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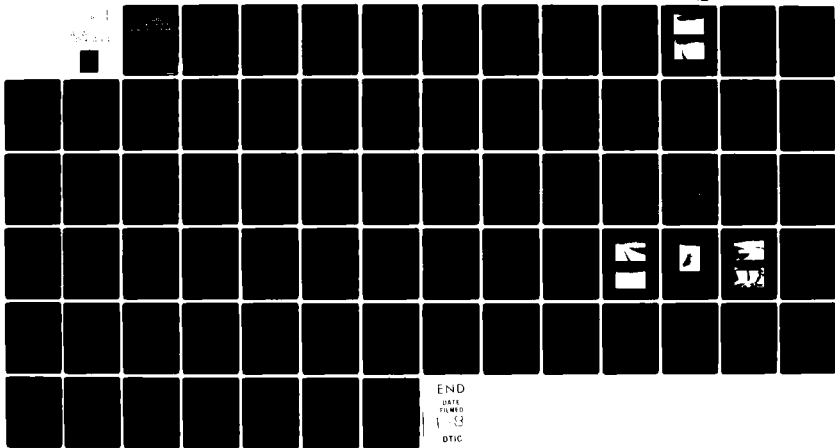
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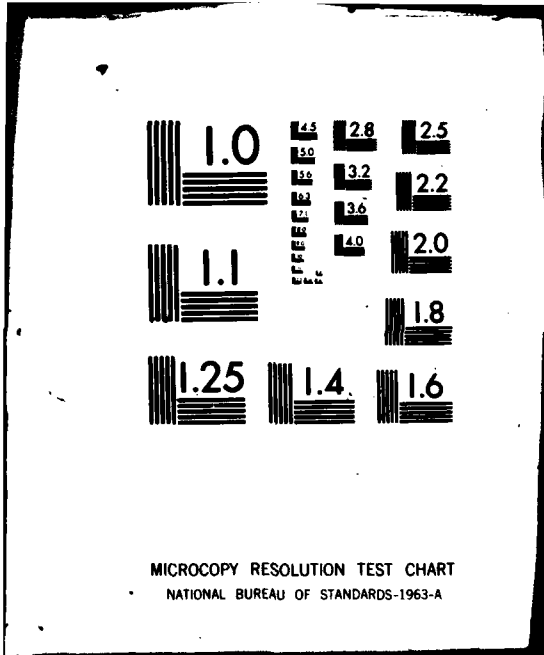
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POTOMAC RIVER BASIN

Name Of Dam: NEWMAN LAKE DAM

Location: HARRISONBURG, VIRGINIA

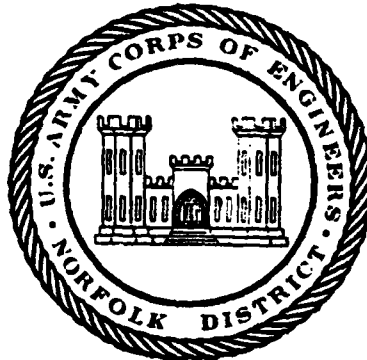
Inventory Number: VA. NO. 66001

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PHASE I INSPECTION REPORT NATIONAL DAM SAFETY PROGRAM



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PREPARED FOR

NORFOLK DISTRICT CORPS OF ENGINEERS
803 FRONT STREET
NORFOLK, VIRGINIA 23510

BY

SCHNABEL ENGINEERING ASSOCIATES, P.C./
J. K. TIMMONS AND ASSOCIATES, INC.

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20. Abstract

Pursuant to Public Law 92-367, Phase I Inspection Reports are prepared under guidance contained in the recommended guidelines for safety inspection of dams, published by the Office of Chief of Engineers, Washington, D. C. 20314. The purpose of a Phase I investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general conditions of the dam is based upon available data and visual inspections. Detailed investigation and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I investigation; however, the investigation is intended to identify any need for such studies.

Based upon the field conditions at the time of the field inspection and all available engineering data, the Phase I report addresses the hydraulic, hydrologic, geologic, geotechnic, and structural aspects of the dam. The engineering techniques employed give a reasonably accurate assessment of the conditions of the dam. It should be realized that certain engineering aspects cannot be fully analyzed during a Phase I inspection. Assessment and remedial measures in the report include the requirements of additional indepth study when necessary.

Phase I reports include project information of the dam and appurtenances, all existing engineering data, operational procedures, hydraulic/hydrologic data of the watershed, dam stability, visual inspection report and an assessment including required remedial measures.

POTOMAC RIVER BASIN

NAME OF DAM: NEWMAN LAKE DAM
LOCATION: HARRISONBURG, VIRGINIA
INVENTORY NUMBER: VA. NO. 66001

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

*Newman Lake Dam (Inventory Number VA
66001), Potomac River Basin
Harrisonburg, Virginia
Potomac River Basin*

PREPARED BY
NORFOLK DISTRICT CORPS OF ENGINEERS
803 FRONT STREET
NORFOLK, VIRGINIA 23510

BY

SCHNABEL ENGINEERING ASSOCIATES, P.C./
J. K. TIMMONS AND ASSOCIATES, INC.

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PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams, for Phase I Investigations. Copies of these guidelines may be obtained from the Office of Chief of Engineers, Washington, D. C., 20314. The purpose of a Phase I Investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspections. Detailed investigation, and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I Investigation; however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team. In cases where the reservoir was lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam will continue to represent the condition of the dam at some point in the future. Only through frequent inspections can unsafe conditions be detected and only through continued care and maintenance can these conditions be prevented or corrected.

Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established Guidelines, the Spillway Test flood is based on the estimated "Probable Maximum Flood" for the region (greatest reasonably possible storm runoff), or fractions thereof. Because of the magnitude and rarity of such a storm event, a finding that a spillway will not pass the test flood should not be interpreted as necessarily posing a highly inadequate condition. The test flood provides a measure of relative spillway capacity and serves as an aid in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition and the downstream damage potential.

Name of Dam: Newman Lake Dam
State: Virginia
City: Harrisonburg
USGS Quad Sheets: Bridgewater and Harrisonburg
Coordinates: Lat 38° 25.9' Long 78° 52.5'
Stream: Branch of Blacks Run
Date of Inspection: April 15, 1980

BRIEF ASSESSMENT OF DAM

Newman Lake Dam is a homogeneous earthfill structure about 400 ft long and 17 ft high. The principal spillway consists of a rectangular concrete overflow weir 20 ft wide by 24 ft long with overflow on three sides. The weir is connected to a double 10 ft by 12 ft culvert which extends through the structure. The top of the dam serves as Virginia Route 331 with a 24 ft wide pavement. The structure is a "small" size dam and has a "significant" hazard potential. The dam is located on a branch of Blacks Run on the James Madison University Campus in Harrisonburg, Virginia. The lake is for recreational purposes and is owned and maintained by the James Madison University.

Based on criteria established by the Department of the Army, Office of the Chief of Engineers (OCE), the approximate Spillway Design Flood (SDF) is the $\frac{1}{2}$ PMF. The spillway will pass 20 percent of the Probable Maximum Flood (PMF) or 40% of the SDF without overtopping the dam. During the SDF, the dam will be overtopped to a depth of 2.3 ft maximum, at a maximum velocity of 8 fps, and will be overtopped for a period of 3.5 hours, assuming no downstream restriction. A roadway embankment with triple box culvert crosses the channel approximately 200 ft downstream. If this downstream restriction

remains intact, it will be overtopped by 4 ft during the SDF creating a tailwater condition upstream at the dam of 6 ft above the low point. The spillway is judged inadequate but not seriously inadequate.

An evaluation of the stability condition could not be made since there is insufficient design and construction data for this structure. The visual inspection revealed the presence of some seepage along the downstream slope. Since the embankment slopes and crest generally meet U.S. Bureau of Reclamation requirements and because the dam has been subjected to rapid drawdown with no adverse effects, a stability check is not required. Overtopping of the dam is not considered detrimental since overtopping will likely occur as a result of tailwater conditions created by the downstream restriction.

It is recommended that the Owner implement the following remedial measures within one year of the date of this report:

- (1) Evaluate the downstream roadway embankment to determine what measures are required to protect the roadway from breaching during periods of overtopping. This should be performed by a qualified Professional Engineer.

- (2) An emergency action plan should be implemented to warn downstream dwellings of any dangers which may be imminent.

The following routine maintenance and observation functions should be initiated:

- (1) The seepage observed along the downstream slope should be monitored quarterly and after periods of high pool levels to detect any increase in flow rates which may cause piping within the embankment.

(2) Trees should not be allowed to grow on the embankment. All existing trees should be cut to the ground. Trees greater than 3 inches in diameter should also have their stumps and root structures removed and resulting holes backfilled with compacted soil.

(3) Uncontrolled vegetation near the outlet structure should be cut and maintained in the future.

(4) Muskrat burrowing in the embankment should be backfilled.

(5) A staff gage should be installed to monitor water levels.

Prepared by:

SCHNABEL ENGINEERING ASSOCIATES, P.C./
J. K. TIMMONS & ASSOCIATES, INC.

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Jack G. Starr, P.E., R.A.
Chief, Engineering Division

Date: _____

AUG 15 1980



Reservoir



Dam

OVERVIEW PHOTO

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
NEWMAN LAKE DAM
VA. NO. 66001

SECTION I - PROJECT INFORMATION

1.1 General:

1.1.1 Authority: Public Law 92-367, 8 August 1972, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a national program of safety inspections of dams throughout the United States. The Norfolk District has been assigned the responsibility of supervising the inspection of dams in the Commonwealth of Virginia.

1.1.2 Purpose of Inspection: The purpose is to conduct a Phase I inspection according to the Recommended Guidelines for Safety Inspection of Dams (see Reference 1, Appendix V). The main responsibility is to expeditiously identify those dams which may be a potential hazard to human life or property.

1.2 Project Description

1.2.1 Dam and Appurtenances: Newman Lake Dam is a homogeneous earthfill structure approximately 400 ft long and 18 ft high.* The top of the dam is 35 ft wide and accommodates Virginia State Route 331 (24 ft wide asphalt pavement) across the length of the dam. Side slopes range from approximately 2.5 horizontal to 1 vertical (2.5:1) to 3 horizontal to 1 vertical (3:1) on both the upstream and downstream sides. The top of the dam is at elevation 1296 ft msl.

* Height is measured from the top of the dam to the downstream toe at the centerline of the stream.

It is not known whether the dam is keyed into the foundation or if there is an internal drainage system. No drain outlets were encountered. Existing vegetation on the embankment slopes provide adequate slope protection.

The principal spillway consists of a 20 ft x 24 ft reinforced concrete overflow weir with overflow on three sides (effective length is 64 ft). The weir is connected to a double 10 ft wide x 12 ft high box culvert which runs through the dam. The weir crest is at elevation 1290 msl. A 24 inch square sluice gate in the weir at elevation 1279.3 msl is used to drain the lake. The double box culvert runs approximately 54 ft through the embankment with an invert elevation at the weir of 1278.8 msl and an invert elevation at the outlet structure of 1278 msl (See Plates No. 5 and 8, Appendix I).

1.2.2 Location: Newman Lake Dam is located on a branch of Blacks Run on the James Madison University Campus in Harrisonburg, Virginia (see Plate No. 1, Appendix I).

1.2.3 Size Classification: The dam is classified as a "small" size structure because of the maximum lake storage potential.

1.2.4 Hazard Classification: The dam is located in a suburban area, however, based upon the downstream proximity of five homes located one-half mile downstream, the dam is assigned a "significant" hazard classification. The hazard classification used to categorize a dam is a function of location only and has nothing to do with its stability or probability of failure.

1.2.5 Ownership: Commonwealth of Virginia, James Madison University, owns and operates the dam.

1.2.6 Purpose: Recreation

1.2.7 Design and Construction History: The dam was designed and constructed under the supervision of the Virginia Department of Highways and Transportation as part of State Route 331. There is no record of who actually constructed the dam, however, construction is believed to have been completed in 1966.

1.2.8 Normal Operational Procedures: The principal spillway is ungated, therefore, water rising above the crest of the weir inlet automatically is discharged downstream. Normal pool is maintained at elevation 1290 msl at the crest of the overflow weir.

1.3 Pertinent Data:

1.3.1 Drainage Areas: The drainage area is 2.88 square miles.

1.3.2 Discharge at Dam Site: Maximum known flood at the dam site occurred in April 1977 and an estimated pool elevation of 1291 was observed.

Principal Spillway Discharges:

Pool Elevation at Crest of Dam (elev 1296) 2822 CFS

1.3.3 Dam and Reservoir Data: See Table 1.1, below:

Table 1.1 DAM AND RESERVOIR DATA

Item	Elevation Feet msl	Reservoir			
		Area Acres	Storage Volume Acre Feet	Watershed Inches	Length Miles
Crest of Dam	1296 (a)	19	220	1.4	.30
Principal Spillway Crest	1290	12	112	.73	.28
Streambed at Down- stream Toe of Dam	1278	-	-	-	-

(a) Low point in dam

SECTION 2 - ENGINEERING DATA

2.1 Design: The dam was designed and constructed under the direction of the Virginia Department of Highways and Transportation (VDHT). Design data and construction specifications are available at the VDHT Staunton District Office. The hydrologic and hydraulic design report was not available and a stability analysis was not performed.

There is no information available concerning the construction and completion date of the dam. A revision date of November 15, 1965 was noted on one drawing and therefore it is assumed construction was completed in 1966. Mr. George L. Marcum, Superintendent of Buildings and Grounds, is not aware of the completion date as he was not employed by James Madison at that time, and there is no information on file concerning construction of the dam.

Comparison of approximate field measurements and the design drawings (Appendix I) indicates the "as built" structure may be slightly different than designed. An upstream slope of 2 horizontal to 1 vertical (2:1) was specified in design (Plate No. 3, Appendix I), however, slopes of $2\frac{1}{2}$:1 to 3:1 were measured in the field. The downstream slope also appears to be more gentle than provided in design. A small berm ranging from $3\frac{1}{2}$ to 6 ft extends along the upstream toe above pool level. This berm is not shown in the design drawings.

A seepage problem was apparently recognized by university personnel, and the USDA, Soil Conservation Service (SCS), was requested to investigate this problem. Conclusions and recommendations developed by the SCS are included as Appendix IV. It is not known whether any of the recommended remedial measures were ever implemented. However, the presence of the berm, and 2½:1 to 3:1 slopes on the upstream side appear to correspond to recommendations made by SCS personnel.

2.2 Evaluation: Engineering calculations are not available and there are no records available for dam performance. Design drawings provided by VDHT appear to be generally representative of the "as-built" structure. There is insufficient information to evaluate the foundation conditions and the embankment stability.

SECTION 3 - VISUAL INSPECTION

3.1 Findings: At the time of inspection, the dam was in good condition. Field observations are outlined in Appendix III.

3.1.1 General: An inspection was made 15 April 1980 and the weather was cloudy with a temperature of 45°F. The pool and tailwater levels at the time of inspection were 1290.1 and 1278.5 msl, respectively, which correspond to normal levels. Ground conditions were damp at the time of inspection. No previous inspection reports were available. A seepage evaluation report prepared in conjunction with a site visit made by SCS personnel in late 1968 or early 1969 is included as Appendix IV.

3.1.2 Dam and Spillway: The embankment slopes were grassed and well maintained at the time of inspection. Slopes ranged from about 2½:1 to 3:1 on both the upstream and downstream side. Numerous decorative trees (several inches in diameter) have recently been planted along the upstream slope of the dam. Slightly larger white pines occur along the crest and downstream embankment slope. Some uncontrolled vegetation was present in the less accessible areas, particularly near the box culvert. The crest of the dam is occupied by a paved road. Only a few small erosion washes (less than 1 ft wide and 1 ft deep) were encountered on the upstream slope. A 6 inch[±] diameter void exists along the northeast corner of the concrete intake structure. This void is believed to be the result of muskrat burrowing. Numerous muskrat holes also exist in a 10 ft[±] wide area, about 85 ft left of the intake structure on the upstream slope, just above pool level. A 2 ft[±]

vertical wave cut notch exists at pool level along the small berm (3½ to 6 ft wide) present along the upstream slope. The above described areas are illustrated on the field sketch, Sheet 2 of Appendix III.

Seepage was observed in an area approximately 60 ft long and 20 ft wide along the toe of the downstream slope at a point roughly 125 ft left of the spillway centerline. Although no flow or turbidity was observed, the area did include iron staining. Much of the area below the downstream slope was water saturated as a result of previous rainfall. Only those areas exhibiting iron staining were identified as seepage. It is likely that the wet marshy area near the box culvert is also the result of seepage through the dam. A field sketch is provided as Sheet 1 of Appendix III.

The embankment ties into the grassed abutment areas. No erosion was observed and there were no soil or bedrock exposures in either abutment. The embankment appears to be constructed with silty clay soils which include some fine to coarse sand, rock fragments and scattered limestone boulders. The only bedrock encountered was several limestone outcrops exposed along the right downstream channel, 300 ft[±] below the double box culvert. The bedrock surface was weathered and irregular. No faults were observed in the field during this inspection and geologic maps of the area do not show the presence of faults in the immediate vicinity.

The intake structure showed no signs of deterioration and the drain gate was reportedly in operational condition. The double 10 ft x 12 ft

outlet culvert showed no signs of deterioration and the riprap outlet pool was intact.

3.1.3 Reservoir Area: The reservoir area was free of debris and the perimeter was grassed. The reservoir is located in a valley with side slopes at approximately 10:1. No sediment buildup was detected near the intake structure.

3.1.4 Downstream Area: The downstream channel consists of a 15 ft wide channel located in a valley with side slopes of 3:1. The channel is grass-lined and lightly wooded. Approximately 200 ft downstream of the outlet structure, a roadway embankment crosses the stream. The embankment has a top elevation at the low point of 1298 msl and a triple 10 ft x 10 ft box culvert through it. Approximately one-half mile downstream there are five homes about 15 ft above the streambed.

3.1.5 Instrumentation: No instrumentation (monuments, observation wells, piezometers, etc.) was encountered for the structure. There was no staff gage for this structure.

3.2 Evaluation:

3.2.1 Dam and Spillway: Overall, the dam was in good condition at the time of inspection. A routine maintenance program exists for this structure. The presence of trees on an embankment may promote the development of deep rooted vegetation and this type of growth can encourage piping within an embankment. All trees presently growing on the embankment should be cut to the ground and trees greater than 3 inches in diameter should also have their stumps and root systems removed. The resulting holes should be backfilled with compacted soil. Uncontrolled vegetation near the outlet structure should be cut and maintained in the future.

The iron-stained water encountered along the downstream slope represents seepage through the dam. No turbidity was noted during the inspection. This condition does not present a hindrance to the normal functioning of the dam at this time, however, it is recommended that the seepage along the downstream slope be monitored quarterly to detect any increase in flow rates which may cause piping within the embankment. Seepage in this same general area was evaluated in 1969 by the SCS and it is not known whether recommended remedial measures were ever implemented. If increased flows should occur, a Professional Engineer with expertise in Geotechnical Engineering should be contacted to evaluate the problem and make recommendations for required corrective measures.

The shallow washes described on the upstream slope do not present any problem. The presence of a good vegetative cover at wave level on the upstream slopes appears to be controlling wave erosion, consequently corrective measures are not believed necessary. The muskrat holes do not presently create an unsafe condition, however, future burrowing could result in numerous voids in the embankment which could be potentially hazardous under certain conditions. It is recommended that the existing holes be backfilled and that any future burrows be backfilled as they appear.

The intake and outlet structures are in good structural condition. Riprap present in the discharge channel is also in good condition.

A staff gage should be installed to monitor pool elevations.

3.2.2 Downstream Area: The location of the roadway embankment immediately downstream of the dam will create a buffer if the dam is breached, however, the roadway would be overtopped sufficiently to create a surge in water level downstream. The dwellings could possibly be jeopardized by a dam breach.

SECTION 4 - OPERATIONAL PROCEDURES

4.1 Procedures: Newman Lake is used for recreational purposes. The normal pool elevation is maintained by an overflow weir inlet acting as the principal spillway. Water flows automatically over the weir as the lake level rises above the crest of the weir.

4.2 Maintenance of Dam and Appurtenances: Maintenance is the responsibility of the James Madison University and the VDHT. Maintenance consisting of inspection, debris removal, mowing of the vegetative cover, and repair is completed routinely. The operating appurtenances are reportedly in working order.

4.3 Warning System: No warning system exists.

4.4 Evaluation: Maintenance of the dam is considered adequate, and complete records of maintenance and inspections should be maintained for future reference. An emergency operation and warning plan should be developed. It is recommended that a formal emergency procedure be prepared and furnished to all operating personnel. This should include:

- a) How to operate the dam during an emergency.
- b) Who to notify, including public officials, in case evacuation from the downstream area is necessary.

SECTION 5 - HYDRAULICS/HYDROLOGIC DATA

5.1 Design: No hydraulic/hydrologic data is available.

5.2 Hydrologic Records: There are no records available.

5.3 Flood Experience: An estimated maximum pool elevation of 1291 msl occurred in April 1977.

5.4 Flood Potentials: In accordance with the established guidelines, the spillway design flood is based on the estimated "Probable Maximum Flood" (flood discharges that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region), or fractions thereof. The Probable Maximum Flood (PMF), $\frac{1}{2}$ PMF and 100 year flood hydrographs were developed by the SCS method (Reference 4, Appendix V). Precipitation amounts for the flood hydrographs of the PMF and 100 year flood are taken from U. S. Weather Bureau Information (References 5 and 6, Appendix V). Appropriate adjustments for basin size and shape were accounted for. These hydrographs were routed through the reservoir to determine maximum pool elevations.

5.5 Reservoir Regulations: For routing purposes, the pool at the beginning of flood was assumed to be at elevation 1290 msl. Reservoir stage-storage data and stage-discharge data were determined from the available plans, field measurement and USGS quadrangle sheets.

Floods were routed through the reservoir using the principal spillway discharge up to a pool storage elevation of 1296 msl and a combined spillway and non-overflow section discharge for pool elevations above 1296. Floods were also routed through the roadway culvert 200⁺ ft downstream of dam using the dam spillway discharge data as the inflow hydrograph. Overtopping of the road embankment was assumed at elevation 1298 msl.

5.6 Overtopping Potential: The predicted rise of the reservoir pool and other pertinent data were determined by routing the flood hydrographs through the reservoir as previously described. The results for the flood conditions (PMF, $\frac{1}{2}$ PMF and 100 year flood) are shown in the following Table 5.1:

TABLE 5.1 RESERVOIR PERFORMANCE

	Normal Flow	Hydrograph		
		100 Year	½ PMF	PMF
Peak Flow, CFS				
Inflow	3	2241	7171	14,341
Outflow	3	2059	7171	14,159
Maximum Pool Elevation				
Ft, msl (a)	-	1294.86	1298.28	1300.35
Non-Overflow Section				
(Elev 1296 msl) (b)				
Depth of Flow, Ft	-	-	2.28	4.35
Duration, Hours	-	-	3.5	5.0
Velocity, fps (c)	-	-	8.0	9.1
Principal Spillway				
(elev 1290 msl)				
Depth of Flow, Ft	-	4.86	8.28	10.35
Duration, Hours	-	12.0	14.0	14.0
Velocity, fps	-	8.6	15.0	18.3
Tailwater Elevation,				
Ft, msl (d)	1278.5	1289.2	1302	1306

- (a) Ignores influence of downstream restriction
- (b) Low point on dam
- (c) Critical velocity at control section
- (d) Control at downstream culvert and road crossing

5.7 Reservoir Emptying Potential: A 24-inch square gate at elevation 1279.3 msl is capable of draining the reservoir through the outlet culverts. Assuming that the lake is at normal pool elevation (1290 msl) and there is 3 cfs inflow, it would take approximately 2 days to lower the reservoir to elevation 1280 msl.

This is equivalent to an approximate drawdown rate of 5.4 ft per day based on the hydraulic height measured from normal pool to gate invert divided by the time to dewater the reservoir.

5.8 Evaluation: The U.S. Army Corps of Engineer's guidelines indicate the appropriate Spillway Design Flood (SDF) for a small size significant hazard dam is the 100 year flood to $\frac{1}{2}$ PMF. Because of the risk involved, the $\frac{1}{2}$ PMF has been selected as the SDF. The spillway will pass 20 percent of the PMF (40% of the SDF). The SDF will overtop the dam a maximum of 2.3 ft, and remain above the dam for 3.5 hours with a critical velocity of 8 fps, if the downstream restriction did not exist. With the downstream restriction, the dam will be submerged for a greater depth and the reservoir elevation will be the same as the water surface elevation at the downstream restriction. The downstream roadway embankment will be overtopped by 4 ft during the SDF creating a tailwater condition upstream at the dam of 6 ft above the low point in the dam.

SECTION 6 - DAM STABILITY

6.1 Foundation and Abutments: The dam is located in the Valley and Ridge Physiographic Province of Virginia. The structure and eastern part of the impoundment are underlain by the Beekmantown Formation of lower Ordovician Age. This formation is up to 2000 ft thick and consists basically of thick-bedded gray dolomite with some blue limestone interbeds and considerable chert. The western portion of the impoundment is believed to be underlain by the younger New Market and Lincolnshire limestones. The impoundment is located on the east limb of the Harrisonburg syncline. Bedrock in the surrounding area strikes to the northeast and generally dips 30 degrees to the northwest. Local dip reversals in the bedrock are likely as indicated in the outcrops exposed immediately downstream, where dips of 70 degrees to the southeast were measured. No bedrock or faults were observed at the site.

There is very little subsurface data available for the structure. It is not known if a cutoff trench exists beneath the dam. Hand auger borings performed by SCS personnel encountered permeable topsoil underlain by essentially impermeable clay. Test borings by VDHT made in the vicinity of the intake structure, box culvert and wing walls, approximate the top of rock as ranging from elev 1281± to elev 1257±. (See Plate No. 9, Appendix I) Detailed descriptions of the overburden soils were not available. Based upon brief examination of the surrounding area, it would appear that the dam rests in part upon a thin stratum of alluvial or stream deposited soils consisting of assorted mixtures of sand, silt and clay materials. Natural permeabilities

ranging from low to medium are likely. The underlying residual soils, which are derived from the in-place weathering of limestone and dolomite bedrock, probably consist of silty clays and clays possessing low to very low natural permeabilities. Gradual consolidation of underlying soils probably had essentially fully developed under the applied load not long after completion of construction. Based upon the performance history of this dam, a stable foundation is assumed.

6.2 Embankment:

6.2.1 Materials: No detailed information is available concerning the soil materials used to construct the dam. Based upon the visual inspection the embankment appears to be constructed with silty clay (CL) residual soils and includes fine to coarse sand and rock fragments. Scattered limestone boulders were exposed in the fill along the downstream slope. The following materials specifications were obtained from Plate No. 3, Appendix I:

- (a) "The fill between Sta. 10+25[±] and Sta. 16+75[±] shall not contain more than 20% rock. Any rock excavation placed in this fill shall be evenly distributed throughout the entire fill so as not to leave any pockets containing more than 20% rock."
- (b) Sta. 10+25 to Sta. 16+75[±] - "Embankment material shall consist of a rock-free clay and compacted to 100% density. Any of the soils on the project will meet the specification if rock-free. The ideal soil for this overlay is found between Sta. 17+00 and 21+00 between 10 ft and 18 ft below the surface."

6.2.2 Subdrains and Seepage: There is no known drainage system and apparently no foundation drain outlets. Seepage observed during this inspection (See Sheet 1, Appendix III) is believed in the same general location as the seepage investigated by SCS personnel in late 1968 or 1969. The report developed in conjunction with this earlier inspection is included as Appendix IV. It was concluded in the SCS report that this seepage did cause "some instability of the dam which could result in failure of the structure." It was proposed that either an upstream cutoff or a downstream drainage system be constructed to help alleviate the seepage problem. Neither James Madison University or the local SCS office have any information verifying whether any of the recommendations were implemented. The presence of the narrow berm along the upstream slope just above water level (See Photograph No. 1 Appendix II) is believed to be the top of the upstream clay berm recommended in the SCS report. This berm was not included in the original dam design.

6.2.3 Stability: There are no available stability calculations. The dam is 17 ft high and has a bottom width of approximately 90 ft and crest width of 35 ft along the principal spillway section of the dam (Sta. 13+50, Plate No. 6, Appendix I). Both the upstream and downstream slopes range from 2.5H:1V to 3H:1V. The dam is subject to rapid draw-down, because the approximate reservoir draw-down rate of 5.4 ft per day exceeds the critical rate of 0.5 ft/day for earth dams. Design drawings (Plate No. 2, Appendix I) show an upstream slope of 1.5 H:1V with a specified compacted blanket of variable thickness (2H:1V slope) and keyed into the ground 2 ft. Based upon existing slopes and the

presence of the berm, it is assumed that the design was altered during construction. For stability purposes, the structure was assumed to be homogeneous and constructed with CL to CH soils. According to the guidelines present in Design of Small Dams, U.S. Department of the Interior Bureau of Reclamation, for small homogeneous dams with a stable foundation subjected to rapid drawdown and composed of CL to CH materials, the recommended slopes range from 2.5H:1V to 3.5H:1V for the downstream and upstream slopes respectively. The recommended crest width is 14 ft.

6.2.4 Seismic Stability: The dam is located in Seismic Zone 2. Therefore, according to the Recommended Guidelines for Safety Inspection of Dams, the dam is considered to have no hazard from earthquakes provided static stability conditions are satisfactory and conventional safety margins exist.

6.3 Evaluation: An accurate check on the stability of this structure cannot be made since there is insufficient design and construction data. The crest width and downstream embankment slope meet the requirements recommended by the U.S. Bureau of Reclamation, however, the upstream slope is slightly steeper than recommended when subject to rapid drawdown. Since no undue settlement, cracking, or seepage was noted at the time of inspection, it appears that the embankment is adequate for maximum control storage with water at elevation 1290 msl. The dam has been subjected to rapid drawdown with no adverse effects on the embankment. Overtopping of the dam would normally be considered detrimental because the critical velocity of 8 fps exceeds the effective eroding velocity (6 fps) for a vegetated

earth embankment. However, tailwater conditions created by the downstream restriction will likely rise to the crest of the dam before overtopping can occur, thus eliminating the potential for erosion. Tailwater conditions will allow approximately 20 percent of the PMF (40 percent of the SDF) to be passed without creating a backwater condition higher than the dam.

The iron-stained water encountered along the downstream slope represents seepage through the dam. This condition does not present a hindrance to the normal functioning of the dam at this time, however, the seepage should be monitored quarterly to detect any increase in flow rates which may cause piping within the embankment. If increased flows should occur, a Professional Engineer with expertise in Geotechnical Engineering should be contacted to evaluate the problem and make recommendations for required corrective measures.

SECTION 7 - ASSESSMENT/REMEDIAL MEASURES

7.1 Dam Assessment: The Newman Lake Dam at the time of inspection appeared in good condition. The appropriate SDF for this dam is the $\frac{1}{2}$ PMF. The spillway will pass 20 percent of the PMF (40% of the SDF) without overtopping, and the dam will be overtopped by 2.3 ft during the SDF. Tailwater conditions will allow approximately 20% of the PMF (40% of the SDF) to be passed without creating a backwater condition higher than the dam. The spillway is judged inadequate but not seriously inadequate.

The roadway embankment immediately downstream of the dam creates a tailwater elevation during the SDF which exceeds the top elevation of the dam by 6 ft. The downstream roadway will be overtopped during the SDF by 4 ft, however, because the low point in the embankment is an area of shallow fill (5 $\frac{1}{2}$ ft), and the roadway is paved with a heavy bituminous concrete surface, a breach in the roadway would be much smaller than normally predicted. The reduced breach potential would reduce the downstream hazard significantly.

There is insufficient design data and no construction records available for this structure, therefore, an accurate check on its stability cannot be made. Since embankment slopes and crest generally meet U.S. Bureau of Reclamation requirements and because the dam has been subjected to rapid drawdown with no adverse effects, a stability check is not required. Overtopping of the dam is not considered detrimental since overtopping will likely occur as a result of tail-

water conditions created by the downstream restriction.

Maintenance of the dam is considered good.

7.2 Recommended Remedial Measures: The following remedial measures should be implemented within one year of the date of this report:

7.2.1 Evaluate the downstream roadway embankment to determine what measures are required to protect the roadway from breaching during periods of overtopping. This should be performed by a qualified Professional Engineer.

7.2.2 An emergency action plan should be implemented to warn downstream dwellings of any dangers which may be imminent.

7.3 Required Maintenance and Observation:

7.3.1 Seepage present along the downstream toe should be monitored quarterly to detect any increase in flow rates which may cause piping within the embankment.

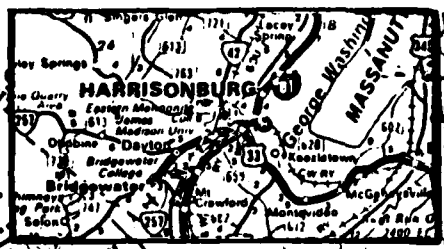
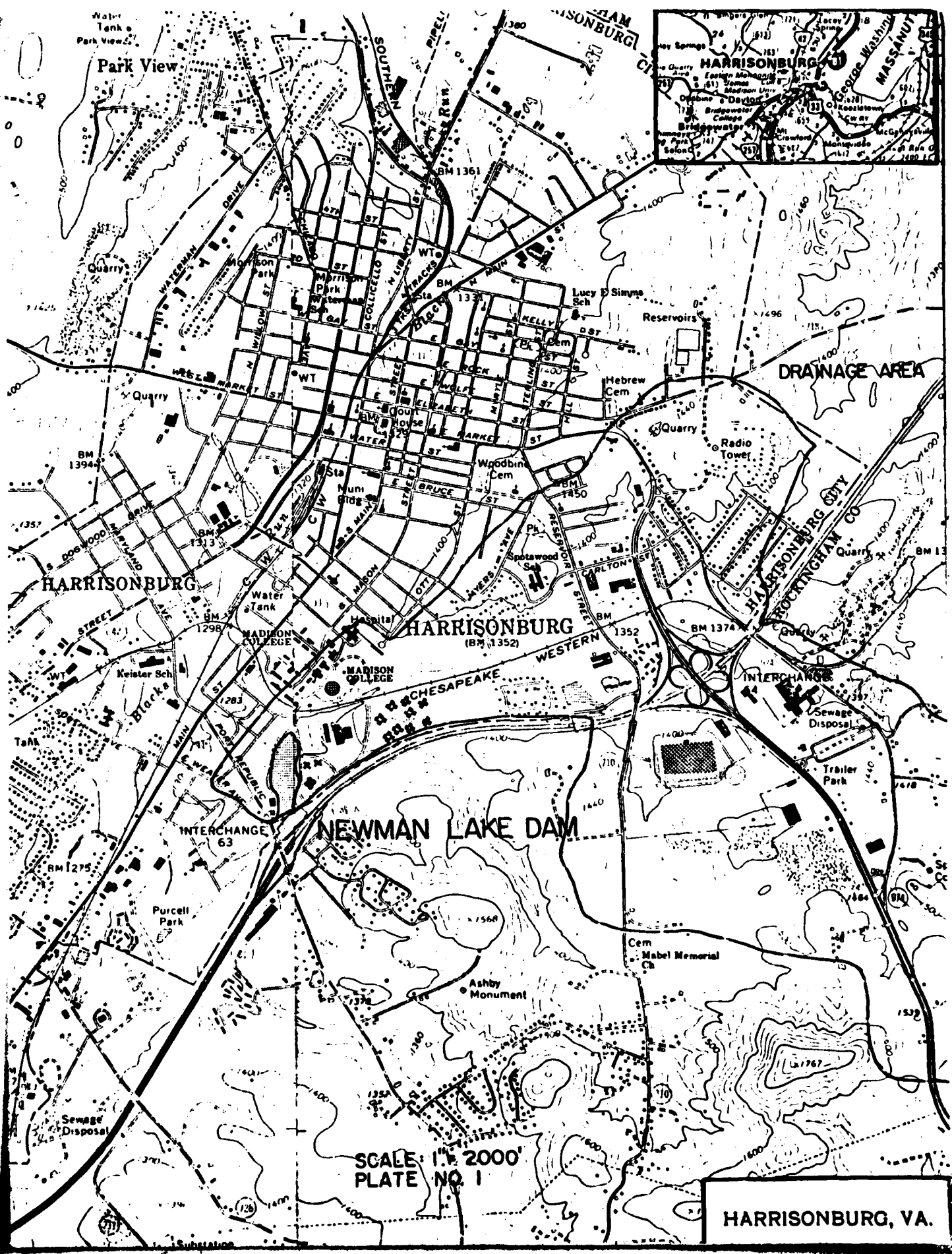
7.3.2 Existing trees on the dam should be cut to the ground. Trees greater than 3 inches in diameter should have their stumps and root structures removed and resulting holes backfilled.

7.3.3 Uncontrolled vegetation near the outlet structure should be cut and maintained in the future.

7.3.4 Muskrat burrowing in the embankment should be backfilled.

7.3.5 A staff gage should be installed to monitor water levels.

APPENDIX I
MAPS AND DRAWINGS

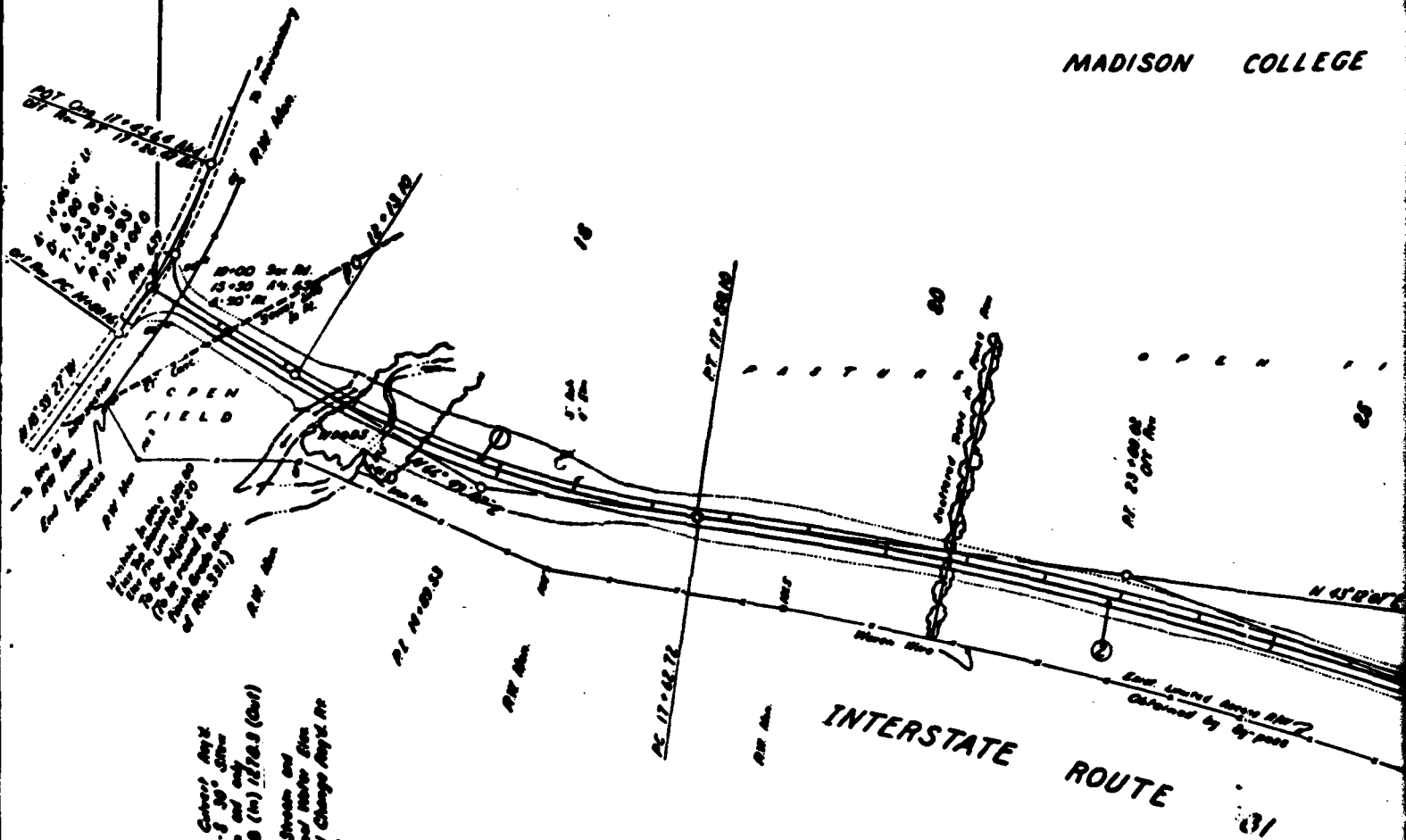


SCALE: 1" = 2000'
 PLATE NO. 1

HARRISONBURG, VA.

REVENUE PROJ. 0031-002-101, C-501
 STA. 10+00.30 Highway Survey

MADISON COLLEGE



Sta. 10+00.30
 St. 10+00.30
 Sta. 10+00.30
 Sta. 10+00.30
 Sta. 10+00.30
 Sta. 10+00.30
 Sta. 10+00.30
 Sta. 10+00.30
 Sta. 10+00.30
 Sta. 10+00.30
 Sta. 10+00.30

①
 Original Survey
 A. 25° 57' 11"
 D. 4' 20"
 T. 276.43'
 L. 505.00'
 R. 1322.21'
 Sup. 1:1
 No Widening

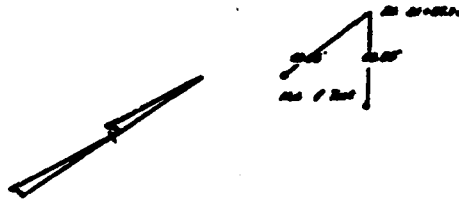
②
 Office Revision
 A. 15° 22' 55"
 D. 1' 15"
 T. 537.48'
 L. 1070.07'
 R. 4503.66'
 No Sup. Elev.
 or Widening

③
 Original Survey
 A. 33° 10'
 D. 1' 15"
 T. 1365.00'
 L. 2653.04'
 R. 4303.66'

④
 College Survey
 A. 106° 00' 00"
 D. 9' 03"
 T. 702.57'
 L. 1003.46'
 R. 500.00'
 Sup. 1:1
 No Widening

INTERSTATE ROUTE 31

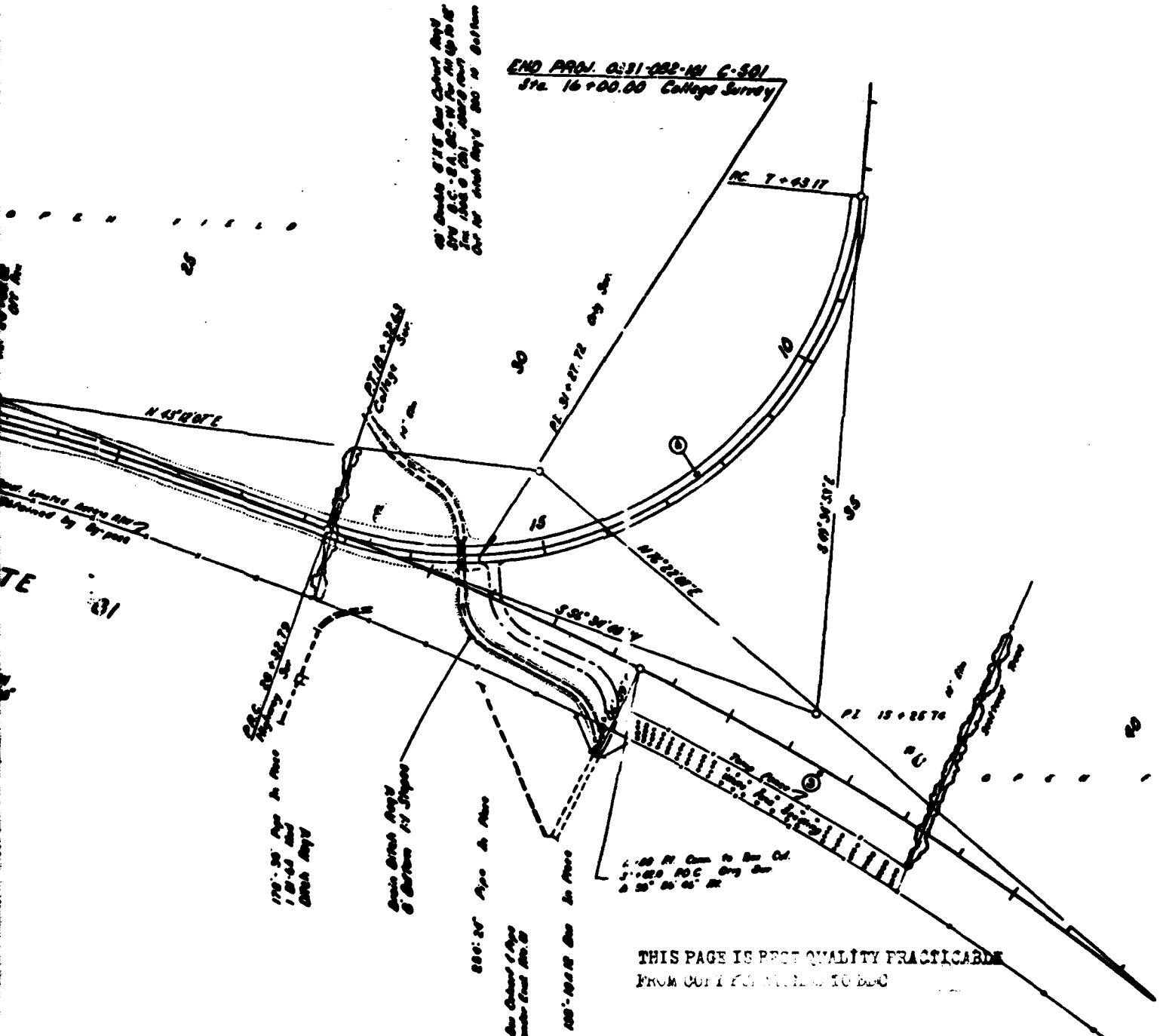
FED. ROAD DISTRICT	STATE	FEDERAL AID		STATE	
		ROUTE	PROJECT	ROUTE	PROJECT
2	VA.			331	033100 C-50



SON COLLEGE FARM

END PROJ. 0:31-002-101 C-501
Sta. 16+00.00 College Survey

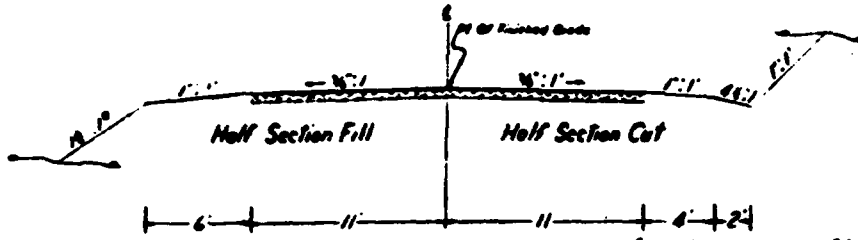
01: Double 6" x 6" Box Culvert 100' Long
Sta. 16+00.00 - 16+100.00
The 100' length of this culvert is shown
on the plan view of the 100' section of the



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SCALE PLATE NO. 2

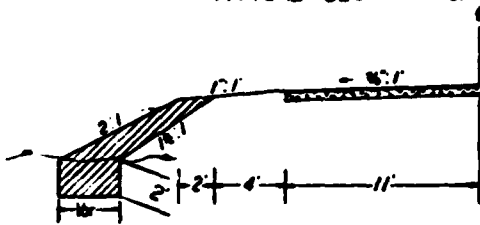
TYPICAL SECTION SERVICE ROAD
PROP 22' x 6" AGGREGATE BASE COURSE TYPE I, GRADING 'B' or 'C'
[PRIME & DOUBLE SEAL CLASS 'A' (To be applied by State Forces at a later date)]



Note: Fill slopes on the left side between sta 10+25 & sta 16+75 are to be 2:1 slopes (See Below)

The fill between sta 10+25 & sta 16+75 shall not contain more than 20% rock. Any rock excavation placed in this fill shall be evenly distributed throughout the entire fill so as not to leave any pockets containing more than 20% rock.

TYPICAL SECTION EMBANKMENT PROTECTION



Sta 10+25 To Sta. 16+75

Note: Embankment Material shall consist of a rock-free clay and compacted to 100% density. Any of the soils on this Proj. will meet the specifications if rock-free. The ideal soil for this overlay is found between sta 17+00 and 21+00 between 10' and 16" below the surface.

CODE NO. 3210

GENERAL NOTES

GRADING

The grade line denotes top of finished pavement unless shown otherwise on typical sections or plans. All slopes are to be finished in accordance with typical sections, except where otherwise noted.

DRAINAGE

The locations of all drainage structures shown on these plans are approximate only with the exception of culverts showing specific stationing.

INCIDENTALS

All necessary clearing & grubbing will be considered incidental to the other items of work and no separate or additional quantities will be allowed therefor.

LANDSCAPE

TOPSOILING - Class "A" depth. Topsoil shall be spread in accordance with Section 518 on all disturbed areas except cut slopes having a gradient of 18% or steeper, except those classified as rock and stabilized shoulders. Cut slopes having a gradient of 18% or steeper, except those classified as rock, shall be roughened before being seeded. Class "A" topsoil may be obtained between 10+00 & 15+00 on a Madison Col. highway.

SEEDING - All disturbed areas except stabilized shoulders and cuts or fills classified as "rock" shall be limed, fertilized and seeded in accordance with Section 520 of the latest Road and Bridge specifications at the rates per acre specified below.

- 2 Tons Agricultural Ground Limestone
- 1500 lbs 10-20-10 or 3000 lbs 5-10-5 Fertilizer (or equivalent)
- 2 Tons Mulch (threshed, dry straw) or other approved material at rates agreed on by the Engineer
- Grass Seed Mixture

Rate	60 lbs	Per Acre (Spring or Fall)
	40 lbs	Kentucky Blue Grass
	15 lbs	Creeping Red Fescue
	2 lbs	Red Top Grass
	3 lbs	White Dutch Clover

Seed and fertilizer must meet Va Dept of Highways specifications as shown by approved lot number or Va. Dept of Agriculture test reports. No seed shall be mixed until satisfactory test reports are received. All fertilizers used on this project shall be accompanied by the manufacturer and no additional materials added prior to its use. Whenever prior to acceptance of the project, the grass and weeds reach a height in excess of 36 inches, the contractor shall mow same.

LOCATION	Sta
SER. ROAD	10+25
DE. DITCH	
TOTALS	

△ Sta taken

LOCATION	Sta
SER. ROAD	10+25
TOTALS	

△ Sta taken

LOCATION	Sta
SER. ROAD	10+25
TOTALS	

△ Sta taken

△ 1-24' dia

Sheet No.	3
TOTALS	

GRADING SUMMARY							
LOCATION	STATION TO STATION		CUT	FILL	BOBBER		REMARKS
			Cu Yds.	Cu Yds.	Cu Yds.	Cu Yds.	
SER. ROAD	10+08.3	28+00	8,754				
	10+17	17+00		13,008			
	25+00	16+00 ^a	260	2,161			
DR. DITCH							
TOTALS			9,014	15,164	6,150		

^aSta. taken from Madison College Survey

PAVEMENT SUMMARY												
LOCATION	STATION TO STATION	EQUALITIES	LENGTH	AGGREGATE BASE COURSE TYPE I, GRAD B & C		PRIME AND DOUBLE SEAL TREATMENT CLASS "A"						REMARKS
				22 Ft. X 6 In.	Flores 50'R 16" depth	PRIME		INITIAL SEAL		FINAL SEAL		
						Bituminous Material No. 68 RT-10 @ 55 gal. per Sq. Yd.	Cover Material No. 68 RT-10 @ 25 lbs. per Sq. Yd.	Bituminous Material No. 68 RT-10 @ 0.25 gal. per Sq. Yd.	Cover Material No. 68 RT-10 @ 25 lbs. per Sq. Yd.	Bituminous Material No. 78 RT-10 @ 0.25 gal. per Sq. Yd.	Cover Material No. 78 RT-10 @ 25 lbs. per Sq. Yd.	
Lin. Ft.	Cu. Yds.	Cu. Yds.	Gals.	Tons	Gals.	Tons	Gals.	Tons				
SER. ROAD	10+08.3-16+00 ^a		2057.12	838.09	19.91	1837.71	64	1286.94	64	1286.94	64	
TOTALS				838.09	19.91	1837.71	64	1286.94	64	1286.94	64	

^aSta. taken from Madison College Survey

Note: Prime & Double Seal Treatment Class "A" Not in Contract, to be applied by State Forces at later date.

DRAINAGE SUMMARY													
LOCATION	STATION	BOX CULVERT REQD.					SPECIAL DESIGN SPREADERS					Excavation for Minor Structures	REMARKS
		LENGTH	SIZE	Class A Conc.	Reinf. Steel	Drainage Structures	Crusher Run Aggr. 1/2" Down Backfiller	Class A Conc.	Reinf. Steel	Struct. Steel	Slide Gate		
SER. ROAD	12+75	54	21.00	178	19610	90	86	18,309	5966	1	50-	574. BC-2A, BC-W, BC-S	
	16+25 ^a	40	21.50	44.7	5768						25-	574. BC-2A, BC-W	
TOTALS				222.7	25,378	90	86	18,309	5966	1	75		

^aSta. taken from Madison College Survey

^a1-24" diameter light duty slide gate Armco model 20-10C or equal

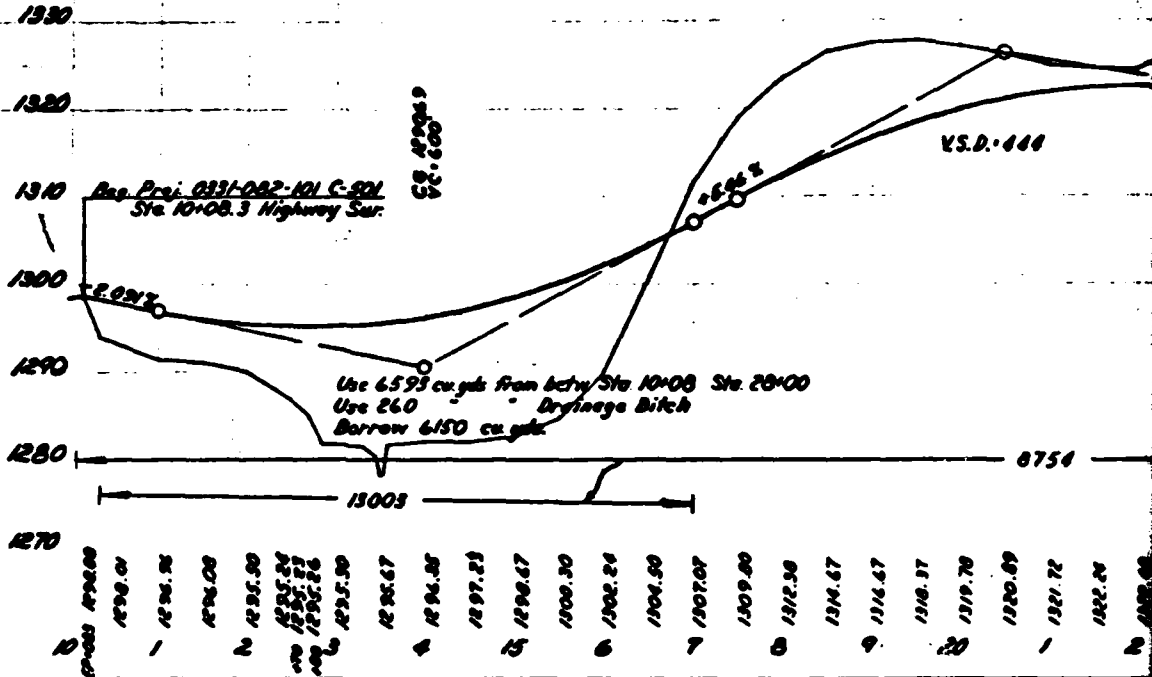
INCIDENTAL SUMMARY									
Sheet No.	Handrail Std. HR-1	Plan Conc. Slab Rip-Rap 6" Thick	Expan. Bolts 1/2"x10"	Class C Conc.	Alloying Dust	Top Soil Class A 2" depth	Seeding	Manhole to be Adjusted	Remarks
	Lin. Ft.	Sq. Yds.	No.	Cu. Yds.	Truck Hrs	Acres	Acres	Lump Sum	
3	93	28	39	10	25	2	2		
TOTALS	93	28	39	10	25	2	2	Lump Sum	

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PLATE NO. 3

12

B.M. Pt. Sta. 11426 Ht. 659
 Chisled "8" on N.E. Corner
 of Bridge
 Elev. 885.79

100 00 1000.0
 VC = 600

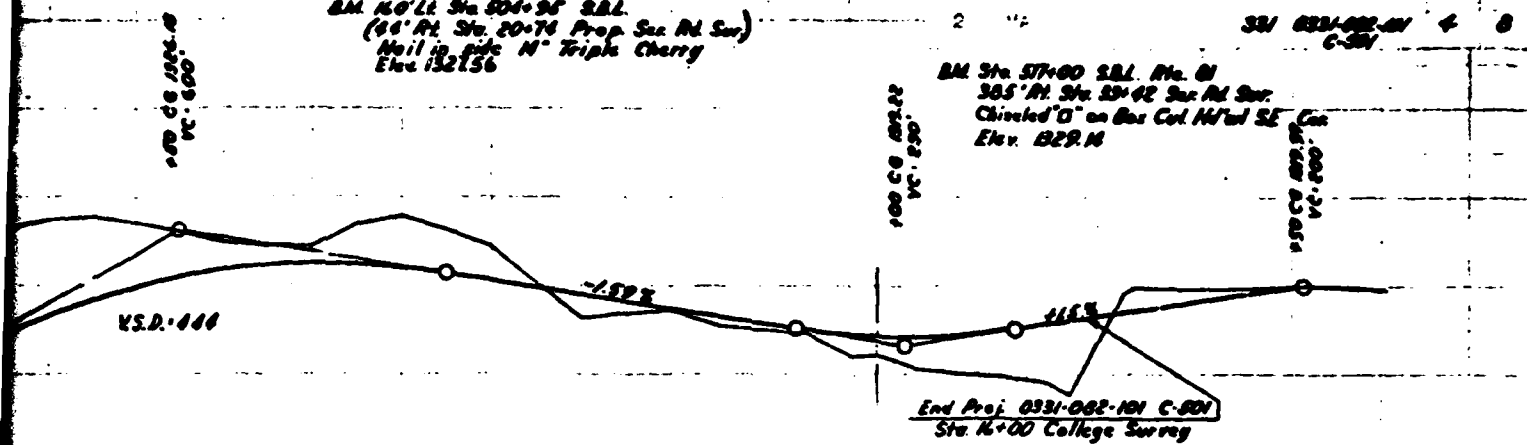


10	1298.01	1296.01	1296.26	1296.08	1295.90	1295.24	1294.27	1293.24	1292.50	1290.67	1294.85	1297.23	1298.67	1300.30	1302.24	1304.30	1307.07	1309.80	1312.58	1314.67	1316.67	1318.37	1319.78	1320.89	1321.72	1322.24	1323.00
----	---------	---------	---------	---------	---------	---------	---------	---------	---------	---------	---------	---------	---------	---------	---------	---------	---------	---------	---------	---------	---------	---------	---------	---------	---------	---------	---------

BM 160'11 Sta 504+95 S.B.L.
 (96' Pl. Sta. 20+76 Prop. Sec. Rd. Surv.)
 Nail in side N^w Triple Cherry
 Elev. 1321.56

331 0331-002-011 4 8
 C-301

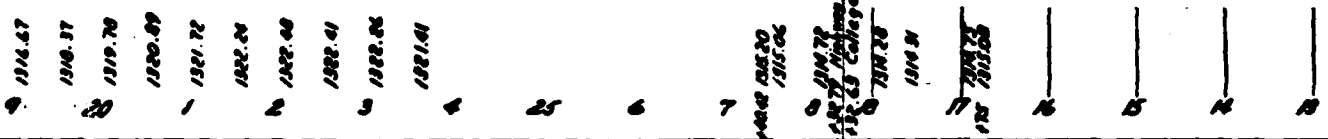
BM Sta 577+00 S.B.L. Pl. 01
 385' Pl. Sta. 32+42 Sec. Rd. Surv.
 Chiseled "C" on Box Col. NW of SE Cor.
 Elev. 1329.18



Use 2161 cu. yds from betw
 Sta. 10+08 Sta. 28+00

0754

2161



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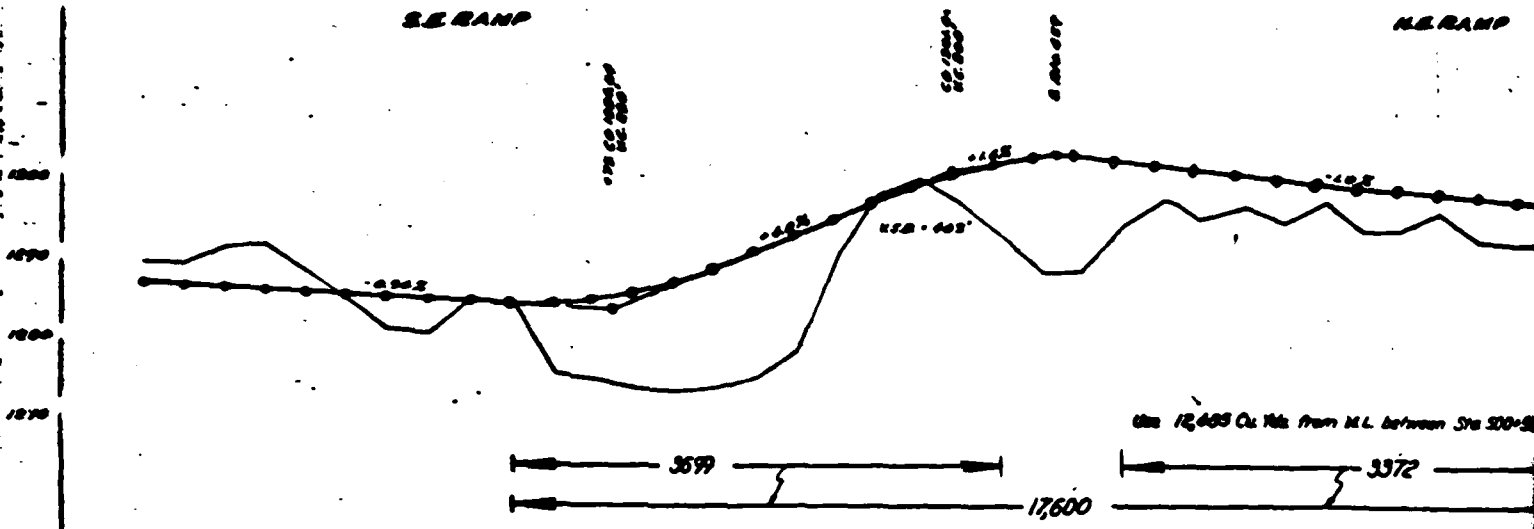
PLATE NO. 41

12

6 1100

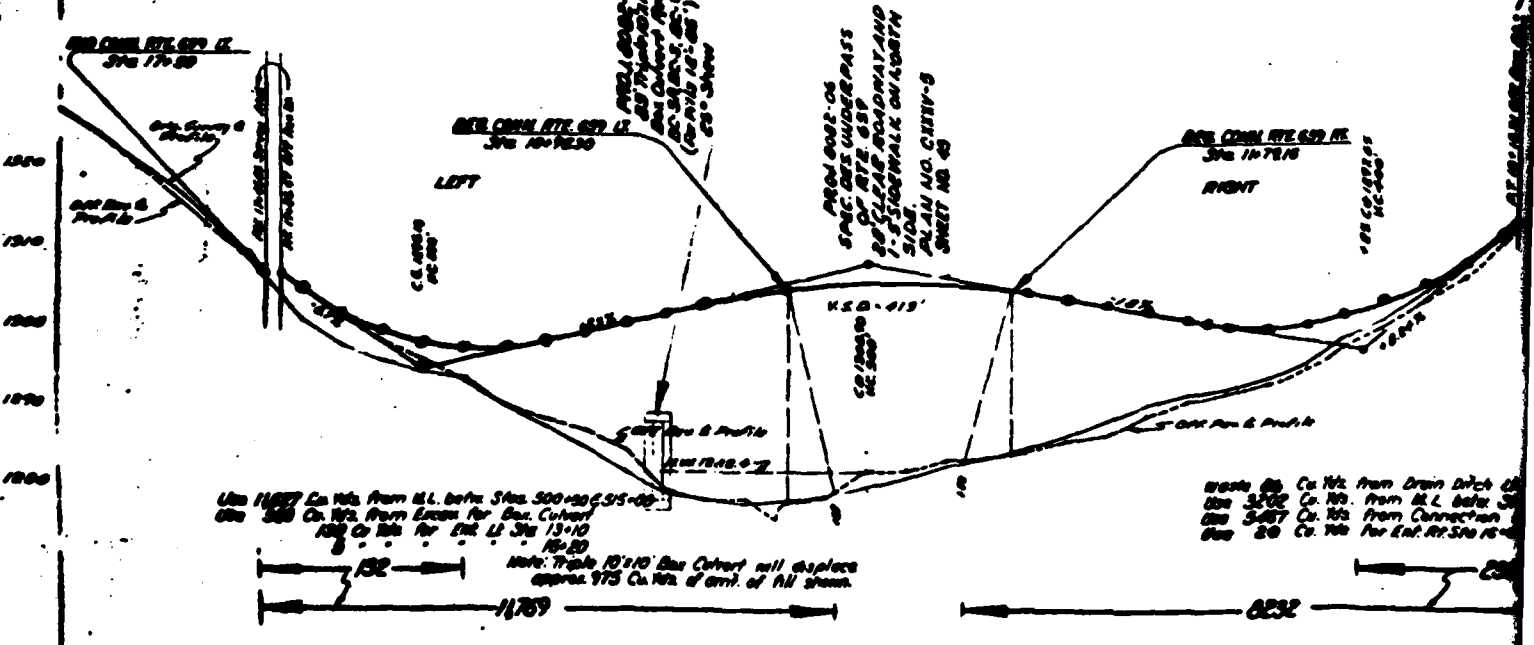
S.E. RAMP

N.E. RAMP



497	498	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44	45	46	47	48	49	50	51	52	53	54	55	56	57	58	59	60	61	62	63	64	65	66	67	68	69	70	71	72	73	74	75	76	77	78	79	80	81	82	83	84	85	86	87	88	89	90	91	92	93	94	95	96	97	98	99	100
-----	-----	---	---	---	---	---	---	---	---	---	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	-----

CONN. RTE 659 LT & RT.
STA. 491+95.26 M.L.



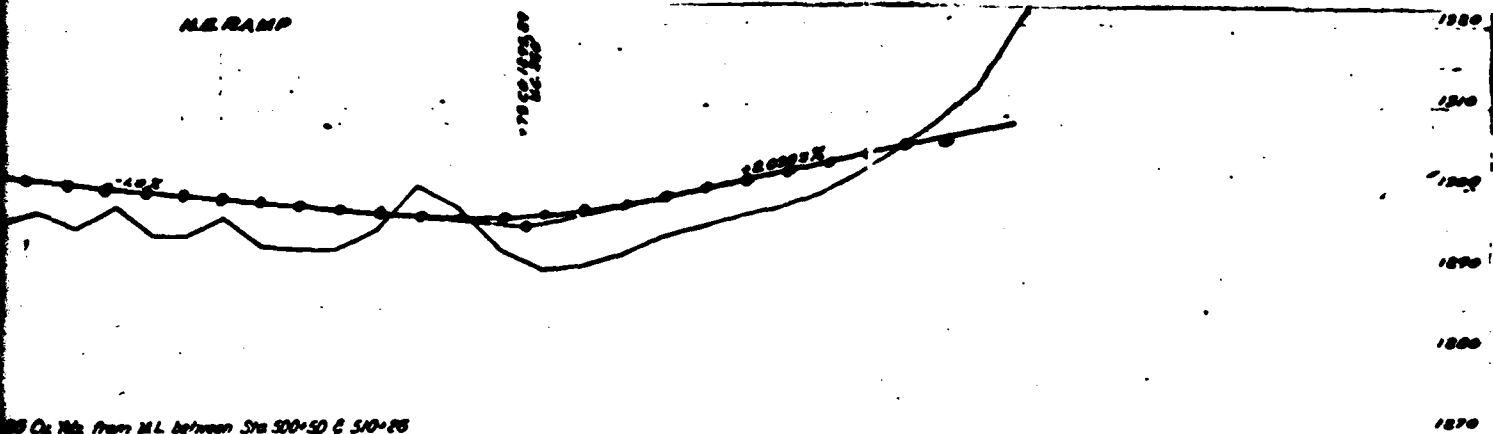
Use 11627 Cu Yds. from M.L. betw. Sta. 500+00 & 515+00
 Use 500 Cu Yds. from Excav. for Box Culvert
 Use 100 Cu Yds. for Ent. Lt. Sta. 13+10
 10+00

Note: Triple 10'x10' Box Culvert will displace approx. 975 Cu Yds. of corr. of All shown.

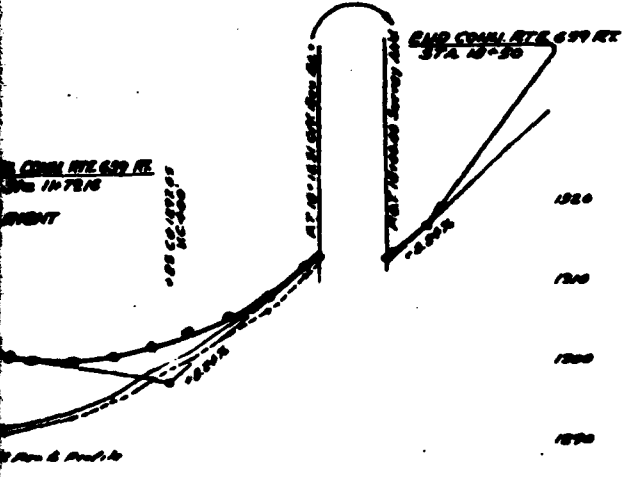
Grate @ Cu Yds. from Drain Ditch @ Sta. 5200 Cu Yds. from M.L. betw. Sta. 5+07 Cu Yds. from Connection @ Sta. 20 Cu Yds. for Ent. Rt. Sta. 15+00

N.E. RAMP

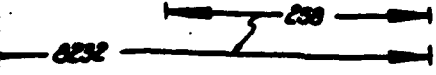
AS SHOWN ON PLAN



See Cut No. from M.L. between Sta 300+50 & 310+25



Sta 10+00 Co. 100 from Drain Ditch LL Sta 10+05 & 10+20
 Sta 10+25 Co. 100 from M.L. below Sta 300+50 & 310+00
 Sta 10+50 Co. 100 from Connection with Rte 933
 Sta 10+75 Co. 100 for End of Sta 10+00

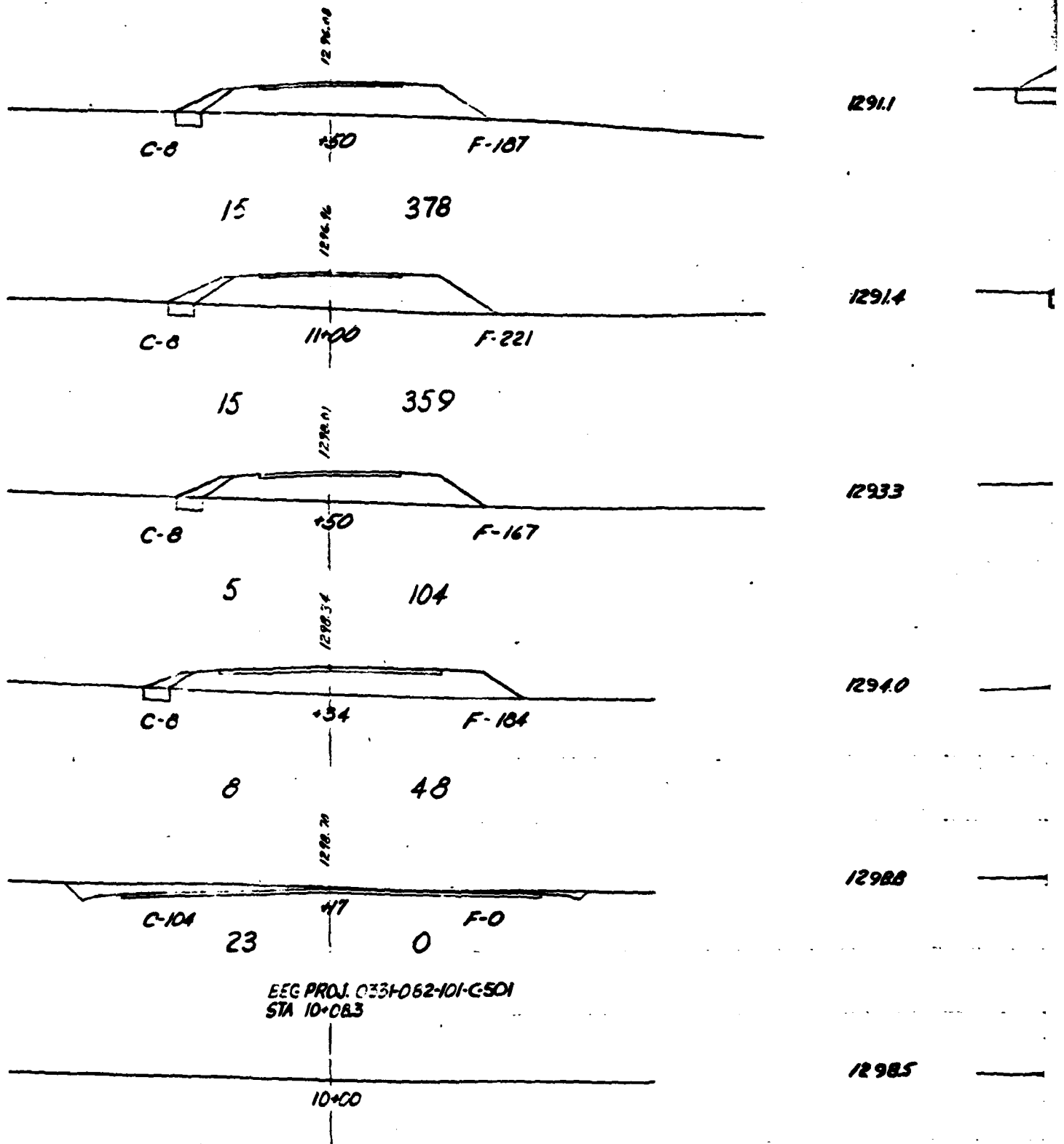


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PLATE NO. 5 27

12

CROSS SECTIONS

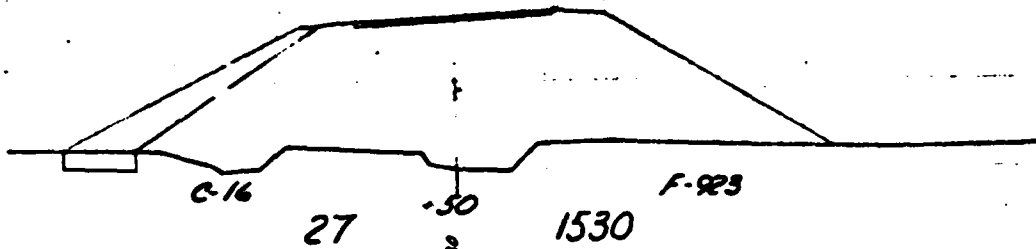


EEG PROJ. 033+062-101-C501
STA 10+08.3

CROSS SECTIONS

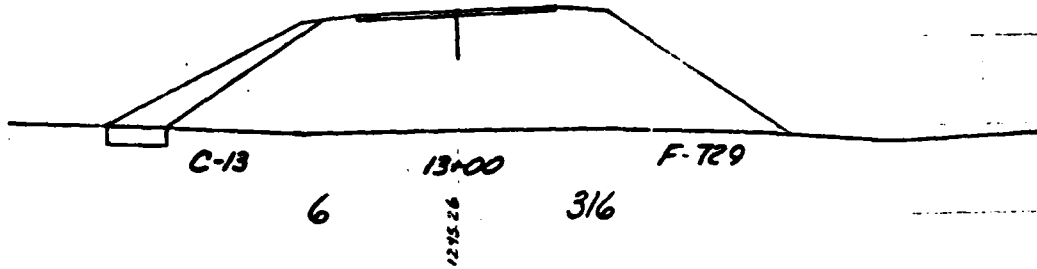
STATE
2 VA. 1 6

12911



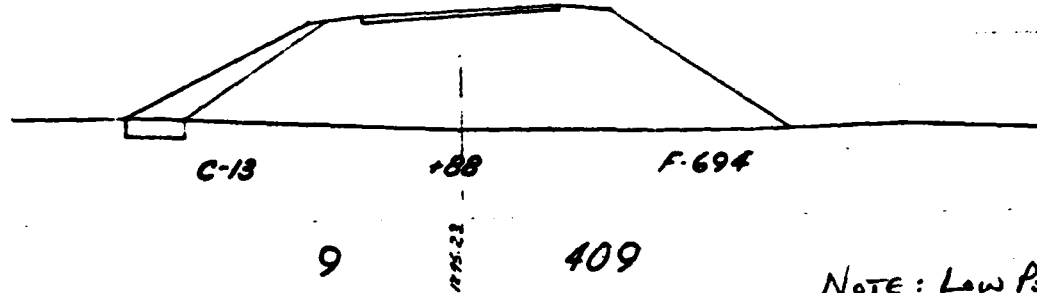
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12914



12819

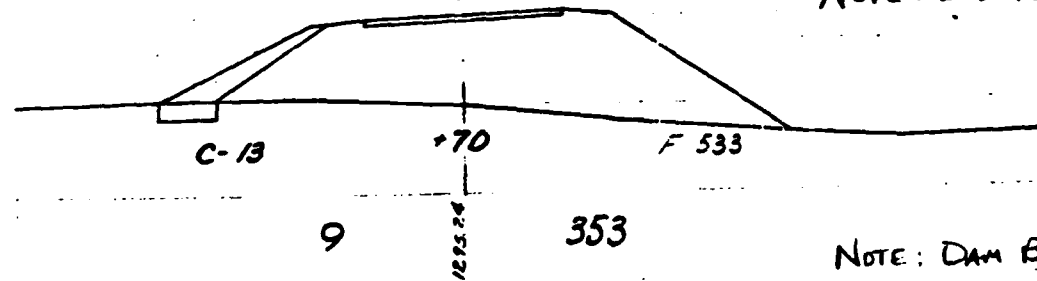
12933



12829

NOTE: LOW POINT

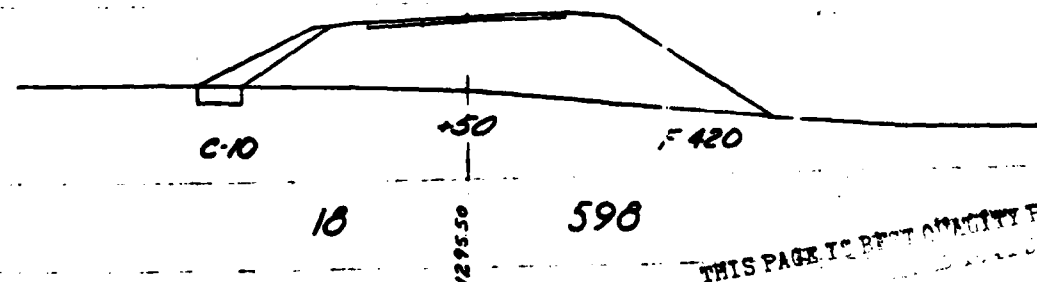
12940



12852

NOTE: DAM BEGINS

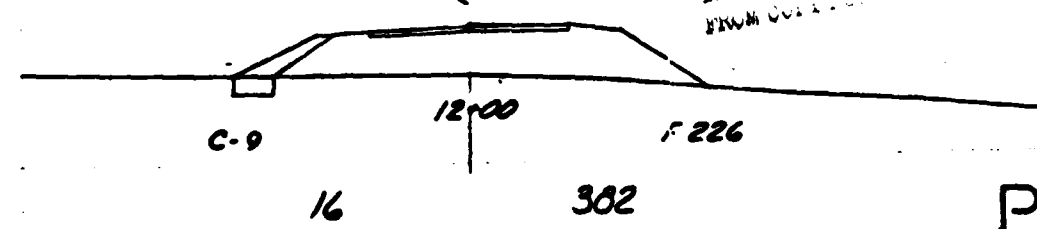
12988



12870

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12985



12901

PLATE NO. 6

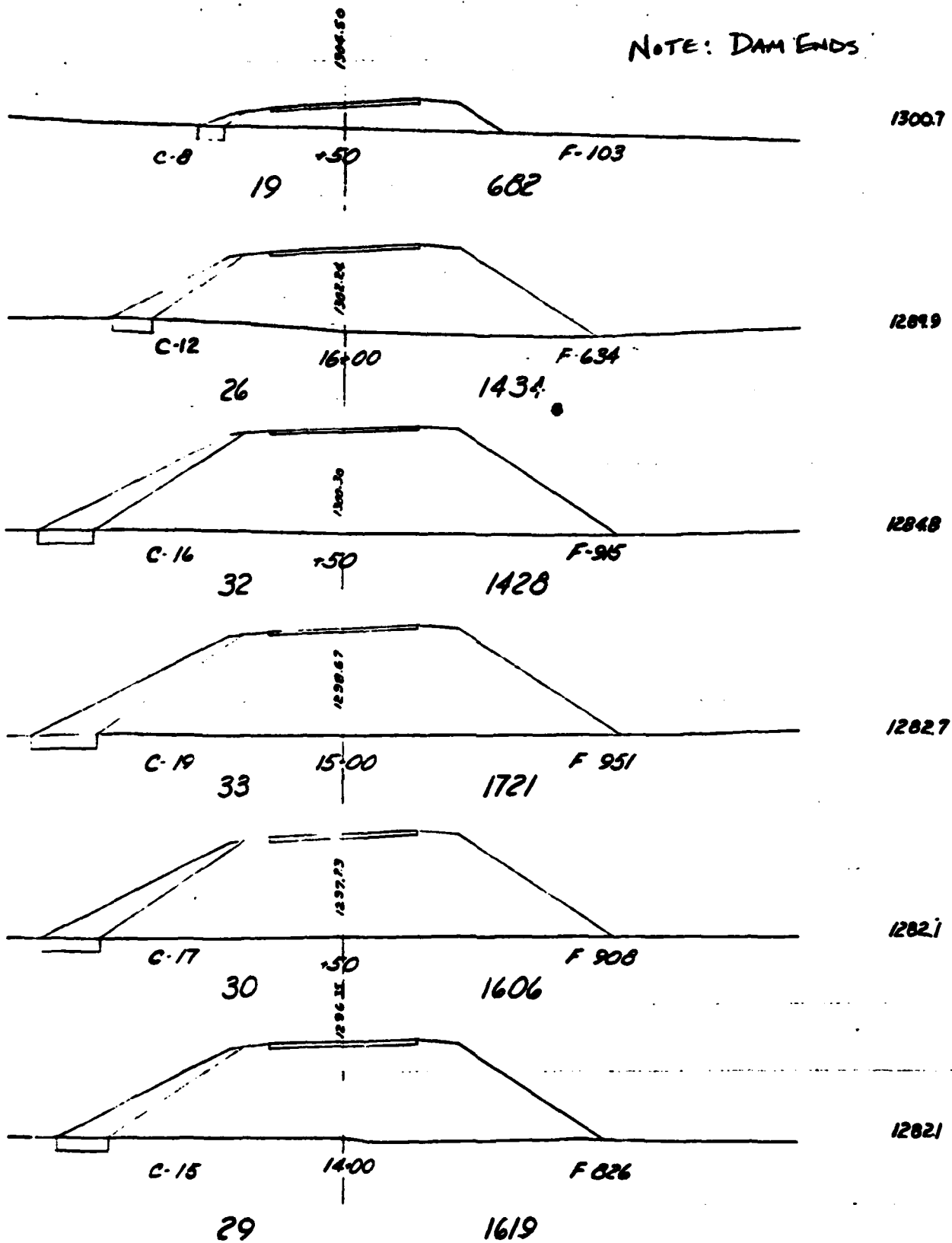
A 6855

12

CROSS SECTIONS

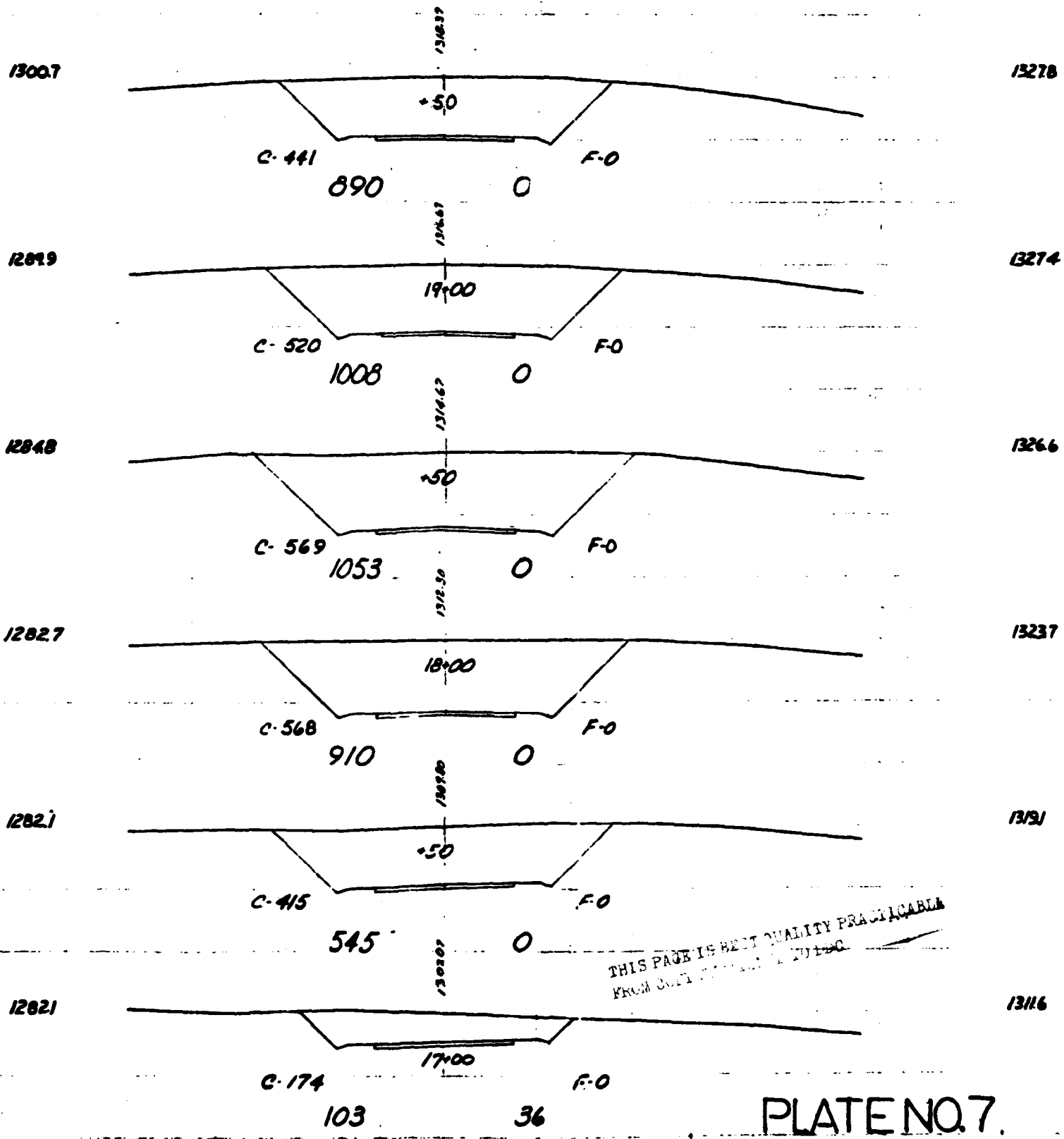
Sta. 100+00 to 100+00

NOTE: DAM ENDS



CROSS SECTIONS

2 VA. 334 6831-002-101 2 4



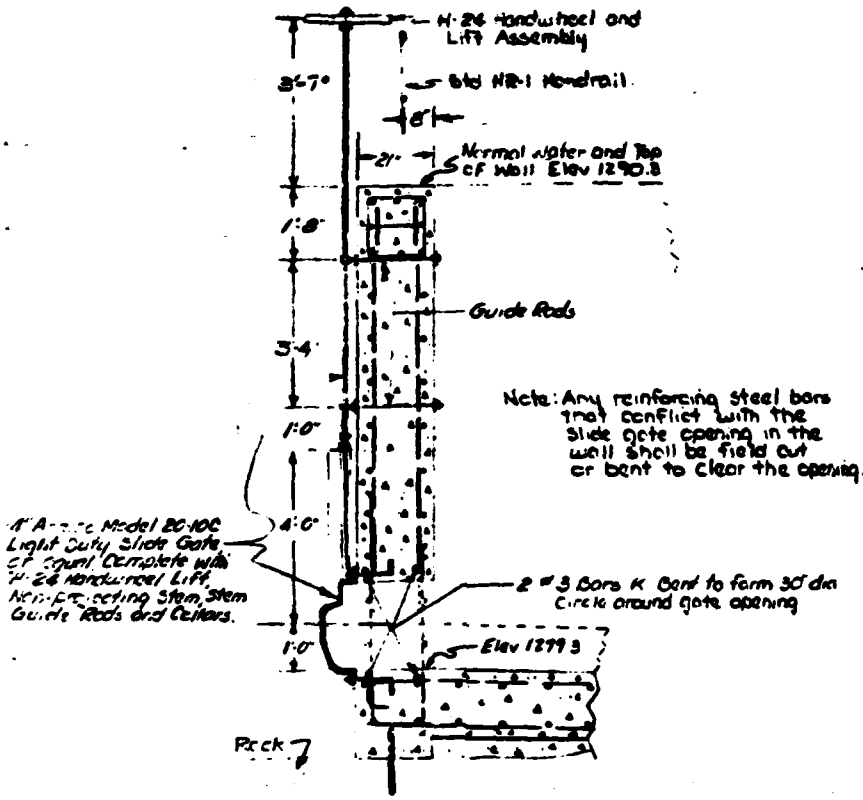
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PLATE NO. 7.

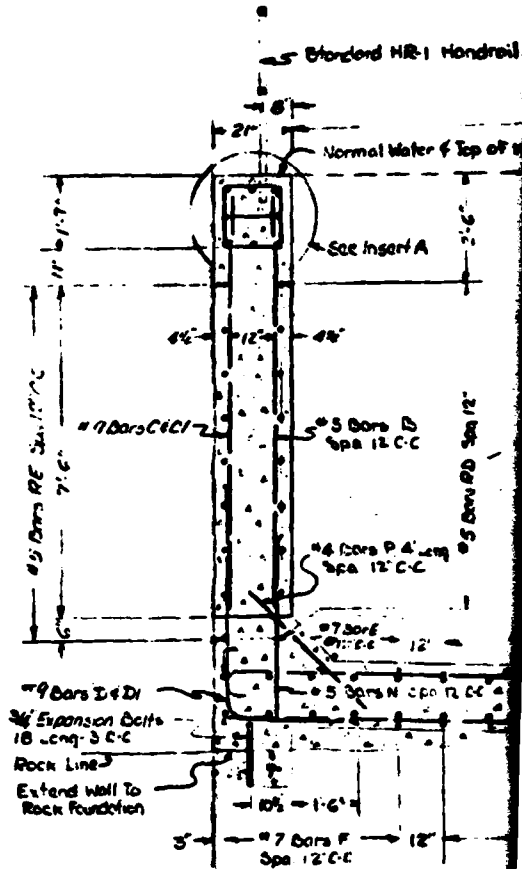
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2

SPECIAL DESIGN SPILLWAY ATTACHED TO INLET END OF STANDARD DOUBLE 10x12 BOX CULVERT



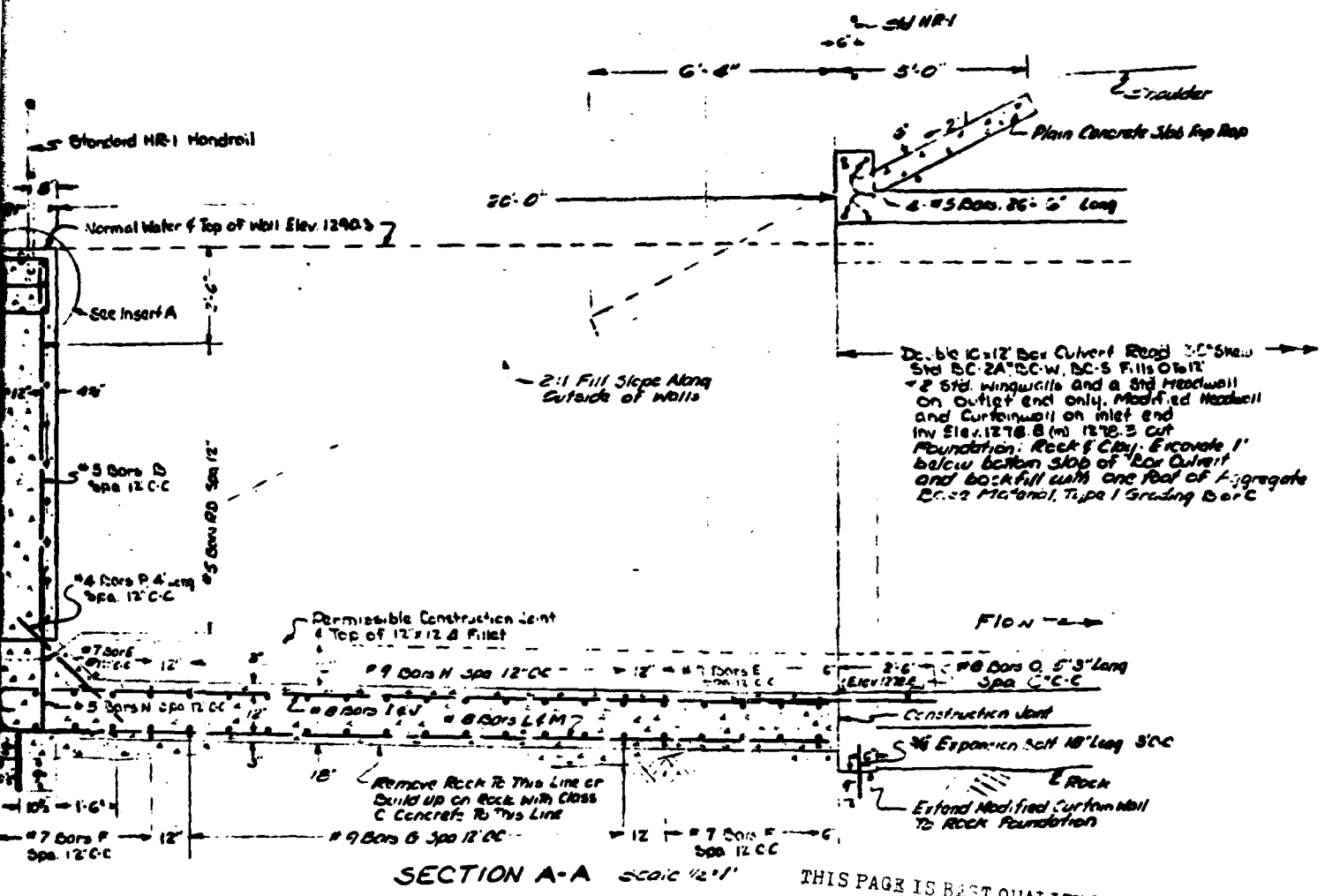
SECTION D-D scale 1/8"=1'
(For details not shown here-on)
See Section A-A



Unit price of equipment for the 24 diameter Ancon Model 20-10C Light Duty Slide Gate or equal complete shall be the unit price bid which price shall include the slide gate, the entire lift assembly and all labor, tools, equipment and materials necessary to complete the installation as planned.

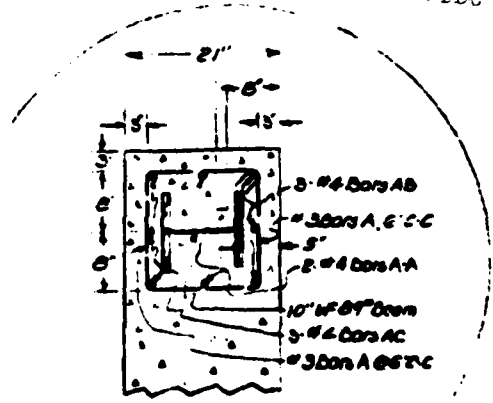
WED TO
12' BOX CULVERT

NO.	DATE	BY	CHKD.	APP'D.	SCALE	NO.	REV.
2	VA.			SSJ	001-000-001 C 201	26	6



SECTION A-A scale 1/2"=1'

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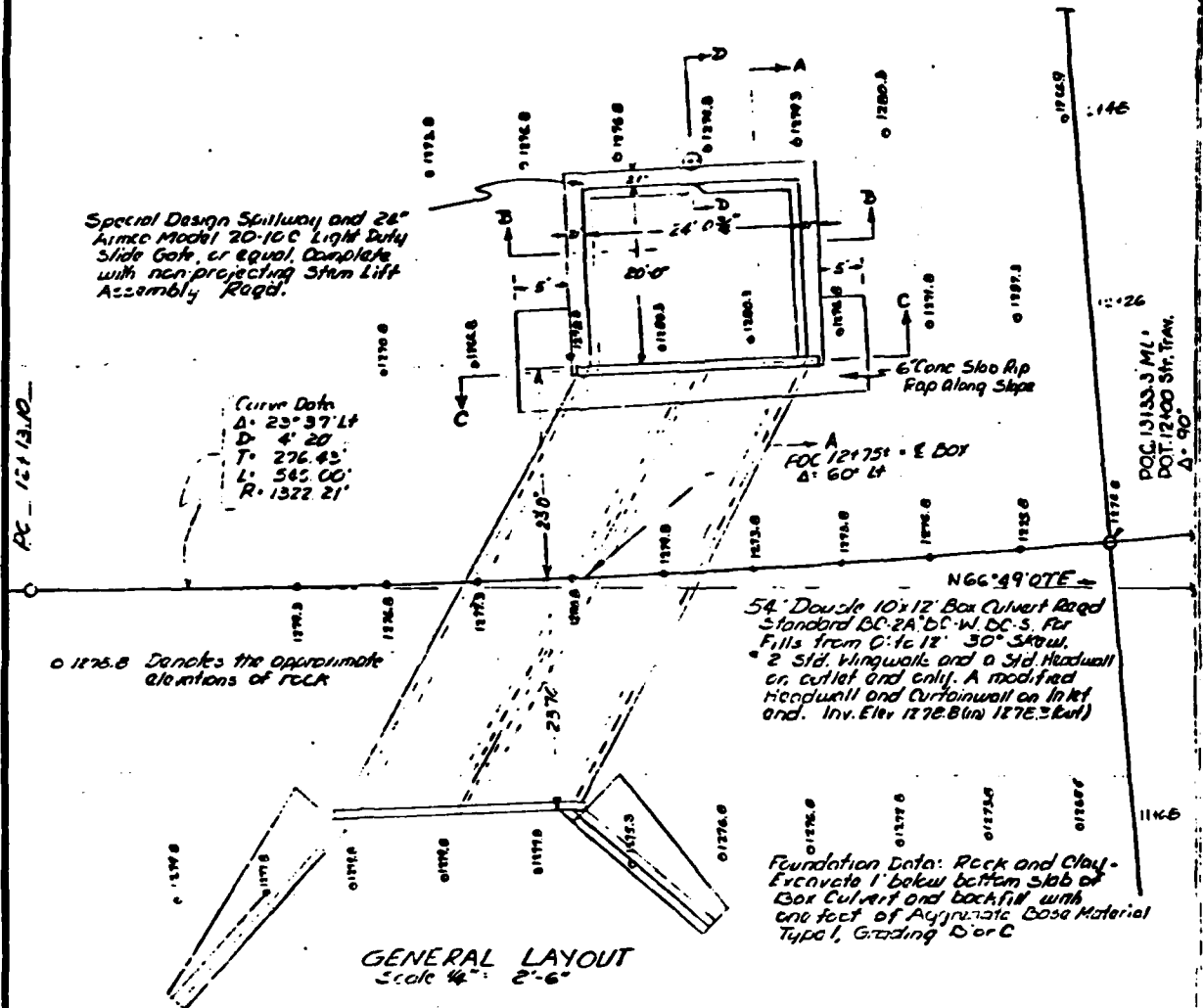


Insert A
scale 1"=1'

PLATE NO. 8

2

DETAILS OF SPECIAL DESIGN SPILLWAY ATTACHED TO
INLET END OF STANDARD DOUBLE 10x12' BOX CULVERT



SUMMARY OF APPROXIMATE QUANTITIES

DOUBLE 10x12 BOX CULVERT

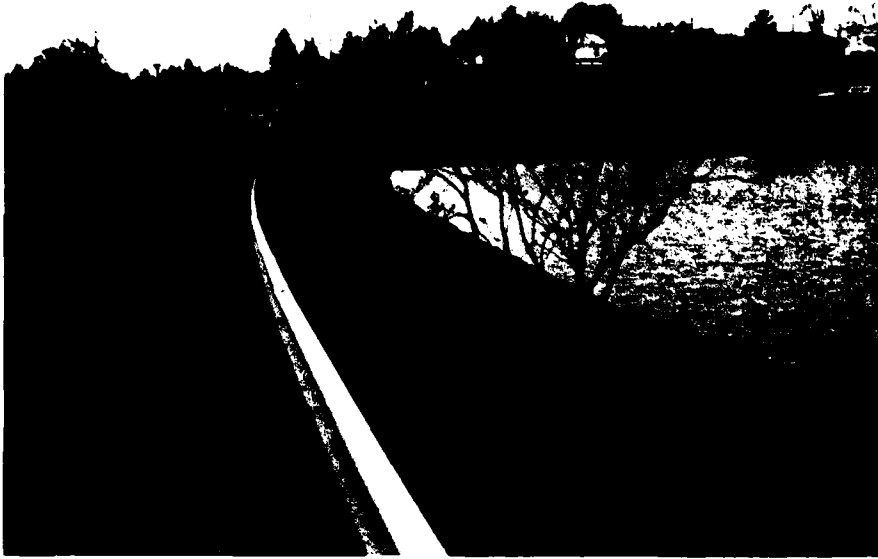
- 178 Cu Yds. Class A Concrete
- 19610 Lbs Reinforcing Steel
- 50 Cu Yds. Excavation
- 50 Cu Yds. Aggregate Base Material Type 1, Grading B or C.

SPECIAL DESIGN SPILLWAY

- 66 Cu Yds. Class A Concrete
- 15000 Yds. Class C Concrete
- 16309 Lbs Reinforcing Steel
- 2966 Lbs Structural Steel (10" x 8" x 29" Beam)
- 29 3/8" Expansion Bolts - 18" Long
- 95 Lin Ft 3/4" HR-1 Handrail
- 28 29 Yds Plain Concrete Slab 210 Rep 6" Thick
- 1 24" Diameter Arisco Model 20-100 Light Duty Slide Gate, or equal, complete.

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APPENDIX II
PHOTOGRAPHS



Upstream Face of Dam

Photograph No. 1



Downstream Face of Dam

Photograph No. 2



Downstream Channel
(Note Interstate 81 in Background)

Photograph No. 3



Intake Structure

Photograph No. 4



Outlet Works (Double 12' X 10' box Culvert)

Photograph No. 5

APPENDIX III
FIELD OBSERVATIONS

Check List
Visual Inspection
Phase I

Lat. 38°-25.9'
Long. 78°-52.5'

Name Dam Harrisonburg Dam City Harrisonburg State Virginia Coordinators _____

Date(s) Inspection 4/15/80 Weather Cloudy Temperature 45° F

Pool Elevation at Time of Inspection 1290.1 msl Tailwater at Time of Inspection 1278.5 msl

Inspection Personnel:

Schnabel Engineering Associates, P.C. J. K. Timmons and Associates, Inc. State Water Control Board
Raymond A. DeStephen, P.E. Robert G. Roop, P.E. Hugh M. Gildea, P.E.
Stephen G. Werner (recorder) Donald Balzer (recorder)

EMBANKMENT

VISUAL EXAMINATION OF

OBSERVATIONS

REMARKS OR RECOMMENDATIONS

SURFACE CRACKS

The slopes, crest, principal spillway, and abutment contacts were inspected and no cracks were noted. A paved road occupies the crest of the dam. The embankment slopes and abutment areas are grassed and well maintained. Numerous decorative trees (several inches in diameter) have been recently planted along the upstream slope and crest. Slightly larger white pines occur along the crest and downstream. Some uncontrolled vegetation was present in the less accessible areas, particularly along the box culvert.

UNUSUAL MOVEMENT OR
CRACKING AT OR BEYOND
THE TOE

No unusual movements or cracking were noted on the dam or downstream beyond the embankment toe.

SLOUGHING OR EROSION OF POCKETS (LESS THAN ONE FT WIDE AND DEEP) WERE ENCOUNTERED ON THE UPSTREAM SLOPE. A SIX INCH
EMBANKMENT AND ABUTMENT DIAMETER VOID EXISTS UNDER THE NORTHEAST CORNER OF THE CONCRETE INTAKE STRUCTURE. THIS VOID IS
APPARENTLY THE RESULT OF MUSKRAT BURROWING. NUMEROUS MUSKRAT HOLES EXIST IN A TEN FT² WIDE AREA,
85 FT² LEFT OF THE INTAKE STRUCTURE ON THE UPSTREAM SLOPE, JUST ABOVE POOL LEVEL. A TWO FT²
VERTICAL WAVE CUT NOTCH EXISTS AT POOL LEVEL ALONG THE UPSTREAM SLOPE. THE THICK GRASS APPEARS TO
PREVENT ANY SERIOUS EROSION PROBLEM. THE EMBANKMENT SLOPES RANGE FROM 2 3/4:1 V TO 3 1/2:1 V ON BOTH THE
UPSTREAM AND DOWNSTREAM SIDES. A SMALL BERM (3 1/2 TO 6 FT WIDE) EXTENDS CONTINUOUSLY FROM THE
UPSTREAM TOE TO POOL LEVEL.

VERTICAL AND HORIZONTAL
ALIGNMENT OF THE CREST

The vertical and horizontal alignment of the dam appeared to be good.

RIPRAP FAILURES

No riprap exists.

EMBANKMENT

VISUAL EXAMINATION OF

JUNCTION OF EMBANKMENT
AND ABUTMENT, SPILLWAY
AND DAM

OBSERVATIONS

The embankment ties into the grassed abutment areas. No erosion was observed and there were no soil or bedrock exposures. The embankment appears to be constructed with silty clay soils (residual), including some fine to coarse sand with rock fragments and scattered limestone boulders. The only bedrock observed consisted of scattered limestone outcrops exposed along the right side of the downstream channel, about 300 ft below the concrete culvert. The bedrock surface was weathered and irregular. Bedrock strike was to the Northeast and dips of 75° Southeast were measured. The bedrock was highly fractured and adjacent residual soils included an abundance of rock fragments. No faults were observed during the inspection.

REMARKS OR RECOMMENDATIONS

ANY NOTICEABLE SEEPAGE

A single area of seepage approximately 60 ft long and 20 ft wide was encountered along the toe of the downstream slope at a point 125 ft± left of the spillway centerline. Although no flow or turbidity was observed, the area did include iron staining. Rainfalls from the previous night resulted in the presence of scattered wet areas. Only areas exhibiting iron staining were positively identified as seepage. It is likely that the wet marshy area near the box culvert is also the result of seepage through the dam. A field sketch is provided as Sheet 1.

STAFF GAGE AND RECORDER

None observed

DRAINS

None observed

UNGATED SPILLWAY

REMARKS AND RECOMMENDATIONS

OBSERVATIONS

VISUAL EXAMINATION OF

CONCRETE WEIR

None

APPROACH CHANNEL

None

DISCHARGE CHANNEL

None

BRIDGE AND PIERS

None

OUTLET WORKS

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CRACKING AND SPALLING OF CONCRETE SURFACES IN OUTLET CONDUIT	No cracking or spalling	Good condition; 10 to 15 years old
INTAKE STRUCTURE	20 ft x 24 ft x 10 ft deep rectangular weir. No signs of deterioration. Overflow on three sides.	Good condition
OUTLET STRUCTURE	45 ft concrete box culvert, 10 ft x 20 ft double Location adjacent to right abutment	Good condition
OUTLET CHANNEL	No erosion, riprap in place	Good condition
EMERGENCY DRAINS	24 inch drain gate. Reported to be operational III-5	Good condition

RESERVOIR

VISUAL EXAMINATION OF

OBSERVATIONS

REMARKS OR RECOMMENDATIONS

SLOPES Gentle (10:1) grassed slopes bound the reservoir.
No erosion was noted

Used for recreation and irrigation. Well maintained by the university.

SEDIMENTATION

No sedimentation observed

DOWNSTREAM CHANNEL

REMARKS OR RECOMMENDATIONS

OBSERVATIONS

VISUAL EXAMINATION OF

CONDITION
(OBSTRUCTIONS,
DEBRIS, ETC.)

Good condition, vegetated
N = 0.05
15 ft wide bottom with 3:1 side slopes
No erosion
A triple box culvert through a roadway embankment occurs
200 ft+ downstream of the outlet

SLOPES

3:1 slopes

Good condition, no erosion

APPROXIMATE NO.
OF HOMES AND
POPULATION

- 1) 5 homes downstream on Butler Street
- 2) Park, no hazards
- 3) Farm building downstream, 5 ft above stream
- 4) Industrial buildings, 15 ft above stream

Possible flooding could occur
from dam breach.

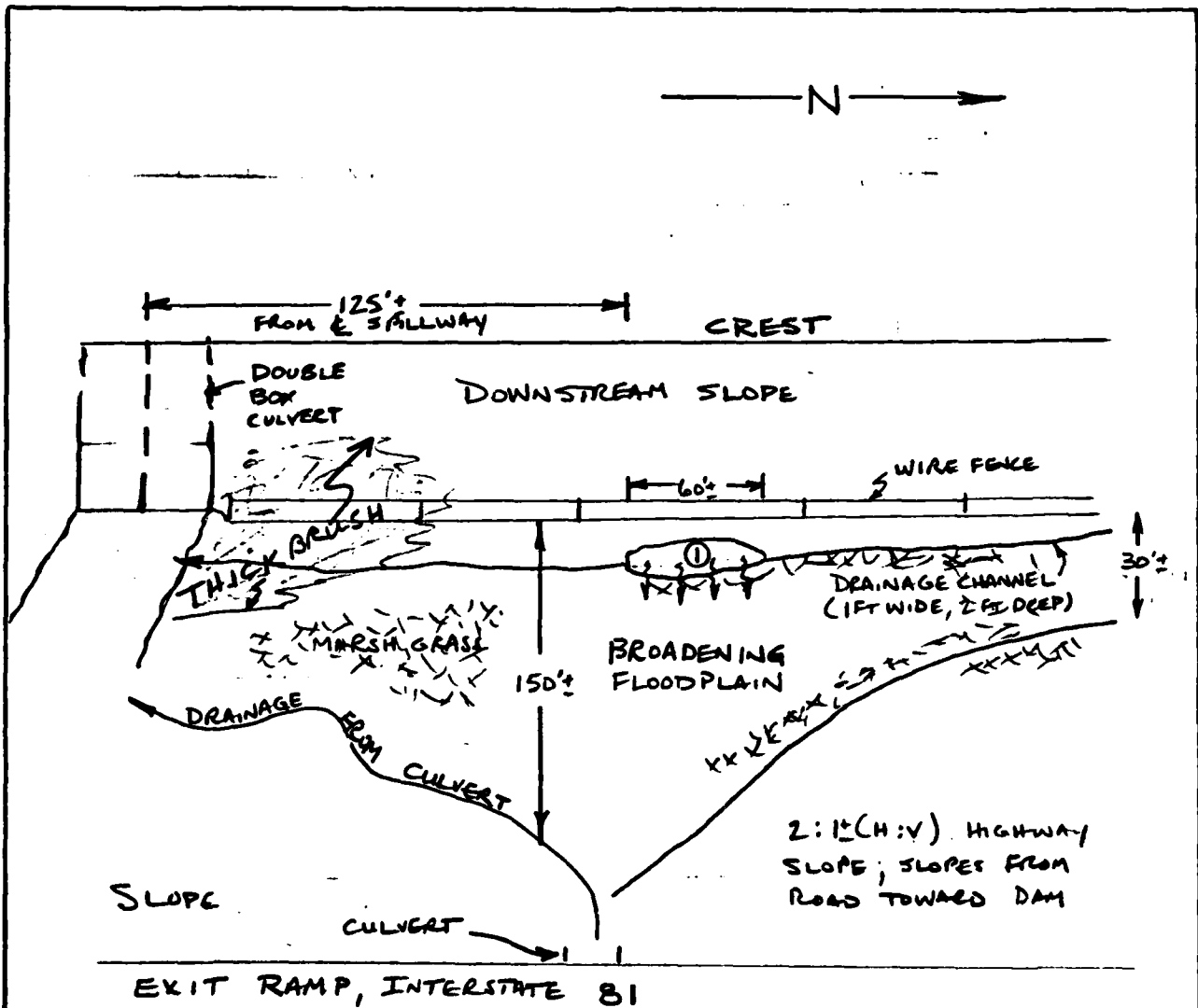
INSTRUMENTATION

VISUAL EXAMINATION	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
MONUMENTATION/SURVEYS	None	-
OBSERVATION WELLS	None	-
WELLS	None	-
PIEZOMETERS	None	-
OTHER	III-8	

BY SW DATE 1/15/84
CHKD. BY DATE
SUBJECT FIELD SKETCH - HARRISONBURG DAM

SCHNABEL ENGINEERING ASSOCIATES
CONSULTING ENGINEERS

SHEET NO. 1 OF 2
JOB NO. V.80106

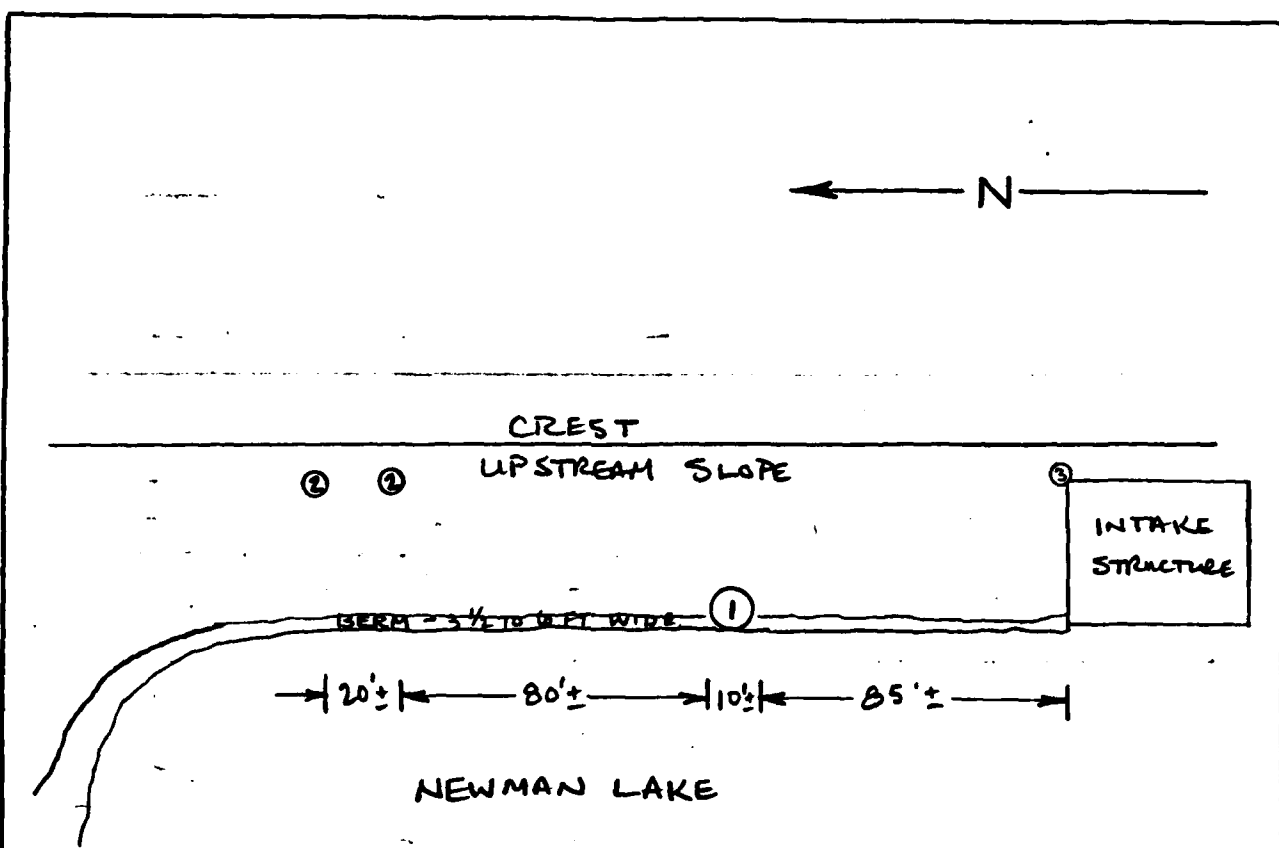


NO SCALE

- ① SEEPAGE AREA APPROXIMATELY 20 FT WIDE. NO FLOW OR TURBIDITY OBSERVED, BUT DOES INCLUDE IRON STAINING.

BY SW DATE 4/15/80 SCHNABEL ENGINEERING ASSOCIATES
CONSULTING ENGINEERS
CHKD. BY DATE
SUBJECT FIELD SKETCH - HARRISONBURG DAM

SHEET NO. 2 OF 2
JOB NO. V90106



NO SCALE

- ① 10 FT ± WIDE AREA, NUMEROUS MUSKRAT (?) HOLES
- ② MINOR SURFACE EROSION; 6" DEEP, 1' WIDE, 2' LONG
- ③ MUSKRAT BURROW

APPENDIX IV
SEEPAGE EVALUATION
BY
USDA SCS

UNITED STATES DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

Box 671
Harrisonburg, Virginia
22801

February 20, 1969

Mr. Gene Wagner
Superintendent of Buildings and Grounds
Madison College
Harrisonburg, Virginia 22801

Dear Mr. Wagner:

Attached is a report of a study of seepage problems at the dam near the Port Republic Road entrance, made at your request.

The report outlines the probable cause and possible dangers of the seepage and recommends that the problem be handled by installation of an earth fill cutoff berm on the upstream face of the dam.

Sincerely yours,

Wm. L. Blair, Jr.
Wm. L. Blair, Jr.,
Area Conservationist

SEEPAGE PROBLEM

at

MADISON COLLEGE DAM

Soil Conservation Service
U. S. Department of Agriculture

SEEPAGE PROBLEM
Madison College Dam

Situation and Procedures Used:

The Soil Conservation Service was requested by Madison College to undertake a study of seepage problems at the dam near the Port Republic Road entrance and make recommendations for their solution. The request was routed through the SCS District Conservationist for Rockingham County to the Area Engineering Staff and the Area Soil Scientist who jointly investigated the situation and arrived at the conclusions and recommendations outlined below. A preliminary topographic survey was made of the area below the dam, and soil borings were made to determine seepage patterns.

Conclusions:

1. There is considerable seepage coming through the portion of the dam north of the concrete inlet, creating a swampy condition downstream from the dam. This seepage also causes some instability of the dam which could result in failure of the structure.
2. Preliminary investigation indicates that the dam rests on a thin layer of permeable topsoil over an almost impermeable subsoil, as shown in Figure 1. Water is moving through the permeable layer and emerging near the downstream toe.
3. If the permeable topsoil had been stripped off and the structure keyed into the impermeable layer as shown in Figure 2, then with the good fill material available at this site and the relatively low water levels involved, probably no appreciable seepage would have occurred.
4. There are two feasible methods of solving the problem apparent at this time. One is with an upstream cutoff, and the other is with downstream drainage. These two alternatives are discussed as follows:

WATER LINE

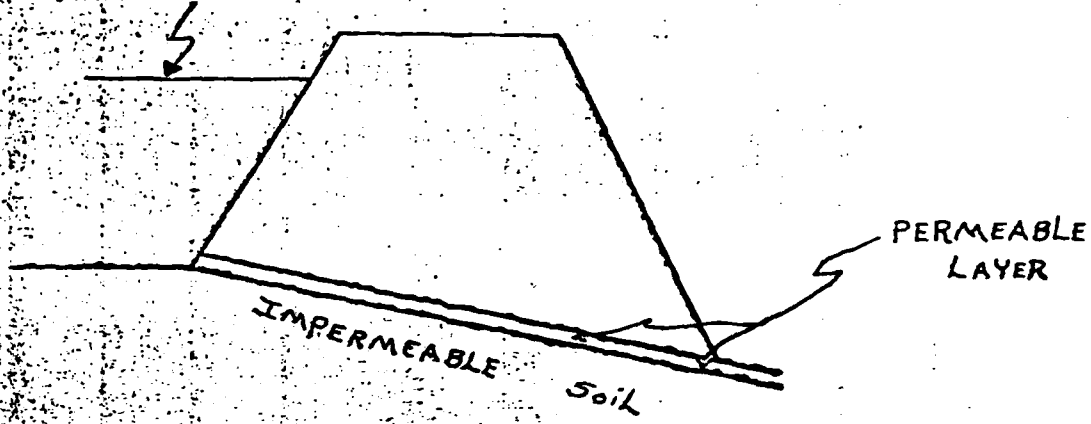


FIG. 1

WATER LINE

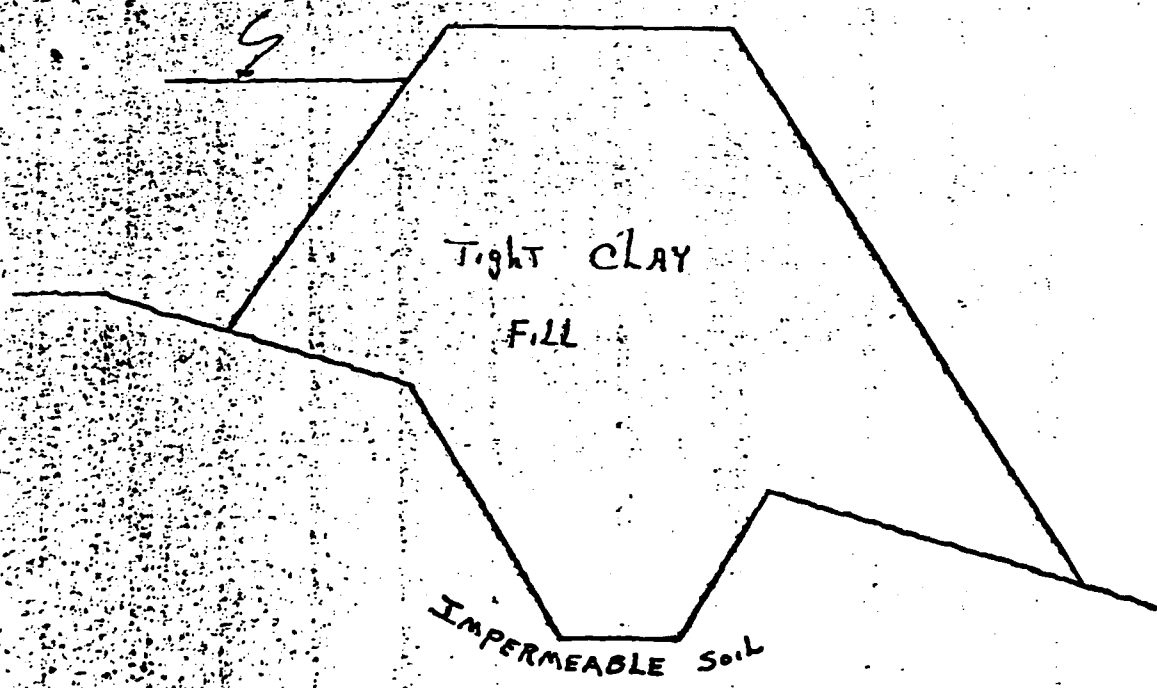


FIG. 2

a. Upstream cutoff:

QUALITY PRACTICABLE

An upstream cutoff, as shown in Figure 3, should adequately stop the flow of water through the dam making downstream drainage unnecessary for a dam this low. Advantages of this system are as follows:

- (1) Most, if not all, excavation and backfill can be done by equipment, lowering costs.
- (2) A positive cutoff should be achieved, conserving as much water as possible.
- (3) The 10' berm will dissipate wave action to protect the dam from erosion. This could be a real problem with a lake of this size.
- (4) Probably, material used here can be taken from around the shoreline, to remove unsightly low water areas.

Possible problems which could be encountered and which would lessen the effectiveness of an upstream cutoff are:

- (1) Unforeseen rock conditions under the dam might make it impossible to achieve a seal.
- (2) Some of the seepage could be from a spring under the dam rather than from leakage.

Neither of these conditions is considered likely, but if either of them were encountered, a drainage system as outlined in alternate method b. would be necessary.

b. Downstream Drainage:

A drainage system as shown in Figure 4 would adequately handle the seepage, protect the dam from failure, and eliminate the marshy area below the dam. The above-ground portion of the drain is necessary to weight the downstream toe, intercept the seepage, and carry it to the subsurface tile system. The subsurface portion of the drain by itself would dry up the marsh but it might not protect the dam adequately. Some disadvantages of this system are:

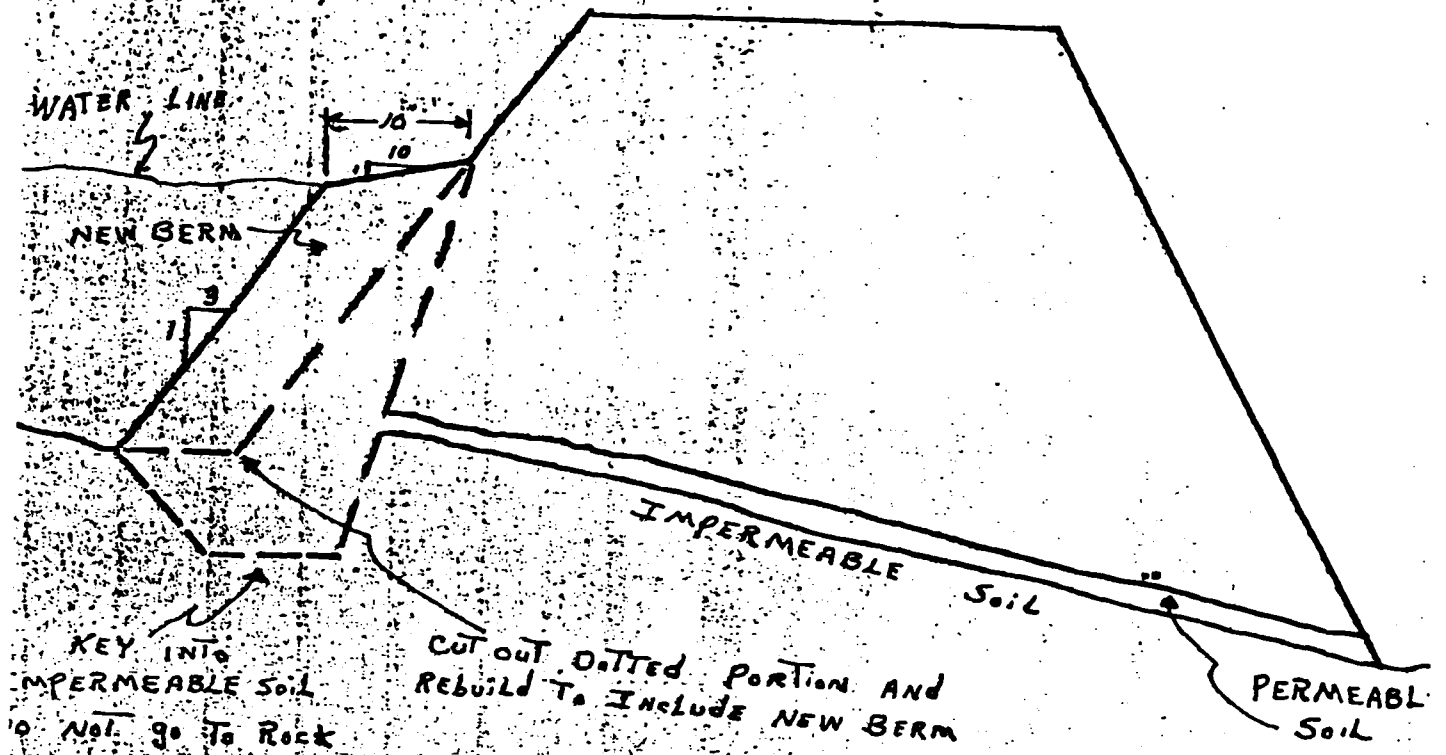


FIG. 3

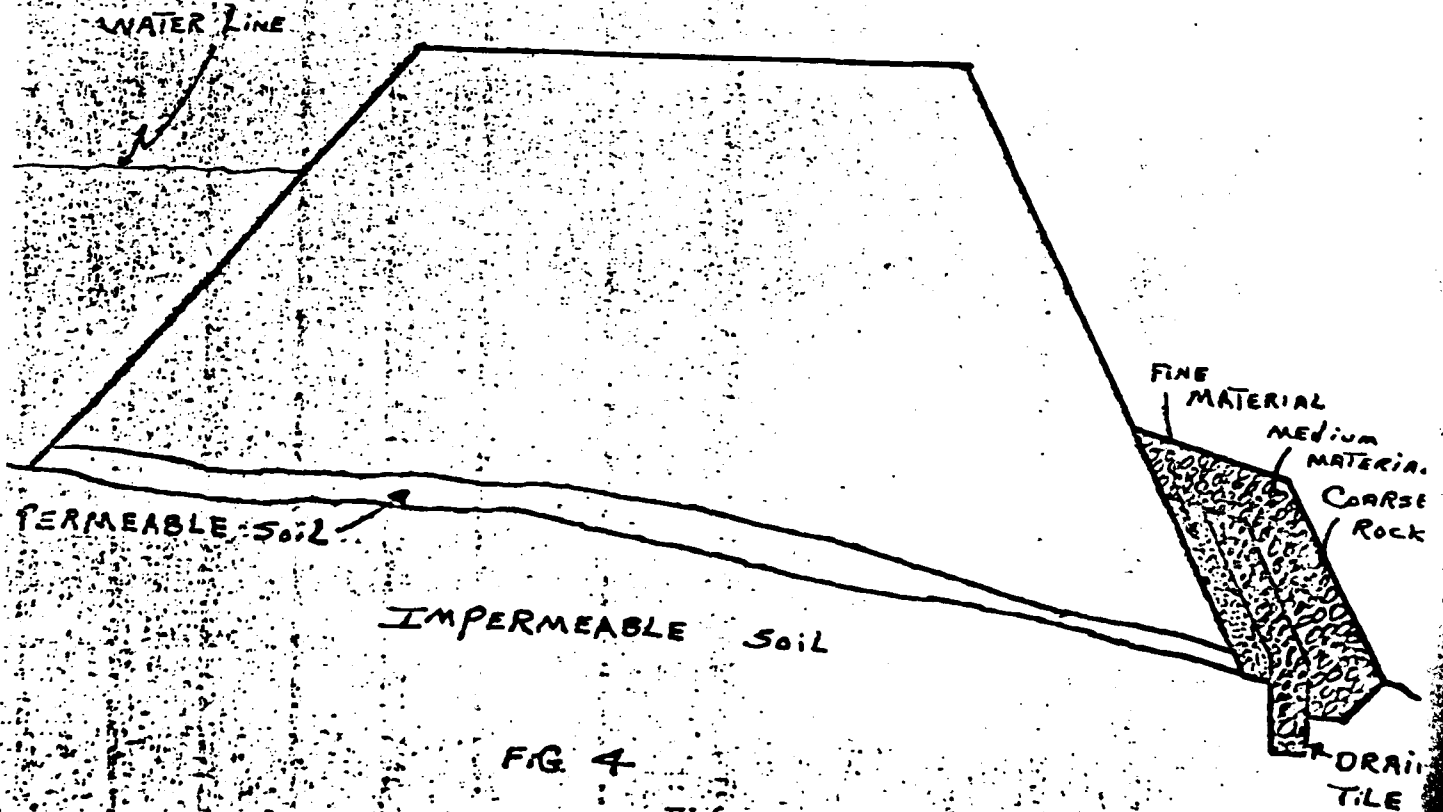


FIG. 4

(1) The filters must be carefully designed, prepared, and installed, and some hand labor will be necessary, all increasing the costs.

(2) An adequate outlet for the tile will be hard to achieve because of the elevation of the channel from the dam to the box culvert under the Port Republic Road. Even if this channel is lowered as much as possible, the tile cannot be buried deeply and some iron or steel pipe will be necessary further increasing the costs.

(3) Construction below the dam may involve disturbing the C&P telephone cable in this area, further increasing the costs.

(4) The dam will still be leaking at an excessive rate which may result in noticeable water lowering at dry times of the year.

(5) This method will not provide a berm to protect the dam from wave damage.

Recommendations:

SCS recommends that:

1. The upstream cutoff method of seepage control be used in this case. It appears to be the easiest way of handling the problem at lowest cost.

2. Consideration be given to installing a similar berm south of the concrete inlet even though no leakage is apparent here. Appearance would be better, and the entire dam would be protected from wave erosion.

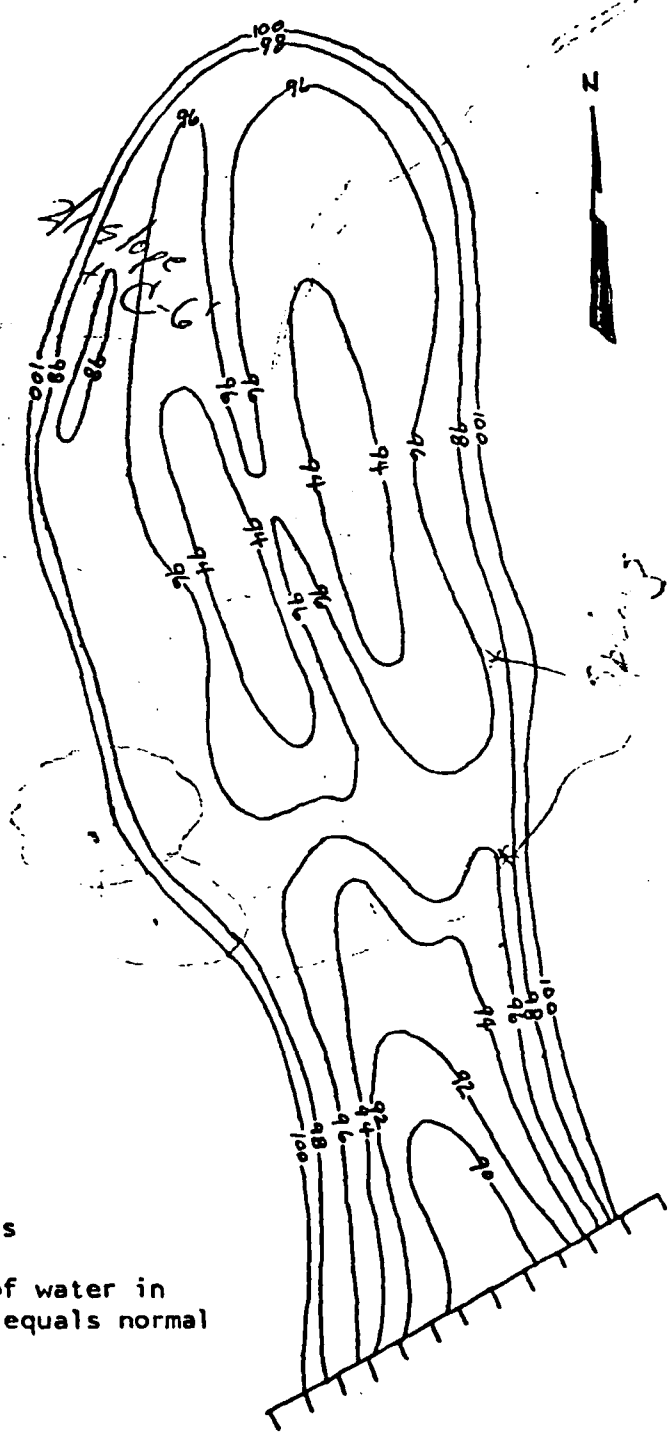
3. No excavation be done near the box inlet. Apparently a good seal was achieved here in construction, and it would be better not to disturb this area.

4. All excavation be done in such a way that no straight up-and-down banks are created. This would make compaction of the backfill very difficult.

5. All backfill material be carefully selected clay, placed in 6" layers in a rather moist--but not wet, condition, and well compacted with a heavy sheepsfoot roller.

#####

LAKE NEWMAN
MADISON COLLEGE



LEGEND

Scale - 1 inch = 50 feet
2 foot contour intervals

Topographic map showing depth of water in
Lake Newman - 100 foot contour equals normal
water level of lake.

Volume = 75 acre feet
Surface = 10 acres.

APPENDIX V - REFERENCES

1. Recommended Guidelines for Safety Inspection of Dams, Department of Army, Office of the Chief of Engineers, 46 pp.
2. Design of Small Dams, U. S. Department of Interior, Bureau of Reclamation, 1974, 816 pp.
3. Geology and Mineral Resources of Rockingham County, Bulletin 76, William B. Brent, Virginia, Division of Mineral Resources, 1960, 174 pp.
4. Section 4, Hydrology, Part 1 Watershed Planning, SCS National Engineering Handbook, Soil Conservation Service, U. S. Department of Agriculture, 1964.
5. Hydrometeorological Report No. 33, U. S. Department of Commerce, Weather Bureau, U. S. Department of Army, Corps of Engineers, Washington, D. C., April 1956.
6. Technical Paper No. 40, U. S. Department of Commerce, Weather Bureau, Washington, D. C., May 1961.