boring No. 9 are presented in Figure No. 8, while the Mohr Envelope for consolidated drained shear tests performed on representative samples taken from test hole No. 17 are presented in Figure No. 9.

It will be observed that the friction angle for the material obtained from test pit No. 9 was 31.1° with 0 cohesion while the Mohr Envelope for the material obtained from test hole No. 17 had a friction angle of 30° and 5 pounds per square inch cohesion.

The shearing strength of the foundation material in the vicinity of the Clearwater Dike was evaluated by performing 3 triaxial shear tests on representative samples obtained from test hole No. 16. The Mohr Envelope corresponding to these 3 triaxial shear tests are presented in Figure No. 10. It will be observed that the friction angle for this material was equal to 32.6^0 with a cohesion of about 4 pounds per square inch. One additional triaxial shear tests was performed on a representative sample of the embankment material obtained from test boring No. 9. The failure circle for this sample is presented in Figure No. 11.

Assuming no cohesion for the above material, a friction angle of 36.9^0 is obtained. It should be noted that this friction angle is substantially greater than other friction angles obtained for the embankment material for these two dikes. It should be recognized, however, that the subsurface material within the embankments is quite heterogeneous and that no 2 samples taken throughout the soil profile are identical. Hence, one would expect some variation in the strength characteristics of the subsurface material.

The results of the triaxial tests outlined above form the basis for the selection of the shear strength parameters in performing the stability computations for the embankments in this area.

IV. STABILITY ANALYSIS

An effective stress stability analysis was performed on the various embankments located in this area. The shear strength parameters used in the stability analysis were based upon the consolidated drained shear test information discussed in the previous section of the report.

In performing the effective stress analysis, pore pressures were estimated based upon the observations of the groundwater level and the environmental conditions throughout the site. The results of the stability analysis for each of the embankments located in the area are discussed below as follows.

1. Clearwater Dike

The idealized profile conditions within the embankment and the foundation at this site along with shear strength parameters used in the stability analysis are presented in Figure No. 12. It is assumed that the embankment is a low plasticity soil with a friction angle of 30^0 and a cohesion of 800 pounds per square foot. The foundation material consisting predominately of cohesive-type soils is designated as F1 in Figure No. 12 and it is assumed to have a uniform thickness throughout the cross-section of the embankment. F2 material is assumed to be the medium dense granular material located below the foundation cohesive soils. For the purpose of the stability analysis, the F1 material is assumed to have a friction angle of 26^0 and a cohesion of either 400 or 100 pounds per square foot. The F2 material is assumed to have a friction angle of 34^0 and 0 cohesion. The 34^0 friction angle for the sandy gravelly material was estimated based upon the results of standard penetration values within the gravelly material.

Stability computations were performed for the case of the downstream slope under steady-state seepage conditions and for the upstream slope under the case of sudden drawdown. The stability analysis was performed for 2 conditions: for both the steady-state seepage case and the sudden drawdown case.

Condition No. 1 assumed that the F1 material had a friction angle of 26° and a cohesion of 400 pounds per square foot, while Condition No. 2 assumed that the F1 material had a friction angle of 26° and a cohesion of 100 pounds per square foot.

The piezometric surface assumed for both the steady-state seepage case and the sudden drawdown seepage case is presented in Figure No. 12. The factors of safety obtained for both the steady-state seepage case and the sudden drawdown case for Conditions No. 1 and No. 2 are also presented in Figure No. 12.

The results of these tests indicate factors of safety of 2.2 and 1.82 for the steady-state seepage case and 1.84 and 1.24 for the sudden drawdown case for Conditions No. 1 and No. 2. It is our opinion that a cohesion of 100 pounds per square foot is very conservative and that the actual cohesion for this material is substantially greater than this value.

It is apparent from the above results that the existing dikes have an adequate factor of safety for both the case of steady-state seepage and the case of sudden drawdown. It is also our opinion that this dike could be raised another 3 feet without endangering the safety of the existing facility. Furthermore, it is our opinion that the reduction in the factor of safety would be relatively small.

2. Lower Refuse Dike

The physical features and the soil parameters used in determining the stability of this facility are presented in Figure No. 13. As indicated in the previous section, the subsurface material within this embankment is very similar to the material in the Clearwater Dike, and the same profile has been assumed for this facility as for the Clearwater Dike.

In its operating condition the water level in the upstream side of the dike is only a few feet greater than the water level in the downstream portion of the dike. The approximate water level through the dike is shown in Figure No. 13. In its existing condition, this facility is in a very stable condition and no attempt was made to determine the factor of safety for the structure with high water on either side of the dike. A stability analysis was performed, however, for a sudden drawdown of the Clearwater on the downstream side of the dam.

The estimated piezometric surface on the downstream side of the dam during the sudden drawdown condition is presented in Figure No. 13. It is our opinion that this is a very conservative estimate of the piezometric surface. The soil parameters used in the stability analysis for this facility are presented in Figure No. 13, and it will be observed that the parameters used for this facility are essentially the same as those for the Clearwater Dike. Three conditions, however, were considered to obtain an indication in the range of the factor of safety for the existing facility under the condition of sudden drawdown on the downstream portion of the structure.

Condition No. 1 assumed that the cohesion of the material in Zone No. 1 was 800 pounds per square foot, while the cohesion for F1 material was 100 pounds per square foot. Condition No. 2 assumed that the cohesion in the Zone No. 1 material was 400 pounds per square foot, while the cohesion for the foundation cohesive material was 400 pounds per square foot. Condition No. 3 assumed that the cohesion for Zone No. 1 was 400 pounds per square foot and that the cohesion in the foundation material for this facility was 100 pounds per square foot.

It will be observed that the factors of safety for this condition were all greater than 1 with Condition No. 3 having the lowest factor of safety. It is our opinion the Condition No. 3 is ultraconservative and that the factors of safety corresponding to Condition No. 1 and Condition No. 2 most reasonably represent the factor of safety of the downstream slope for this dike under a sudden drawdown condition.

Based upon the results of the stability analysis performed herein, it is our opinion that this dike could be raised 3 feet without endangering the safety of the structure and that raising the structure 3 feet would not result in a substantial decrease in the factors of safety.

3. <u>Upper Refuse Dike</u>

The physical conditions for this facility along with shear strength parameters used in the stability analysis are presented in Figure No. 14. The idealized profile throughout the embankment and foundation is also presented in Figure No. 14. The material within the embankment is assumed to be coal refuse while the foundation material immediately below the coal refuse is assumed to be a low plasticity cohesive material. This material is designated as F1 in Figure No. 14.

The material below the cohesive material is assumed to be a granular soil in a medium dense state and is designated as F2 in Figure No. 14. The friction angle for the coal refuse material has been assumed equal to 28.5^0 in accordance with the shear test on this material, while the friction angle for the F1 material and F2 material is 35^0 and 34^0, respectively. None of this material was assumed to have cohesive characteristics.

In its operating condition, tailings and water exist on both sides of this embankment, and the facility is entirely stable under the existing conditions.

In order to obtain an indication of the stability under severe operating conditions, it was assumed that all of the water downstream from this facility was drawn down and that a steady-state seepage case would occur. The piezometric surface assumed in performing the stability computations is presented in Figure No. 14. The results of the stability analysis performed for this condition indicated a factor of safety of 1.22.

Since no danger or loss of life would occur if there was a failure for this facility, it is our opinion that a factor of safety of 1.22 is entirely satisfactory for the proposed structure. Furthermore, it is our opinion that this embankment can be raised by 3 feet without seriously endangering the safety of the structure.

4. North Dike

The North Dike was formed by dumping the material excavated for a trench as shown in Figure No. 14. As indicated earlier in the report, the embankment formed from the excavated material was randomly placed and insofar as we can ascertain, no attempt was made to compact this material in accordance with well accepted engineering standards. The embankment is apparently stable under the conditions in which it exists. However, it is our opinion that this dike should not be raised further without attention being given to a more satisfactory design procedure.

In order to obtain an indication of the characteristics of the material within the dike, three test pits were excavated to depths of between

12 and 15 feet. The logs for these three test pits are presented in Figure No. 15. It will be observed that the subsurface material within the dike consists predominately of brown silty clay with shale fragments.

In-place density tests were also performed on the subsurface material within this embankment, and the results of these tests are presented on the boring logs. It will be observed that the in-place density of the subsurface material at this location varied from 97 pounds per cubic foot to 108 pounds per cubic foot. The in-place densities obtained during this investigation are substantially less than would be expected for this material at 95 percent of the proctor density. In view of the relatively low density characteristics of this material, we recommend that if this dike is used to retain water that it be reorganized in a manner that would result in a stable condition.

We recommend that the material within the dike either be spread out and compacted to form the base of a new dike in this area or that a new dike be constructed on the downstream portion of this area in accordance with accepted design standards.

V. RIPRAP MATERIAL

A visual observation was made of the riprap material used on the Clearwater Dike and the Lower Refuse Dike.

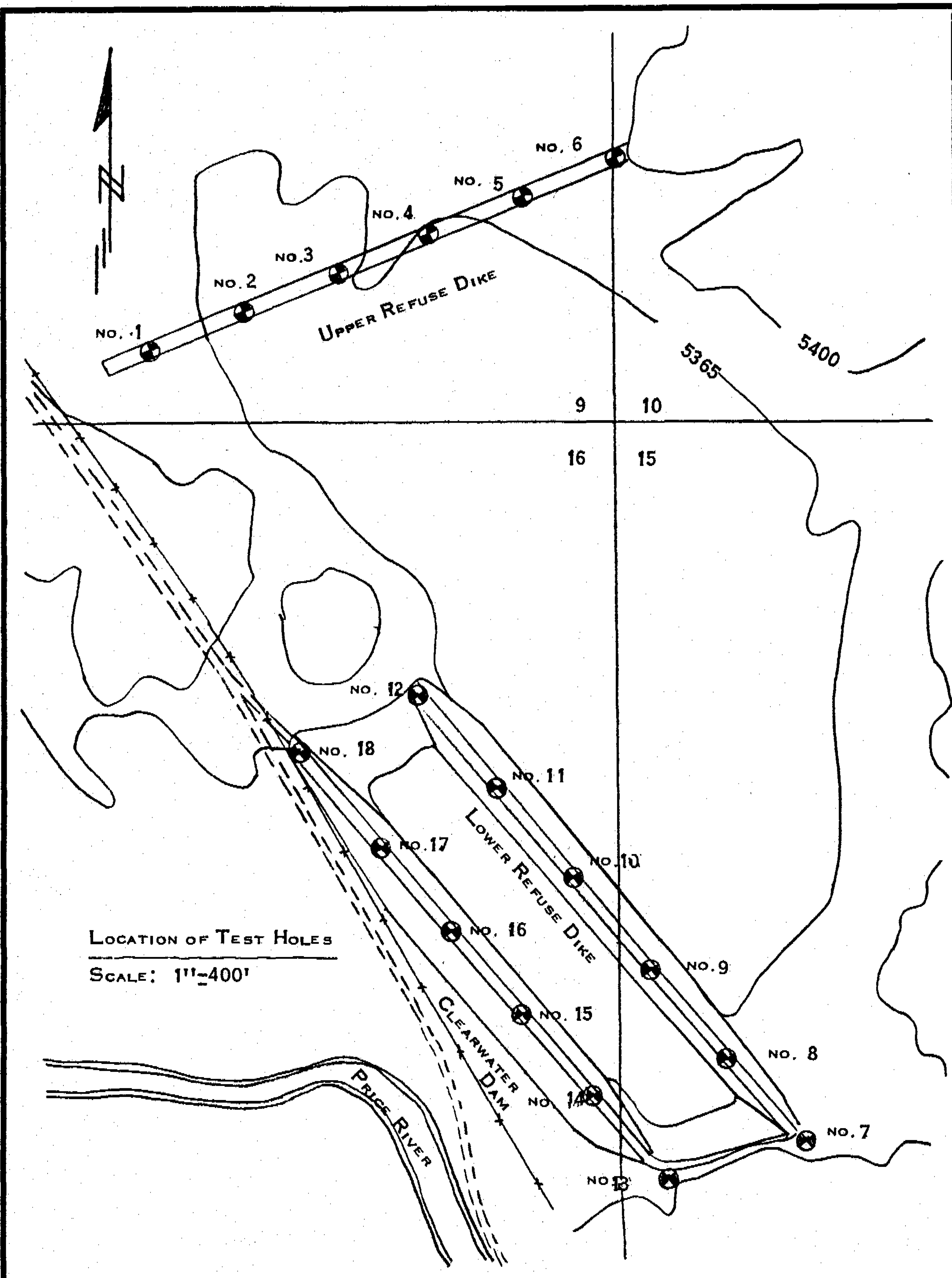
1. Clearwater Dike

The downstream face of this dike has a 1 to 1.5 foot layer of brown silty sandy gravel overlying the embankment material. The maximum gravel size is approximately 5 inches. The average size is less than 1 inch.

The upstream face has this same material from the crest to a height of 3 to 5 feet below the crest. This is followed by a layer of sandstone riprap which extends an unknown distance down the face. The sandstone particles are up to 3 feet in diameter with an average size of approximately 1 foot. The sandstone shows some erosion but it is our opinion that it is providing adequate protection. Periodic inspections should be made to evaluate the sandstone performance and the need for maintenance.

2. Lower Refuse Dike

From the crest to a distance of 5 to 7 feet down the face on the downstream side of the dike (enclosing the clearwater pond) there is a layer of black shale fragments and coal refuse on the southern half. This layer is followed by a sandstone layer from 5 to 7 feet down the face to an unknown distance. There are areas on the southern half of the dike where the water level is over the sandstone. It is our opinion that the sandstone riprap should be increased to a height of at least 3 feet above the water level. The northern half has a layer of silty sandy gravel similar to the Clearwater Dike down to the sandstone layer. The upstream slope is covered with coal refuse to the water level at a distance of approximately 4 feet down the face. No qualitative analysis could be made as to the riprap performance because of the high water level. It is apparent, however, that the water level is above any sandstone layer that may exist. No evidence of erosion appeared to be occurring.



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LOCATION OF TEST HOLES FOR U.S. STEEL CORPORATION COAL PREP PLANT REFUSE DIKES WELLINGTON, UTAH

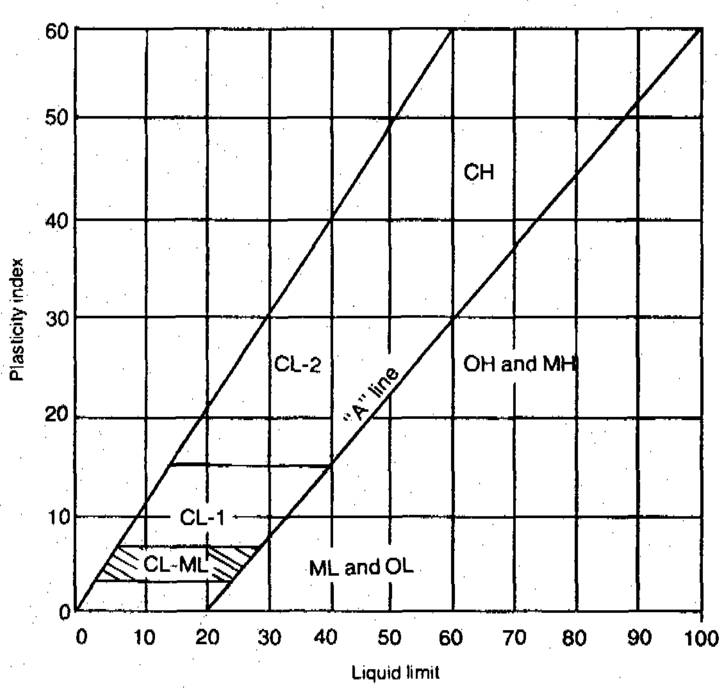
FIGURE No. 1

Bruning "500"

Figure No. 2

Unified Soil Classification System

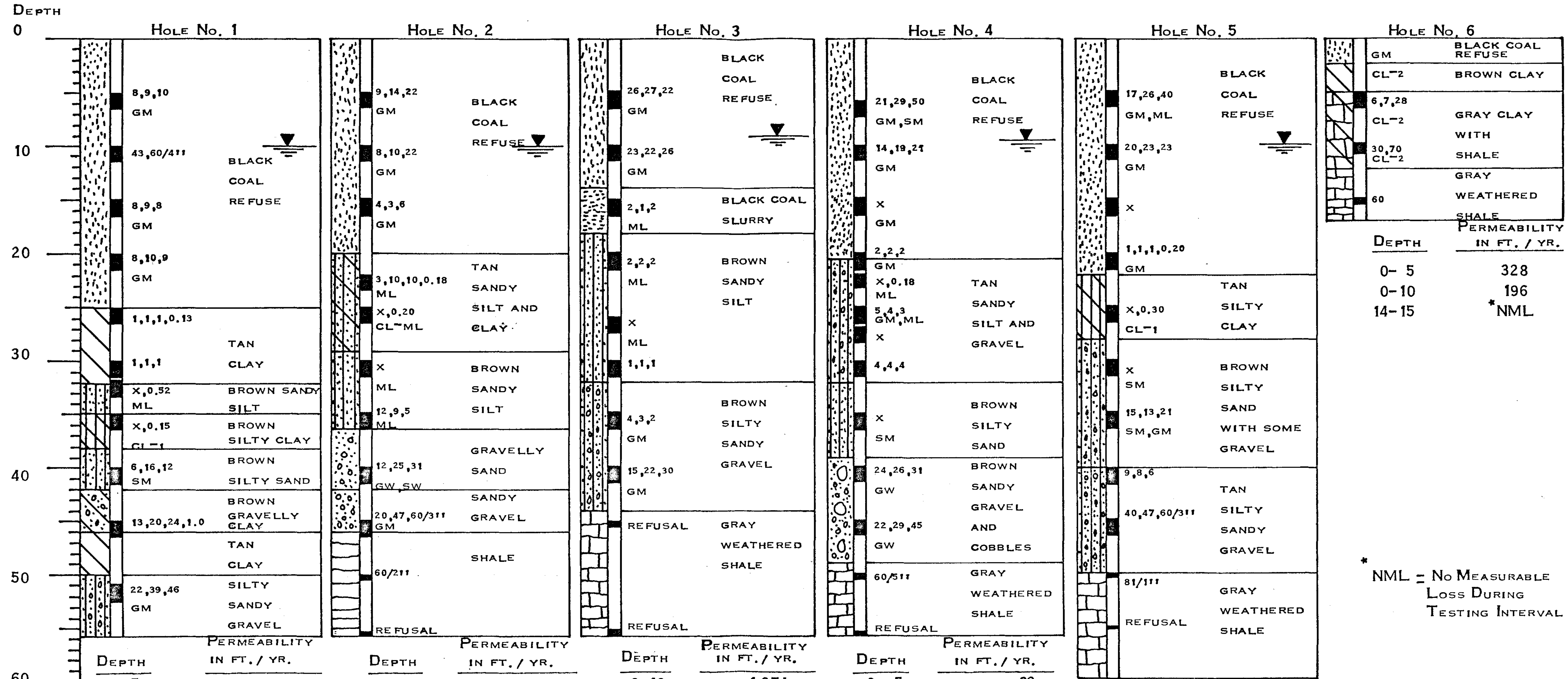
Major divisions			Group symbols	Typical names	Laboratory classification criteria		
Coarse-grained soils (More than half of material is larger than No. 200 sieve size)	Gravels (More than half of coarse fraction is larger than No. 4 sieve size)	Clean gravels (Little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines	Determine percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows: Less than 5 percent ... GW, GP, SW, SP; More than 12 percent ... GM, GC, SM, SC; 5 to 12 percent ... Borderline cases requiring dual symbols**	$Cu = \frac{D_{60}}{D_{10}}$ greater than 4, $Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines		Not meeting all gradation requirements for GW	
		Gravels with fines (Appreciable amount of fines)	GM* d/u	Silty gravels, gravel-sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Above "A" line with P.I. between 4 and 7 are borderline cases requiring use of dual symbols.
			GC	Clayey gravels, gravel-sand-clay mixtures		Atterberg limits above "A" line with P.I. greater than 7	
	Sands (More than half of coarse fraction is smaller than No. 4 sieve size)	Clean sands (Little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines		$Cu = \frac{D_{60}}{D_{10}}$ greater than 6, $Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			SP	Poorly graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for SW	
		Sands with fines (Appreciable amount of fines)	SM* d/u	Silty sands, sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Limits plotting in hatched zone with P.I. between 4 and 7 are borderline cases requiring use of dual sysbols.
			SC	Clayey sands, sand-clay mixtures		Atterberg limits above "A" line with P.I. less than 7	
Fine-grained soils (More than half of material is smaller than No. 200 sieve)	Silts and clays (Liquid limit less than 50)		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity			
			CL 1/2	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays			
			OL	Organic silts and organic silty clays of low plasticity			
	Silts and clays (Liquid limit greater than 50)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts			
			CH	Inorganic clays of high plasticity, fat clays			
			OH	Organic clays of medium to high plasticity, organic silts			
Highly organic soils			Pt	Peat and other highly organic soils			



Plasticity Chart

*Division of GM and SM groups into subdivisions of d and u for roads and airfields only. Subdivision is based on Atterberg limits, uffix d used when L. L. is 28 or less and the P.I. is 6 or less, the suffix u used when L. L. is greater than 28.

** Borderline classifications, used for soils possessing characteristics of two groups, are designated by combinations of group symbols. 'or example: GW-GC, well-graded gravel-sand mixture with clay binder.



Hole No. 1

Depth	Permeability in ft./yr.
0- 5	984
0-10	49
10-15	112
15-20	>15,000
20-25	>15,000
25-30	3,727
30-35	567
35-40	301
40-45	12
45-47	>20,000

Hole No. 2

Depth	Permeability in ft./yr.
0- 5	328
0-10	110
10-15	>15,000
15-20	>15,000
20-25	5,206
25-30	142
30-35	6,248
35-40	7,587
40-45	>20,000
46-50	1,670
46-55	3,693

Hole No. 3

Depth	Permeability in ft./yr.
0-10	1,274
20-26	7,353
25-30	8,945
30-35	12,449
35-40	17,891
45-47	31,104
46.5-50	13
46.5-55	800

Hole No. 4

Depth	Permeability in ft./yr.
0- 5	82
0-10	12
10-15	2,894
15-20	33
20-25	6,517
25-30	4,768
30-35	5,750
35-40	24,437
40-45	720,000
45-50	11,500
50-55	13
50-60	21

Hole No. 5

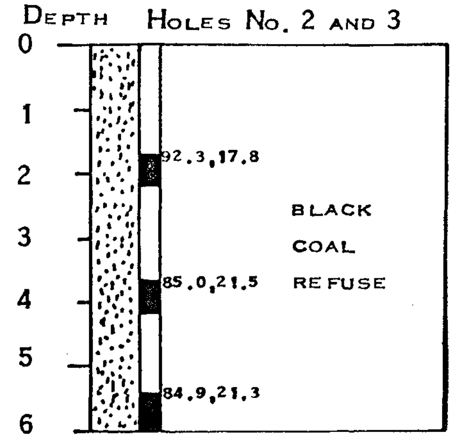
Depth	Permeability in ft./yr.
0- 5	246
0-10	98
9-15	11,086
15-20	9
20-25	7,455
25-30	10,543
30-35	14,365
35-40	15,083
40-45	351
45-50	>21,000
50-55	465
55-60	1,881

Hole No. 6

Depth	Permeability in ft./yr.
0- 5	328
0-10	196
14-15	*NML

*NML = No Measurable Loss During Testing Interval

Test Pit Between Holes No. 2 and 3



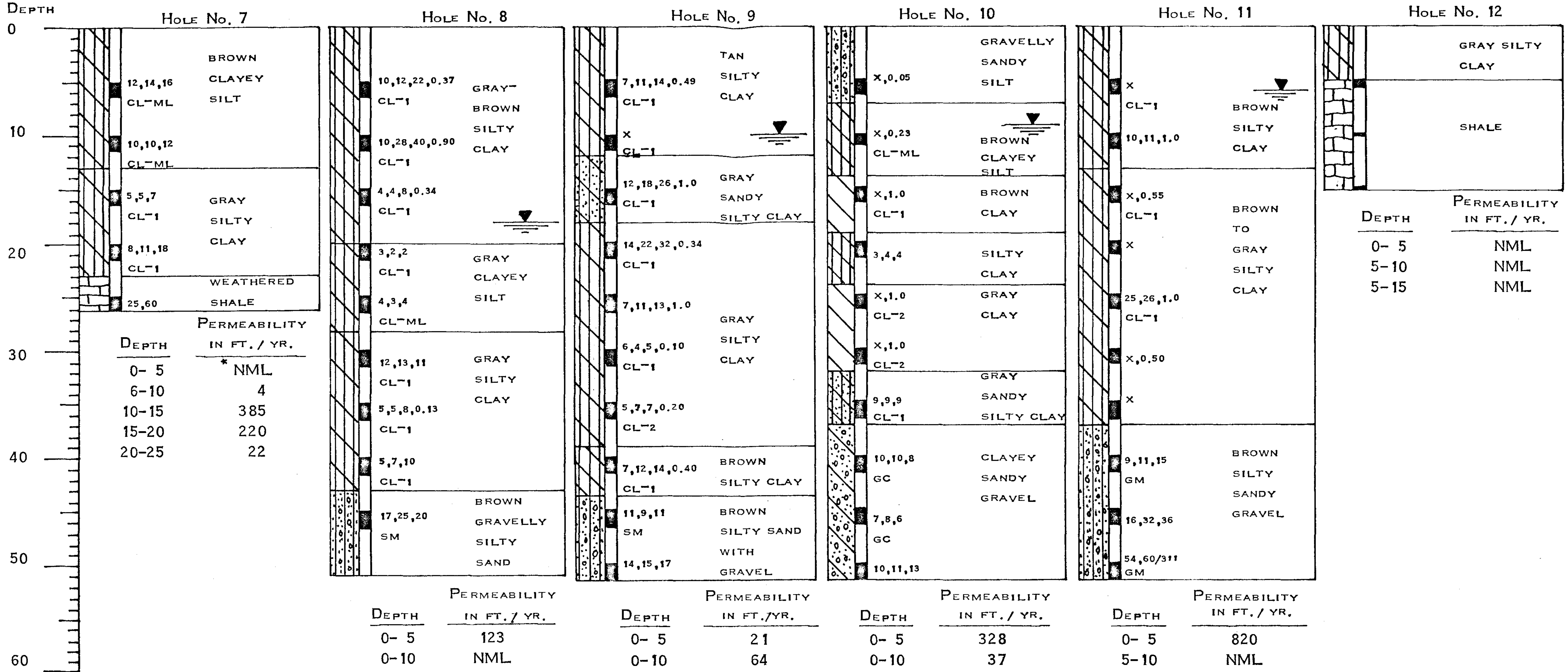
Scale	
Designed	Checked
Drawn	Date
Approved	License No.

ROLLINS, BROWN & GUNNELL, Inc.
CONSULTING ENGINEERS

LOG OF BORINGS FOR THE UPPER REFUSE DIKE
U.S. STEEL CORPORATION, WELLINGTON, UTAH

FIGURE NO. 3

Bruning "500"



Hole No. 7

Depth	Permeability in ft./yr.
0- 5	*NML
6-10	4
10-15	385
15-20	220
20-25	22

Hole No. 8

Depth	Permeability in ft./yr.
0- 5	123
0-10	NML
10-15	NML
15-20	1,108
20-25	1,097
25-30	43
30-35	659
35-40	775
40-45	1,550
45-50	3,765

Hole No. 9

Depth	Permeability in ft./yr.
0- 5	21
0-10	64
10-15	38
15-20	58
20-25	NML
25-30	22
30-35	15
35-40	137
40-45	1,302
45-50	1,065

Hole No. 10

Depth	Permeability in ft./yr.
0- 5	328
0-10	37
10-15	68
15-20	
20-25	
25-30	30
30-35	598
31-40	244
40-45	1,144
45-50	1,004

Hole No. 11

Depth	Permeability in ft./yr.
0- 5	820
5-10	NML
10-15	4
15-20	2,392
20-25	NML
25-30	26
30-35	68
35-40	5,325
40-45	9,535
45-50	7,193

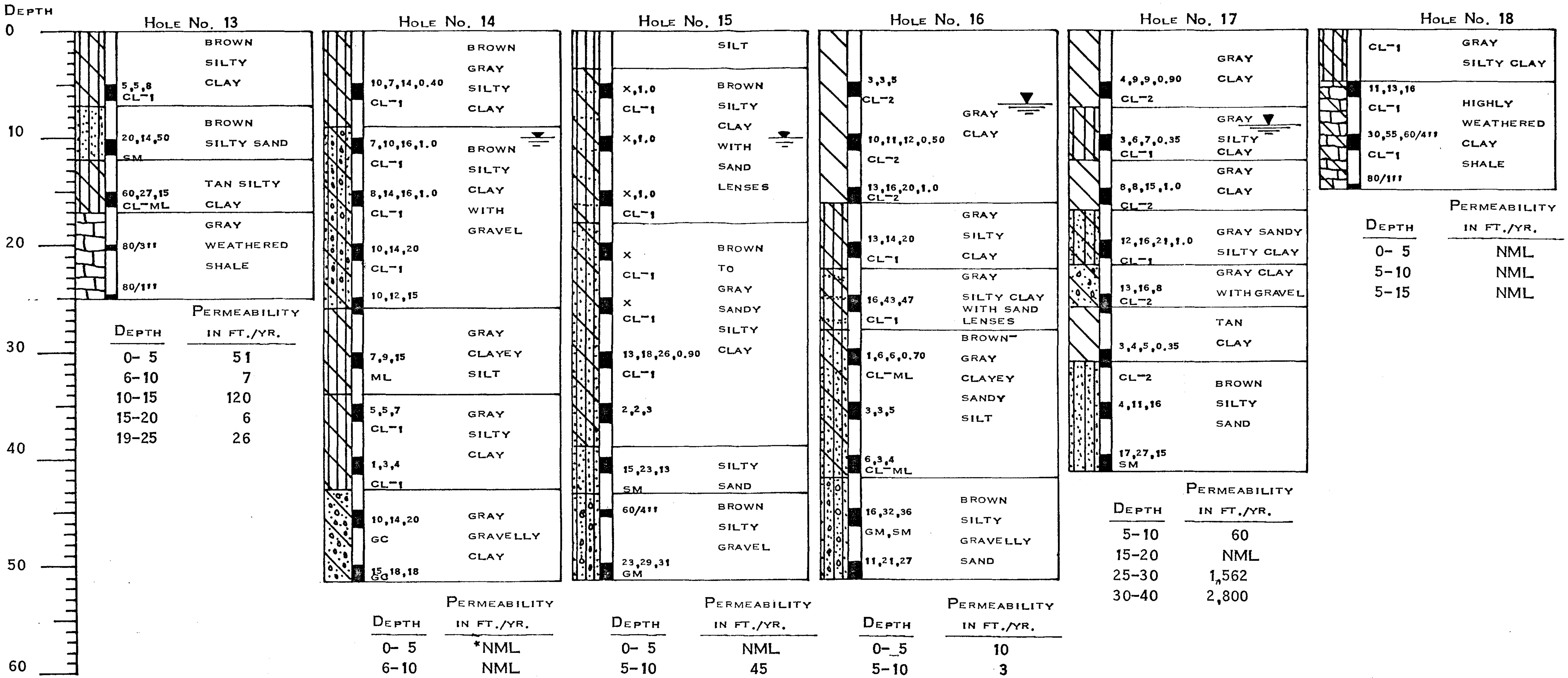
Hole No. 12

Depth	Permeability in ft./yr.
0- 5	NML
5-10	NML
5-15	NML

*NML= No Measurable Loss During Testing Interval

SCALE DESIGNED ____ CHECKED ____ DRAWN ____ DATE ____ APPROVED ____ LICENSE NO. ____	ROLLINS, BROWN & GUNNELL, Inc. CONSULTING ENGINEERS	LOG OF BORINGS FOR THE LOWER REFUSE DIKE U.S. STEEL CORPORATION, WELLINGTON, UTAH	FIGURE NO. 4

Bruning "500"



Hole No. 13

DEPTH	PERMEABILITY IN FT./YR.
0- 5	51
6-10	7
10-15	120
15-20	6
19-25	26

Hole No. 14

DEPTH	PERMEABILITY IN FT./YR.
0- 5	*NML
6-10	NML
10-15	2
15-20	2
20-25	NML
25-30	259
30-35	8
35-40	51
40-45	1,885
45-50	1,449

Hole No. 15

DEPTH	PERMEABILITY IN FT./YR.
0- 5	NML
5-10	45
10-15	NML
15-20	22
20-25	NML
25-30	NML
30-35	61
35-40	752
40-45	1,028
45-50	1,268

Hole No. 16

DEPTH	PERMEABILITY IN FT./YR.
0- 5	10
5-10	3
10-15	6
15-20	NML
20-25	3
25-30	7
30-35	6
35-40	64
40-45	1,414
45-50	709

Hole No. 17

DEPTH	PERMEABILITY IN FT./YR.
5-10	60
15-20	NML
25-30	1,562
30-40	2,800

Hole No. 18

DEPTH	PERMEABILITY IN FT./YR.
0- 5	NML
5-10	NML
5-15	NML

*NML= No Measurable Loss During Testing Interval

SCALE DESIGNED ____ CHECKED ____ DRAWN ____ DATE ____ APPROVED ____ LICENSE NO. ____	**ROLLINS, BROWN & GUNNELL, Inc.** CONSULTING ENGINEERS	LOG OF BORINGS FOR THE CLEARWATER DIKE U.S. STEEL CORPORATION, WELLINGTON, UTAH	FIGURE NO. 5

Bruning "500"

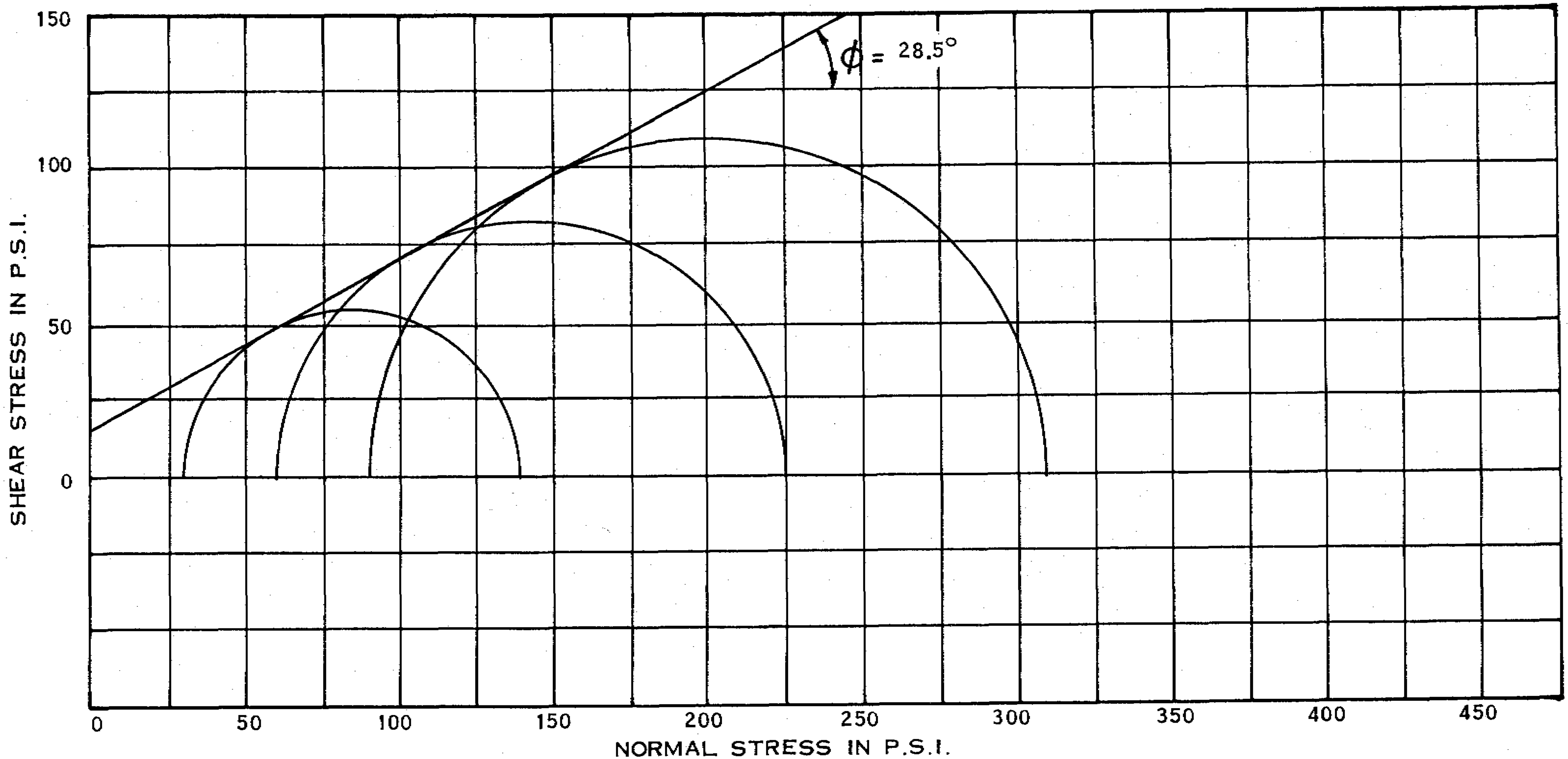


FIGURE 6 SHEAR TEST RESULTS

TYPE OF TEST Consolidated Drained

TEST PIT NO. COAL REFUSE DEPTH 0-10'

MAX. DENSITY ______ LBS/FT.^{3} MOISTURE 18.0 %

SAMPLE DENSITY 103.0 LBS/FT.^{3} SAMPLE SIZE 1.3" DIA.

PROJECT U.S. Steel, Wellington, Utah

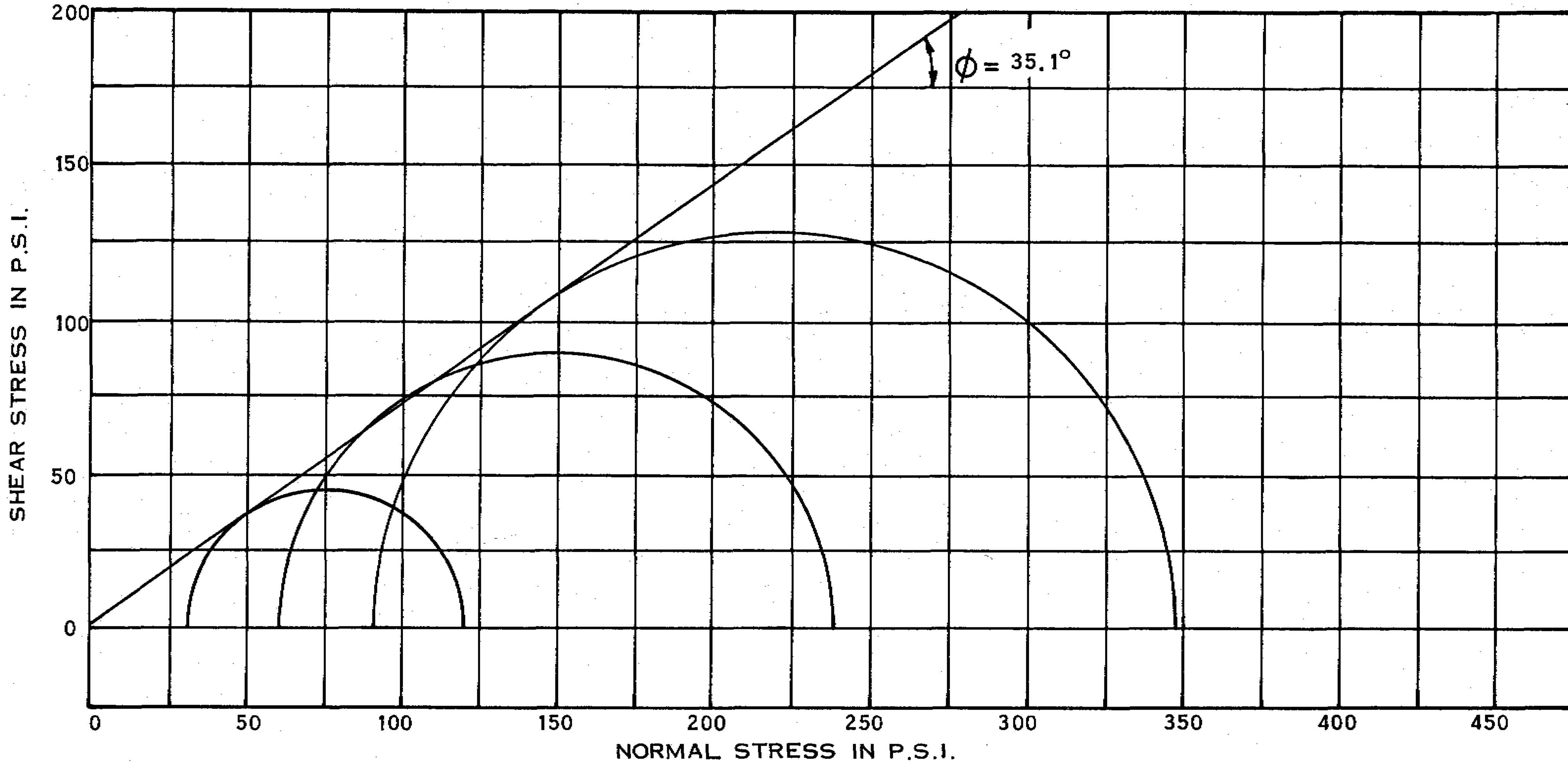


FIGURE 7 SHEAR TEST RESULTS

TYPE OF TEST CONSOLIDATED DRAINED

TEST PIT NO. 3 DEPTH 25-26.5'

MAX. DENSITY ______ LBS/FT.^{3} OPT. MOISTURE 20.0 %

SAMPLE DENSITY 118.0 LBS/FT.^{3} SAMPLE SIZE 1.3" DIA.

PROJECT U.S. STEEL, WELLINGTON, UTAH

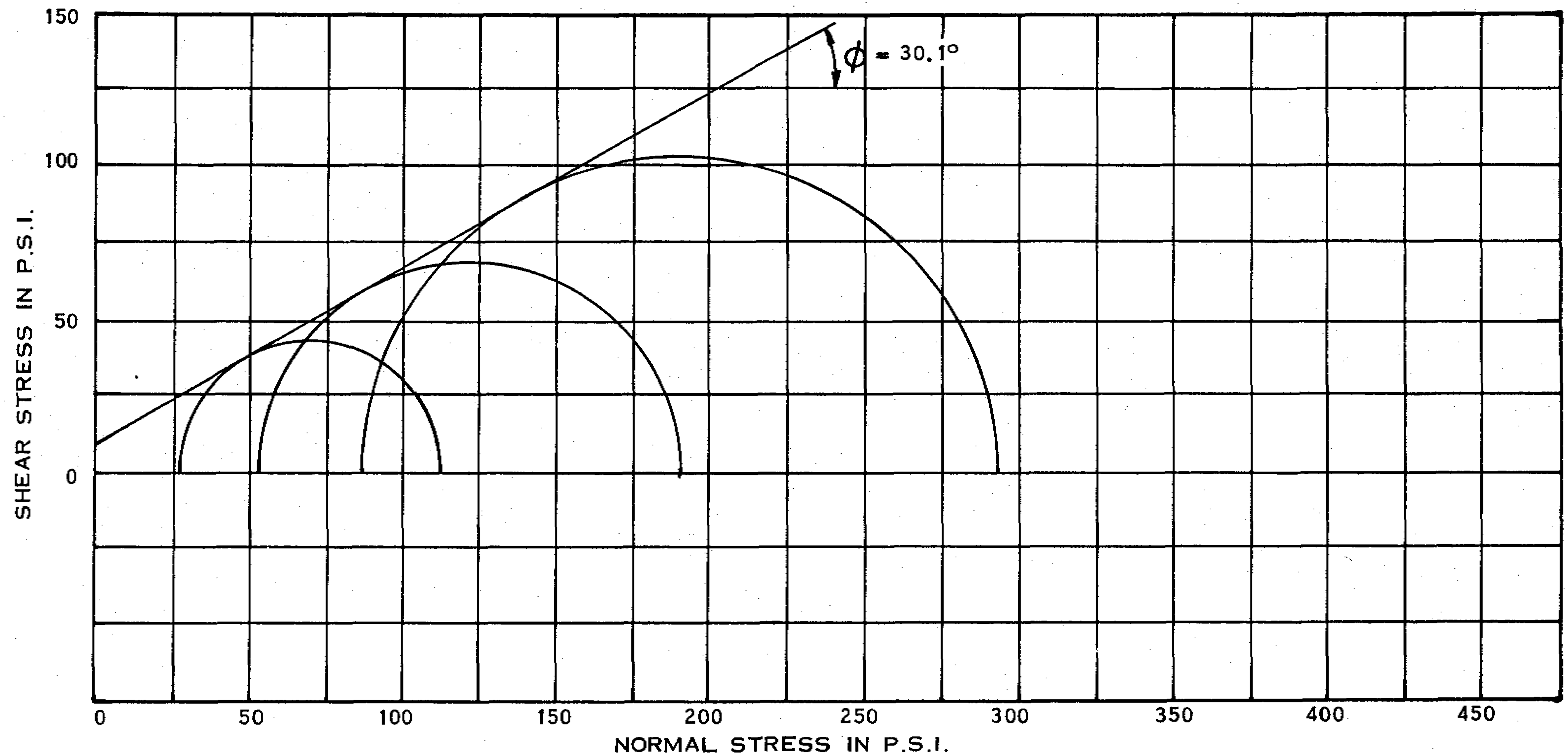


Figure	9 — SHEAR TEST RESULTS
Type of Test	Consolidated Drained
Test Pit No.	17
Depth	10–11.5'
Max. Density	lbs/ft.³
Moisture	10.0 %
Sample Density	132.0 lbs/ft.³
Sample Size	1.3" dia.
Project	U.S. Steel, Wellington, Utah

VS STEEL (X)

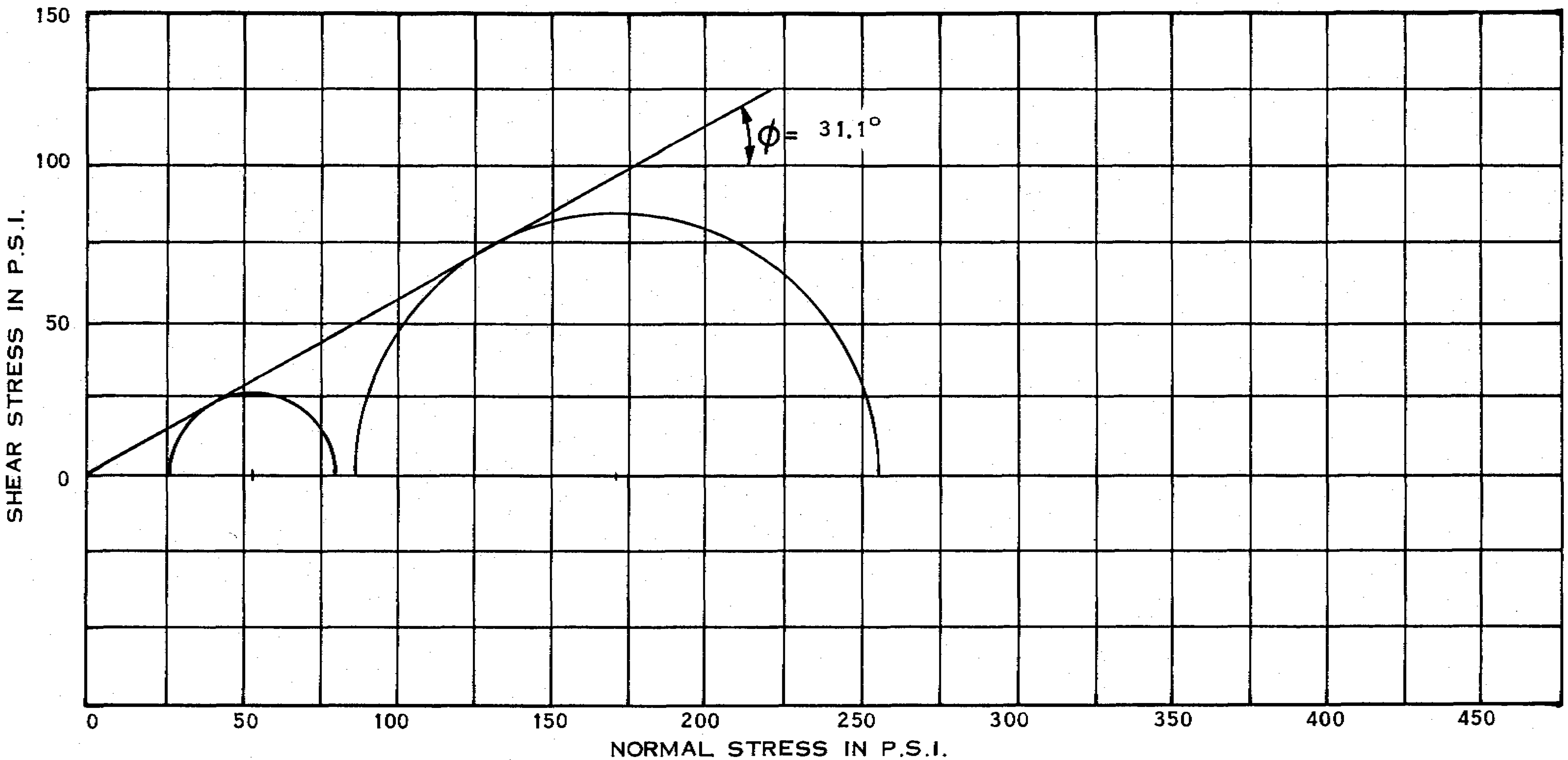


FIGURE 8 SHEAR TEST RESULTS

TYPE OF TEST CONSOLIDATED DRAINED

TEST PIT NO. 9 DEPTH 25-30'

MAX. DENSITY ______ LBS/FT.³ MOISTURE 16.0 %

SAMPLE DENSITY 110.0 LBS/FT.³ SAMPLE SIZE 1.3" DIA.

PROJECT U.S. STEEL, WELLINGTON, UTAH

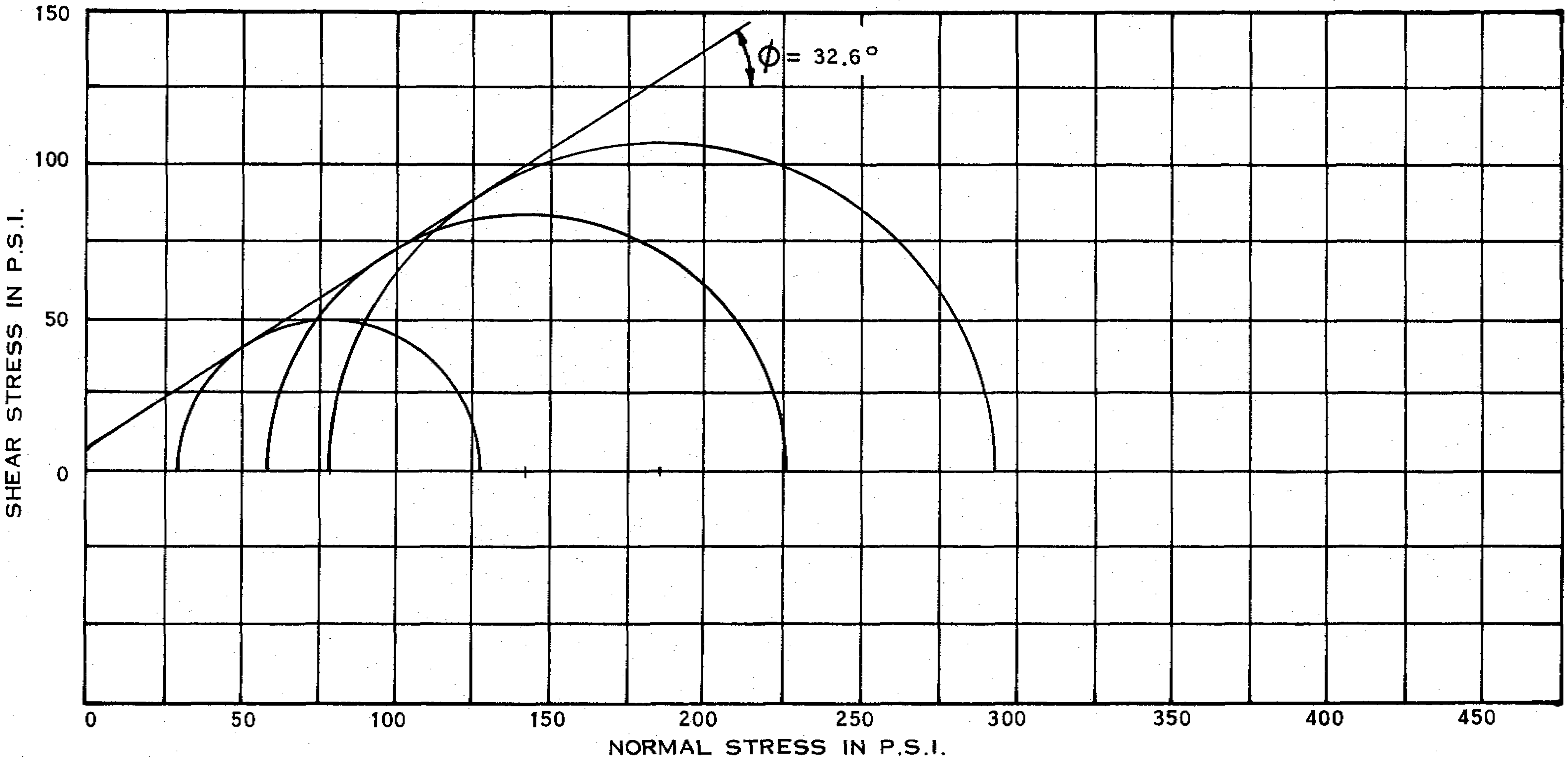


FIGURE 10 SHEAR TEST RESULTS

TYPE OF TEST CONSOLIDATED DRAINED

TEST PIT NO. 16 DEPTH 35-36.5'

MAX. DENSITY ____ LBS/FT.³ MOISTURE 19.0 %

SAMPLE DENSITY 97.3 LBS/FT.³ SAMPLE SIZE 1.3" DIA.

PROJECT U.S. STEEL, WELLINGTON, UTAH.

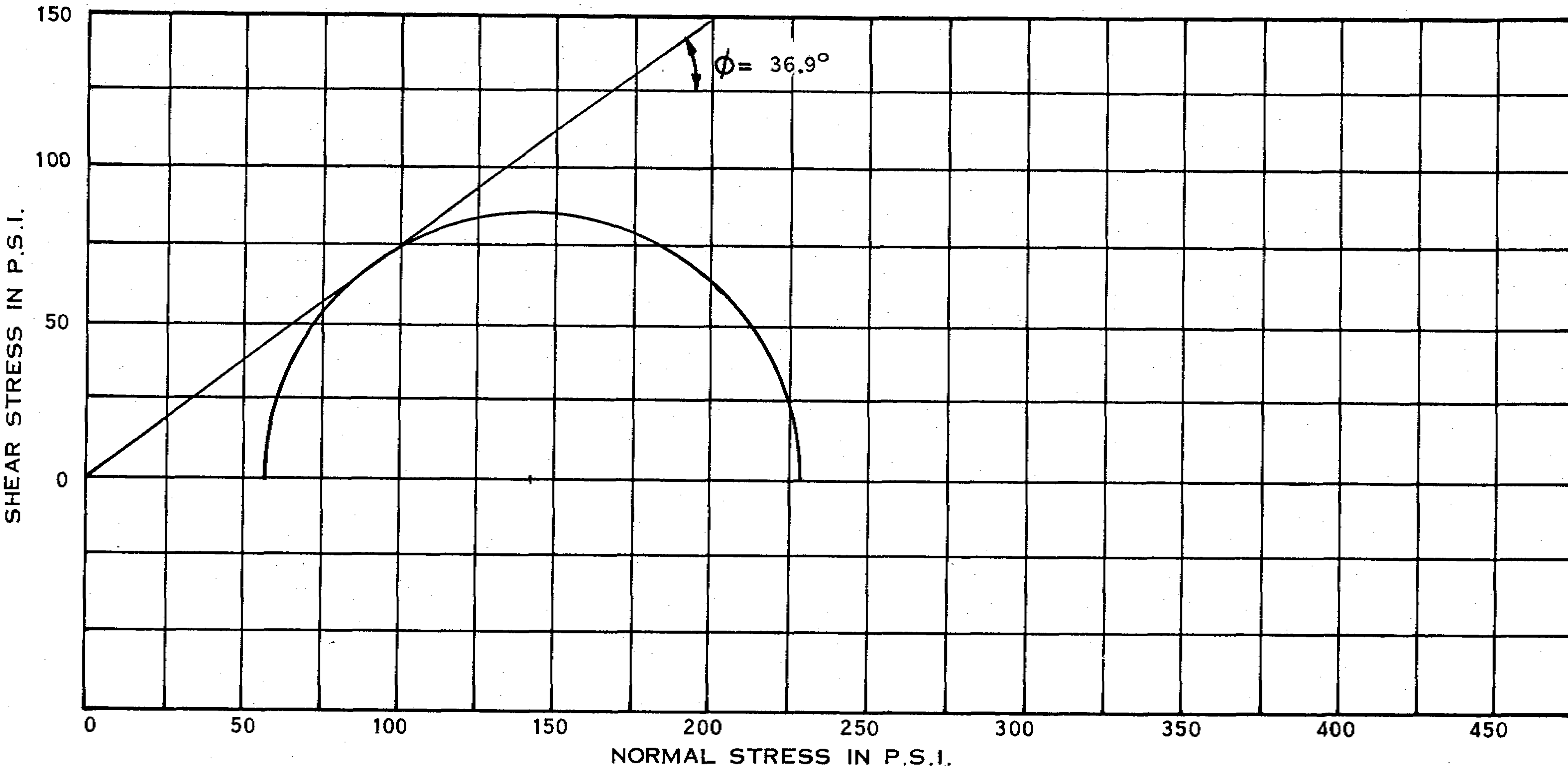


FIGURE 11 SHEAR TEST RESULTS

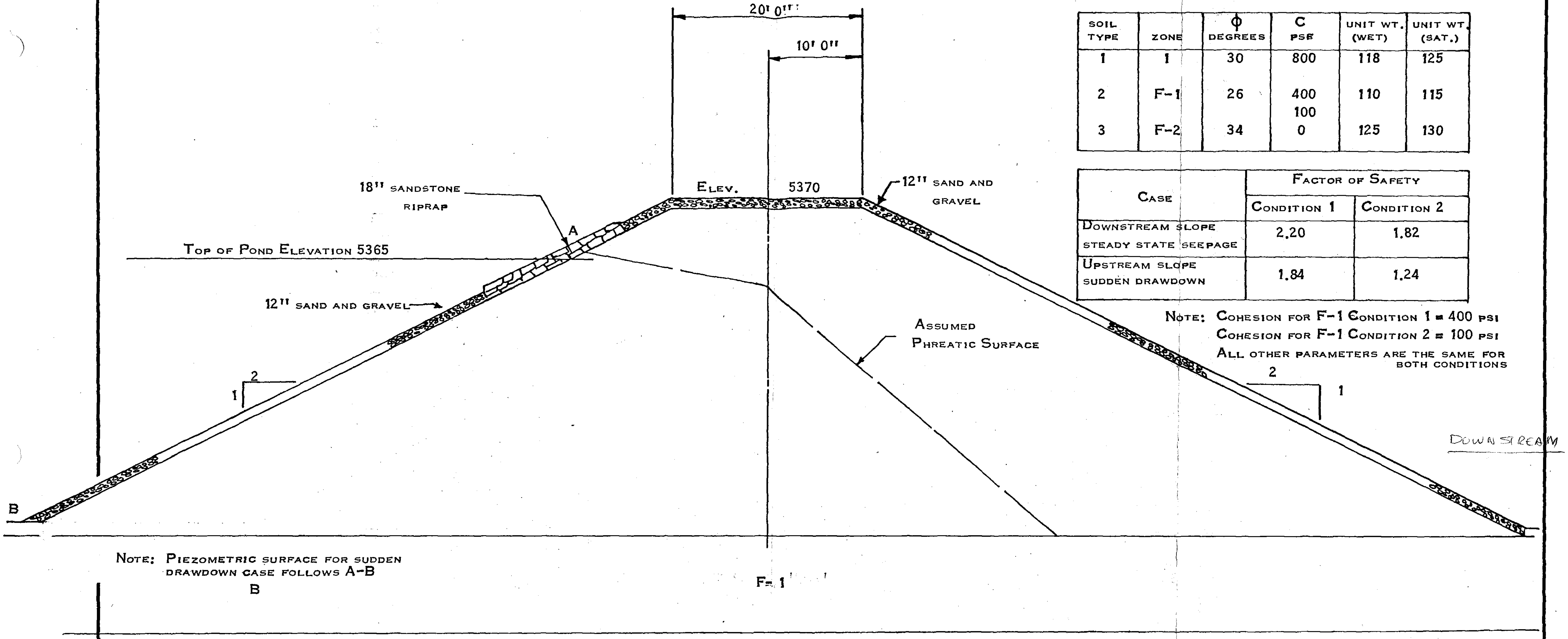
TYPE OF TEST CONSOLIDATED DRAINED

TEST PIT NO. 9 DEPTH 15'

MAX. DENSITY ______ LBS/FT.³ MOISTURE 11.0 %

SAMPLE DENSITY 122.4 LBS/FT.³ SAMPLE SIZE 1.3" DIA.

PROJECT U.S. STEEL, WELLINGTON, UTAH



Soil Type	Zone	Φ Degrees	C PSF	Unit Wt. (Wet)	Unit Wt. (Sat.)
1	1	30	800	118	125
2	F-1	26	400 100	110	115
3	F-2	34	0	125	130

Case	Factor of Safety	
	Condition 1	Condition 2
Downstream slope steady state seepage	2.20	1.82
Upstream slope sudden drawdown	1.84	1.24

Note: Cohesion for F-1 Condition 1 = 400 psi
Cohesion for F-1 Condition 2 = 100 psi
All other parameters are the same for both conditions

Note: Piezometric surface for sudden drawdown case follows A-B

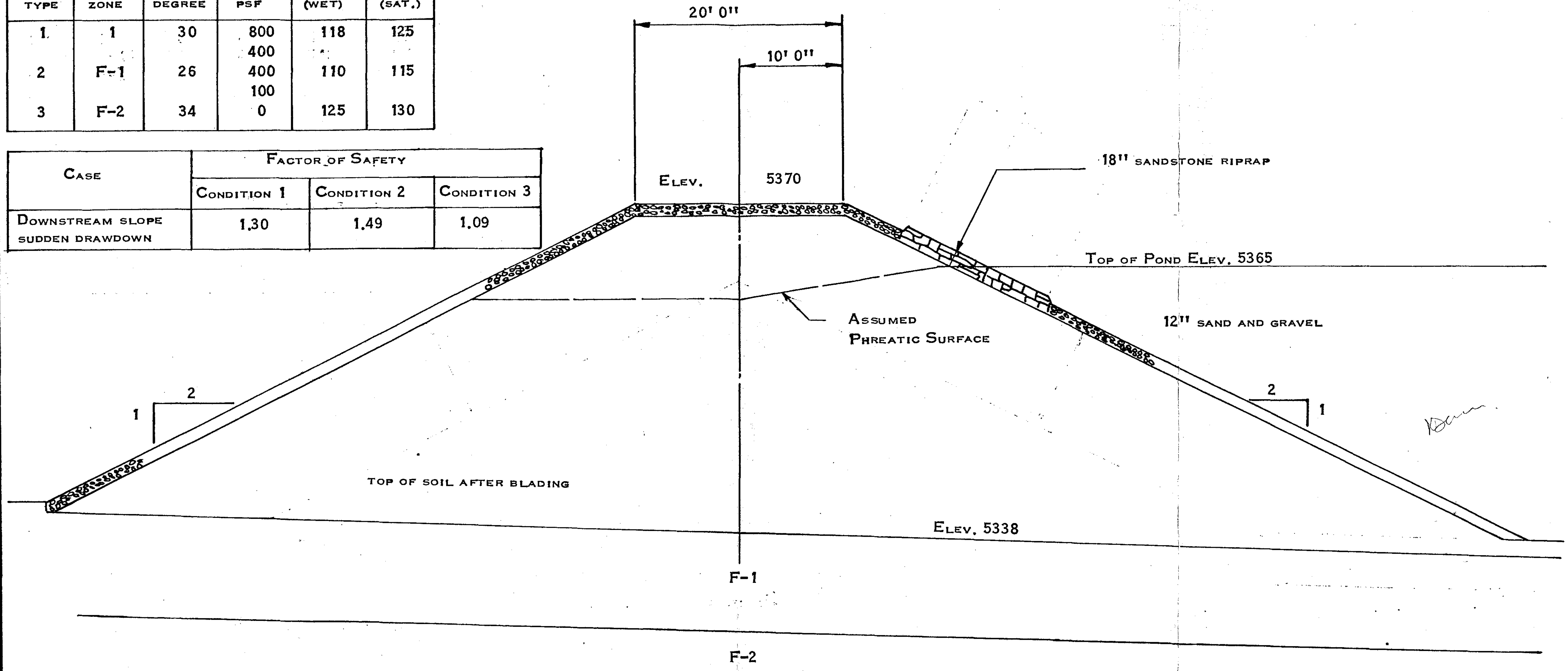
Scale: 1" = 10'

SCALE ___ DESIGNED ___ CHECKED ___ DRAWN ___ DATE ___ APPROVED ___ LICENSE NO. ___	**ROLLINS, BROWN & GUNNELL, Inc.** CONSULTING ENGINEERS	PHYSICAL CHARACTERISTICS AND MATERIAL PROPERTIES USED IN STABILITY ANALYSIS FOR CLEARWATER DIKE U.S. STEEL CORPORATION, WELLINGTON, UTAH	FIGURE NO. 12

Bruning "500"

SOIL TYPE	ZONE	Φ DEGREE	C PSF	UNIT WT. (WET)	UNIT WT. (SAT.)
1	1	30	800 400	118	125
2	F-1	26	400 100	110	115
3	F-2	34	0	125	130

Case	Factor of Safety		
	Condition 1	Condition 2	Condition 3
Downstream slope sudden drawdown	1.30	1.49	1.09



Note: Condition 1 Cohesion for Zone 1 = 800 psf, Cohesion for F-1 = 100 psf
Condition 2 Cohesion for Zone 1 = 400 psf, Cohesion for F-1 = 400 psf
Condition 3 Cohesion for Zone 1 = 400 psf, Cohesion for F-1 = 100 psf
All other parameters are the same for all conditions.

Scale: 1" = 10'

SCALE
DESIGNED CHECKED
DRAWN DATE
APPROVED LICENSE NO.

ROLLINS, BROWN & GUNNELL, Inc.
CONSULTING ENGINEERS

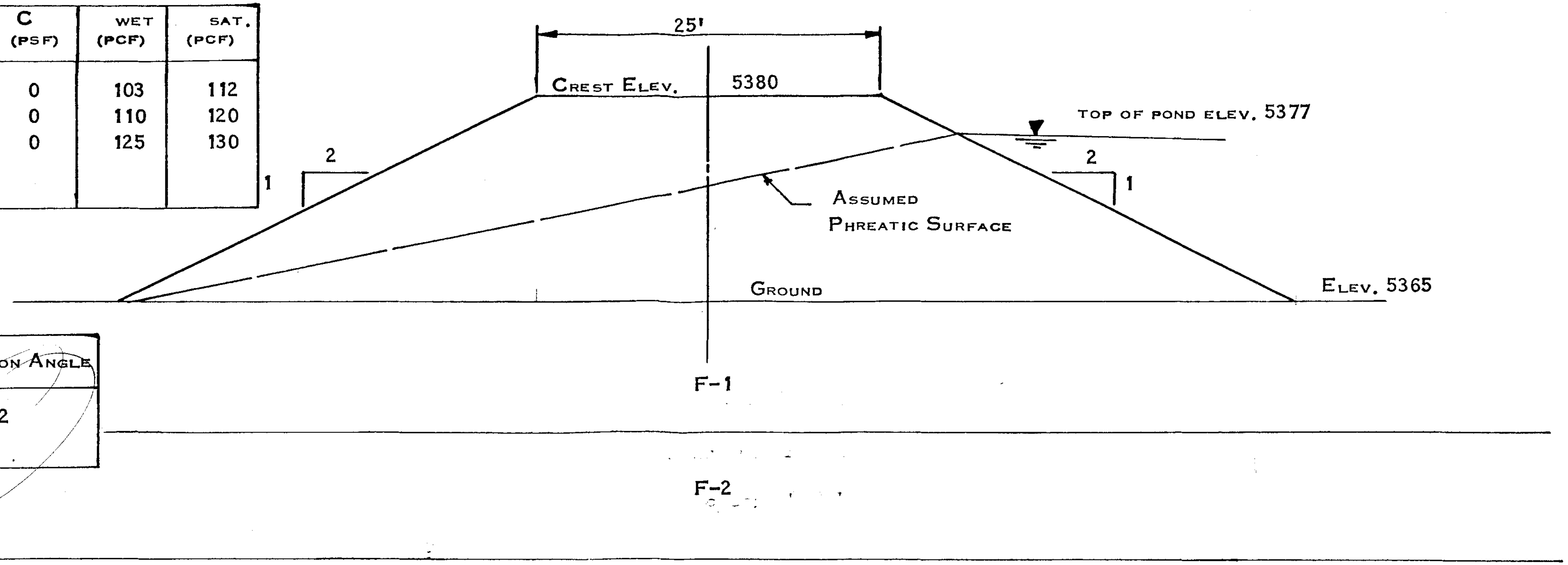
PHYSICAL CHARACTERISTICS AND MATERIAL PROPERTIES USED IN STABILITY ANALYSIS FOR LOWER REFUSE DIKE
U.S. STEEL CORPORATION, WELLINGTON, UTAH

FIGURE NO. 13

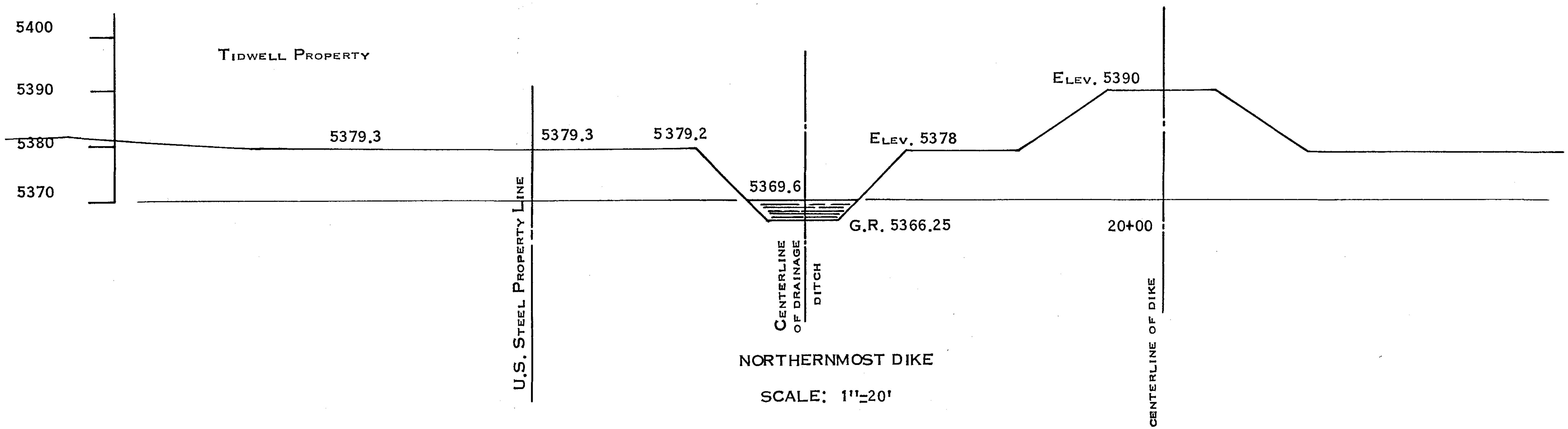
Bruning "500"

SOIL TYPE	ZONE	φ DEGREES	C (PSF)	WET (PCF)	SAT. (PCF)
1	1	28.5	0	103	112
2	F-1	35.0	0	110	120
3	F-2	34.0	0	125	130

Case	Friction Angle
Steady State Seepage	1.22



UPPER REFUSE DIKE
Maximum Section
Scale: 1" = 20'



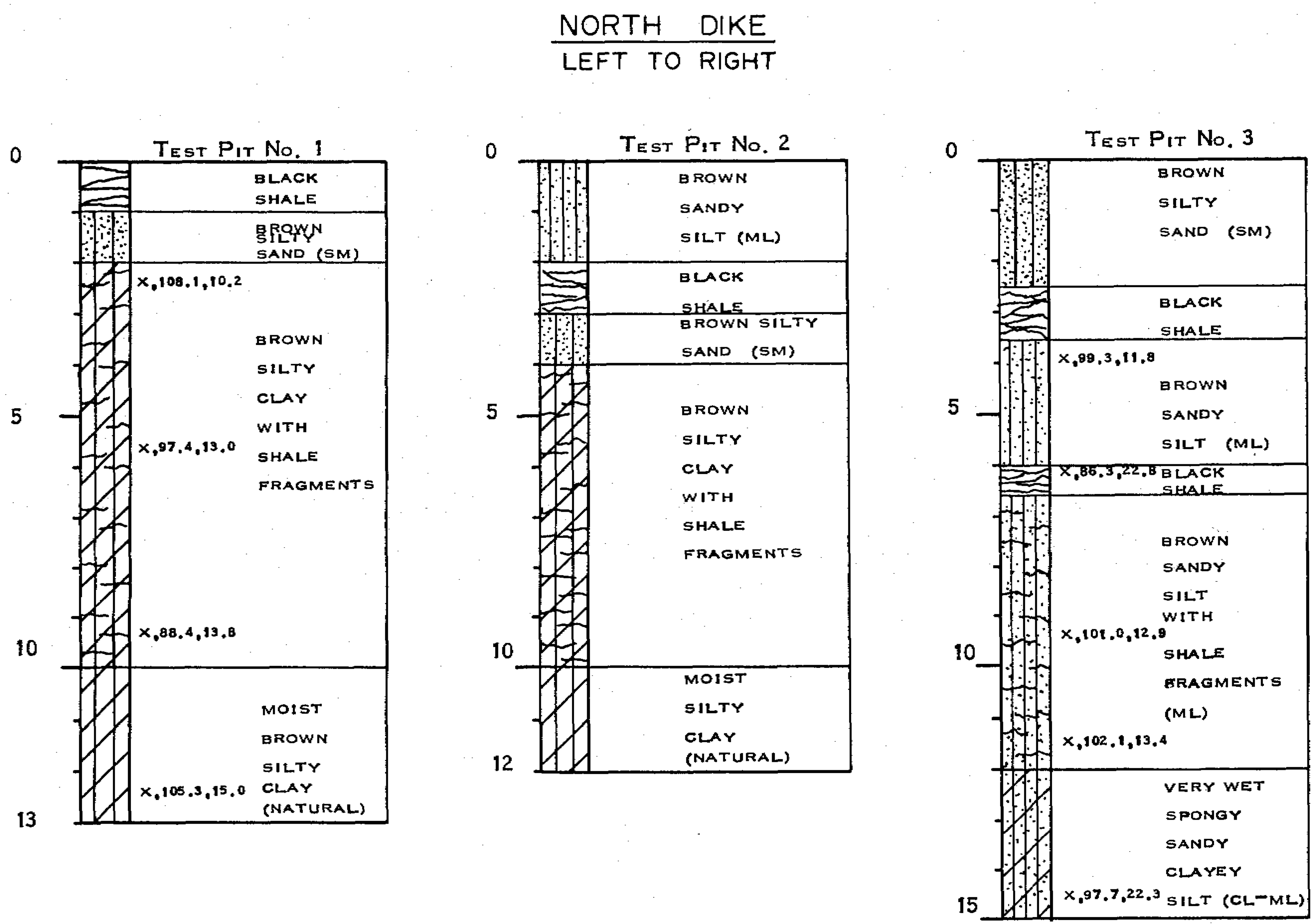
NORTHERNMOST DIKE

SCALE: 1"=20'

SCALE		ROLLINS, BROWN & GUNNELL, Inc. CONSULTING ENGINEERS	PHYSICAL CHARACTERISTICS AND MATERIAL PROPERTIES USED IN STABILITY ANALYSIS FOR NORTHERN-MOST DIKE U.S. STEEL CORPORATION, WELLINGTON, UTAH
DESIGNED	CHECKED		
DRAWN	DATE		
APPROVED	LICENSE NO.		

FIGURE NO. 14

Bruning "500"



LOG OF TEST PITS FOR THE NORTHERN DIKE
U.S. STEEL CORPORATION, WELLINGTON, UTAH

FIGURE NO. 15

TABLE 1.1 SUMMARY OF TEST DATA

PROJECT U.S. Steel FEATURE Tailings Dams LOCATION Wellington, Utah

HOLE NO.	DEPTH BELOW GROUND SURFACE	STANDARD PENETRA. BLOWS PER FT.	IN-PLACE			UNCONFINED COMPRESSIVE STRENGTH LB/FT³	FRICTION ANGLE Φ	CONSISTENCY LIMITS			MECHANICAL ANALYSIS			SOIL CLASSIFICATION UNIFIED SYSTEM
			UNIT WEIGHT LB/FT³	MOISTURE PERCENT	VOID RATIO			L.L. %	P.L. %	P.I. %	% GRAVEL	% SAND	% SILT & CLAY	
3	25-26.5							21.4	19.8	6.4				ML
4	6-7.5										29.9	42.7	27.4	GM, SM
	10-15										37.7	40.4	21.9	GM
	20-21.5										57.2	27.4	15.4	GW
	25-26.5							25.5	18.6	6.9				CL-ML
	40-41										56.6	34.8	8.6	GW
5	0-11.5										52.3	30.5	17.2	GM
	5-6.5										45.3	39.3	15.4	GM
	15-16										35.5	43.3	21.2	GM
	35-36.5							19.4	17.2	2.2				ML
	40-41										40.6	49.7	9.7	GW, SW
	45-46.5										47.6	34.8	17.5	GM

TABLE I.2 SUMMARY OF TEST DATA

PROJECT U.S. Steel FEATURE Tailings Dams LOCATION Wellington, Utah

HOLE NO.	DEPTH BELOW GROUND SURFACE	STANDARD PENETRA. BLOWS PER FT.	IN-PLACE			UNCONFINED COMPRESSIVE STRENGTH LB/FT^3	FRICTION ANGLE ϕ	CONSISTENCY LIMITS			MECHANICAL ANALYSIS			SOIL CLASSIFICATION UNIFIED SYSTEM
			UNIT WEIGHT LB/FT^3	MOISTURE PERCENT	VOID RATIO			L.L. %	P.L. %	P.I. %	% GRAVEL	% SAND	% SILT & CLAY	
5	22-23.5	3-10-10						20.3	18.3	2.0				ML
9	10-11.5							23.1	15.3	7.8				CL-1
	15-16.5							23.0	16.5	6.5				CL-ML
	25-26.5							25.1	18.9	6.2				CL-ML
	30-31.5							33.9	19.0	14.9				CL-1
	35-36.5										27.4	29.2	43.4	SC
	40-41.5										54.8	33.4	11.8	GM-GC
16	5-6.5							20.2	18.2	2.0				CL-ML
	15-16.5							23.3	18.9	4.4				CL-ML
	25-26.5							33.4	19.5	13.9				CL-1
17	5-6.5							28.7	18.9	9.8				CL-1
	10-11.5							28.8	17.3	11.5				CL-1

TABLE 1.3 SUMMARY OF TEST DATA

PROJECT U.S. Steel FEATURE Tailings Dams LOCATION Wellington, Utah

HOLE NO.	DEPTH BELOW GROUND SURFACE	STANDARD PENETRA. BLOWS PER FT.	IN-PLACE			UNCONFINED COMPRESSIVE STRENGTH LB/FT³	FRICTION ANGLE φ	CONSISTENCY LIMITS			MECHANICAL ANALYSIS			SOIL CLASSIFICATION UNIFIED SYSTEM
			UNIT WEIGHT LB/FT³	MOISTURE PERCENT	VOID RATIO			L.L. %	P.L. %	P.I. %	% GRAVEL	% SAND	% SILT & CLAY	
17	15-16.5							33.8	20.2	13.6				CL-1
	20-21.5										41.7	25.8	32.5	SM
	25-26.5							22.3	15.5	6.8				CL-1/CL-ML
	30-31.5							23.0	19.0	4.0				CL-ML
	40-41.5										0	74.6	25.4	SC

ROLLINS, BROWN AND GUNNELL, INC.
PROFESSIONAL ENGINEERS

ROLLINS, BROWN AND GUNNELL, INC.
PROFESSIONAL ENGINEERS

April 16,1980

U.S. Steel Corporation
P.O. Box 807
East Carbon, Utah 84520

ATTENTION: Glenn A. Sides

Gentlemen:

We have reviewed a letter dated February 15,1980, sent to your organization from John W. Barton, Mine Safety and Health Administration, along with a memorandum prepared by Kenneth E. Cudworth, Civil Engineer, Mine Waste Branch. The following comments are made regarding these letters.

1. Our investigation of the Upper Refuse Pond embankment was based upon construction drawings supplied to our organization by United States Steel Corporation. At the time of our investigation, the downstream toe of the Upper Refuse Dam was submerged, and it was not possible to determine the exact height of this facility. Based upon our conversation with officials of the United States Steel Corporation and our observation of the site, we used a height of 15 feet for this facility. The side slopes used in our analysis were based upon side slopes obtained from the construction drawings.

2. In our original investigation, we concluded that a catastrophic failure of the entire system was not likely even if the Upper Refuse Dam failed. Our conclusion was based upon the following analysis. Recent contour maps prepared by Aerographics for the United States Steel Corporation indicate that the water surface area upstream from the Upper Refuse Dam is about 80 acres. The total water surface area impounded by both the Clearwater Dam and the Lower Refuse Dam is approximately 90 acres. Based upon our discussions with officials of the United States Steel Corporation, along with our own observations throughout the storage area, the depth of free water above the slurry material varies from 0 to approximately 2 feet. The minimum freeboard for both the Clearwater and the Lower Refuse dikes is approximately 2 feet. It is apparent, therefore, that since the water surface area for the Clearwater and the Lower Refuse dikes exceeds the water surface area for the Upper Refuse Dike, the lower dikes are capable of storing at least 2 feet of water from their Upper Refuse dike storage basin in the event that this facility failed. In view of the above reasoning, it is still our opinion that a catastrophic failure of the entire system is unlikely.

3. Mr. Cudworth in his memorandum of August 29,1978, indicated that he has computed a factor of safety of less than 1.0 and that in his opinion the Upper Refuse dike was unstable. He further states that when he visited the facility on September 1, 1976, he found no ponded water surface at the downstream toe. Since the dike had not failed under these conditions, it is apparent that the factor of safety of the existing structure was greater than 1.0 and that the shear strength parameters that Mr. Cudworth used to obtain a factor of safety less than 1.0 were well on the con-

servative side.

4. In a memorandum to J.W. Barton, dated October 16,1978, Mr. Cudworth indicated that by using "corrected" parameters, the DTCS arrived at a factor of safety of less than 1.0. The stability analysis performed during our original investigation were based upon shear strength parameters obtained from triaxial tests performed on both the foundation clays and the refuse material in the dike. Mr. Cudworth indicated that he had used "corrected" shear strength parameters. We have no way of knowing what shear strength parameters Mr. Cudworth used. However, since the dam existed in a stable condition with no water on the downstream side of the dam and with water within two feet of the crest on the upstream side of the dam with no failure occuring we question whether the shear strength parameters used by Mr. Cudworth were better than the valves we used.

5. In compliance with the request from the Mine Safety and Health Administration by Mr. John Barton in his letter dated February 15, 1980, we have completed a supplemental stability analysis of the Upper Refuse dike. In performing this analysis, we have modified the geometric parameters to agree with those determined by Mr. Cudworth during his visit to the site when the downstream slope was not inundated. The downstream slope has been changed to an angle of 34 degrees, rather than a slope of 2 horizontal to 1 vertical used in the previous analysis. Mr. Cudworth measured the embankment height at 12 feet rather than 15 feet and, therefore, we have used both embankment heights in performing the new analysis, to assimilate the existing condition and a condition if the embankment were raised 3 feet.

The stability analysis has been performed using Spencer's method, which provides a rigorous solution satisfying both force and moment equilibrium. The search routine utilized in the computer program provides three modes of search used in the iterative process to locate the critical failure surface with a minimum safety factor. It is our understanding from a previous discussion with Mr. Cudworth at the time the original stability analysis was performed that the DTSC was just at that time instigating a computer program utilizing Spencer's method.

It should be recongnized that the results obtained from the stability analysis must be analyzed relative to the location of the critical failure surfaces. It has been our esperience that for embankments consisting of granular materials where no cohesion is used in the shear strength parameters and where the side slopes are steeper than 2 horizontal to 1 vertical, the computer program iterates until a minimum factor of safety is obtained for a failure surface approaching an infinite slope, located within inches of the slope profile. Both observation and experience indicates that a failure surface, only inches deep, is not a true indication of the actual factor of asfety against slope stability failure. **Due to this fact, a significant effort has been expended to eliminate very shallow insignificant failure surfaces and to obtain the minimum factor of safety for failure surfaces well within** the embankment.

In plotting the failure surfaces for the existing facility, it is apparent how Mr. Cudworth could obtain a safety factor less than 1.0. By using a low friction angle and zero cohesion for the embankment refuse material, a failure surface only inches deep in the downstream slope does produce a safety factor less than 1.0.

By using a low friction angle and zero cohesion for the embankment refuse material, a failure surface only inches deep in the downstream slope does produce a safety factor less than 1.0. It is obvious however that this slope failure has not occurred to date for the existing facility, and it is our experience that such failure surfaces are meaningless in defining the actual minimum safety factor against slope failure.

In our original analysis no cohesion value was used. It is our experience from numerous other investigations of tailings dikes constructed of coal refuse material that the unsaturated refuse material in its insitu condition does exhibit some cohesion. It can be observed form Figure No. 6 of our original stability analysis that a significant amount of cohesion is depicted by the Mogr envelope obtained from the triaxial shear tests. Therefore, the shear strength parameters used in this analysis were varied in order to evaluate the affect of cohesion on the minimum factor of safety against slope failure. The results of our slope stability analysis are tabulated below.

UPPER REFUSE DIKE
RESULTS OF SLOPE STABILITY ANALYSIS
(STEADY STATE SEEPAGE CONDITIONS)

Embankment Height (ft)	Material Type	Angle Ø (Degrees)	Cohesion C (psf)	Wet (pcf)	Sat (pcf)	Minimum Factor of Safety	Search Limitations (Surface depth)
12	Embankment refuse	28.5	0	103	112	1.22	3' depth
12		30.0	0			1.32	3' depth
12		28.5	100			1.48	Complete search
15		28.5	100			1.38	Complete search
12' with Berm		28.5	0			1.38	>3' depth
15' with 2.5:1 downstream slope		28.5	0			1.33	Complete search

It is our opinion that a friction angle of 28.5 degrees, with a cohesion of zero, for the refuse material is very conservative based on the results of the triaxial shear tests. Using an effective stress analysis and with the major portion of the failure surface existing within the unsaturated portion of the refuse material, it is our opinion that a more realistic and yet, satisfactorily conservative estimate of the shear strength parameters is a friction angle of 28.5 and a cohesion of 100 pounds per square foot. Our judgement that these parameters are still significantly conservative is based on the results of a number of triaxial shear tests which we have performed in the past on similar coal refuse material from other tailings dikes. In general, friction angles of 34 and 35 degrees have been obtained with a small amount of cohesion.

It can be observed from the results tabulated above that the existing structure, 12 feet high as measured by Mr. Cudworth, exhibits a factor of safety of 1.48 when 100 psf of cohesion is used. It should also be noted that if U. S. Steel Corporation were to raise this facility by 3 feet to an embankment height of 15 feet, the minimum safety factor obtained is 1.38.

Mr. John W. Barton indicated in his letter that for approval to be granted for the upper refuse embankment the minimum safety factor must be 1.30. If 100 psf of cohesion is accepted it is apparent that the stability of the existing facility under steady state seepage conditions exceeds the minimum requirement and that the embankment height could be increased by 3 feet and still exceed the minimum required factor of safety.

It will be noted from the above table that the factor of safety has been determined for the existing facility with a 4' high berm 12' wide, using a friction angle of 28.5 degrees and no cohesion. This crossection meets the requirement specified by Mr. Barton as necessary for approval. If the embankment is raised 3' and if the shear strength parameters are limited to a friction angle of 28.5 degrees and no cohesion, the above table also indicates that a downstream slope of 2.5 horizontal to 1 vertical will be required to obtain a factor of safety greater than 1.30.

2.5

While it is our opinion that the downstream slope of the existing structure under steady state seepage case has a factor of safety of 1.30 without considering cohesion, we recommend that a berm 4' high and 12' wide be constructed at the downstream toe of the Upper Refuse Dike to insure compliance with the Mine Safety and Health Administration.

Please advise us if we can be of further assistance to you.

Yours truly,

ROLLINS, BROWN & GUNNELL, INC.

Ralph L. Rollins

Ralph L. Rollins
President

APPENDIX - A

STORM RUNOFF CALCULATIONS

Inserted Appendix B of Plans 6/15/87 Jec

CALCULATION NOTES

Subject Storm Calculations
Wellington Coal Cleaning Plant
100 Year 24 Hour Storm

Summary

By V. R. Watts
Checked
Acc't
Date May 27 1983
Sheet No. 1 of 4 Sheets

Calculation 1

Ponds Completed as Proposed

These calculations are based on the following:

1. The Lower Refuse Dike modifications are completed as proposed including the installation of the final pond discharge structure.
2. The proposed discharge structure for the Upper Refuse Pond is installed.
3. 100 year 24 hour storm.
4. No changes to the Clear Water Pond.
5. The proposed permanent diversion is constructed as proposed.
6. The discharges from upper ponds are added to any storm inflows.

Results

Pond	Time of Occurrence Hrs.	Maximum Accumulation Ac. Ft.	Maximum Increase in Pond Depth ft.	Plant Discharge cfs	Peak Pond Discharge cfs
Upper Refuse Pond	24.0	11.31	0.22	7.58	10.01
Lower Refuse Pond	24.0	38.28	0.64	*	18.15
Clear Water Pond	24.0	14.28	1.16	*	19.39

*Plant Discharge is included inflows from upper ponds.

CALCULATION NOTES

Subject Storm Calculations
Wellington Coal Cleaning Plant
100 Year 24 Hour Storm

Summary

By V. R. Watts
Checked
Acc't
Date May 27 19 83
Sheet No. 2 of 4 Sheets

Calculation 2

Ponds During Construction

These calculations are based on the following:

1. The Lower Refuse Dike is under construction. The present overflow structure is still in service. The top of the pipe has been cut off to increase the flow capacity. This will result in an open end 18" vertical pipe as the overflow structure.

2. The proposed discharge structure for the Upper Refuse Pond is installed.

3. 100 year 24 hour storm.

4. No changes to the Clear Water Pond.

5. The proposed permanent diversion is constructed as proposed.

6. The discharges from upper ponds are added to any storm inflows.

Results

Pond	Time of Occurrence Hrs.	Maximum Accumulation Ac. Ft.	Maximum Increase in Pond Depth ft.	Plant Discharge cfs	Peak Pond Discharge cfs
Upper Refuse Pond[1]	24.0	11.31	0.22	7.58	10.01
Lower Refuse Pond[2]	24.0	44.61	0.75	*	16.38
Clear Water Pond				*	

*Plant Discharge is included inflows from upper ponds.

1 Same as Calculation 1
2 The Clear Water Pond was not calculated since the peak inflow is less than the peak outflow with 3 feet of freeboard.

CALCULATION NOTES

Subject Storm Calculations
Wellington Coal Cleaning Plant
100 Year 24 Hour Storm

Summary

By V. R. Watts
Checked
Acc't
Date May 27 19 83
Sheet No. 3 of 4 Sheets

Calculation 3

Ponds During Construction

These calculations are based on the following:

1. The proposed discharge structure is installed in the Lower Refuse Dike. The construction is still in progress. An 18" horizontal pipe (part of the proposed discharge structure) is being used as a water flow by-pass until construction is complete.

2. The proposed discharge structure for the Upper Refuse Pond is installed.

3. 100 year 24 hour storm.

4. No changes to the Clear Water Pond.

5. The proposed permanent diversion is constructed as proposed.

6. The discharges from upper ponds are added to any storm inflows.

Results

Pond	Time of Occurrence Hrs.	Maximum Accumulation Ac. Ft.	Maximum Increase in Pond Depth ft.	Plant Discharge cfs	Peak Pond Discharge cfs
Upper Refuse Pond 1	24.0	11.31	0.22	7.58	10.01
Lower Refuse Pond 2	24.0	36.36	0.61	*	20.49
Clear Water Pond				*	

*Plant Discharge is included inflows from upper ponds.

1 Same as calculation 1
2 The Clear Water Pond was not calculated since the peak inflow is less than the peak discharge with 3 feet of freeboard.

Subject Storm Calculations

Wellington Coal Cleaning Plant

100 Year 24 Hour Storm

Summary

CALCULATION NOTES

By V. R. Watts

Checked

Acc't

Date May 27 19 83

Conclusion

In conclusion the storm calculations for the design storm (100 year 24 hour), show that the proposed modifications to the refuse ponds can contain and pass the storm runoff with an acceptable rise in the pond water level. The design storm can also be contained and passed during construction.

CALCULATION NOTES

By V R Watts

Checked

Acc't

Date May 27 1983

Sheet No. 1 of 2 Sheets

Subject Storm Calculations

100 year 24 hour Storm

Wellington Coal Cleaning Plant

Ponds Completed as Proposed

General Data - Calculation 1

Number of units of storm data	75
Number of hours per unit of storm data	2.0
Design storm magnitude in inches	2.74
Storm duration in hours	24.0
Cleaning Plant Discharge	7.58

Upper Refuse Pond Data

Pond area in acres	52.37
Number of overflow weirs	2
Weir length in feet	1.44
Elevation difference between weir and pond	-0.94

Lower Refuse Pond Data

Pond area in acres	59.87
Number of overflow weirs	3
Weir length in feet	1.44
Elevation difference between weir and pond	-0.70

Clear Water Pond Data

Pond area in acres	12.29
Number of overflow pipes	1
Inside pipe diameter	3.0
Roughness factor	0.012
Slope as a ratio	0.01
Elevation difference between pipe and pond	0.1

CALCULATION NOTES

Subject Storm Calculations
100 year 24 hour Storm
Wellington Coal Cleaning Plant
Ponds Completed as Proposed

By V R Watts
Checked
Acc't
Date May 23 1983
Sheet No. 2 of 2 Sheets

Summary of Results

Design Storm 100 year 24 hour

Storm Magnitude 2.74 inches

Pond	Time of Occurrence Hrs.	Maximum Accumulation Ac. Ft.	Maximum Increase in Pond Depth ft.	Plant Discharge cfs	Peak Pond Discharge cfs
Upper Refuse Pond	24.0	11.31	0.22	7.58	10.01
Lower Refuse Pond	24.0	38.28	0.64	*	18.15
Clear Water Pond	24.0	14.28	1.16	*	19.39

*Plant Discharge is included inflows from upper ponds.

```
LOO
 THIS PROGRAM MODELS INFLOWN AND OUTFLOWS FROM THE
 WELLINGTON REFUSE PONDS
  ENTER THE NUMBER OF UNITS OF STORM DATA =
? 75
  ENTER THE NUMBER OF HOURS FOR EACH UNIT OF STORM DATA =
? 2.0
   ENTER THE POND AREA IN ACRES  =
? 52.37
   ENTER THE DESIGN STORM MAGNITUDE IN INCHES =
? 2.74
   ENTER THE STORM DURATION IN HOURS =
[illegible].0
   ENTER THE DISCHARGE FROM THE COAL CLEANING PLANT
   IN CFS  =
? 7.58
   ENTER THE TYPE OF STORM DATA DESIRED
   (1) ACCUMULATED INCHES OF RUN OFF
   (2)  RUN OFF IN CFS
? 2
   ENTER THE STORM DATA =
? 0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0
? 0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0
? 0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0
  (1) CALCULATES DISCHARGE THRU A SUPPRESSED WEIR
   (2) CALCULATES DISCHARGE THRU OPEN END VERTICAL PIPES
   (3) CALCULATES DISCHARGE THRU HORIZONTAL PIPES
   (4) CALCULATES DISCHARGE THRU AN OPEN SPILLWAY
   (5) COMBINE (1) AND (3)
   (6) COMBINE(1) AND (4)
   (7) COMBINE (2) AND (3)
   (8) COMBINE (2) AND (4)


 ENTER THE TYPECALCULATIONS DESIRED
? 1
   ENTER THE NUMBER OF OVERFLOW WEIRS =
? 2
   ENTER THE WEIR LENGTH =
? 1.44
 ENTER THE ELEVATION DIFFERENCE BETWEEN THE WEIR
   AND THE POND IN FEET  =
? -0.94
```

TIME HOURS	STORM DATA CFS OR IN	ACCUM RUN OFF AC FT	PRECIP IN POND AC FT	TOTAL ACCUM IN POND AC FT	INCREASE IN POND DEPTH FT	PLANT DISCHARGE CFS	POND DISCHARGE CFS	NET ACCUM DISCHARGE AC FT
2.0	0.00	1.25	1.00	2.25	.04	7.58	8.07	.67
4.0	0.00	2.51	1.99	3.35	.07	7.58	8.40	2.03
6.0	0.00	3.76	2.99	4.72	.09	7.58	8.5[illegible]	3.43
8.0	0.00	5.01	3.99	5.56	.11	7.58	8.72	4.87
10.0	0.00	6.26	4.98	6.38	.12	7.58	8.95	6.33
12.0	0.00	7.52	5.98	7.16	.14	7.58	9.11	7.82
14.0	0.00	8.77	6.98	7.92	.15	7.58	9.[illegible]	9.34
16.0	0.00	10.02	7.97	8.65	.17	7.58	9.43	10.89
18.0	0.00	11.28	8.97	9.35	.18	7.58	9.58	12.46
20.0	0.00	12.53	9.96	10.03	.19	7.58	9.75	14.06
22.0	0.00	13.78	10.96	10.68	.20	7.58	9.87	15.68
24.0	0.00	15.03	11.96	11.31	.22	7.58	10.[illegible]	17.32
26.0	0.00	16.29	11.96	10.92	.21	7.58	9.92	18.97
28.0	0.00	17.54	11.96	10.53	.20	7.58	9.84	20.60
30.0	0.00	18.79	11.96	10.15	.19	7.58	9.76	22.22
32.0	0.00	20.05	11.96	9.78	.19	7.58	9.68	23.83
34.0	0.00	21.30	11.96	9.43	.18	7.58	9.60	25.42
36.0	0.00	22.55	11.96	9.09	.17	7.58	9.53	27.00
38.0	0.00	23.80	11.96	8.76	.17	7.58	9.46	28.57
40.0	0.00	25.06	11.96	8.45	.16	7.58	9.39	30.13
42.0	0.00	26.31	11.96	8.14	.16	7.58	9.32	31.67
44.0	0.00	27.56	11.96	7.85	.15	7.58	9.26	33.21
46.0	0.00	28.82	11.96	7.57	.14	7.58	9.20	34.73
48.0	0.00	30.07	11.96	7.29	.14	7.58	9.14	36.25
50.0	0.00	31.32	11.96	7.03	.13	7.58	9.08	37.76
52.0	0.00	32.58	11.96	6.78	.13	7.58	9.03	39.25
54.0	0.00	33.83	11.96	6.53	.12	7.58	8.98	40.74
56.0	0.00	35.08	11.96	6.30	.12	7.58	8.93	42.22
58.0	0.00	36.33	11.96	6.07	.12	7.58	8.88	43.69
60.0	0.00	37.59	11.96	5.85	.11	7.58	8.83	45.16
62.0	0.00	38.84	11.96	5.64	.11	7.58	8.79	46.61
64.0	0.00	40.09	11.96	5.44	.10	7.58	8.74	48.06
66.0	0.00	41.35	11.96	5.24	.10	7.58	8.70	49.50
68.0	0.00	42.60	11.96	5.05	.10	7.58	8.66	50.94
70.0	0.00	43.85	11.96	4.87	.09	7.58	8.62	52.37
72.0	0.00	45.10	11.96	4.69	.09	7.58	8.59	53.79
74.0	0.00	46.36	11.96	4.52	.09	7.58	8.55	55.21
[illegible].0	0.00	47.61	11.96	4.36	.08	7.58	8.52	56.62
[illegible].0	0.00	48.86	11.96	4.20	.08	7.58	8.48	58.02
80.0	0.00	50.12	11.96	4.05	.08	7.58	8.45	59.42
82.0	0.00	51.37	11.96	3.90	.07	7.58	8.42	60.82
84.0	0.00	52.62	11.96	3.76	.07	7.58	8.3[illegible]	62.21
86.0	0.00	53.87	11.96	[illegible]	.07	7.58	8.3[illegible]	63.59
88.0	0.00	55.13	11.96	3.50	.07	7.58	8.3[illegible]	64.97
90.0	0.00	56.38	11.96	3.37	.06	7.58	8.[illegible]	66.34
92.0	0.00	57.63	11.96	3.25	.06	7.58	8.[illegible]	67.7[illegible]
94.0	0.00	58.89	11.96	3.13	.06	7.58	8.[illegible]	69.08
96.0	0.00	60.14	11.96	3.01	.06	7.58	8.25	70.44
98.0	0.00	61.39	11.96	2.91	.06	7.58	8.2[illegible]	71.80

102.0	0.00	63.90	11.96	2.70	.05	7.58	8.16	74.51
104.0	0.00	65.15	11.96	2.60	.05	7.58	8.14	75.86
106.0	0.00	66.40	11.96	2.50	.05	7.58	8.12	77.20
108.0	0.00	67.66	11.96	2.41	.05	7.58	8.10	78.54
110.0	0.00	68.91	11.96	2.32	.04	7.58	8.09	79.88
112.0	0.00	70.16	11.96	2.24	.04	7.58	8.07	81.22
114.0	0.00	71.41	11.96	2.16	.04	7.58	8.05	82.55
116.0	0.00	72.67	11.96	2.08	.04	7.58	8.03	83.88
118.0	0.00	73.92	11.96	2.00	.04	7.58	8.02	85.20
120.0	0.00	75.17	11.96	1.93	.04	7.58	8.00	86.53
[illegible]	0.00	76.43	11.96	1.86	.04	7.58	7.99	87.85
[illegible]	0.00	77.68	11.96	1.79	.03	7.58	7.97	89.17
126.0	0.00	78.93	11.96	1.72	.03	7.58	7.96	90.49
128.0	0.00	80.19	11.96	1.66	.03	7.58	7.95	91.80
130.0	0.00	81.44	11.96	1.60	.03	7.58	7.93	93.11
132.0	0.00	82.69	11.96	1.54	.03	7.58	7.92	94.42
134.0	0.00	83.94	11.96	1.48	.03	7.58	7.91	95.73
136.0	0.00	85.20	11.96	1.42	.03	7.58	7.90	97.04
138.0	0.00	86.45	11.96	1.37	.03	7.58	7.89	98.34
140.0	0.00	87.70	11.96	1.32	.03	7.58	7.87	99.64
142.0	0.00	88.96	11.96	1.27	.02	7.58	7.86	100.94
144.0	0.00	90.21	11.96	1.22	.02	7.58	7.85	102.24
146.0	0.00	91.46	11.96	1.18	.02	7.58	7.84	103.54
148.0	0.00	92.71	11.96	1.13	.02	7.58	7.84	104.84
150.0	0.00	93.97	11.96	1.09	.02	7.58	7.83	106.13

```
THE MAGNITUDE OF THE DESIGN STORM =  2.74 IN
 THE DESIGN STORM DURATION = 24.0  HOURS
THE DISCHARGE FROM THE PLANT =       7.58  CFS
THE POND AREA =   52.4 ACRES
  DO YOU WISH TO RUN THE PROGRAM AGAIN WITH THE SAME
  STORM DATA
  ENTER 1 FOR YES AND 2 FOR NO
? 2
  DO YOU WISH TO RUN THE NEXT LOWER POND USING THE
  THE DISCHARGES JUST CALULATED AS A FLOW TO THE
  LOWER POND
  ENTER 1 FOR YES AND 2 FOR NO
?

  1
  ENTER THE NUMBER OF NEW STORM DATA POINTS
? 31
  ENTER THE NEW STORM DATA
? 1.61,5.07,10.46,17.66,22.55,24.20,23.77,21.17,18.02,13.48
? 10.12,7.75,6.01,4.68,3.56,2.78,2.16,1.66,1.25,.98,.76,.58
? .47,.38,.29,.23,.17,.13,.08,.05,.01
  ENTER THE NEW POND AREA IN ACRES
? 59.87
 (1) CALCULATES DISCHARGE THRU A SUPPRESSED WEIR
  (2) CALCULATES DISCHARGE THRU OPEN END VERTICAL PIPES
  (3) CALCULATES DISCHARGE THRU HORIZONTAL PIPES
  (4) CALCULATES DISCHARGE THRU AN OPEN SPILLWAY
  (5) COMBINE (1) AND (3)
  (6) COMBINE(1) AND (4)
  (7) COMBINE (2) AND (3)
  (8) COMBINE (2) AND (4)

 ENTER THE TYPECALCULATIONS DESIRED
? 1
  ENTER THE NUMBER OF OVERFLOW WEIRS =
? 3
  ENTER THE WEIR LENGTH =
? 1.44
 ENTER THE ELEVATION DIFFERENCE BETWEEN THE WEIR
  AND THE POND IN FEET  =
? -0.70
```

TIME HRS	STORM DATA CFS OR IN	ACCUM RUN OFF AC FT	PRECIP IN POND AC FT	TOTAL ACCUM IN POND AC FT	INCREASE IN POND DEPTH FT.	PLANT DISCHARGE CFS	POND DISCHARGE CFS	NET ACCUM DISCHARGE AC FT
2.0	9.68	1.60	1.14	2.74	.05	7.58	8.30	.69
4.0	13.47	3.83	2.28	5.42	.09	7.58	9.00	2.12
6.0	19.05	6.98	3.42	8.28	.14	7.58	9.76	3.67
8.0	26.43	11.35	4.56	12.24	.20	7.58	10.62	5.37
10.0	31.50	16.55	5.70	16.88	.28	7.58	12.09	7.26
12.0	33.31	22.06	6.84	21.63	.36	7.58	13.41	9.37
14.0	33.05	27.52	7.97	26.13	.44	7.58	14.58	11.69
16.0	30.60	32.58	9.11	30.00	.50	7.58	15.70	14.21
18.0	27.60	37.14	10.25	33.19	.55	7.58	16.67	16.89
20.0	23.21	40.98	11.39	35.48	.59	7.58	17.55	19.70

26.0	15.93	[illegible]				[illegible]	[illegible]	[illegible]
28.0	14.52	[illegible]				[illegible]	[illegible]	[illegible]
30.0	13.32	[illegible]		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
32.0	12.46	56.51		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
34.0	11.76	58.45	[illegible]	[illegible]	.56	7.58	[illegible]	[illegible]
36.0	11.19	60.30	[illegible]	[illegible]	[illegible]	7.58	[illegible]	43.04
38.0	10.71	62.07	13.67	[illegible]	.55	7.58	[illegible]	45.80
40.0	10.37	63.78	13.67	[illegible]	.53	7.58	16.25	48.51
42.0	10.08	65.45	13.67	[illegible]	.51	7.58	15.95	51.17
44.0	9.85	67.08	13.67	[illegible]	.49	7.58	[illegible]	[illegible]
46.0	9.67	68.68	13.67	[illegible]	.48	7.58	15.37	56.35
48.0	9.52	70.25	13.67	[illegible]	.46	7.58	[illegible]	58.87
50.0	9.37	71.80	13.67	[illegible]	.44	7.58	14.81	61.34
52.0	9.26	73.33	13.67	25.67	.43	7.58	[illegible]	[illegible]
54.0	9.15	74.84	13.67	[illegible]	.41	7.58	14.29	66.15
56.0	9.06	76.34	13.67	21.67	.40	7.58	13.94	68.49
58.0	8.96	77.82	13.67	[illegible]	.38	7.58	[illegible]	[illegible]
60.0	8.88	79.29	13.67	[illegible]	.37	7.58	13.56	73.05
62.0	8.80	80.74	13.67	21.37	.36	7.58	[illegible]	[illegible]
64.0	8.74	82.19	13.67	[illegible]	.34	7.58	13.12	77.46
66.0	8.70	83.63	13.67	17.84	.33	7.58	12.91	79.61
68.0	8.66	85.06	13.67	19.12	.32	7.58	[illegible]	[illegible]
70.0	8.62	86.49	13.67	18.43	.31	7.58	12.52	83.81
72.0	8.59	87.91	13.67	17.76	.30	7.58	[illegible]	[illegible]
74.0	8.55	89.32	13.67	[illegible]	[illegible]	7.58	12.16	87.89
76.0	8.52	90.73	13.67	16.51	.28	7.58	11.99	89.86
78.0	8.48	92.13	13.67	15.92	.27	7.58	11.82	91.85
80.0	8.45	93.53	13.67	15.34	.26	7.58	11.67	93.79
82.0	8.42	94.92	13.67	14.79	.25	7.58	11.52	95.71
84.0	8.39	96.30	13.67	14.27	.24	7.58	[illegible]	97.60
86.0	8.36	97.69	13.67	13.76	.23	7.58	11.23	99.47
88.0	8.33	99.06	13.67	13.27	.22	7.58	11.10	101.31
90.0	8.31	100.44	13.67	12.79	.21	7.58	[illegible]	103.14
92.0	8.28	101.80	13.67	12.34	.21	7.58	10.85	104.94
94.0	8.26	103.17	13.67	11.90	.20	7.58	[illegible]	106.72
96.0	8.23	104.53	13.67	11.48	.19	7.58	10.61	108.49
98.0	8.21	105.89	13.67	11.07	.18	7.58	[illegible]	110.25
100.0	8.19	107.24	13.67	10.68	.18	7.58	[illegible]	111.98
102.0	8.16	108.59	13.67	10.30	.17	7.58	[illegible]	[illegible]
104.0	8.14	109.94	13.67	9.94	.17	7.58	10.20	[illegible]
106.0	8.12	111.26	13.67	9.59	.16	7.58	10.10	117.04
108.0	8.10	112.67	13.67	9.25	.15	7.58	10.01	118.76
110.0	8.09	113.95	13.67	8.92	.15	7.58	[illegible]	[illegible]
112.0	8.07	115.29	13.67	8.61	.14	7.58	9.84	121.99
114.0	8.05	116.62	13.67	8.30	.14	7.58	9.76	[illegible]
116.0	8.03	117.95	13.67	8.01	.13	7.58	9.68	125.21
118.0	8.02	119.27	13.67	7.73	.13	7.58	9.61	126.81
120.0	8.00	120.59	13.67	7.47	.12	7.58	9.54	128.39
122.0	7.99	121.91	13.67	7.20	.12	7.58	9.47	129.95
124.0	7.97	123.23	13.67	6.94	.12	7.58	9.40	131.52
126.0	7.96	124.55	13.67	6.70	.11	7.58	9.34	133.07
128.0	7.95	125.86	13.67	6.46	.11	7.58	9.28	134.61
130.0	7.93	127.17	13.67	6.24	.10	7.58	9.22	136.13
132.0	7.92	128.46	13.67	6.02	.10	7.58	9.16	137.65
134.0	7.91	129.79	13.67	5.81	.10	7.58	9.10	139.16
136.0	7.90	131.09	13.67	5.60	.09	7.58	9.05	140.66
138.0	7.89	132.40	13.67	5.41	.09	7.58	9.00	142.15
140.0	7.87	133.70	13.67	[illegible]	.09	7.58	[illegible]	[illegible]
142.0	7.86	135.00	[illegible]	[illegible]	.08	7.58	[illegible]	[illegible]
144.0	7.85	136.30	13.67	[illegible]	.08	7.58	[illegible]	146.58
146.0	7.84	137.59	[illegible]	[illegible]	.08	7.58	8.81	148.04
148.0	7.84	138.89	13.67	[illegible]	.08	7.58	[illegible]	149.49
150.0	7.83	140.18	13.67	4.36	.07	7.58	[illegible]	[illegible]

THE MAGNITUDE OF THE DESIGN STORM = [illegible] IN
THE DESIGN STORM DURATION = 24.0 HOURS
THE DISCHARGE FROM THE PLANT = 7.58 CFS
THE POND AREA = 59.9 ACRES
DO YOU WISH TO RUN THE PROGRAM AGAIN WITH THE SAME
STORM DATA
ENTER 1 FOR YES AND 2 FOR NO
? 2
DO YOU WISH TO RUN THE NEXT LOWER POND USING THE
THE DISCHARGES JUST CALCULATED AS A FLOW TO THE
LOWER POND
ENTER 1 FOR YES AND 2 FOR NO
?

```
 1
 ENTER THE NUMBER OF NEW STORM DATA POINTS
? 5
 ENTER THE NEW STORM DATA
? 0,0,0,0,0
 ENTER THE NEW POND AREA IN ACRES
? 12.29
 (1) CALCULATES DISCHARGE THRU A SUPPRESSED WEIR
 (2) CALCULATES DISCHARGE THRU OPEN END VERTICAL PIPES
 (3) CALCULATES DISCHARGE THRU HORIZONTAL PIPES
 (4) CALCULATES DISCHARGE THRU AN OPEN SPILLWAY
 (5) COMBINE (1) AND (3)
 (6) COMBINE(1) AND (4)
 (7) COMBINE (2) AND (3)
 (8) COMBINE (2) AND (4)


 ENTER THE TYPECALCULATIONS DESIRED
? 3
 ENTER THE INSIDE PIPE DIAMETER IN FEET=
? 3.0
 ENTER THE ROUGHNESS FACTOR
? 0.012
 ENTER THE PIPE SLOPE AS A RATIO =
? 0.01
 ENTER THE NUMBER OF OVERFLOW PIPES =
? 1
 ENTER THE ELEVATION DIFFERENCE BETWEEN THE POND
 AND THE BOTTOM OF THE OVERFLOW PIPES =
? 0.1
```

TIME HOURS	STORM DATA CFS OR IN	ACCUM RUN OFF AC FT	PRECIP IN POND AC FT	TOTAL ACCUM IN POND AC FT	INCREASE IN POND DEPTH FT	PLANT DISCHARGE CFS	POND DISCHARGE CFS	NET ACCUM DISCHARGE AC FT
2.0	8.30	1.37	.23	1.61	.13	7.58	.01	.00
4.0	9.00	2.86	.47	3.33	.27	7.58	.46	.04
6.0	9.76	4.47	.70	5.13	.42	7.58	1.70	.22
8.0	10.82	6.26	.94	6.98	.57	7.58	3.80	.67
10.0	12.09	8.26	1.17	8.76	.71	7.58	6.59	1.53
12.0	13.41	10.48	1.40	10.35	.84	7.58	9.63	2.88
14.0	14.68	12.90	1.64	11.66	.95	7.58	12.63	4.72
16.0	15.78	15.51	1.87	12.66	1.03	7.58	15.07	7.01
18.0	16.69	18.27	2.10	13.36	1.09	7.58	16.90	9.55
20.0	17.35	21.13	2.34	13.82	1.12	7.58	18.14	12.55
22.0	17.81	24.08	2.57	14.10	1.15	7.58	18.91	15.61
24.0	18.15	27.00	2.81	14.28	1.16	7.58	19.39	18.78
26.0	18.05	30.06	2.81	14.09	1.15	7.58	18.89	21.94
28.0	17.88	33.02	2.81	13.89	1.13	7.58	18.31	25.01
30.0	17.66	35.94	2.81	13.73	1.12	7.58	17.89	28.01
32.0	17.41	38.82	2.81	13.62	1.11	7.58	17.56	30.94
34.0	17.13	41.65	2.81	13.52	1.10	7.58	17.31	33.82
36.0	16.85	44.43	2.81	13.42	1.09	7.58	17.05	36.66
38.0	16.55	47.17	2.81	13.31	1.08	7.58	16.77	39.46
40.0	16.25	49.85	2.81	13.20	1.07	7.58	16.48	42.21
42.0	15.95	52.49	2.81	13.09	1.07	7.58	16.19	44.91
44.0	15.66	55.08	2.81	12.98	1.06	7.58	15.90	47.56
46.0	15.37	57.62	2.81	12.87	1.05	7.58	15.61	50.16
48.0	15.09	60.11	2.81	12.76	1.04	7.58	15.33	52.72
50.0	14.81	62.56	2.81	12.65	1.03	7.58	15.05	55.23
52.0	14.55	64.96	2.81	12.54	1.02	7.58	14.78	57.69
54.0	14.29	67.33	2.81	12.44	1.01	7.58	14.52	60.12
56.0	14.04	69.65	2.81	12.34	1.00	7.58	14.27	62.49
58.0	13.80	71.93	2.81	12.24	1.00	7.58	14.02	64.83
60.0	13.56	74.17	2.81	12.14	.99	7.58	13.79	67.13
62.0	13.34	76.37	2.81	12.05	.98	7.58	13.56	69.39
64.0	13.12	78.54	2.81	11.96	.97	7.58	13.33	71.61
66.0	12.91	80.68	2.81	11.87	.97	7.58	13.12	73.80
68.0	12.71	82.78	2.81	11.78	.96	7.58	12.92	75.95
70.0	12.52	84.85	2.81	11.70	.95	7.58	12.72	78.07
72.0	12.33	86.88	2.81	11.62	.95	7.58	12.53	80.16
74.0	12.16	88.89	2.81	11.54	.94	7.58	12.35	82.21
76.0	11.99	90.87	2.81	11.47	.93	7.58	12.17	84.24
78.0	11.82	92.83	2.81	11.40	.93	7.58	12.01	86.24
80.0	11.67	94.76	2.81	11.33	.92	7.58	11.84	88.21
82.0	11.52	96.66	2.81	11.26	.92	7.58	11.69	90.15
84.0	11.37	98.54	2.81	11.19	.91	7.58	11.54	92.07
86.0	11.23	100.40	2.81	11.13	.91	7.58	11.40	93.97
88.0	11.10	102.23	2.81	11.07	.90	7.58	11.26	95.84
90.0	10.97	104.04	2.81	11.01	.90	7.58	11.13	97.69
92.0	10.85	105.84	2.81	10.95	.89	7.58	11.00	99.52
94.0	10.73	107.61	2.81	10.90	.89	7.58	10.87	101.33
96.0	10.61	109.36	2.81	10.84	.88	7.58	10.76	103.11
98.0	10.50	111.10	2.81	10.79	.88	7.58	10.64	104.88
100.0	10.40	112.82	2.81	10.74	.87	7.58	10.53	106.63
102.0	10.30	114.52	2.81	10.69	.87	7.58	10.43	108.37
104.0	10.20	116.21	2.81	10.65	.87	7.58	10.33	110.08
106.0	10.10	117.88	2.81	10.60	.86	7.58	10.23	111.78
108.0	10.01	119.53	2.81	10.56	.86	7.58	10.13	113.46
110.0	9.93	121.17	2.81	10.52	.86	7.58	10.04	115.13
112.0	9.84	122.80	2.81	10.47	.85	7.58	9.96	116.78
114.0	9.76	124.41	2.81	10.44	.85	7.58	9.87	118.42
116.0	9.68	126.01	2.81	10.40	.85	7.58	9.79	120.05
118.0	9.61	127.60	2.81	10.36	.84	7.58	9.71	121.66
120.0	9.54	129.18	2.81	10.32	.84	7.58	9.64	123.26
122.0	9.47	130.74	2.81	10.29	.84	7.58	9.57	124.85

126.0	9.34	133.84	2.81	10.22	.33	7.58	9.43	127.99
128.0	9.28	135.37	2.81	10.19	.33	7.58	9.37	129.54
130.0	9.22	136.90	2.81	10.16	.33	7.58	9.30	131.08
132.0	9.16	138.41	2.81	10.13	.32	7.58	9.24	132.61
134.0	9.10	139.91	2.81	10.11	.32	7.58	9.18	134.14
136.0	9.05	141.41	2.81	10.08	.32	7.58	9.13	135.65
138.0	9.00	142.90	2.81	10.05	.32	7.58	9.07	137.16
140.0	8.95	144.38	2.81	10.03	.32	7.58	9.02	138.65
142.0	8.90	145.85	2.81	10.00	.31	7.58	8.97	140.14
144.0	8.85	147.31	2.81	9.98	.31	7.58	8.92	141.62
146.0	8.81	148.77	2.81	9.96	.31	7.58	8.88	143.09
148.0	8.77	150.22	2.81	9.93	.31	7.58	8.83	144.55
150.0	8.72	151.66	2.81	9.91	.31	7.58	8.79	146.01

```
THE MAGNITUDE OF THE DESIGN STORM =  2.74 IN
 THE DESIGN STORM DURATION = 24.0  HOURS
THE DISCHARGE FROM THE PLANT =      7.58  CFS
THE POND AREA =   12.3 ACRES
  DO YOU WISH TO RUN THE PROGRAM AGAIN WITH THE SAME
  STORM DATA
  ENTER 1 FOR YES AND 2 FOR NO
? 2
  DO YOU WISH TO RUN THE NEXT LOWER POND USING THE
  THE DISCHARGES JUST CALULATED AS A FLOW TO THE
  LOWER POND
  ENTER 1 FOR YES AND 2 FOR NO
? 2
..
```

CALCULATION NOTES

By	VRWatts
Checked	
Acc't	
Date	May 27 1983
Sheet No.	1 of 2 Sheets

Subject Storm Calculations

100 year 24 hour Storm

Wellington Coal Cleaning Plant

Lower Refuse Pond During Construction

Horizontal Pipe Over Flow

General Data - Calculation 2

Number of units of storm data	75
Number of hours per unit of storm data	2.0
Design storm magnitude in inches	2.74
Storm duration in hours	24.0
Cleaning Plant Discharge	7.58

Upper Refuse Pond Data

Pond area in acres	52.37
Number of overflow weirs	2
Weir length in feet	1.44
Elevation difference between weir and pond	-0.94

Lower Refuse Pond Data

Roughness factor	0.012
Pond area in acres	59.87
Number of overflow pipes	1
Pipe diameter	1.44
Elevation difference between pipes and pond	-0.35
Slope as a ratio	0.03

Clear Water Pond Data

Pond area in acres	12.29
Number of overflow pipes	1
Inside pipe diameter	3.0
Roughness factor	0.012
Slope as a ratio	0.01
Elevation difference between pipe and pond	0.1

Subject *Storm Calculations*
100 year 24 hour Storm
Wellington Coal Cleaning Plant
Lower Refuse Pond During Construction
Horizontal Pipe Over Flow

CALCULATION NOTES

By *V R Watts*

Checked

Acc't

Date *May 27* 19*83*

Sheet No. *2* of *2* Sheets

Summary of Results

Design Storm *100 year 24 hour Storm*

Storm Magnitude *2.74*

Pond	Time of Occurrence Hrs.	Maximum Accumulation Ac. Ft.	Maximum Increase in Pond Depth ft.	Plant Discharge cfs	Peak Pond Discharge cfs
Upper Refuse Pond [1]	24.0	11.31	0.22	7.58	10.01
Lower Refuse Pond	24.0	44.61	0.75	*	16.38
Clear Water Pond [2]				*	

*Plant Discharge is included inflows from upper ponds.

[1] *Same as calculation 1*

[2] *The Clearwater Pond was not calculated since the peak inflow is less than the peak discharge with 3 ft of free board*

(1) CALCULATES DISCHARGE THRU A SUPPRESSED WEIR
(2) CALCULATES DISCHARGE THRU OPEN END VERTICAL PIPES
(3) CALCULATES DISCHARGE THRU HORIZONTAL PIPES
(4) CALCULATES DISCHARGE THRU AN OPEN SPILLWAY
(5) COMBINE (1) AND (3)
(6) COMBINE(1) AND (4)
(7) COMBINE (2) AND (3)
(8) COMBINE (2) AND (4)

```
 ENTER THE TYPECALCULATIONS DESIRED
? 3
   ENTER THE INSIDE PIPE DIAMETER IN FEET=
? 1.44
   ENTER THE ROUGHNESS FACTOR
? 0.012
   ENTER THE PIPE SLOPE AS A RATIO  =
? 0.03
   ENTER THE NUMBER OF OVERFLOW PIPES =
? 1
   ENTER THE ELEVATION DIFFERENCE BETWEEN THE POND
   AND THE BOTTOM OF THE OVERFLOW PIPES =
? -0.35
```

TIME HOURS	STORM DATA CFS OR IN	ACCUM RUN OFF AC FT	PRECIP IN POND AC FT	TOTAL ACCUM IN POND AC FT	INCREASE IN POND DEPTH FT	PLANT DISCHARGE CFS	POND DISCHARGE CFS	NET ACCUM DISCHARGE AC FT
2.0	9.68	1.60	1.14	2.74	.05	7.58	2.92	.24
4.0	13.47	3.83	2.28	5.86	.10	7.58	3.71	.79
6.0	19.05	6.98	3.42	9.60	.16	7.58	4.76	1.49
8.0	26.43	11.35	4.56	14.41	.24	7.58	6.24	2.40
10.0	31.50	16.55	5.70	19.85	.33	7.58	8.04	3.58
12.0	33.31	22.06	6.84	25.31	.42	7.58	9.95	5.07
14.0	33.05	27.52	7.97	30.43	.51	7.58	11.76	6.86
16.0	[illegible].60	32.58	9.11	34.83	.58	7.58	13.29	8.93
18.0	27.60	37.14	10.25	38.46	.64	7.58	14.50	11.22
20.0	23.21	40.98	11.39	41.14	.69	7.58	15.35	13.69

26.0	15.93	49.85	13.67	44.57	.74	7.58	16.37	21.66
28.0	14.52	52.25	13.67	44.26	.74	7.58	16.28	24.35
30.0	13.32	54.45	13.67	43.77	.73	7.58	16.14	27.03
32.0	12.46	56.51	13.67	43.15	.72	7.58	15.96	29.69
34.0	11.76	58.45	13.67	42.44	.71	7.58	15.75	32.31
36.0	11.19	60.30	13.67	41.67	.70	7.58	15.51	34.89
38.0	10.71	62.07	13.67	40.85	.68	7.58	15.26	37.43
40.0	10.37	63.78	13.67	40.02	.67	7.58	15.00	39.93
42.0	10.08	65.45	13.67	39.19	.65	7.58	14.73	42.39
44.0	9.85	67.08	13.67	38.36	.64	7.58	14.47	44.80
46.0	9.67	68.68	13.67	37.54	.63	7.58	14.20	47.17
48.0	9.52	70.25	13.67	36.75	.61	7.58	13.93	49.50
50.0	9.37	71.80	13.67	35.97	.60	7.58	13.67	51.78
52.0	9.26	73.33	13.67	35.22	.59	7.58	13.42	54.02
54.0	9.15	74.84	13.67	34.50	.58	7.58	13.17	56.22
56.0	9.06	76.34	13.67	33.80	.56	7.58	12.93	58.37
58.0	8.96	77.82	13.67	33.12	.55	7.58	12.70	60.49
60.0	8.88	79.29	13.67	32.47	.54	7.58	12.47	62.57
62.0	8.80	80.74	13.67	31.84	.53	7.58	12.25	64.61
64.0	8.74	82.19	13.67	31.25	.52	7.58	12.04	66.62
66.0	8.70	83.63	13.67	30.68	.51	7.58	11.84	68.60
68.0	8.66	85.06	13.67	30.13	.50	7.58	11.65	70.54
70.0	8.62	86.49	13.67	29.62	.49	7.58	11.47	72.45
72.0	8.59	87.91	13.67	29.13	.49	7.58	11.30	74.33
74.0	8.55	89.32	13.67	28.66	.48	7.58	11.13	76.18
76.0	8.52	90.73	13.67	28.21	.47	7.58	10.97	78.01
78.0	8.48	92.13	13.67	27.79	.46	7.58	10.82	79.81
80.0	8.45	93.53	13.67	27.38	.46	7.58	10.68	81.59
82.0	8.42	94.92	13.67	27.00	.45	7.58	10.54	83.34
84.0	8.39	96.30	13.67	26.63	.44	7.58	10.41	85.08
86.0	8.36	97.69	13.67	26.28	.44	7.58	10.29	86.79
88.0	8.33	99.06	13.67	25.95	.43	7.58	10.17	88.48
90.0	8.31	100.44	13.67	25.63	.43	7.58	10.06	90.15
92.0	8.28	101.80	13.67	25.33	.42	7.58	9.95	91.80
94.0	8.26	103.17	13.67	25.04	.42	7.58	9.85	93.44
96.0	8.23	104.53	13.67	24.76	.41	7.58	9.75	95.06
98.0	8.21	105.89	13.67	24.50	.41	7.58	9.66	96.66
100.0	8.19	107.24	13.67	24.24	.40	7.58	9.57	98.25
102.0	8.16	108.59	13.67	24.00	.40	7.58	9.49	99.83
104.0	8.14	109.94	13.67	23.78	.40	7.58	9.41	101.39
106.0	8.12	111.28	13.67	23.56	.39	7.58	9.33	102.94
108.0	8.10	112.62	13.67	23.35	.39	7.58	9.26	104.48
110.0	8.09	113.95	13.67	23.15	.39	7.58	9.19	106.00
112.0	8.07	115.29	13.67	22.96	.38	7.58	9.12	107.51
114.0	8.05	116.62	13.67	22.78	.38	7.58	9.06	109.02
116.0	8.03	117.95	13.67	22.60	.38	7.58	9.00	110.51
118.0	8.02	119.27	13.67	22.43	.37	7.58	8.94	111.99
120.0	8.00	120.59	13.67	22.27	.37	7.58	8.88	113.46
122.0	7.99	121.91	13.67	22.12	.37	7.58	8.83	114.93
124.0	7.97	123.23	13.67	21.98	.37	7.58	8.78	116.38

128.0	7.95	125.86	13.67	21.70	.36	7.58	8.68	119.27
130.0	7.93	127.17	13.67	21.58	.36	7.58	8.64	120.70
132.0	7.92	128.48	13.67	21.45	.36	7.58	8.60	122.12
134.0	7.91	129.79	13.67	21.34	.36	7.58	8.55	123.54
136.0	7.90	131.09	13.67	21.23	.35	7.58	8.52	124.95
138.0	7.89	132.40	13.67	21.12	.35	7.58	8.48	126.35
140.0	7.87	133.70	13.67	21.01	.35	7.58	8.44	127.75
142.0	7.86	135.00	13.67	20.92	.35	7.58	8.41	129.15
144.0	7.85	136.30	13.67	20.82	.35	7.58	8.38	130.53
146.0	7.84	137.59	13.67	20.73	.35	7.58	8.35	131.92
148.0	7.84	138.89	13.67	20.64	.34	7.58	8.32	133.29
150.0	7.83	140.18	13.67	20.56	.34	7.58	8.29	134.66

```
 THE MAGNITUDE OF THE DESIGN STORM =  2.74 IN
  THE DESIGN STORM DURATION = 24.0  HOURS
 THE DISCHARGE FROM THE PLANT =      7.58  CFS
 THE POND AREA =   59.9 ACRES
   DO YOU WISH TO RUN THE PROGRAM AGAIN WITH THE SAME
   STORM DATA
   ENTER 1 FOR YES AND 2 FOR NO
? 2
   DO YOU WISH TO RUN THE NEXT LOWER POND USING THE
   THE DISCHARGES JUST CALULATED AS A FLOW TO THE
   LOWER POND
   ENTER 1 FOR YES AND 2 FOR NO
? 2
..
```

CALCULATION NOTES

By V R Watts

Checked

Acc't

Date May 27 1983

Sheet No. 1 of 2 Sheets

Subject Storm Calculations

100 year 24 hour Storm

Wellington Coal Cleaning Plant

Lower Refuse Pond during Construction

Vertical Pipe from Lower Refuse Pond

General Data - Calculation 3

Number of units of storm data	75
Number of hours per unit of storm data	2.0
Design storm magnitude in inches	2.74
Storm duration in hours	24.0
Cleaning Plant Discharge	7.58

Upper Refuse Pond Data

Pond area in acres	52.37
Number of overflow weirs	2
Weir length in feet	1.44
Elevation difference between weir and pond	-0.94

Lower Refuse Pond Data

Pond area in acres	59.87
Number of overflow pipe(s)	1
Pipe diameter	1.44
Elevation difference between pipes and pond	-0.62

Clear Water Pond Data

Pond area in acres	12.29
Number of overflow pipes	1
Inside pipe diameter	3.0
Roughness factor	0.012
Slope as a ratio	0.01
Elevation difference between pipe and pond	0.1

CALCULATION NOTES

Subject Storm Calculations
100 year 24 hour Storm
Wellington Coal Cleaning Plant
Lower Refuse Pond during Construction
Vertical Pipe from Lower Refuse Pond

By V R Watts
Checked
Acc't
Date May 27 1983
Sheet No. 2 of 2 Sheets

Summary of Results

Design Storm 100 year 24 hour Storm

Storm Magnitude 2.74 inches

Pond	Time of Occurrence Hrs.	Maximum Accumulation Ac. Ft.	Maximum Increase in Pond Depth ft.	Plant Discharge cfs	Peak Pond Discharge cfs
Upper Refuse Pond [1]	24.0	11.31	0.22	7.58	10.01
Lower Refuse Pond	24.0	36.36	0.61	*	20.49
Clear Water Pond [2]				*	

*Plant Discharge is included inflows from upper ponds.

1 Same as Calculation 1

2 The Clearwater Pond was not calculated since the peak inflow is less than the peak discharge with 3 ft. of freeboard.

```
  ENTER THE NUMBER OF NEW STORM DATA POINTS
? 31
  ENTER THE NEW STORM DATA
? 1.61,5.07,10.46,17.66,22.55,24.20.23.77,21.17,18.02.13.48.10.12.
? 7.75$6.01,4.68,3.56,2.78,2.16,1.66,1.25..98..76,.59,.47,.38..29
? .23,.17,.13,.08,.05,.01
  ENTER THE NEW POND AREA IN ACRES
? 59.87
 (1) CALCULATES DISCHARGE THRU A SUPPRESSED WEIR
  (2) CALCULATES DISCHARGE THRU OPEN END VERTICAL PIPES
  (3) CALCULATES DISCHARGE THRU HORIZONTAL PIPES
  (4) CALCULATES DISCHARGE THRU AN OPEN SPILLWAY
  (5) COMBINE (1) AND (3)
  (6) COMBINE(1) AND (4)
  (7) COMBINE (2) AND (3)
  (8) COMBINE (2) AND (4)


 ENTER THE TYPECALCULATIONS DESIRED
? 2
  ENTER OUT FLOW PIPE DIAMETER =
? 1.44
  ENTER THE NUMBER OF VERTICAL PIPES
? 1
  ENTER THE ELEVATION DIFFERENCE BETWEEN THE POND
  AND THE DISCHARGE PIPES  =
? -0.62
```

TIME HOURS	STORM DATA CFS OR IN	ACCUM RUN OFF AC FT	PRECIP IN POND AC FT	TOTAL ACCUM IN POND AC FT	INCREASE IN POND DEPTH FT	PLANT DISCHARGE CFS	POND DISCHARGE CFS	NET ACCUM DISCHARGE AC FT
2.0	9.68	1.60	1.14	2.74	.05	7.58	8.18	.68
4.0	13.47	3.83	2.28	5.43	.09	7.58	9.03	2.10
6.0	19.05	6.98	3.42	8.30	.14	7.58	9.95	3.67
8.0	26.43	11.35	4.56	12.23	.20	7.58	11.28	5.42
10.0	31.50	16.55	5.70	16.83	.28	7.58	12.88	7.42
12.0	33.31	22.06	6.84	21.47	.36	7.58	14.59	9.69
14.0	[illegible].05	27.52	7.97	25.81	.43	7.58	16.23	12.24
16.0	30.60	32.58	9.11	29.46	.49	7.58	17.67	15.04
18.0	27.60	37.14	10.25	32.36	.54	7.58	18.83	18.05

24.0	17.76	47.22	13.67	36.36	.61	7.58	20.49	27.88
26.0	15.93	49.85	13.67	35.64	.60	7.58	20.18	31.24
28.0	14.52	52.25	13.67	34.68	.58	7.58	19.78	34.54
30.0	13.32	54.45	13.67	33.58	.56	7.58	19.33	37.78
32.0	12.46	56.51	13.67	32.40	.54	7.58	18.85	40.93
34.0	11.76	58.45	13.67	31.19	.52	7.58	18.36	44.01
36.0	11.19	60.30	13.67	29.96	.50	7.58	17.87	47.00
38.0	10.71	62.07	13.67	28.74	.48	7.58	17.38	49.91
40.0	10.37	63.78	13.67	27.54	.46	7.58	16.91	52.75
42.0	10.08	65.45	13.67	26.37	.44	7.58	16.45	55.51
44.0	9.85	67.08	13.67	25.24	.42	7.58	16.02	58.19
46.0	9.67	68.68	13.67	24.16	.40	7.58	15.60	60.80
48.0	9.52	70.25	13.67	23.12	.39	7.58	15.20	63.35
50.0	9.37	71.80	13.67	22.12	.37	7.58	14.83	65.83
52.0	9.26	73.33	13.67	21.17	.35	7.58	14.47	68.25
54.0	9.15	74.84	13.67	20.26	.34	7.58	14.14	70.62
56.0	9.06	76.34	13.67	19.40	.32	7.58	13.82	72.93
58.0	8.96	77.82	13.67	18.57	.31	7.58	13.51	75.18
60.0	8.88	79.29	13.67	17.78	.30	7.58	13.23	77.39
62.0	8.80	80.74	13.67	17.02	.28	7.58	12.95	79.56
64.0	8.74	82.19	13.67	16.30	.27	7.58	12.70	81.68
66.0	8.70	83.63	13.67	15.62	.26	7.58	12.46	83.76
68.0	8.66	85.06	13.67	14.97	.25	7.58	12.23	85.80
70.0	8.62	86.49	13.67	14.36	.24	7.58	12.01	87.80
72.0	8.59	87.91	13.67	13.78	.23	7.58	11.81	89.77
74.0	8.55	89.32	13.67	13.22	.22	7.58	11.61	91.70
76.0	8.52	90.73	13.67	12.69	.21	7.58	11.43	93.61
78.0	8.48	92.13	13.67	12.19	.20	7.58	11.26	95.49
80.0	8.45	93.53	13.67	11.71	.20	7.58	11.10	97.33
82.0	8.42	94.92	13.67	11.25	.19	7.58	10.94	99.15
84.0	8.39	96.30	13.67	10.82	.18	7.58	10.79	100.95
86.0	8.36	97.69	13.67	10.41	.17	7.58	10.65	102.72
88.0	8.33	99.06	13.67	10.01	.17	7.58	10.52	104.47
90.0	8.31	100.44	13.67	9.63	.16	7.58	10.40	106.20
92.0	8.28	101.80	13.67	9.27	.15	7.58	10.28	107.91
94.0	8.26	103.17	13.67	8.93	.15	7.58	10.16	109.60
96.0	8.23	104.53	13.67	8.60	.14	7.58	10.05	111.27
98.0	8.21	105.89	13.67	8.29	.14	7.58	9.95	112.92
100.0	8.19	107.24	13.67	7.99	.13	7.58	9.85	114.56
102.0	8.16	108.59	13.67	7.70	.13	7.58	9.76	116.18
104.0	8.14	109.94	13.67	7.42	.12	7.58	9.67	117.79
106.0	8.12	111.28	13.67	7.16	.12	7.58	9.58	119.38
108.0	8.10	112.62	13.67	6.91	.12	7.58	9.50	120.95
110.0	8.09	113.95	13.67	6.67	.11	7.58	9.42	122.52
112.0	8.07	115.29	13.67	6.44	.11	7.58	9.35	124.07
114.0	8.05	116.62	13.67	6.22	.10	7.58	9.28	125.61
116.0	[illegible].03	117.95	13.67	6.01	.10	7.58	9.21	127.14
118.0	[illegible].02	119.27	13.67	5.80	.10	7.58	9.15	128.65
120.0	8.00	120.59	13.67	5.61	.09	7.58	9.08	130.16
122.0	7.99	121.91	13.67	5.42	.09	7.58	9.02	131.66

124.0	7.97	123.23	13.67	[illegible]	[illegible]	7.58	[illegible]	[illegible]
126.0	7.96	124.55	13.67	5.07	.08	7.58	8.91	134.62
128.0	7.95	125.86	13.67	4.91	.08	7.58	8.86	136.09
130.0	7.93	127.17	13.67	4.75	.08	7.58	8.81	137.55
132.0	7.92	128.48	13.67	4.60	.08	7.58	8.76	139.00
134.0	7.91	129.79	13.67	4.46	.07	7.58	8.72	140.45
136.0	7.90	131.09	13.67	4.32	.07	7.58	8.67	141.89
138.0	7.89	132.40	13.67	4.18	.07	7.58	8.63	143.32
140.0	7.87	133.70	13.67	4.05	.07	7.58	8.59	144.74
142.0	7.86	135.00	13.67	3.93	.07	7.58	8.55	146.16
144.0	7.85	136.30	13.67	3.81	.06	7.58	8.52	147.57
146.0	7.84	137.59	13.67	3.70	.06	7.58	8.48	148.97
148.0	7.84	138.89	13.67	3.59	.06	7.58	8.45	150.37
150.0	7.83	140.18	13.67	3.48	.06	7.58	8.41	151.76

```
THE MAGNITUDE OF THE DESIGN STORM =  2.74 IN
 THE DESIGN STORM DURATION = 24.0  HOURS
THE DISCHARGE FROM THE PLANT =      7.58  CFS
THE POND AREA =   59.9 ACRES
  DO YOU WISH TO RUN THE PROGRAM AGAIN WITH THE SAME
  STORM DATA
  ENTER 1 FOR YES AND 2 FOR NO
? 1
```

CALCULATION NOTES

Subject UNIT HYDROGRAPH ANALYSIS
SUMMATION of ORDINATES
R[illegible]ACH 1 + 2 - Upper Refuse Pond
STORM DURATION: 24 HOURS

By M.O.A
Checked
Acc't
4-25-1983
Sheet No. 21 of 22 Sheets

TIME INCREM. HOURS	ORDINATES OF UNIT HYDROGRAPH CFS	10-24	25-24	100-24	TIME INCREMENT HOURS	ORDINATES OF UNIT HYDROGRAPH CFS	10-24	25-24	100-24
Q →	1"				Q =	1"			
0		0	0	0	60		0.152	0.203	0.345
2		1.27	1.78	2.97	62		0.11	0.16	0.26
4		4.17	5.85	9.72	64		0.07	0.10	0.17
6		8.29	11.63	19.32	66		0.05	0.06	0.11
8		14.17	19.89	33.03	68		0.02	0.03	0.05
10		19.30	27.08	44.98	70		0	0	0
12		22.34	31.31	52.01					
14		22.82	31.97	53.12	cfs. Hrs.		421.639	590.355	980.924
16		22.04	30.85	51.26	Ac. Ft.		34.844	48.787	81.063
18		19.59	27.42	45.56					
[illegible]		16.78	23.46	39.00					
22		13.08	18.28	30.39					
24		9.78	13.67	22.74					
26		7.80	10.90	18.10					
28		6.18	8.64	14.36					
30		4.89	6.83	11.36					
32		3.91	5.47	9.09					
34		3.08	4.30	7.16					
36		2.42	3.38	5.62					
38		1.92	2.68	4.46					
40		1.52	2.12	4.31					
42		1.18	1.66	2.76					
44		0.95	1.33	2.21					
46		0.72	1.00	1.66					
48		0.61	0.84	1.39					
50		0.48	0.67	1.11					
52		0.37	0.51	0.85					
[illegible]		0.29	0.40	0.67					
56		0.23	0.31	0.52					
58		0.183	0.26	0.42					

Subject *Unit Hydrograph Analysis*
Summation of Ordinates for Design Storms
Reach A+B 24 hour Storms

Lower Refuse Ponds

CALCULATION NOTES
By *MOA*
Checked
Acc't
4-26 19*83*
Sheet No. of Sheets

Time Increment hours	10 yr 24 hr cfs	25 yr 24 hr cfs	100 yr 24 hr cfs				
0	0	0	0				
2	0.73	1.05	1.61				
4	2.30	3.32	5.07				
6	4.74	6.84	10.46				
8	8.00	11.54	17.66				
10	10.22	14.74	22.55				
12	10.96	15.82	24.20				
14	10.76	15.54	23.77				
16	9.59	13.84	21.17				
18	8.17	11.78	18.02				
20	6.14	8.86	13.48				
22	4.59	6.62	10.12				
24	3.52	5.07	7.75				
26	2.73	3.93	6.01				
28	2.12	3.06	4.68				
30	1.61	2.32	3.56				
32	1.26	1.82	2.78				
34	1.12	1.41	2.16				
36	0.86	1.08	1.66				
38	0.65	0.82	1.25				
40	0.51	0.64	0.98				
42	0.40	0.50	0.76				
44	0.30	0.38	0.59				
46	0.24	0.30	0.47				
48	0.20	0.25	0.38				
50	0.15	0.19	0.29				
52	0.12	0.16	0.23				
54	0.09	0.11	0.17				
56	0.06	0.08	0.13				
58	0.04	0.05	0.08				
60	0.03	0.03	0.05				
62	0.01	0.01	0.01				

APPENDIX - B

EXPLANATION OF COMPUTER CALCULATIONS

CALCULATION NOTES

Subject Computer Model of Storm Runoff Through Ponds-Backup Information

By VRW
Checked
Acc't
Date May 17 1983
Sheet No. 1 of 10 Sheets

Assumptions

1. The program assumes a 24 hour operation of the Coal Cleaning Plant including a 7.58 cfs discharge to the refuse ponds.

2. The 7.58 cfs plant effluent is discharging from the Upper and Lower Refuse Ponds at the beginning of the storm.

3. The precipitation from the design storm is evenly distributed throughout the storm duration. This assumption only applies to precipitation fall directly into the pond.

4. There is no evaporation or seepage out of the refuse ponds.

5. There is no water going to the plant from the Clear Water Pond.

These assumptions are very conservative. The calculations based on these assumptions will develop the "worst case" evaluation of the design storm.

Explanation

The computer program is designed to model the flow of storm run off through the refuse ponds at the Wellington Coal Cleaning Plant. The computations of the program is broken down into steps as follows:

1. Input the necessary data for calculating the inflows and outflows from the first pond to be calculated.

2. Calculate pond inflows, pond discharges, increase in pond depth, etc.

3. The program allows the next lower pond to be calculated using the discharge from the upper pond as an inflow. When this option is used any storm inflows are input. The program adds the storm inflows to the next upper pond's discharges. Step 2 is repeated.

4. Repeat steps 2 and 3 as necessary.

CALCULATION NOTES

Subject Computer Model of Storm Runoff Through Ponds

Back Up Information

By V. R. Watts

Checked MOA

Acc't

Date May 17 19 83

Sheet No. 2 of Sheets

Program Input

Symbol	Definition
	General
A	Pond area in acres.
M	Design storm magnitude in inches.
D	Design storm duration in hours.
P	Plant discharge in cfs.
S	Storm data in cfs
Ti	Number of hours per unit of storm data.
	Discharge through Weir
Nw	Number of overflow weirs.
Lw	Weir length.
Ew	Difference in elevation between the weir and the pond in feet.
	Discharge through a Horizontal Pipe
D_H	Inside pipe diameter.
R	Roughness factor.
S_H	Pipe slope as a ratio.
N_H	Number of overflow pipes
E_H	Difference in elevation between the overflow pipe and the pond in feet
	Discharge through an Open End Vertical Pipe
D_V	Pipe diameter in feet.
N_V	Number of overflow pipes.
E_V	Elevation difference between the pond and the overflow structure.

CALCULATION NOTES

Subject Computer Model of Storm Runoff Through Ponds

Back Up Information

By V. R. Watts
Checked MOA
Acc't
Date May 17 1983
Sheet No. 3 of Sheets

Program Output

Symbol	Location on Print Out	Description
T	Column 1	Time from beginning of storm
S	Column 2	Storm data
F_1	Column 3	Accumulation of inflows to pond
F_2	Column 4	Accumulated precipitation falling in pond
F_T	Column 5	Accumulated water in pond less discharges
H	Column 6	Increase in pond water depth
P	Column 7	Plant discharge
O	Column 8	Pond discharge through overflow structure
O_T	Column 9	Accumulated discharge from pond

Subject Computer Model of Storm Run Off Through Ponds

Back Information

CALCULATION NOTES

By V R Watts

Checked MAA

Acc't

May 17 1983

Calculation of Program Output

All variables calculated for n = 1 to Z
where Z = the number of storm data points

Variable	Calculated Definition
T	$T_n = n \times T_{in}$
S	S_n = input data
F_1	$X_n = \frac{S_n(P_n)(T_{in})(3600)}{43560}$
	for n = 1
	$F_{1_n} = X_n$
	for n = 2 to Z
	$F_{1_n} = X_n + F_{1_{n-1}}$
F_2	For times less than the storm duration
	$F_{2_n} = \frac{M_n H_n A_n}{D_n}$
	For times greater than the storm duration
	$F_{2_n} = F_{2_{n-1}}$

Subject Computer Model of Storm Run Off Through Ponds

Back Up Information

CALCULATION NOTES
By V R Watts
Checked MOA
Acc't
May 17 1983
Sheet No. 5 of 9 Sheets

Calculation of Program Output (cont)

Variable	Calculated Definition
O	Calculation of Discharge through Weirs

*General weir equation (Rectangular weir end contractions

$$q = 3.33(l - 0.2h)h^{1.5} \, \omega$$

where q = flow through weir in cfs
l = weir length in feet
h = height of water over weir in feet
ω = number of weirs

Total Pond Inflows $= Y_n = F_{1_n} + F_{2_n}$

$q_n = O_n$
$h_n = H_n - E_{\omega_n}$
$l_n = L_{\omega_n}$
$\omega_n = N_{\omega_n}$

$$O_n = 3.33\left[L_{\omega_n} - 0.2(H_n - E_{\omega_n})\right](H_n - E_{\omega_n})^{1.5} \cdot N_{\omega_n}$$

O_T — for $n = 1$

$$O_{T_n} = O_n\left(\frac{3600\, T_{i_n}}{43560}\right)(0.5)$$

Calculates average flow in acre feet

* Reference Stevens Water Resources Data Book 3rd Edition

Subject Computer Model of Storm Run Off Through Ponds

Back Up Information

CALCULATION NOTES
By V R Watts
Checked moa
Acc't
May 17 1983

Calculation of Program Output (Cont)

Variable	Calculated Definition
O_T (cont)	for $n = 2$ to Z $O_{T_n} = \left(\frac{O_n + O_{n-1}}{2}\right)\left(\frac{3600\ T_i}{43560}\right) + O_{T_{n-1}}$ Calculates average accumulated discharge in acre feet
H	for $n = 1$ $H_n = (F_{1_n} + F_{2_n}) / A$ for $n = 1$ to Z $H_n = \frac{(F_{1_n} + F_{2_n} - O_{T_{n-1}})}{A}$
P	P = Program input
F_T	for $n = 1$ $F_{T_n} = F_{1_n} + F_{2_n} - O_{T_{n-1}}$

CALCULATION NOTES

Subject: Computer Model of Storm Run Off Through Ponds

Back Up Information

By: V R Watts

Checked: [illegible]

Acc't:

May 17 1983

Sheet No. 7 of 9 Sheets

Variable	Calculated Definition
O	Calculates Discharge through a Horizontal Pipe

General equations from SCS National Engineering Handbook

for pipe < half full

$$Area = \frac{d^2}{8}\left(\frac{\pi\theta}{180} - \sin\theta\right)$$

$$Hydraulic\ Radius = \frac{45d}{\pi\theta}\left(\frac{\pi\theta}{180} - \sin\theta\right)$$

$$\theta = 4\sin^{-1}\sqrt{b/d}$$

for pipe > half full

$$Area = \frac{d^2}{8}\left(2\pi - \frac{\pi\theta}{180} + \sin\theta\right)$$

$$r = Hydraulic\ Radius = \frac{45d}{\pi(360-\theta)}\left(2\pi - \frac{\pi\theta}{180} + \sin\theta\right)$$

$$\theta = 4\cos^{-1}\sqrt{b/d}$$

Manning Equation

$$v = \frac{1.486}{n} r^{2/3} s^{1/2}$$

Subject Computer Model of Storm Run Off Through Ponds

Back Up Information

CALCULATION NOTES
By V R Watts
Checked jmm
Acc't
May 17 1983
Sheet No. 8 of 9 Sheets

Variable	Calculated Definition
	where b = depth of water flowing in pipe in feet
	d = pipe diameter in feet
	r = hydraulic radius
	a = area
	s = pipe slope as a ratio
	v = velocity of water flowing in pipe
	ρ = coefficient of roughness
	$b = H_n - E_{H_n}$
	$d = D_H$ = input data
	$s = S_H$ = input data
	$\rho = R$ = input data
	$O_n = v_n a_n N_{H_n}$

Subject Computer Model of Storm Run Off through Ponds

Back Up Information

CALCULATION NOTES
By V R Watts
Checked MOA
Acc't
May 17 1983
Sheet No. 9 of 9 Sheets

Variable	Definition
O	Calculates the Discharge through an Open End Vertical Pipe For $n = 1$ to Z $W_n = Depth - E_V$ Circumference $= C_n = \frac{2\pi D_H}{2}$ General Equation $Q = 3.33\, L\, H^{1.5}$ where Q = quantity in cfs L = pipe circumference H = height of water flowing over the pipe Therefore: $O_n = 3.33\, C_n\, W_n^{1.5}$

APPENDIX - C

WELLINGTON COAL PREPARATION PLANT
Wellington, Utah

HYDROLOGIC ANALYSIS OF DESIGN STORM RUNOFF INTO THE REFUSE PONDS

RUNOFF AND UNIT HYDROGRAPH COMPUTATIONS FOR A 100 YEAR 6 HOUR DESIGN STORM
AMC-II

Runoff By Reach Area

M. O. Anderson

May 1983

CALCULATION NOTES

Subject Data Sheet
Hydrologic Analysis of Design Storms
Runoff Into Upper and Lower Refuse Ponds
Wellington Coal Preparation Plant

By M.O.A.
Checked
Acc't
Date April 11 19 83
Sheet No. A of 2 Sheets

GRAPHICAL LOCATION:

The Wellington Coal Preparation Plant is located in the center and south quarter of Carbon County; in T.15S., R.11E. of the S.L.B&M. About 7 miles from Price, Utah the local weather reporting station (see Exhibits 5, attached) .

HYDROLOGIC DATA:

The design storms computed are:

10 year 24 hour at 1.82 inches of rainfall.
25 year 24 hour at 2.18 inches of rainfall.
100 year 24 hour at 2.74 inches of rainfall.
100 year 6 hour at 1.91 inches of rainfall.

These values were obtained from Exhibit 5, attached, "Estimated Return Periods for Short-Duration Precipitation in Utah." The values were also substantiated by the "National Weather Service, River Forecast Center", Salt Lake City, Utah and the 100 year 6 hour storm from "Design of Small Dams", General Storm 6 Hour Duration fig. 17, page 50, and Assumption A, (page 73), reduction, fig. 23, page 56 as follows: 6 Hour = 3.80" divide by 2.0 obtained from fig. 23. 3.80 ÷ 2 = 1.90 inches.

The Hydrologic Soil classification and climate, Exhibit 6, were obtained from "Soil Survey - Carbon-Emery Area, Utah), from which Drawing C9-1283, was prepared.

The curve numbers (CN), AMC-II, used on "Storm Runoff Calculations" sheets were obtained by U. S. Steel Mining Co., Inc. Engineering personnel, using data from the "Soil Survey" book, aerial photographs, visual knowledge and the following tables from S.C.S.N.E. Handbook, Hydrology, Section 4, attached.

Table 9.1, Runoff Curve Numbers for Hydrologic Soil-Cover Complexes
Table 9.6, Ground Cover Density in Percent, Juniper-Grass and Sage Grass.

(Note: larger replacement Table 9.6, from Design of Small Dams)

COMPUTATIONS AND FORMULAS:

Form sheets have been prepared to perform the necessary calculations presented herein. They contain necessary formulas to perform computations and are pretty much self explanatory.

CALCULATION NOTES

Subject Data Sheet Con'td.

Hydrologic Analysis

Wellington Coal Cleaning Plant

By M.O.A.

Checked

Acc't

Date April 11 19 83

Sheet No. B of 2 Sheets

STORM RUNOFF CALCULATIONS:

Runoff calculations are computed by both the weighted curve number (CN) and the weighted Quantity (Q) of runoff. The largest value is used and tabulated at the bottom of the sheet under the design storm.

Volume, Acre Feet = Q x Acres ÷ 12 =

Volume, cubic feet = Acre feet x 43560 = (where shown)

UNIT HYDROGRAPH COMPUTATIONS:

The formulas for computing the unit Hydrographs are from the S.C.S., N.E. Handbook, Hydrology, Section 4, and are included in brief in Exhibit 7.

Form sheets are:

1. Time of concentration - Tc, Form
2. Design storm, unit hydrograph calculations, page 1 of 2
3. Design storm, unit hydrograph calculations, page 2 of 2
4. Ordinates for unit hydrograph
5. Summation of ordinates of unit hydrographs. Ordinates of design storms
6. Other summation sheets of storm input for computer data

Other sources of information include:

"Design of Small Dams" Bureau of Reclamation.
"Engineering and Design Manual, Coal Refuse Disposal Facilities" MESA
Other personal text books and reference books. Some of this material will be included in "Misc. Exhibit" attached.

Other backup source material will be included in the "Misc. Exhibit.

Subject Upper Refuse Pond
Data Sheet
Wellington Coal Cleaning Plant

CALCULATION NOTES

By MOA
Checked
Acc't
Date April 12 19 83
Sheet No. D-1 of 2 Sheets

DATA:

Reach 1 and 2 flow into Upper Refuse Pond.

AREAS:

Reach 1 = 604.224 acres
Reach 2 = 46.832 acres

Total 651.056 acres

Point 1 = 11.019 acres, (runoff into pond)

Upper Refuse Pond = 52.365 Acres

The pond is dry except for plant circulation water flowing through pond in a ditch. The pond has not been used for over two years and a drag scraper is being used to excavate refuse fines from the pond.

Plant Circulation Water - 3400 gpm used to pump 1½" x 0 refuse from plant.

3400 x .002228 = 7.5752 cfs

Discharge Pipes through Upper Refuse Dam to Lower Refuse Pond.

4 each 10" dia. steel pipes with Weir arrangement.
Pipe length = 50 ft. Weir Crest L = 0.79 H= 5.0 ft.
Weir consists of the pipe wall being cut out. Pipe sleeves were used to regulate the pond water level when in use.

Present Condition:

Plant Flow = 7.5752 cfs ÷ 4 pipes = 1.8938 cfs each.
Weir crest for 1.894 cfs = 0.97 ft. Depth for plant flow = 0.97 ft ea.

Pipe Capacity = $Q = A\sqrt{\frac{2gH}{1+km+kpL}}$

di = 10.02" I.D.
L = 50 ft.
n = 0.013
km = 1.0

D = 0.8350 ft.
A = 0.5476 sq. ft.
kp = 0.0399
r = .20875

kpL = 0.0399 x 50 = 1.995

$Q = 0.5476\sqrt{\frac{64.4H}{3.995}}$ =

(H-1) = 2.199 cfs x 4 = 8.794 cfs Total
(H-3) = 3.808 cfs x 4 = 15.232 cfs Total
(H-5) = 4.916 cfs x 4 = 19.664 cfs Total
(H-7.5) = 6.021 cfs x 4 = 24.084 cfs Total

CALCULATION NOTES

Subject Upper Refuse Pond
Data Sheet Cont'd
Wellington Coal Cleaning Plant

By MOA
Checked
Acc't
Date April 12 1983
Sheet No. D-2 of 2 Sheets

Future Condition:

Install two (2) 18" dia. steel discharge pipes through west end of dam. Pipes to have Weir type over flow to control water level when pond is cleaned and reactivated. See Exhibit - 1, "Typical Discharge and Primary Overflow".

2 ea. 18" dia steel pipes

di = 17.25 in. I.D.
r = 0.3593
n = 0.013
km = 1.0

L = 1.4375 feet
A = 1.623 sq. ft.
D = 1.4375 ft.
kp = 0.0193
kpL = .0193 x 60 = 1.158

Plant Circulation = 7.5752 cfs ÷ 2 = 3.7876 cfs ea. pipe

Weir Crest L = 1.4375' H = 0.94 ft.

See Exhibit - 2, Upper Refuse Pond - 2 each 18" dia. Weir over flow pipes and Exhibit - 3, Plan View - Weir Type Overflow Pipes

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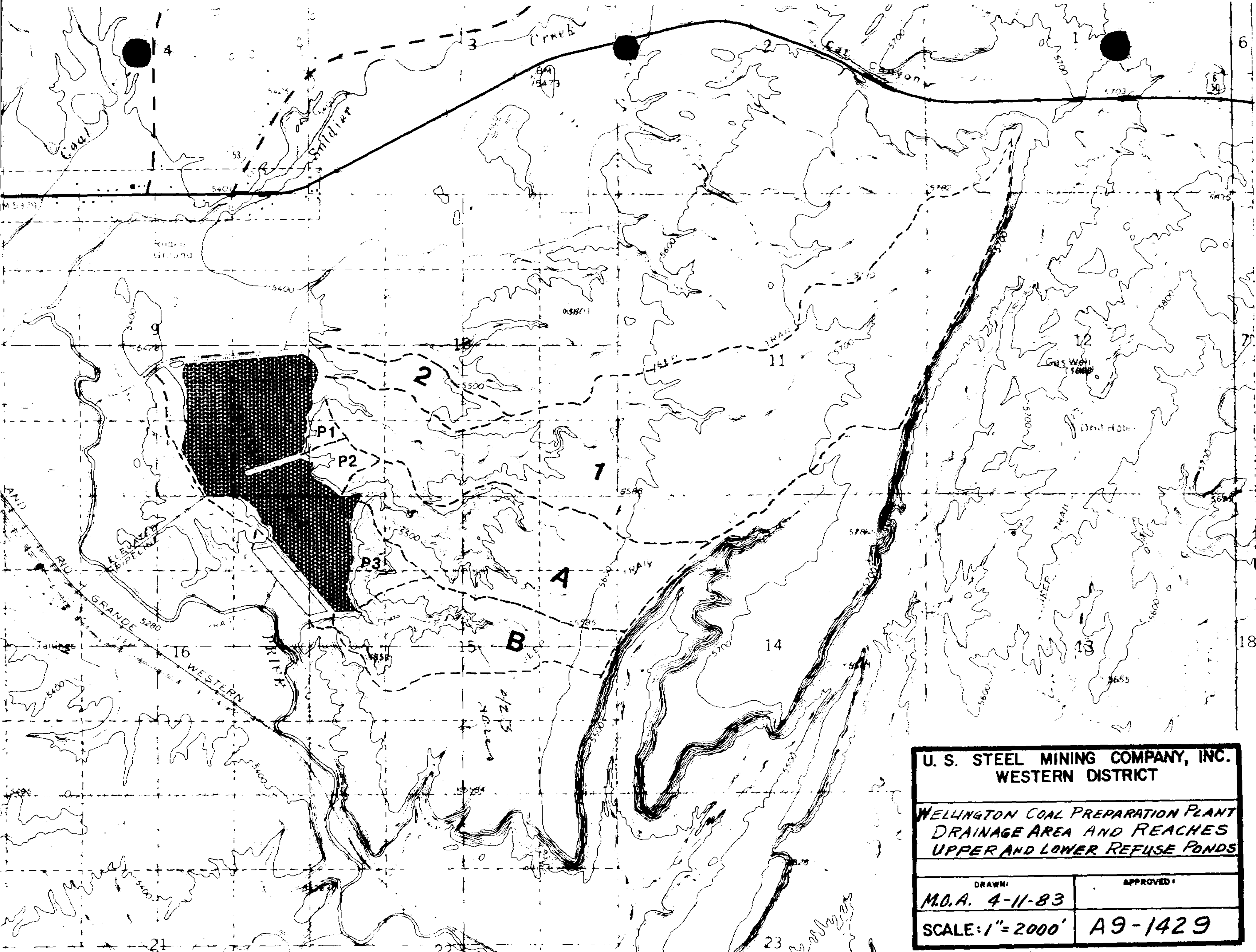
Storm Input into pond other than reach 1 and 2.

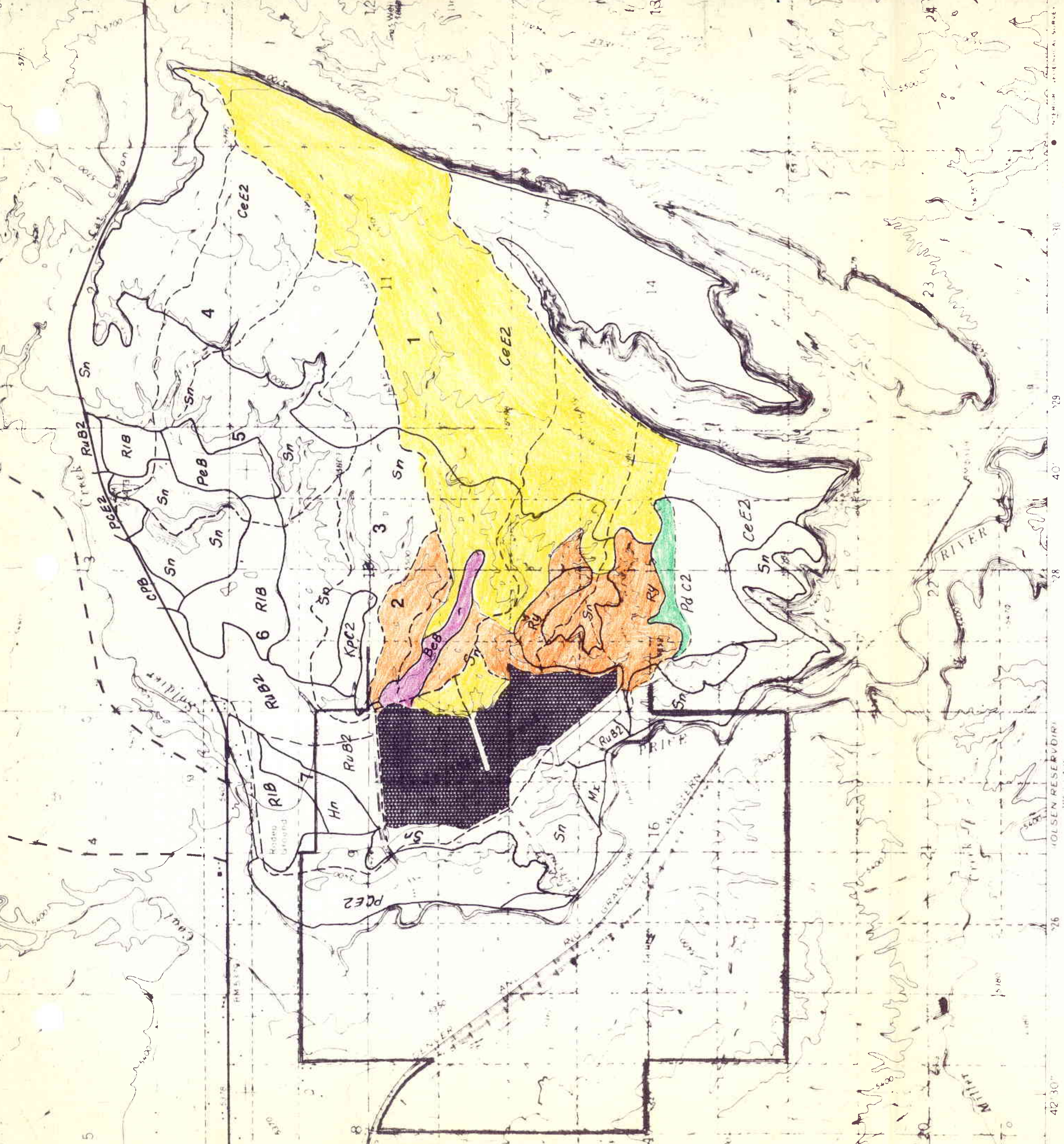
Runoff	10-24(1.81)	25-24(2.18)	100-24(2.74)	100-6(1.91)
Point 1 - sheet 23	0.804	1.078	1.530	0.871
Rainfall into Pond (52.365 s)	7.942	9.513	11.957	8.335
Total	8.746	10.591	13.487	9.206



U. S. STEEL MINING COMPANY, INC.
WESTERN DISTRICT

WELLINGTON COAL PREPARATION PLANT
DRAINAGE AREA AND REACHES
UPPER AND LOWER REFUSE PONDS

DRAWN:	APPROVED:
M.O.A. 4-11-83	
SCALE: 1" = 2000'	A9-1429



SOIL SURVEY - CARBON-EMERY AREA, UTAH (Book)

SOILS TABLE (Reference 1)

Soil Symbol	Name	Hyd-Group	Classification
BeB	Beebe	A	Loamy Fine Sand
CeE2	Castle Valley	D	Very Find Sandy Loam
Hn	Hunting	B	Loam
KpC2	Killpack	C	Clay Loam or Loam
PCE2	Persayo (2)	D	Loam and Silty Clay
PdC2	Palisade	B	Very Fine Sandy Loam
PeB	Penoyer	B	Loam
RlB	Ravola	B	Loam
RuB2	Ravola (3)	B	Loam
Ry	Rocky Land (4)	C	A Miscellaneous Land Type
Sn	Shaly Colluvial (5)	C	A Mixture of Soil Material
Sn	Shaly Colluvial (6)	D	A Mixture of Soil Material

References and Notes:

(1) Above Table extracted from Table 3. "Estimated soil properties significant to Engineering" Page 25-27 Soil Survey - Carbon-Emery Area Utah.

(2) Associated with the Chipeta soil series.

(3) Associated with the Bunderson soil series.

(4) Use C soil because of proximity of C and B soils.

(5) Use C soil because of proximity of C and B soils.

(6) Use D soil because of proximity of D soil

Hydrologic Soil Group - Color Code

Soil Type A

Soil Type B

Soil Type C

Soil Type D

U. S. STEEL MINING COMPANY, INC.
WESTERN DISTRICT

WELLINGTON COAL PREPARATION PLANT
HYDROLOGIC SOIL MAP
DRAINAGE AREA TO REFUSE PONDS

DRAWN	APPROVED
M.D.A 4-11-83	
SCALE: 1" = 2000'	C9-1283

Subject STORM RUNOFF CALCULATIONS

REACH 1 - Upper Refuse Pond

WELLINGTON COAL PREPARATION PLANT

CALCULATION NOTES

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Sheet No. 1 of 23 Sheets

Climatic Condition = AMC-I

Map Scale 1" = 2000'

Acres Per Sq. Inch = 91.82736

AMC-II

P = 1.82 (10-24) P = 2.74 (100-24)

P = 2.18 (25-24) P = 1.91 (100-6)

$$Q = \frac{(P-0.2S)^2}{P+0.8S}$$

Table I Area- Acre Conversion and Weighted Curve Numbers

Area	Sq. In.	Acres	Fraction of Total Acres	CN	Weighted CN
Ce E2 (D)	5.15	472.911	0.78267	86	67.31
Be B (A)	0.24	22.033	0.03648	49	1.79
Sn (C)	0.45	41.322	0.06839	82	5.61
Sn (D)	0.74	67.553	0.11246	84	9.45
Totals		604.224	1.00000		84.158

Runoff Calculation Data

AMC-II
CN= 84
S= 1.90
Q(10.24)= 0.621
Q(25.24)= 0.875
Q(100.24)= 1.307
Q(100-6)= 0.692

AMC-I
CN= 68
S= 4.70
Q(10-24)= 0.139
Q(25-24)= 0.255
Q(100-24)= 0.492
Q(100-6)= 0.166

Table 2 Weighted Quantity of Runoff = Q x Fraction of Total Acres

AMC-II					
CN	S	10-24 Storm Q	10-24 Storm Wtd. Q	25-24 Storm Q	25-24 Storm Wtd. Q
86	1.63	0.714	0.559	0.987	.772
49	10.4	(1)	—	0.001	.000
82	2.20	0.532	0.036	0.768	.053
84	1.90	0.620	0.069	0.876	.098
Totals			0.665		0.923

AMC-II					
CN	S	100-24 Storm Q	100-24 Storm Wtd. Q	100-6 Storm Q	100-6 Storm Wtd. Q
		1.441	1.128	0.781	.611
		0.039	.001	0.003	.000
		1.176	.080	0.589	.040
		1.307	.147	0.682	.077
Totals			1.357		0.728

STORM RUNOFF COMPUTATIONS:
(Use largest Q. Value from Tables 1 & 2)

	AMC-I 10-24	25-24	AMC- 100-24	100-6
Quantity, inches runoff =	0.665	0.923	1.357	0.728
Volume, Acre-feet =	33.48	46.47	68.32	36.66
Cubic-feet =				
Peak Discharge =				

(1) Curve begins higher than P.

CALCULATION NOTES

Subject *UNIT HYDROGRAPH*
Time of Concentration - Tc
REACH 1
W.C.P.P
AMC-II

By *M.O.A.*
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Acc't
-Copy- *3-10-1982*
Sheet No. *2* of *23* Sheets

TABLE $T_t = \frac{l}{3600\,V}$ $T_c = \sum T_t$ (increments)

Contour Elev.	Elev. Diff H.	Incremental Reach Length-L	Grade	Fig. 15.2 Velocity	Tt Hours	Remarks
5820						
	120	3600	.03333	1.84	.543	
5700						
	100	3600	.02777	1.69	.592	
5600						
	100	2900	.03448	1.85	.435	
5500						
	100	4600	.02167	1.48	.863	
5400						
	18.8	1550	.0121	1.10	.391	
5381.2						
	6.2	600	.0103	1.02	.163	
5375						
	1.0	350	.00285	0.53	.183	
5374						
Total		17200			3.170	= Tc
5820						
	446	17200	.02593	1.61	2.968	Do Not Use.
5374						

CALCULATION NOTES

Subject Design Storms

Unit Hydrograph Calculations

Reach / UPPER REFUSE POND

Storm Duration= 24 Hours

See page 2 of calc. of 3-10-82

By M.O.A

Checked

Acc't

4-22- 1983

Sheet No. 3 of 23 Sheets

Page 1 of 2, Calculation Sheets

DATA:

Design Storm, 10 year, 24 hours, Precipitation= 1.82 inches.

Drainage Area, A= 604.22 Acres, 0.9441 Square Miles

Reach / , Hydraulic Length, 1720 feet.

Time of Concentration, T_c = 3.17 hours. (Computations page 2)(3-10-82 Calc)

CN = 84 Weighted Moisture Condition = AMC - II

Quantity, Q = 1 inch for unit Hydrograph.

Design Storm(s) Runoff = (10 - 24) = 1.82" Q = 0.665" (100-24) = 2.74" Q = 1.357"

(25 - 24) = 2.18" Q = 0.923"

FORMULAS: (References - NEH, Section 4, Hydrology)

(1) $T_p = 0.5(D) + 0.6\,(T_c)$ (Eq. 17-45)

(2) $T_r = 1.67\,T_p$ (pg. 16.6)

(3) $T_b = T_p + T_r$ (pg. 16.6)

(4) $q_p = \frac{484AQ}{T_p}$ (Eq. 16.6)

Volume Under Unit Hydrograph

(5) V = 645.33 x A = cfs - hours (pg. 16.10)

Tp=Time to peak hours.
D =Storm Duration, Total
Tc=Time of Concentration, Σ of Reach Travel Time (Tt).
Tr=Time of Recession, Triangular Hydro.
Tb=Time of Base, Triangular Hydrograph
A =Area of Drainage Basin, Sq. Miles

645.33=Rate of Discharge, 1 inch from 1 sq. mile in 1 hour.

qp = Peak Flow, cfs.

CALCULATIONS:

(1) $T_p = 0.5 \times 24 + 0.6 \times 3.17 = 12 + 1.902 = 13.902$ Hrs.

(2) $T_r = 13.902 \times 1.67 = 23.21634$ hrs use 23.22 hrs.

(3) $T_b = 13.902 + 23.21634 = 37.11834$ hrs use 37.12 hrs

(4) $q_p = \frac{484 \times .9441 \times 1}{13.902} = 32.869$ cfs

(5) $V = 645.33 \times .9441 = 609.256$ cfs-hrs. × .03264 =

Design Storms - Peak flows

10-24 Q = 0.665" = qp = 21.858 cfs

25-24 Q = 0.923" = qp = 30.338 cfs.

100-24 Q = 1.537" = qp = 44.603 cfs.

Subject Design Storms

[Un]it Hydrograph Calculations

Reach 1 *Upper Refuse Pond*

Storm Duration = 24 Hours

CALCULATION NOTES

By *M.O.A.*

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Acc't

4-22-1983

Sheet No. *4* of *23* Sheets

Page 2 of 2, Calculation Sheets

Reference Chapter 16, Pages 16.7 and 16.8

FORMULAS:

$L = 0.6\ T_{c2}$ (Eq. 15.3, Page 15.6)

Point of inflection, P.I. = 1.7 Tp

$\Delta D = 0.2\ T_p$ (Pg. 16.8)

$T_p = \frac{\Delta D}{2} + L$ (Eq. 16.7)

$T_{c2} + \Delta D = 1.7\ T_p$ (Eq. 16.10)
Therefore $T_{c2} = 1.7\ T_p - \Delta D$

$\frac{\Delta D}{2} + 0.6\ T_{c2} = T_p$ (Eq. 16.11)

$\Delta D = 0.133\ T_{c2}$ (Eq. 16.12)

L = Lag, is time from center of excessive rainfall to peak rate of runoff.

T_{c2} = Time of concentration. Time from end of excess rainfall to point of inflection of unit hydrograph.

ΔD = Duration of excessive rainfall.

Tp = Time to peak

CALCULATIONS: $T_p = 13.902$ hrs

$P.I. = 1.7 \times 13.902 = 23.6334$

$\Delta D = 0.2 \times 13.902 = 2.7804$

$L = 13.902 - \frac{2.7804}{2} = 12.5118$

$T_{c2} = 23.6334 - 2.7804 = 20.853$

$T_p = \frac{2.7804}{2} + .6 \times 20.853 = 13.902$ check

$\Delta D = 0.133 \times 20.853 = 2.7734 < 2.7804$ good

Summary:

$\Delta D = 2.78$ hrs. $\frac{\Delta D}{2} = 1.39$ hrs.

$P.I. = 23.633$ hrs.

$T_{c2} = 20.853$ hrs.

$L = 12.51$ hrs.

CALCULATION NOTES

Subject Unit Hydrograph

ORDINATES FOR UNIT HYDROGRAPH

Reach 1 Upper Refuse Pond

Storm Duration = 24 Hours

By M.O.A

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Acc't

4-22-1983

Sheet No. 5 of 23 Sheets

Table **Computation of coordinates for unit hydrograph**

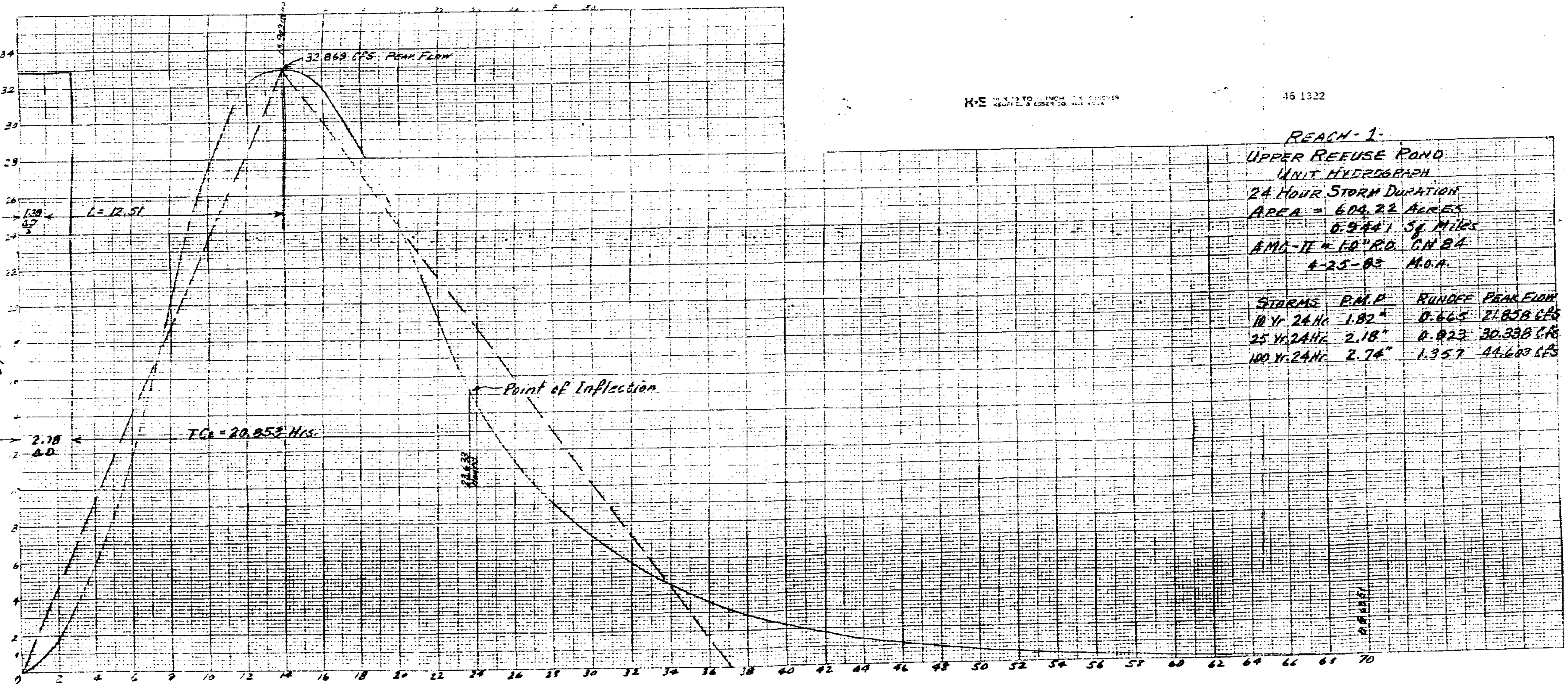
1	2	3	4
Time Ratios (table 16.1)	**Time (col 1 x Tp 13.902)**	**Discharge Ratios (table 16.1)**	**Discharges (col 3 x Qp 32.8694)**
(t/T_p)	(hours)	(q/q_p)	(cfs)
.0	0	0	0
.1	1.39	.030	.99
.2	2.78	.100	3.29
.3	4.17	.190	6.25
.4	5.56	.310	10.19
.5	6.95	.470	15.45
.6	8.34	.660	21.69
.7	9.73	.820	26.95
.8	11.12	.930	30.57
.9	12.51	.990	32.54
1.0	13.90	1.000	32.869
1.1	15.29	.990	32.54
1.2	16.68	.930	30.57
1.3	18.07	.860	28.27
1.4	19.46	.780	25.64
1.5	20.85	.680	22.35
1.6	22.24	.560	18.41
1.7	23.63	.460	15.12
1.8	25.02	.390	12.82
1.9	26.41	.330	10.84
2.0	27.80	.280	9.20
2.2	30.58	.207	6.80
2.4	33.36	.147	4.83
2.6	36.14	.107	3.52
2.8	38.93	.077	2.53
3.0	41.71	.055	1.81
3.2	44.49	.040	1.31
3.4	47.27	.029	.95
3.6	50.05	.021	.69
3.8	52.83	.015	.49
4.0	55.61	.011	.36
4.5	62.56	.005	.16
5.0	69.51	0	0

See Unit Hydrograph "Reach-1- Upper Refuse Pond" Pg. 5A
24 Hour Storm Duration
AMC-II - 1.0" R.O. CN 84.

REACH - 1 -

UPPER REFUSE POND
UNIT HYDROGRAPH
24 HOUR STORM DURATION
AREA = 604.22 ACRES
0.9441 Sq. Miles
AMC-II = 1.0" R.O. CN 84
4-25-83 M.O.A.

STORMS	P.M.P.	RUNOFF	PEAK FLOW
10 Yr. 24 Hr.	1.82"	0.665	21.858 CFS
25 Yr. 24 Hr.	2.18"	0.923	30.338 CFS
100 Yr. 24 Hr.	2.74"	1.357	44.603 CFS



CALCULATION NOTES

By M.O.A.

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4-22-1983

Sheet No. 6 of 23 Sheets

Subject *UNIT HYDROGRAPH ANALYSIS*

SUMMATION of ORDINATES

[illegible]ACH 1, Upper Refuse Pond

STORM DURATION: 24 HOURS

Design

TIME INCREM. HOURS	ORDINATES OF UNIT HYDROGRAPH CFS	cfs 10-24	cfs 25-24	cfs 100-24	TIME INCREMENT HOURS	ORDINATES OF UNIT HYDROGRAPH CFS	cfs 10-24	cfs 25-24	cfs 100-24
Q →	1"	0.665	0.923	1.537	Q =	1"	0.665	0.923	1.537
0	0	0	0	0	60	0.22	0.15	0.20	0.34
2	1.30	1.20	1.66	2.77	62	0.17	0.11	0.16	0.26
4	5.95	3.96	5.49	9.15	64	0.11	0.07	0.10	0.17
6	11.80	7.85	10.89	18.14	66	0.07	0.05	0.06	0.11
8	20.23	13.45	18.67	31.09	68	0.03	0.02	0.03	0.05
10	27.64	18.38	25.51	42.48	70	0	0	0	0
12	32.10	21.35	29.63	49.34					
14	32.86	21.85	30.33	50.51		304.65			
16	31.83	21.17	29.38	48.92		×2 = 609.3			
18	28.35	18.85	26.17	43.57	cfs-Hrs	609.256	405.153	562.343	936.426
20	24.40	16.23	22.52	37.50	Ac. Ft	50.349	33.482	46.472	77.386
22	19.05	12.67	17.58	29.28					
24	14.42	9.47	13.14	21.89					
26	11.35	7.55	10.48	17.45					
28	9.01	5.99	8.32	13.85					
30	7.13	4.74	6.58	10.96					
32	5.72	3.80	5.28	8.79					
34	4.50	2.99	4.15	6.92					
36	3.54	2.35	3.27	5.44					
38	2.81	1.87	2.59	4.32					
40	2.22	1.48	2.05	4.21					
42	1.75	1.16	1.62	2.69					
44	1.40	0.93	1.29	2.15					
46	1.05	0.70	0.97	1.61					
48	0.88	0.59	0.81	1.35					
50	0.70	0.47	0.65	1.08					
52	0.54	0.36	0.50	0.83					
54	0.42	0.28	0.39	0.65					
56	0.33	0.22	0.30	0.51					
58	0.27	0.18	0.25	0.41					

Subject STORM RUNOFF CALCULATIONS

REACH 2, Upper Refuse Pond

WELLINGTON COAL PREPARATION PLANT

See Sheet 5-3-3-82 -Copy-

Climatic Condition = AMC-I

CALCULATION NOTES

By M.O.A.

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-Copy- March 3, 1982

Sheet No. 11 of 23 Sheets

Map Scale 1" = 2000 P = 1.82 (10-24) P = 2.74 (100-24)

Acres Per Sq. Inch = 91.82736 P = 2.18 (25-24) P = 1.91 (100-6)

AMC-II

$$Q = \frac{(P-0.2S)^2}{P+0.8S}$$

Table I Area- Acre Conversion and Weighted Curve Numbers

Area	Sq. In.	Acres	Fraction of Total Acres	CN	Weighted CN
Sn (C)	0.47	43.16	0.92159	79	72.81
BeB (A)	0.035	3.21	0.06854	49	3.36
RuB2 (D)	0.005	0.46	0.00982	89	0.87
Totals	0.51	46.832	.99995		77.04

Runoff Calculation Data

AMC-II
CN= 77
S= 2.99
Q(10.24)= 0.354
Q(25.24)= 0.547
Q(100.24)= 0.894
Q(100-6)= 0.400

AMC-I
CN=
S=
Q(10-24)=
Q(25-24)=

Table 2 Weighted Quantity of Runoff = Q x Fraction of Total Acres

AMC-I					
CN	S	10-24 Storm Q	10-24 Storm Wtd. Q	25-24 Storm Q	25-24 Storm Wtd. Q
79	2.66	0.42	0.387	0.630	.581
49	10.4	(1)	—	0.001	.000
89	1.24	0.778	0.007	1.177	.012
Totals			0.394		0.593

AMC-					
CN	S	100-24 Storm Q	100-24 Storm Wtd. Q	100-6 Storm Q	100-6 Storm Wtd. Q
		1.001	.923	0.470	.433
		0.039	.003	0.003	.000
		1.664	.016	0.952	.009
Totals			0.942		0.443

STORM RUNOFF COMPUTATIONS:
(Use largest Q. Value from Tables 1 & 2)

	AMC-II 10-24	25-24	AMC-II 100-24	100-6
Quantity, inches runoff =	0.394	0.593	0.942	0.443
Volume, Acre-feet =				
Cubic-feet =				
Peak Discharge =				

CALCULATION NOTES

Subject *UNIT HYDROGRAPH*

By *M.O.A.*

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Acc't

REACH 2, Upper Refuse Pond

— COPY — *3-11-1982*

See pg. 6 of 3-11-82 ✓

Sheet No. *12* of *23* Sheets

TABLE $T_t = \frac{L}{3600\ V}$ $T_c = \sum T_t$ (increments)

Contour Elev.	Elev. Diff H.	Incremental Reach Length-L	Incremental Reach Grade	Fig. 15.2 Velocity	T_t Hours	Remarks
5580						Head Elev.
	80	510	.15686	3.94	.036	
5500						
	100	2350	.0425	2.06	.317	
5400						
	18.8	610	.0308	1.76	.096	
5381.2						Jct. Reach 1.
	6.2	600	.0103	1.02	.163	
5375						Point B.
	1.0	350	.00285	0.53	.183	
5374						Point E. (Pond)
Total	206	4420		2.17	0.795	use 0.8 hours

CALCULATION NOTES

Subject Design Storms

Unit Hydrograph Calculations

Reach 2 Upper Refuse Pond

Storm Duration= 24 Hours

Refer Sheets 5 & 6, March 3, & 11, 1982.

By M.O.A

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4-22 1983

Sheet No. 13 of 23 Sheets

Page 1 of 2, Calculation Sheets

DATA:

Design Storm, 10 year, 24 hours, Precipitation= 1.82 inches.

Drainage Area, A= 46.832 Acres, 0.0732 Square Miles

Reach 2 , Hydraulic Length, 4420 feet.

Time of Concentration, T_c = 0.795 hours. (Computations page 6)

CN = 77 Weighted Moisture Condition = AMC - II

Quantity, Q = 1 inch for unit Hydrograph.

Design Storm(s) Runoff = (10 - 24) = 1.82 Q = 0.394 (100 -24) = 2.74" Q = 0.942"

(25 - 24) = 2.18 Q = 0.593

FORMULAS: (References - NEH, Section 4, Hydrology)

(1) $T_p = 0.5(D) + 0.6\ (T_c)$ (Eq. 17-45)

(2) $T_r = 1.67\ T_p$ (pg. 16.6)

(3) $T_b = T_p + T_r$ (pg. 16.6)

(4) $q_p = \frac{484AQ}{T_p}$ (Eq. 16.6)

Tp=Time to peak hours.
D =Storm Duration, Total
Tc=Time of Concentration, Σ of Reach Travel Time (Tt).
Tr=Time of Recession, Triangular Hydro.
Tb=Time of Base, Triangular Hydrograph
A =Area of Drainage Basin, Sq. Miles

Volume Under Unit Hydrograph

(5) $V = 645.33 \times A$ = cfs - hours (pg. 16.10)

645.33=Rate of Discharge, 1 inch from 1 sq. mile in 1 hour.

CALCULATIONS:

(1) $T_p = .5 \times 24 + 0.6 \times 0.795 =$
$12 + 0.477 = 12.477$

(2) $T_r = 1.67 \times 12.477 = 20.83659$

(3) $T_b = 12.477 + 20.83659 = 33.31359$

(4) $q_p = \frac{484 \times .0732 \times 1}{12.477} = 2.840$ cfs.

(5) $V = 645.33 \times .0732 = 47.238$ cfs-hrs $\times .08264 = 3.904$ Ac.ft

Design Storms - Peak Flow

10-24 = 1.82 Q = 0.394" qp = 1.119 cfs.

25-24 = 2.18 Q = 0.593" qp = 1.684 cfs.

100-24 = 2.74 Q = 0.942" qp = 2.675 cfs.

Subject Design Storms

Unit Hydrograph Calculations

Reach 2

Storm Duration = 24 Hours

CALCULATION NOTES

By M.O.A

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4-22-1983

Sheet No. 14 of 23 Sheets

Page 2 of 2, Calculation Sheets

Reference Chapter 16, Pages 16.7 and 16.8

FORMULAS:

(4) $L = 0.6\ Tc_2$ (Eq. 15.3, Page 15.6)

(1) Point of inflection, P.I. = 1.7 Tp

(2) $\Delta D = 0.2\ Tp$ (Pg. 16.8)

(5) $Tp = \frac{\Delta D}{2} + L$ (Eq. 16.7)

$Tc_2 + \Delta D = 1.7\ Tp$ (Eq. 16.10)

(3) Therefore $Tc_2 = 1.7\ Tp - \Delta D$

(5) $\frac{\Delta D}{2} + 0.6\ Tc_2 = Tp$ (Eq. 16.11)

(6) $\Delta D = 0.133\ Tc_2$ (Eq. 16.12)

L = Lag, is time from center of excessive rainfall to peak rate of runoff.

Tc_2 = Time of concentration. Time from end of excess rainfall to point of inflection of unit hydrograph.

ΔD = Duration of excessive rainfall.

Tp = Time to peak

CALCULATIONS: Tp = 12.477 hrs.

(1) P.I. = 1.7 × 12.477 = 21.2109 hrs.

(2) ΔD = 0.2 × 12.477 = 2.4954 hrs. $\frac{\Delta D}{2}$ = 1.2477

(3) Tc_2 = 21.2109 − 2.4954 = 18.7155

(4) L = 0.6 × 18.7155 = 11.2293

(5) Tp = 1.2477 + 11.2293 = 12.477 check

(6) ΔD = 0.133 × 18.7155 = 2.48916 < 2.4954 good check

Summary

ΔD = 2.50 hrs. $\frac{\Delta D}{2}$ = 1.25 hrs.

P.I. = 21.21 hrs.

Tc_2 = 18.72 hrs.

L = 11.23 hrs.

Subject Unit Hydrograph

Reach

Storm Duration = Hours

CALCULATION NOTES

By M.O.A.

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Acc't

4-22-1983

Sheet No. 15 of 23 Sheets

Table Computation of coordinates for unit hydrograph

1	2	3	4
Time Ratios (table 16.1)	Time (col 1 x Tp) 12.477	Discharge Ratios (table 16.1)	Discharges (col 3 x Qp) 2.84 cfs.
(t/T_p)	(hours)	(q/q_p)	(cfs)
.0	0	0	0
.1	1.25	.030	0.09
.2	2.50	.100	0.28
.3	3.74	.190	0.54
.4	4.99	.310	0.88
.5	6.24	.470	1.33
.6	7.49	.660	1.87
.7	8.73	.820	2.33
.8	9.98	.930	2.64
.9	11.23	.990	2.81
1.0	12.48	1.000	2.84
1.1	13.72	.990	2.81
1.2	14.97	.930	2.64
1.3	16.22	.860	2.44
1.4	17.47	.780	2.22
1.5	18.71	.680	1.93
1.6	19.96	.560	1.59
1.7	21.21	.460	1.31
1.8	22.46	.390	1.11
1.9	23.71	.330	.94
2.0	24.95	.280	.80
2.2	27.45	.207	.59
2.4	29.94	.147	.42
2.6	32.44	.107	.30
2.8	34.94	.077	.22
3.0	37.43	.055	.16
3.2	39.93	.040	.11
3.4	42.42	.029	.08
3.6	44.92	.021	.06
3.8	47.41	.015	.04
4.0	49.91	.011	.03
4.5	56.15	.005	.01
5.0	62.39	0	0

SEE UNIT HYDROGRAPH - REACH 2 - Upper Refuse Pond" pg. 15A

24 Hour Storm Duration

AMC-II - 1.0" R.O. CN77

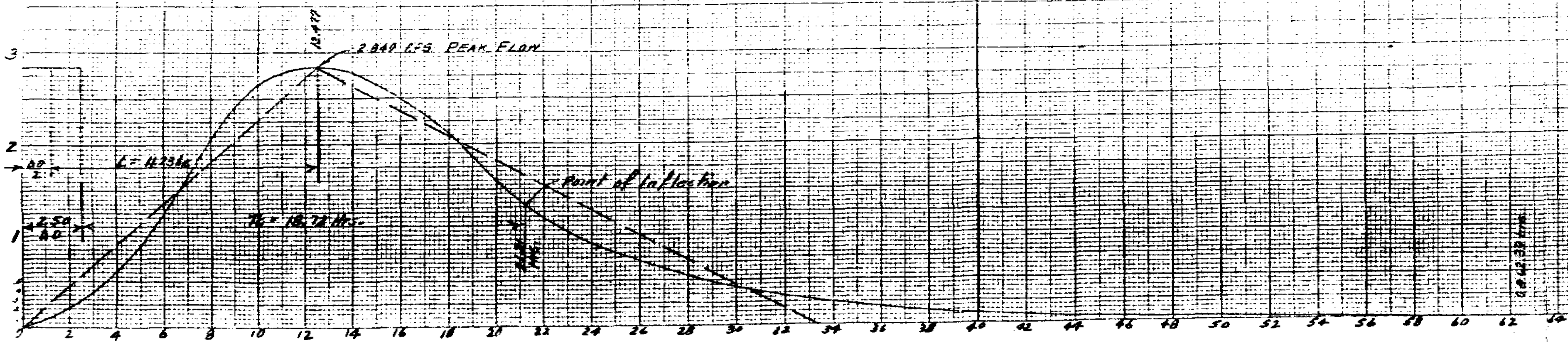
K+E 10 X 10 TO 1/2 INCH 7 X 10 INCHES KEUFFEL & ESSER CO. MADE IN U.S.A. 46 1322

K+E 10 X 10 TO 1/2 INCH 7 X 10 INCHES KEUFFEL & ESSER CO. MADE IN U.S.A. 46 1322

REACH - 2

UPPER REFUSE POND
UNIT HYDROGRAPH
24 HOUR STORM DURATION
AREA = 46.832 ACRES
0.0732 Sq. MILES
AMC-II = 1.0" R.O. CN 77
4-25-83 M.O.A.

STORMS	P.M.P.	RUNOFF	PEAK FLOW
10 Yr. 24 Hr.	1.82	0.394	1.119 CFS
25 Yr. 24 Hr.	2.18	0.593	1.634 "
100 Yr. 24 Hr.	2.74	0.942	2.675 "



CALCULATION NOTES

Subject *UNIT HYDROGRAPH ANALYSIS*
SUMMATION of ORDINATES
REACH 2 Upper Refuse Pond
STORM DURATION: 24 HOURS

By *M. O. A.*
Checked
Acc't
4-22- 19*83*
Sheet No. *16* of *23* Sheets

TIME INCREM. HOURS	ORDINATES OF UNIT HYDROGRAPH CFS	CFS 10-24	CFS 25-24	CFS 100-24	TIME INCREMENT HOURS	ORDINATES OF UNIT HYDROGRAPH CFS	CFS 10-24	CFS 25-24	CFS 100-24
Q →	1"	0.349	0.593	0.942	Q =	1"	0.349	0.593	0.942
0	0	0	0	0	50	0.035	0.01	0.02	0.03
2	0.21	0.07	0.12	0.20	52	0.025	0.01	0.01	0.02
4	0.61	0.21	0.36	0.57	54	0.02	0.01	0.01	0.02
6	1.25	0.44	0.74	1.18	56	0.015	0.01	0.01	0.01
8	2.06	0.72	1.22	1.94	58	0.01	0.003	0.01	0.01
10	2.65	0.92	1.57	2.50	60	0.005	0.002	0.003	0.005
12	2.83	0.99	1.68	2.67	62	0	0	0	0
14	2.77	0.97	1.64	2.61					
16	2.48	0.87	1.47	2.34		23.675			
18	2.11	0.74	1.25	1.99		47.35			
20	1.59	0.55	0.94	1.50	cfs-hours	47.238	16.486	28.012	44.498
22	1.18	0.41	0.70	1.11	Ac. Ft.	3.904	1.362	2.315	3.677
24	0.90	0.31	0.53	0.85					
26	0.71	0.25	0.42	0.67					
28	0.545	0.19	0.32	0.51					
30	0.42	0.15	0.25	0.40					
32	0.32	0.11	0.19	0.30					
34	0.25	0.09	0.15	0.24					
36	0.19	0.07	0.11	0.18					
38	0.145	0.05	0.09	0.14					
40	0.11	0.04	0.07	0.10					
42	0.07	0.02	0.04	0.07					
44	0.065	0.02	0.04	0.06					
46	0.055	0.02	0.03	0.05					
48	0.045	0.02	0.03	0.04					
	23.565								

CALCULATION NOTES

By M.O.A

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Acc't

4-25-1983

Sheet No. 21 of 22 Sheets

Subject UNIT HYDROGRAPH ANALYSIS

SUMMATION OF ORDINATES

REACH 1+2- Upper Refuse Pond

STORM DURATION: 24 HOURS

TIME INCREM. HOURS	ORDINATES OF UNIT HYDROGRAPH CFS	10-24	25-24	100-24
Q →	1"			
0		0	0	0
2		1.27	1.78	2.97
4		4.17	5.85	9.72
6		8.29	11.63	19.32
8		14.17	19.89	33.03
10		19.30	27.08	44.98
12		22.34	31.31	52.01
14		22.82	31.97	53.12
16		22.04	30.85	51.26
18		19.59	27.42	45.56
20		16.78	23.46	39.00
22		13.08	18.28	30.39
24		9.79	13.67	22.74
26		7.80	10.90	18.10
28		6.18	8.64	14.36
30		4.89	6.83	11.36
32		3.91	5.47	9.09
34		3.08	4.30	7.16
36		2.42	3.38	5.62
38		1.92	2.68	4.46
40		1.52	2.12	4.31
42		1.18	1.66	2.76
44		0.95	1.33	2.21
46		0.72	1.00	1.66
48		0.61	0.84	1.39
50		0.48	0.67	1.11
52		0.37	0.51	0.85
54		0.29	0.40	0.67
56		0.23	0.31	0.52
58		0.183	0.26	0.42

TIME INCREMENT HOURS	ORDINATES OF UNIT HYDROGRAPH CFS	10-24	25-24	100-24
Q =	1"			
60		0.152	0.203	0.345
62		0.11	0.16	0.26
64		0.07	0.10	0.17
66		0.05	0.06	0.11
68		0.02	0.03	0.05
70		0	0	0
Cfs. Hrs.		421.639	590.355	980.924
Ac. Ft.		34.844	48.787	81.063

CALCULATION NOTES

Subject STORM RUNOFF CALCULATIONS

~~REACH~~ Point 1 - Upper Refuse Pond

WELLINGTON COAL PREPARATION PLANT

By M.A.A.

Checked

Acc't

19

Climatic Condition = AMC-II

Sheet No. 23 of 23 Sheets

Map Scale 1" = 2000 ft. P = 1.82 (10-24) P = 2.74 (100-24)

Acres Per Sq. Inch = 91.82736 P = 2.18 (25-24) P = 1.91 (100-6)

AMC-II

$$Q = \frac{(P-0.2S)^2}{P+0.8S}$$

Table I Area- Acre Conversion and Weighted Curve Numbers

Area	Sq. In.	Acres	Fraction of Total Acres	CN	Weighted CN	Runoff Calculation Data
Sn (D)	0.12	11.019	1.0000	89	89	AMC-II CN= 89 S= 1.24 Q(10.24)= 0.876 Q(25.24)= 1.174 Q(100.24)= 1.666 Q(100.6)= 0.949 AMC-I CN= 76 S= 3.16 Q(10-24)= Q(25-24)= Q(100-24) Q(100-6)
Totals		11.019				

Table 2 Weighted Quantity of Runoff = Q x Fraction of Total Acres

AMC-I						AMC-					
		10-24	Storm	25-24	Storm				Storm		Storm
CN	S	Q	Wtd. Q	Q	Wtd. Q	CN	S	Q	Wtd. Q	Q	Wtd. Q
89	1.24										
Totals						Totals					

STORM RUNOFF COMPUTATIONS:
(Use largest Q. Value from Tables 1 & 2)

	AMC-II 10-24	25-24	AMC-II 100-24	100-6
Quantity, inches runoff =	0.376	1.174	1.666	0.949
Volume, Acre-feet = 11.019 Ac.	0.804	1.078	1.530	0.871
Cubic-feet =	35039.10	46,958.79	66639.28	37959.02
~~Peak Discharge~~ =				

CALCULATION NOTES

Subject Lower Refuse Pond	By MOA
Data Sheet	Checked
Wellington Coal Cleaning Plant	Acc't
	Date April 12 1983
	Sheet No. D-1 of 1 Sheets

DATA:

Reach A and B flow into the Lower Refuse Pond

AREAS

Reach - A = 212.121 acres		Point 2 = 15.611 acres
Reach - B = 133.150 acres		Point 3 = 26.171 acres
Totals	345.271 acres	41.782 acres

Lower Refuse Pond = 59.867 acres

This pond was taken out of service March 30, 1983, except for plant circulation water presently flowing through pond in a ditch. To increase dam height is purpose of this study.

Plant Circulation Water = 7.5752 cfs

Discharge Pipe through Lower Refuse Dam - Present

One (1) 18" dia. pipe with weir type control arrangement
Crest L = 1.00 ft.

Future Discharge Pipes

Three 18" dia. pipes with weir type water level control, will be installed to control the water level in the proposed new dam and to by pass the storm runoff into the pond from Reaches A & B and from discharge of Upper Refuse Pond. See Schematic Dwg.s Exhibit-1, Typical Discharge and Primary Overflow, Exhibit-3, Plan View-Weir Type Overflow Pipe, and Exhibit-4, Lower Refuse Dam Enlargement.

Plant circulation through 3 pipes = 7.5752 ÷ 3 = 2.5251 cfs.

Depth of flow over weir to carry flow = 0.70 ft.

Storm Input into Pond Other Than Reach Inflow:

Runoff	10-24(1.82")	25-24(2.18")	100-24(2.74")	100-6(1.91")
Point 2, ac ft.	1.144	1.531	2.165	1.238
Point 3, ac ft.	1.082	1.577	2.440	1.202
Rainfall into pond (59.867 ac)	9.080	10.876	13.670	9.529
Total	11.306	13.984	18.275	11.969

Subject STORM RUNOFF CALCULATIONS

[redacted] POINT-2 RUNOFF INTO LOWER REFUSE POND

WELLINGTON COAL PREPARATION PLANT

CALCULATION NOTES

By M.O. Anderson

Checked

Acc't

4-12-1998

Sheet No. 2 of Sheets

Climatic Condition = AMC-

Map Scale 1" = 2000'

Acres Per Sq. Inch =

AMC-

P = 1.82 (10-24) P = 2.74 (100-24)

P = 2.18 (25-24) P = 1.91 (100-6)

$$Q = \frac{(P-0.2S)^2}{P+0.8S}$$

AMC-II

Table I Area- Acre Conversion and Weighted Curve Numbers

Area	Sq. In.	Acres	Fraction of Total Acres	CN	Weighted CN	Runoff Calculation Data
① Sn-D-	0.17	15.611	1.00	89		AMC-11 CN= 89 S= 1.24 Q(10-24)= 0.879 Q(25-24)= 1.177 Q(100-24)= 1.664 Q(100-6)= 0.952 AMC-I CN= 76 S= 3.16 Q(10-24)= 0.325 Q(25-24)= 0.509 Q(100-24)= 0.844 Q(100-6)= 0.368
Totals				89		

Table 2 Weighted Quantity of Runoff = Q x Fraction of Total Acres

AMC-I						AMC-					
		10-24 Storm		25-24 Storm				Storm		Storm	
CN	S	Q	Wtd. Q	Q	Wtd. Q	CN	S	Q	Wtd. Q	Q	Wtd. Q
89	1.24										
Totals						Totals					

STORM RUNOFF COMPUTATIONS:
(Use largest Q. Value from Tables 1 & 2)

	AMC-II 10-24	25-24	AMC-II 100-24	100-6
Quantity, inches runoff =	0.879	1.177	1.664	0.952
Volume, Acre-feet =	1.144	1.531	2.165	1.238
Cubic-feet =	49811.1	66898.2	94295.4	53947.9
Peak Discharge Cfs = Moderate CN 90	19.0	23.0	34.0	

Subject STORM RUNOFF CALCULATIONS

[redacted] POINT 3 - RUNOFF INTO LOWER REFUSE POND.

WELLINGTON COAL PREPARATION PLANT

CALCULATION NOTES

By M.O.A

Checked

Acc't

4 - 12 - 1948

Sheet No. 3 of ____ Sheets

Climatic Condition = AMC-

Map Scale 1" = 2000'

Acres Per Sq. Inch =

AMC-

P = 1.82 (10-24) P = 2.74 (100-24)

P = 2.18 (25-24) P = 1.91 (100-6)

$$Q = \frac{(P-0.2S)^2}{P+0.8S}$$

Table I Area- Acre Conversion and Weighted Curve Numbers

Area	Sq. In.	Acres	Fraction of Total Acres	CN	Weighted CN	Runoff Calculation Data
Sn - C -	0.035	3.214	.12281	79	9.70	AMC-II CN= 81 S= 2.34 Q(10-24)= 0.495 Q(25-24)= 0.723 Q(100-24)= 1.119 Q(100-6)= 0.550
Ry - C -	0.175	16.070	.61404	81	49.73	AMC-I CN= 64 S= 5.62 Q(10-24)= 0.077 Q(25-24)= 0.167 Q(100-24)= 0.360 Q(100-6)= 0.096
Sn - C -	0.075	6.887	.26316	82	21.58	
Totals	0.285	26.171			81.01	

Table 2 Weighted Quantity of Runoff = Q x Fraction of Total Acres

CN	S	AMC-I 10-24 Storm Q 1.82	AMC-I 10-24 Storm Wtd. Q	AMC-I 25-24 Storm Q 2.18	AMC-I 25-24 Storm Wtd. Q	CN	S	AMC- 100-24 Storm Q 2.74	AMC- 100-24 Storm Wtd. Q	AMC- 100-6 Storm Q 1.91	AMC- 100-6 Storm Wtd. Q
79	2.66	.420	.052	.630	.077			1.001	.123	0.470	.058
81	2.34	.495	.304	.723	.444			1.119	.687	0.550	.338
82	2.20	.532	.140	.768	.202			1.176	.309	0.588	.155
Totals			0.496		0.723	Totals			1.119		0.551

STORM RUNOFF COMPUTATIONS:
(Use largest Q. Value from Tables 1 & 2)

	AMC-II 10-24	25-24	AMC-II 100-24	100-6
Quantity, inches runoff =	0.496	0.723	1.119	0.551
Volume, Acre-feet =	1.082	1.577	2.440	1.202
Cubic-feet =	47120.4	68685.5	106305.8	52345.4
Peak Discharge Cfs = Moderate CN 80	10.6	14.0	22.2	

Subject STORM RUNOFF CALCULATIONS

REACH A Lower Refuse Pond.

WELLINGTON COAL PREPARATION PLANT

CALCULATION NOTES

By M.O.A.

Checked

Acc't

4 - 13 - 1983

Sheet No. 4 of Sheets

Climatic Condition = AMC-

Map Scale 1" = 2000'

Acres Per Sq. Inch = 91.827365

AMC-II

P = 1.82 (10-24) P = 2.74 (100-24)

P = 2.18 (25-24) P = 1.91 (100-6)

$$Q = \frac{(P-0.2S)^2}{P+0.8S}$$

Table I Area- Acre Conversion and Weighted Curve Numbers

Area	Sq. In.	Acres	Fraction of Total Acres	CN	Weighted CN	Runoff Calculation Data
Sn-C.	0.17	15.611	.07359	82	6.035	AMC-II CN= 83 S= 2.05 Q(10-24)= 0.575 Q(25-24)= 0.820 Q(100-24)= 1.239 Q(100-6)= 0.634
Sn-C	0.08	7.346	.03463	79	2.736	
Sn-D	0.65	59.688	.29139	84	23.636	
Sn-C	0.11	10.101	.04762	82	3.905	
Ry-C	0.235	21.579	.10173	76	7.732	
CeE2-D	1.065	97.796	.46104	84	38.727	AMC-I CN= 67 S= 4.92 Q(10-24)= 0.121 Q(25-24)= 0.234 Q(100-24) = 0.462 Q(100-6) = 0.147
Totals	2.310	212.1212			82.77	

Combine CN 82 x .121212

84 x .747424

Table 2 Weighted Quantity of Runoff = Q x Fraction of Total Acres

AMC-II						AMC-II					
		10-24 Storm		25-24 Storm				100-24 Storm		100-6 Storm	
CN	S	Q 1.82	Wtd. Q	Q 2.18	Wtd. Q	CN	S	Q 2.74	Wtd. Q	Q 1.91	Wtd. Q
82	2.20	.532	.064	.768	.093			1.176	.142	.539	.071
79	2.66	.420	.015	.630	.022			1.001	.035	.470	.016
84	1.90	.621	.461	.876	.650			1.307	.971	.682	.507
76	3.16	.325	.033	.509	.052			.844	.086	.368	.037
Totals			0.573		0.817	Totals			1.234		0.631

STORM RUNOFF COMPUTATIONS:
(Use largest Q. Value from Tables 1 & 2)

	AMC-II 10-24	AMC-II 25-24	AMC-II 100-24	AMC-II 100-6
Quantity, inches runoff =	0.575	0.820	1.239	0.634
Volume, Acre-feet =	10.164	14.495	22.856	11.207
Cubic-feet =	442,750.0	631,400.0	995,609.9	488,180.0
Peak Discharge cfs. = Flat. CN 80-85 Int.	37.8			

Subject STORM RUNOFF CALCULATIONS

REACH *B*

WELLINGTON COAL PREPARATION PLANT

CALCULATION NOTES

By *N.O.A.*

Checked

Acc't

4-13-1983

Sheet No. *5* of Sheets

Climatic Condition = AMC-

Map Scale 1" =

Acres Per Sq. Inch =

AMC-

P = *1.82* (10-24) P = *2.74 (100-24)*

P = *2.18* (25-24) P = *1.91 (100-6)*

$$Q = \frac{(P-0.2S)^2}{P+0.8S}$$

Table I Area- Acre Conversion and Weighted Curve Numbers

Area	Sq. In.	Acres	Fraction of Total Acres	CN	Weighted CN
Ry C	0.67	61.524	.46207	77	35.6
Sn C	0.09	8.264	.06207	82	5.09
Sn D	0.05	4.591	.03443	84	2.90
Pd C2 - B	0.21	19.284	.14483	75	10.34
Ce E2 - D	0.40	36.731	.27586	84	23.17
Sn C	0.03	2.755	.02069	86	1.78
Totals	1.450	133.150			79.379

Runoff Calculation Data

AMC-II
CN= 79
S= 2.66
Q(10.24)= 0.420
Q(25.24)= 0.630
Q(100-24)= 1.001
Q(100-6)= 0.470

AMC-I
CN= 62
S= 6.13
Q(10-24)= 0.052
Q(25-24)= 0.128
Q(100-24)= 0.300
Q(100-6)= 0.069

84 = .31034

Table 2 Weighted Quantity of Runoff = Q x Fraction of Total Acres

AMC-I						AMC-					
CN	S	10-24 Storm Q 1.82	Wtd. Q	25-24 Storm Q 2.18	Wtd. Q	CN	S	100-24 Storm Q 2.74	Wtd. Q	100-6 Storm Q 1.91	Wtd. Q
77	2.99	.355	.164	.547	.253			.894	.413	.400	.185
82	2.20	.532	.033	.768	.048			1.176	.073	.539	.037
84	1.90	.621	.193	.876	.272			1.307	.406	.682	.212
75	3.33	.297	.043	.473	.069			.796	.115	.338	.049
86	1.63	.714	.015	.987	.020			1.441	.030	.731	.016
Totals			0.448		0.662	Totals			1.037		0.499

STORM RUNOFF COMPUTATIONS:
(Use largest Q. Value from Tables 1 & 2)

	AMC-II 10-24	25-24	AMC-II 100-24	100-6
Quantity, inches runoff =	0.448	0.662	1.037	0.499
Volume, Acre-feet =	4.971	7.345	11.506	5.537
Cubic-feet =	216,533.9	319967.4	501217.9	241193.9
Peak Discharge cfs = Moderate CN80 ES-1027 #12	31.5	41.0	78.0	

CALCULATION NOTES

Subject *UNIT HYDROGRAPH*
Time of Concentration - Runoff
REACH A
" B

By *MOA.*
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Acc't
4-13 19*83*
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TABLE $T_t = \frac{L}{3600\,V}$ $T_c = \sum T_t$ (increments)

Contour Elev.	Elev. Diff H.	Incremental Reach Length-L-	Incremental Reach Grade	Fig. 15.2 * Velocity	Tt Hours	Remarks
						* nearly bare-untilled
5640						
	100	1620	.0617	2.50	0.18	
5540						
	100	2520	.0397	2.00	0.35	
5440						
	67	2620	.0256	1.61	0.45	
5373						
					0.982	$\sum T_t = T_c$
	267	6800	.03926	1.98	0.954	Tc over [illegible]
REACH B						
5620						
	60	1070	.0561	2.33	.125	
						.375
5560						
	120	2140	.0561	2.33	.249	
5440						
	68	1610	.0422	2.08	.215	
5372					0.510	$\sum T_t = T_c$
	248	4820	.05145	2.29	0.5847	Tc over water course

CALCULATION NOTES

Subject DESIGN STORMS
UNIT HYDROGRAPH CALCULATIONS
REACH - A
STORM DURATION = 24 HOURS

By M.O.A.
Checked
Acc't
4-18-1983
Sheet No. 7 of ___ Sheets

DATA:

Design Storm, 10 year, 24 Hours, Precipitation = 1.82 inches.
Drainage Area, A = 212.1212 Acres, .3314 Square Miles.
Reach A, Hydraulic Length, 6800 feet.
Time of Concentration, T_c = 0.982 hours. (Computations page 6)
CN = 83 Weighted Moisture Condition = AMC - II
Quantity, Q = 1 inch for unit Hydrograph.
Design Storm(s) Runoff = (10 - 24) = 1.82 inches Q = 0.575"
(25 - 24) = 2.18 " Q = 0.820"
(100 - 24) = 2.74 " Q = 1.239"

Formulas: (References - NEH, Section 4, Hydrology)

(1) $T_p = 0.5(D) + 0.6(T_c)$ (Eq. 17-45) Pg. 11.84

(2) $q_p = \frac{484\,AQ}{T_p}$

T_p = Time to peak hours.
D = Storm Duration, Total
T_c = Time of Concentration, (See Data above)
A = Area Sq. miles
645.33 = the rate to discharge one inch from one square mile in one hour

Volume under unit Hydrograph
(3) V = 645.33 × A = cfs-hours (pg. 16.10)

CALCULATIONS:

V = 645.33 × .3314 = 213.862 cfs-hrs
× .08264 = 17.674 A.F

(1) $T_p = 0.5 \times 24 + 0.6 \times .982$
12 + 0.5892 = 12.589 hours

$T_r = 1.67\,T_p$ = 21.024 hours

$T_b = T_p + T_r$ = 12.589 + 21.024 = 33.613 hours

(2) $q_p = \frac{484 \times .3314 \times 1}{12.589}$ = 12.741 cfs.

Design Storms Peak Discharge
10-24 Q = 0.575 × 12.741 = 7.326 cfs
25-24 Q = 0.820 × " = 10.448 cfs.
... = 15.786 cfs

CALCULATION NOTES

Subject DESIGN STORMS Cont.
REACH A
STORM DURATION = 24 Hrs

By M.O.A.
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Reference Chapter 16, Pages 16.7 & 16.8

Formulas:

$Lag = 0.6\,T_c$ (Eq. 15.3, page 15.6)

$T_p = \frac{\Delta D}{2} + L$ (Eq. 16.7)

Point of inflection, $P.I. = 1.7\,T_p$

$\Delta D = 0.2\,T_p$ (Page 16.8)

$T_c + \Delta D = 1.7 T_p$ (Eq. 16.10, Pg. 16.8)

$\frac{\Delta D}{2} + .6\,T_c = T_p$ (Eq. 16.11, Pg. 16.8)

$\Delta D = 0.133\,T_c$ (Eq. 16.12, Pg. 16.8)

CALCULATIONS:

$T_p = 12.5892$ Hours.

$P.I. = 1.7 \times 12.5892 = 21.40164$ hrs.

$\Delta D = 0.2 \times 12.5892 = 2.51784$ hrs.

$T_c + \Delta D = 1.7\,T_p$.

$\therefore T_c = 1.7 T_p - \Delta D = 21.40164 - 2.51784 = 18.8838$ hrs.

$\frac{\Delta D}{2} + 0.6\,T_c = T_p \quad \therefore T_p = \frac{2.51784}{2} + .6 \times 18.8838 = 12.5892$

$\Delta D = .133 \times 18.8838 = 2.5115 < 2.51784$ but a good check

$Lag = 0.6\,T_c = .6 \times 18.8839 = 11.3303$ hrs.

$\Delta D = 2.51784 \quad \frac{\Delta D}{2} = 1.25892 = 1.26$

CALCULATION NOTES

Subject *UNIT HYDROGRAPH*
REACH A
24 hour storm Duration

By *M.O.A.*
Checked
Acc't
4-18-1983
Sheet No. *9* of ____ Sheets

Table — Computation of coordinates for unit hydrograph

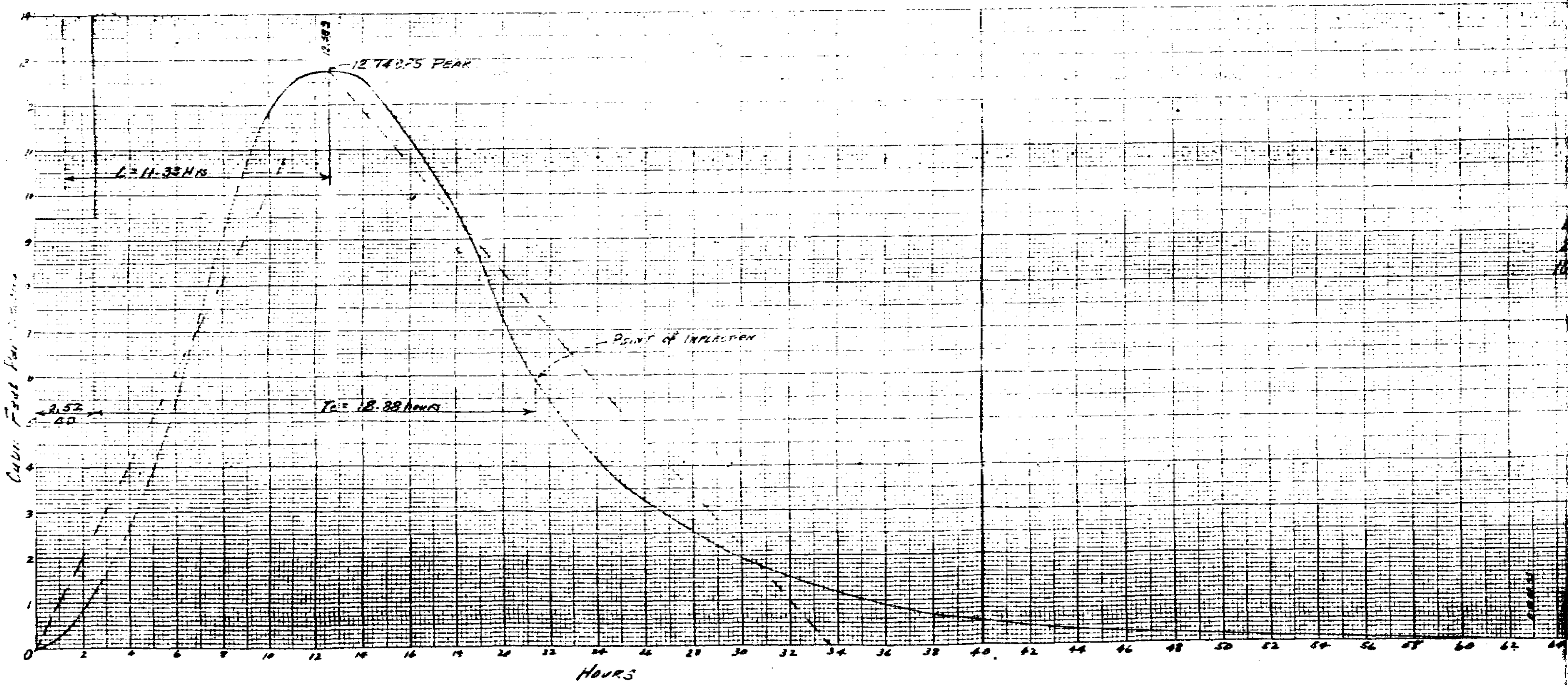
1	2	3	4
Time Ratios (table 16.1)	Time (col 1 x *Tp* *12.589*)	Discharge Ratios (table 16.1)	Discharges (col 3 x *qp* *12.741*)
(t/T_p)	(hours)	(q/q_p)	(cfs)
.0	*0*	0	
.1	*1.26*	.030	*0.38*
.2	*2.52*	.100	*1.27*
.3	*3.78*	.190	*2.42*
.4	*5.04*	.310	*3.95*
.5	*6.29*	.470	*5.99*
.6	*7.55*	.660	*8.41*
.7	*8.81*	.820	*10.45*
.8	*10.07*	.930	*11.85*
.9	*11.33*	.990	*12.61*
1.0	*12.59*	1.000	*12.74*
1.1	*13.85*	.990	*12.61*
1.2	*15.11*	.930	*11.85*
1.3	*16.37*	.860	*10.96*
1.4	*17.62*	.780	*9.94*
1.5	*18.88*	.680	*8.66*
1.6	*20.14*	.560	*7.13*
1.7	*21.40*	.460	*5.86*
1.8	*22.66*	.390	*4.97*
1.9	*23.92*	.330	*4.20*
2.0	*25.18*	.280	*3.56*
2.2	*27.70*	.207	*2.64*
2.4	*30.21*	.147	*1.87*
2.6	*32.73*	.107	*1.36*
2.8	*35.25*	.077	*0.98*
3.0	*37.77*	.055	*0.70*
3.2	*40.28*	.040	*0.51*
3.4	*42.80*	.029	*0.37*
3.6	*45.32*	.021	*0.27*
3.8	*47.34*	.015	*0.19*
4.0	*50.36*	.011	*0.14*
4.5	*56.65*	.005	*0.06*
5.0	*62.95*	0	*0*

See Unit Hydrograph "Reach-A- Lower Refuse Pond" Pg. 9A,
24 Hour Storm Duration
AMC-II - 1.0" R.O. CN 83.

REACH A

LOWER REFUSE POND
UNIT HYDROGRAPH
24 HOUR STORM DURATION
AREA = 212.12 ACRES
0.3314 SQ MILES
AMC-II ~ 1.0" R.O., CN 83
4-19-83 M.O.A.

STORM	P.M.P.	RUNOFF	PEAK FLOW
10 Yr. 24 Hr.	1.82"	0.575"	7.326 CFS.
25 Yr. 24 Hr.	2.18"	0.820"	10.443 CFS.
100 Yr. 24 Hr.	2.74"	1.239"	15.786 CFS.



Subject *UNIT HYDROGRAPH ANALYSIS*
SUMMATION of Ordinates
REACH A
STORM DURATION = 24 HOUR

CALCULATION NOTES
By *M.O.A.*
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Acc't
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TIME INCRE. Hrs	CSF Ordinate Unit	CSF 10-24	CSF 25-24	CSF 100-24	Time Increment Hrs	CSF Ordinate Unit	CSF 10-24	CSF 25-24	CSF 100-24
Q →	1"	0.575"	0.820"	1.239"	31	1.71	0.98	1.40	2.12
0	0	0	0	0	32	1.51	0.87	1.24	1.87
1	0.25	.14	.21	.31	33	1.33	0.76	1.09	1.65
2	0.83	.48	.68	1.03	34	1.17	0.67	.96	1.45
3	1.64	.94	1.34	2.03	35	1.03	0.59	.84	1.28
4	2.65	1.52	2.17	3.28	36	0.90	0.52	.74	1.12
5	3.92	2.25	3.21	4.86	37	0.78	0.45	.64	0.97
6	5.41	3.11	4.43	6.69	38	0.68	0.39	.56	.84
7	7.30	4.20	5.99	9.04	39	0.60	0.35	.49	.74
8	9.20	5.29	7.54	11.40	40	0.53	0.30	.43	.66
9	10.70	6.15	8.77	13.26	41	0.47	0.27	.39	.59
[illegible]	11.82	6.80	9.69	14.64	42	0.41	0.24	.34	.51
11	12.48	7.18	10.23	15.46	43	0.36	0.21	.30	.45
12	12.72	7.31	10.43	15.76	44	0.32	0.18	.26	.40
13	12.72	7.31	10.43	15.76	45	0.29	[illegible]	.24	.36
14	12.54	7.21	10.29	15.55	46	[illegible]	0.15	.21	.32
15	11.92	6.85	9.77	14.77	47	0.23	0.15	.19	.29
16	11.21	6.45	9.19	13.89	48	0.21	0.12	.17	.26
17	10.44	6.01	8.56	12.94	49	0.18	0.10	.15	.22
18	9.60	5.52	7.87	11.89	50	0.16	0.09	.13	.20
19	8.52	4.90	6.99	10.56	51	0.14	0.08	.11	.17
20	7.25	4.17	5.95	8.92	52	0.13	0.07	.11	.16
21	6.21	3.57	5.09	7.69	53	0.11	0.06	.09	.14
22	5.44	3.13	4.46	6.74	54	0.10	0.06	.08	.12
23	4.77	2.74	3.91	5.91	55	0.08	0.05	.07	.10
24	4.15	2.39	3.40	5.14	56	0.07	0.04	.06	.09
25	3.63	2.09	2.98	4.50	57	0.06	0.03	.05	.07
26	3.23	1.86	2.65	4.00	58	0.05	0.03	.04	.06
[illegible]	2.87	1.60	2.28	3.44	59	0.04	0.02	.03	.05
28	2.52	1.45	2.07	3.12	60	0.03	0.02	.02	.04
29	2.21	1.27	1.81	2.74	61	0.02	0.01	.01	.02
30	1.92	1.10	1.57	2.38	62	0.01	0.01	.01	.01
TOTAL	200.06					214.03			

CALCULATION NOTES

Subject DESIGN STORMS

UNIT HYDROGRAPH CALCULATIONS

REACH B

STORM DURATION = 24 Hours

By M.O.A.

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Acc't

4-19-1983

Sheet No. 15 of ____ Sheets

DATA:

Design Storm, 10 year, 24 Hours, Precipitation = 1.82 inches.

Drainage Area, A = 133.150 Acres, 0.2080 Square Miles

Reach B, Hydraulic Length, 4820 feet.

Time of Concentration, Tc = 0.59 hours. (Computations page)

CN = 79 Weighted Moisture Condition = AMC - II

Quantity, Q = 1 inch for unit Hydrograph.

Design Storm(s) Runoff = (10 - 24) = 1.82"

(25 - 24) = 2.18"

(100 - 24) = 2.74"

Formulas: (References - NEH, Section 4, Hydrology)

(1) $T_p = 0.5(D) + 0.6(T_c)$ (Eq. 17-45) Pg. 17.84

(2) $q_p = \frac{484\,AQ}{T_p}$

T_p = Time to peak hours.

D = Storm Duration, Total

T_c = Time of Concentration, (see Data above)

A = Area Sq. miles

645.33 = the rate to discharge one inch from one square mile in one hour

Volume under unit Hydrograph

(3) $V = 645.33 \times A =$ cfs-hours (pg. 16.10)

CALCULATIONS:

(1) $T_p = .5 \times 24 + .6 \times .590 =$

$12 + .345 = 12.345$ Hrs

$T_r = 1.67\,T_p = 1.67 \times\ = 20.384$ Hrs.

$T_b = T_p + T_r = 32.738$ Hrs.

(2) $q_p = \frac{484 \times .208 \times 1}{12.345} = 8.155$ cfs

(3) $V = 645.33 \times .208 = 134.2286$ cfs-Hrs.

$\times .08264 = 11.093$ A.F.

actual Storm peak flow

$q_p = 8.155 \times Q_{(storm)}$

$Q_{(10-24)} = .448,\ q_p = 3.653$

$Q_{(25-24)} = .662,\ q_p = 5.398$

$Q_{(100-24)} = 1.037,\ q_p = 8.457$

CALCULATION NOTES

Subject *DESIGN STORMS CONT.*
REACH B
STORM DURATION = 24 HOURS

By *M.O.A*
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Reference Chapter 16, Pages 16.7 & 16.8.

Formulas:

$Lag = 0.6\ T_c$ (Eq. 15.3, Page 15.6)

Point of Inflection, $P.I. = 1.7\ T_p$ (Pg. 16.7 & 16.3)

$\Delta D = 0.2\ T_p$ (Pg. 16.8)

$T_p = \frac{\Delta D}{2} + L$ (Eq. 16.7) $\therefore L = T_p - \frac{\Delta D}{2}$

$T_c + \Delta D = 1.7\ T_p$ (Eq. 16.10) Therefore $T_c = 1.7T_p - \Delta D$

$\frac{\Delta D}{2} + 0.6T_c = T_p$ (Eq. 16.11)

$\Delta D = 0.133\ T_c$ (Eq. 16.12)

Calculations: $T_p = 12.345$ Hrs.

$P.I. = 1.7T_p = 1.7 \times 12.345 = 20.9865$ hrs

$\Delta D = 0.2T_p = 0.2 \times 12.345 = 2.469$ hrs use

$L = T_p - \frac{\Delta D}{2} = 12.345 - \frac{2.469}{2} = 11.1105$ hrs

$T_c = 1.7T_p - \Delta D = 20.9865 - 2.496 = 18.5175$ hrs.

$T_p = \frac{\Delta D}{2} + .6T_c = \frac{2.469}{2}$ (1.2345) $+ .6 \times 18.5175 = 12.345$ hrs. check

$\Delta D = .133\ T_c = .133 \times 18.5175 = 2.4628$ check

$L = 0.6\ T_c = .6 \times 18.5175 = 11.1105$ hrs check.

$\Delta D = 2.47$ hrs $\frac{\Delta D}{2} = 1.235$ hrs

CALCULATION NOTES

Subject: *UNIT HYDROGRAPH*
REACH B
STORM DURATION – 24 HOURS

By *MOA*
Checked
Acc't
4-19-1983
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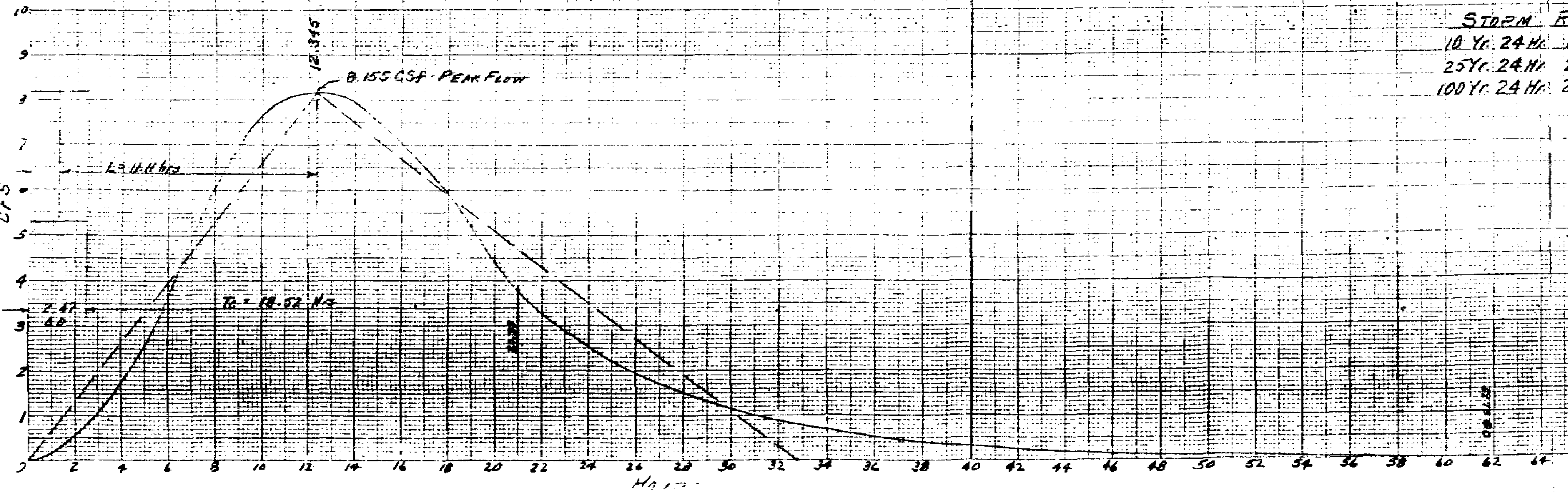
Table Computation of coordinates for unit hydrograph

1	2	3	4
Time Ratios (table 16.1)	Time (col 1 x *Tp*) *12.345 hr*	Discharge Ratios (table 16.1)	Discharges (col 3 x *qp*) *8.155 cfs*
(t/T_p)	(hours)	(q/q_p)	(cfs)
.0	*0*	0	
.1	*1.23*	.030	[illegible]
.2	[illegible]	.100	[illegible]
.3	[illegible]	.190	
.4	*4.94*	.310	
.5	[illegible]	.470	*3.83*
.6	[illegible]	.660	*5.38*
.7		.820	[illegible]
.8	[illegible]	.930	*7.58*
.9	[illegible]	.990	*8.07*
1.0	*12.35*	1.000	*8.16*
1.1	*13.58*	.990	*8.07*
1.2	[illegible]	.930	*7.58*
1.3	[illegible]	.860	*7.01*
1.4	[illegible]	.780	*6.36*
1.5	[illegible]	.680	*5.55*
1.6	[illegible]	.560	*4.57*
1.7	[illegible]	.460	*3.75*
1.8	[illegible]	.390	*3.18*
1.9	[illegible]	.330	*2.69*
2.0	[illegible]	.280	*2.28*
2.2	[illegible]	.207	*1.69*
2.4	[illegible]	.147	*1.20*
2.6	[illegible]	.107	[illegible]
2.8	[illegible]	.077	*0.63*
3.0	[illegible]	.055	*0.45*
3.2	[illegible]	.040	*0.33*
3.4	[illegible]	.029	*0.24*
3.6	*44.44*	.021	*0.17*
3.8	*46.91*	.015	*0.12*
4.0	*49.38*	.011	*0.09*
4.5	*55.55*	.005	*0.04*
5.0	*61.73*	0	*0*

See Unit Hydrograph "Reach-B – Lower Refuse Pond" Pg. 17A.
24 Hour Storm Duration
AMC-II – 1.0" R.O. CN-79

REACH-B-
LOWER REFUSE POND
UNIT HYDROGRAPH
24 HOUR STORM DURATION
AREA = 133.15 ACRES
0.2080 Sq. MILES
AMC-II ≈ 1.0" R.O. CN = 79
4-21-83 M.O.A.

STORM	RMP.	RUNOFF	PEAK FLOW
10 Yr. 24 Hr.	1.82"	0.448"	3.653 CFS.
25 Yr. 24 Hr.	2.18"	0.662"	5.358 CFS.
100 Yr. 24 Hr.	2.74"	1.037"	8.457 CFS.



CALCULATION NOTES

Subject *UNIT HYDROGRAPH ANALYSIS*
SUMMATION OF ORDINATES
REACH B
STORM DURATION: 24 HOURS

By *M.O.A.*
Checked
Acc't
4-21-1983
Sheet No. *18* of ____ Sheets

TIME INCREM. HOURS	CFS ORDINATE HYDROGRAPH	CFS 10-24	CFS 25-24	CFS 100-24	TIME INCREMENT HOURS	CFS ORDINATE HYDROGRAPH	CFS 10-24	CFS 25-24	CFS 100-24
Q →	UNIT	0.448	0.662	1.037	Q =	UNIT	0.448	0.662	1.037
1	.16	.07	.11	.17	31	1.0	.45	.66	1.04
2	.56	.25	.37	.58	32	0.88	.39	.58	.91
3	1.08	.48	.71	1.12	33	0.77	.34	.51	.80
4	1.73	.78	1.15	1.79	34	0.68	.30	.45	.71
5	2.55	1.14	1.61	2.64	35	0.60	.27	.40	.62
6	3.64	1.63	2.41	3.77	36	0.52	.23	.34	.54
7	4.90	2.20	3.24	5.08	37	0.46	.21	.30	.48
8	6.04	2.71	4.00	6.26	38	0.40	.18	.26	.41
9	6.97	3.12	4.61	7.23	39	0.35	.16	.23	.36
10	7.63	3.42	5.05	7.91	40	0.31	.14	.21	.32
11	8.02	3.59	5.31	8.32	41	0.28	.13	.19	.29
12	8.14	3.65	5.39	8.44	42	0.245	.11	.16	.25
13	8.13	3.64	5.38	8.43	43	0.215	.10	.14	.22
14	7.93	[illegible]	5.25	8.22	44	0.18	.08	.12	.19
15	7.52	3.37	4.98	7.80	45	0.16	.07	.11	.17
16	7.02	3.14	4.65	7.28	46	0.14	.06	.09	.15
17	6.50	[illegible]	4.30	6.74	47	0.13	.06	.09	.13
18	5.91	2.65	3.91	6.13	48	0.115	.05	.08	.12
19	5.18	2.32	3.43	5.37	49	0.10	.0448	.0662	.1037
20	4.40	[illegible]	2.91	4.56	50	0.09	.04	.06	.09
21	3.75	1.68	2.48	3.89	51	0.08	.04	.05	.08
22	3.26	1.46	2.16	3.38	52	0.07	.03	.05	.07
23	2.88	1.29	1.91	2.99	53	0.06	.03	.04	.06
24	2.52	1.13	1.67	2.61	54	0.05	.02	.03	.05
25	2.21	0.99	1.46	2.29	55	0.04	.02	.03	.04
26	1.94	0.87	1.28	2.01	56	0.035	.02	.02	.04
27	1.71	0.77	1.13	1.77	57	0.03	.01	.02	.03
28	1.50	0.67	0.99	1.56	58	0.02	.01	.01	.02
29	1.31	0.58	.87	1.36	59	0.015	.01	.01	.02
30	1.14	0.51	.75	1.18	60	0.01	.00	.01	.01
					61	0.005	.00	.00	.01

Subject Unit Hydrograph Analysis
Summation of Ordinates for Design Storms
Reach A+B 24 hour Storms

Lower Refuse Ponds

CALCULATION NOTES
By MOA
Checked
Acc't
4-26 1983
Sheet No. of Sheets

Time Increment hours	10 yr 24 hr cfs	25 yr 24 hr cfs	100 yr 24 hr cfs				
0	0	0	0				
2	0.73	1.05	1.61				
4	2.30	3.32	5.07				
6	4.74	6.84	10.46				
8	8.00	11.54	17.66				
10	10.22	14.74	22.55				
12	10.96	15.82	24.20				
14	10.76	15.54	23.77				
16	9.59	13.84	21.17				
18	8.17	11.78	18.02				
20	6.14	8.86	13.48				
22	4.59	6.62	10.12				
24	3.52	5.07	7.75				
26	2.73	3.93	6.01				
28	2.12	3.06	4.68				
30	1.61	2.32	3.56				
32	1.26	1.82	2.78				
34	1.12	1.41	2.16				
36	0.86	1.08	1.66				
38	0.65	0.82	1.25				
40	0.51	0.64	0.98				
42	0.40	0.50	0.76				
44	0.30	0.38	0.59				
46	0.24	0.30	0.47				
48	0.20	0.25	0.38				
50	0.15	0.19	0.29				
52	0.12	0.16	0.23				
54	0.09	0.11	0.17				
56	0.06	0.08	0.13				
58	0.04	0.05	0.08				
60	0.03	0.03	0.05				
62	0.01	0.01	0.01				

CALCULATION NOTES

Subject Clear Water Pond

Data and Calculations:

Wellington Coal Cleaning Plant

By MOA

Checked

Acc't

Date May 12 1983

Sheet No. 1 of 1 Sheets

DATA:

There are no Reaches flowing into this pond. The surface runoff is so small it will be negligible. Ignoring any surface runoff, except reaches, into this pond or any pond will have little effect. Plant circulation of 7.5752 cfs is included with no time break. However, the plant operates only 5 days per week maximum and only 8 hours per day. When the plant operates 4000 gpm is pumped from the Clear Water Pond. This draw down is not being considered. We have been over conservative in the storm inflow handling capabilities for all 24 hour storms and the 6 hour storms.

AREA

Clear Water Pond = 12.287 acres

Discharge Pipe - (Overflow)

A 36" concrete horizontal discharge pipe, at 1.0% slope extends through the dam at 5 feet from dam crest to flow line of pipe. It changes to a 24" dia. conc. pipe down face of 2:1 slope of dam to a ditch, outby the toe of the dam, to the Price River.

Plant Circulation

During Plant operation 4000 gpm water is pumped to the plant from the clear water pond of which 3400 gpm is recirculated to the pond with 1½ x 0 refuse. This circulation stops when plant is idle.

Rainfall into Pond = 12.287 acres x rainfall ÷ 12 = Ac. ft.

Design Storms	10-24(1.82")	25-24(2.18")	100-24(2.74")	100-6(1.91")
Acre-feet	1.863	2.232	2.806	1.956

Subject Permanent Diversion
Wellington Coal Cleaning Plant
Upper Refuse Pond

Notes on Calculations

CALCULATION NOTES
By
Checked
Acc't
19
Sheet No. of Sheets

The proposed permanent diversion will divert runoff from undisturbed areas in reaches 1 and 2 to the Siaperas ditch. The flow will then coarse through a culvert under the County road and on to the Price River. The following calculations demonstrate that the proposed diversion structure and the Siaperas ditch will handle the flow from the design storm (100 year 24 hour).

Reaches 3 thru 7 flow into the Siaperas ditch in addition to reaches 1 and 2 which will be diverted (refer to Map C9-1283). Peak flows were determined similar to the calculations for reaches 1 and 2. It was assumed for design purposes that all of the peak flows occurred simultaneously.

The existing culvert under the County road is too small to pass the flow from the design storm. The culvert is in poor condition and requires replacement. The County proposes to install a new culvert approximately 7 feet high by 10 feet wide at its base. The following calculations demonstrate that this culvert is adequate to pass the design flow.

CALCULATION NOTES

Subject *PEAK FLOW CALCULATIONS REACH'S 1, 2, 3, 4, 5a, 5b, 6, & 7. Determine Culvert Size for Siaperas Ditch at County Road (Refer to C9-1283)*

By *M. O. A.*

Checked

Acc't

4 - 29 - 19*83*

Sheet No. *1* of ___ Sheets

Reach	Q Inches R.O	100 year 24 Hour — 24 Hour Storm Data Tp. Hours	PEAK FLOW-cfs. Unit Hyd.	Runoff	Q Inches R.O.	100 year 6 Hour — 6 Hour Storm Data Tp Hours	PEAK FLOW CFS Unit Hyd.	Runoff
1	1.357	13.90	32.869	44.603	0.728	4.90	93.206	67.854
2	0.942	12.48	2.840	2.675	0.443	3.48	10.189	4.514
3	1.234	12.97	12.133	14.972	0.640	3.97	39.674	25.391
4	1.281	14.15	9.175	11.753	0.664	5.15	25.195	16.729
5a	1.240	13.19	12.454	15.443	0.635	4.19	39.129	24.847
5b	0.848	13.95	4.557	3.864	0.378	4.95	12.849	4.857
6	0.972	13.56	16.261	15.806	0.460	4.56	48.372	22.251
7	1.096	12.79	8.007	8.776	0.551	3.79	27.021	14.889
Total cfs.				117.892			295.635	181.332

CALCULATION NOTES

Subject *DIVERSION DITCH*
BY-PASS UPPER REFUSE POND
TO SIAPERAS DITCH
REACH 1 & 2 -
WELLINGTON COAL PREPARATION PLANT

By *M.O.A.*
Checked ______
Acc't ______
5-24-1983
Sheet No. *1* of ____ Sheets

DATA:

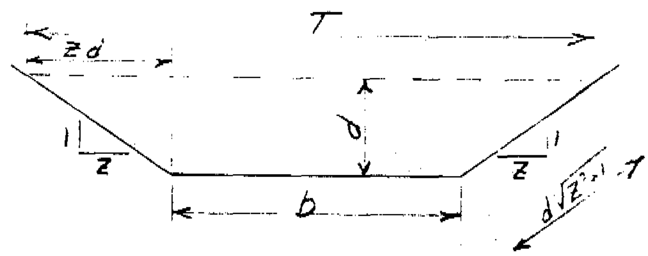
Peak Runoff From Design storms - cfs

DESIGN STORM	10 Yr. 24 Hr.	25 Yr. 24 Hr	100 Yr. 24 Hr	100 Yr. 6 Hr
Precipitation-inches	1.82"	2.18"	2.74"	1.91"
Reach 1, R.O.	21.858 Cfs.	30.338 Cfs	44.603 Cfs	67.861 Cfs
Reach 2, R.O.	1.119 "	1.684 "	2.675 "	4.514 "
TOTAL - Reach 1 + 2	22.977 Cfs.	32.022 Cfs	47.278 Cfs	72.375 Cfs

Use 75 Cfs for Design Storm flow.

Upstream Area = 20.0 Sq. ft (19.92 sq ft) Flow = 156.8 Cfs.
slope = 1.43 %

Ditch Design



Side Slopes, $Z = 1\frac{1}{2}$ to 1

base width, b = 10.0 ft.

depth, d = See Ditch Discharge Calculation sheet, attached.

$n = 0.025$ (earth straight & Uniform - Maximum)

Alternate $n = 0.0225$ and 0.028

Formulas:

Mannings: $V = \frac{1.486}{n} r^{.67} s^{.5}$ $Q = aV$

$a = bd + zd^2$

$P = b + 2d\sqrt{z^2+1}$

$r = \frac{a}{P}$

$T = b + 2zd$

$R = V^2\sqrt{a} + 40$

Critical Velocity

$V_c = \sqrt{g\,d_m}$

a = area, sq. ft.
V = Velocity, fps.
Q = Quantity, cfs.
n = roughness Coef.
r = hydraulic radius = $\frac{a}{P}$
s = Hyd. Grade, Ditch Slope
P = Wetted perimeter
T = Top width.

R = Minimum Radius of horizontal Curves.

Subject *DIVERSION DITCH*

CALCULATION NOTES

By *M.O.A*

Checked

Acc't

6-2-1983

Sheet No. *2* of ____ Sheets

Refer Page 3

DATA CONT'D:

Compute two Flows: 75 cfs.± and 156.8 cfs.±

Critical Velocities - $V_c = \sqrt{g d_m}$

upper ditch. $S = .008$ $d = 1.20$ $d_m = \frac{a}{T} = \frac{19.16}{10 + 2 \times 1.5 \times 1.2} = 1.04$

75± cfm

$V_c = \sqrt{32.2 \times 1.04}$

$V_c = 5.787 > 5.274$ ok. Vel. in Sub critical

$d = 1.85$ $d_m = \frac{a}{T} = \frac{23.633}{10 + 3 \times 1.85} = 1.52$

156.8 cfm

$V_c = \sqrt{32.2 \times 1.52}$

$V_c = 6.996 > 6.717$ ok

Lower ditch $S = .0025$

75± cfm $d = 1.70$ $d_m = \frac{21.835}{10 + 3 \times 1.70} = 1.41'$

$V_c = \sqrt{32.2 \times 1.41}$

$= 6.738 > 3.584$ - good.

156.8± cfm $d = 2.55$ $d_m = \frac{35.253}{10 + 3 \times 2.55} = 1.997$

$V_c = \sqrt{32.2 \times 1.997}$

$= 8.019 > 4.466$ = good.

NOTE: Velocities for both flow volumes 75± cfs and 256.8± cfs in both upper ditch, slope 0.8%, and lower ditch, slope 0.25%, are less than critical velocity to keep erosion to a minimum.

CALCULATION NOTES

Subject DIVERSION DITCH

By M.O.A

Checked

Acc't

6-2-1983

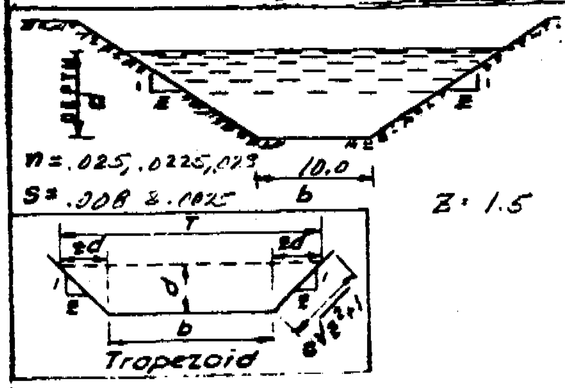
Sheet No. 3 of ___ Sheets

Upper Ditch - S = .008

Lower " - S = .0025

Q = 75 cfs & 156.8 cfs. ±

Ditch Discharge- Manning's Formula.



Manning's formula is one of the most widely accepted and commonly used of the open channel formulas.

$$v = \frac{1.486}{n} r^{2/3} s^{1/2}$$

v = mean velocity of flow in ft. per sec.
r = hydraulic radius in ft.
s = slope of the energy gradient.
n = coefficient of roughness. See back.

	n	Slope	Bottom Width	Depth	AREA	WETTED PERIMETER	HYDRAULIC RADIUS	VELOCITY	QUANTITY
	Coeff. of roughness	S feet/foot	b ft.	d ft.	$bd+zd^2$ a	$b+2d\sqrt{z^2+1}$ P	$\frac{bd+zd^2}{b+2d\sqrt{z^2+1}}$ r	$\frac{1.486}{n} r^{2/3} s^{1/2}$ feet Per Second	aV Cu. Ft./Second
	Upper Ditch								
use →	.025	.008	10.0	1.20	14.16	14.3266	0.9883	5.274	74.63
	.0225	"	"	"	"	"	"	5.860	82.99
	.028	"	"	1.25	14.34375	14.5096	1.0232	4.920	71.55
	.028	"	"	1.30	15.535	14.6872	1.05772	4.9[illegible]	74.57
use →	.025	.008	10.0	1.85	23.633	16.6703	1.4177	6.717	158.75
	.0225	"	"	"	"	"	"	7.463	176.[illegible]
	.028	"	"	1.95	25.20375	17.0303	1.480	6.172	155.57
	Lower Ditch								
use →	.025	.0025	10.0	1.70	21.335	16.1294	1.3227	3.584	76.48
	.0225	"	"	"	"	"	"	3.982	84.97
	.028	"	"	1.80	22.860	16.48999	1.3863	3.303	75.50
use →	.025	.0025	10.0	2.55	35.25375	19.1942	1.8367	4.466	157.46
	.0225	"	"	2.50	34.375	19.0139	1.8079	4.910	168.79
	.028	"	"	2.65	37.034	19.5547	1.8938	4.071	150.75
	.028	"	"	2.70	37.935	19.735	1.9222	4.111	155.96

CALCULATION NOTES

Subject DIVERSION DITCH DESIGN SUMMARY

By M.O.A.

Checked

Acc't

Date 6-2-1983

Sheet No. 4 of Sheets

Reference Dwg. E9-3427

DITCH = Trapezoidal Section - See Pg. 1.

Side Slopes - Ditch interior = 1.5 to 1 = 1.5:1 entire length of ditch.

Ditch embankment Crest = 10.0 wide, min. Tapers 10' to 18' wide @ channel.

Outer embankment Slopes = 2 to 1 = 2:1

UPPER DITCH Sta 0+00 to Sta 2+50 length = 250 ft.

bottom slope = 0.008 ft. per foot = 0.8%

bottom width = 10.0 ft.

depth of Ditch = 3.0 ft sta. 0+00 to Sta 1+12.5

varies from 3.0 @ 1+12.5 to 2.5 @ 2+12.5 to 0 @ 2+25±

Radius of Curve = 200 ft.

Riprap = 4" Min. Thickness (3 5/8" if Conc Block Used, see note)

Along outer side slope from sta. 0+00 to sta 1+20

Extend 4" Min. below bottom of ditch.

Riprap Weir – Sta. 2+00 to Sta 2+12.5 Sides and bottom.

4" Min. Thickness (5 5/8" if solid Concrete blocks used)

LOWER DITCH Sta. 5+17.8 to 9+32.8 Length = 415 ft.

bottom Slope = 0.0025 ft per foot = 0.25%

bottom width = 10.0

Depth of Ditch = 3.25 ft Sta. 5+17.8 to 7+70± Fill

plus Cut . Fill on west (low) side ditch only.

Sta. 7+70± to 8+90± = Cut up to 5.0+ ft. Daylight at sta. 9+32.8 in existing ditch.

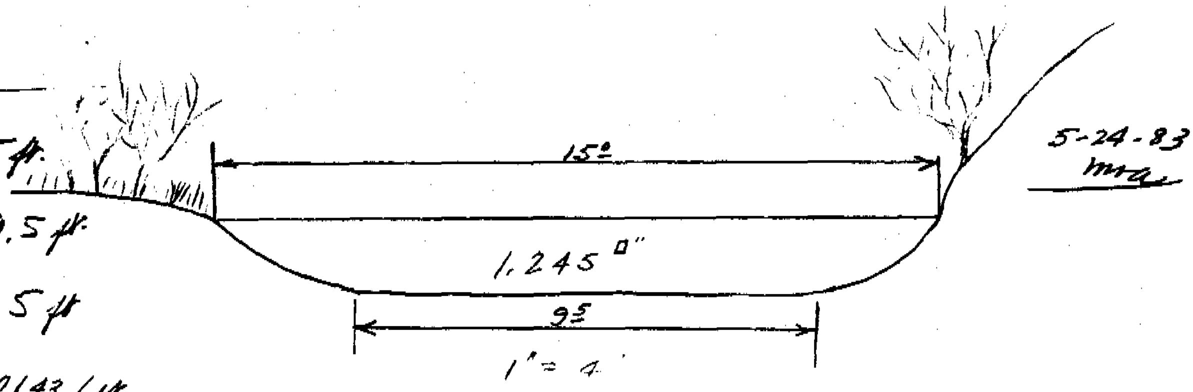
Radius of Curve = 130 ft.

Riprap Weir - Sta. 9+20 to 9+32. Sides & bottom. Same as above.

NOTE: Use of 8 x 16 x 4, Concrete block, hollow core suitable for Slope Riprap if cores are filled with concrete with #4 rebar on 48"

Reach 1

Top 15 ft.
bottom 9.5 ft.
depth 1.5 ft
slope = 0.0143/ft.



5-24-83
mra

$4^2 \times 1.245 = 19.92$ sq ft

A = 19.92
P = 16.0
$r = \frac{19.92}{16} = 1.245$
$S = \frac{1}{70} = .0143$
$n = 0.025$

$V = C\sqrt{rs}$

$$C = \frac{\frac{1.811}{n} + 41.66 + \frac{.00281}{s}}{1 + \frac{n}{\sqrt{r}}\left(41.66 + \frac{.00281}{s}\right)}$$

$$\left(41.66 + \frac{.00281}{.0143}\right) = 41.85650$$

$$C = \frac{\frac{1.811}{.025} + 41.85650}{1 + \frac{.025}{\sqrt{1.245}}\left(41.85650\right)} = \frac{114.2965}{1.93132} = 58.982$$

$$V = 58.932\sqrt{1.245 \times .0143} = 7.86996 = 7.87 \text{ ft/sec.}$$

$$Q = aV = 19.92 \times 7.87 = 156.76960 \text{ cfs.}$$

CALCULATION NOTES

Subject COUNTY ROAD CULVERT
100 year 24 Hour Storm
W.C.P. Plant.

By M.O.A.

Checked

Acc't

4-29-1983

Sheet No. 3 of ___ Sheets

DATA:

100 year 24 Hour Storm

Peak Flow from Area = 117.392 cfs ~ 120 cfs.

Slope = .005

Length = 75

From SCS Chart - n = .025

$Q = 120 \quad S = .005 \quad D = 66'' = 5'6$

Calculation

$$Q = \frac{.000614}{.025} \times .005^{.5} = .0017366 \times 66^{2.6667} = 123.56 \text{ cfs}$$

good.

From Corrug. Steel Pipe - Technical Manual - Mannings

$n = .027 \quad D = 5.5 \quad \text{Area} = 23.8 \quad P = \pi D = \pi \times 5.5 = 17.279$

$r = \frac{A}{P} = 1.377$

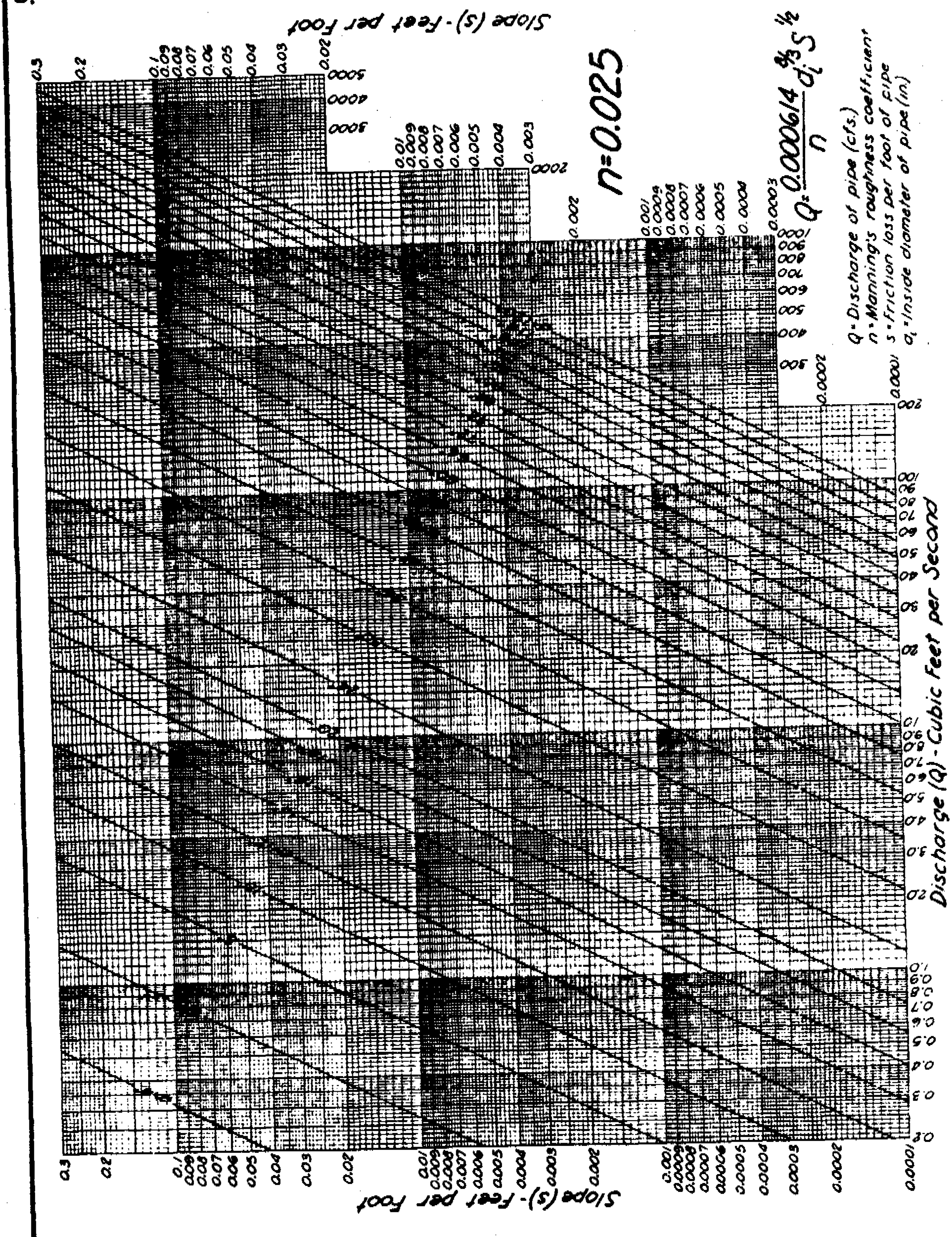
$$Q = \frac{1.486}{.027} \times .005^{.5} \times 1.377^{.6667} \times 23.8 = 114.64 < 120$$

$n = .025$

$$Q = \frac{1.486}{.025} \times .005^{.5} \times 1.377^{.6667} \times 23.8 = 123.814 \text{ cfs}$$

HYDRAULICS: DISCHARGE OF CIRCULAR PIPE FLOWING FULL

5.5-40



$n = 0.025$

$$Q = \frac{0.000614}{n} d_i^{8/3} S^{1/2}$$

Q = Discharge of pipe (cfs.)
n = Manning's roughness coefficient
s = Friction loss per foot of pipe
d_i = Inside diameter of pipe (in.)

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE H. H. Bennett, Chief ENGINEERING STANDARDS UNIT	STANDARD DWG. NO. ES-54 SHEET 4 OF 4 DATE 4-27-51

TABLE 5.4-1. VALUES OF ROUGHNESS COEFFICIENT, n

Type of Conduit and Description	Values of n Min.	Design	Max.	References
Pipe				
Cast-iron, coated	0.010	0.012 - 0.014	0.014	1
Cast-iron, uncoated	0.011	0.013 - 0.015	0.015	1
Wrought iron, galvanized	0.013	0.015 - 0.017	0.017	1
Wrought iron, black	0.012		0.015	1
Steel, riveted and spiral	0.013	0.015 - 0.017	0.017	1
Corrugated	0.021	0.025	0.0255	2
Wood stave	0.010	0.012 - 0.013	0.014	1
Neat cement surface	0.010		0.013	1
Concrete	0.010	0.012 - 0.017	0.017	1,6
Vitrified sewer pipe	0.010	0.013 - 0.015	0.017	1
Clay, common drainage tile	0.011	0.012 - 0.014	0.017	1
Lined Channels				
Metal, smooth semicircular	0.011		0.015	1,5
Metal, corrugated	0.0228	0.024	0.0244	2
Wood, planed	0.010	0.012	0.015	1,5
Wood, unplaned	0.011	0.013	0.015	1,5
Neat cement-lined	0.010		0.013	1,5
Concrete	0.012	0.014 - 0.016	0.018	1,5
Cement rubble	0.017		0.030	1,5
Vegetated, small channels, shallow depths				
Bermuda grass; long - 13", green	0.042			3
Long - 13", dormant	0.035		0.28	3
Short - 3", green	0.034			3
Short - 3", dormant	0.034			3
Sericea Lespedeza; long -16", green	0.076		0.22	3
Long - 16", dormant	0.050			3
Short - 2", green	0.033			3
Short - 2", dormant	0.034			3
Unlined Channels				
Earth; straight and uniform	0.017	0.0225	0.025	1
Dredged	0.025	0.0275	0.033	1
Winding and sluggish	0.0225	0.025	0.030	1
Stony bed, weeds on bank	0.025	0.035	0.040	1
Earth bottom, rubble sides	0.028	0.030 - 0.033	0.035	1
Unlined Channels—Continued				
Rock cuts; smooth and uniform	0.025	0.033	0.035	1
Jagged and irregular	0.035		0.045	1
Natural Streams				
(1) Clean, straight banks, full stage, no rifts or deep pools	0.025		0.033	1,4
(2) Same as (1) but more weeds and stones	0.030		0.040	1,4
(3) Winding, some pools and shoals, clean	0.033		0.045	1,4
(4) Same as (3), lower stages, more ineffective slopes and sections	0.040		0.055	1,4
(5) Same as (3), some weeds and stones	0.035		0.050	1,4
(6) Same as (4), stony sections	0.045		0.060	1,4
(7) Sluggish reaches, rather weedy, very deep pools	0.050		0.080	1,4
(8) Very weedy reaches	0.075		0.150	1,4

CALCULATION NOTES

Subject *Siaperas Ditch.*

By *M.O.A.*

Checked ____

Acc't ____

5-3-1983

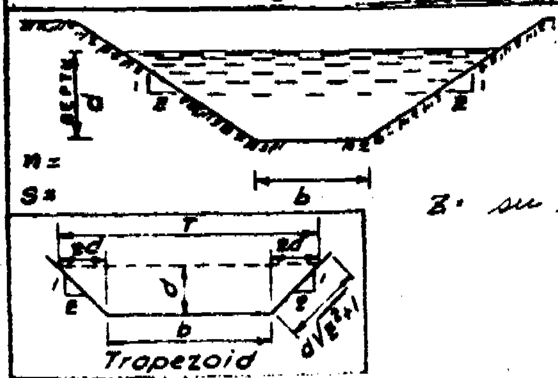
Sheet No. *1* of *5* Sheets

Peak Flow 100-24 = 117.9 - use 120 cfs

" " 100-6 = 181.4 cfs.

Sta 7+64 - Cross Sections from Topo Map.

Ditch Discharge- Manning's Formula.



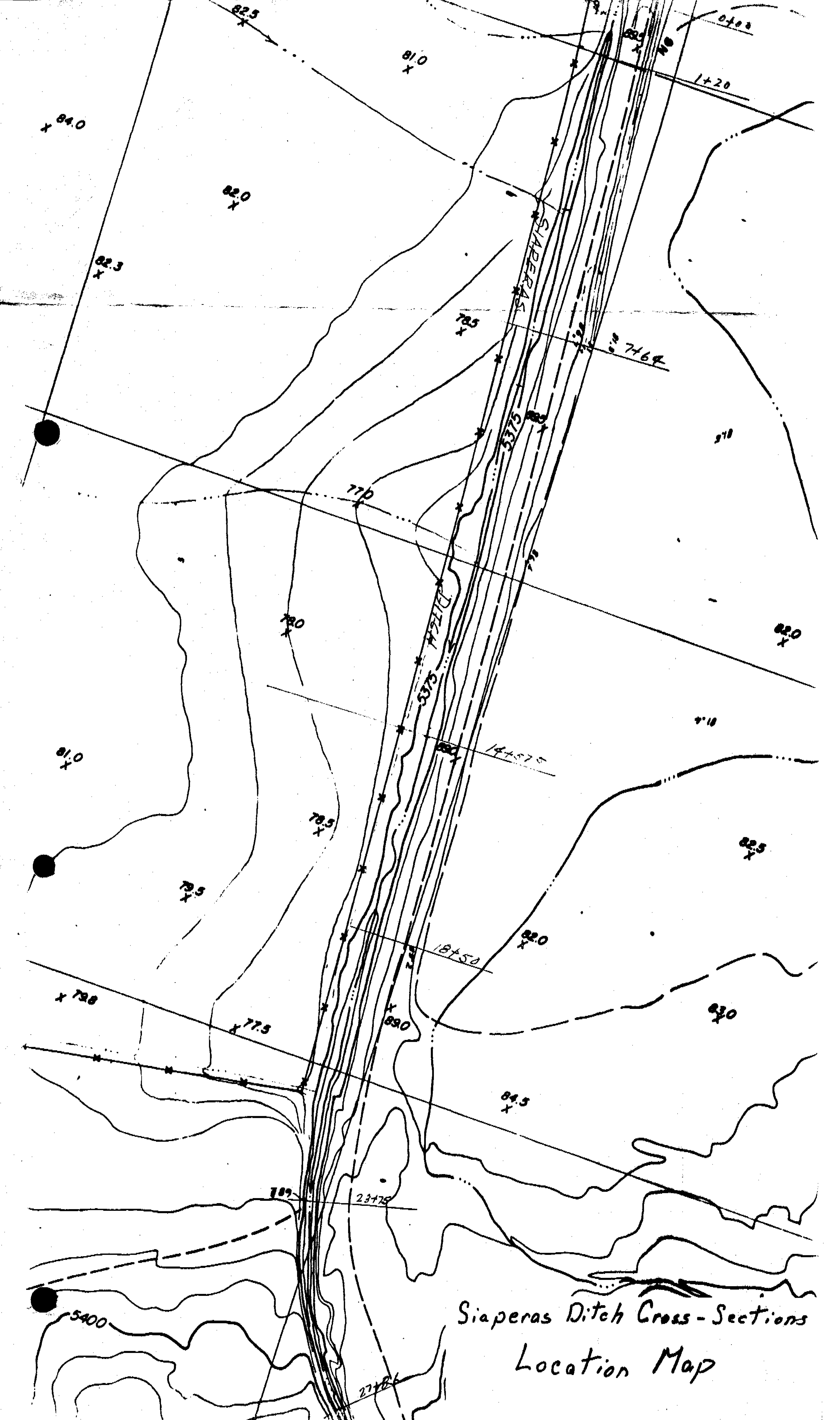
Z - see back.

Manning's formula is one of the most widely accepted and commonly used of the open channel formulas:

$$v = \frac{1.486}{n} r^{2/3} s^{1/2}$$

v = mean velocity of flow in ft. per sec.
r = hydraulic radius in ft.
s = slope of the energy gradient.
n = coefficient of roughness. *See back.*

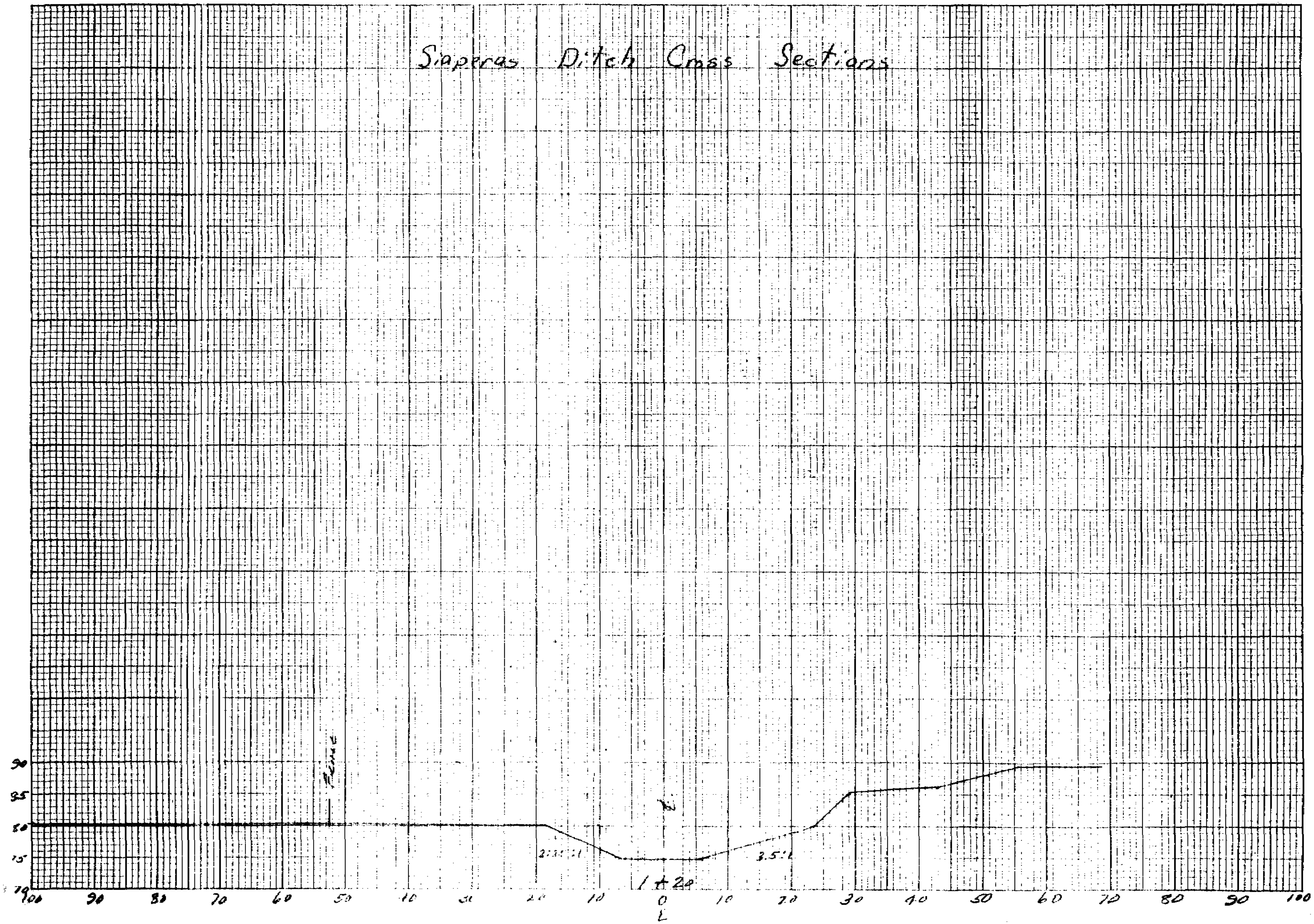
	n	Slope	Bottom Width	Depth	AREA	WETTED PERIMETER	HYDRAULIC RADIUS	VELOCITY	QUANTITY
	Coeff. of roughness	S feet/foot	b ft.	d ft.	$bd+zd^2$ a	$b+2d\sqrt{z^2+1}$ P	$\frac{bd+zd^2}{b+2d\sqrt{z^2+1}}$ r	$\frac{1.486}{n} r^{2/3} S^{1/2}$ Feet Per Second	aV Cu. Ft./Second
+64	.035	.0029	14	2	37.0	23.833	1.549	3.061	113.255
"	.035	.0029	14	3	68.0	40.048	1.698	3.254	221.288
"	.035	.0029	14	4	112.5	51.24[illegible]	2.195	3.862	434.447



Siaperas Ditch Cross-Sections

Location Map

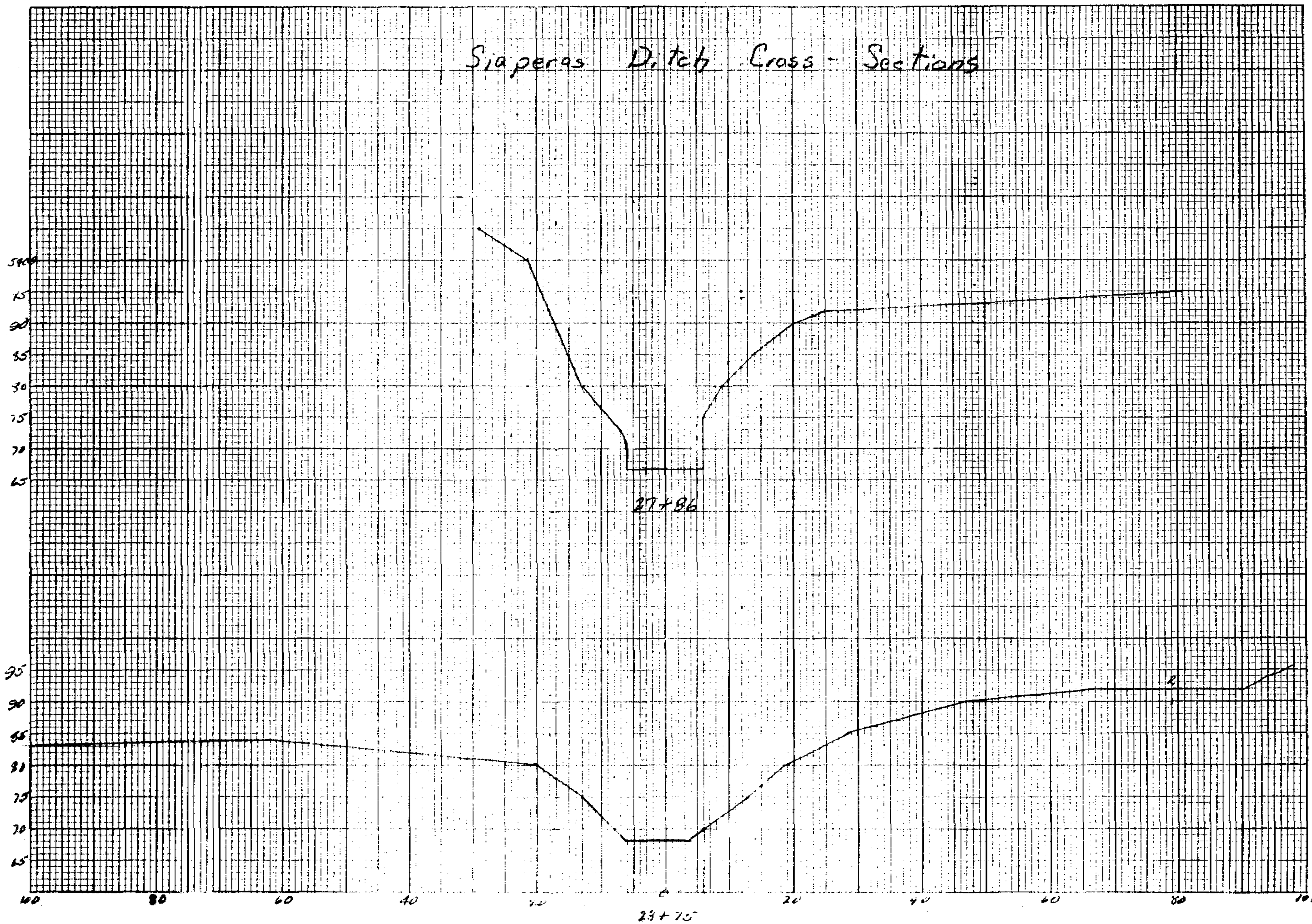
46 1322


Siaperas Ditch Cross Sections
90
85
80
75
70
Fence
3.5:1
1+20
100 90 80 70 60 50 40 30 20 10 0 10 20 30 40 50 60 70 80 90 100
L

5-2-83

K+E 10 X 10 TO ½ INCH 7 X 10 INCHES KEUFFEL & ESSER CO. MADE IN U.S.A. 46 1322

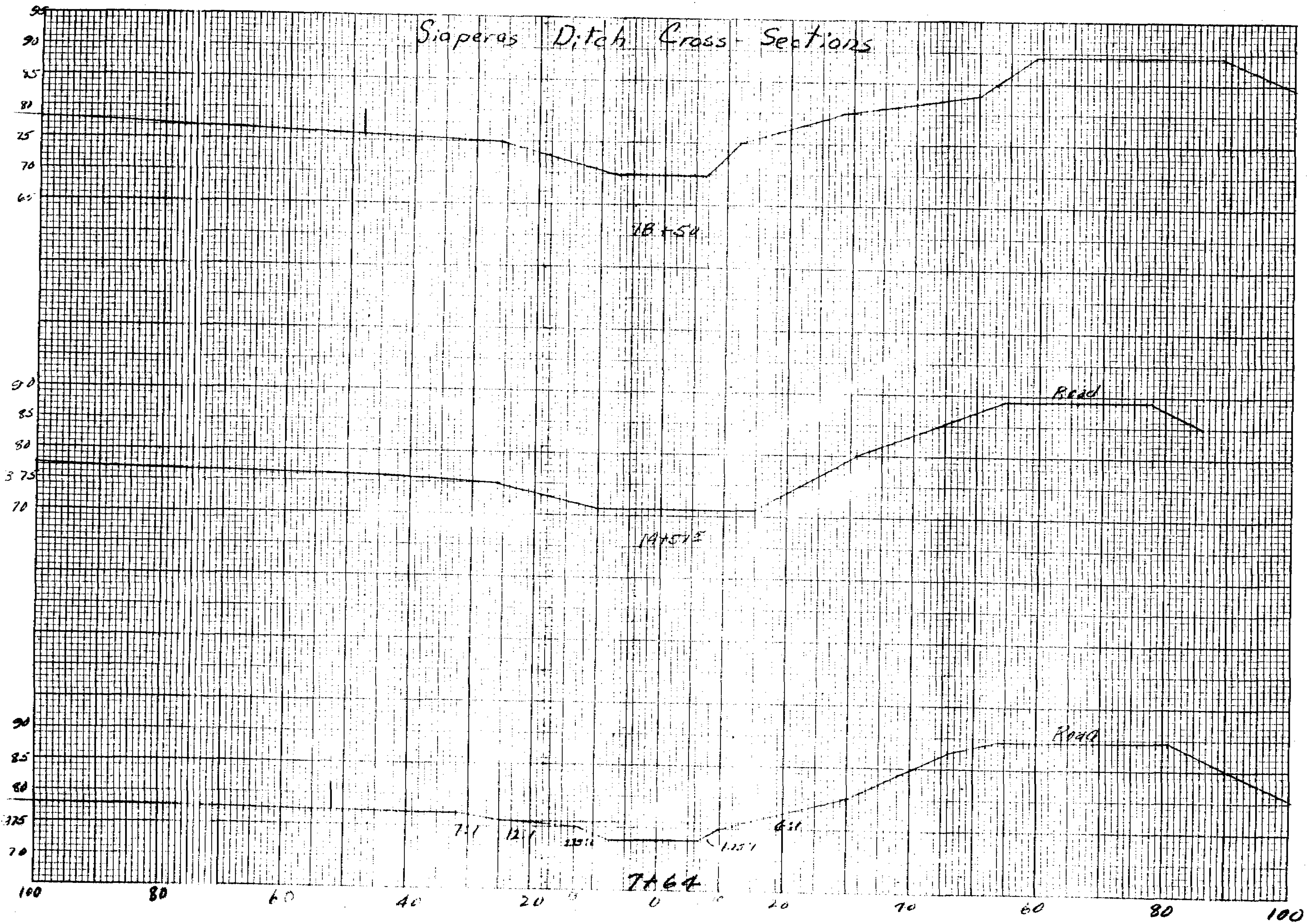
Siaperas Ditch Cross-Sections



5-2-83

K+E 10 X 10 TO 1/2 INCH 7 X 10 INCHES KEUFFEL & ESSER CO. MADE IN U.S.A.

46 1322



APPENDIX - D

WELLINGTON COAL PREPARATION PLANT
Wellington, Utah

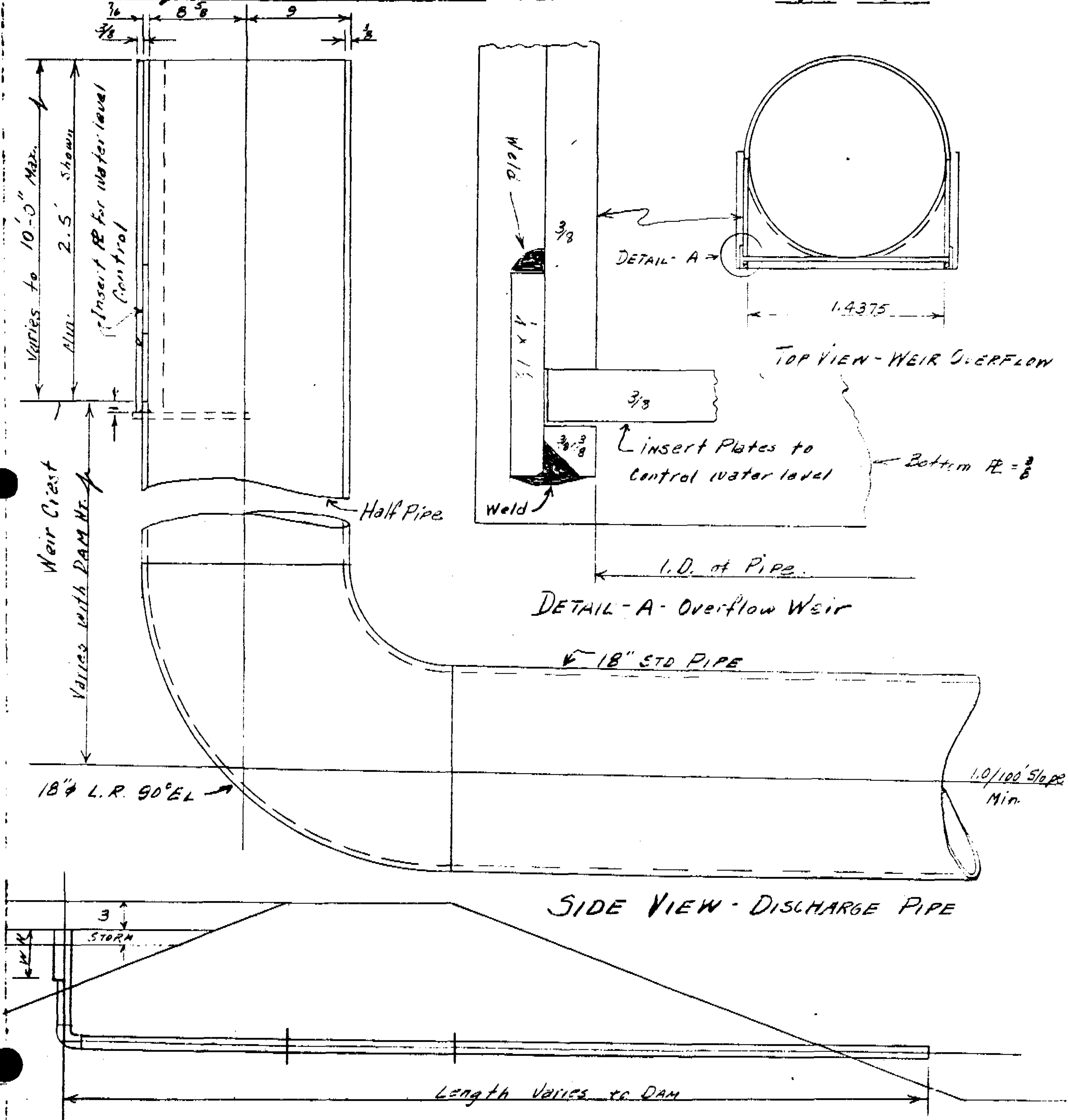
REFUSE PONDS

EXHIBITS AND SUPPORTING DATA

EXHIBIT-1

Subject TYPICAL DISCHARGE And PRIMARY OVERFLOW Rectangular Weir with End Contractions and adjustable Crest Height to Control Water Level

CALCULATION NOTES
By M.O.A.
Checked
Acc't
5 - 4 1983
Sheet No. 1 of 2 Sheets



SCHEMATIC OF OVERFLOW DISCHARGE WEIR and Pipe

CALCULATION NOTES

Subject: *Typical Discharge and Primary Overflow*

By: *M.O.A.*

Checked:

Acc't:

Date: *5-5-1983*

Sheet No. 2 of 2 Sheets

DISCHARGE FORMULAS:

WEIR FORMULA

Rectangular Weir with End Contractions:

$$Q = 3.33(L - 0.2H)H^{\frac{3}{2}} = \text{cfs.}$$

Q = Flow in cfs.

L = Length of weir in feet.

H = Height of water above weir crest in feet.

FLOW IN DISCHARGE PIPES

$$Q = a\sqrt{\frac{2gH}{1 + K_m + K_p L}} \qquad K_p = \frac{5087\, n^2}{d_i^{4/3}}$$

Q = Flow in cfs.

a = Area in cubic feet.

g = Acceleration of gravity = 32.2 ft/sec^2

H = Elevation Head differential feet.

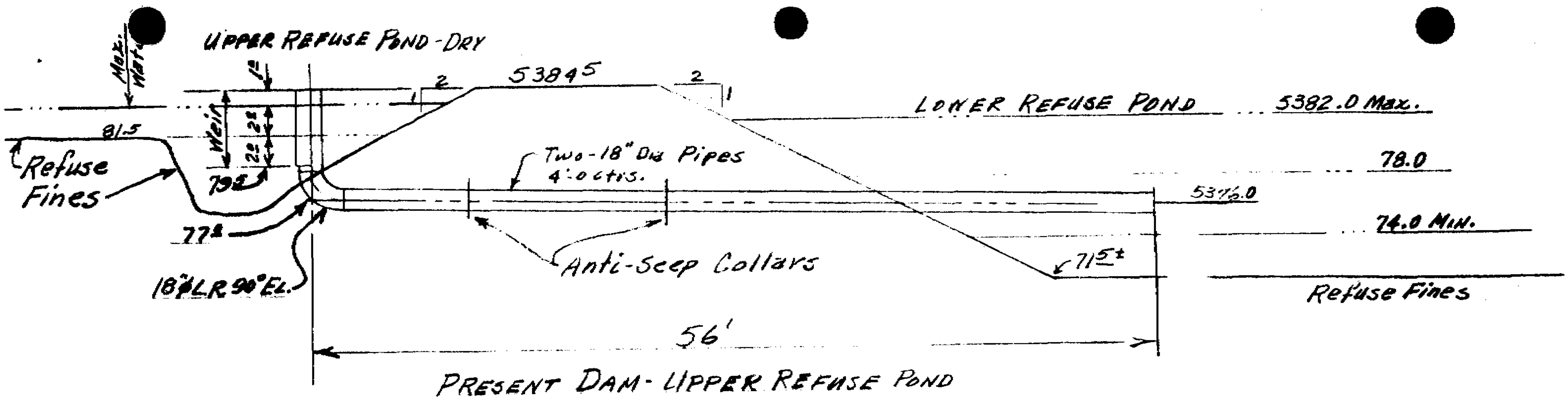
K_m = Coefficient of minor losses

K_p = Pipe friction coefficients

L = Pipe Length - feet.

d_i = Diameter of pipe in inches.

NOTE: Weir flow will control pipe flow until maximum capacity of pipe is reached. In most cases weir flow will reach a constant flow before pipe capacity is acheived.



Weir Flow $cfs = 3.3(L-.2H)H^{\frac{3}{2}}$

Height	ea. Pipe		Combined Pipes	
0.1	0.149	cfs.	0.398	cfs
0.5	1.575	"	3.149	"
1.0	4.121	"	8.241	"
1.5	6.959	"	13.917	"
2.0	9.772	"	19.543	"
2.5	12.340	"	24.680	"
3.0	14.491	"	28.982	"
3.5	16.081	"	32.162	"
4.0	16.983	"	33.965	"

Pipe Flow $Q = a\sqrt{\frac{2gH}{1+K_m+K_pL}}$

$a = 1.623$ sq ft. $K_m = 1$ $K_p = 0.0193$

$K_pL = 0.0193 \times 56 = 1.0808$

$Q = 1.623\sqrt{20.904H}$

$Q = (H = 7.7) = 20.591$ cfs. each pipe

$(H = 8.7) = 21.887$ "

$(H = 9.7) = 23.111$ "

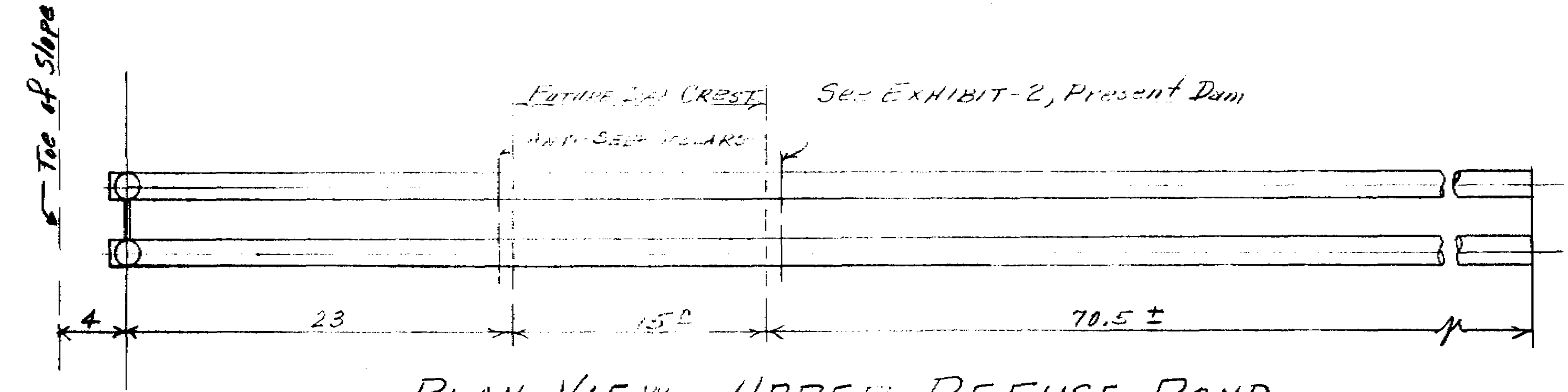
$(H = 10.7) = 24.273$ "

EXHIBIT-2

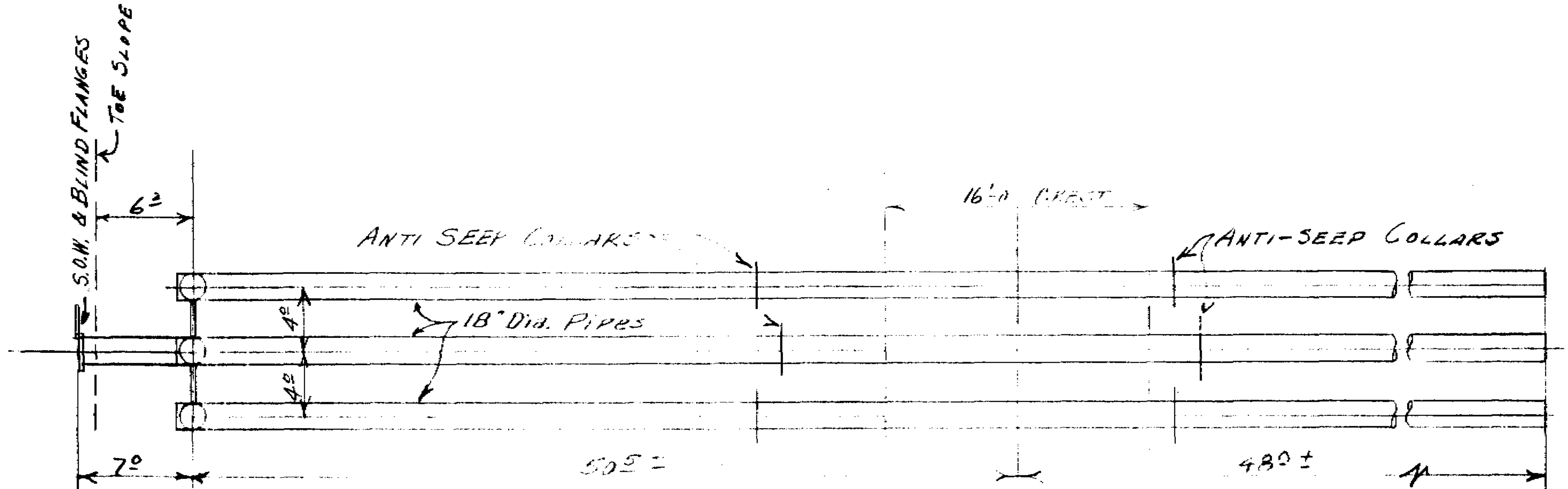
UPPER REFUSE POND

Two-18" Dia. Weir Type Overflow Pipes

1" = 10'   5-16-83   M.O.A.



PLAN VIEW - UPPER REFUSE POND
WEIR TYPE OVERFLOW PIPES - 18" DIA. PIPE
1"=10'



PLAN VIEW- LOWER [illegible] POND
WEIR TYPE OVERFLOW PIPES [illegible] FLOW THROUGH
END [illegible] CENTER PIPE [illegible]
1"=10'

PLAN VIEW - WEIR TYPE OVERFLOW PIPES
UPPER and LOWER REFUSE PONDS
M.O.A. 5-19-83
EXHIBIT - 3

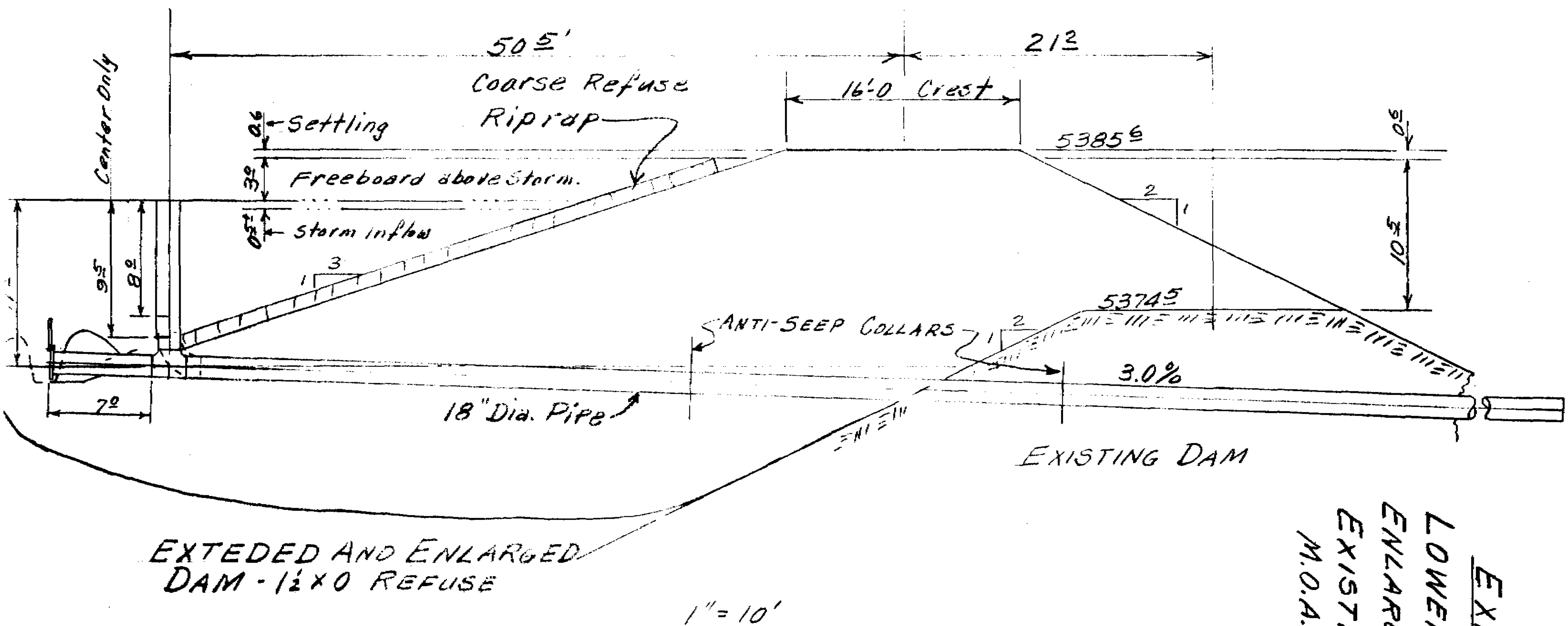


EXHIBIT - 4

LOWER REFUSE DAM
ENLARGEMENT ABOVE
EXISTING DAM

M.O.A. 5-19-83

EXHIBIT-5-
3 Pages

UTAH STATE UNIVERSITY
Logan, Utah

ESTIMATED RETURN PERIODS FOR SHORT-DURATION PRECIPITATION IN UTAH

E. Arlo Richardson
NOAA Climatologist
Utah State University, Logan

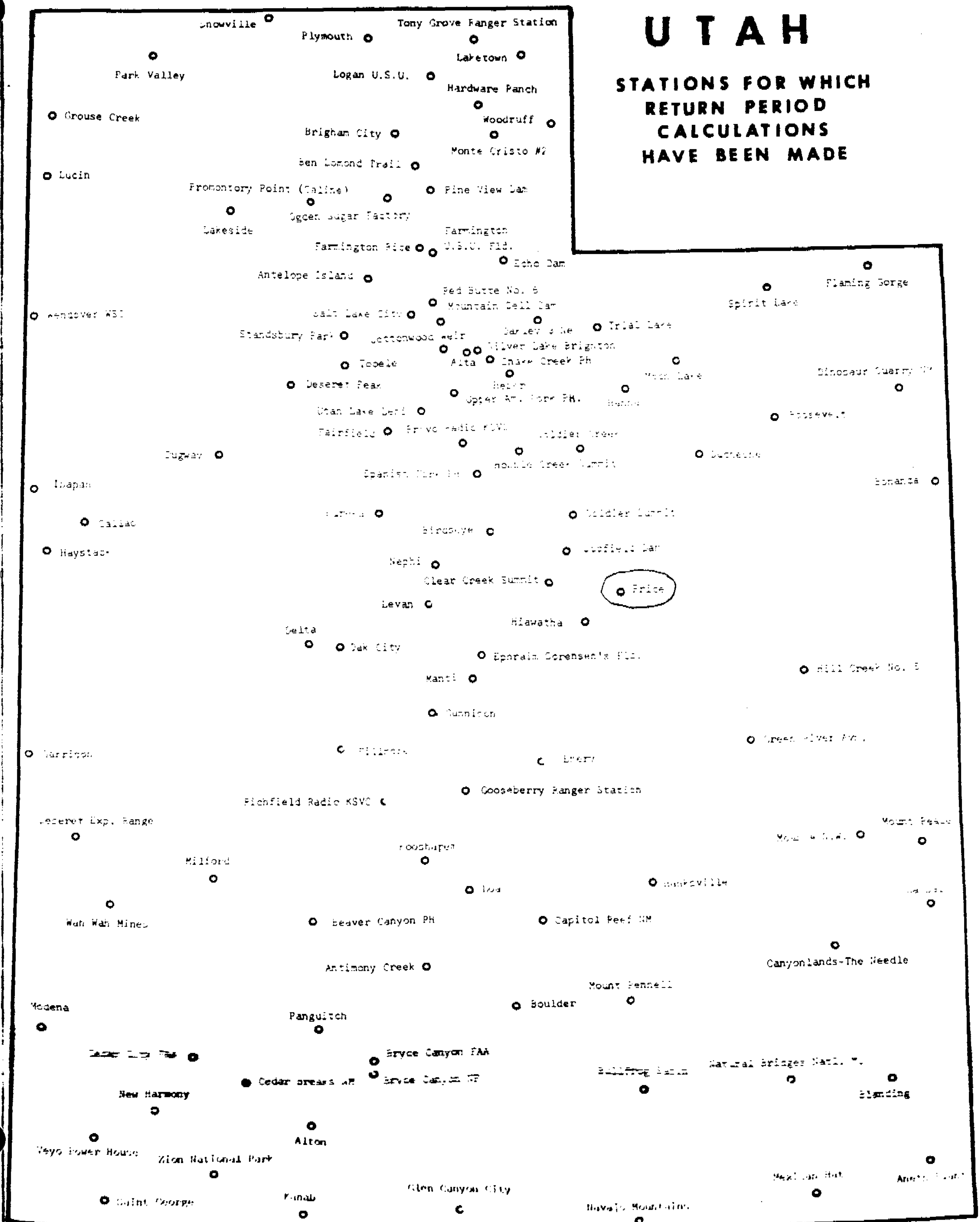
DEPARTMENT OF SOILS AND BIOMETEOROLOGY
Bulletin No. 1

March 1971

3 Pages

UTAH

STATIONS FOR WHICH RETURN PERIOD CALCULATIONS HAVE BEEN MADE



ESTIMATED RETURN PERIODS FOR SHORT DURATION PRECIPITATION
(inches)

Station: Price Elevation: 5680
Latitude: 39° 37' Longitude: 110° 50'

DURATION

RETURN PERIOD (years)	5 Min	10 Min	15 Min	30 Min	1 Hr	2 Hr	3 Hr	6 Hr	12 Hr	24 Hr
1	.08	.13	.17	.23	.29	.37	.44	.62	.78	.95
2	.12	.18	.23	.32	.40	.49	.58	.80	1.00	1.20
5	.16	.25	.32	.44	.56	.68	.79	1.07	1.32	1.58
10	.20	.31	.39	.54	.68	.81	.94	1.25	1.53	1.82
25	.24	.37	.47	.65	.82	.98	1.13	1.50	1.83	2.18
50	.28	.43	.54	.75	.95	1.12	1.29	1.71	2.08	2.47
100	.31	.49	.62	.85	1.08	1.27	1.45	1.91	2.32	2.74

Station: Promontory Point (Saline) Elevation: 4230
Latitude: 41° 13' Longitude: 112° 29'

DURATION

RETURN PERIOD (years)	5 Min	10 Min	15 Min	30 Min	1 Hr	2 Hr	3 Hr	6 Hr	12 Hr	24 Hr
1	.16	.24	.31	.43	.54	.57	.60	.68	.75	.82
2	.18	.27	.35	.48	.61	.67	.72	.85	.97	1.09
5	.21	.32	.40	.56	.71	.80	.89	1.11	1.31	1.51
10	.22	.34	.43	.60	.76	.88	.99	1.28	1.53	1.80
25	.26	.40	.50	.70	.88	1.03	1.17	1.53	1.85	2.18
50	.27	.42	.53	.73	.93	1.11	1.28	1.71	2.09	2.49
100	.29	.45	.57	.79	1.00	1.21	1.41	1.90	2.34	2.80

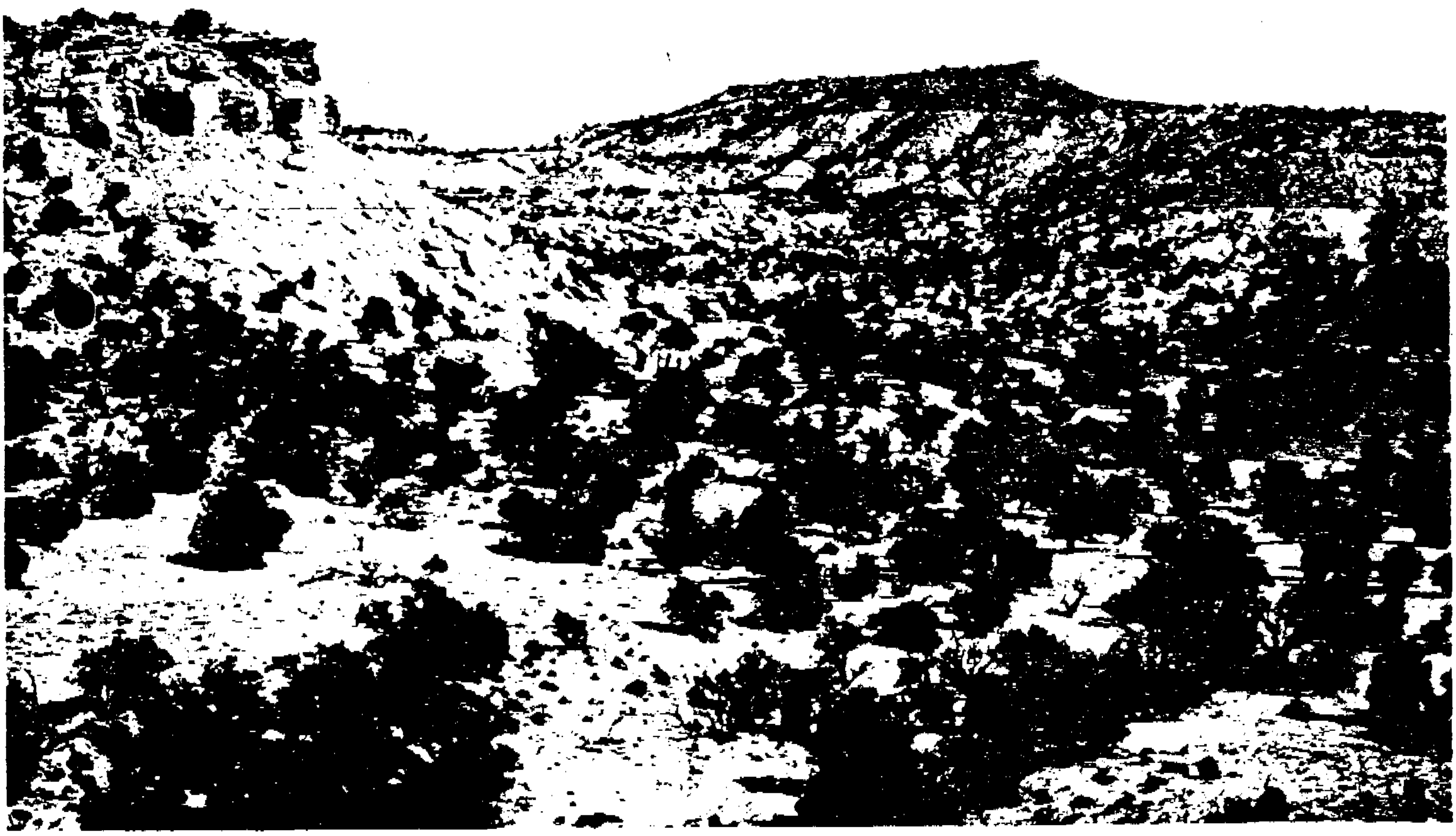
Issued December 1970

EXHIBIT - 6

9 Pages

SOIL SURVEY

Carbon-Emery Area, Utah



UNITED STATES DEPARTMENT OF AGRICULTURE
Soil Conservation Service
and
UNITED STATES DEPARTMENT OF THE INTERIOR
Bureau of Land Management
In cooperation with
UTAH AGRICULTURAL EXPERIMENT STATION

EXHIBIT - 6 -

ponds, mainly with rainbow trout. The lack of good springs and of underground water of good quality restricts the construction and development of farm ponds. Most fishponds in existence are fed by irrigation water.

Engineering Properties and Behavior of Soils [5]

Some soil properties are of special interest to engineers because they affect the construction, maintenance, and performance of roads, airports, pipelines, building foundations, facilities for water storage, erosion-control structures, drainage systems, and sewage disposal systems. Soil properties that are most important to the engineer are permeability to water, shear strength, compaction characteristics, soil drainage, shrink-swell characteristics, grain size, plasticity, and reaction. Also important, however, are depth to the water table, to bedrock, or to a hardpan; content of salt and alkali; and topography.

The information in this soil survey can be used by engineers to—

1. Make studies of soil and land use that will aid in selecting and developing sites for industries, businesses, residences, and recreation.
2. Obtain estimates of the amount of runoff and the erosion characteristics of the soils, for use in designing drainage structures and in planning dams and other structures for use in conserving soil and water.
3. Make reconnaissance surveys of soil and site conditions that will aid in selecting locations for highways and airports and in planning detailed soil surveys for the intended locations.

[5] WILLIAM J. MORGAN, (deceased), engineer, Soil Conservation Service, assisted in preparing this section.

TABLE 3.—*Estimated soil properties*

[Badland (Ba), Gullied land (Gu), Mixed alluvial land (Mx), Riverwash (Rv), Rock land (Ry), Shaly colluvial land (Sn), and Stony alluvial land (Ry), the depth to a seasonal water table is more

Soil series and map symbols	Depth to seasonal water table	Depth to bedrock	Depth from surface (typical profile)	Classification		
				USDA texture	Unified	AASHO
	Inches	*Inches*	*Inches*			
Abbott (Ab, As)	6–40	60+	0–60	Silty clay and silty clay loam.	CL	A-7
Beebe (BbB, BeB, BeC2, BfA)	72+	60+	0–71	Loamy fine sand	SM	A-4
Billings (BlB, BlC2, BsB, BtB, BuB2) (For properties of the Bunderson soil in mapping unit BuB2, refer to the Bunderson series.)	(1)	60+	0–72	Silty clay loam	CL	A-6
Bunderson. (Mapped only in complexes with the Billings and Ravola soils.)	72+	60+	0–72	Loam	CL	A-4
Cache (Ca)	20–40	60+	0–60	Silty clay	CL	A-7
Castle Valley (CeE2)	72+	10–20	0–10 10	Very fine sandy loam Sandstone.	ML	A-4
Cedar Mountain (CmF2)	72+	10–20	0–14 14	Shaly silty clay, clay loam, and silt loam. Shale.	ML–CL	A-6
Chipeta (CBF2, CPB, CPE2) (For properties of the Persayo soils in mapping units CPB and CPE2, refer to the Persayo series. Badland in mapping unit CBF2 is too variable to rate.	72+	10–20	0–17 17	Silty clay loam Shale.	CL	A-6 or A-7
Ferron:						
(Fr)	6–36	60+	0–60	Loam and very fine sandy loam.	ML–CL	A-4
(Fe)	6–36		0–60	Silty clay loam and silt loam.	CL	A-6
Green River (Gr)	20–40	60+	0–45 45–60	Stratified loam and very fine sandy loam. Fine sand	CL–ML SM	A-4 A-2

See footnotes at end of table.

4. Locate probable sources of sand and gravel for use in structures and as a base for both flexible and rigid pavements.
5. Correlate pavement performance with kinds of soil, and thus develop information that will be useful in designing and maintaining pavements.
6. Determine the suitability of soil mapping units for cross-country movement of vehicles and construction equipment.
7. Supplement the information obtained from other published maps and reports and from aerial photographs.
8. Become aware of hazards or of useful properties of soils to be used for highways and earth construction where definite laboratory data are not available.

With the soil map for identification, the engineering interpretations reported here can be useful for many purposes. It should be emphasized, however, that they may not eliminate the need for sampling and testing at the site of specific engineering works that involve heavy loads or where the excavations are deeper than the depth of layers here reported. Even in these situations, however, the soil map is useful for planning more detailed field investigations and for suggesting the kinds of problems that may be expected.

Much of the information in this subsection is in tables 3, 4, and 5. In table 3 properties of soils that are important to engineering are estimated. Table 4 indicates the suitability of soils for various engineering uses. Table 5 gives test data for soils of several soil types that are extensive in the survey area.

significant to engineering

land (St) are omitted from this table, because their properties generally are too variable to estimate. For Badland (Ba) and Rock than 72 inches and bedrock is at or near the surface]

Percentage passing sieve—			Percentage larger than 3 inches	Permeability	Available water capacity	Reaction (paste)	Salinity	Dispersion	Shrink-swell potential	Hydrologic groupings
No. 4 (4.7 mm.)	No. 10 (1.0 mm.)	No. 200 (0.074 mm.)								
				Inches per hour	*Inches per inch of soil depth*	*pH*				
100	100	90–95	--------	0. 05–0. 2	0. 19	7. 1–7. 9	Slight to strong.	Low to moderate.	Moderate---	D
100	100	35–45	--------	5. 0–10. 0	0. 06–0. 10	7. 9–9. 7	None to moderate.	None------	Low-------	A
100	100	85–95	--------	0. 05–0. 2	0. 17–0. 2	7. 6–8. 6	Slight to moderate.	Moderate---	Moderate---	C
100	100	70–90	--------	(3)	0. 13–0. 15	8. 5–10. 0	Moderate to high.	High-------	Low-------	D
100	100	90–100	--------	0. 05–0. 2	0. 15–0. 17	7. 9–8. 5	Very strong---	High-------	Moderate---	D
95–100	85–100	50–60	5–10	2. 5–5. 0	0. 13–0. 15	7. 5–8. 0	None--------	Low-------	Low-------	D
100	95–100	85–90	5–10	0. 05–0. 2	0. 12–0. 14	8. 0–8. 5	None--------	Moderate---	Moderate---	D
100	100	90–97	--------	0. 05–0. 2	0. 15–0. 17	7. 4–8. 0	Moderate to strong.	Moderate---	Moderate---	D
100	100	80–90	--------	0. 8–2. 5	0. 17–0. 19	7. 7–8. 5	Slight to strong.	Low-------	Low-------	B
100	100	90–95	--------	0. 2–0. 8	0. 17–0. 19	7. 7–8. 5	Moderate-----	Moderate---	Moderate---	B
100	100	70–80	--------	0. 8–2. 5	0. 17–0. 19	7. 8–8. 2	None to slight.	Low-------	Low-------	B
100	90–100	15–20	--------	2. 5–5. 0	0. 08–0. 11	7. 8–8. 2	None to slight.	Low-------	None------	B

TABLE 3.—*Estimated soil properties*

Soil series and map symbols	Depth to seasonal water table	Depth to bed-rock	Depth from surface (typical profile)	Classification		
				USDA texture	Unified	AASHO
	Inches	*Inches*	*Inches*			
Harding (Ha)	72+	60+	0–10	Clay loam	CL	A-6
			10–20	Clay	CL	A-7
			20–52	Clay loam and loam	CL	A-6
Hunting (Hn, Hs, Hu)	20–40	60+	0–60	Loam	CL	A-4
Kenilworth (KeE2)	72–	60+	0–34	Stony sandy loam	SM	A-2 or A-4
Killpack (KlB, KlC2, KmB, KpB, KpC2)	(1)	20–40	0–23	Clay loam or loam	ML–CL	A-4
			23–29	Shaly silty clay loam	CL	A-6
			29	Shale.		
Libbings (Lb, Ls)	10–30	20–40	0–34	Silty clay loam, clay, and silty clay.	CL	A-6
			34	Shale.		
Minchey (McB, MlB, MsB, MsC2) (For properties of the Sanpete soils in mapping units MsB and MsC2, refer to the Sanpete series.)	72–	60+	0–32	Clay loam	CL	A-6
			32–64	Gravelly sandy loam	MG–GC	A-4 and A-2.
Palisade (PaB, PdB, PdC2)	(1)	60+	0–41	Very fine sandy loam	ML–CL	A-4
			41–60	Very fine sandy loam	SM	A-4
Penoyer:						
(PeB, PeC2, PhD, PnA, PsB, PsC2, PvB2)	72+	60+	0–60	Loam	CL	A-6
(PrA, PoB)			0–14	Silty clay loam	CL	A-4
			14–60	Loam	CL–ML	A-4
Persayo (PCE2) (For properties of the Chipeta soil in this mapping unit, refer to Chipeta series.)	72+	6–20	0–12	Loam and silty clay loam.	CL	A-4
			12	Shale.		
Rafael (Ra)	6–30	60+	0–70	Silty clay loam and loam.	CL	A-4
Ravola (RlB, RlB2, RlC2, RnD, RsA, RsB, RtB, RuB2) (For properties of the Bunderson soil in mapping unit RuB2, refer to the Bunderson series.)	72+	60+	0–60	Loam	ML–CL	A-4
Saltair (Sa, Sb)	6–60	60+	0–60	Silty clay loam and silt loam.	CL	A-4
Sanpete (SlB, SlD2, SmD2) (For properties of the Minchey soil in mapping unit SmD2, refer to the Minchey series.)	72+	60+	0–14	Gravelly sandy clay loam.	ML–CL	A-4
			14–30	Very cobbly sandy clay loam.	SM	A-4
Woodrow (Wo)	72+	60+	0–60	Silty clay loam.	CL	A-4

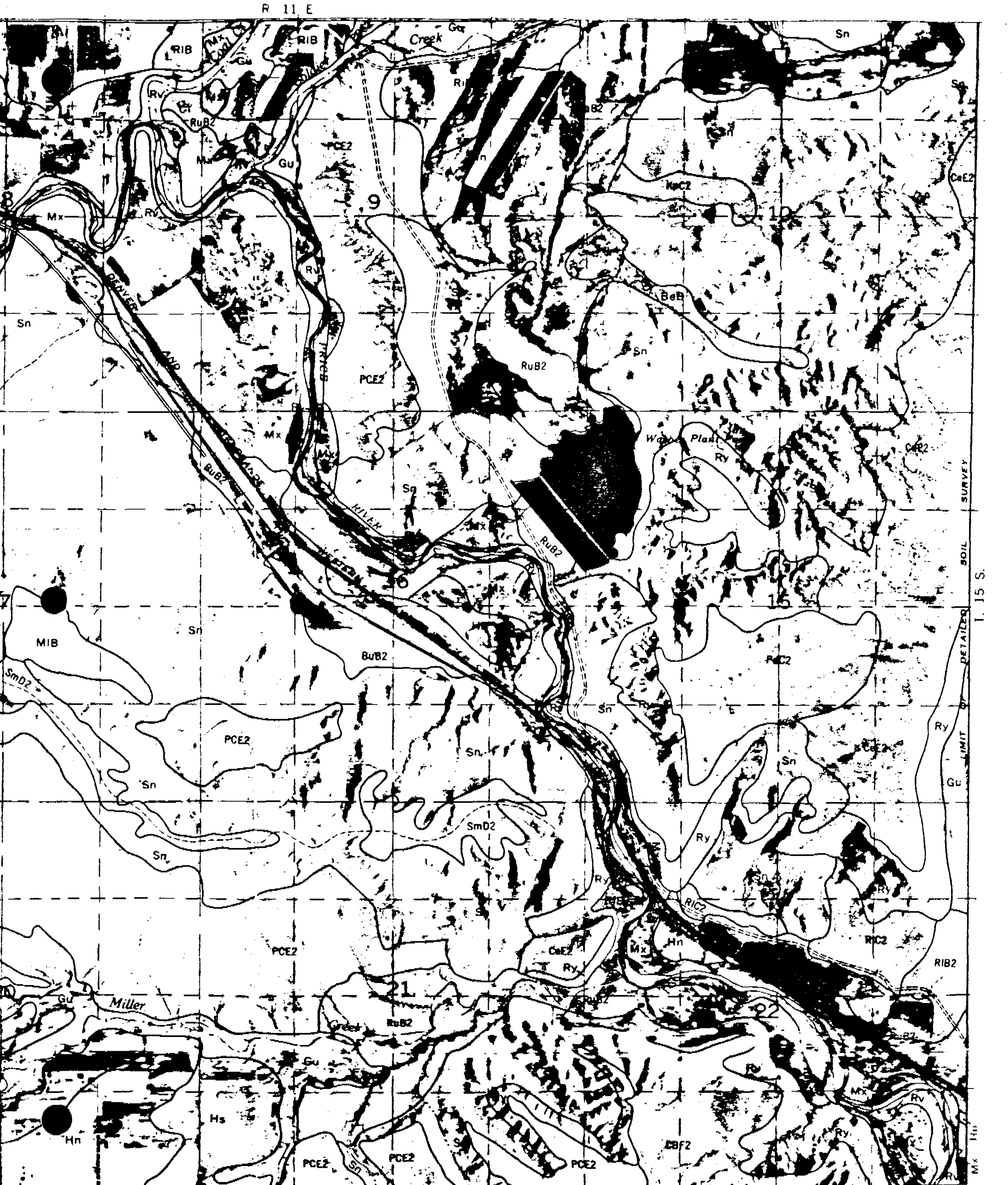
[1] In the Billings and Killpack soils, the seasonal water table is at a depth of more than 72 inches, except that it is between 36 and 60 inches in mapping unit BsB and between 20 and 40 inches in mapping unit KmB.

[2] Less than 0.05 inch per hour in surface layer; 0.5 to 2.5 inches per hour below a depth of 10 to 20 inches.

significant to engineering—Continued

Percentage passing sieve— No. 4 (4.7 mm.)	No. 10 (1.0 mm.)	No. 200 (0.074 mm.)	Percentage larger than 3 inches	Permeability	Available water capacity	Reaction (paste)	Salinity	Dispersion	Shrink-swell potential	Hydrologic groupings
				Inches per hour	*Inches per inch of soil depth*	*pH*				
95–100	95–100	75–85		0.05–0.8	0.19–0.21	8.1–8.7	Moderate	High	Moderate	C
95–100	95–100	75–85		(3)	0.16–0.18	8.1–8.7	Moderate to strong.	High	Moderate	C
75–80	65–75	70–80		0.8–2.5	0.16–0.18	8.3–8.7	Moderate to strong.	High	Moderate	C
100	100	70–80		0.8–2.5	0.17–0.19	7.8–8.3	Slight to strong.	Low	Moderate	B
50–75	45–70	25–40	20–50	0.8–2.5	0.10–0.12	7.7–8.5	None	Low	Low	B
100	100	70–80		0.2–0.8	0.19–0.21	7.7–8.0	Slight to moderate.	Moderate	Moderate	C
80–95	70–90	65–85		0.2–0.8	0.19–0.20	7.7–8.0	Slight to moderate.	Moderate	Moderate	C
100	80–95	90–100		0.05–0.2	0.16–0.18	8.2–8.9	Very strong	Moderate	Moderate	D
95–100	95–100	60–75		0.8–2.5	0.19–0.21	7.9–8.3	None	Low	Moderate	B
55–85	50–80	30–40	5	1.25–5.0	0.06–0.09	7.9–8.3	None	Low	Low	
85–95	80–90	50–60		0.8–2.5	0.17–0.19	7.5–8.0	None	Low	Low	B
85–95	80–90	35–45	0	2.5–5.0	0.07–0.10	7.5–8.0	None	Low	Low	B
100	100	70–80		0.8–2.5	0.17–0.19	7.7–8.2	None	Low	Low	B
100	100	90–95		0.2–0.8	0.19–0.21	7.7–8.2	None	Low	Moderate	B
100	100	70–80		0.8–2.5	0.17–0.19	7.8–8.2	Slight	Low	Low	B
80–85	70–80	65–75		0.8–2.5	0.17–0.19	7.5–8.0	Slight to strong.	Moderate	Moderate	D
100	95–100	75–85		0.05–0.2	0.17–0.19	7.7–8.6	Moderate to strong.	Moderate	Moderate	D
100	100	75–85		0.8–2.5	0.17–0.19	7.7–8.0	None to moderate.	Low	Low	B
100	100	85–95		0.05–0.2	0.16–0.18	8.3–8.9	Very strong	High	Moderate	D
99–100	90–100	50–60	10–20	2.5–5.0	0.10–0.13	7.9–8.5	None	Low	Low	A
70–80	65–75	40–50	20	2.5–5.0	0.06–0.08	7.9–8.5	None	Low	Low	A
100	100	90–95		0.05–0.2	0.19–0.21	7.6–7.9	None to slight.	Low	Moderate	C

[3] Less than 0.05 inch per hour.
[4] In the Palisade soils, mapping unit PaB has a seasonal water table at a depth between 10 and 45 inches. In all other Palisade soils, the water table is lacking.


R. 11 E.
T. 15 S.
LIMIT OF DETAILED SOIL SURVEY
Creek
Miller
Creek
PRICE
RIVER
DENVER AND RIO GRANDE WESTERN
Water Plant
8
9
10
15
16
17
20
21
22
Sn
RuB2
PCE2
Mx
Rv
Gu
Ry
MIB
SmD2
BuB2
RIB
RIC2
RIB2
CeE2
BeB
Hn
Hs
CBF2

Climate

The Carbon-Emery Area has a semiarid, continental type of climate. Humidity is low. Daily and seasonal temperatures vary over a wide range, and there is a large amount of sunshine. The growing season is 110 to 130 days, except that it is 140 to 160 days in an area near Green River. Because of the small amount of rainfall, a long period of time is required for climate to influence the formation of the soils. The effects of low rainfall are most strongly expressed in shallow soil profiles, light-colored A1 horizons, a low content of organic matter, and strongly developed Cca horizons.

Climatic records show that the average monthly precipitation is about 0.5 of an inch during the period of October through June and that it is about 1 inch in July, August, and September. The total yearly average precipitation is about 8 inches. During winter, when evaporation and transpiration are low, the precipitation that falls is largely available for storage if it enters the soil. During the period of November to March, the average precipitation is about 2.5 inches. Sanpete, Minchey, and Palisade soils, on the older land surfaces, have a strong Ca horizon at a depth of 7 to 20 inches. This is the depth to which winter precipitation moistens these soils.

In March, April, and May, the Carbon-Emery Area frequently has winds of moderate to high velocity that dry the soils and increase the rates of evaporation and transpiration. These winds are important in the movement and redistribution of calcareous sediment because they occur before the soil warms up and protective vegetation has grown enough to help break the force of the wind. The widespread deposition of calcareous sediment by wind probably accounts for the calcareous A horizon in the Minchey, Sanpete, Palisade, and Castle Valley soils. The A1 horizon in the Castle Valley soils is also slightly more calcareous than the B horizon, and this indicates that calcareous sediment has been recently added to the surface.

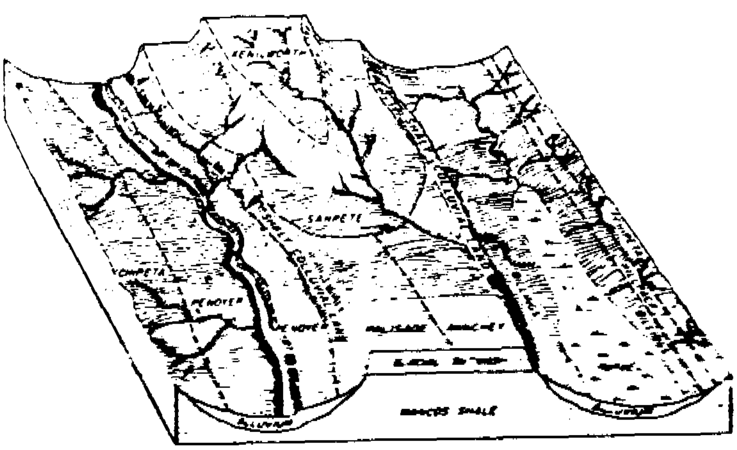


Figure 15.—Schematic cross section showing soils and underlying material north of Castle Dale.

Time

In a semiarid climate the soil-forming processes are slow and require a much longer time to modify the parent material than is required in a wetter climate. The Castle Valley soils seem to be the oldest in the Area: that is, they have been subjected to horizon differentiation for the longest time. They have a moderately developed Bt horizon and contain accumulations of calcium carbonate.

The Minchey and Sanpete soils are intermediate in development. They show increased amounts of clay in the subsoil, but they lack clay films. They also have horizons that contain a strong accumulation of lime.

The least horizonation occurs in soils that have formed in alluvium—those of the Billings, Ravola, and Penoyer series. These soils are on alluvial fans and flood plains. The length of time the soil material has been in place is too short for genetic horizons to have formed in these soils, but some organic matter has accumulated in the surface layer to form a weakly developed A1 horizon. A buried A1 horizon is not uncommon.

The old dissected glacial outwash plains on which the Kenilworth soils occur appear to be old eroded land surfaces. The profile looks like the ca horizon of an eroded soil from which most of the solum has been removed. Kenilworth soils are developing in this ca horizon, which has been modified only by the translocation of lime and an increase in content of organic matter.

The following is evidence that an old soil may have existed on these dissected outwash plains. In places, the surface layer is sandy clay loam, and this might be a relict B horizon. Typically, the present surface layer contains slightly more clay than the substratum. In many places lime-coated gravel and cobblestones are on the surface. In a few places lime coatings on stones extend 2 to 6 inches above the surface. This evidence indicates that soil material has been removed from around the coarse surface material.

Relief

Drainage, aeration, exposure, and susceptibility to erosion are factors of relief that affect the soil-forming processes. For example, the Kenilworth soils have a weakly developed A1 horizon that is slightly darker and more prominent on north and east exposures than on south and west exposures. Furthermore, similar vegetation is more vigorous on north and east exposures than on other exposures. These differences in vegetation probably are the result of less direct exposure to the sun on north- and east-facing slopes than on other slopes. In addition, soil temperature is lower, surface evaporation is less, and water for transpiration is less restricted on north- and east-facing slopes than on south- and west-facing slopes.

Most soils in the survey area apparently were well drained before they were irrigated. Since irrigation was started, a high water table has developed and salts have accumulated, resulting in the formation of the somewhat poorly drained and poorly drained saline soils of the Ferron, Hunting, and Saltair series. In many places this change in drainage can be attributed to differences in the surface of the underlying shale, which is undulating or uneven. Shale restricts the movement of underground water and causes this water to accumulate.

In some places, especially in depressions, the Saltair and Libbings soils have a crust of salt as much as 1½ inches thick on the surface. In those places salty water has seeped into the depressions and has left salt when it evaporated. These soils are bare or have only a sparse stand of pickleweed and greasewood.

Plant and animal life

The natural vegetation in the Carbon-Emery Area consists of a sparse stand of desert shrubs and some bunch grasses. Along the streams are squawbush, cottonwood, and willow. Because plant growth is sparse, such soils as the Minchey, Sanpete, and Harding typically contain less than 1 percent organic matter under natural conditions. After a period of cultivation and irrigation, the organic matter in such soils as the Billings, Penoyer, and Ravola increases to 2 or 3 percent. This increase is the result of applications of some barnyard manure and of increases in the amount of plant residue returned to the soil through cultivation and irrigation. The content of organic matter does not increase above 2 or 3 percent. This level seems to represent the equilibrium in these climatic conditions for soils under irrigation.

Some areas that have a high water table support a growth of sedges and grasses. In such areas the content of organic matter in the O horizon of the Ferron, Rafael, and Abbott soils is 12 to 16 percent. Organic matter in the A1 horizon ranges from 3 to 6 percent, and it decreases with depth until there is only 1 to 2 percent at depths below 5 inches.

Greasewood and shadscale are two common salt-tolerant plants having fleshy leaves and thorns that exert considerable influence on the content of salt in the soils. This is especially true for the Beebe and Bunderson soils but to a less extent for the Ravola and Billings soils. The leaves of both greasewood and shadscale contain salts that have been assimilated from the soil, and they are salty to the taste. As the dropped leaves decompose, the mineral residue is again incorporated into the soil. Some of the salts are leached away, but sodium salts react with the soil to increase the amount of exchangeable sodium. Where sodium salts have accumulated, areas called slickspots appear. Slickspots are evident for many years after the greasewood plants have disappeared. A soil in these slickspots, designated "Beebe No. 1," in table 7, has pH paste values between 8.0 and 9.7. About 50 feet away, an irrigated, leached soil, designated "Beebe No. 2," has pH and exchangeable sodium values less than those of Beebe No. 1.

TABLE 7.—*A comparison of a Beebe soil before and after irrigation*

Beebe No. 1, before irrigation			Beebe No. 2, after irrigation		
Depth from surface	pH paste	Exchangeable sodium	Depth from surface	pH paste	Exchangeable sodium
Inches		*Percent*	*Inches*		*Percent*
0–2	8.0	3	0–6	7.8	4
2–12	9.7	50	6–12	7.9	4
12–18	9.6	52	12–18	7.9	3
18–34	8.5	19	18–36	7.7	3
34–39	7.9	3	36–60	8.0	7
39–71	8.2	8			

Animals, as well as plants, have affected the formation of soils in the survey area. Prairie dogs, for example, have dug burrows that are especially noticeable in this semiarid area because of the sparse cover of vegetation. The prairie dog towns consist of a system of burrows and mounds. The subsoil becomes a mound when it is brought to the surface by prairie dogs when they dig their burrows. After a burrow is abandoned, the soil material on or near the surface fills the cavities and washes into the deeper soil horizons.

Classification of Soils

Soils are classified so that we can more easily remember their significant characteristics. Classification enables us to assemble knowledge about the soils, to see their relationship to one another and to the whole environment, and to develop principles that help us to understand their behavior and their response to manipulation. First through classification, and then through use of soil maps, we can apply our knowledge of soils to specific fields and other tracts of land.

Thus, in classification soils are placed in narrow categories that are used in detailed soil surveys so that knowledge about the soils can be organized and used in managing farms, fields, and woodlands; in developing rural areas; in performing engineering work; and in many other ways. Soils are placed in broad classes to facilitate study and comparison in large areas, such as countries and continents.

Two systems of classifying soils have been used in the United States in recent years. The older system was adopted in 1938 (*6*) and later revised (*5*). The system currently used was adopted for general use by the National Cooperative Survey in 1965. The current system is under continual study. Therefore, readers interested in developments of the current system should search the latest literature available (*9,4*).

The current system of classification is based on morphological characteristics that reflect the genesis of soils. In this system the criteria used as a basis for classification are soil properties that are observable and measurable. The properties are chosen, however, so that the soils of similar genesis, or mode of origin, are grouped together.

The current system of classification has six categories. Beginning with the broadest, these categories are order, suborder, great group, subgroup, family, and series. In table 8 some of the classes in the current system are given for each soil series in the Carbon-Emery survey area.

Laboratory Analyses

The results of laboratory analyses of samples, taken at the same location as the typical soil, are shown by horizons in tables 9 and 10. The analyses in table 9 were made by the Soil Conservation Service and Utah State University Cooperative Soils Laboratory, Logan, Utah. Those in table 10 were made by the Agricultural Research Service.

Methods of Analyses

All samples were air dried in the laboratory. They were then sieved through round openings 2 millimeters in diameter. Samples that appeared to contain no appreciable amount of pebbles or stones, that is, less than 5 percent, were poured through a mechanical crusher that has openings about 4 millimeters in diameter. Samples that contained an appreciable amount of pebbles or stones were broken up in an iron mortar without crushing the pebbles or stones. Where it was necessary to reduce the size of the sample, a Riffle sampler was used. Each laboratory sample was mixed thoroughly to insure uniformity, and all subsequent analyses were made on the fraction that was less than 2 millimeters in diameter. The percentage of material greater than 2 millimeters in diameter was calculated by dividing the weight of the fraction retained on the 2-millimeter sieve by the initial weight of the air-dry sample. Subsamples less than 2 millimeters in diameter were ground small enough to pass a sieve of 0.3 millimeter by use of a mortar and pestle. These subsamples were used to determine organic carbon and the calcium carbonate equivalent.

The reaction, or pH, was measured with a line-operated pH meter using a glass electrode with a calomel reference electrode. In determining the pH of soil-water suspensions in a ratio of one to five, the suspensions were stirred vigorously immediately before the electrodes were inserted. At the first indication of stabilization, the pH was read; then, the process was repeated until duplicate readings were obtained. Distilled water, or water free of carbon dioxide, was used for all soil-water suspensions.

In determining the content of soluble salts, a standard Bureau of Soils cup was used to obtain the ohms of resistance of the soil paste at saturation moisture content. The percentage of total soluble salts was then obtained from standard tables after correcting for soil texture and temperature.

Gypsum

The amount of calcium plus magnesium found in 1:10, 1:20, 1:50 or 1:100 water extract, less than found in saturation extract, was considered to be derived from gypsum. Calcium plus magnesium in both cases was determined by titration with versenate, using Eriochrome Black T as an indicator. Exchangeable sodium percentage (ESP) value was obtained by dividing the amount of exchangeable sodium by the cation-exchange capacity and multiplying the results by 100.

TABLE 8.—*Classification of soil series*

Series	Family	Subgroup	Order
Abbott	Fine, montmorillonitic, calcareous, mesic	Fluventic Haplaquepts	Inceptisols.
Beebe	Sandy, mixed, mesic	Typic Torrifluvents	Entisols.
Billings	Fine-silty, mixed, calcareous, mesic	Typic Torrifluvents	Entisols.
Bunderson	Fine-silty, mixed, calcareous, mesic	Typic Torrifluvents	Entisols.
Cache	Fine, mixed, mesic	Typic Salorthids	Aridisols.
Castle Valley	Loamy, mixed, mesic	Lithic Xerollic Haplargids	Aridisols.
Cedar Mountain	Loamy, mixed, mesic	Lithic Camborthids	Aridisols.
Chipeta	Clayey, mixed, calcareous, mesic, shallow	Typic Torriorthents	Entisols.
Ferron	Coarse-silty, mixed, calcareous, mesic	Fluventic Haplaquepts	Inceptisols.
Green River	Coarse-loamy, mixed, calcareous, mesic	Aquic Ustifluvents	Entisols.
Harding	Fine, mixed, mesic	Typic Natrargids	Aridisols.
Hunting	Fine-silty, mixed, calcareous, mesic	Aquic Ustifluvents	Entisols.
Kenilworth	Loamy-skeletal, mixed, mesic	Xerollic Calciorthids	Aridisols.
Killpack	Fine-silty, mixed, calcareous, mesic	Typic Torriorthents	Entisols.
Libbings	Fine, mixed, mesic	Ustollic Salorthids	Aridisols.
Minchey	Fine-loamy, mixed, mesic	Typic Calciorthids	Aridisols.
Palisade	Coarse-loamy, mixed, mesic	Xerollic Calciorthids	Aridisols.
Persayo	Loamy, mixed, calcareous, mesic, shallow	Typic Torriorthents	Entisols.
Penoyer	Coarse-silty, mixed, calcareous, mesic	Typic Torrifluvents	Entisols.
Rafael	Fine-silty, mixed, calcareous, mesic	Typic Haplaquepts	Inceptisols.
Ravola	Fine-silty, mixed, calcareous, mesic	Typic Torrifluvents	Entisols.
Saltair	Fine-silty, mixed, mesic	Typic Salorthids	Aridisols.
Sanpete	Loamy-skeletal, mixed, mesic	Xerollic Calciorthids	Aridisols.
Woodrow	Fine-silty, mixed, calcareous, mesic	Typic Torrifluvents	Entisols.

<u>Elements of a Unit Hydrograph</u>
The dimensionless curvilinear unit hydrograph (figure 16.1) has 37.5% of the total volume in the rising side, which is represented by one unit of time and one unit of discharge. This dimensionless unit hydrograph also can be represented by an equivalent triangular hydrograph having the same units of time and discharge, thus having the same percent of volume in the rising side of the triangle (figure 16.2).

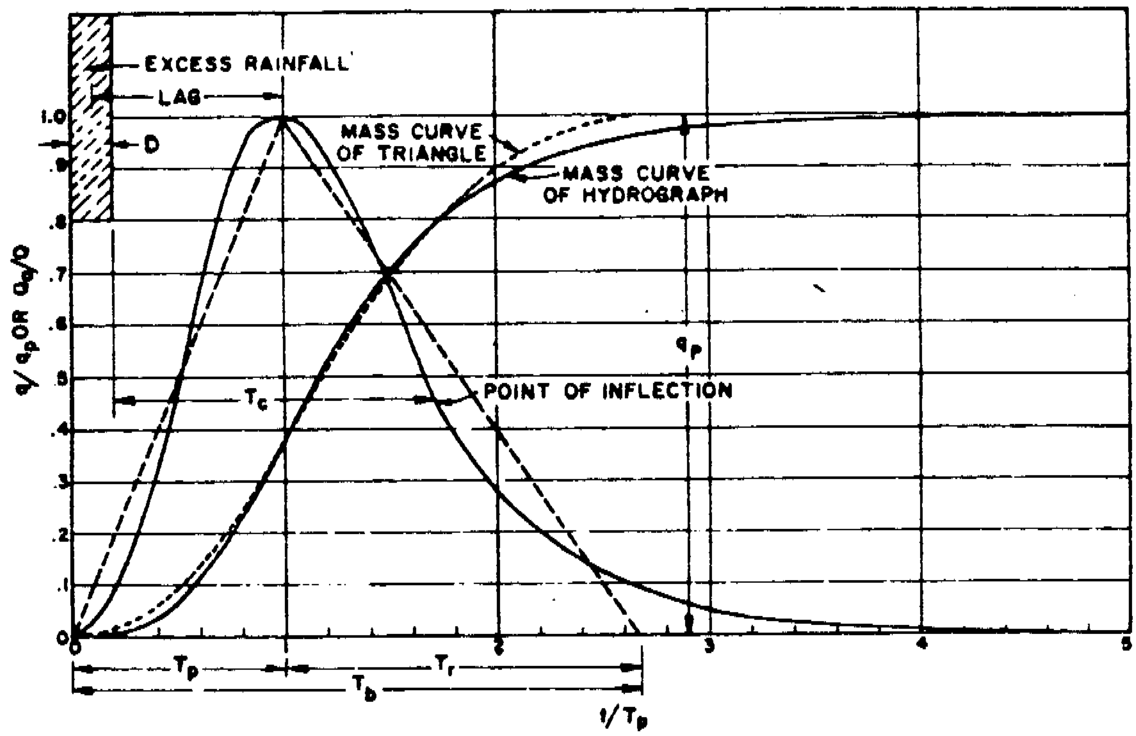


Figure 16.2 Dimensionless curvilinear unit hydrograph and equivalent triangular hydrograph

This allows the base of the triangle to be solved in relation to the time to peak using the geometry of triangles. Solving for the base length of the triangle, if one unit of time T_p equals .375 of volume:

$$T_b = \frac{1.00}{.375} = 2.67 \text{ units of time,}$$

$$T_r = T_b - T_p = 1.67 \text{ units of time or } 1.67\ T_p.$$

These relationships are useful in developing the peak rate equation for use with the dimensionless unit hydrograph.

Peak Rate Equation

From figure 16.2 the total volume under the triangular unit hydrograph is:

$$Q = \frac{q_p\ T_p}{2} + \frac{q_p\ T_r}{2} = \frac{q_p}{2}(T_p + T_r) \qquad \text{(Eq. 16.1)}$$

With Q in inches and T in hours, solve for peak rate q_p in inches per hour:

$$q_p = \frac{2Q}{T_p + T_r} \qquad \text{(Eq. 16.2)}$$

$$\text{Let } K = \frac{2}{1 + \frac{T_r}{T_p}} \qquad \text{(Eq. 16.3)}$$

$$\text{Therefore } q_p = \frac{KQ}{T_p} \qquad \text{(Eq. 16.4)}$$

In making the conversion from inches per hour to cubic feet per second and putting the equation in terms ordinarily used, including drainage area "A" in square miles, and time "T" in hours, equation 16.4 becomes the general equation:

$$q_p = \frac{645.33 \times K \times A \times Q}{T_p} \qquad \text{(Eq. 16.5)}$$

Where q_p is peak discharge in cubic feet per second (cfs) and the conversion factor 645.33 is the rate required to discharge one inch from one square mile in one hour.

The relationship of the triangular unit hydrograph, $T_r = 1.67\ T_p$, gives K = 0.75. Then substituting into equation 16.5 gives:

$$q_p = \frac{484\ A\ Q}{T_p} \qquad \text{(Eq. 16.6)}$$

Since the volume under the rising side of the triangular unit hydrograph is equal to the volume under the rising side of the curvilinear dimensionless unit hydrograph in figure 16.2, the constant 484, or peak rate factor, is valid for the dimensionless unit hydrograph in figure 16.1.

Any change in the dimensionless unit hydrograph reflecting a change in the percent of volume under the rising side would cause a corresponding change in the shape factor associated with the triangular hydrograph and therefore a change in the constant 484. This constant has been known to vary from about 600 in steep terrain to 300 in very flat swampy country. The E&WP Unit hydrologist should concur in the use of a dimensionless unit hydrograph other than figure 16.2. If for some reason it becomes necessary to vary the dimensionless shape of the hydrograph to perform a special job, the ratio of the percent of total volume in the rising side of the unit hydrograph to the rising side of a triangle is a useful tool in arriving at the peak rate factor.

Figure 16.2 shows that:

$$T_p = \frac{\Delta D}{2} + L \qquad \text{(Eq. 16.7)}$$

where ΔD is the duration of unit excess rainfall and L is the watershed lag in hours. The lag (L) of a watershed is defined (chapter 15) as the time from the center of mass of excess rainfall (ΔD) to the time to peak (T_p) of a unit hydrograph. From equation 16.6:

$$q_p = \frac{484\ A\ Q}{\frac{\Delta D}{2} + L} \qquad \text{(Eq. 16.8)}$$

The average relationship of lag (L) to time of concentration (T_c) is $L = 0.6\ T_c$ (chapter 15).

Substituting in equation 16.8, the peak rate equation becomes:

$$q_p = \frac{484\ A\ Q}{\frac{\Delta D}{2} + 0.6\ T_c} \qquad \text{(Eq. 16.9)}$$

The time of concentration is defined in two ways in chapter 15: 1) the time for runoff to travel from the furthermost point in the watershed to one point in question, and 2) the time from the end of excess rainfall to the point of inflection of the unit hydrograph.

These two relationships are important since T_c is computed under the first definition and ΔD, the unit storm duration, is used to compute the time to peak (T_p) of the unit hydrograph. This in turn is applied to all of the points on the abscissa of the dimensionless unit hydrograph using the ratio t/T_p as shown in table 16.1.

The dimensionless unit hydrograph shown in figure 16.2 has a time to peak at one unit of time and point of inflection at approximately 1.7 units of time. Using the relationships Lag = 0.6 T_c and the point of inflection = 1.7 T_p, ΔD will be .2 T_p. A small variation in ΔD is permissible, however, it should be no greater than .25 T_p. See example 1.

Using the relationship shown on the dimensionless unit hydrograph, figure 16.2 to compute the relationship of ΔD to T_c:

$$T_c + \Delta D = 1.7\ T_p \qquad \text{(Eq. 16.10)}$$

$$\frac{\Delta D}{2} + .6\ T_c = T_p \qquad \text{(Eq. 16.11)}$$

Solving these two equations:

$$T_c + \Delta D = 1.7\left(\frac{\Delta D}{2} + .6\ T_c\right)$$

$$.15\ \Delta D = .2\ T_c$$

$$\Delta D = .133\ T_c \qquad \text{(Eq. 16.12)}$$

Application of Unit Hydrograph

The unit hydrograph can be constructed for any location on a uniformly shaped watershed, once the values of q_p and T_p are defined (figure 16.3, areas A and B).

Area C in figure 16.3 is an irregularly shaped watershed having two uniformly shaped areas (C2 and C1) with a big difference in their time of concentration. This watershed requires the development of two unit hydrographs which may be added together forming one irregularly shaped unit hydrograph. This irregularly shaped unit hydrograph may be used to develop a flood hydrograph in the same way as the unit hydrograph developed from the dimensionless form (figure 16.1) is used to develop the flood hydrograph. See example 1 for area shown in figure 16.3. Also, each of the two unit hydrographs developed for areas C2 and C1 in figure 16.3 may be used to develop a flood hydrograph for its respective C2 and C1 areas. The flood hydrographs from each area are then combined to form the hydrograph at the outlet of area C.

There are many variables integrated into the shape of a unit hydrograph. Since a dimensionless unit hydrograph is used and the only parameters readily available from field data are drainage area and time of concentration, consideration should be given to dividing the watershed into hydrologic units of uniformly shaped areas. These divisions, if at all possible, should be no greater than 20 square miles in area and should have a homogeneous drainage pattern.

The "storm duration" is the actual time duration of precipitation excess. This time duration varies with actual storms and should not be confused with the unit time or unit hydrograph duration.

Effects of storm duration and time of concentration (Section 17, Pg. 17.

When the effects of a change in either the storm duration or the time of concentration must be taken into account, one way to do it is to use the following relation from Chapter 16:

$$T_p = a(D) + b(T_c) \qquad \text{(Eq. 17-44)}$$

where T_p = time to peak, in hours

a = a constant

D = storm duration, in hours, during which runoff is generated; it is usually less than the total storm duration

b = a constant

T_c = time of concentration, in hours

As shown in Chapter 1-, the constants a and b can be taken as 0.5 and 0.6 respectively, for most problems, in which case Equation 17-44 becomes:

$$T_p = 0.5\ D + 0.6\ T_c \qquad \text{(Eq. 17-45)}$$

Using Equation 17-45 in equations 17-37, 17-38, and 17-39 produces working equations in which either the storm duration or the time of concentration can be changed and the effect of the change determined. Such equations are not often used because the main comparison is usually between present and future conditions in which only runoff amount and drainage area will change. In special problems where storm duration must be taken into account there are other approaches that are more applicable (see the section titled "Use of Equation 17-43 on large watersheds").

Storm Duration

(Section 4, Pg. 4.11

The total duration of a storm is used in estimating a peak rate of runoff or in developing a hydrograph. The duration is always known for a design storm, but for natural storms, such as those used in some methods of watershed evaluation, the duration may be difficult to determine. Methods of estimating the duration of natural storms will be briefly discussed.

NATURAL STORMS

Durations of specific actual storms can generally be estimated to the nearest hour by use of Weather Bureau publications of hourly precipitation data. With these data, or even with instrument charts from a recording gage, it is often difficult to decide on the beginning or ending times of a storm. Furthermore, if there are periods of no rain within the storm, the duration may need to be arbitrarily defined. The problem of hydrograph construction is simplified by using storm increments and, in general, this is the best way of using natural storms (for hydrograph construction in this manner, see chapter 16).

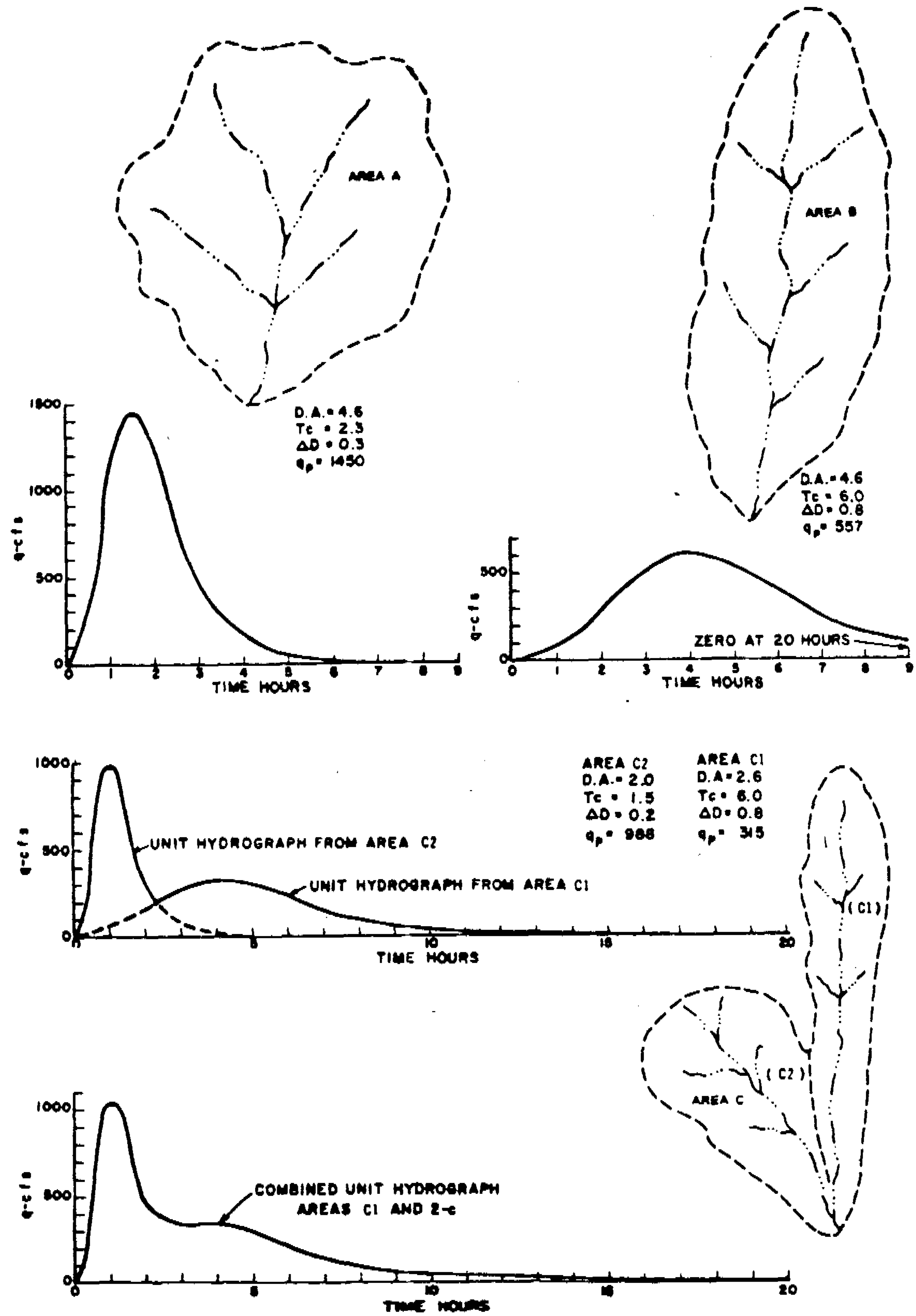


Figure 16.3 The effect of watershed shape on the peaks of unit hydrographs

MISCELLANEOUS EXHIBITS

Back Up, Source Materials

Table 9.1.--Runoff curve numbers for hydrologic soil-cover complexes

(Antecedent moisture condition II, and $I_a = 0.2\ S$)

Land use	Cover: Treatment or practice	Cover: Hydrologic condition	Hydrologic soil group: A	B	C	D
Fallow	Straight row	----	77	86	91	94
Row crops	"	Poor	72	81	88	91
	"	Good	67	78	85	89
	Contoured	Poor	70	79	84	88
	"	Good	65	75	82	86
	"and terraced	Poor	66	74	80	82
	" " "	Good	62	71	78	81
Small grain	Straight row	Poor	65	76	84	88
		Good	63	75	83	87
	Contoured	Poor	63	74	82	85
		Good	61	73	81	84
	"and terraced	Poor	61	72	79	82
		Good	59	70	78	81
Close-seeded legumes 1/ or rotation meadow	Straight row	Poor	66	77	85	89
	" "	Good	58	72	81	85
	Contoured	Poor	64	75	83	85
	"	Good	55	69	78	83
	"and terraced	Poor	63	73	80	83
	"and terraced	Good	51	67	76	80
Pasture or range		Poor	68	79	86	89
		Fair	49	69	79	84
		Good	39	61	74	80
	Contoured	Poor	47	67	81	88
	"	Fair	25	59	75	83
	"	Good	6	35	70	79
Meadow		Good	30	58	71	78
Woods		Poor	45	66	77	83
		Fair	36	60	73	79
		Good	25	55	70	77
Farmsteads		----	59	74	82	86
Roads (dirt) 2/		----	72	82	87	89
(hard surface) 2/		---	74	84	90	92

1/ Close-drilled or broadcast.
2/ Including right-of-way.

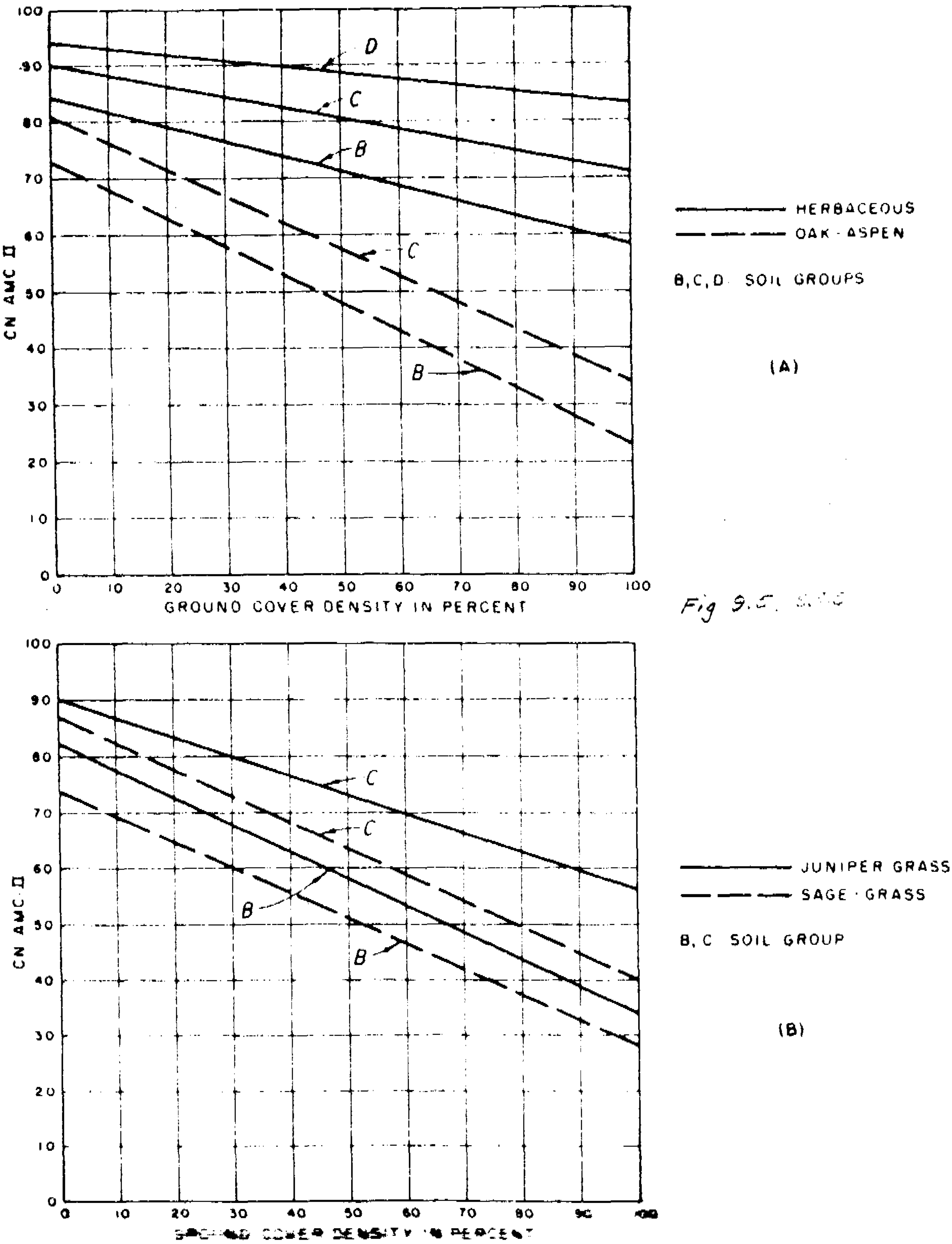


Figure A-3 Determining CN for forest range in the western United States. 288-D-2824.

Same as Fig. 9.6, S.C.S. N.E.H. Hyd. Sec. 4. except larger

Table 10.1. Curve numbers (CN) and constants for the case $I_a = 0.2\ S$

1	2	3	4	5	1	2	3	4	5
CN for condi-tion II	CN for conditions I	III	S values*	Curve* starts where P =	CN for condi-tion II	CN for conditions I	III	S values*	Curve* starts where P =
			(inches)	(inches)				(inches)	(inches)
100	100	100	0	0	60	40	78	6.67	1.33
99	97	100	.101	.02	59	39	77	6.95	1.39
98	94	99	.204	.04	58	38	76	7.24	1.45
97	91	99	.309	.06	57	37	75	7.54	1.51
96	89	99	.417	.08	56	36	75	7.86	1.57
95	87	98	.526	.11	55	35	74	8.18	1.64
94	85	98	.638	.13	54	34	73	8.52	1.70
93	83	98	.753	.15	53	33	72	8.87	1.77
92	81	97	.870	.17	52	32	71	9.23	1.85
91	80	97	.989	.20	51	31	70	9.61	1.92
90	78	96	1.11	.22	50	31	70	10.0	2.00
89	76	96	1.24	.25	49	30	69	10.4	2.08
88	75	95	1.36	.27	48	29	68	10.8	2.16
87	73	95	1.49	.30	47	28	67	11.3	2.26
86	72	94	1.63	.33	46	27	66	11.7	2.34
85	70	94	1.76	.35	45	26	65	12.2	2.44
84	68	93	1.90	.38	44	25	64	12.7	2.54
83	67	93	2.05	.41	43	25	63	13.2	2.64
82	66	92	2.20	.44	42	24	62	13.8	2.76
81	64	92	2.34	.47	41	23	61	14.4	2.88
80	63	91	2.50	.50	40	22	60	15.0	3.00
79	62	91	2.66	.53	39	21	59	15.6	3.12
78	60	90	2.82	.56	38	21	58	16.3	3.26
77	59	89	2.99	.60	37	20	57	17.0	3.40
76	58	89	3.16	.63	36	19	56	17.8	3.56
75	57	88	3.33	.67	35	18	55	18.6	3.72
74	55	88	3.51	.70	34	18	54	19.4	3.88
73	54	87	3.70	.74	33	17	53	20.3	4.06
72	53	86	3.89	.78	32	16	52	21.2	4.24
71	52	86	4.08	.82	31	16	51	22.2	4.44
70	51	85	4.28	.86	30	15	50	23.3	4.66
69	50	84	4.49	.90					
68	48	84	4.70	.94	25	12	43	30.0	6.00
67	47	83	4.92	.98	20	9	37	40.0	8.00
66	46	82	5.15	1.03	15	6	30	56.7	11.34
65	45	82	5.38	1.08	10	4	22	90.0	18.00
64	44	81	5.62	1.12	5	2	13	190.0	38.00
63	43	80	5.87	1.17	0	0	0	infinity	infinity
62	42	79	6.13	1.23					
61	41	78	6.39	1.28					

*For CN in column 1.

CONVERSIONS

THIS:	TIMES THIS:	GIVES YOU THIS:
cfs days	1.983	AF
cfs days	0.03719	inches depth on 1 square mile
cfs days per square mile	0.03719	inches depth
cfs hours	0.08264	AF
cfs hours per square mile	0.001550	inches depth
cfs	1.983	AF per day
cfs	724.0	AF per year (365 days)
cfs	448.8	U. S. gallons per minute
cfs	0.6463	million U. S. gallons per day
csm	0.03719	inches depth per day
csm	13.57	inches depth per year (365 days)
inches per hour	645.3	csm
inches per hour	1.008	cfs per acre
inches depth	53.33	AF per square mile
inches depth on 1 sq. mi.	53.33	AF
AF	0.5042	cfs days
AF	12.10	cfs hours
AF	0.01875	inches depth on 1 square mile
AF	0.3258	million U. S. gallons
AF per day	0.5042	cfs
AF per square mile	0.01875	inches depth
U. S. gallons per minute	0.002228	cfs
million U. S. gallons per day	1.547	cfs
million U. S. gallons per day	3.069	AF
feet per second	0.6818	miles per hour
centimeters	0.3937	inches
hectares	2.471	acres
liters	0.2642	U. S. gallons
kilograms	2.205	pounds
cubic feet	7.480	U. S. gallons
imperial gallons	1.200	U. S. gallons

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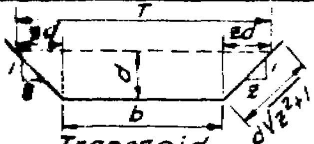
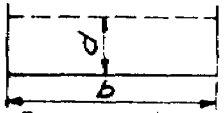
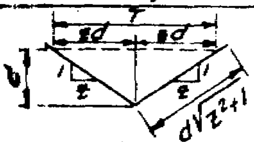
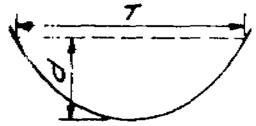
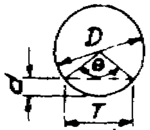
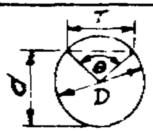
HYDRAULICS: ELEMENTS OF CHANNEL SECTIONS

Section	Area a	Wetted Perimeter p	Hydraulic Radius r	Top Width T
 Trapezoid	$bd+zd^2$	$b+2d\sqrt{z^2+1}$	$\dfrac{bd+zd^2}{b+2d\sqrt{z^2+1}}$	$b+2zd$
 Rectangle	bd	$b+2d$	$\dfrac{bd}{b+2d}$	b
 Triangle	zd^2	$2d\sqrt{z^2+1}$	$\dfrac{zd}{2\sqrt{z^2+1}}$	$2zd$
 Parabola	$\frac{2}{3}dT$	$T+\dfrac{8d^2}{3T}$ [1]	$\dfrac{2dT^2}{3T^2+8d^2}$ [1]	$\dfrac{3a}{2d}$
 Circle - < ½ full [2]	$\dfrac{D^2}{8}\left(\dfrac{\pi\theta}{180}-\sin\theta\right)$	$\dfrac{\pi D\theta}{360}$	$\dfrac{45D}{\pi\theta}\left(\dfrac{\pi\theta}{180}-\sin\theta\right)$	$D\sin\dfrac{\theta}{2}$ or $2\sqrt{d(D-d)}$
 Circle - > ½ full [3]	$\dfrac{D^2}{8}\left(2\pi-\dfrac{\pi\theta}{180}+\sin\theta\right)$	$\dfrac{\pi D(360-\theta)}{360}$	$\dfrac{45D}{\pi(360-\theta)}\left(2\pi-\dfrac{\pi\theta}{180}+\sin\theta\right)$	$D\sin\dfrac{\theta}{2}$ or $2\sqrt{d(D-d)}$

[1] Satisfactory approximation for the interval $0<\frac{d}{T}\leqq 0.25$
When $d/T>0.25$, use $p=\frac{1}{2}\sqrt{16d^2+T^2}+\frac{T^2}{8d}\sinh^{-1}\frac{4d}{T}$

[2] $\theta=4\sin^{-1}\sqrt{d/D}$
[3] $\theta=4\cos^{-1}\sqrt{d/D}$
} Insert θ in degrees in above equations

REFERENCE

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
H. H. Bennett, Chief
ENGINEERING STANDARDS UNIT

STANDARD DWG. NO. ES-33
SHEET 1 OF 1
DATE 6-6-50

5.4-1a

HYDRAULICS: HEAD LOSS COEFFICIENTS FOR CIRCULAR AND SQUARE CONDUITS FLOWING FULL

HEAD LOSS COEFFICIENT, K_p, FOR CIRCULAR PIPE FLOWING FULL $\quad K_p = \frac{5087\, n^2}{d_i^{4/3}}$

Pipe diam. inches	Flow area sq. ft.	MANNING'S COEFFICIENT OF ROUGHNESS "n" 0.010	0.011	0.012	0.013	0.014	0.015	0.016	0.017	0.018	0.019	0.020	0.021	0.022	0.023	0.024	0.025
6	0.196	0.0467	0.0565	0.0672	0.0789	0.0914	0.1050	0.1194	0.1348	0.151	0.168	0.187	0.206	0.226	0.247	0.269	0.292
8	0.349	.0318	.0385	.0458	.0537	.0623	.0715	.0814	.0919	.1030	.1148	.1272	.140	.154	.168	.183	.199
10	0.545	.0236	.0286	.0340	.0399	.0463	.0531	.0604	.0682	.0765	.0852	.0944	.1041	.1143	.1249	.136	.148
12	0.785	.0185	.0224	.0267	.0313	.0363	.0417	.0474	.0535	.0600	.0668	.0741	.0817	.0896	.0980	.1067	.1157
14	1.069	.0151	.0182	.0217	.0255	.0295	.0339	.0386	.0436	.0488	.0544	.0603	.0665	.0730	.0798	.0868	.0942
15	1.23	.0138	.0166	.0198	.0232	.0270	.0309	.0352	.0397	.0446	.0496	.0550	.0606	.0666	.0727	.0792	.0859
16	1.40	.0126	.0153	.0182	.0213	.0247	.0284	.0323	.0365	.0409	.0455	.0505	.0556	.0611	.0667	.0727	.0789
18	1.77	.01078	.0130	.0155	.0182	.0211	.0243	.0276	.0312	.0349	.0389	.0431	.0476	.0522	.0570	.0621	.0674
21	2.41	.00878	.01062	.0126	.0148	.0172	.0198	.0225	.0254	.0284	.0317	.0351	.0387	.0425	.0464	.0506	.0549
24	3.14	.00735	.00889	.01058	.0124	.0144	.0165	.0188	.0212	.0238	.0265	.0294	.0324	.0356	.0389	.0423	.0459
27	3.98	.00628	.00760	.00904	.01061	.0123	.0141	.0161	.0181	.0203	.0227	.0251	.0277	.0304	.0332	.0362	.0393
30	4.91	.00546	.00660	.00786	.00922	.01070	.01228	.0140	.0158	.0177	.0197	.0218	.0241	.0264	.0289	.0314	.0341
36	7.07	.00428	.00518	.00616	.00723	.00839	.00965	.01096	.0124	.0139	.0154	.0171	.0189	.0207	.0226	.0246	.0267
42	9.62	.00348	.00422	.00502	.00589	.00683	.00784	.00892	.01007	.0129	.0126	.0139	.0154	.0169	.0184	.0201	.0218
48	12.57	.00292	.00353	.00420	.00493	.00572	.00656	.00747	.00843	.00945	.01053	.01166	.0129	.0141	.0154	.0168	.0182
54	15.90	.00249	.00302	.00359	.00421	.00488	.00561	.00638	.00720	.00808	.00900	.00997	.01099	.0121	.0132	.0144	.0156
60	19.63	.00217	.00262	.00312	.00366	.00424	.00487	.00554	.00626	.00702	.00782	.00866	.00955	.01048	.0115	.0125	.0135

HEAD LOSS COEFFICIENT, K_c, FOR SQUARE CONDUIT FLOWING FULL $\quad K_c = \frac{29.16\, n^2}{r^{4/3}}$

Conduit Size feet	Flow area sq. ft.	MANNING'S COEFFICIENT OF ROUGHNESS "n" 0.012	0.013	0.014	0.015	0.016
2×2	4.00	0.01058	0.01242	0.01440	0.01653	0.01880
2½×2½	6.25	0.00786	0.00922	0.01070	0.01228	0.01397
3×3	9.00	.00616	.00723	.00839	.00963	.01096
3½×3½	12.25	.00502	.00589	.00683	.00784	.00892
4×4	16.00	.00420	.00493	.00572	.00656	.00746
4½×4½	20.25	.00359	.00421	.00488	.00561	.00638
5×5	25.00	.00312	.00366	.00425	.00487	.00554
5½×5½	30.25	.00275	.00322	.00374	.00429	.00488
6×6	36.00	.00245	.00287	.00333	.00382	.00435
6½×6½	42.25	.00220	.00258	.00299	.00343	.00391
7×7	49.00	.00199	.00234	.00271	.00311	.00354
7½×7½	56.25	.00182	.00213	.00247	.00284	.00323
8×8	64.00	.00167	.00196	.00227	.00260	.00296
8½×8½	72.25	.00154	.00180	.00209	.00240	.00273
9×9	81.00	.00142	.00167	.00194	.00223	.00253
9½×9½	90.25	.00133	.00156	.00180	.00207	.00236
10×10	100.00	.00124	.00145	.00168	.00193	.00220

$$H_1 = (K_p \text{ or } K_c)\, L\, \frac{v^2}{2g}$$

Nomenclature:

- a = Cross-sectional area of flow in sq. ft.
- d_i = Inside diameter of pipe in inches
- g = Acceleration of gravity = 32.2 ft per sec.
- H_1 = Loss of head in feet due to friction in length L.
- K_c = Head loss coefficient for square conduit flowing full.
- K_p = Head loss coefficient for circular pipe flowing full.
- L = Length of conduit in feet
- n = Manning's coefficient of roughness.
- Q = Discharge or capacity in cu ft. per sec.
- r = Hydraulic radius in feet.
- v = Mean velocity in ft. per sec.

Example 1: Compute the head loss in 300 ft. of 24 in. diam. concrete pipe flowing full and discharging 30 c.f.s. Assume $n = 0.015$

$$v = \frac{Q}{a} = \frac{30}{3.14} = 9.55 \text{ f.p.s.}; \quad \frac{v^2}{2g} = \frac{(9.55)^2}{64.4} = 1.42 \text{ ft.}$$

$$H_1 = K_p L \frac{v^2}{2g} = 0.0165 \times 300 \times 1.42 = 7.03 \text{ ft}$$

Example 2: Compute the discharge of a 250 ft., 3×3 square conduit flowing full if the loss of head is determined to be 2.25 ft. Assume $n = 0.014$.

$$H_1 = K_c L \frac{v^2}{2g}; \quad \frac{v^2}{2g} = \frac{H_1}{K_c L} = \frac{2.25}{0.00839 \times 250} = 1.073 \text{ ft.}$$

$$v = \sqrt{64.4 \times 1.073} = 8.31; \quad Q = 9 \times 8.31 = 74.8 \text{ c.f.s.}$$

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE H. H. Bennett, Chief ENGINEERING STANDARDS UNIT	STANDARD DWG. NO. ES - 42 SHEET 1 OF 1 DATE 7-17-50

Revised 4-11-51

APPENDIX B

HYDROLOGIC EVALUATION

1981

DEVELOPMENT OF EVAPORATION AND

OTHER LOSSES - WELLINGTON COAL CLEANING PLANT

1981 Pump House Diversion (acre feet)[1]	832
1981 Precipitation $(\frac{9.64}{12} \times 88.2)$ (AF)[2]	71
Total Water Entering Circuit (AF)	903

Heat Dryer Losses

Total coal shipments	905,037 tons
Percent of coal to heat dryer	40%
Tons of coal from heat dryer	362,014
Moisture in coal from heat dryer	4.5%
Moisture in coal shipped	16,291
Tons of coal dry basis to heat dryer	345,723
Moisture in coal to heat dryer	9.5%
Tons of coal wet basis to heat dryer	382,015
Total water evaporated (tons)	20,001
Acre feet of water $(\frac{20,001 \times 2000}{62.4 \times 43,560})$	14.7

Pond Evaporation

Average evaporation loss-annual — 5 feet[3]

Surface Area of Ponds (acres)

Clearwater	12.3
Lower Refuse	65.6
Fields	9.3
Ditches	1.0
Total	88.2

Evaporation loss — 441.0 AF

Other Losses During 1981 — 462 AF

1 - Coal Preparation Plant Records - reported to the Price River Water Users Association for 1981.

2 - Precipitation at Wellington for 1981, Utah State Climatologist's Office, Logan, Utah.

3 - Pan Evaporation Tests - Coal Preparation Plant.

CALCULATION NOTES

Subject Hydrologic Evaluation

By BAF
Checked
Acc't
Date June 30 1983
Sheet No. 1 of 2 Sheets

GENERAL

The Wellington Coal Cleaning Plant is located some two to three miles southeast of Wellington, Carbon County, Utah. The area is semi-arid with an annual precipitation of about six to eight inches, most of which is received in the form of short duration thunderstorms.

The plant and adjacent areas west of the Price River are located in the past and present floodplain of the Price River. Currently, the floodplain is confined to that area between the river and the mainline of the Denver and Rio Grande Western Railroad tracks. In the past, however, the river probably occupied a channel in the present plant location. The resulting floodplain occupies all of the low areas west of the Price River which are now utilized by U. S. Steel Mining Co., Inc. for coal processing, equipment and material storage, and coarse refuse accumulation.

A floodplain, by definition, is a flat area which is built up by stream deposition. The plant area is located within the Price River floodplain and is, therefore, subject to sediment deposition rather than erosion. As such, the plant area is subject to sediment accumulation, not contributions.

The slurry pond area east of the Price River is located in a stream channel which at one time coursed irrigation and storm drainage to the Price River. Since construction of the impoundments and the diversion ditch along the North Dike, irrigation water is intercepted and passed to the river through the diversion ditch. Storm runoff from disturbed areas is contained in the slurry pond system. Hydrologic evaluation of the slurry pond system and adjacent diversion ditch, pursuant to the proposed modifications, for a 100 year 24 hour storm is included in Technical Revision No. 1.

FIELD RECONNAISSANCE

The permit area and adjacent areas were evaluated by U. S. Steel Mining Co., Inc. engineering personnel in May of 1983 for specific drainage and vegetation characteristics. Conservative vegetative cover values were assigned (generally about 60 percent of that observed during the field reconnaissance) due to the relatively wet spring. It is expected that some of the low ground cover now growing will disappear with the arrival of summer.

Additional field reconnaissance in June of 1983 by three U. S. Steel Mining Co. engineers and two Division of Oil, Gas and Mining hydrologists established that the May 1983 survey was a prudent evaluation and that it was appropriate to proceed with the data for the hydrologic evaluation.

CALCULATION NOTES

Subject Hydrologic Evaluation

By BAF

Checked

Acc't

Date June 30 19 83

Sheet No. 2 of 2 Sheets

Field Reconnaissance continued

Drawing No. F9-177, sheets 1 and 2 of 2, summarizes the results of the field reconnaissance.

DEVELOPMENT OF HYDROLOGIC CURVE NUMBERS

Ground cover types and densities, established during field reconnaissance, and hydrologic soil groups were used in developing the curve numbers shown on F9-177. Methodology used in developing the curve numbers is shown on page B-3.

STORM RUNOFF CALCULATION

Storm runoff calculations are based on a 10 year 24 hour precipitation event, which at the plant area is 1.82 inches (page B-17). The runoff estimate is calculated from a weighted average curve number for the entire drainage area. This method of evaluation is demonstrated in the SCS-National Engineering Handbook - Section 4, Chapter 10, pages 10 and 11. Storm runoff is then calculated using the formula:

$$Q = \frac{(P-0.2S)^2}{P+0.8S}$$

Where: Q = inches of runoff
P = inches of rainfall for the design storm = 1.82" for 10-24 (p. B-17)
$S = \frac{1000}{\text{curve no.}} - 10$

Local weather patterns indicate a dry antecedent moisture condition (AMC I). However, all calculations in this section are based on the antecedent moisture condition II (AMC II) at the Divisions request.

SOIL LOSS CALCULATION

Soil loss calculations are based on the Universal Soil Loss Equation (Soil Conservation Service, January, 1976) where k = RkLSC. Factors used in developing the soil loss are included on pages B-28 thru B-35 . Soil losses in the plant area are expected to be insignificant due to the depositional rather than erosive nature of the site.

CALCULATION NOTES

Subject Development of Hydrologic Curve Numbers

By BAF
Checked
Acc't
Date June 30 19 83
Sheet No. 1 **of** 2 **Sheets**

Soil(1)(2) Class	Hyd. Soil(2) Group	Cover(3) Complex	Cover(3) Density	Curve Number
SmD2	A	Sage/Grass	10	49(4)
MlB	B	Sage/Grass	10	69(4)
Mx			15	67(4)
BuB2	C	Sage/Grass	0	87(4)
Sn(c)			5	85(4)
			10	83(4)
			15	80(4)
			20	78(4)
			25	76(4)
			30	73(4)
			40	68(4)
			50	64(4)
			60	59(4)
		Dirt Roads	-	87(5)
		Paved Roads	-	98(5)
		Building Roofs	-	98(5)
		Pond Surface	-	100
PCE2	D	Sage/Grass	10	86(4)
Sn(D)			15	84(4)

(1) Drawing No.'s E9-3339 and A9-1431.

(2) SOIL SURVEY Carbon-Emery Area, Utah, Soil Conservation Service and Bureau of Land Management in cooperation with Utah Agricultural Experiment Station, December, 1970; pp. B-18, 19.

(3) Field reconnaissance of the plant and adjacent areas during May and June of 1983 by U. S. Steel Mining Co., Inc. engineering personnel.

(4) National Engineering Handbook, Section 4, Chapter 9, Fig. 9.6, Soil Conservation Service, pp. B-20, 21

(5) Urban Hydrology for Small Watersheds, TR No. 55, Table 2.2, Soil Conservation Service, January, 1975, p. B-22

CALCULATION NOTES

Subject: Development of Hydrologic Curve Numbers	By: BAF
	Checked:
	Acc't:
	Date: June 30 1983
	Sheet No. 2 of 2 Sheets

NOTE:

All areas where coarse materials have been consolidated into a pile are assumed to have no runoff from a 10 year 24 hour storm. These areas are:

1. Coarse refuse pile south of the main plant building.

2. Coarse refuse pile west of the Upper Refuse Pond.

3. Track ballast.

This assumption is supported as follows:

- Coarse material has an extremely high infiltration rate due to void spaces, inherant to piles of coarse material, which preclude a drainage path along the surface of the pile.

- Water which lands on a pile of coarse material will percolate through the pile, saturating the surface area of the materials prior to reaching the soil foundation.

- The thickness of the refuse piles generally exceeds 20 feet and track ballast generally a foot or more. This indicates a significant surface area that must be saturated before water reaches the foundation.

- Each pile is located on a soil foundation. Should moisture from a precipitation event reach the soil foundation, the soil itself will absorb its respective volume of moisture.

- The soil foundation is the only location where runoff can occur due to the percolating rather than runoff affect of the coarse material. Since the pile is located on top, the foundation soil has a 100 percent ground cover. This corresponds to a curve number of 40 (reference 4, p. B-3) which is significantly lower than the minimum curve number where runoff will occur from a storm of 1.82 inch intensity.

CALCULATION NOTES

Subject: Diversion Ditch Drainage

Ref. Dwg. A9-1341, p. B-24

By: BAF
Checked:
Acc't:
Date: May 27 1983
Sheet No. 1 of 5 Sheets

Soil (1)(3) Type	Curve No. (2)	Acres (3)	Weighted CN (4)
SmD2	49	13.8	2.4
MlB	69	20.2	5.0
Sn(c)	73	7.0	1.8
Sn(c)	78	7.4	2.1
Sn(c)	80	25.9	7.4
Sn(c)	83	8.4	2.5
PCE2	86	197.8	60.5
Sn(D)	86	0.8	0.2
		281.0	81.9

$$CN = 82$$

$$S = \frac{1000}{82} - 10 = 2.20$$

$$Q = \frac{(1.82 - 0.2(2.20))^2}{1.82 + 0.8(2.20)} = 0.532 \text{ in.}$$

$$\text{Volume} = \frac{0.532 \text{ in. } (281.0 \text{ acres})}{12 \text{ in/ft}} = \underline{\underline{12.5 \text{ acre feet}}}$$

The diversion ditch outlet capacity of 22.6 AF (reference Dwg. E9-3431) is adequate to contain the 12.5 AF discharge from a 10 year 24 hour storm.

CALCULATION NOTES

Subject Drainage Ditch
East of Coarse Refuse Pile

Ref. Dwg. F9-177 Sheet 1 of 2

By BAF
Checked
Acc't
Date May 27 1983
Sheet No. 2 of 5 Sheets

Soil(1)(3) Type	Curve No.(2)	Acres(3)	Weighted CN(4)
Refuse	NO RUNOFF		
Sn/BuB2	78	4.97	29.8
BeB2	80	1.50	9.2
Sn	83	3.99	25.5
Sn/BuB2	87	2.26	15.1
Blacktop/Bldgs.	98	0.28	2.1
		13.00	81.7

$$CN = 82$$

$$S = \frac{1000}{82} - 10 = 2.20$$

$$Q = \frac{(1.82 - 0.2(2.20))^2}{1.82 + 0.8(2.20)} = 0.532 \text{ in.}$$

$$\text{Volume} = \frac{0.532 \text{ in } (13.00\text{Ac})(43560 \text{ ft}^2/\text{ac})}{12 \text{ in/ft}} = 25{,}105 \text{ cf}$$

Discussion on pages B-10, 11 of this Appendix.

Subject Plant Area-Drainage into Auxiliary Pond and Road Pond

Ref. Dwg. F9-177

CALCULATION NOTES

By BAF

Checked

Acc't

Date May 27 1983

Sheet No. 3 of 5 Sheets

Runoff Requirement

Soil (1) (3) Type	Curve No. (2)	Acres (3)	Weighted CN (4)
BuB2	78	0.45	5.5
Sn (c)	83	0.51	6.6
Sn (c)	85	0.16	2.1
Sn/BuB2	87	2.40	32.8
Blacktop/Bldgs.	98	2.42	37.2
Pond Surface	100	0.43	6.8
		6.37	91.0

CN = 91

S = 0.99

Q10-24 = 1.007 in.

Q25-24 = 1.322 in.

$$\text{Volume 10-24} = \frac{(1.007\text{ in})\ (6.37\text{ ac})\ (43560\text{ sf/ac})}{12\text{ in/ft}} = \underline{23{,}240}\text{ CF}$$

$$\text{Volume 25-24} = \frac{(1.322\text{ in})\ (6.37\text{ ac})\ (43560\text{ sf/ac})}{12\text{ in/ft}} = \underline{30{,}569}\text{ CF}$$

Soil Loss Requirement (acreage excludes blacktop, buildings and pond surface areas)

R = 20

k = 0.50

LS = 0.14

C = 0.45

(20) (0.50) (0.14) (0.45) = 0.630 tons/acre/year

$$\text{3 year soil loss} = \frac{(0.630\text{ T/AC/YR})\ (3.52\text{ AC})\ (3\text{ YR})\ (2000\text{ \#/T})}{85\text{ \#/CF}} = \underline{157}\text{ CF}$$

Containment is provided in the Road Pond (reference Dwg. E9-3429)

Subject	Heat Dryer Drainage Area Ref. Dwg. F9-177

CALCULATION NOTES
By BAF
Checked
Acc't
Date May 27 1983
Sheet No. 4 of 5 Sheets

Runoff Requirement

Soil (1) (3) Type	Curve No. (2)	Acres (3)	Weighted CN (4)
BuB2	87	0.97	73.4
Blacktop/ Bldgs.	98	0.18	15.3
		1.15	88.7

CN = 89 Q10-24 = 0.879 in.
S = 1.24 Q25-24 = 1.177 in.

$$\text{Volume 10-24} = \frac{(0.879 \text{ in.})\ (1.15 \text{ ac})\ (43560 \text{ sf/ac})}{12 \text{ in/ft}} = 3{,}669 \text{ cf}$$

$$\text{Volume 25-24} = \frac{(1.177 \text{ in.})\ (1.15 \text{ ac})\ (43560 \text{ sf/ac})}{12 \text{ in/ft}} = 4{,}913 \text{ cf}$$

Soil Loss Requirement

F = 20 LS = 0.10
K = 0.50 C = 0.45

(20) (0.50) (0.10) (0.45) = 0.450 tons/acre/year

$$\text{3 year soil loss} = \frac{(0.450 \text{ T/ac/yr})\ (0.97 \text{ ac})\ (3 \text{ yr})\ (2000 \text{ \#/T})}{85 \text{ \#/CF}} = 31 \text{ CF}$$

Containment is provided in the Heat Dryer Pond (reference Dwg. E9-3433). where is this

Rev. 1: 6-30-83
Rev. 2: 12-30-83
Rev. 3: 2-24-84

CALCULATION NOTES

Subject Non Coal Waste Holding Area Drainage

Ref. Dwg. E9-3431, F9-177

By BAF

Checked

Acc't

Date May 27 19 83

Sheet No. 5 of 5 Sheets

Soil(1)(3) Type	Curve No.(2)	Acres(3)	Weighted CN(4)
BuB2	87	0.26	87.0

$$CN = 87$$

$$S = \frac{1000}{87} - 10 = 1.49$$

$$Q = \frac{(1.82 - 0.2(1.49))^2}{1.82 + 0.8(1.49)} = 0.769 \text{ in.}$$

$$\text{Volume} = \frac{(0.769 \text{ in})(0.26AC)(43560 \text{ ft.}^2/\text{ac})}{12 \text{ in/ft.}} = \underline{\underline{726 \text{ cf}}}$$

The non coal waste holding area capacity of 1138 cf (reference Dwg. E9-3431) is adequate to contain the 726 cf of runoff from a 10 year 24 hour storm.

(1) SOIL SURVEY Carbon-Emery Area, Utah,Soil Conservation Service and Bureau of Land Management in Cooperation with Utah Agricultural Experimentation Station, December 1980, pp B-18, 19.

(2) Pages B-3, 4 of this Appendix.

(3) Drawing No. F9-177 and/or E9-3339.

(4) Curve number times acres divided by total acres.

CALCULATION NOTES

Subject Drainage Ditch Evaluation

By BAF
Checked
Acc't
Date June 30 1983
Sheet No. 1 of 2 Sheets

The drainage ditch east of the refuse pile provides an escape for excess water which may accumulate from the plant equipment and material storage yard area. Due to the flatness of the drainage area, runoff velocities are slow and in many areas nonexistent,resulting in puddling. Culverts located under roadways preclude significant puddling in roadways, and act to equalize standing water in most areas. Soil infiltration is high due to the slow velocities which allow moisture more time to percolate into the soils rather than run off.

The equipment and material storage yard is located on an old floodplain of the Price River and as such is subject to sediment deposition rather than erosion. The soil carrying capacity on these relatively flat surfaces is essentially nil.

Hydrologic evaluation for the drainage area feeding the drainage ditch is found on page B-6.. Cross sections of the ditch are found on Drawing No. E9-3431. Since the drainage area is not subject to sediment contributions, the Operator proposes that a silt fence be installed at the location of cross section K-K. As such, sediments that may be carried into the drainage ditch will be filtered prior to discharge. This area is considered a small area in terms of sediment contribution potential, and exemption from sedimentation pond installations is requested.

The drainage ditch is not subject to significant water velocities which would wash out the silt fence. Like the surrounding area, the ditch has only a slight grade which results in a maximum velocity of 2.8 feet per second. It should be noted that approximately one half of the total storm runoff (assuming all the runoff reached the drainage ditch) can be contained in the ditch from section K-K upstream while maintaining 0.3 feet of freeboard. The Geofab silt fence has a capacity to pass some 470 gallons per square foot of fence. Specifications for this silt fence is included on page B-27.

Ditch velocities were calculated using the manning formula given on page B-13. Velocities are calculated using Q = AV as follows:

- **Section JJ'**

  A = 19.8 SF (calculated from survey notes)
  P = 16.7 FT (measured when surveyed)
  R = A/P = 1.18 FT.

  $$Q = \frac{1.486}{0.035}(19.8)(1.18)^{2/3}(0.0027)^{1/2} = 49 \text{ cfs}$$

  $$V = \frac{Q}{A} = 2.5 \text{ ft./sec.}$$

*high infiltration? based on?

CALCULATION NOTES

Subject: Drainage Ditch Evaluation

By: BAF

Checked:

Acc't:

Date: June 30 1983

Sheet No. 2 of 2 Sheets

- Section KK'

A = 24.1 SF (calculated from survey notes)
P = 16.7 FT (measured when surveyed)
R = A/P = 1.44 FT

$$Q = \frac{1.486}{0.035}(24.1)(1.44)^{2/3}(0.0027)^{1/2} = 68 \text{ cfs}$$

$$V = \frac{Q}{A} = 2.8 \text{ ft./sec.}$$

CALCULATION NOTES

Subject Sedimentation Control at the River Pumphouse

By BAF

Checked

Acc't

Date June 30 19 83

Sheet No. 1 of 1 Sheets

The River Pumphouse is located near the clear water dike between the Price River and the county road. The total drainage area is about one acre. Approximately half the area drains into an incised pit shown on Drawing No. F9-177. The adjacent area is bermed along the river, but the concrete roadway (reference Dwg. E9-3430) along the diversion dam is not bermed.

The Operator requests exemption from a sedimentation pond installation due to the small area (0.47 acres) involved. It is proposed that a silt fence or straw filter be installed across the roadway to filter sediments carried by runoff before entering the Price River.

Either a row of straw bales or a silt fence (reference B-27) approximately 15 feet long will provide effective sediment protection. The proposed location is shown on F9-177.

CALCULATION NOTES

Subject Diversion Ditch Adequacy

By BAF

Checked

Acc't

Date June 27 19 83

Sheet No. 1 **of** 2 **Sheets**

DITCH CAPACITY

Flow of water in open channels is most commonly computed using the Manning Formula (B-23):

$$Q = \frac{1.486}{n} AR^{2/3} S^{\frac{1}{2}}$$

Where:
Q = quantity of flow in cfs
n = coefficient of roughness
A = cross sectional area in sf
R = hydraulic radius (area/ wetted perimeter) in ft.
S = channel slope in ft./ft.
P = wetted perimeter

Referring to Drawing E9-3431, Sections FF', GG' and HH', and assuming a roughness coefficient of 0.035 (p. B-23), the minimum capacity of this ditch is 341 cfs. Detailed calculation is as follows:

- Section FF'

  A = 43.7 SF (calculated from survey notes)

  P = 30.1 FT (measured when surveyed)

  R = A/P = 1.45 ft.

$$Q = \frac{1.486}{0.035} (43.7)(1.45)^{2/3}(0.0209)^{\frac{1}{2}} = \underline{\underline{344 \text{ cfs}}}$$

- Section GG'

  A = 75.3 SF (calculated from survey notes)

  P = 38.2 FT (measured when surveyed)

  R = A/P = 1.97 ft.

$$Q = \frac{1.486}{0.035}(75.3)(1.97)^{2/3}(0.0105)^{\frac{1}{2}} = \underline{\underline{515 \text{ cfs}}}$$

- Section HH'

A = 46.3 SF (calculated from survey notes)

P = 21.4 FT (measured when surveyed)

R = A/P = 2.16 ft.

$$Q = \frac{1.486}{0.035}(46.3)(2.16)^{2/3}(0.0108)^{1/2} = \underline{\underline{341\ cfs}}$$

MAXIMUM DITCH CAPACITY = 341 cfs

DITCH ADEQUACY

Referring to the hydrologic evaluation of the diversion ditch on page B-5 , the average curve number is 82 and the drainage area is 281.0 acres. A 10 year 24 hour storm is 1.82 inches intensity. The nomographs on pages B-25 and B-26 indicate peak flows as follows: (assuming steep slopes)

CN = 80 peak flow = 83 cfs

CN = 85 peak flow = 148 cfs

Interpolating: CN = 82 peak flow = 109 cfs

The ditch capacity of 341 cfs is adequate to pass the 109 cfs peak flow from a 10 year 24 hour storm.

CALCULATION NOTES

Subject Storm Runoff into Catch Basin

By B.A.F.

Checked

Acc't

8-27 1984

Sheet No. 2 of 2 Sheets

Method referenced on Page B-2 of ORP.

Acres	Curve No.	Weighted CN
3.10	84	45.9
2.57	89	40.3
5.67		86.2

CN = 86

$S = 1000/86 - 10 = 1.63$

$Q_{10-24} = \frac{(1.82 - 0.2(1.63))^2}{1.82 + 0.8(1.63)} = 0.714 \text{ In.}$

$\text{Volume} = 0.714 \text{ in } \left(\frac{1 \text{ Ft.}}{12 \text{ In.}}\right) \; 5.67 \text{ Acres}$

= 0.34 Acre-Ft. for 10 Yr. - 24 Hr. Storm

$Q_{25-24} = \frac{(2.18-0.2(1.63))^2}{2.18 = 0.8(1.63)} = 0.987 \text{ In.}$

$\text{Volume} = 0.987 \text{ In } \left(\frac{1 \text{ Ft.}}{12 \text{ In.}}\right) \; 5.67 \text{ Acres}$

= 0.47 Acre-Ft. for 25 Yr - 24 Hr Storm

Both the existing and modified basins are adequate to contain a 25 year-24 hour storm.

Subject Storm Runoff into Catch Basin

CALCULATION NOTES
By B.A.F.
Checked
Acc't
8-27 1984
Sheet No. 1 of 2 Sheets

All drawing and page references in this section refer to either drawings submitted in this section, or drawings and pages included in the Operation and Reclamation Plan - Wellington Coal Cleaning Plant - ACT/007/012.

* The entire drainage area is located within the Sn soil group (DWG. E9-3339).

* Due to the proximity to the PCE2 series, the Sn series is assumed to be soil group D (PP. B-18,19).

* Undisturbed areas have a 15% sage-grass cover. (Cover density is estimated at approx. 60% of actual cover noted in field reconnaissance, August 1984, to account for seasonal changes.) This corresponds to a hydrologic curve number of 84 (P. B-3).

* Disturbed areas have no appreciable cover, which corresponds to a hydrologic curve number of 89 (P. B-21,22). The Containment area within the basin also has a curve number of 89 because the pond is generally dry.

* A 10 year-24 hour storm is 1.82 inches. A 25 year-24 hour storm is 2.18 inches (P. B-17).

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Source: Estimated Return Periods for Short-Duration Precipitation in Utah, Utah State University, Logan, Utah, Department of Soils and Biometeorology, Bulletin No. 1, March, 1971.

ESTIMATED RETURN PERIODS FOR SHORT DURATION PRECIPITATION
(inches)

Station: Price
Latitude: 39° 37'

Elevation: 5680
Longitude: 110° 50'

DURATION

RETURN PERIOD (years)	5 Min	10 Min	15 Min	30 Min	1 Hr	2 Hr	3 Hr	6 Hr	12 Hr	24 Hr
1	.08	.13	.17	.23	.29	.37	.44	.62	.78	.95
2	.12	.18	.23	.32	.40	.49	.58	.80	1.00	1.20
5	.16	.25	.32	.44	.56	.68	.79	1.07	1.32	1.58
10	.20	.31	.39	.54	.68	.81	.94	1.25	1.53	1.82
25	.24	.37	.47	.65	.82	.98	1.13	1.50	1.83	2.18
50	.28	.43	.54	.75	.95	1.12	1.29	1.71	2.08	2.47
100	.31	.49	.62	.85	1.08	1.27	1.45	1.91	2.32	2.74

TABLE 3.—*Estimated soil properties significant to engineering*

[Badland (Ba), Gullied land (Gu), Mixed alluvial land (Mx), Riverwash (Rv), Rock land (Ry), Shaly colluvial land (Sn), and Stony alluvial land (St) are omitted from this table, because their properties generally are too variable to estimate. For Badland (Ba) and Rock land (Ry), the depth to a seasonal water table is more than 72 inches and bedrock is at or near the surface]

Soil series and map symbols	Depth to seasonal water table	Depth to bedrock	Depth from surface (typical profile)	Classification: USDA texture	Classification: Unified	Classification: AASHO	Percentage passing sieve— No. 4 (4.7 mm.)	No. 10 (1.0 mm.)	No. 200 (0.074 mm.)	Percentage larger than 3 inches	Permeability	Available water capacity	Reaction (paste)	Salinity	Dispersion	Shrink-swell potential	Hydrologic groupings
	Inches	*Inches*	*Inches*								*Inches per hour*	*Inches per inch of soil depth*	*pH*				
Abbott (Ab, As)	6-40	60+	0-60	Silty clay and silty clay loam.	CL	A-7	100	100	90-95	------	0.05-0.2	0.19	7.1-7.9	Slight to strong.	Low to moderate.	Moderate	D
Beebe (BbB, BeB, BeC2, BfA)	72+	60+	0-71	Loamy fine sand	SM	A-4	100	100	35-45	------	5.0-10.0	0.06-0.10	7.9-9.7	None to moderate.	None	Low	A
Billings (BlB, BlC2, BsB, BtB, BuB2). (For properties of the Bunderson soil in mapping unit BuB2, refer to the Bunderson series.)	([1])	60+	0-72	Silty clay loam	CL	A-6	100	100	85-95	------	0.05-0.2	0.17-0.2	7.6-8.6	Slight to moderate.	Moderate	Moderate	C
Bunderson. (Mapped only in complexes with the Billings and Ravola soils.)	72+	60+	0-72	Loam	CL	A-4	100	100	70-90	------	([3])	0.13-0.15	8.5-10.0	Moderate to high.	High	Low	D
Cache (Ca)	20-40	60+	0-60	Silty clay	CL	A-7	100	100	90-100	------	0.05-0.2	0.15-0.17	7.9-8.5	Very strong	High	Moderate	D
Castle Valley (CaE2)	72+	10-20	0-10	Very fine sandy loam	ML	A-4	95-100	85-100	50-60	5-10	2.5-5.0	0.13-0.15	7.5-8.0	None	Low	Low	D
			10	Sandstone.													
Cedar Mountain (CmF2)	72+	10-20	0-14	Shaly silty clay, clay loam, and silt loam	ML-CL	A-6	100	95-100	85-90	5-10	0.05-0.2	0.12-0.14	8.0-8.5	None	Moderate	Moderate	D
			14	Shale													
Chipeta (CBF2, CPB, CPE2). (For properties of the Persayo soils in mapping units CPB and CPE2, refer to the Persayo series. Badland in mapping unit CBF2 is too variable to rate.	72+	10-20	0-17	Silty clay loam	CL	A-6 or A-7	100	100	90-97	------	0.05-0.2	0.15-0.17	7.4-8.0	Moderate to strong.	Moderate	Moderate	D
			17	Shale													
Ferron: (Fr)	6-36	60+	0-60	Loam and very fine sandy loam	ML-CL	A-4	100	100	80-90	------	0.8-2.5	0.17-0.19	7.7-8.5	Slight to strong.	Low	Low	B
(Fe)	6-36	------	0-60	Silty clay loam and silt loam.	CL	A-6	100	100	90-95	----	0.2-0.8	0.17-0.19	7.7-8.5	Moderate	Moderate	Moderate	B
Green River (Gr)	20-40	60+	0-45	Stratified loam and very fine sandy loam	CL-ML	A-4	100	100	70-80	------	0.8-2.5	0.17-0.19	7.8-8.2	None to slight.	Low	Low	B
			45-60	Fine sand	SM	A-2	100	90-100	15-20	------	2.5-5.0	0.08-0.11	7.8-8.2	None to slight.	Low	None	B

See footnotes at end of table.

TABLE 3.—*Estimated soil properties significant to engineering*—Continued

Soil series and map symbols	Depth to seasonal water table	Depth to bedrock	Depth from surface (typical profile)	Classification: USDA texture	Classification: Unified	Classification: AASHO	Percentage passing sieve—: No. 4 (4.7 mm.)	Percentage passing sieve—: No. 10 (2.0 mm.)	Percentage passing sieve—: No. 200 (0.074 mm.)	Percentage larger than 3 inches	Permeability	Available water capacity	Reaction (paste)	Salinity	Dispersion	Shrink-swell potential	Hydrologic groupings
	Inches	*Inches*	*Inches*								*Inches per hour*	*Inches per inch of soil depth*	*pH*				
Harding (Ha)	72+	60+	0-10	Clay loam	CL	A-6	95-100	95-100	75-85		0.05-0.8	0.19-0.21	8.1-8.7	Moderate	High	Moderate	C
			10-20	Clay	CL	A-7	95-100	95-100	75-85		(3)	0.16-0.18	8.1-8.7	Moderate to strong.	High	Moderate	C
			20-32	Clay loam and loam	CL	A-6	75-80	65-75	70-80		0.8-2.5	0.16-0.18	8.3-8.7	Moderate to strong.	High	Moderate	C
Hunting (Ha, He, Hu)	20-40	60+	0-60	Loam	CL	A-4	100	100	70-80		0.8-2.5	0.17-0.19	7.8-8.3	Slight to strong.	Low	Moderate	B
Kenilworth (KeE2)	72+	60+	0-34	Stony sandy loam	SM	A-2 or A-4	50-75	45-70	25-40	20-50	0.8-2.5	0.10-0.12	7.7-8.5	None	Low	Low	B
Killpack (K B, KlC2, KmB, KpB, KpC2)	(1)	20-40	0-23	Clay loam or loam	ML-CL	A-4	100	100	70-80		0.2-0.8	0.19-0.21	7.7-8.0	Slight to moderate.	Moderate	Moderate	C
			23-29	Shaly silty clay loam	CL	A-6	80-95	70-90	65-85		0.2-0.8	0.19-0.20	7.7-8.0	Slight to moderate.	Moderate	Moderate	C
			29	Shale.													
Libbings (Lb, Lt)	10-30	20-40	0-34	Silty clay loam, clay, and silty clay.	CL	A-6	100	80-95	90-100		0.05-0.2	0.16-0.18	8.2-8.9	Very strong	Moderate	Moderate	D
			34	Shale.													
Minchey (McB, MlB, MsB, MsC2) (For properties of the Sanpete soils in mapping units MsB and MsC2, refer to the Sanpete series.)	72+	60+	0-32	Clay loam	CL	A-6	95-100	95-100	60-75		0.8-2.5	0.19-0.21	7.9-8.3	None	Low	Moderate	B
			32-64	Gravelly sandy loam	MG-GC	A-4 and A-2.	55-85	50-80	30-40	5	1.25-5.0	0.06-0.09	7.9-8.3	None	Low	Low	
Palisade (PaB, PdB, PdC2)	(4)	60+	0-41	Very fine sandy loam	ML-CL	A-4	85-95	80-90	50-60		0.8-2.5	0.17-0.19	7.3-8.0	None	Low	Low	B
			41-60	Very fine sandy loam	SM	A-4	85-95	80-90	35-45	0	2.5-5.0	0.07-0.10	7.5-8.0	None	Low	Low	B
Penoyer:																	
(PeB, PeC2, PnD, PnA, PsB, PsC2, PvB2)	72+	60+	0-60	Loam	CL	A-6	100	100	70-80		0.8-2.5	0.17-0.19	7.7-8.2	None	Low	Low	B
(PrA, PsB)			0-14	Silty clay loam	CL	A-4	100	100	90-95		0.2-0.8	0.19-0.21	7.7-8.2	None	Low	Moderate	B
			14-60	Loam	CL-ML	A-4	100	100	70-80		0.8-2.5	0.17-0.19	7.8-8.2	Slight	Low	Low	B
Persayo (PCE2) (For properties of the Chipeta soil in this mapping unit, refer to Chipeta series.)	72+	6-20	0-12	Loam and silty clay loam.	CL	A-4	80-85	70-80	65-75		0.8-2.5	0.17-0.19	7.5-8.0	Slight to strong.	Moderate	Moderate	D
			12	Shale.													
Rafael (Ra)	6-30	60+	0-70	Silty clay loam and loam	CL	A-4	100	95-100	75-85		0.05-0.2	0.17-0.19	7.7-8.6	Moderate to strong.	Moderate	Moderate	D
Ravola (RtB, RlB2, RlC2, RnD, RsA, RsB, RtB, RuB2) (For properties of the Billings soil in mapping unit RuB2, refer to the Billings series.)	72+	60+	0-60	Loam	ML-CL	A-4	100	100	75-85		0.8-2.5	0.17-0.19	7.7-8.0	None to moderate.	Low	Low	B
Saltair (Sa, Sb)	6-60	60+	0-60	Silty clay loam and silt loam	CL	A-4	100	100	85-95		0.05-0.2	0.16-0.18	8.3-8.9	Very strong	High	Moderate	D
Sanpete (S B, SrD2, SmD2) (For properties of the Minchey soil in mapping unit SmD2, refer to the Minchey series.)	72+	60+	0-14	Gravelly sandy clay loam.	ML-CL	A-4	99-100	90-100	50-60	10-20	2.5-5.0	0.10-0.13	7.9-8.5	None	Low	Low	A
			14-30	Very cobbly sandy clay loam.	SM	A-4	70-80	65-75	40-50	20	2.5-5.0	0.06-0.08	7.9-8.5	None	Low	Low	A
Woodrow (Wo)	72+	60+	0-60	Silty clay loam	CL	A-4	100	100	90-95		0.05-0.2	0.19-0.21	7.6-7.9	None to slight.	Low	Moderate	C

[1] In the Billings and Killpack soils, the seasonal water table is at a depth of more than 72 inches, except that it is between 38 and 60 inches in mapping unit BaB and between 10 and 40 inches in mapping unit KmB.

[2] Less than 0.06 inch per hour in surface layer; 0.8 to 2.5 inches per hour below a depth of 10 to 20 inches.

[3] Less than 0.05 inch per hour.

[4] In the Palisade soils, mapping unit PaB has a seasonal water table at a depth between 10 and 45 inches. In all other Palisade soils, the water table is lacking.

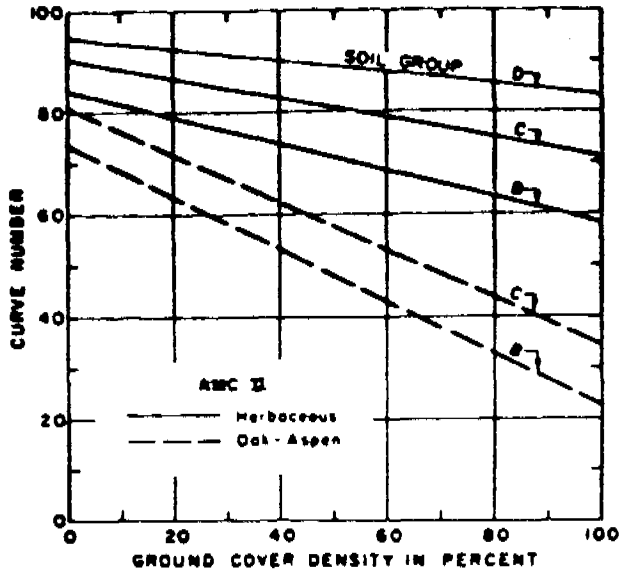


Figure 9.5.--Graph for estimating runoff curve numbers of forest-range complexes in western United States: herbaceous and oak-aspen complexes.

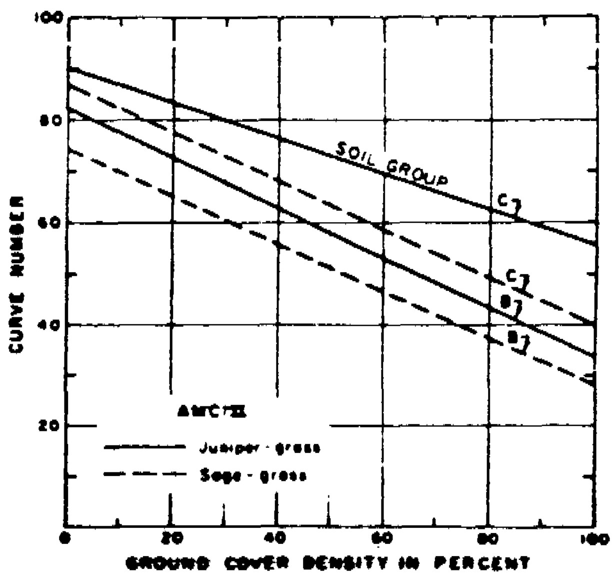
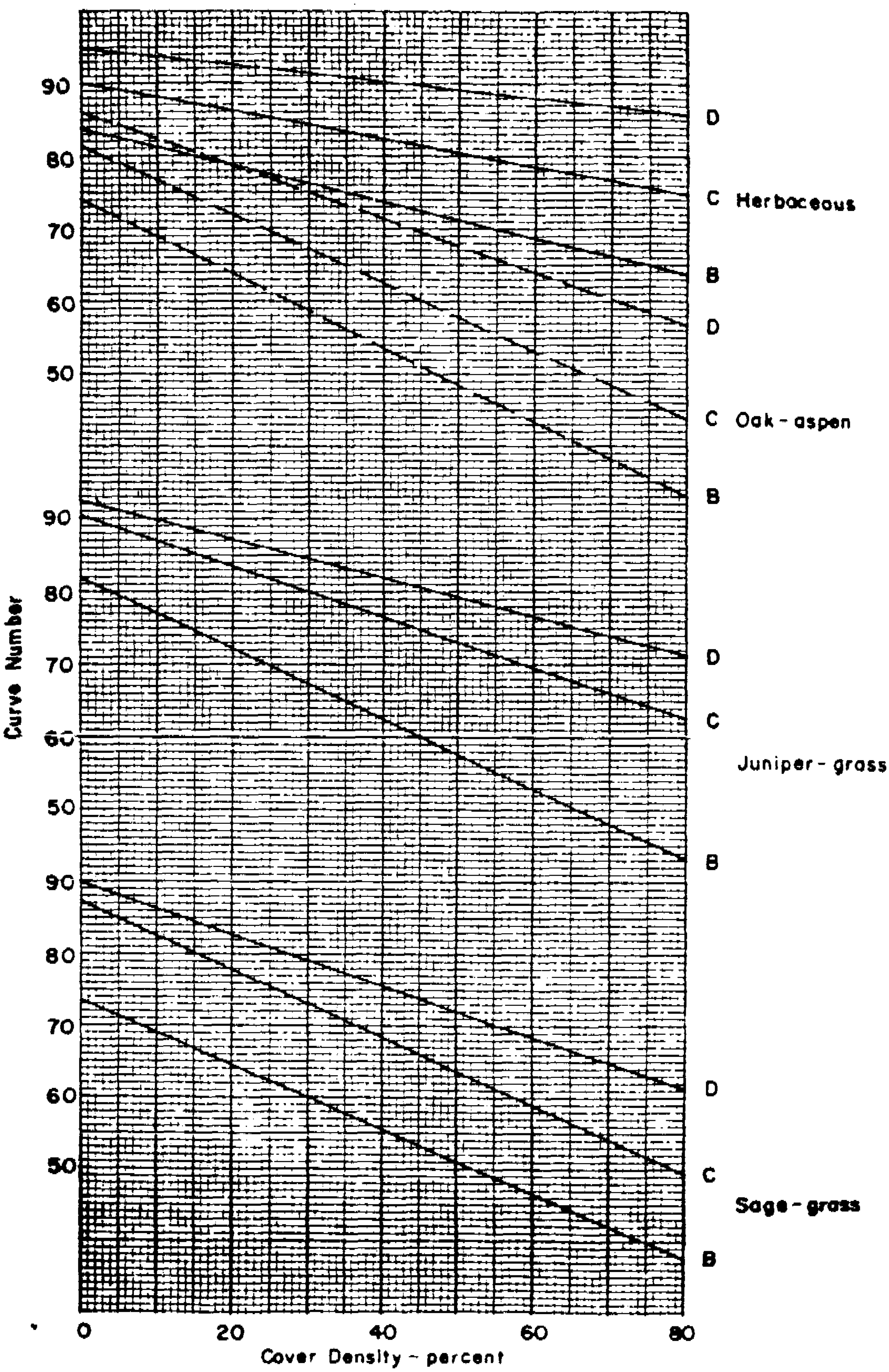


Figure 9.6.--Graph for estimating runoff curve numbers of forest-range complexes in western United States: juniper-grass and sage-grass complexes.



HYDROLOGIC SOIL COVER COMPLEX & ASSOCIATED CURVE NUMBERS

Suppliment to Figure 9.6

Table 2-2.--Runoff curve numbers for selected agricultural, suburban, and urban land use. (Antecedent moisture condition II, and $I_a = 0.2S$)

LAND USE DESCRIPTION	HYDROLOGIC SOIL GROUP			
	A	B	C	D
Cultivated land[1]: without conservation treatment	72	81	88	91
: with conservation treatment	62	71	78	81
Pasture or range land: poor condition	68	79	86	89
good condition	39	61	74	80
Meadow: good condition	30	58	71	78
Wood or Forest land: thin stand, poor cover, no mulch	45	66	77	83
good cover[2]	25	55	70	77
Open Spaces, lawns, parks, golf courses, cemeteries, etc.				
good condition: grass cover on 75% or more of the area	39	61	74	80
fair condition: grass cover on 50% to 75% of the area	49	69	79	84
Commercial and business areas (85% impervious)	89	92	94	95
Industrial districts (72% impervious).	81	88	91	93
Residential:[3]				
Average lot size — Average % Impervious[4]				
1/8 acre or less — 65	77	85	90	92
1/4 acre — 38	61	75	83	87
1/3 acre — 30	57	72	81	86
1/2 acre — 25	54	70	80	85
1 acre — 20	51	68	79	84
Paved parking lots, roofs, driveways, etc.[5]	98	98	98	98
Streets and roads:				
paved with curbs and storm sewers[5]	98	98	98	98
gravel	76	85	89	91
dirt	72	82	87	89

1/ For a more detailed description of agricultural land use curve numbers refer to National Engineering Handbook, Section 4, Hydrology, Chapter 9, Aug. 1972.

2/ Good cover is protected from grazing and litter and brush cover soil.

3/ Curve numbers are computed assuming the runoff from the house and driveway is directed towards the street with a minimum of roof water directed to lawns where additional infiltration could occur.

4/ The remaining pervious areas (lawn) are considered to be in good pasture condition for these curve numbers.

5/ In some warmer climates of the country a curve number of 95 may be used.

TABLE 10-1 Values of the Roughness Coefficient n

Channel material	n
Plastic, glass, drawn tubing	0.009
Neat cement, smooth metal	0.010
Planed timber, asbestos pipe	0.011
Wrought iron, welded steel, canvas	0.012
Ordinary concrete, asphalted cast iron	0.013
Unplaned timber, vitrified clay	0.014
Cast-iron pipe	0.015
Riveted steel, brick	0.016
Rubble masonry	0.017
Smooth earth	0.018
Firm gravel	0.023
Corrugated metal pipe	0.022
Natural channels in good condition	0.025
Natural channels with stones and weeds	0.035
Very poor natural channels	0.060

noted that V and Q are proportional to $1/n$ and S proportional to n^2 so that values from the nomograph may be readily adjusted to any other value of n.

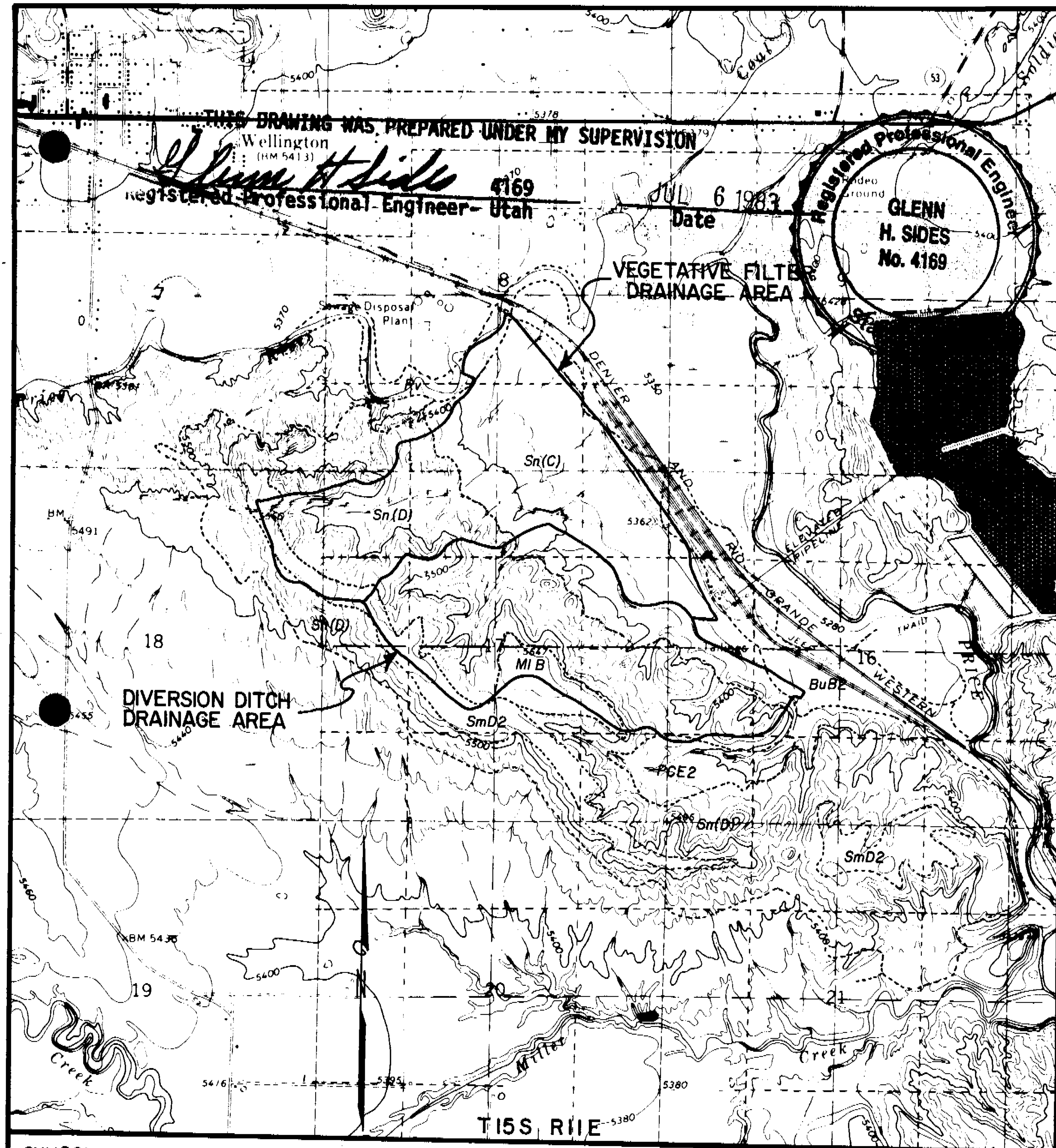
A situation often encountered in hydraulic engineering, particularly in the case of sewers, is that of a closed conduit flowing partly full. Under this condition the liquid surface is at atmospheric pressure, and the flow is the same as that in an open channel. It is often inconvenient to compute R and A for partially full sections, and it is simpler to calculate V or Q for the pipe flowing full and to adjust to partly full conditions by use of a chart[1] such as Fig. 10-3. When the depth of flow in a circular pipe increases above $0.8D$, the wetted perimeter increases more rapidly than the cross-sectional area because of the convergence of the pipe walls. Hence R, and consequently V, decreases. Maximum discharge occurs when $d = 0.94D$.

10-2 Normal depth Normal depth d_n is the depth at which uniform flow will occur in an open channel. Normal depth may be determined by writing the Manning equation for discharge,

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2} \qquad (10\text{-}5)$$

and substituting for A and R expressions involving d and other necessary dimensions of the channel cross section. The resulting equation requires

[1] The roughness coefficient n varies somewhat with depth of flow. This variation is reflected in Fig. 10-3. See "Design and Construction of Sanitary and Storm Sewers," *ASCE Manual of Practice 37*, 2d ed., pp. 87–95, American Society of Civil Engineers, New York, 1969.



SYMBOL	SOIL SERIES	HYD. GROUP
BuB2	Billings-Bunderson Complex	C
MI B	Minchey Loam	B
PCE2	Persayo-Chipeta Association	D
R[illegible]	Riverwash	C
[illegible]2	Sanpete-Minchey Complex	D
Sn(C)	Shaley Colluvial Land	C
Sn(D)	Shaley Colluvial Land	D

Source: SOIL SURVEY Carbon-Emery Area Utah, SCS & BLM

U. S. STEEL MINING COMPANY, INC.
WESTERN DISTRICT
WELLINGTON COAL CLEANING PLANT

DRAINAGE AREAS FOR HYDROLOGIC EVALUATION

Reference Drawings: F9-177

DRAWN:		APPROVED:	
B.A.F.	6·21·83	G.H.S.	6·21·83

SCALE: 1" = 2000'

A9-1431

PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION

SLOPES - STEEP
CURVE NUMBER - 85

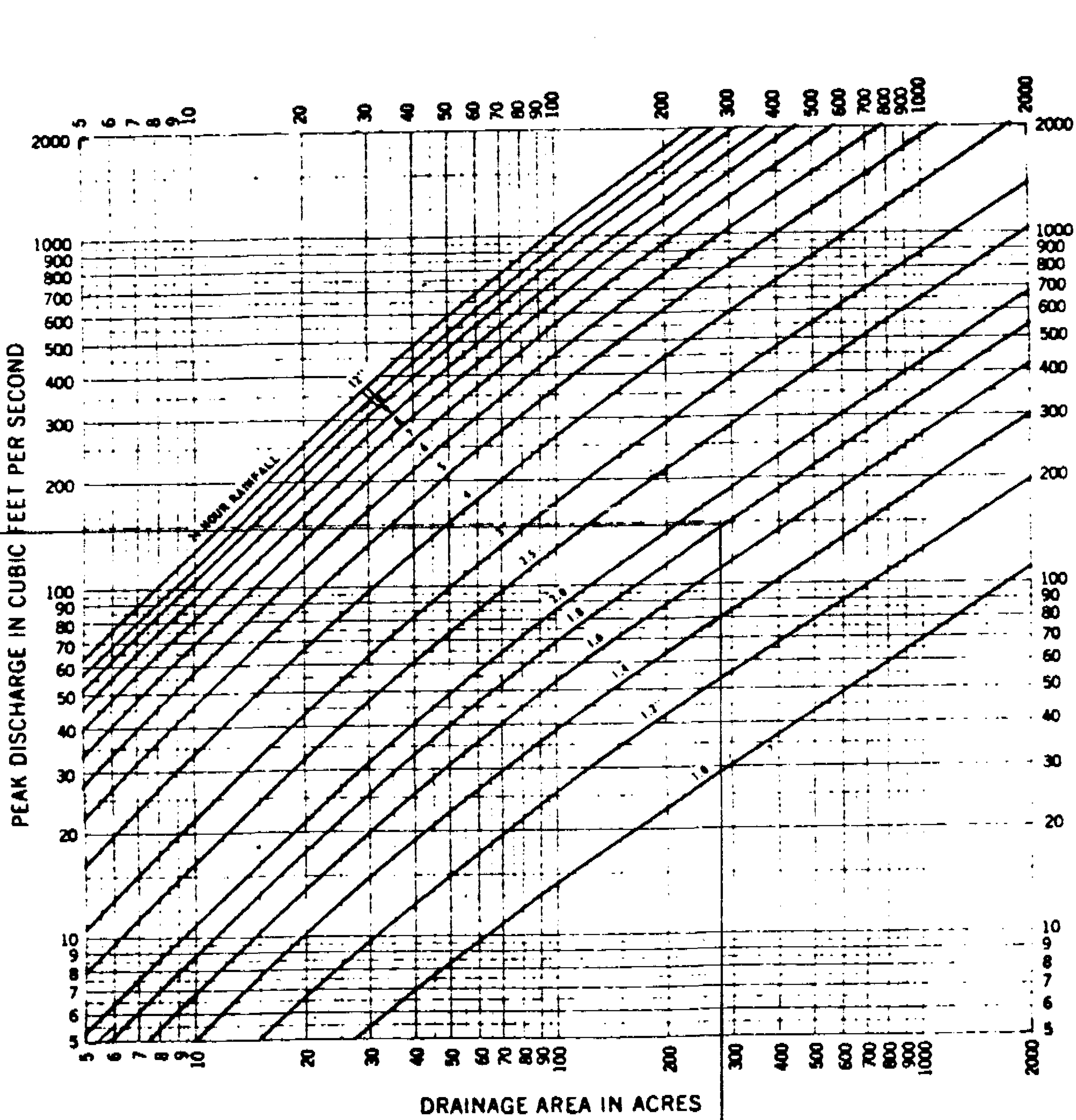


Exhibit 2-10

EFM Notice-4, 5/71

REFERENCE		STANDARD DWG. NO.
Chapter 2, Engineering Field Manual for Conservation Practices	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING DIVISION HYDROLOGY BRANCH	ES-1027 SHEET 20 OF 2_ DATE 2-15-71

PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION

SLOPES - STEEP
CURVE NUMBER - 80

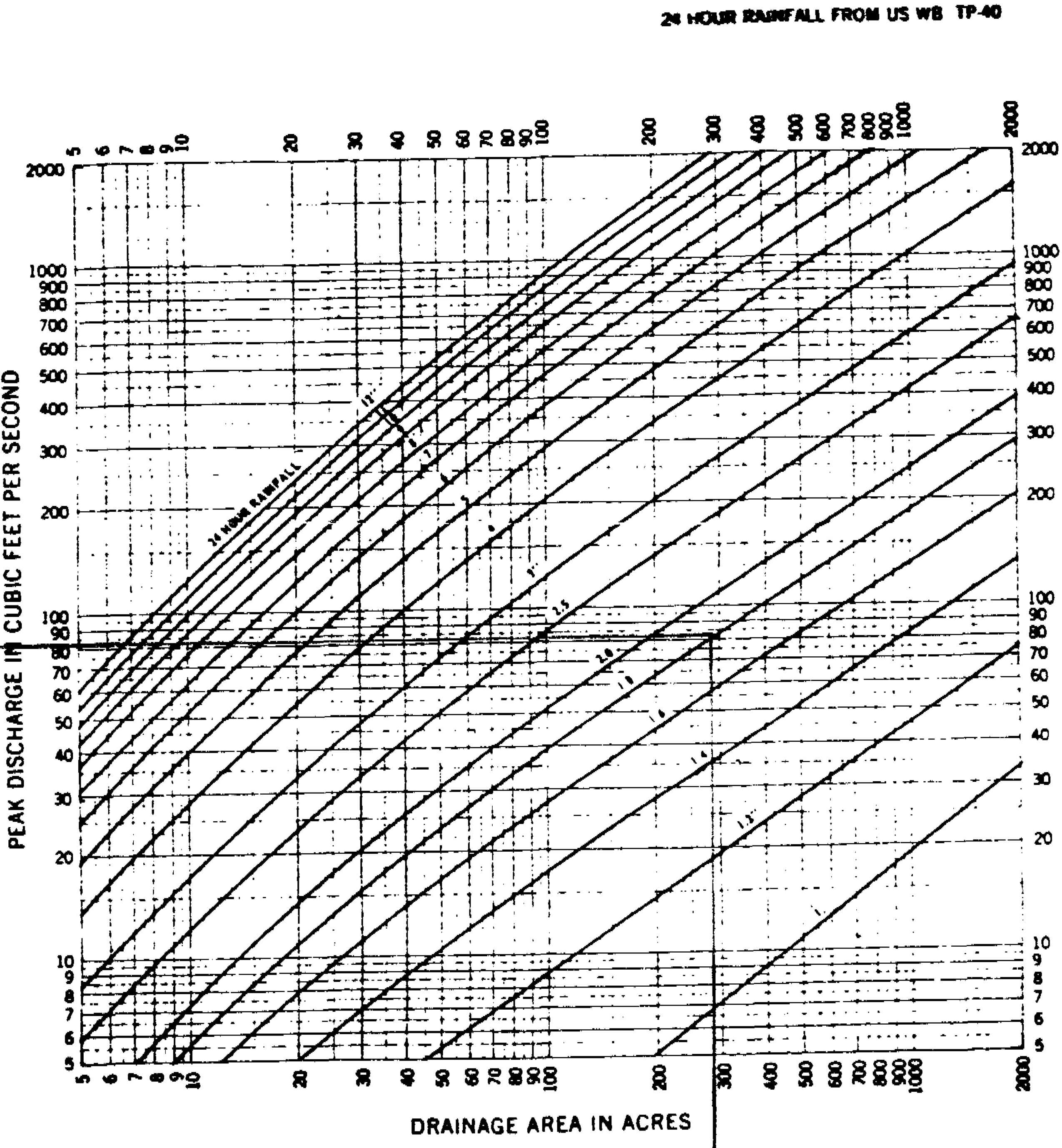


Exhibit 2-10

EFM Notice-4, 5/71

REFERENCE		STANDARD DWG. NO.
"Chapter 2, Engineering Field Manual for Conservation Practices"	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING DIVISION - HYDROLOGY BRANCH	ES-1027 SHEET 19 OF 21 DATE 2-19-71

GEOFAB™ SILT FENCE

PATENT PENDING

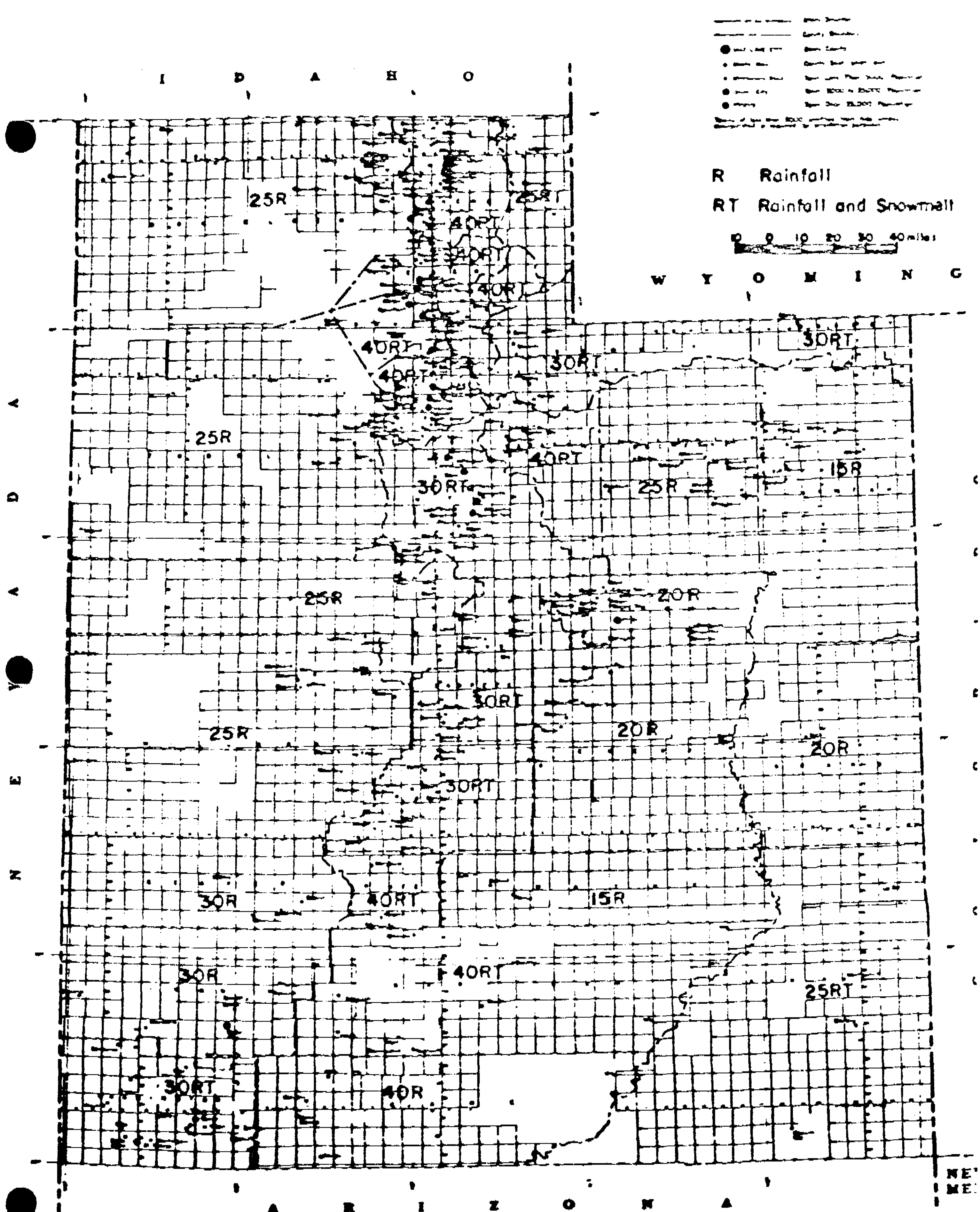
TYPICAL FABRIC PROPERTIES*

Material	100% spunbonded nylon reinforced with polyester netting	
Weight	4.2 oz./yd.²	ASTM D 1910
Thickness	10 mils	ASTM D 1977
Grab Tensile	130 lbs.	ASTM D 1682
Elongation To Break	25% plus or minus 3%	ASTM D 1682
Mullen Burst	210 lbs.	ASTM D 774-46
Water Flow Rate	470 gallons per minute/foot²	
E.O.S.	Equivalent Opening Size 70-100 U.S. Corps of Engineers Guide Spec. CW02215 Nov. 1977	
Cord	1/8" nylon braided installed full length of fabric	
Strength Retention	Ultraviolet treated for outdoor application	
Fabric Width	Supplied in widths to meet local State D.O.T. and/or Federal requirements	
Standard Roll Length	150 ft.	

*The above properties are average results and should not be construed as minimum or maximum properties.

For more information on GEOFAB™ Silt Fence or other civil engineering fabrics in the GEOFAB™ line, please contact Mercantile Development, Inc. at (203) 226-7803, or write:

MERCANTILE DEVELOPMENT, INC.
274 Riverside Avenue
Westport, CT 06880



GENERALIZED R & RT VALUES

UNIVERSAL SOIL LOSS EQUATION

STATE: UTAH | RECORD NO: | AUTHOR(S): WN-RB | DATE: 8-73 | REVISED: | UNIT MODIFIER: DEEP WATER TABLE | KIND OF UNIT: PHASE | UNIT NAME: BILLINGS

CLASSIFICATION AND BRIEF SOIL DESCRIPTION

THE BILLINGS SERIES, DEEP WATER TABLE PHASE, ARE DEEP, MODERATELY WELL DRAINED SOILS FORMED IN ALLUVIUM FROM SHALE ON FANS AND FLOOD PLAINS UNDER GALLETA GRASS, SALTGRASS, SHADSCALE AND GREASEWOOD. THE MEAN ANNUAL TEMPERATURE IS 47 TO 52 F. AVERAGE ANNUAL PRECIPITATION IS 7 TO 11 INCHES. FROST FREE PERIOD IS 110 TO 160 DAYS. THE SURFACE LAYER IS LIGHT BROWNISH-GRAY SILTY CLAY LOAM 8 TO 14 INCHES THICK. THE UNDERLYING LAYER IS LIGHT BROWNISH-GRAY SILTY CLAY LOAM TO 60 INCHES. SLOPES RANGE FROM 1 TO 3 PERCENT.

ESTIMATED SOIL PROPERTIES

DEPTH (IN.)	USDA TEXTURE	UNIFIED	AASHO	FRACT. >3 IN. (PCT)	Passing sieve 4	10	40	200	LIQUID LIMIT	PLASTICITY INDEX
0-60	SICL	CL	A-6	0	100	100	95-100	90-95	25-40	7-20

(Percent of material less than 3 in. passing sieve)

DEPTH (IN.)	PERMEABILITY (IN/HR)	AVAILABLE WATER CAPACITY (IN/IN)	SOIL REACTION (pH)	SALINITY (MMHOS/CM)	SHRINK-SWELL POTENTIAL	CORROSIVITY STEEL	CORROSIVITY CONCRETE	EROSION FACTORS K	EROSION FACTORS T	WIND EROD. GROUP
SAME DEPTH AS ABOVE	0.06-0.2	0.17-0.2	7.4-9.0	4-16	MODERATE	HIGH	HIGH	.43	5	4L

FLOODING FREQUENCY	FLOODING DURATION	FLOODING MONTHS	HIGH WATER TABLE DEPTH (FT)	HIGH WATER TABLE KIND	HIGH WATER TABLE MONTHS	CEMENTED PAN DEPTH (IN)	CEMENTED PAN HARDNESS	BEDROCK DEPTH (IN)	BEDROCK HARDNESS	SUBSIDENCE INITIAL (IN)	SUBSIDENCE TOTAL (IN)	HYD GRP	POTENTIAL FROST ACTION
RARE	V. BRIEF	JUL-AUG	2.5-4.0	APPARENT	JAN-DEC	-	-	>60				C	HIGH

SANITARY FACILITIES

Use	Footnote	Rating
SEPTIC TANK ABSORPTION FIELDS		SEVERE - PERCS SLOWLY, WET
SEWAGE LAGOONS	1	SEVERE - WET
SANITARY LANDFILL (TRENCH)		SEVERE - WET MODERATE - TOO CLAYEY, FLOODS
SANITARY LANDFILL (AREA)		SEVERE - WET MODERATE - FLOODS
DAILY COVER FOR LANDFILL		FAIR - TOO CLAYEY

COMMUNITY DEVELOPMENT

Use	Footnote	Rating
SHALLOW EXCAVATIONS		MODERATE - WET, FLOODS
DWELLINGS WITHOUT BASEMENTS	1	SEVERE - FROST ACTION, FLOODS MODERATE - WET, LOW STRENGTH, SHRINK-SWELL
DWELLINGS WITH BASEMENTS	1	SEVERE - WET, FLOODS MODERATE - SHRINK-SWELL, LOW STRENGTH
SMALL COMMERCIAL BUILDINGS		
LOCAL ROADS AND STREETS	1	SEVERE - FROST ACTION MODERATE - SHRINK-SWELL, FLOODS, LOW STRENGTH

SOURCE MATERIAL

Keying only	Use	Rating
FILL 19	ROADFILL	POOR - FROST ACTION FAIR - SHRINK-SWELL, WET, LOW STRENGTH
SAND 20	SAND	UNSUITED
GRAVEL 21	GRAVEL	UNSUITED
SOIL 22	TOPSOIL	POOR - EXCESS SALT FAIR - TOO CLAYEY

WATER MANAGEMENT

Keying only	Use	Limitations
PONDRS 23	POND RESERVOIR AREA	WET
DIKES 24	EMBANKMENTS DIKES AND LEVEES	LOW STRENGTH, PIPING, HARD TO PACK
PONDAQ 25	EXCAVATED PONDS AQUIFER FED	SLOW REFILL, DEEP TO WATER
DRAIN 26	DRAINAGE	EXCESS SALT, PERCS SLOWLY
IRRIG 27	IRRIGATION	EXCESS SALT, WET
TERRAC 28	TERRACES AND DIVERSIONS	NOT NEEDED
WATERW 29	GRASSED WATERWAYS	NOT NEEDED

REGIONAL INTERPRETATIONS

FOOTNOTES

SOIL SURVEY INTERPRETATIONS

MLRA(S) [illegible] — KIND OF UNIT: SERIES — UNIT NAME: [illegible] LOAM

[illegible] UTAH — PROJECT NO. — AUTHOR(S): [illegible] — DATE: 2-75 — REVISED — UNIT MODIFIER: 1 TO 3% SLOPES

[illegible] AND BRIEF SOIL DESCRIPTION

[illegible]AGON SERIES ARE VERY DEEP, WELL-DRAINED, CALCAREOUS, STRONGLY ALKALI SOILS FORMED IN ALLUVIUM FROM MARINE SHALES AND SANDSTONE ON ALLUVIAL FANS AND FLOOD PLAINS UNDER SPARSE STANDS OF PICKLEWEED AND GREASEWOOD. MAST IS 47 TO 54°F. MAP IS 6 TO 10 INCHES. FFP IS 110 TO 130 DAYS. A TYPICAL PROFILE HAS A LIGHT GRAY AND LIGHT BROWNISH-GRAY LOAM SURFACE LAYER 11 INCHES THICK. THE UNDERLYING LAYER IS LIGHT BROWNISH-GRAY SILT LOAM AND LOAM TO 60 INCHES OR MORE. SLOPES ARE 1 TO 3 PERCENT.

ESTIMATED SOIL PROPERTIES

DEPTH (IN.)	USDA TEXTURE	UNIFIED	AASHO	FRACT. >3 IN. (PCT)	PERCENT OF MATERIAL LESS THAN 3 IN. PASSING SIEVE: 4	10	40	200	LIQUID LIMIT	PLASTICITY INDEX
0-11	L	CL, CL-ML	A-4	0	100	100	85-95	60-75	20-30	5-10
11-60	L	CL, CL-ML	A-4	0	100	100	85-95	60-75	20-30	5-10

DEPTH (IN.)	PERMEABILITY (IN/HR)	AVAILABLE WATER CAPACITY (IN/IN)	SOIL REACTION (PH)	SALINITY (MMHOS/CM)	SHRINK-SWELL POTENTIAL	CORROSIVITY: STEEL	CONCRETE	EROSION FACTORS: K	T	WIND EROD. GROUP
SAME DEPTHS AS ABOVE	0.06 - 0.2	0.02 - 0.12	>9.0	8 - >16	LOW	HIGH	HIGH	.55	1	4L
	0.6 - 2.0	0.02 - 0.12	8.5 - 9.0	8 - >16	LOW	HIGH	HIGH	.55		

AWC = 1.0 - 7.0

WSC = 2.0 - 4.0

PERMEABILITY IS SLOW

FLOODING: FREQUENCY	DURATION	MONTHS	HIGH WATER TABLE: DEPTH (FT)	KIND	MONTHS	CEMENTED PAN: DEPTH (IN)	HARDNESS	BEDROCK: DEPTH (IN)	HARDNESS	SUBSIDENCE: INITIAL (IN)	TOTAL (IN)	HYD GRP	POTENTIAL FROST ACTION
NONE			>6.0			-		>60		-		D	LOW

SANITARY FACILITIES

SEPTIC TANK ABSORPTION FIELDS	MODERATE - PERCS SLOWLY
SEWAGE LAGOONS	1-2%: MODERATE - SEEPAGE 2+%: MODERATE - SLOPE, SEEPAGE
SANITARY LANDFILL (TRENCH)	MODERATE - SEEPAGE
SANITARY LANDFILL (AREA)	SLIGHT
DAILY COVER FOR LANDFILL	GOOD

COMMUNITY DEVELOPMENT

SHALLOW EXCAVATIONS	SLIGHT
DWELLINGS WITHOUT BASEMENTS	MODERATE - LOW STRENGTH
DWELLINGS WITH BASEMENTS	MODERATE - LOW STRENGTH
SMALL COMMERCIAL BUILDINGS	MODERATE - LOW STRENGTH
LOCAL ROADS AND STREETS	MODERATE - LOW STRENGTH

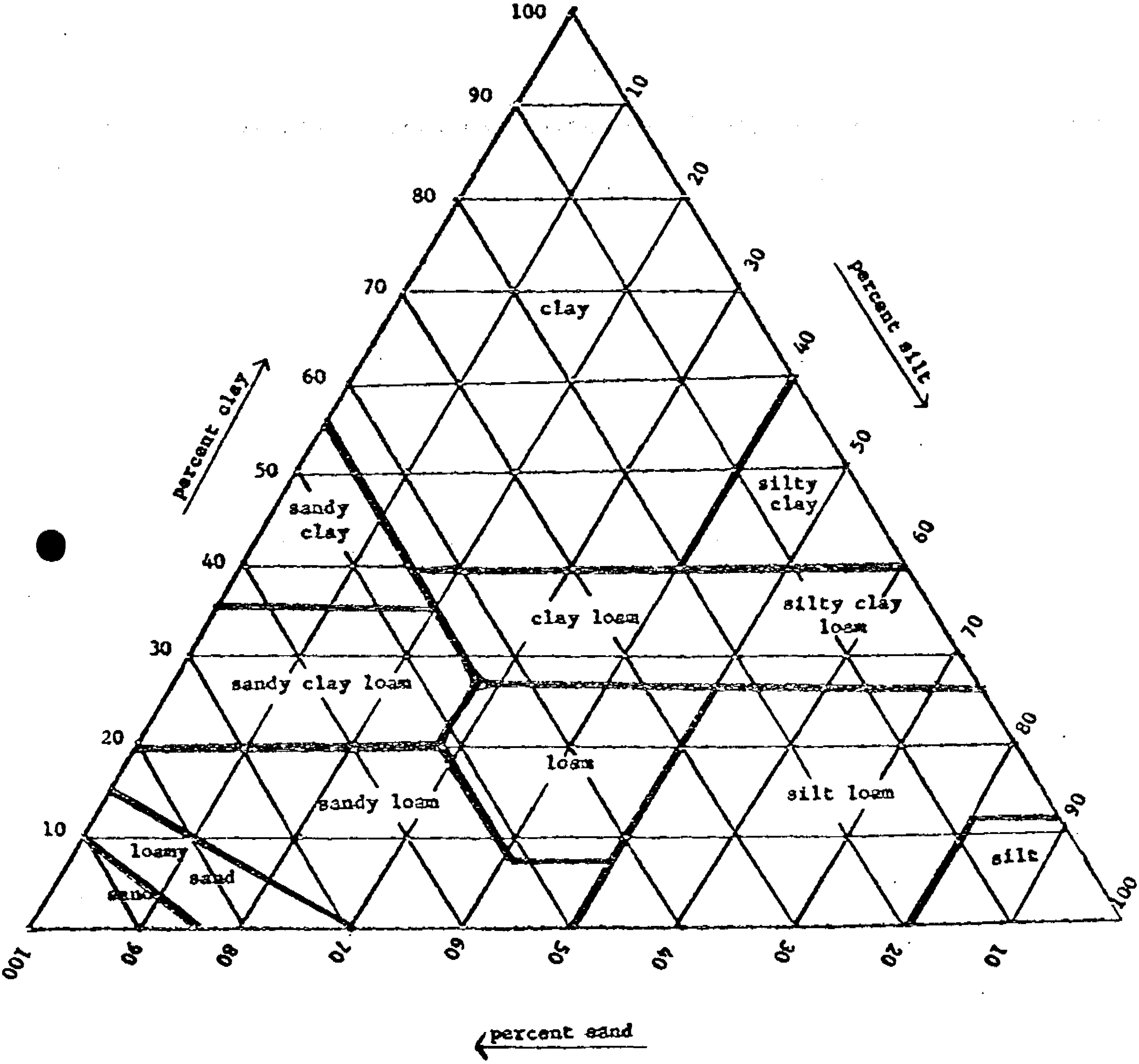
SOURCE MATERIAL

ROADFILL	FAIR - LOW STRENGTH
SAND	UNSUITED
GRAVEL	UNSUITED
TOPSOIL	POOR - EXCESS SALT, EXCESS ALKALI

WATER MANAGEMENT

POND RESERVOIR AREA	SEEPAGE
EMBANKMENTS DIKES AND LEVEES	LOW STRENGTH, PIPING
EXCAVATED PONDS AQUIFER FED	NO WATER
DRAINAGE	
IRRIGATION	
TERRACES AND DIVERSIONS	ERODES EASILY
GRASSED WATERWAYS	ERODES EASILY, EXCESS SALT, EXCESS ALKALI

REGIONAL INTERPRETATIONS


100
90
80
70
60
50
40
30
20
10
percent clay
10
20
30
40
50
60
70
80
90
100
percent silt
100
90
80
70
60
50
40
30
20
10
percent sand
clay
sandy clay
silty clay
clay loam
silty clay loam
sandy clay loam
loam
sandy loam
silt loam
loamy sand
sand
silt

Typical topsoil K Values[1] (SCS, 1978).

Surface Layer Texture	Estimated K Value
Clay, clay loam, loam, silty clay	.32
Fine sandy loam, loamy very fine sand, sandy loam	.24
Loamy fine sand, loamy sand	.17
Sand	.15
Silt loam, silty clay loam, very fine sandy loam	.37

1/ Note: These values are typical based only on textural information. Values for an actual soil can be considerably different due to different structure and infiltration.

Source: "Applied Hydrology and Sedimentology for Disturbed Areas"; Barfield, Warner and Haan; 1981, Table 5-5, p. 330.

Percent Slope	Slope Length in Feet 300	400	500	600	700	800	900	1000	1100	1200	1300	1500	1700	2000
0.2	0.11	0.12	0.13	0.14	0.15	0.15	0.16	0.16	0.17	0.17	0.18	0.19	0.19	0.20
0.3	0.12	0.13	0.14	0.15	0.16	0.16	0.17	0.18	0.18	0.18	0.19	0.20	0.21	0.22
0.4	0.13	0.14	0.15	0.16	0.17	0.17	0.18	0.19	0.19	0.20	0.20	0.21	0.22	0.23
0.5	0.14	0.15	0.16	0.17	0.18	0.18	0.19	0.20	0.20	0.21	0.21	0.22	0.23	0.24
1.0	0.18	0.20	0.21	0.22	0.23	0.24	0.25	0.26	0.27	0.27	0.28	0.29	0.30	0.32
2.0	0.28	0.31	0.33	0.34	0.36	0.38	0.39	0.40	0.41	0.42	0.43	0.45	0.47	0.49
3.0	0.40	0.44	0.47	0.49	0.52	0.54	0.56	0.57	0.59	0.61	0.62	0.65	0.67	0.71
4.0	0.62	0.70	0.76	0.82	0.87	0.92	0.96	1.01	1.04	1.08	1.12	1.18	1.24	1.33
5.0	0.93	1.07	1.20	1.31	1.42	1.52	1.61	1.69	1.78	1.86	1.93	2.07	2.21	2.40
6.0	1.17	1.35	1.50	1.65	1.78	1.90	2.02	2.13	2.23	2.33	2.43	2.61	2.77	3.01
8.0	1.72	1.98	2.22	2.43	2.62	2.81	2.98	3.14	3.29	3.44	3.58	3.84	4.09	4.44
10.0	2.37	2.74	3.06	3.36	3.62	3.87	4.11	4.33	4.54	4.74	4.94	5.30	5.65	6.13
12.0	3.13	3.61	4.04	4.42	4.77	5.10	5.41	5.71	5.99	6.25	6.51	6.99	7.44	8.07
14.0	3.98	4.59	5.13	5.62	6.07	6.49	6.88	7.26	7.61	7.95	8.27	8.89	9.46	10.26
16.0	4.92	5.68	6.35	6.95	7.51	8.03	8.52	8.98	9.42	9.83	10.24	11.00	11.71	12.70
18.0	5.95	6.87	7.68	8.41	9.09	9.71	10.30	10.86	11.39	11.90	12.38	13.30	14.16	15.36
20.0	7.07	8.16	9.12	9.99	10.79	11.54	12.24	12.90	13.53	14.13	14.71	15.80	16.82	18.24
25.0	10.20	11.78	13.17	14.43	15.59	16.66	17.67	18.63	19.54	20.41	21.24	22.82	24.29	26.35
30.0	13.78	15.91	17.79	19.48	21.04	22.50	23.86	25.15	26.38	27.55	28.68	30.81	32.80	
40.0	21.92	25.31	28.30	31.00	33.48									
50.0	30.87													
60.0														

SLOPE-EFFECT TABLE (TOPOGRAPHIC FACTOR, LS)

Percent Slope	Slope Length in Feet													
	10	20	40	60	80	100	110	120	130	140	150	160	180	200
0.2	0.04	0.05	0.06	0.07	0.08	0.08	0.08	0.09	0.09	0.09	0.09	0.09	0.10	0.10
0.3	0.04	0.05	0.07	0.08	0.08	0.09	0.09	0.09	0.09	0.10	0.10	0.10	0.10	0.11
0.4	0.05	0.06	0.07	0.08	0.09	0.09	0.10	0.10	0.10	0.10	0.11	0.11	0.11	0.11
0.5	0.05	0.06	0.08	0.08	0.09	0.10	0.10	0.10	0.11	0.11	0.11	0.11	0.12	0.12
1.0	0.06	0.08	0.10	0.11	0.12	0.13	0.13	0.14	0.14	0.14	0.15	0.15	0.15	0.16
2.0	0.10	0.12	0.15	0.17	0.19	0.20	0.21	0.21	0.22	0.22	0.23	0.23	0.24	0.25
3.0	0.14	0.18	0.22	0.25	0.27	0.29	0.30	0.30	0.31	0.32	0.32	0.33	0.34	0.35
4.0	0.16	0.21	0.28	0.33	0.37	0.40	0.42	0.43	0.44	0.46	0.47	0.48	0.51	0.53
5.0	0.17	0.24	0.34	0.41	0.48	0.54	0.56	0.59	0.61	0.63	0.66	0.68	0.72	0.76
6.0	0.21	0.30	0.43	0.52	0.60	0.67	0.71	0.74	0.77	0.80	0.82	0.85	0.90	0.95
8.0	0.31	0.44	0.63	0.77	0.89	0.99	1.04	1.09	1.13	1.17	1.21	1.25	1.33	1.40
10.0	0.43	0.61	0.87	1.06	1.23	1.37	1.44	1.50	1.56	1.62	1.68	1.73	1.84	1.94
12.0	0.57	0.81	1.14	1.40	1.61	1.80	1.89	1.98	2.06	2.14	2.21	2.28	2.42	2.55
14.0	0.73	1.03	1.45	1.78	2.05	2.29	2.41	2.51	2.62	2.72	2.81	2.90	3.08	3.25
16.0	0.90	1.27	1.80	2.20	2.54	2.84	2.98	3.11	3.24	3.36	3.48	3.59	3.81	4.01
18.0	1.09	1.54	2.17	2.66	3.07	3.43	3.60	3.76	3.92	4.06	4.21	4.34	4.61	4.86
20.0	1.29	1.82	2.58	3.16	3.65	4.08	4.28	4.47	4.65	4.83	5.00	5.16	5.47	5.77
25.0	1.86	2.63	3.73	4.56	5.27	5.89	6.18	6.45	6.72	6.97	7.22	7.45	7.90	8.33
30.0	2.52	3.56	5.03	6.16	7.11	7.95	8.34	8.71	9.07	9.41	9.74	10.06	10.67	11.25
40.0	4.00	5.66	8.00	9.80	11.32	12.65	13.27	13.86	14.43	14.97	15.50	16.01	16.98	17.90
50.0	5.64	7.97	11.27	13.81	15.94	17.82	18.69	19.53	20.32	21.09	21.83	22.55	23.91	25.21
60.0	7.32	10.35	14.64	17.93	20.71	23.15	24.28	25.36	26.40	27.39	28.36	29.29	31.06	32.74

"C" VALUES FOR PERMANENT PASTURE, RANGELAND, AND IDLE LAND 1/

Vegetal Canopy			Cover that Contacts the Surface					
Type and Height of Raised Canopy 2/	Canopy Cover 3/ %	Type 4/	Percent Ground Cover 0	20	40	60	80	95-100
Column No.	2	3	4	5	6	7	8	9
No appreciable canopy		G	.45	.20	.10	.042	.013	.003
		W	.45	.24	.15	.090	.043	.011
Canopy of tall weeds or short brush (0.5 m fall ht.)	25	G	.36	.17	.09	.038	.012	.003
		W	.36	.20	.13	.082	.041	.011
	50	G	.26	.13	.07	.035	.012	.003
		W	.26	.16	.11	.075	.039	.011
	75	G	.17	.10	.06	.031	.011	.003
		W	.17	.12	.09	.067	.038	.011
Appreciable brush or bushes (2 m fall ht.)	25	G	.40	.18	.09	.040	.013	.003
		W	.40	.22	.14	.085	.042	.011
	50	G	.34	.16	.085	.038	.012	.003
		W	.34	.19	.13	.081	.041	.011
	75	G	.28	.14	.08	.036	.012	.003
		W	.28	.17	.12	.077	.040	.011
Trees but no appreciable low brush (4 m fall ht.)	25	G	.42	.19	.10	.041	.013	.003
		W	.42	.23	.14	.087	.042	.011
	50	G	.39	.18	.09	.040	.013	.003
		W	.39	.21	.14	.085	.042	.011
	75	G	.36	.17	.10	.039	.012	.003
		W	.36	.20	.13	.083	.041	.011

1/ All values shown assume: (1) random distribution of mulch or vegetation, and (2) mulch of appreciable depth where it exists.

2/ Average fall height of waterdrops from canopy to soil surface: m = meters.

3/ Portion of total-area surface that would be hidden from view by canopy in a vertical projection, (a bird's-eye view).

4/ G: Cover at surface is grass, grasslike plants, decaying compacted duff, or litter at least 2 inches deep.

W: Cover at surface is mostly broadleaf herbaceous plants (as weeds) with little lateral-root network near the surface, and/or undecayed residue.

PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION

SLOPES - FLAT
CURVE NUMBER - 75

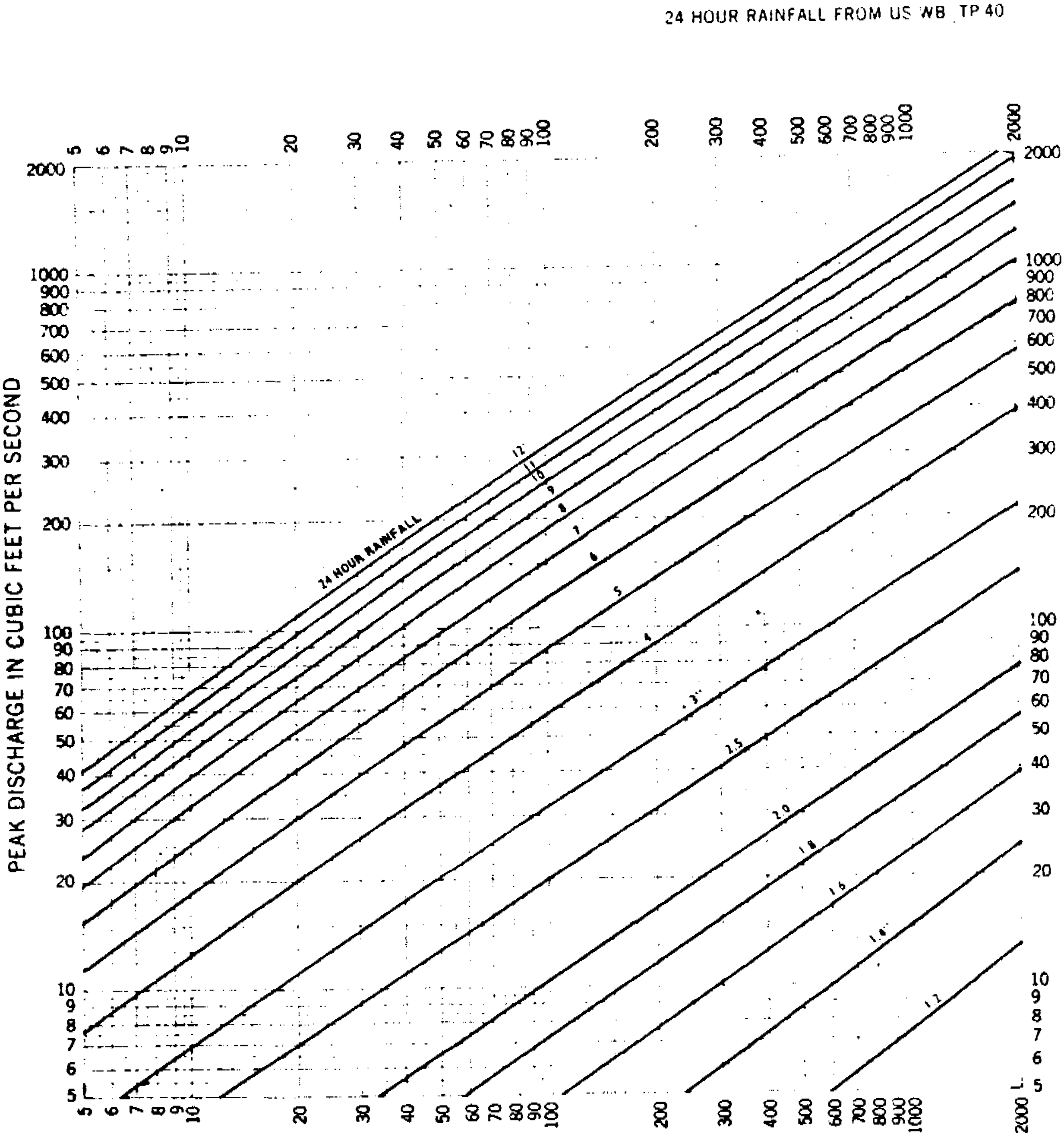


Exhibit 2-10

EFM Notice-4, 5/71

REFERENCE		STANDARD DWG. NO.
"Chapter 2. Engineering Field Manual for Conservation Practices"	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING DIVISION - HYDROLOGY BRANCH	ES-1027 SHEET 4 OF 21 DATE 2-15-71

PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION

SLOPES - FLAT
CURVE NUMBER - 80

24 HOUR RAINFALL FROM US WB TP-40

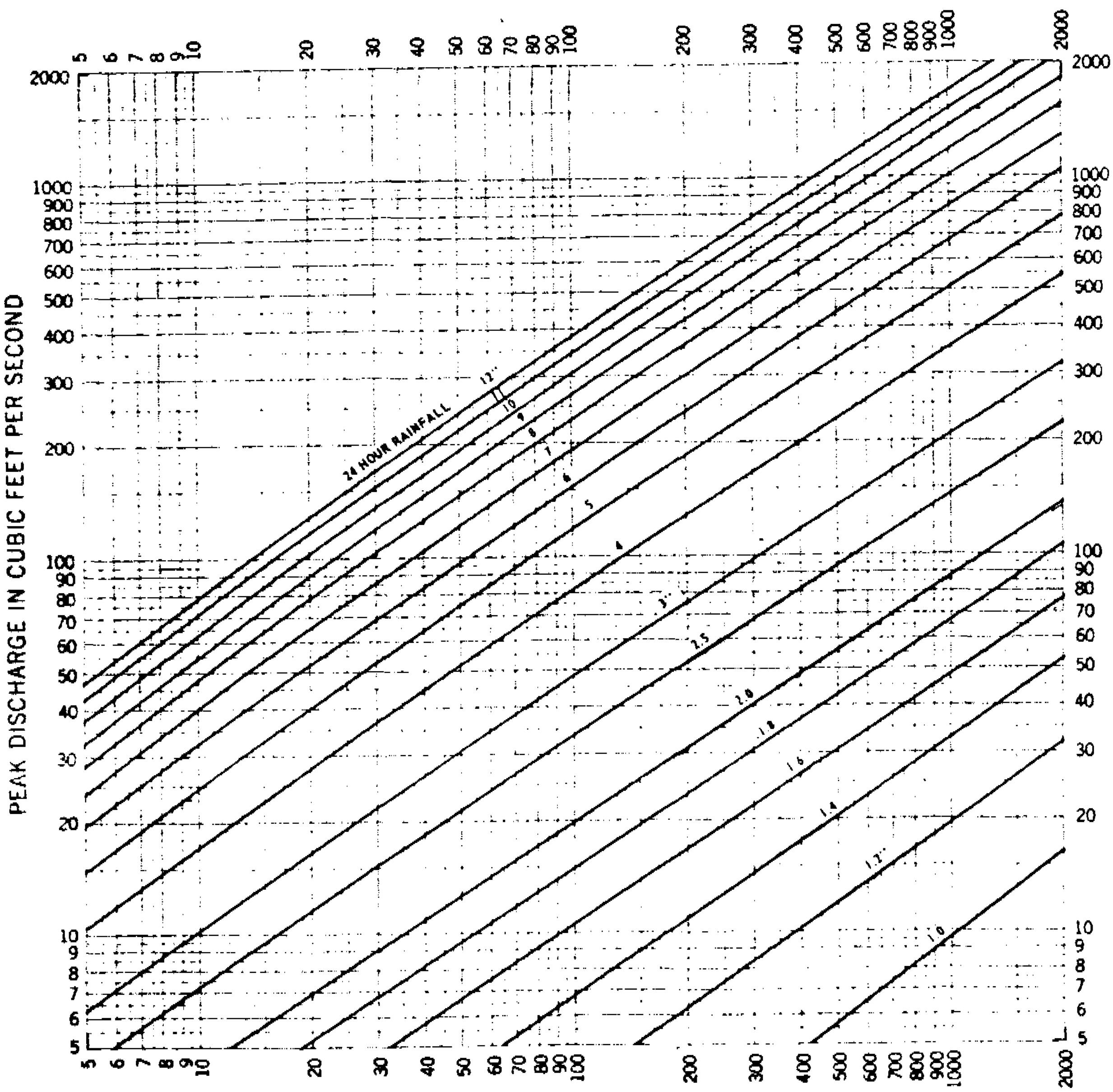


Exhibit 2-10

EFM Notice-4, 5/71

REFERENCE		STANDARD DWG. NO.
"Chapter 2, Engineering Field Manual for Conservation Practices"	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING DIVISION - HYDROLOGY BRANCH	ES-1027 SHEET 5 OF 21 DATE 2-15-71

PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION

SLOPES - FLAT
CURVE NUMBER - 85

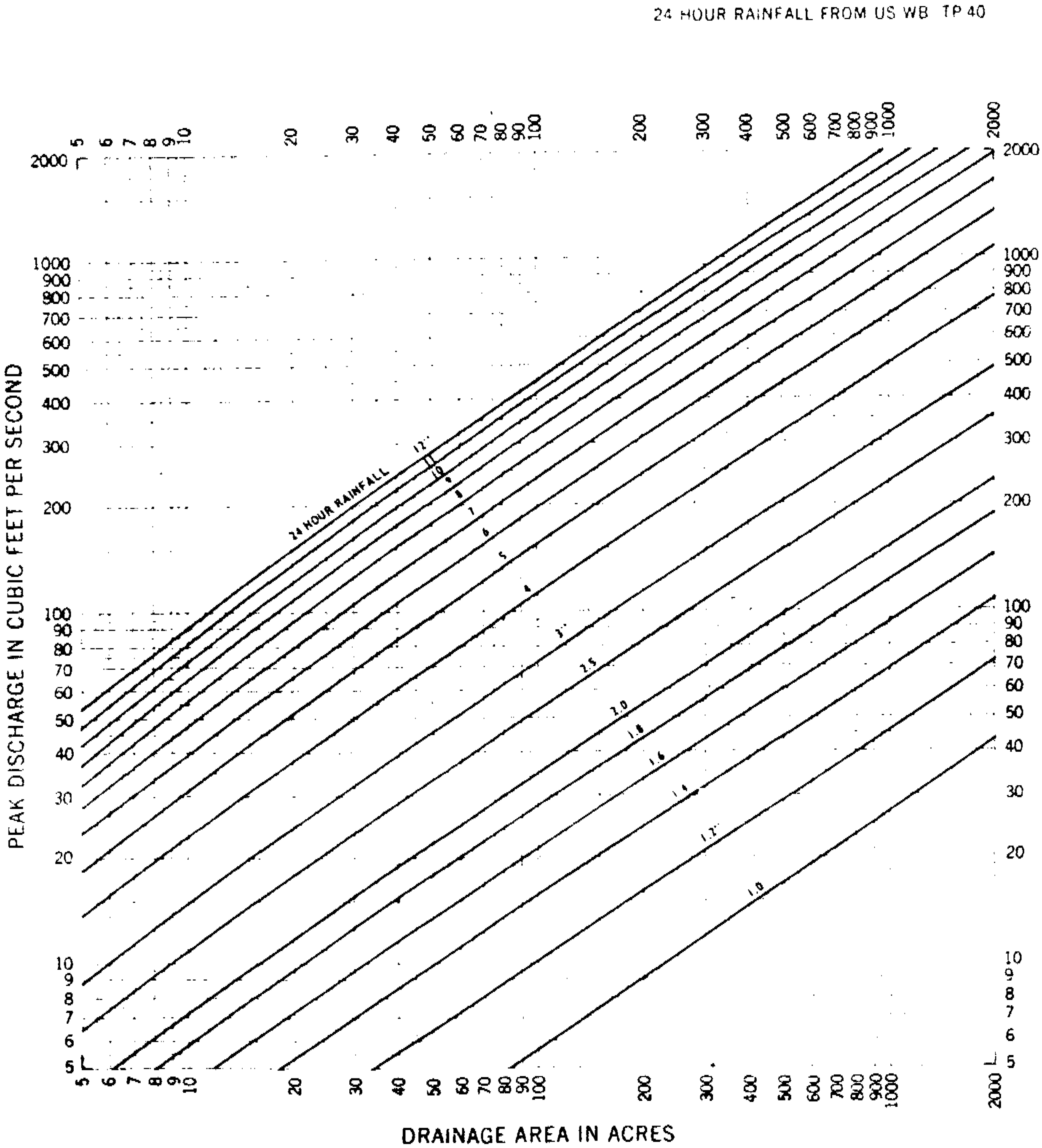


Exhibit 2-10

EFM Notice-4, 5/71

REFERENCE		STANDARD DWG. NO.
"Chapter 2, Engineering Field Manual for Conservation Practices"	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING DIVISION - HYDROLOGY BRANCH	ES-1027 SHEET 6 OF 21 DATE 2-15-71

PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION

SLOPES - FLAT
CURVE NUMBER - 90

24 HOUR RAINFALL FROM US WB TP 40

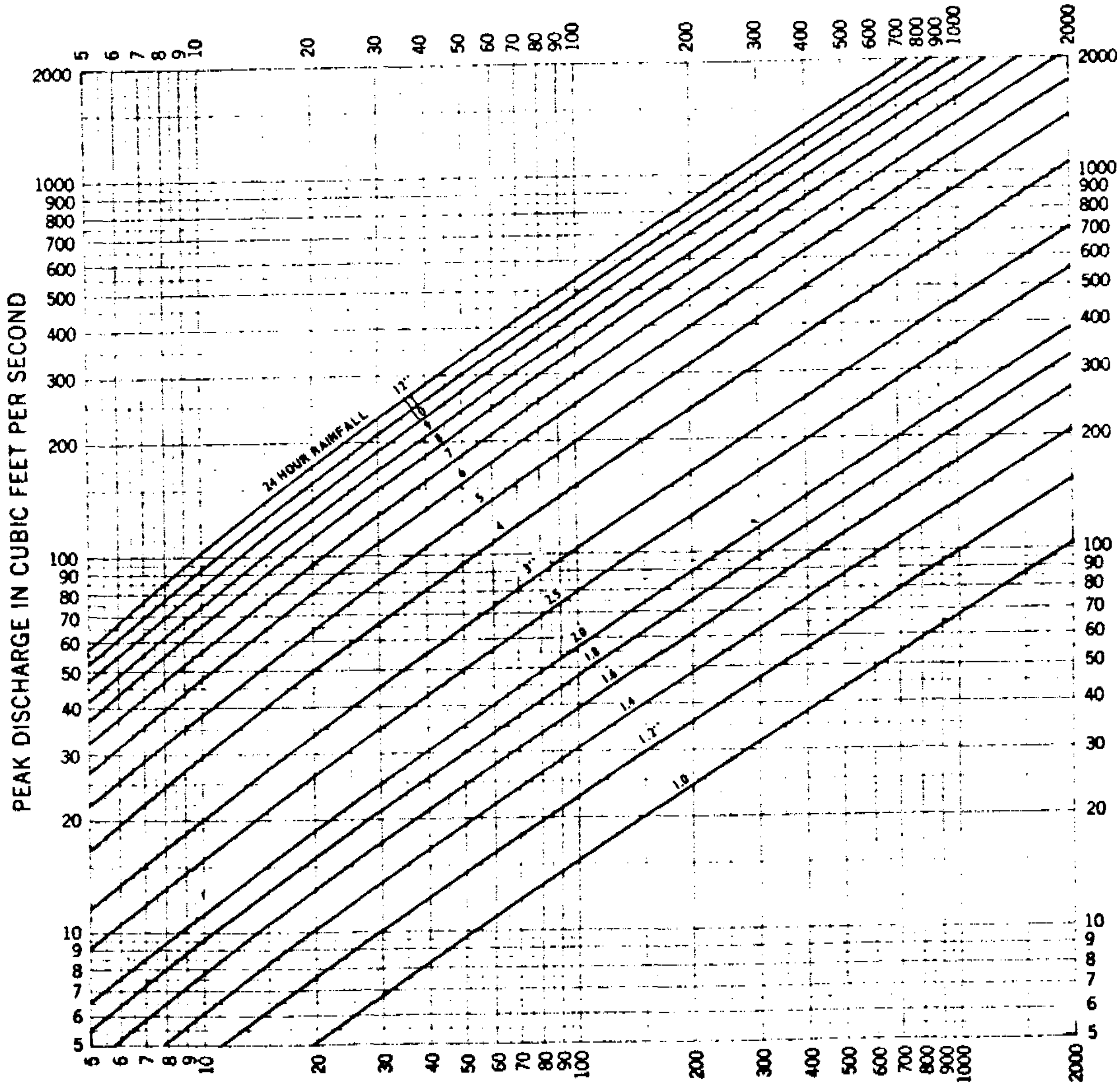


Exhibit 2-10

EFM Notice-4, 5/71

REFERENCE		STANDARD DWG. NO.
"Chapter2 , Engineering Field Manual for Conservation Practices"	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING DIVISION HYDROLOGY BRANCH	ES-1027 SHEET 7 OF 21 DATE 2-15-71

CALCULATION NOTES

Subject Culvert Adequacy for Vegetative Filter Drainage Area (A9-1431) - Culverts Installed in Parallel Under Access Road

By BAF
Checked
Acc't
Date December 18 1983
Sheet No. 1 of 5 Sheets

Culvert capacities are calculated using the Manning Equation for Discharge: Q = 1.486/n AR 2/3 S1/2 where Q = Capacity in CFS, n - roughness coefficient, A = area in square feet, R - hydraulic radius = area/wetted perimeter, and S = slope in feet per feet.

Culvert No.(1)	Culvert Material	Roughness Coefficient(2)	Dia. (in)	Slope (ft/ft)	Area (ft^2)	Wetted Perimeter (ft)	Capacity (cfs)
C1	Corrugated	0.022	24	0.0333	3.14	6.28	24.39
C2	Corrugated	0.022	18	0.0472	1.77	4.71	13.63
C5	Corrugated	0.022	18	0.0315	1.77	4.71	11.13
C8	Cement	0.010	24	0.0040	3.14	6.28	18.60
				TOTAL CAPACITY			67.75 cfs

Peak flows through culverts are based on a 10 year - 24 hour precipitation event. Peak flow values are developed from SCS nomographs based on drainage area, drainage slope and hydrologic curve numbers as follows:

Drainage Area	Acres(3)	Curve No.(4)	Peak Flow(CFS)(5)
Vegetative Filter	310	82	44

Culvert capacities exceed the 10 year - 24 hour peak flow

Subject Culvert Adequacy for Vegetative Filter Drainage Area (A9-1431) - Culverts Installed in Parallel under Raw Coal Tracks

CALCULATION NOTES

By BAF

Checked

Acc't

Date December 18 1983

Sheet No. 2 of 5 Sheets

Culvert capacities are calculated using the Manning Equation for Discharge: Q = 1.486/n AR 2/3 S1/2 where Q = Capacity in CFS, n - roughness coefficient, A = area in square feet, R - hydraulic radius = area/wetted perimeter, and S = slope in feet per feet.

Culvert No.(1)	Culvert Material	Roughness Coefficient(2)	Dia. (in)	Slope (ft/ft)	Area (ft^2)	Wetted Perimeter (ft)	Capacity (cfs)
C3	Cement	0.010	24	0.0020	3.14	6.28	13.15
C9	Cement	0.010	24	0.0023	3.14	6.28	14.10
C14	Cement	0.010	24	0.0100	3.14	6.28	29.41
				TOTAL CAPACITY			56.66 cfs

Note: There is one additional 24" culvert under the D&RGW mainline in parallel with these culverts.

Peak flows through culverts are based on a 10 year - 24 hour precipitation event. Peak flow values are developed from SCS nomographs based on drainage area, drainage slope and hydrologic curve numbers as follows:

Drainage Area	Acres(3)	Curve No.(4)	Peak Flow(CFS)(5)
Vegetative Filter	310	82	44

Culvert Capacities exceed the 10 year 24 hour peak flow

CALCULATION NOTES

Subject Culvert Adequacy for Culverts Entering the Auxiliary Pond (F9-177)

By BAF
Checked
Acc't
Date December 18 19 83
Sheet No. 3 of 5 Sheets

Culvert capacities are calculated using the Manning Equation for Discharge: Q = 1.486/n AR 2/3 S1/2 where Q = Capacity in CFS, n - roughness coefficient, A = area in square feet, R - hydraulic radius = area/wetted perimeter, and S = slope in feet per feet.

Culvert No.(1)	Culvert Material	Roughness Coefficient(2)	Dia. (in)	Slope (ft/ft)	Area (ft^2)	Wetted Perimeter (ft)	Capacity (cfs)
C10	Cement	0.010	36	0.0010	7.07	9.42	27.44
C11	Cement	0.010	24	0.0069	3.14	6.28	24.43

Note: Two culverts, one under the main access road and one under the access to the car dumping station at the track hopper, are evaluated together (C10) because they act in series with no significant incramental drainage area.

Peak flows through culverts are based on a 10 year - 24 hour precipitation event. Peak flow values are developed from SCS nomographs based on drainage area, drainage slope and hydrologic curve numbers as follows:

Drainage Area	Acres(3)	Curve No.(4)	Peak Flow(CFS)(5)
C10	0.45	78	<5
C11	2.27	86	<5

Note: Culvert C11 will not be subject to peak flow runoff after the Road Pond is constructed.

Culvert capacities exceed the 10 year 24 hour peak flow

CALCULATION NOTES

Subject Culvert Adequacy for Culvert Under Road to Refuse Pile (F9-177) and Culvert Under Road by Oil House (F9-177)

By BAF
Checked
Acc't
Date December 18 1983
Sheet No. 4 of 5 Sheets

Culvert capacities are calculated using the Manning Equation for Discharge: Q = 1.486/n AR 2/3 S1/2 where Q = Capacity in CFS, n - roughness coefficient, A = area in square feet, R - hydraulic radius = area/wetted perimeter, and S = slope in feet per feet.

Culvert No.(1)	Culvert Material	Roughness Coefficient(2)	Dia. (in)	Slope (ft/ft)	Area (ft^2)	Wetted Perimeter (ft)	Capacity (cfs)
C12	Corrugated	0.022	24	0.0300	3.14	6.28	23.15
C13	Corrugated	0.022	18	0.0100	1.77	4.71	6.27

Peak flows through culverts are based on a 10 year - 24 hour precipitation event. Peak flow values are developed from SCS nomographs based on drainage area, drainage slope and hydrologic curve numbers as follows:

Drainage Area	Acres(3)	Curve No.(4)	Peak Flow(CFS)(5)
C12	7.40	83	<5
C13	1.85	87	<5

Culvert capacities exceed the 10 year 24 hour peak flow

Subject Culvert Adequacy for Vegetative Filter Drainage Area (A9-1431) - Culverts Which Allow Railroad Culverts to Act in Parallel

CALCULATION NOTES
By BAF
Checked
Acc't
Date December 18 1983
Sheet No. 5 of 5 Sheets

Culvert capacities are calculated using the Manning Equation for Discharge: Q = 1.486/n AR 2/3 S1/2 where Q = Capacity in CFS, n - roughness coefficient, A = area in square feet, R - hydraulic radius = area/wetted perimeter, and S = slope in feet per feet.

Culvert No.(1)	Culvert Material	Roughness Coefficient(2)	Dia. (in)	Slope (ft/ft)	Area (ft^2)	Wetted Perimeter (ft)	Capacity (cfs)
C4	Cement	0.010	24	0.0141	3.14	6.28	34.92
C6	Corrugated	0.022	24	0.0320	3.14	6.28	23.91
C7	Corrugated	0.022	18	0.0100	1.77	4.71	6.27

Culverts are adequate to equalize runoff

References:

(1) Drawing No. F9-177 Sheet 1 of 2
(2) Page B-23
(3) Drawing No. A9-1341 for Vegetative Filter Drainage Area; Drawing No. F9-177 for all other drainage areas
(4) Drawing No. F9-177 and page B-3
(5) Pages B-35 thru B-39

DESCRIPTION OF POST MINING DRAINAGE

West of the Price River

During reclamation, all structures and facilities, excluding the Road Pond and Heat Dryer Pond, will be removed and the surface graded to the configuration shown on E9-3342. It is the Operators intent to maintain approximately the same drainage patterns shown on F9-177:

Approximately 310 acres drain through the northwest end of the permit area and pass through a vegetative filter between the D&RGW mainline and the river (see A9-1431). The area disturbed by the Operator is minimal compared to the entire drainage area. The disturbed areas will not require surface grading, but will be revegetated according to the revegetation plan. Post mining drainage patterns will continue to be coursed through the vegetative filter both during and after reclamation.

Approximately 281 acres of undisturbed land drain into the diversion ditch west of the plant area. The Operator intends to maintain this ditch during surface reclamation to minimize surface flow over newly graded areas. Long range plans for the coarse refuse pile encroach on the outlet basin of this ditch. The Operator has committed to submit plans in a timely manner for extending this ditch past the refuse pile as a technical revision when it becomes necessary to inundate the outlet basin. During reclamation, the drainage pattern shown on A9-1431 and F9-177 will be maintained. When reclamation is considered successful in accord with the success standards, the ditch will be graded to the configuration shown on E9-3342, where all drainage will pass through the culverts under the D&RGW mainline.

The drainage ditch adjacent to the east side of the course refuse pile has a drainage area of some 13 acres. This area primarily includes the equipment and material storage yard. The Operator does not anticipate any surface grading in this area except on the Course Refuse Pile. Areas will be revegetated in accord with the revegetation plan. During reclamation, the silt fence (or equal sediment filter) will be maintained in the ditch drainage to trap any sediment carried by runoff. When revegetation is successful, the sediment filter will be removed and the ditch slopes graded. Surface drainage will continue to flow this direction after reclamation because of the presence of the clean coal tracks which belong to the Denver and Rio Grande Western Railroad.

The Road Pond will be maintained during reclamation for sediment control. Since the plant area will require extensive building demolition and grading it will be necessary to maintain the pond until the area is adequately revegetated.

The 6 acre drainage area shown on F9-177 will be graded such that runoff from the area will enter the Road Pond. The culvert between the Auxiliary Pond and Road Pond will be removed at the time the Auxiliary Pond is graded. The Operator will then install the discharge structure, described in this section, for pond dewatering (E9-3429). When revegetation is successful, the decant structure and pipe will be removed and the pond graded to the configuration shown on E9-3342. Surface drainage will flow to the culvert under the tracks by the Heat Dryer.

The Heat Dryer Pond will be maintained during reclamation for sediment control. Dryer demolition and grading necessitates that the pond be maintained until the area is revegetated. The one acre drainage area shown on F9-177 will be graded to course surface runoff into the pond. The scrubber discharge ditch and outside sump will be removed during building demolition and the decant structure and pipe described in this section will be installed (E9-3433). When revegetation is successful, the decant will be removed and the pond graded to the configuration shown on E9-3342. Surface drainage will then course through the adjacent culvert under the D&RGW tracks and into the fields by the river.

East of the Price River

When the slurry pond area is reclaimed, two diversion ditches will channel the runoff from undisturbed areas around the regraded surface. The drainage areas shown on Technical Revision No. 1 for the pond area will be maintained, except for the diversion ditch around the lower pond. Referring to F9-177, and TR No. 1:

The pump house and deep well will be demolished, graded and revegetated in accord with the plan. The silt fence (or equal sediment filter) will be maintained until a successful stand of vegetation is established. Once revegetated, the sediment filter will be removed and surface runoff will course into the Price River.

Refer to TR No. 1 for a discussion on the permanent diversion ditch adjacent to the North Dike.

A permanent diversion is proposed in this section to channel runoff from the undisturbed area away from the reclaimed pond surface. This ditch will outlet into the Clear Water Pond during reclamation so that the impoundment does not have to be altered. When revegetation is successful, the Clear Water Pond will be reclaimed and the diversion ditch channel extended to discharge directly into the Price River. why

The slurry pond area will be graded such that as much of the surface runoff as is possible will drain into the Clear Water Pond. When reclamation of the ponds is successful, the impoundment will be graded to the configuration shown on E9-3342. After the Clear Water dike is graded, surface runoff will pass through the culvert (shown on C9-1292) under the county road and discharge in the undisturbed area between the road and the river.

Revegetation success standards are discussed in the reclamation plan. The Operator will sample surface water inflows into sediment control structures. When inflows meet effluent limitations and revegetation is considered successful, sediment control structures will be removed.

The area is generally flat both east and west of the Price River. When sediment control structures are removed, surface runoff will course toward the Price River from all areas.

All runoff from the reclaimed areas west of the river must pass through a culvert under the D&RGW mainline. Because the area is flat, flow velocities are not expected to be significant. Drainage channels are expected to be stable and erosion will not be significant.

The major drainages east of the river are diverted away from the reclaimed pond surface. Erosion protection and channel stability are evaluated in the ditch design. The grade over the reclaimed pond surface is not significant and drainage channels are expected to be stable.

Subject MAINTAIN ROAD POND FOR SEDIMENTATION CONTROL DURING RECLAMATION

CALCULATION NOTES
By BAF
Checked
Acc't
12-20 1983
Sheet No. 1 of 5 Sheets

THE 6.37 ACRE DRAINAGE AREA INTO THE ROAD POND IS SHOWN ON F9-177. SURFACE GRADING DURING RECLAMATION WILL ENSURE APPROPRIATE DRAINAGE INTO THE POND.

FOR RECLAMATION, THE FOLLOWING CHANGES ARE MADE TO THE HYDROLOGIC EVALUATION ON B-7:

- ACREAGES ASSIGNED TO BLACKTOPPED AREAS AND BUILDING ROOFS WILL BE GRADED WITH LOCAL SOIL (BuB2 AND/OR Sm). ASSUMING NO VEGETAL COVER, B-3 INDICATES A CN = 87 FOR THE 2.42 ACRES.

- ACREAGES ASSIGNED TO THE POND SURFACE AREA WILL BE ADJUSTED TO CORRESPOND TO MAINTAINING THE ROAD POND AND RECLAIMING THE AUXILIARY POND. THE EVALUATION ON B-7 ASSUMES THE CONDITION WHERE THE AUXILIARY POND CONTAINS WATER AND THE ROAD POND IS DRY. SINCE THE SURFACE AREA OF THE TWO PONDS IS NOT SIGNIFICANTLY DIFFERENT, THE 0.43 AC IS ASSIGNED A CN=100.

RUNOFF IS CALCULATED ACCORDING TO THE METHODS DESCRIBED ON PAGE B-2 AND SHOWN ON B-7. THE WEIGHTED FOR RECLAMATION IS CN = 87. THEREFORE:

$$Q_{10\text{-}24} = \frac{(1.82 - 0.2(1.49))^2}{1.82 + 0.8(1.49)} = 0.769 \text{ in}$$

$$Q_{25\text{-}24} = \frac{(2.18 - 0.2(1.49))^2}{2.18 + 0.8(1.49)} = 1.050 \text{ in}$$

Subject MAINTAIN ROAD POND
& SEDIMENTATION CONTROL
DURING RECLAMATION

CALCULATION NOTES
By BAF
Checked
Acc't
12-20 1983
Sheet No. 2 of 5 Sheets

$$VOLUME_{10\cdot 24} = \frac{(0.769\,IN)(6.37\,AC)(43560\,ft^2/AC)}{12\,IN/ft} = \underline{\underline{17,782\,cf}}$$

$$VOLUME_{25\cdot 24} = \frac{(1.050\,IN)(6.37\,AC)(43560\,ft^2/AC)}{12\,IN/ft} = \underline{\underline{24,279\,cf}}$$

SOIL LOSS CALCULATION WILL NOW INCLUDE THE 2.42 ACRES WHICH WERE NOT INCLUDED ON B-7 (BLDGS & BLACKTOP):

$A = RKLSC$

$A = (20)(0.50)(0.04)(0.45)$

$= 0.180\ T/AC/YR$

$$3\,YR\ SOIL\ LOSS = \frac{(0.180\,T/AC/YR)(2000\,\#/T)(5.94\,AC)(3\,YR)}{85\,\#/ft^3} = \underline{\underline{75\,cf}}$$

CAPACITY REQUIREMENT:

10 YR 24 HR = 17782 + 75 = **17,857 cf**

25 YR 24 HR = 24279 + 75 = **24,354 cf**

ROAD POND WILL CONTAIN THE 25 YR 24 HR EVENT

Subject MAINTAIN ROAD POND & SEDIMENTATION CONTROL DURING RECLAMATION

CALCULATION NOTES
By BAF
Checked
Acc't
12-20 1983
Sheet No. 3 of 5 Sheets

THE OVERFLOW STRUCTURE ON THE ROAD POND IS DESIGNED TO PASS THE PEAK FLOW FROM A 25 YR 24 HR EVENT. FROM NOMOGRAPH (PP. B-), USING 6.37 ACRES, 2.18" RAINFALL, FLAT SLOPES, AND CN = 87:

AT CN = 85 Q = 5.8 cfs

AT CN = 90 Q = 8.5 cfs

INTERPOLATING TO CN = 87 Q = 6.9 cfs

A 25 YR 24 HOUR STORM WILL FILL THE POND POND TO APPROXIMATELY ELEV. 5339.3'.

5339.3 (MAX. WATER - 25-24)
5337.9 (BOTTOM OF DECANT PIPE)
1.4'

∴ LET h OF DISCHARGE STRUCTURE = 1.5'

THE UPSTREAM END OF THE DECANT WILL BE BUILT AT ELEV. 5337.9. THE DOWNSTREAM END WILL DISCHARGE INTO THE 48" CULVERT UNDER THE D&RGW MAINLINE, AT ELEV. 5331.5. THE RESULTING GRADE ON THE PIPE IS 1.12%.

CALCULATION NOTES

Subject: MAINTAIN ROAD POND AND SEDIMENTATION CONTROL DURING RECLAMATION

By: BAF

Checked:

Acc't:

12-22 1983

Sheet No. 4 of 5 Sheets

CHECK MANNING FOR 12" DIA PVC PIPE, WHERE $n = 0.009$ (p. B-23):

$$\frac{1.486}{0.009}(0.785)(0.250)^{2/3}(0.0112)^{1/2} = 5.45 \text{ cfs}$$

" " $5.45 < 6.9$ ∴ OVERFLOW STRUCTURE WILL FILL WITH WATER TO INCREASE THE HYDRAULIC GRADE FROM 6.4'/570' TO 7.9'/570'.

CHECK BERNOULLI FOR MAX. CAPACITY WITH INCREASED HEAD:

$$\frac{V_1^2}{2g} + \frac{P_1}{\gamma_1} + Z_1 = \frac{V_2^2}{2g} + \frac{P_2}{\gamma_2} + Z_2$$

$$\frac{\left(\frac{6.9 \text{ cfs}}{1.0 \text{ sf}}\right)^2}{2(32.2)} + 7.9 = \frac{V_2^2}{2(32.2)}$$

$V_2 = 22.7$ fps (MAX.)

$Q_2 = 22.7 \text{ fps}(0.785 \text{ ft}^2) = 17.8$ cfs (CAPACITY IS OK)

$$V_{OUTLET} = \frac{6.9 \text{ cfs}}{0.785 \text{ ft}^2} = 8.8 \text{ fps}$$

Subject MAINTAIN ROAD POND
FOR SEDIMENTATION CONTROL
DURING RECLAMATION

CALCULATION NOTES
By BAF
Checked
Acc't
12-22 1983

THE OVERFLOW DISCHARGES INTO THE 48" DIA. CONCRETE CULVERT UNDER THE D&RGW MAINLINE. THIS CULVERT WILL ADEQUATELY DISSIPATE THE DISCHARGE VELOCITY.

DECANT/OVERFLOW DIMENSIONS:

$h = 1.5'$
$d = 1.0'$
$\phi = 1.0'$
$S = 1.123\%$
$w = 1''$ OR AS NECESSARY

$l = 10''$
$O = 8''$
$t = 1/4''$ OR AS NECESSARY
$p = 4\frac{1}{2}'' \Rightarrow 4$ PLATES REQ'D

Subject MAINTAIN HEAT DRYER POND FOR SEDIMENTATION CONTROL DURING RECLAMATION

CALCULATION NOTES
By BAF
Checked
Acc't
12-20 1983

THE 1.15 ACRE DRAINAGE AREA INTO THE HEAT DRYER POND IS SHOWN ON F9-177. SURFACE GRADING DURING RECLAMATION WILL ENSURE APPROPRIATE DRAINAGE INTO THE POND.

THE ENTIRE DRAINAGE AREA WILL HAVE A CN = 87 WHEN THE BLACKTOP AND BUILDINGS ARE REMOVED. UTILIZING THE METHODS DESCRIBED ON B-2 AND SHOWN ON B-8:

$Q_{10\text{-}24} = 0.769 \text{ IN}$

$Q_{25\text{-}24} = 1.050 \text{ IN}$

$\text{VOLUME}_{10\text{-}24} = \frac{(0.769\text{ IN})(1.15\text{ AC})(43560\text{ FT}^2/\text{AC})}{12\text{ IN/FT}} = \underline{3{,}210\text{ cf}}$

$\text{VOLUME}_{25\text{-}24} = \frac{(1.050\text{ IN})(1.15\text{ AC})(43560\text{ FT}^2/\text{AC})}{12\text{ IN/FT}} = \underline{4{,}383\text{ cf}}$

SOIL LOSS:

$A = RKLSC$

$A = (20)(0.50)(0.04)(0.45)$

$= 0.180 \text{ T/AC/YR}$

$\text{3 YR SOIL LOSS} = \frac{(0.180\text{ T/AC/YR})(2000\text{ \#/T})(1.15\text{ AC})(3\text{ YR})}{85\text{ \#/FT}^3} = \underline{15\text{ cf}}$

Subject MAINTAIN HEAT DRYER
[illegible]D FOR SEDIMENTATION
CONTROL DURING RECLAMATION

CALCULATION NOTES
By BAF
Checked
Acc't
12-20 1983

CAPACITY REQUIREMENT:

10YR 24HR = 3210 + 15 = 3225 cf

25yr 24HR = 4383 + 15 = 4398 cf

HEAT DRYER POND WILL CONTAIN THE 25YR 24HR EVENT

THE SUMP OVERFLOW ON THE HEAT DRYER POND WILL BE REMOVED AND RECLAIMED WITH THE PLANT STRUCTURES. THE OPERATOR PROPOSES THE DECANT/OVERFLOW STRUCTURE SHOWN ON A9- .

THE OVERFLOW STRUCTURE ON THE HEAT DRYER POND IS DESIGNED TO PASS THE PEAK FLOW FROM A 25 YEAR 24 HOUR EVENT FROM NOMOGRAPH (PP B-), USING 10 AND 5 ACRES AND INTERPOLATING TO 1.15 ACRES, 2.18" RAINFALL, FLAT SLOPES AND CN = 87:

AT 10 ACRES
CN = 85 Q = 8.2
CN = 90 Q = 13.0
INTEROPOLATING: CN = 87
Q = 10.1 cfs

AT 5 ACRES
CN = 85 Q = 5.0
CN = 90 Q = 7.5
INTERPOLATING CN = 87
Q = 6.0 cfs

INTERPOLATING ACREAGES TO 1.15 ACRES:

Q_{25-24} = 0.7 cfs

REV. 2: 12-30-83

Subject MAINTAIN HEAT DRYER POND FOR SEDIMENTATION CONTROL DURING RECLAMATION

CALCULATION NOTES
By BAF
Checked
Acc't
12-22 1983

THE UPSTREAM END OF THE DECANT WILL BE BUILT AT ELEV. 5333.0. THE DOWNSTREAM END WILL DISCHARGE INTO THE 48" CULVERT UNDER THE D&RGW MAINLINE AT ELEV. 5331.5. THE RESULTING GRADE ON THE PIPE IS 1.71%

A 25 YEAR 24 HOUR STORM WILL FILL THE HEAT DRYER POND TO APPROXIMATELY ELEV. 5336.4'.

5336.4' (MAX WATER - 25-24)
5333.0' (BOTTOM OF DECANT PIPE)
3.4'

∴ LET h OF DISCHARGE STRUCTURE = 3.5'

CHECK MANNING FOR 12" DIA PVC PIPE, WHERE $n = 0.009$ (p. B-23):

$$\frac{1.486}{0.009}(0.785)(0.250)^{2/3}(0.0171)^{1/2} = 6.73 \text{ cfs}$$

DISCHARGE PIPE IS ADEQUATE UNDER OPEN FLOW CONDITIONS.

Subject MAINTAIN HEAT DRYER [illegible]D FOR SEDIMENTATION CONTROL DURING RECLAMATION

CALCULATION NOTES
By BAF
Checked
Acc't
12-22 1983
Sheet No. 4 of 4 Sheets

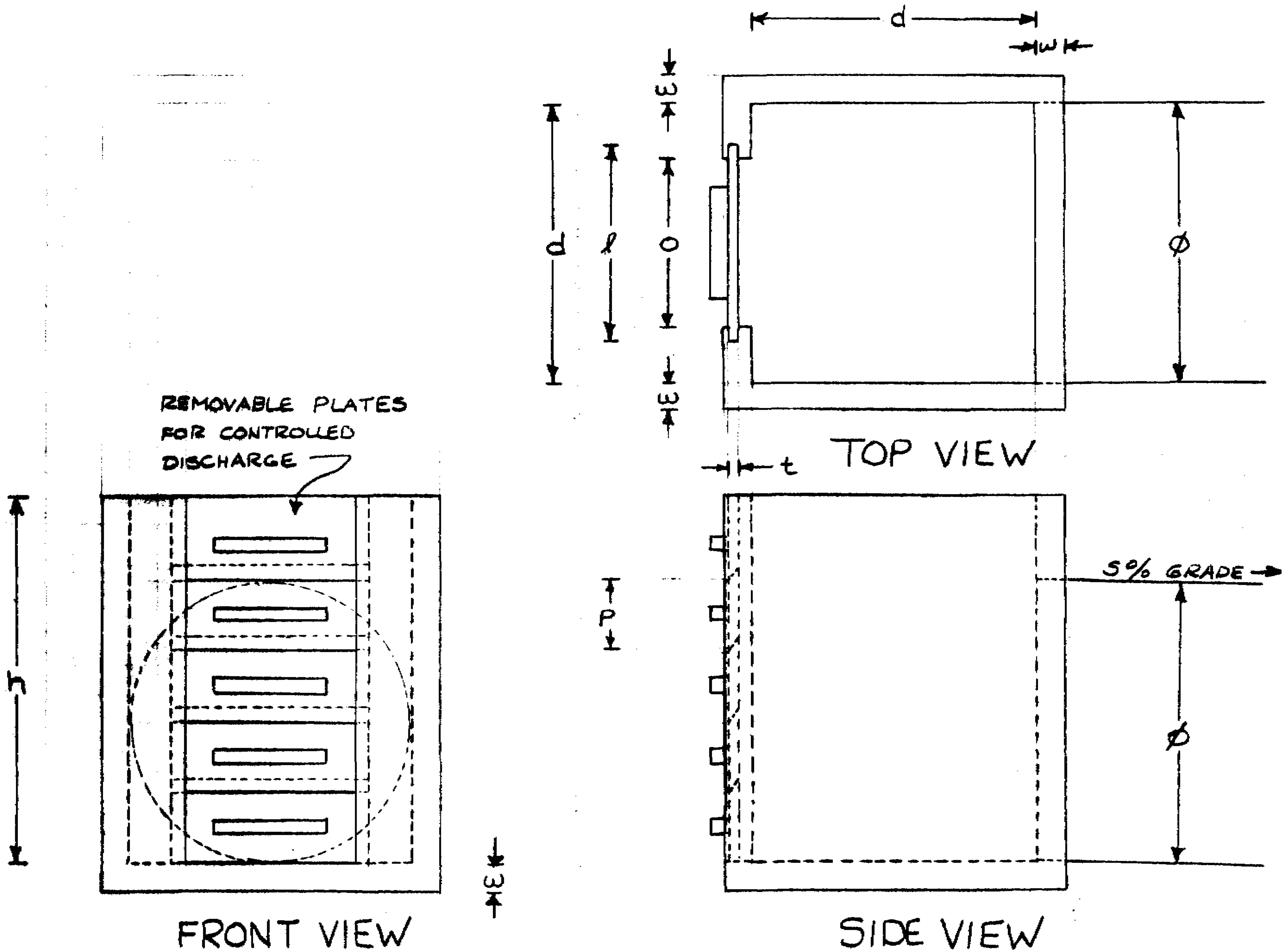
DECANT/OVERFLOW DIMENSIONS:

h = 3.5'
d = 1.0'
∅ = 1.0'
S = 1.71%
w = 1" OR AS NECESSARY

l = 10"
o = 8"
t = 1/4" OR AS NECESSARY
p = 6" ⇒ 7 PLATES REQ^D

BY BAF DATE 12/20/83 SUBJECT TYPICAL DECANT AND OVERFLOW STRUCTURE (DURING RECLAMATION ONLY) SHEET NO. 1 OF 1

CHKD. BY ...... DATE ...... JOB NO. ......



B-57
REV. 2: 12-30-83

CALCULATION NOTES

Subject Clear Water Pond Adequacy For Sedimentation Control During Reclamation	By BAF
	Checked
	Acc't
	Date December 16 19 83
	Sheet No. 1 of 4 Sheets

The Clear Water Pond will be maintained during reclamation for sedimentation control. A complete hydrologic evaluation of the slurry pond area is included in Technical Revision No. 1. Runoff quantities developed in TR No. 1 will be used in this evaluation with the exception of the pond surface areas, which are recalculated.

The Operator proposes to use topsoil which will be borrowed from the designated area to reclaim the pond surface area. This soil type is a mixed alluvial land deposited by river flooding. The "Soil Survey - Carbon Emery Area, Utah", SCS and BLM, does not assign this soil type to a specific hydrologic soil group due to the variability in locations. The majority of the soils in the plant area are hydrologic group C, with some A, B, and D classifications. Because of the predominance of the C group, the Operator has assigned the Mx series as hydrologic soil group C.

Drainage areas into the Clear Water Pond include Reach A, Reach B, Point 1, Point 2, Point 3 and both the Upper and Lower Pond surface areas. These drainage areas are shown on drawing No. A9-1429 of TR No. 1. It should be noted that, Reaches 1 and 2 drain into the permanent diversion ditch discussed in TR No. 1. The following runoff volumes are developed in TR No. 1:

	100 Year-24 Hour Event
Point 1	1.530
Point 2	2.165
Point 3	2.440
Reach A	22.856
Reach B	11.506
Total	40.497 Acre Feet

The area capacity curve for the Clear Water Pond is included on Page B- 62. The water intake tower, shown on page B-63 shows that the live storage volume in the pond is 18.25 feet below the overflow. From the capacity curve, this indicates that the pond contains some 22 acre feet of dead storage and 164 acre feet of live storage.

CALCULATION NOTES

Subject Clear Water Pond Adequacy For Sedimentation Control During Reclamation

By BAF
Checked
Acc't
Date December 16 1983
Sheet No. 2 of 4 Sheets

RUNOFF CALCULATION (Reclaimed surface only)

Soil type: Mixed Alluvial Land (Mx)(ref. dwg. E9-3339)
Hydrologic Soil Group: C
Curve No.: 78 (ref. page B-22 for cultivated land with conservation treatment)
Drainage area: 209.38 acres
Runoff Volume: 16.53 acre feet for 100 year-24 hour event (ref. pages B-2 for methodology and B-17 for storm intensities - Appendix B, ORP)

Soil loss is evaluated as described on page B-2 (revised persuant to this submittal). Factors used in the Universal Soil Loss Equation (USLE) are referenced on pages B-28 thru B-35.

Soil erodability factors (k) used in the USLE were developed based on soil texture descriptions from the "Soil Survey - Carbon Emery Area, Utah". The Sn (Shaley Colluvial Land) and Mx (Mixes Alluvial Land) series are not associated with a specific soil type, so the data from the Operator's soil testing program was utilized. All other pertinant soil types within the drainage area are associated with a specific soil texture.

Soil Series(1)	Soil Texture				k Factor(3)
CeE2	Very fine sandy loam(2)				0.24
PdC2	Very fine sandy loam(2)				0.24
Ry	Rock land with sandy loam(2)				0.24
Sn	Soil Sample(4)	%Sand	%Clay	%Silt	
	1WT	49.2	27.2	23.4	
	10WT	25.4	35.8	38.7	
	Average	37.3	31.5	31.1	
	Clay loam(5)				0.32
Mx	11WP	18.7	37.8	43.4	
	Silty clay loam(5)				0.37

(1) Technical Revision No. 1, Dwg. No. C9-1283
(2) "Soil Survey-Carbon Emery Area, Utah", SCS and BLM, Dec. 1970
(3) Reference page B-32.
(4) Reference Appendix H, page 44
(5) Reference page B-31.

CALCULATION NOTES

Subject Clear Water Pond Adequacy For Sedimentation Control During Reclamation

By BAF
Checked
Acc't
Date December 16 1983
Sheet No. 3 of 4 Sheets

SOIL LOSS CALCULATION (Total drainage area)

Drainage Area	Soil Series	Acres	R	k	LS	C	3 Year Soil Loss (CF)
Point 1	Sn	11.02	20	0.32	0.31	0.45	694
Point 2	Sn	15.61	20	0.32	0.21	0.45	666
Point 3	Sn	10.10	20	0.32	0.43	0.45	883
	Ry	16.07	20	0.24	0.43	0.45	1,054
Reach A	Sn	92.66	20	0.32	0.17	0.45	3,202
	Ry	21.58	20	0.24	0.17	0.45	559
	CeE2	97.80	20	0.24	0.17	0.45	2,535
Reach B	Sn	15.61	20	0.32	0.17	0.45	539
	Ry	61.52	20	0.24	0.17	0.45	1,595
	PdC2	19.28	20	0.24	0.17	0.45	500
	CeE2	36.73	20	0.24	0.17	0.45	952
Reclaimed Surface	Mx	209.38	20	0.37	0.06	0.45	2,953
							16,132 CF

3 Year Soil Loss = 0.37 AF

POND STORAGE REQUIREMENT

Surface runoff-undisturbed	40.50 acre-ft.
Surface runoff-reclaimed ponds	16.53 acre-ft.
Soil loss-3 year accumulation	0.37 acre-ft.
Total Requirement	57.40 acre-ft.

The Clear Water Pond can adequately contain the 100 year-24 hour precipitation event with no outflow (page B-62). (Note: the overflow structure described in TR No. 1 will be maintained as an emergency overflow. Because of the pond capacity, however, the Operator does not anticipate the water level approaching the overflow elevation during reclamation.)

Subject Clear Water Pond Adequacy For Sedimentation Control During Reclamation

CALCULATION NOTES

By BAF

Checked

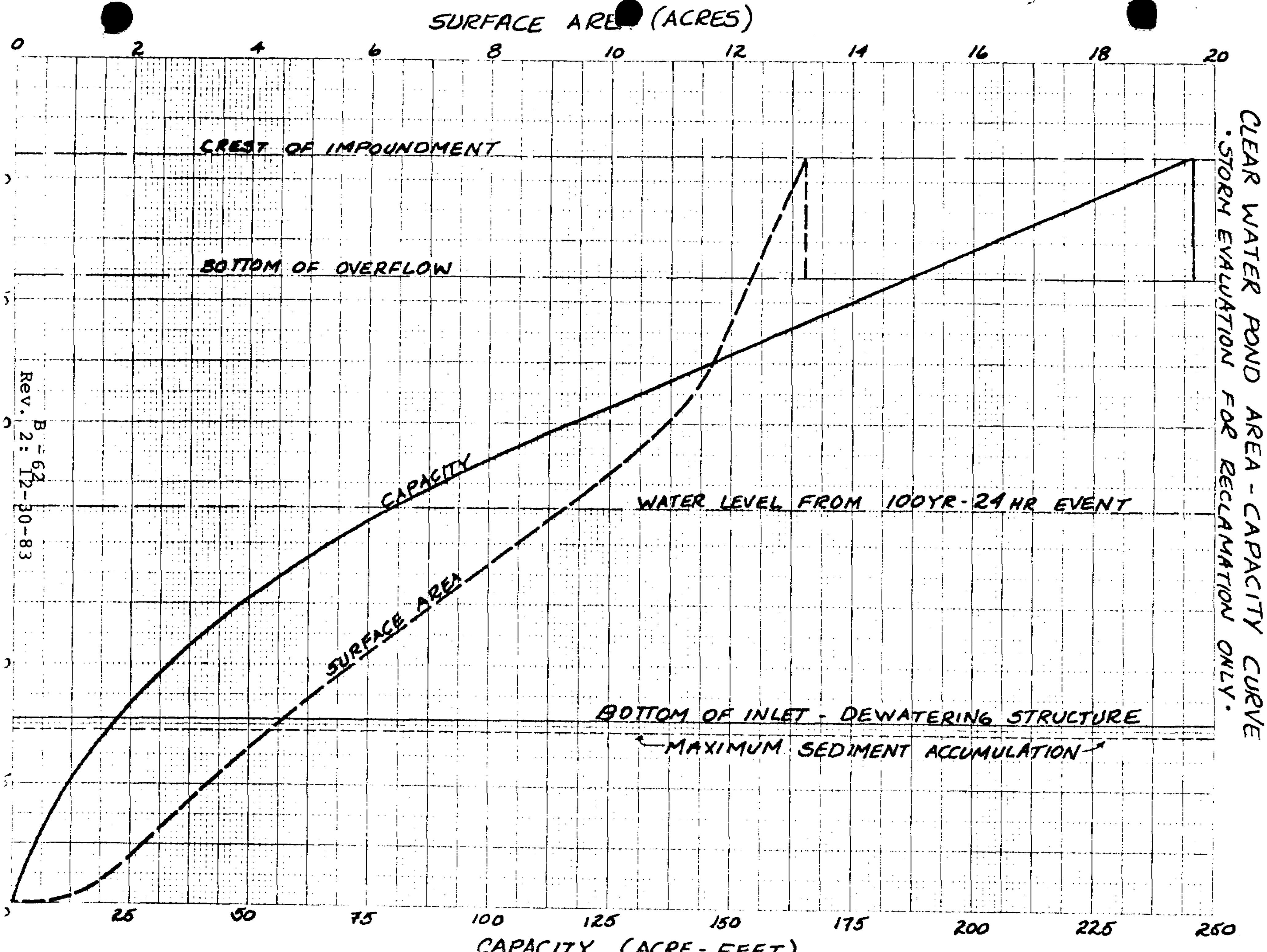
Acc't

Date December 16 1983

Sheet No. 4 of 4 Sheets

The pond dewatering structure is shown on Drawing No. C9-1293 . The Operator will maintain a sediment level in the pond not to exceed six inches from the bottom of the intake of the dewatering structure. Pond dewatering will occur after a minimum of 24 hours detention time.

46 1320


SURFACE AREA (ACRES)
0
2
4
6
8
10
12
14
16
18
20
CREST OF IMPOUNDMENT
BOTTOM OF OVERFLOW
CAPACITY
WATER LEVEL FROM 100YR-24 HR EVENT
SURFACE AREA
BOTTOM OF INLET - DEWATERING STRUCTURE
MAXIMUM SEDIMENT ACCUMULATION
25
50
75
100
125
150
175
200
225
250
CAPACITY (ACRE-FEET)
CLEAR WATER POND AREA - CAPACITY CURVE
*STORM EVALUATION FOR RECLAMATION ONLY.
B-62
Rev. 2: 12-30-83

U. S. STEEL (USS) MINING CO. | C9-1293

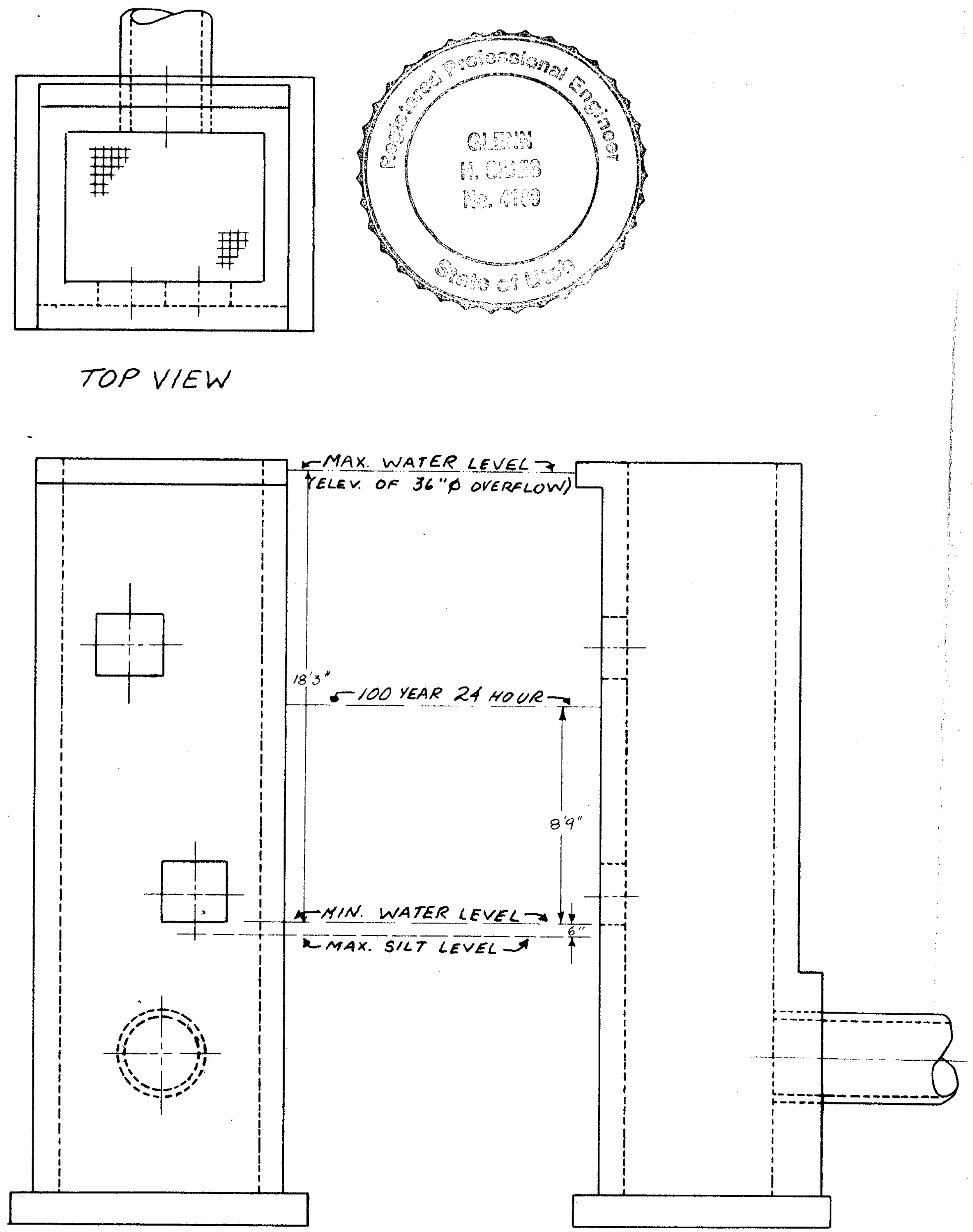
CLEAR WATER POND DECANT STRUCTURE FOR RECLAMATION

C9-1293

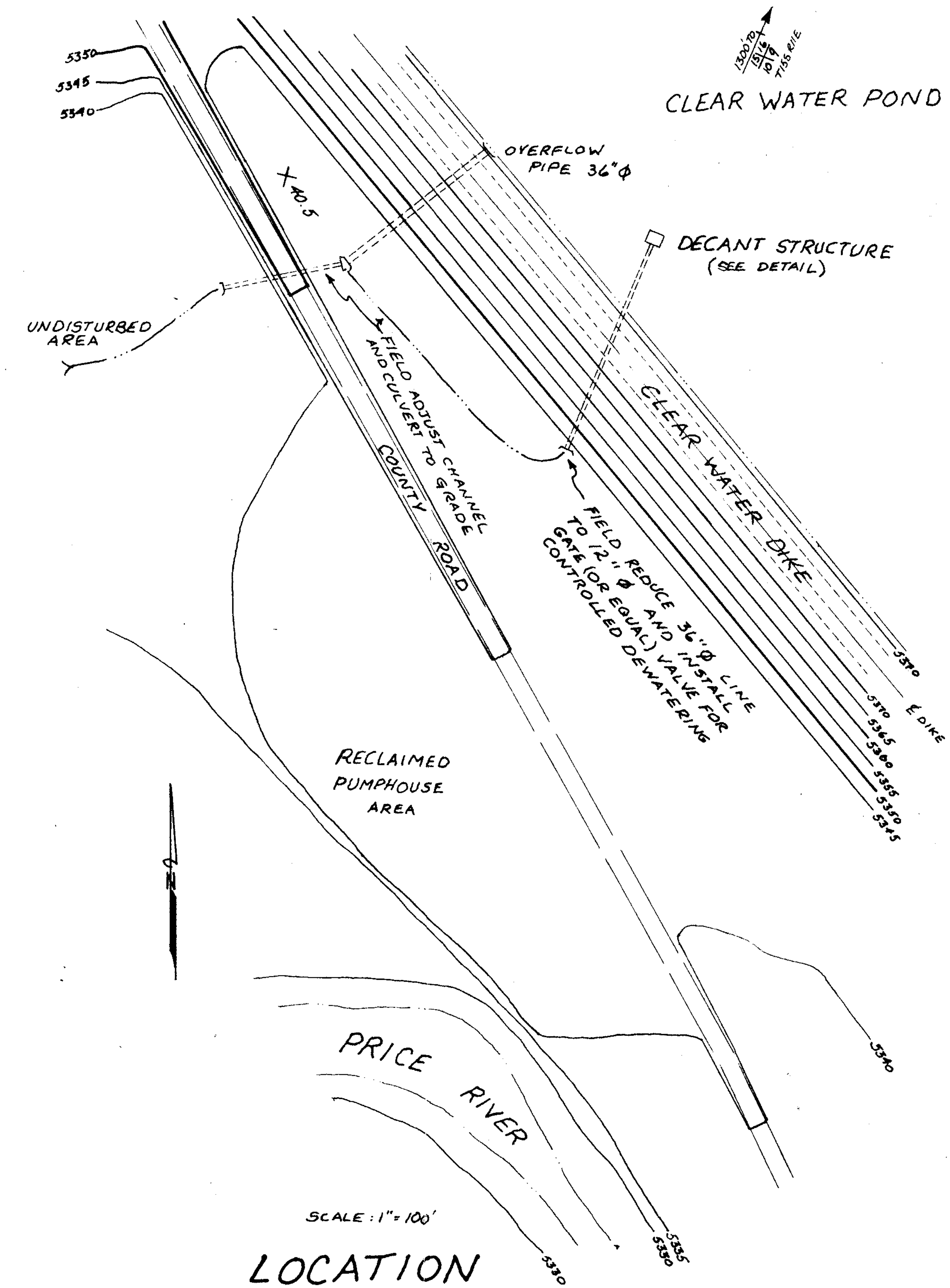
ENGINEERING DEPARTMENT
WESTERN DISTRICT – COAL

Registered Professional Engineer
GLENN H. GILES
No. 4169
State of Utah

THIS DRAWING WAS PREPARED UNDER MY SUPERVISION
Glenn H. Giles
Registered Professional Engineer- Utah 4169
DEC 28 1983
Date



DECANT STRUCTURE



LOCATION

DRAWN: B.A.F. 12-23-83 CHECKED:
TRACED: APPROVED FOR SAFETY: APPROVED: G.H.S. 12-27-83

REVISIONS

B-63

OSBORNE'S

Rev. 2: 1230-83

CALCULATION NOTES

Subject Diversion Ditch Around Lower Refuse Pond Reclaimed Surface

By BAF

Checked

Acc't

Date December 18 1983

Sheet No. 1 of 2 Sheets

The Operator proposes to construct a permanent diversion ditch to divert runoff from the undisturbed areas in Reaches A and B at the time the slurry pond area is reclaimed. The proposed ditch will be coursed around the southeast perimeter of the Lower Refuse Pond reclaimed surface as shown on Drawing No. E9-3342. The Operator feels that a permanent diversion is necessary to insure that the pond reclamation scheme is not altered by the potential erosion hazards of coursing a significant drainage area across the reclaimed surfaces.

A complete hydrologic evaluation for the slurry pond area is included in Technical Revision No. 1, Appendix C. Design capacities in the proposed ditch are based on the peak flow of 24.20 cfs, which was established by combining the unit hydrographs for Reach A and B for a 100 year - 24 hour precipitation event.

The Operator proposes to construct the ditch such that it discharges into the Clear Water Pond during reclamation (sizing of the Clear Water Pond as a sedimentation Pond includesthe 100 year storm inflow from Reaches A and B). When the Clear WAter Pond is reclaimed, the diversion ditch will be extended as shown on Drawing No. E9-3342.

Timing of the diversion ditch construction is as follows:

1. Construct ditch lengths D1 to D2 as designed prior to surface grading of the Lower Refuse Pond. The ditch will discharge into the Clear Water Pond at the approximate location of length D2 to D3.

2. Construct ditch lengths D3 to D4 and D5 to D6 after grading the Clear Water Dike to the configuration shown on E9-3342. Install the culvert under the County Road. (Note: this portion of the diversion ditch will be built after slurry pond reclamation is considered successful - ie. when water entering the Clear Water Pond meets effluent limitations).

Subject Diversion Ditch Around Lower Refuse Pond Reclaimed Surface

CALCULATION NOTES

By BAF

Checked

Acc't

Date December 18 1983

Sheet No. 2 of 2 Sheets

Design criteria is based on a peak flow of 24.2 cfs, and the following ditch segments referenced on E9-3342:

Ditch Segment	Length (ft)	Elevation Drop (ft)	Slope (ft/ft)	Design
D1 - D2	2950	5	0.0017	Open Channel
D2 - D3	420	35	0.0833	Open Channel
D3 - D4	390	6	0.0154	Open Channel
D4 - D5	50	2.5	0.0500	Culvert
D5 - D6	50	12.5	0.2500	Open Channel

CALCULATION NOTES

Subject DIVERSION DITCH AROUND RECLAIMED SLURRY POND AREA

By BAF

Checked

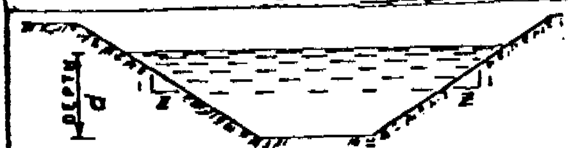
Acc't

12-19 1983

Sheet No. 1 of 8 Sheets

REF. DWG. E9-3342

Ditch Discharge- Manning's Formula.

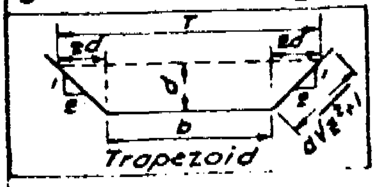


n =

S =

b

Z·



Manning's formula is one of the most widely accepted and commonly used of the open channel formulas.

$$v = \frac{1.486}{n} r^{2/3} s^{1/2}$$

v = mean velocity of flow in ft. per sec.
r = hydraulic radius in ft.
s = slope of the energy gradient.
n = coefficient of roughness. *See back*

D1-D2 CHANNEL CHARACTERISTICS

n Coeff. of Roughness	Slope S feet/foot	Bottom Width b ft.	Depth d ft.	AREA $bd+zd^2$ a	WETTED PERIMETER $b+2d\sqrt{z^2+1}$ P	HYDRAULIC RADIUS $\frac{bd+zd^2}{b+2d\sqrt{z^2+1}}$ r	VELOCITY $\frac{1.486}{n} r^{2/3} S^{1/2}$ feet Per Second	QUANTITY aV Cu. Ft./Second
0.035	0.0017	5.0	0.5	2.88	6.80	0.42	0.99	2.8
			1.0	6.50	8.61	0.76	1.45	9.4
			1.5	10.88	10.41	1.04	1.80	19.6
PEAK FLOW:			1.7	12.84	11.13	1.15	1.93	24.7

B-66

REV. 2: 12-30-83

Subject DIVERSION DITCH AROUND RECLAIMED SLURRY POND AREA

REF. DWG E9-3342

CALCULATION NOTES

By BAF

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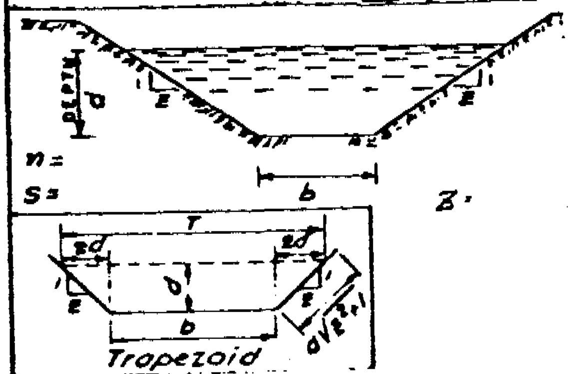
Acc't

12-19 1983

Sheet No. 2 of 8 Sheets

Ditch Discharge- Manning's Formula.



Manning's formula is one of the most widely accepted and commonly used of the open channel formulas

$$v = \frac{1.486}{n} r^{2/3} s^{1/2}$$

v = mean velocity of flow in ft. per sec.
r = hydraulic radius in ft.
s = slope of the energy gradient.
n = coefficient of roughness. See back.

D2-D3 CHANNEL CHARACTERISTICS

n Coeff. of roughness	Slope S feet/foot	Bottom Width b Ft.	Depth d ft.	AREA $bd+Zd^2$ a	WETTED PERIMETER $b+2d\sqrt{Z^2+1}$ P	HYDRAULIC RADIUS $\frac{bd+Zd^2}{b+2d\sqrt{Z^2+1}}$ r	VELOCITY $\frac{1.486}{n} r^{\frac{2}{3}} S^{\frac{1}{2}}$ Feet Per Second	QUANTITY aV Cu. Ft./Second
0.0395	0.0833	5.0	0.25	1.34	5.90	0.23	4.05	5.4
			0.50	2.88	6.80	0.42	6.11	17.6
PEAK FLOW:			0.61	3.61	7.20	0.50	6.85	24.7

B-67

REV 2: 12-30-83

CALCULATION NOTES

By BAF

Checked

Acc't

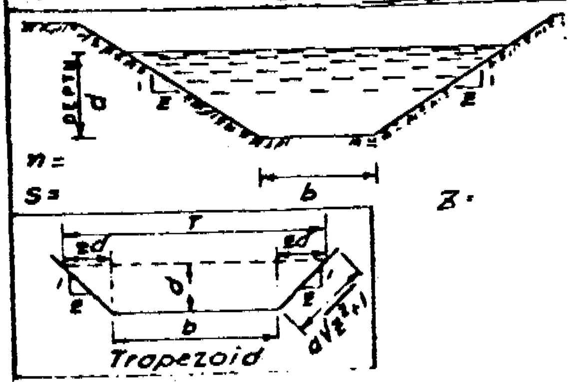
12-19 1983

Sheet No. 3 of 8 Sheets

Subject DIVERSION DITCH AROUND RECLAIMED SLURRY POND AREA

REF. DWG E9-3342

Ditch Discharge- Manning's Formula.



Manning's formula is one of the most widely accepted and commonly used of the open channel formulas

$$v = \frac{1.486}{n} r^{2/3} s^{1/2}$$

v = mean velocity of flow in ft. per sec.
r = hydraulic radius in ft.
s = slope of the energy gradient.
n = coefficient of roughness. See back

D3 - D4 CHANNEL CHARACTERISTICS

n Coeff. of Roughness	Slope S feet/foot	Bottom Width b ft.	Depth d ft.	AREA $bd+zd^2$ a	WETTED PERIMETER $b+2d\sqrt{z^2+1}$ P	HYDRAULIC RADIUS $\frac{bd+zd^2}{b+2d\sqrt{z^2+1}}$ r	VELOCITY $\frac{1.486}{n} r^{2/3} S^{1/2}$ Feet Per Second	QUANTITY aV Cu. Ft./Second
0.033	0.0154	5.0	0.25	1.34	5.90	0.23	2.08	2.8
			0.50	2.88	6.80	0.42	3.15	9.0
			0.75	4.59	7.70	0.60	3.96	18.2
PEAK	FLOW:		0.89	5.64	8.21	0.69	4.35	24.5

REV 2: 12-30-83

Subject DIVERSION DITCH AROUND RECLAIMED SLURRY POND AREA

REF. DWG. E9-3342

CALCULATION NOTES

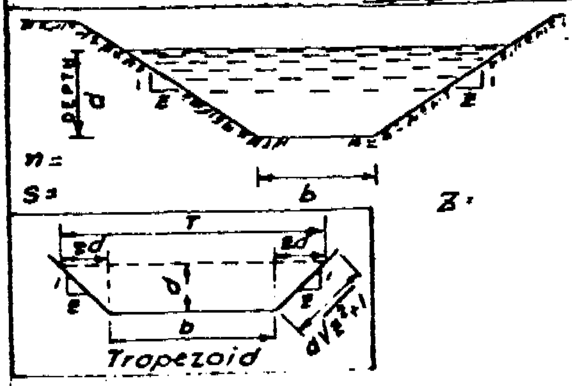
By BAF

Checked

Acc't

12-19 1983

Ditch Discharge- Manning's Formula.



Manning's formula is one of the most widely accepted and commonly used of the open channel formula.

$$v = \frac{1.486}{n} r^{2/3} s^{1/2}$$

v = mean velocity of flow in ft. per sec

r = hydraulic radius in ft.

s = slope of the energy gradient.

n = coefficient of roughness. See back.

D5-D6 CHANNEL CHARACTERISTICS

n Coeff. of Roughness	Slope S feet/foot	Bottom Width b ft.	Depth d ft.	AREA $bd+Zd^2$ a	WETTED PERIMETER $b+2d\sqrt{Z^2+1}$ P	HYDRAULIC RADIUS $\frac{bd+Zd^2}{b+2d\sqrt{Z^2+1}}$ r	VELOCITY $\frac{1.486}{n} r^{2/3} S^{1/2}$ Feet Per Second	QUANTITY aV Cu. Ft./Second
0.046	0.2500	10.0	0.10	1.02	10.36	0.10	3.43	3.5
			0.20	2.06	10.72	0.19	5.38	11.1
			0.30	3.14	11.08	0.28	6.96	21.8
PEAK FLOW:			0.32	3.35	11.15	0.30	7.25	24.3

REV 2: 12-30-83

Subject DIVERSION DITCH AROUND RECLAIMED SLURRY POND AREA	CALCULATION NOTES
	By BAF
	Checked
	Acc't
REF. DWG. E9-3342	12-19 1983
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RIPRAP REQUIREMENTS

D1-D2 – MAX. VELOCITY = 1.93 fps

∴ NO RIPRAP IS REQUIRED

D2-D3 – MAX. VELOCITY = 6.85 fps

∴ USE 1' AVERAGE DIA RIPRAP – 2.6 SP. GR.

MANNING $n = 0.0395\, D_{50}^{1/6} = 0.0395(1)^{1/6} = 0.0395$

$d = 0.61'$ AT PEAK FLOW

$\tau = \gamma d S = 62.4(0.61)(0.0833) = 3.17\ \text{lb/ft}^2$

$$\eta_b = \frac{21\tau}{\gamma(SG-1)(D_{50})} = \frac{21(3.17)}{62.4(2.6-1)(1)} = 0.667$$

FROM TABLE (P. B-75) : $\phi = 42°$

$\theta = \tan^{-1} 0.0833 = 4.76°$

$$SF = \frac{\cos 4.76 \tan 42}{\sin 4.76 + 0.667 \tan 42} = \underline{1.31 \text{ SAFETY } \ldots}$$

$D_{50} > d \qquad 1.0 > 0.61 \quad$ ∴ CHANNEL BANK IS STABLE

D3-D4 – MAX. VELOCITY = 4.54 fps

USE 4" AVERAGE DIA. RIPRAP – 2.6 SP. GR.

MANNING $n = 0.0395(0.33)^{1/6} = 0.033$

$d = 0.89'$ AT PEAK FLOW

$\tau = 62.4(0.89)(0.0154) = 0.855\ \text{lb/ft}^2$

Subject: DIVERSION DITCH AROUND RECLAIMED SLURRY POND AREA	CALCULATION NOTES
REF. DWG. E9-3342	By: BAF
	Checked:
	Acc't:
	12-19 1983
	Sheet No. 6 of 8 Sheets

$$\eta_b = \frac{21(0.855)}{62.4\,(2.6-1)(0.33)} = 0.540$$

FROM TABLE (p B-75): $\phi = 41°$

$\theta = \tan^{-1} 0.0154 = 0.88°$

$$SF = \frac{\cos 0.88 \tan 41}{\sin 0.88 + 0.540 \tan 41} = \underline{\underline{1.8 \text{ SAFETY } ok}}$$

$0.33' < 0.89'$

CK. CHANNEL BANK STABILITY:

$\tau_{MAX} = 0.74\,\gamma d s$ (p. B-75)

$$0.74(62.4)(0.89)(0.0154) = 0.633\ lb/ft^2$$

$$\eta = \frac{21(\tau_{MAX})}{\gamma(SG-1)D_{50}} = \frac{21(0.633)}{62.4(2.6-1)(0.33)} = 0.399$$

$\lambda = \theta = 0.88°$

$$\delta = \tan^{-1}\frac{1}{z} = \tan^{-1}\frac{1}{1.5} = 33.69°$$

$$\beta = \tan^{-1}\left[\frac{\cos\lambda}{\left(\frac{2\sin\delta}{\eta\tan\phi}\right)+\sin\lambda}\right]$$

$$= \tan^{-1}\left[\frac{\cos 0.88}{\left(\frac{2\sin 33.69}{0.399\tan 41}\right)+\sin 0.88}\right] = 17.30°$$

$$\eta' = \eta\left(\frac{1+\sin(\lambda+\beta)}{2}\right) = 0.399\left(\frac{1+\sin(0.88 + 41.30)}{2}\right)$$

$= 0.261$

CALCULATION NOTES

Subject DIVERSION DITCH AROUND RECLAIMED SLURRY POND AREA

REF. DWG E7-3342

By BAF

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Acc't

12-19 1983

$$SF = \frac{\cos\delta \tan\phi}{\eta' \tan\phi + \sin\delta \cos\beta}$$

$$= \frac{\cos 33.69 \tan 41}{0.261 \tan 41 + \sin 33.69 \cos 17.80} = 1.0 \text{ SAFETY}$$

CHANNEL SIDE SLOPE SAFETY FACTOR OF 1.0 WILL BE OK FOR A MAX. VELOCITY OF 4.54 fps.

D5-D6 - MAX. VELOCITY = 7.25 fps

USE 2.5' AVERAGE DIA. RIPRAP - 2.6 SP. GR.

MANNING $n = 0.0395(2.5)^{1/6} = 0.046$

$d = 0.32'$ AT PEAK FLOW

$\tau = 62.4(0.32)$ [illegible]

$\eta_b = \frac{21(4.99)}{[illegible]} = 0.420$

FROM [illegible] 75 : $\phi = 42°$

$\theta = \tan^{-1} 0.2500 = 14°$

$$SF = \frac{\cos 14 \tan 42}{\sin 14 + 0.420 \tan 42} = 1.4 \text{ SAFETY OK}$$

$2.5' > 0.32$ ∴ CHANNEL BANK IS STABLE

REFERENCE: "APPLIED HYDROLOGY AND SEDIMENTOLOGY FOR DISTURBED AREAS", BARFIELD WARNER & HAAN, 1981, CHAPTER 3.

Subject DIVERSION DITCH AROUND RECLAIMED SLURRY POND AREA

REF. DWG. E9-3342

CALCULATION NOTES

By BAF

Checked

Acc't

12-19 198[illegible]

Sheet No. 8 of 8 Sheets

D4 - D5

THE DIVERSION DITCH WILL PASS UNDER THE COUNTY ROAD IN A 24" CORRUGATED STEEL CULVERT. THE CULVERT WILL BE INSTALLED AT 5% GRADE. THE CULVERT CAPACITY, FROM THE MANNING EQUATION:

$$Q = \frac{1.486}{n} A R^{2/3} S^{1/2}$$

$n = 0.022$ (p. B-23)

$$A = \frac{\pi 2^2}{4} = 3.14 \, ft^2$$

$P = 2\pi = 6.28 \, ft$

$R = 0.50 \, ft$

$$S = \frac{5}{100} = 0.050$$

$$Q = \frac{1.486}{0.022}(3.14)(0.50)^{2/3}(0.050)^{1/2}$$

$= 29.9$ cfs FLOWING FULL

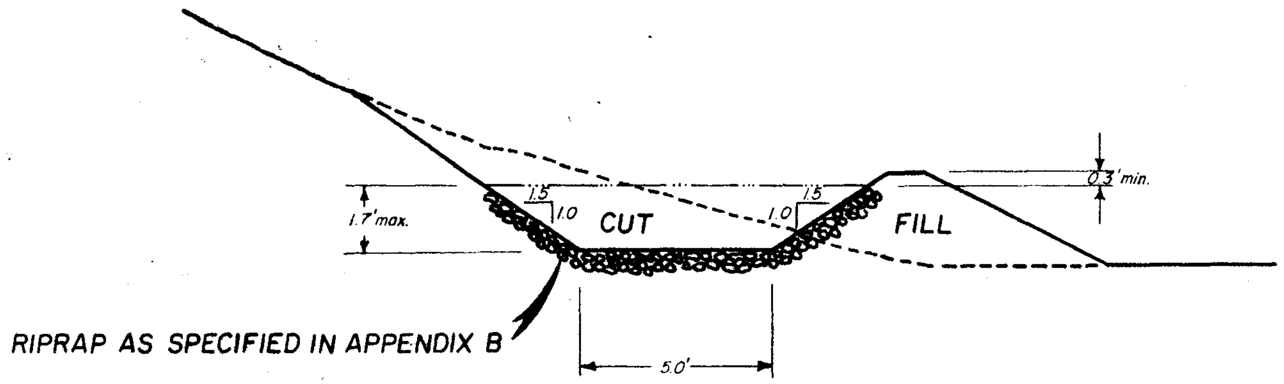
$$SF = \frac{29.9}{24.2} = 1.2 \text{ SAFETY OK}$$

U. S. STEEL (USS) MINING CO. | C9-1292 | DIVERSION DITCH CROSS SECTIONS FOR RECLAMATION | C9-1292

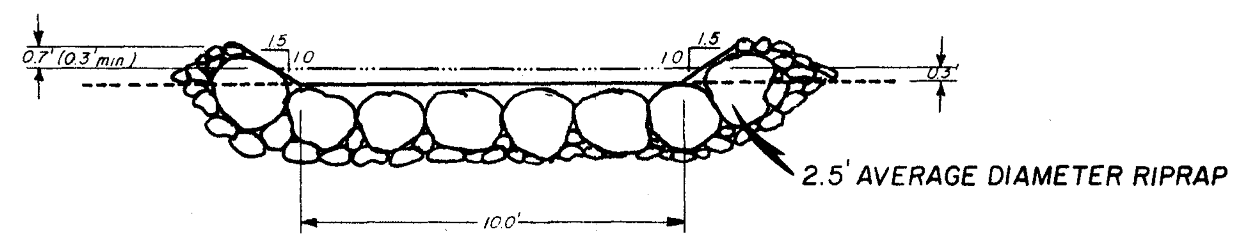
ENGINEERING DEPARTMENT
WESTERN DISTRICT - COAL

--------- ORIGINAL CONTOUR

——— PROPOSED FINAL CONTOUR



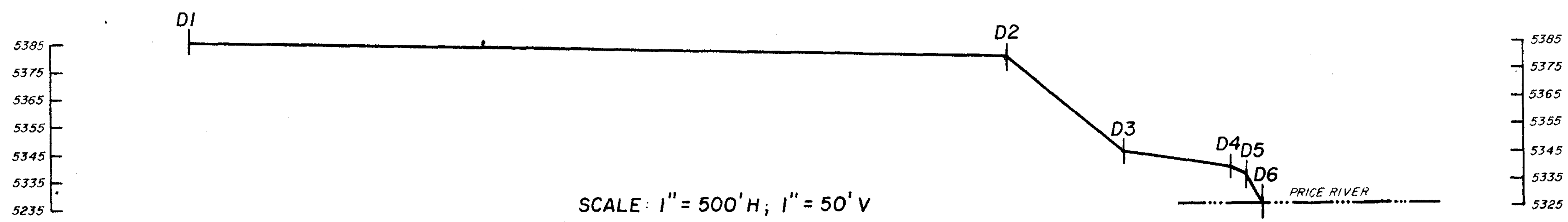
SCALE: 1" = 5' H&V



SCALE: 1" = 5' H&V

TYPICAL DITCH SECTION FROM D1 TO D4

TYPICAL DITCH SECTION FROM D5 TO D6



SCALE: 1" = 500' H; 1" = 50' V

HYDRAULIC GRADIENT OF PERMENANT DIVERSION

NOTE: D4 TO D5 IS CONTAINED IN A 24" CORRUGATED STEEL CULVERT

THIS DRAWING WAS PREPARED UNDER MY SUPERVISION

4169 DEC 2 8 1983

Registered Professional Engineer- Utah | Date

Registered Professional Engineer GLENN H. SIDES No. 4169 State of Utah

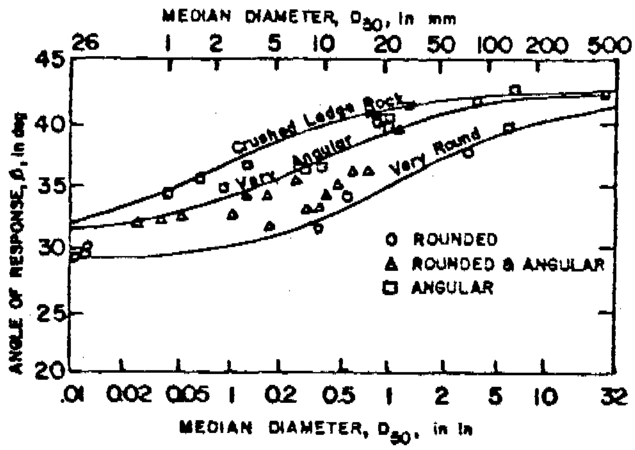
DRAWN: R.A.F. 12-23-83 CHECKED: APPROVED: G.H.S. 12-23-83

TRACED: APPROVED FOR SAFETY:

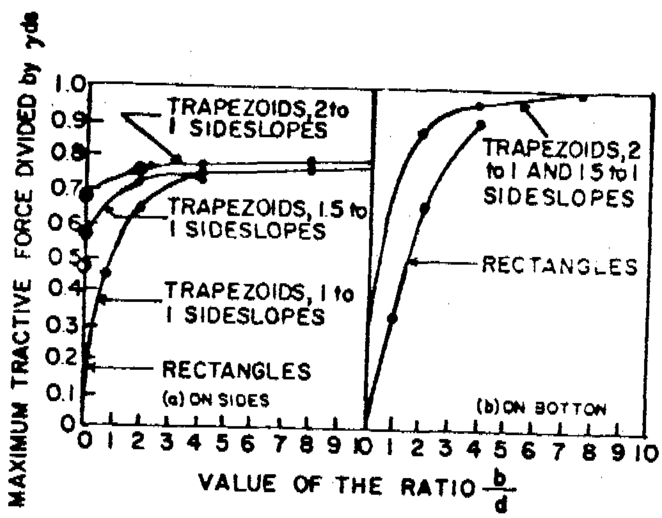
REVISIONS		REFERENCE DRAWINGS	E9-3342	B-74

REV. 2: 12-30-83

OSBORNE'S



Angle of repose of dumped riprap. (After Simons and Senturk, 1977)



Maximum tractive force as related to γdS on trapezoidal channels. (After Lane, 1953)

Source: "Applied Hydrology and Sedimentology for Disturbed Areas"; Barfield, Warner and Haan; 1981, Figure 3.14 and Figure 3.16, pp 187, 192.

DIVERSION DITCH DISCHARGE STRUCTURES

There are three diversion ditches proposed in this plan. Two are designed as permanent diversions to course runoff from undisturbed areas away from the slurry pond area. One is a temporary diversion to be maintained until surface reclamation is achieved.

Temporary Diversion West of Plant Area:

The temporary diversion ditch (described on pages B-5, 13, 14) west of the plant area currently discharges into the basin shown on E9-3431. Long range plans for the adjacent coarse refuse pile will eventually inundate the outlet basin. The Operator has committed to submitting a technical revision for ditch modification when the coarse refuse pile is expanded to a point that the basin is no longer adequate.

The discharge basin has shown little or no signs of erosion to date. The basin appears to be an effective discharge structure.

Permanent Diversion Ditch Around Lower Refuse Pond:

The permanent diversion described on pages B-64 - 74 is designed to pass the peak flow from a 100 year - 24 hour storm. Channel stability evaluations for riprap sizing are included. The proposed discharge into the Price River consists of a 10 foot wide bed of 2.5 foot average diameter riprap. Calculations using the Manning equation indicate a channel depth of approximately 0.3 feet and a maximum velocity of 7.25 fps. The large riprap bed will adequately minimize erosion and dissipate energy in the ditch prior to confluence with the Price River.

Permanent Diversion Ditch Adjacent to the North Dike:

The permanent diversion ditch adjacent to the North Dike is described in Technical Revision No. 1. The ditch passes through a culvert under the county road then drops approximately 30 feet in 500 feet. This corresponds to average grade of 6% between the culvert and the river. The ditch channel existed prior to the diversion described in TR No. 1. No discharge structure was installed at the time the original ditch was excavated (prior to 1977). Over the years, the channel has cut into the hillside such that there are significant highwalls in the channel in some locations.

The general area around the Coal Cleaning Plant is either a relatively flat flood plain of the Price River, or weathering shale hillsides which are susceptible to erosion. The discharge end of the diversion ditch has eroded to a stabilized configuration which is consistant with the surrounding surface configuration. Because of the near vertical channel sides for about half the length of the discharge, the disturbance required to slope the channel sides to flatter slopes would probably cause greater sediment contributions to the Price River be than the potential future contributions from additional erosion.

Rough cross sections of the ditch discharge are shown on page B-78. The maximum velocity required to pass the peak flow is approximately 7.5 fps. Because the channel erosion has already occurred and the current configuragion is stabilized, this is not considered excessive.

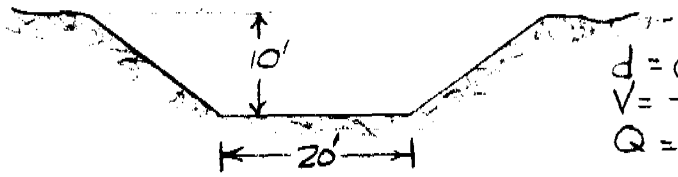
BY BAF DATE 12-23-83 SUBJECT DISCHARGE OF PERMENANT DIVERSION SHEET NO. 1 OF 1

CHKD. BY DATE JOB NO.

APPROX. DIST. FROM CULVERT

V & Q FROM MANNING
PEAK FLOW 100-24
= 120cfs (TR #1)
use n=0.035 (pB-23)

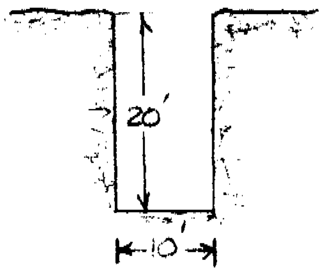
0' TO 180'



d = 0.8'
V = 7.0 fps
Q = 117 fps

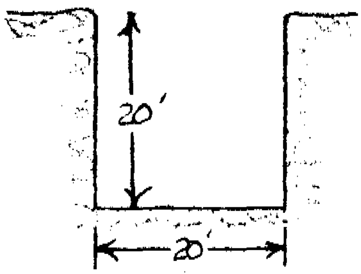
NOTE: 10' WATERFALL AT 180'

180' TO 330'



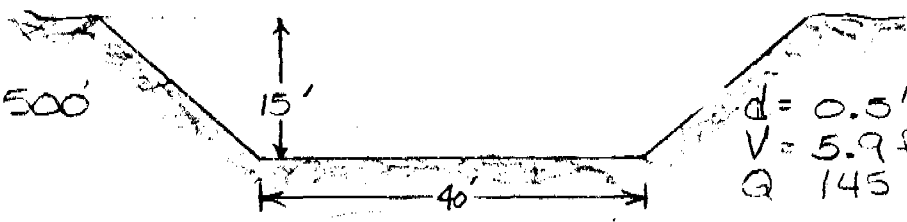
d = 1.8'
V = 7.1 fps
Q = 127 cfs

330 TO 450'



d = 0.8'
V = 7.5 fps
Q = 135 cfs

450' TO 500'



d = 0.5'
V = 5.9 fps
Q 145 cfs

500' IS APPROX LOCATION OF PRICE RIVER

NOTE: V & Q ARE CALCULATED AT 4% SLOPE - THIS AVERAGE GRADE DISCOUNTS THE 10' WATERFALL ((30-10)/500)

Subject Discharge Structures For Sedimentation Ponds During Reclamation

CALCULATION NOTES

By BAF

Checked

Acc't

Date 2-14 1984

Sheet No. ____ **of** ____ **Sheets**

Road Pond and Dryer Pond

The overflow structures on the Road Pond and Dryer Pond are designed to pass 6.9 cfs (p. B-50) and 0.7cfs (p. B-54) respectively. Both structures will discharge into the 48" corrugated (This culvert is said, in error, to be concrete on p. B-52) culvert under the D&RGW Mainune (REF. DWG. E9 - 3433). The culvert grade is 0.5%.

Check Manning for 48" DIA corrugated pipe, where N = 0.022 (p. B-23):

x-sec of 48" pipe

$$\frac{1.486}{0.022} (12.57) (1.00)^{2/3} (0.005)^{1/2} = 60.04 \text{ cfs}$$ ← full pipe flow? chart = 60 cfs OK

Pipe capacity exceeds combined peaks

$$\frac{60.04 \text{ cfs}}{12.57 \text{ ft}^2} = 4.78 \text{ fps}$$

Velocity is not excessive