04008904142E

40-8904

INTERA

Intera Technologies Inc.
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Austin, Texas 78731

Tel.: (512)-346-2000
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RETURN ORIGINAL TO PDR HQ.

September 14, 1988

SEP 1988 RECEIVED

DOCKETED USNRC SEP 20 1988 MAIL SECTION DOCKET CLERK

Mr. Edward Hawkins, Chief
Licensing Branch 1
Uranium Recovery Field Office
Region IV
Box 25325
Denver, Colorado 80225

RE: Docket No. 04008904 1800, Responses to July 26, 1988 NRC Comments

Dear Mr. Hawkins:

Please find enclosed five (5) copies of INTERA/BP AMERICA responses to comments 1 and 2 from the NRC letter of July 26, 1988 identifying surface water hydrologic deficiencies in the L-Bar Reclamation Plan. These responses come as a result of several discussions between our hydrologic consultant, Dr Alan Kuhn, and Ray Gonzales of your staff. We believe these responses answer the concerns presented in your letter and we therefore assume that all outstanding deficiencies have been resolved. We look forward to final approval of the L-Bar Reclamation Plan, which we understand is forthcoming shortly.

Based on your letter of May 27, 1988 expressing no fatal flaws in the Reclamation Plan, your letter of July 26, 1988 expressing near completion of Reclamation Plan review and the two deficiencies (addressed herein) and telephone conversations with Scott Grace which indicated he knew of no other outstanding deficiencies or issues and could see no reason why construction should not start, BP America awarded a reclamation construction contract to Twin Mountain Rock Company effective August 16, 1988. Reclamation activities have begun

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DESIGNATED ORIGINAL

Certified By Mary C. Hood

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Addl Info
88-1249

at the site and the preliminary schedule indicates reclamation activities will be completed by April 10, 1989.

INTERA and BP AMERICA appreciate the thorough review NRC has given this Reclamation Plan. We also appreciate the assistance NRC has provided to BP AMERICA in carrying out its commitment to an environmentally sound site closure.

We look forward to final Reclamation Plan approval and remain available to assist the NRC in any way to expedite the process.

Sincerely,

T.G. Osborn
Project Coordinator

TGO:lli

cc: G.E. Crisak
Ralph DeLeonardis

H01100C399

INTERA

INTERA RESPONSE TO NRC COMMENTS OF JULY 26, 1988

COMMENT 1.

The design basis flood used to design the shape of the top of the pile and the swale where the flood passes over the impoundment was not conservatively derived. Based on our independent evaluations, the proposed rock size for the swale would not be adequate. The design basis flood should be recalculated, as we have discussed with your consultant, and the swale redesigned. The changes in the design could include increasing the size of the rock, widening the swale, or a combination of these. Also, you may wish to consider redesigning the top to eliminate the swale and instead, direct flows over the entire length of the embankment outslopes. This option, however, would necessitate that the rock size on the embankment outslopes be increased to accommodate the larger design flows.

RESPONSE TO COMMENT 1.

The design basis flood of the top of the pile has been recalculated using an ultraconservative runoff coefficient of 1.0. As a result the cover swale has been redesigned and the rip rap criteria of NUREG/CR-4651 have been incorporated. The calculation and sketches for this redesign are enclosed.

We must point out that our decision to redesign to the above criteria does not reflect our agreement with the appropriateness of the criteria. We believe using a runoff coefficient of 1.0 for an area of vegetated ground with a very slight slope is unreasonably conservative. A tiled roof or asphalt parking lot would yield a lower runoff coefficient than 1.0. Using a runoff coefficient of 1.0, which assumes that every drop of water turns into runoff, should also greatly reduce any concerns regarding infiltration, which was mentioned as a possible concern in the July 26, 1988 NRC letter.

NUREG/CR-4651 is a report of results of rip rap tests in plumes on materials no larger than about six inches. The appropriateness of extrapolating those results to non-plume situation and larger materials is yet to be demonstrated. We therefore question its use as the regulatory guideline under circumstances such as those that exist at the L-Bar site.

COMMENT 2.

Our independent evaluations of the flows used to design the diversion ditches indicate that they are acceptable. However, the method used (Manning's equation for uniform flow) to estimate flow depths and velocities in the channels is not conservative, tending to underestimate the need for erosion protection. Our evaluations, which were performed using gradually varied flow calculations (the computer program HEC-2 was used) indicate that there are a few locations in the north channel where erosion protection may be required because velocities exceed 3 feet per second (fps). Velocities above 3 fps on bare soils are assumed to be erosive. Likewise, in the southern channel, flows exceed 3 fps, particularly in areas from the sedimentation/stilling basin to the outfall areas. The upper end of the sedimentation/stilling basin and the "G" portion of the southern channel may also need additional erosion protection. You should re-estimate velocities and water surface elevations in the diversion channels using gradually varied flow conditions and redesign the channels accordingly. The redesigns may consist of placing rock in certain sections of the channels, widening and/or flattening the channels, or some combination thereof.

REPLY TO COMMENT 2.

The design of the diversion channels at L-Bar incorporated numerous levels of conservation which, when taken together, clearly result in a design which is more than adequately conservative to address the concerns expressed in the July 26 NRC letter. A letter explaining these conservatisms from our hydrologic consultant, Alan Kuhn, is attached.

ALAN K. KUHN, Ph.D., P.E.
CONSULTANT IN GEOLOGICAL ENGINEERING AND APPLIED GEOSCIENCES
13212 Manitoba Drive NE, Albuquerque, NM 87111-2955 505-298-9839

8803

August 19, 1988

RECEIVED AUG 24 1988 INTERA TECHNOLOGIES

FILE COPY

Dr. Tom Osborn
Intera Technologies
6850 Austin Center Blvd.
Suite 300
Austin, TX 78731

CONSERVATISMS INCORPORATED IN THE DESIGN OF DIVERSION CHANNELS
L-BAR URANIUM OPERATIONS RECLAMATION PLAN

Dear Tom:

On August 4 I had a meeting at the Denver NRC office with Ray Gonzales concerning the NRC letter to BP America of July 26. That letter stated that the L-Bar Reclamation Plan was "deficient" in two areas related to hydraulic design. The first area had to do with the design basis flood for runoff control from the top of the covered pile. Mr. Gonzales stated that the NRC required that a runoff coefficient, C, of 1.0 be used to calculate the runoff from a PMP event. A value of 1.0 means that it is assumed that every drop of rainfall turns into a drop of runoff, with no infiltration, detention or retention of any water on the pond cover. This assumption is conservative in the extreme. However, I have completed a redesign of the top swale on the pond cover and the front slope swale for the control and discharge of the runoff assuming a C = 1.0.

In addition to the use of the C = 1.0 value for determining runoff, the NRC now requires that the Mannings coefficient, n, and the sizing of riprap follow the results of test reported in NUREG/CR-4651, a report of results of riprap tests performed in flumes at Colorado State University and first published in May, 1987. The size of material tested in the CSU program included no sizes larger than about six inches. I have called NRC's attention to the fact that much of the riprap to be used at the L-Bar will be larger than the maximum size tested by CSU, and therefore, the results of the CSU tests might not be applicable to all of the L-Bar riprap. NRC (Ray Gonzales) has responded by saying that while our riprap sizes exceed the range of sizes tested by CSU, they believe that there are no better criteria to use and, therefore, the design guidelines in NUREG/CR-4651 should be used for the design of riprap at the L-Bar. The redesign of the top and front slope swales which I have just completed follows the design criteria of NUREG/CR-4651 and uses discharges resulting from a runoff coefficient of 1.0.

The other "deficient" area sited by NRC's letter states that NRC believes the diversion channel designs are "acceptable," but that the method used for design is "not conservative." NRC based their evaluation, based on calculations they made assuming gradually varied flow and using the computer program HEC-2.

In my meeting with Ray Gonzales on August 4, I described to him the conservatisms incorporated in the present design of the diversion channels. He stated that he was not aware that we had already incorporated several levels of conservatism and suggested that I write a letter enumerating the conservatisms in this design. I believe that the conservatisms are already clearly discernible in the design summary and the detailed calculations appended to that summary. However, for the sake of expediting the review and approval of the diversion system designs, it may be useful to list the design assumptions, methods and parameters taht have already led to a very conservative design of surface water diversion channels. These designs include the following sources of conservatism:

1. The probable maximum precipitation (PMP) one-hour local storm event was used for computing the design runoff to the diversion system. The PMP estimates were based on HMR #55A and provide an estimate of the largest-ever predicted storm event of a duration most likely to produce the greatest runoff for an area of less than one square mile, applicable to the L-Bar site. The authors of NUREG/CR-4620 have stated that the computational method for PMP rainfall intensities in such small watersheds are "extremely conservative" (NUREG/CR-4620, p. 12). The entire hydrograph for this storm and runoff event would last only a few hours, with the peak discharge rate providing the design basis discharge for all diversion channels. This peak discharge would last for much less than an hour. Therefore, for a design protection period of 200 to 1000 years, we have used the greatest-ever storm event with a recurrence interval well over 1000 years and for which the duration is less than an hour. Even if left unprotected against such a storm event, the diversion channels would suffer a relatively minor amount of scour during this very short period of peak discharge. Therefore, using this runoff event as the design basis event is a very conservative assumption and produces an extremely conservative input parameter (i.e., the design discharge).
2. The runoff coefficient, C, used for the calculation of runoff from the tributary areas to the diversion channels was 0.7. This value is equivalent to the lower end of the range used for asphalt pavements or roofs. Compared to natural ground it is roughly equivalent to the upper end of bare clay surfaces. The selected C value of 0.7 is very conservative when

compared to the recommended values presented in Tables 4.4, 4.5 and 4.6 in NUREG/CR-4620.

3. To simplify calculations an assumption was made that all flow entering a reach of a channel from the area tributary to that reach entered at the top of the reach as a slug or injection of discharge. As a result, even though uniform flow was assumed (again in order to simplify calculations), the channel dimensions in each reach were based on the maximum discharge that would occur at the bottom of the reach rather than the average discharge for the entire reach. Therefore, in every reach the dimensions are conservatively large, and this conservatism can be quantified roughly as the difference between the average discharge that would occur along the reach and the maximum discharge at the downstream end of the reach. For the south channel, for example, from station 0+00 to the discharge point, the discharge was assumed to be constant at 5185 cfs. This value is approximately 21 percent higher than the calculated discharge at the upper end of the south channel (4284 cfs) and is approximately equal to the discharge that would exit the bottom end of the channel.

4. As a result of a meeting with Ray Gonzales on December, 15, 1987 the maximum permissible velocity for all channels was changed from higher values given in standard hydraulics references for the materials expected in the channel bottom to a uniform and conservative value of 3.0 fps, a value deemed by Headquarters NRC to be appropriate for exposed soils. The test pit excavations and other site explorations have all indicated that we can expect most if not all channel beds to be on hardpan clay or shale. The maximum permissible velocity for such material is listed by most investigators (Fortier and Scobey, 1926; Lane, 1955; and Brater and King, 1976) to be 6.0 fps. The NRC calculations using the gradually varied flow method of calculation and the HEC-2 computer program indicate that velocities could be as high as 5.0 to 6.0 fps. My own calculations assuming gradually varied flow produce a maximum velocity near the discharge end of the south channel of 5.72 fps. Therefore, the present design still results in maximum velocities that are below the maximums considered permissible by most experts in the field. It should also be emphasized that these peak velocities would occur only during the peak of the PMF hydrograph (i.e., for less than an hour in 200 to 1000 years, if ever). Therefore, the channels have been designed to protect against the

greatest erosional stress which might occur in virtually an instant of time within a span of thousands of years.

5. The layout of the diversion channels is such that the erosion of the channel beds themselves will never expose tailings. None of these channels are located immediately adjacent to or on top of covered tailings. Therefore, even if the channel bed should erode, the eroded material will be natural ground, not contaminated materials.

The conservatisms described above are the most important of those used for the design of the diversion systems at the L-Bar. Some others include the conservative rounding of calculated numbers and the assumption of lineal flow (as opposed to sinuous natural channel flow) in determining the times of concentrations for discharge calculations. Given these multiple levels of conservatism already incorporated in the present design, it is quite apparent to me that the concerns expressed in item #2 of the NCR's letter of July 26 have been adequately addressed in the existing design. There is no need to change the design in the direction of greater conservatism; and, therefore, I recommend that a copy of this letter be forwarded to the NRC as an explanation of the current design and the adequacy of its conservatism.

Yours truly,

Alan K. Kuhn

Alan K. Kuhn

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8803

August 19, 1988

Mr. Scott Grace
Uranium Recovery Field Office
U.S. Nuclear Regulatory Commission
P.O. Box 25325
Denver, CO 80225

REDESIGN OF COVER SWALES USING NUREG/CR-4651 CRITERIA
L-BAR URANIUM OPERATIONS RECLAMATION PLAN

Dear Scott:

At Intera's request I am submitting the enclosed sketches and calculations for the redesign of the L-Bar cover swales for NRC review. This submittal responds to item #1 of NRC's letter of July 26, 1988. I met with Ray Gonzales on August 4 to discuss that letter and his concerns about the hydraulic designs. Following that meeting I redesigned the cover swales (top and front slope) using a runoff coefficient of 1.0 and the criteria of NUREG/CR-4651, which apparently represents the current NRC technical position on design of riprap. The decision to redesign to these criteria was taken after consultation with Intera and was based on considerations of expediency; I believe that NUREG/CR-4651 criteria have not been demonstrated to be applicable to riprap larger than d50=6", and a runoff coefficient of 1.0 is unreasonably conservative.

In a separate letter to Intera, I have addressed item #2 of NRC's July 26 letter. I understand that they will forward a copy of that letter to you. I am confident that it will demonstrate that the diversion channel design included more than sufficient conservatism.

NRC's expeditious review of the attached material will be greatly appreciated.

Yours truly,

Alan K. Kuhn

Alan K. Kuhn

~~8810130199~~ 13PP

PURPOSE: TO re-calculate runoff from the radon barrier cover using runoff coeff. (C) of 1.0 per NRC staff position (Ref. 1 below) and to redesign swales per criteria of ref. 3 below

References:

1) Telcon AKK to Ray Gonzales of USNRC, 7/15/88. Notes in AKK telcon record #5.

2) Calc. 84-103.C8 of 1/22/88 (Rev.0)

3) NUREG/CR-4651, May 1987

4) NUREG/CR-4620

5) French, 1985, Open-Channel Hydraulics

CALCULATIONS:

1) Hydrologic parameters for design storm (PMP) and its runoff from the cover.

1.1) Parameters from previous calcs. per Ref. 2,

One hour local PMP = 10.96"
L, longest flow path = 2800'
S, slope gradient = 0.002
T_c = 0.64 hrs = 38.5 min
rainfall depth = 9.95"
i intensity = 15.5"/hr.

1.2) Revised Parameters

→ C = runoff coeff. increased from 0.5 to 1.0

q = unit width discharge
= C i a

where a = 2800 ft²/43560 ft²/acre

CALCS (1.2 cont'd)

$q = 1.0(15.5)\,2800/43560 = 0.996$, say 1.0 cfs

Q = area discharge = CiA
area = A = 130 acres (Ref. 2)

$Q = CiA = 2015$ cfs

y = sheet flow depth

$= \left[\frac{q\,n}{1.486\,S^{1/2}}\right]^{3/5}$, $n = 0.03$

$= \left[\frac{1.0(0.03)}{1.486(0.002)^{1/2}}\right]^{3/5} = 0.62$ ft

V = sheet flow velocity

$= q/y = 1.0/0.62 = 1.61$ fps, < 3.0 fps allowed, OK

1.3) Flow convergence/concentration

q' for $V_{max} = 3.0$

$= V y' = 3.0\left[\frac{q'(0.03)}{1.486(0.002)^{1/2}}\right]^{3/5}$

$= 1.86\, q'^{3/5}$

$q'^{2/5} = 1.86$

$q' = 4.72$ cfs

$y' = 1.57$ ft, same as Rev. 0

r, radius from center of curvature to point of $V = 3.0$;
arc length at r, $\theta = 180°$

$= Q/q' = 2015/4.72 = 427'$

calcs (1.3 cont'd)

$$r = \frac{\text{arc length}}{\pi} = \frac{427}{\pi} = \underline{136 \text{ ft.}}$$

Alternative method, for check

$Q = AV = 3.0(A)$

$Y \simeq R$, hyd. radius

$$R = \left[\frac{V\,n}{1.486\,(S)^{1/2}}\right]^{1.5}$$

$= 1.58 = y'$, as in Rev. 0

$A = 2015/3.0 = \underline{672 \text{ ft}^2}$

arc length or x-section width of flow at $V = 3.0$

$= 672/y' = 672/1.58 = \underline{425'}$

vs. 427' by other method, OK

(Note: Because C = 1.0 is used as a very conservative value, the Rev. 0 conservatism of using a 120° arc for the flow source to calculate r for rock protection can be eliminated)

→ Swale with riprap is needed from center of drainage contours to <u>136' distance</u>

2.0) FRONT SLOPE SWALE DESIGN (Portion on 5H:1V front slope)

Q = 2015 cfs + rainfall on swale

= 2015 cfs + 0.78 cfs/A (Ref. 2 p.ll) × B × 2 (for increase to C = 1.0)

For estimating total Q assume B = 130', then

Q = 2219 cfs on front slope

CALCS (2.0 cont'd)

2.1) Select d_{50}, d_{100} of riprap

If b (channel bed width) $\simeq$ 130', the $q = 2219/130$

$q \simeq 17.1$ cfs/ft

This exceeds tested range of NUREG 4651, so use:

→ $\underline{d_{50} = 12''}$, $\underline{d_{100} = 17''}$

2.2) Determine n of riprap channel

$n = 0.0456 (d_{50} \times S)^{0.159}$ Ref. 3, Eqn 4.8

for $d_{50} = 12''$ & $S = 0.2$,

→ $\underline{n = 0.052}$

2.3) Determine allowable unit discharge using $q_{failure}$ of riprap per Fig. 4.3, Ref. 3 as upper limit.

· q_f for $S = 0.2$, $d_{50} = 12''$

→ $q_f = \underline{16.1 \text{ cfs}}$ extrapolating the CSU data

2.4) Determine necessary channel width, B

→ $B = Q/q_f = 2219/16.1 = \underline{138 \text{ ft.}}$

2.5) Determine amount of interstitial flow, q_i and Q_i

$$q^* = 0.079 \left(C_u^{-0.94} S^{0.46} \tfrac{1.07}{?} \right)^{0.999} \times (g\, d_{50})^{0.5}$$

$C_u = 1.62 = \frac{d_{60}}{d_{10}}$, assuming linear gradation & $d_{50} = 12''$, $d_{100} = 17''$ (See Attachment A)

CALCS (2.5 cont'd)

$S = 0.2$

$n_p = 0.45$ (Ref. 3, Table B.1)

$q^* = 0.53$ cfs/in

$q_i = q^* \times 17''$ thickness of riprap $= 9.0$ cfs

Q_i for $B = 138' = 9.0 \times 138' = 1242$ cfs

2.6) Determine Open Channel Flow, Q_o
$Q_{overflow}$ (open channel flow above riprap)

⟶ $Q_o = 138\,(16.1 - q_i) = \underline{980 \text{ cfs}}$

2.7) Determine depth of open channel flow

$$y_o = \left[\frac{q_o\, n}{1.486\sqrt{S}}\right]^{3/5}$$ Ref. 4 Eqn 4.46

$= \underline{0.70 \text{ ft.}}$

$y = 1.15$ ft. if no interstitial flow occurs

⟶ Use design depth of d_{100} of riprap, 17" and form swale banks with one lift of $d_{100} = 17''$ riprap (rectangular section)

CALCS (3.0)

3.0) TOP SWALE DESIGN

Design to provide a control section upstream of front swale to channelize flow, and a transition section upstream, between control section and the cover at S = 0.002 for directing radial flow to control section

3.1) Control section

Use B = B of front slope swale, 138'

Assume q = 16.1 cfs/ft although actually lower (2015/138 = 14.6 cfs/ft for an added 10% safety factor

→ S = 0.01 selected to be steeper than cover, flatter than radial section, to provide reduced velocities.

$d_{50} = 4''$ (Ref 3, Fig. 4.3)

$d_{100} = 6''$

→ $n = \underline{0.027}$ (Ref. 3, Eqn. 4.8)

y_0 = flow depth assuming no interstitial flow (conservative)

→ $y_0 = \left[\frac{16.1\ (0.027)}{1.486\sqrt{0.01}}\right]^{3/5} = \underline{2.05'}$

$V_0 = q_0/y_0 = 7.85$ fps

CALCS (3.1 cont'd)

Length of control section, set at 1/2 B, = 69'

(See Figure 84-103.C8.3)
Elevation at east end is
$5197 + 69(0.01) = 6197.69$

3.2) Transition or Radial Flow section

Outer limit = limit of riprap protection = 136 ft. from center of curvature (C.O.C) (page 3 these calcs)
See Figure 84-103.C8.3
Contour elev. is $6199 + 136 \times 1.002 = 6199.27$
Inner limit = outer boundary of control section

S, slope, will vary around this section, being steepest along 45° ∡ radii from COC:

$\Delta h / \Delta l = (6199.27 - 6197.69) / 44.5'$ (scaled)

$= \underline{0.0355}$

flatest along ℄:

$(6199.27 - 6197.69)/67' = \underline{0.0236}$

and $(6199.27 - 6197)/67 = \underline{0.0339}$
along break in slope (top of front slope)

3.2.1) Determine riprap sizes needed

At r = 69 from C.O.C
arc length = πr = 217

$q = 2015/217 = 9.3$ cfs/ft.

CALCS (3.2.1 cont'd)

q will range from 4.72 cfs/ft. at r = 136' from C.O.C. to 9.3 cfs/ft. at r = 69' from COC

For S = 0.02 to ~0.04, and q = 9.3, d_{50} = 4" will be suitable based on interpolations on Fig. 4.3, Ref. 3

3.2.2) Determine flow depth (disregarding interstitial flow) using max q. and S

$$Y = \left[\frac{9.3\,(0.033)}{1.486\sqrt{0.0355}}\right]^{3/5} = \underline{\underline{1.06'}}$$

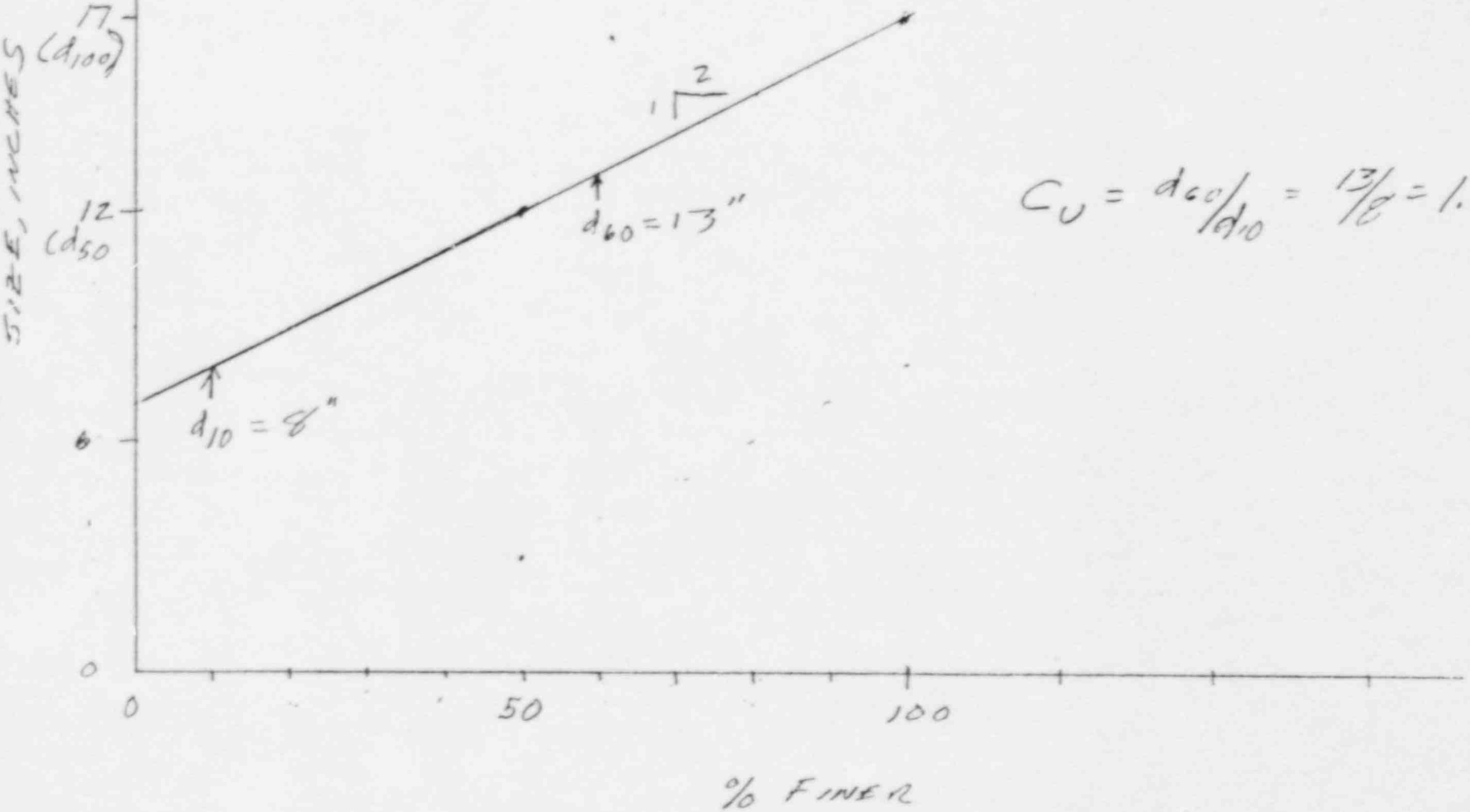
where $n = 0.0456\left(4" \cdot 0.0355\right)^{0.159} = 0.033$

<u>Contour</u> transition or radial

section to provide smooth, gradual changes from radially converging flow to channelized flow in control section per Figures 84-103.C8.3, C8.4, and C8.5)

CALC. 84-103.C8 Rev.1
ATTACHMENT A

UNIFORMITY COEFFICIENT OF RIPRAP WITH d_{50}=12", d_{100}=17", ASSUMING LINEAR DISTRIBUTION



$$C_U = d_{60}/d_{10} = 13/8 = 1.62$$

FIGURE 64-103.C8.3
L-BAR REC. PLAN
COVER AND FRONT SLOPE
SWALE DESIGN DIMENSIONS
REVISION OF 8/18/88
A. K. KUHN

PLAN VIEW
1" = 40'

RADIAL INFLOW SECTION
radius 136'
S = 0.0236 to 0.0355
Riprap $d_{50} = 4''$, $d_{100} = 6''$
Design Q = 2015

CONTROL SECTION
138' wide, 69' long
S = 0.01
Riprap $d_{50} = 4''$, $d_{100} = 6''$
Design Q = 2219

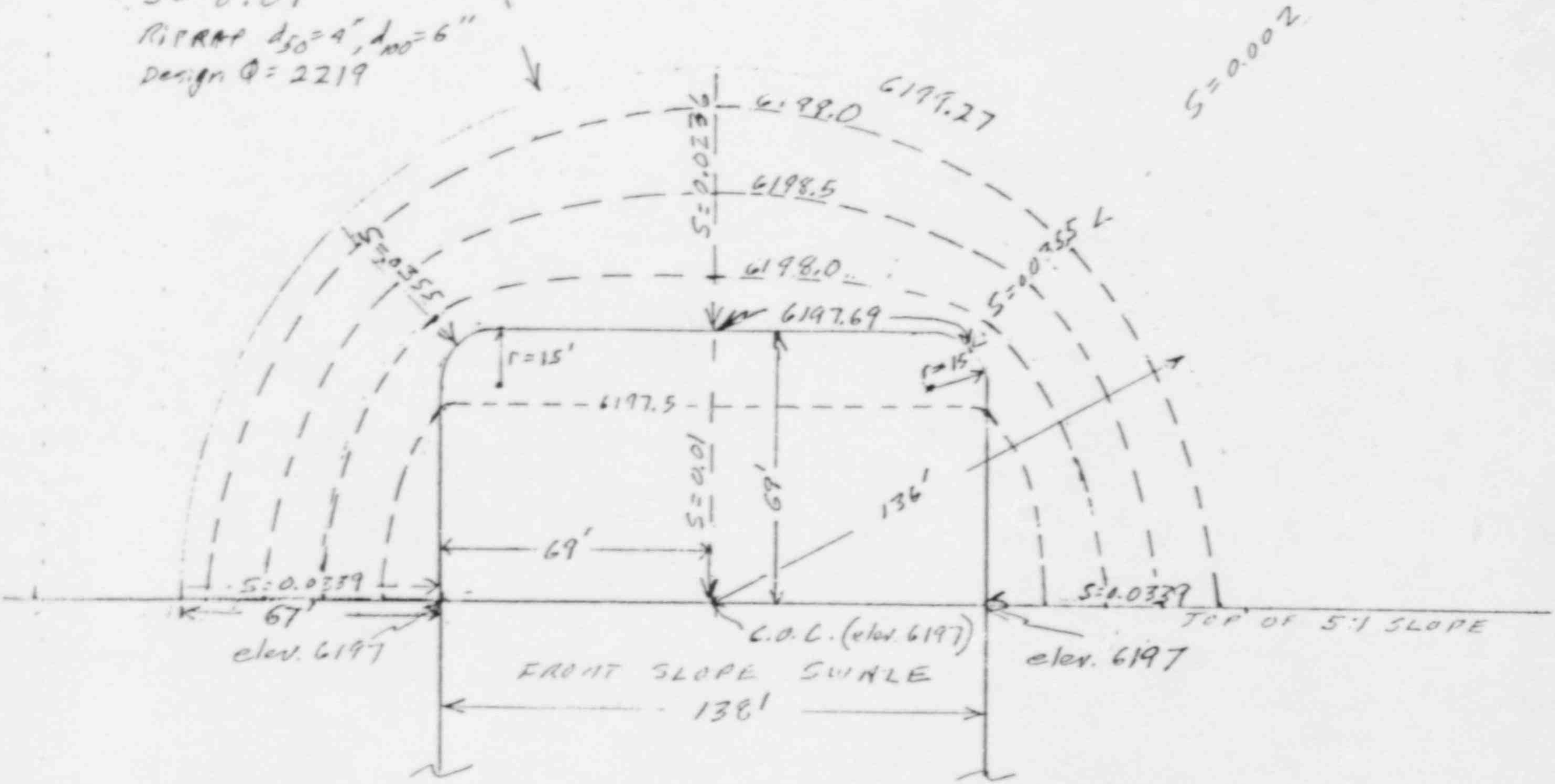
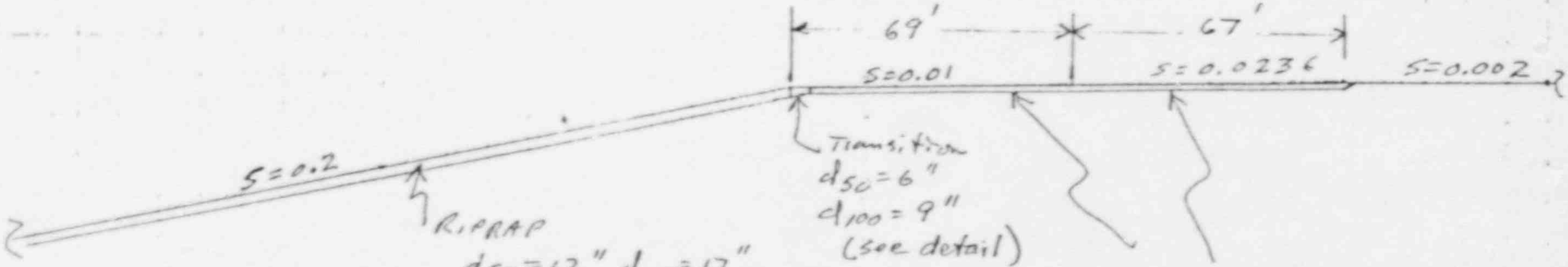


FIGURE 84-103.C8.4

L-BAR REC. PLAN
COVER AND FRONT SLOPE.
SWALE DESIGN DIMENSIONS
REVISION OF 8/18/88
A.K. KUHN

SECTION ALONG CENTERLINE
1" = 40, H & V



Transition
$d_{50}=6"$
$d_{100}=9"$
(see detail)

RIPRAP
$d_{50}=12"$, $d_{100}=17"$
thickness 17" min.
filter layer 6" min.

RIPRAP
$d_{50}=4"$, $d_{100}=6"$
thickness 12" min.
filter layer 6" min.

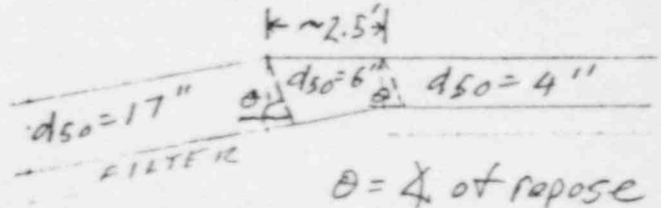
FILTER LAYERS

$$\frac{d_{15}\ (\text{riprap})}{d_{85}\ (\text{filter})} < 5$$

$$5 < \frac{d_{15}\ (\text{riprap})}{d_{15}\ (\text{filter})} < 40$$

$$\frac{d_{50}\ (\text{riprap})}{d_{50}\ (\text{filter})} < 50$$

TRANSITION DETAIL



θ = ∠ of repose

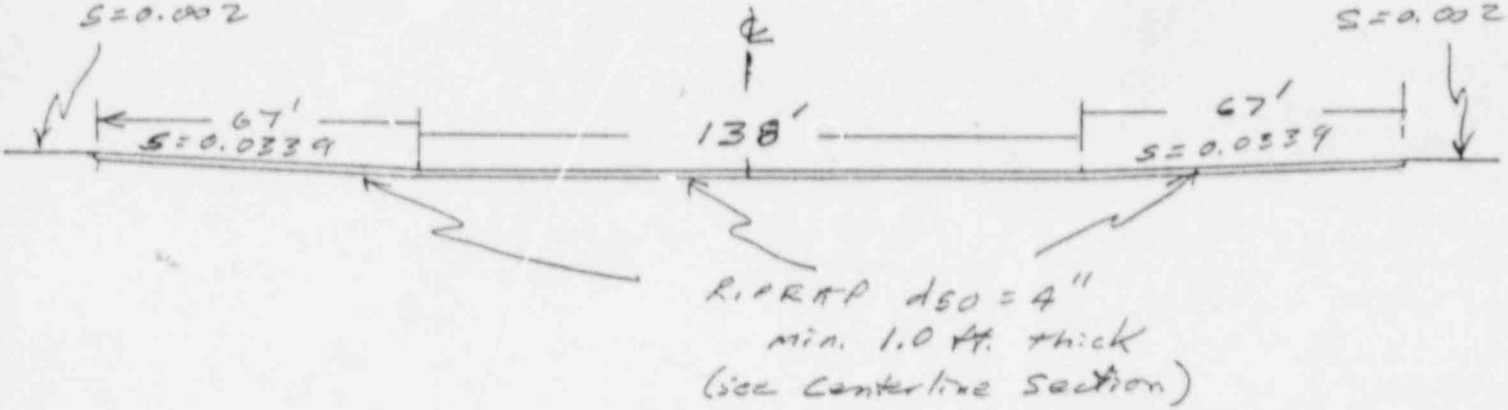
1" = 4'

FIGURE 84-103.C8.5

L-BAR REC. PLAN
COVER AND FRONT SLOPE
SWALE DESIGN DIMENSIONS

REVISION OF 8/18/88
A.K. KUHN

SECTION ACROSS TOP SWALE AT CREST
1" = 40'



SECTION ACROSS FRONT SLOPE SWALE
1" = 40'