**Report on Seismic Stability, Onondaga Dam, New York.**

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**Abstract:**
This report is a reevaluation of the stability of the existing Onondaga Dam which is located near the city of Syracuse, NY. This is an earth and rockfill dam and provides flood protection for the city of Syracuse which is downstream of the dam. This evaluation includes a seismic evaluation (Attachment-2) and a stability analysis based upon current Corps criteria. This document contains a Main Report, six appendices (A-F), and two attachments. The Main Report contains the local and regional geology at the project site and summarizes the results of the stability analysis. Appendix A contains pertinent data (Cont'd).
about Onondaga Dam. Appendix B contains the rationale for the selected soil parameters used in the analysis. Appendix C contains hand calculations of the critical slip surfaces. Appendix D contains the computer input files and sample output for the slip circle analysis. Appendix E contains the wave runup analysis for the storage pool. Appendix F contains the slope protection calculations. Attachment No. 1 is the Concrete Spillway Stability analysis and Attachment No. 2 is the seismic evaluation.
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ATTACHMENTS

1. Spillway Stability Analysis
2. Report on Seismic Stability, Onondaga Dam, NY
1. INTRODUCTION

1.1 Authority.

Authority for continuing evaluation of a completed civil works structure, whose failure or partial failure would endanger the lives of the public or cause substantial property damage, is contained in ER 1110-2-100. Authority for a seismic investigation is contained in ER 1110-2-1800. This stability investigation has been performed in accordance with these regulations and North Central Division request contained in NCDED-T 1st Indorsement, periodic Inspection Report, Onondaga Dam dated 20 December 1978.

1.2 Scope.

This report includes a stability analysis for a cross section of the embankment (to include an evaluation of the slope protection) stability analysis of the spillway and a seismic stability report.

1.3 Purpose.

The purpose of this investigation is to comply with ER 1110-2-100, “Engineering and Design, Periodic Inspection and Continuing Evaluation of Completed Civil Works Structures,” Appendix A, paragraph 6d and ER 1110-2-1806, Engineering and Design, Earthquake Analysis for Corps of Engineers projects. These regulations call for a review of the stability of principle structures based on current criteria and a seismic evaluation of existing projects.

1.4 Background.

The Onondaga Dam Flood Control Project was authorized by the Flood Control Act of 1941 (Public Law 228, 77th Congress, 1st Session). Construction of the Dam was initiated in May 1947 and was completed in August 1949.

1.5 General Description of Onondaga Dam.

Onondaga Dam is located on Onondaga Creek about 4 miles southwest of the southern limits of Syracuse, NY (See Plate 1). The structure consists of a rolled earthfill embankment with a concrete overflow side channel spillway on the right abutment (See Plate 2). The overall length of the rolled fill embankment is 1,782 feet, having a maximum height of 67 feet. A 1,100-foot long spillway channel has been cut into bedrock along the right abutment. For additional pertinent data, see Appendix A.
2. GEOLOGY

2.1 Regional Geology.

2.1.1 Surficial Geology - Onondaga Dam is located in the northern part of the Allegheny Plateau Physiographic Province. The physiographic province, which lies immediately north of the dam, is a region of low relief and is termed the Lakes Plains Province. See Plate 3 for locations of physiographic provinces. The Allegheny Plateau is characterized as a region of moderate to high relief with elevations ranging from about 2,000 to 4,000 feet above sea level. The topography of the region has been produced by erosion of the underlying sedimentary strata and later modified by glacial processes. Glacial deposits are relatively thin on the upland portions of this province; however, the prominent north-south U-shaped valleys have been deeply eroded and filled by extensive moraine, lacustrine, and glacial outwash deposits. Some of the valleys are plugged at both ends by glacial deposits, thereby forming the Finger Lakes, while other valleys have subsequently drained.

2.1.2 Bedrock Geology - The stratigraphy of central New York consists of relatively undeformed, flat-lying sedimentary rocks ranging in age from Upper Ordovician to Upper Devonian. The predominant rock formations of the region consist of interbedded limestone, sandstone, and shale. The stratigraphic sequence dips gently southward at approximately 40 to 50 feet per mile such that the oldest units are exposed to the north with progressively younger formations exposed southward. Devonian rocks are by far the most extensively exposed in central New York. Devonian strata underlie all of the Allegheny Plateau region and much of the regions adjacent to it. Carbonates dominate the Lower to Middle Devonian with shales comprising the remainder of the section.

During the Pleistocene Epoch which began about 3 million years ago, central New York was covered by glacial ice, approximately 2 miles thick. The erosional effect of the ice mass was to deeply scour the valley of Onondaga Creek and other north-south trending valleys. As the ice receded northward, its margin paused south of the Onondaga damsite, depositing the Tully Moraine, near Tully, NY. With continued northward withdrawal of the glacial ice, a thin veneer of ground moraine in the form of glacial till was deposited over the bedrock surface. Glacial lakes formed south of the ice front and filled the valleys north of the Tully Moraine. One of many glacial lakes existed in the valley south of Syracuse. As the lake received meltwater from the west, a dramatic series of glacial lake deltas were deposited. As the glacial ice receded north into the Lake Ontario Basin many of the glacial lakes drained eastward into the Mohawk River. Those which did not drain formed the present day Finger Lakes.

2.1.3 Structural Geology - Generally, the sedimentary rocks in the area are horizontally bedded with a regional dip of approximately $1/2^\circ$ to the south. Superimposed on this dip are low-amplitude anticlines and synclines and faults of low displacement. The folds in the Syracuse and central New York region are related either to mild compression as a result of the Appalacian Orogeny or to removal of salt and gypsum in the Salina and Bertie Groups due to migrating groundwater. Some of the smaller folds may be the
result of subsidence. A number of small faults occur in the Syracuse central New York region. They are all small in lateral extent and displacement or offset. Generally, they strike N70°W, dip southward and are thrust faults that form due to compression. The overlying glacial deposits are not offset, indicating that no motion has taken place on these faults since the last ice retreat.

The joint system in the Syracuse region contains two main sets, one nearly north-south and the other nearly east-west. Most of the joints in these sets are nearly vertical. Two minor sets that strike northeast and northwest are also present.

In central New York, small dikes from a few inches to a few feet wide occupy the north-south joint set. They are composed of kimberlite and alnoite, a high temperature, ultra-basic igneous rock high in iron and magnesium silicates and low in aluminum silicates such as potash feldspars.

2.1.4 Earthquake Activity - Between 1720 and 1980, more than 330 earthquakes with a maximum Modified Mercalli intensity (Io) greater than II are known to have occurred in New York State (Mitronovas, 1981). New York State has been subdivided into three areas, a relatively high seismic activity separated by a large area of very low or no activity in the center of the State (which includes the Onondaga Dam site).

Details regarding the distribution of earthquakes, the evaluation for active faults and the intensity and effects of earthquake shaking at Onondaga Dam are summarized in "Report on Seismic Stability, Onondaga Dam, New York: Geological and Seismological Investigations at Onondaga Dam, New York" (Attachment 2). This report concludes that the dam is situated in an area that is structurally simple and tectonically stable.

2.2 Site Geology.

2.2.1 Surficial Geology

2.2.1.1 General - Geologic conditions at the dam have been largely influenced by the advance and waning of continental ice sheets during the Pleistocene. Plate 4 shows the local surficial geology. With recession of the glacial ice, a large terminal moraine was deposited, filling the valley at Tully and blocking drainage to the south. As the ice slowly retreated northward, vast quantities of meltwater flowed eastward along the front of the glacier, ponding a preglacial lake in the valleys south of Syracuse. The eastward-moving currents poured into Onondaga Valley through the well-defined Cedarvale Channel near West Onondaga, carrying large amounts of sediment which were rapidly deposited in a large delta at the point of entry into the lake. As the ice receded further, lake outlet channels to the east were uncovered at progressively lower elevations. This caused the delta to grow outward in a series of descending steps. Finally, the lake waters completely disappeared and the existing drainage, including Onondaga Creek, formed.

At the dam site, Onondaga Creek flows almost due north through a narrow steep-walled, post-glacial valley. The valley floor at the dam site consists of a floodplain approximately 600 feet wide.
2.2.1.2 Fluvial Overbank Deposits - The fluvial overbank deposits consist of gray and brown fine sandy silt, clay to silty clay, with an occasional layer of silty fine sand, small amounts of organic material, and are the original near-surface soils of the valley bottom. See Plates 7 and 8 for the extent of this material. Laboratory test results indicate the fluvial overbank deposits consist of 0-10 percent fine gravel, 15 to 37 percent sand, 25-40 percent silt, and 25 to 60 percent clay. As a result of the visual descriptions and laboratory test results, the fluvial overbank deposits would be classified as ML, CL, and SM soils according to the USCS. These are the weakest soils in the dam foundation. Appendix B summarizes the soil strength parameters.

2.2.1.3 Deltaic Deposits - Deltaic deposits underlie the overbank deposit in each of the recent test borings and generally consist of brown, silty, coarse to fine sand, gravel, to coarse to fine, sandy gravel, silt, with occasional layers of medium to fine sand. Plates 7 and 8 show the extent of the deltaic deposits. Grain size distribution curves of this deposit indicate the material consists of 0 to 49 percent gravel, 36 to 73 percent sand, and 9 to 27 percent silt. As a result of the above visual descriptions and laboratory test results, the deltaic deposits would be classified as GM, GC, SW, SM, and SP soils, according to the USCS. For the discussion of soil strength parameters, see Appendix B.

2.2.1.4 Lacustrine Deposits - Lacustrine sediments underly the deltaic deposits. These sediments consist of red-brown, silty fine sand, coarse to medium sand and fine gravel, to silty clay, coarse to fine sand, with occasional layers of coarse to fine sand, gravel, and silt. The extent of the lacustrine deposits are shown on Plates 7 and 8. Grain size distributions of representative samples of coarser portions of this deposit (silty fine sand) contained 60 to 72 percent fine sand and 28 to 40 percent silt. The hydrometer analysis of silty clay consisted of 10 percent sand, 30 percent silt, and 60 percent clay. As a result of the above visual descriptions and laboratory test results, the lacustrine deposits would be classified as SM, SF, ML, and CL, according to the USCS. For the discussion of soil strength parameters, see Appendix B.

2.2.1.5 Glacial Till - Glacial till underlies the lacustrine deposit. Descriptions of recovered samples range from gray, gravelly, coarse to fine sand, silt to brown, fine sandy silt, coarse to medium sand and gravel. Numerous cobbles and boulders were indicated during casing advance and sampling. No grain size distributions of this deposit were obtained. As a result of the above visual descriptions, the glacial till would be described as SM, GM, and GW soils, according to the USCS.

2.2.2 Bedrock Geology

2.2.2.1 Dam - The stratigraphic sequence of rocks exposed in the vicinity of the dam is represented by Lower and Middle Devonian Limestone and shales of the Helderberg Group, Onondaga Limestone, and Hamilton Group. Plate 5 shows the bedrock geology of central New York State. The Onondaga Limestone is exposed in the walls of the spillway cut. In this region, the Onondaga Formation is described as a series of light bluish grey
semi-crystalline limestone occurring in even continuous layers from 1 inch to 2 feet thick, separated by thin seams of dark calcareous shales. Flattened nodules of dark blue or black chert, sometimes in continuous sheets or beds, are unevenly distributed throughout the formation, but is predominant in the upper part.

The total thickness of the Onondaga Limestone is approximately 70 feet. The formation, subdivided into five members, from older to younger are: the Edgecliff Member, the Nedrow Member, the Moorehouse Member, the Tioga Bentonite Member, and the Seneca Member. Borings indicate that the top of rock occurs at 117 feet or deeper along the central portion of the dam embankment.

2.2.2.2. Spillway - Both the Nedrow and overlying Moorehouse Members are exposed within the spillway cut at Onondaga Dam. The Nedrow Member is characterized as medium grey, thin bedded, shaly limestone. A small amount of chert is unevenly distributed at the top of the unit. The Nedrow is about 10 feet thick and gradational with the overlying Moorehouse Member. The Moorehouse is described as a medium grey, very fine grained limestone, with 2-inch to 5-foot thick beds separated in many places by thin shale partings. Chert is common, but is variable in amount. The total thickness of the Moorehouse Member is about 25 feet.

Testing of rock was not conducted for the design analysis of the spillway. Strength parameters and permeability were not determined.

Based upon the descriptions of the rock from the 1945 subsurface exploration program, numerous horizontal and vertical seams are present. Generally, the material appears unweathered.

2.2.3 Structural Geology

2.2.3.1 Structural Deformations - Bedrock in the study area exhibits a gentle dip of 1/2° to the south. A gentle anticline is exposed in the spillway cut at the dam. The Manlius-Onondaga Contact rises to a maximum of 12 feet above the floor of the valley from the spillway northward, then descends further northward such that it is near creek level at the northern extremity of the exposure.

2.2.3.2 Joints - The joint system in the study area contains two main sets, one nearly north-south and the other nearly east-west. Most of the joints are nearly vertical. Two minor sets that strike northeast and northwest are also present. The results of the 1945 subsurface exploration program describe the rock core as containing numerous horizontal and vertical seams. Orientations of these defects were not discussed.

2.2.3.3 Faults - A number of small faults occur in the general vicinity of central New York; however, none were identified specifically at the project site.

2.2.3.4 Dikes - Dikes have been identified in isolated locations in central New York. None have been identified at the project site.
2.3 Groundwater.

Borings that were conducted for the original design analysis in 1945 along the dam embankment indicate an artesian flow of sulphur water occurred in the Manlius Limestone. The flow was spontaneous with little pressure after the first run, but increased slightly and became constant after drilling a total of 10 feet into rock. The overflow amounted to 1/2 gallon per minute through a 2-1/2-inch casing. The flow was sealed in the rock with Oakum and Portland cement. Artesian flow of sulphur water from the overburden was also encountered at a depth of 55 feet in one hole and in 105 feet in another hole. These holes were both sealed with Oakum and backfilled after the casing was pulled to prevent seepage.

The left portion of the dam rests against a steeply pitching deltaic terrace. This terrace is capped with a 40-foot layer of pervious sandy gravel underlain by a uniform sand bed averaging 45 feet in thickness. Immediately below, a bed of red silty clay occurs, 20 feet thick, serving as an impervious barrier to groundwater seepage. Plates 7 and 8 show this sequence of sediments. The thick sand beds also serve as a reservoir for subsurface drainage, and a perched water table has been formed with springs issuing from the terrace front. The perched water table was found at a depth of 71.0 feet in the uniform sand above the silty clay stratum. Groundwater was not encountered again after the casing reached the clay stratum. This perched water table probably has seasonal variations and finds outlet in seepage about halfway down the slope. The seepage upstream of the dam flows through the riprap slope protection to a cutoff trench at the base. It then runs to the creek channel. Downstream of the dam, the water runs along the rock toe to the old creek bed and runs northward until it merges with the creek channel.

At the time of the dam construction, a series of piezometers and settlement gages were installed. The settlement gages were constructed such that subsurface water level readings could also be obtained. Readings indicated a subsurface water level of approximately 460+ to 465+, closely corresponding to the original ground surface. Plate 7 shows where groundwater was encountered in the 1982 subsurface explorations. During the 1982 subsurface exploration program, regular water level observations were made during drilling operations and the water level in adjacent settlement gages were measured. These levels correspond closely with the original ground surface.

Twelve piezometers were installed to measure the porewater pressures under the downstream toe of the dam. Eight are located in the downstream rock toe of the dam embankment, and four are located behind the east wall of the spillway. The stability of the embankment slope depends on the porewater pressure realized.

Since the construction of the dam, water storage has never exceeded about one-third of the depth below spillway crest. Saturation levels are generally low because of the prevailing low stage. Drainage of the spillway wall backfill is provided by a perforated pipe drain and filter at the heel of the wall. The four piezometers behind the wall have not shown water levels higher than the drain elevation which was also used as the design saturation level.
3. EMBANKMENT STABILITY ANALYSIS

3.1 General.

The stability analysis was performed using EM 1110-2-1902 and WES Slope Stability Program 10009. The WES program utilizes the Modified Swedish Method. In this program, the sliding mass is divided into finite slices and a number of circular failure arcs are investigated to determine which is the most critical. In the Modified Swedish Method, the earth forces acting on the sides of the slice are considered. The direction of these side forces is assumed to be parallel to the average slope of the embankment and are changed to horizontal at the heel and toe. The program conducts a systematic search for the critical slip circle tangent to a specified depth. By varying the tangent elevation and the grid, the minimum factor of safety can be obtained. The grid was generally between 9 and 25 points per tangent elevation in order to insure that the minimum factor of safety was within the inside of the grid. The WES program is also designed to handle all cases outlined in the EM with the exception of Case VI - Surcharge Pool.

3.2 Stability Criteria.

The stability criteria for earth dams is set forth in EM 1110-2-1902. The minimum factor of safety for each case is given in Table 3.1

<table>
<thead>
<tr>
<th>Case Number</th>
<th>Design Condition</th>
<th>Minimum Factor of Safety</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>End of Construction</td>
<td>1.3</td>
<td>Upstream and Downstream Slopes</td>
</tr>
<tr>
<td>II</td>
<td>Sudden Drawdown From Maximum Pool</td>
<td>1.0</td>
<td>Upstream Slope</td>
</tr>
<tr>
<td>III</td>
<td>Sudden Drawdown From Spillway Crest</td>
<td>1.2</td>
<td>Upstream Slope</td>
</tr>
<tr>
<td>IV</td>
<td>Partial Pool With Steady Seepage</td>
<td>1.5</td>
<td>Upstream Slope</td>
</tr>
<tr>
<td>V</td>
<td>Steady Seepage With Maximum Storage Pool</td>
<td>1.5</td>
<td>Downstream Slope</td>
</tr>
<tr>
<td>VI</td>
<td>Steady Seepage With Storage Pool</td>
<td>1.4</td>
<td>Downstream Slope</td>
</tr>
<tr>
<td>VII</td>
<td>Earthquake (Cases I, IV, and V With Seismic Loading)</td>
<td>1.0</td>
<td></td>
</tr>
</tbody>
</table>

Table 3.1
3.3 Selection of Cross Section.

The cross section used in the analysis is at Station 6+02. This section was selected based on the following:

- It is in the area of maximum embankment height.
- Cross sectional data was readily available.
- It is at a location where a recent subsurface exploration was conducted.

3.4 Selection of Soil Parameters.

The embankment cross section and foundation details are shown on Plate 8. The parameters selected for the analysis are listed in Table 3.4.1. The rationale for these selected parameters is in Appendix A. A detailed description of the foundation materials is in paragraph 2.2.1.

The embankment is primarily made up of random fill materials excavated from borrow areas in the vicinity of the dam. The random fill materials may generally be described as brown, silty coarse to fine sand and gravel with an occasional layer of medium to fine sand and coarse to fine sandy gravel. Samples consist of 34 to 56 percent gravel, 31 to 39 percent sand, and 13 to 28 percent silt. As a result of the above visual descriptions and laboratory test results, the random fill materials would be classified as CM, SM, or SP soils, according to the Unified Soils Classification System (USCS). These materials are considered to be relatively pervious and compact to very compact.

Table 3.4.1

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>c</th>
<th>O</th>
<th>Unit Weight (pcf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Embankment</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Riprap</td>
<td>0</td>
<td>40°</td>
<td>105</td>
</tr>
<tr>
<td>Random Fill</td>
<td>0</td>
<td>36°</td>
<td>145</td>
</tr>
<tr>
<td>Impervious</td>
<td>0</td>
<td>34°</td>
<td>145</td>
</tr>
<tr>
<td>Foundation</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fluvial Overbank</td>
<td>0</td>
<td>23°</td>
<td>105</td>
</tr>
<tr>
<td>Deltaic Deposit</td>
<td>0</td>
<td>35°</td>
<td>119</td>
</tr>
<tr>
<td>Lacustrine</td>
<td>0</td>
<td>27°</td>
<td>124</td>
</tr>
</tbody>
</table>

These parameters are based on test results and boring log descriptions from the original subsurface exploration and testing program, and on a Corps of Engineers subsurface exploration program conducted in July and August 1982.
3.5 Results of the Analysis.

Table 3.5.1 summarizes the results of the analysis. Discussions of individual case are contained in paragraph 3.7.

Table 3.5.1

<table>
<thead>
<tr>
<th>Case</th>
<th>Condition</th>
<th>Min. F.O.S</th>
<th>Calculated F.O.S</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>As is (No Pool) US Slope</td>
<td>1.3</td>
<td>2.13</td>
</tr>
<tr>
<td></td>
<td>US Slope</td>
<td>1.3</td>
<td>1.65</td>
</tr>
<tr>
<td>II</td>
<td>Sudden Drawdown From Maximum Pool - US Pool</td>
<td>1.0</td>
<td>1.25</td>
</tr>
<tr>
<td>III</td>
<td>Sudden Drawdown From Spillway - Crest - US Pool</td>
<td>1.2</td>
<td>1.34</td>
</tr>
<tr>
<td>IV</td>
<td>Partial Pool With Steady Seepage - US Slope</td>
<td>1.5</td>
<td>1.83</td>
</tr>
<tr>
<td>V</td>
<td>Steady Seepage With May Storage - Pool - DS Slope</td>
<td>1.5</td>
<td>1.51</td>
</tr>
<tr>
<td>VI</td>
<td>Steady Seepage With Surcharge Pool - DS Slope</td>
<td>1.4</td>
<td>NA*</td>
</tr>
<tr>
<td>VII</td>
<td>Earthquake Cases I (US, DS) VI, V</td>
<td>1.0</td>
<td>1.8, 1.4, 1.5, 1.3**</td>
</tr>
</tbody>
</table>

* See Paragraph 3.7.6
** See Paragraph 3.7.7

The input data files used in the computer analysis and a sample output run are contained in Appendix D. It should be noted that for the analysis the cross sections were simplified somewhat in order to codify the soil profiles. In general, for the upstream slope cases, the simplifications included:

- elimination of the riprap slope protection and filter layer.
- simplification of the geometry of the impervious layer.
- elimination of downstream slope features (i.e. riprap toe and variable slope).

The downstream slope simplification includes:

- elimination of filter layer.
- simplification of the riprap toe geometry.
- elimination of upstream slope features.
A more detailed discussion is contained in subsequent paragraphs.

3.6 Hand Check of the Results.

The results of the computer analyses were checked by hand using the Simplified Bishops Method. This method was chosen for ease of hand calculation and to provide a check by an alternate stability method. The hand calculation checks were made on the critical slip surfaces obtained by computer. The calculations are contained in Appendix C. Table 4.4.2 summarizes the results.

Table 4.4.2

<table>
<thead>
<tr>
<th>Case</th>
<th>Min.</th>
<th>Factor of Safety</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Computer</td>
</tr>
<tr>
<td>I</td>
<td>US</td>
<td>1.3</td>
</tr>
<tr>
<td>I</td>
<td>DS</td>
<td>1.3</td>
</tr>
<tr>
<td>II</td>
<td></td>
<td>1.0</td>
</tr>
<tr>
<td>III</td>
<td></td>
<td>1.2</td>
</tr>
<tr>
<td>IV</td>
<td></td>
<td>1.5</td>
</tr>
<tr>
<td>V</td>
<td></td>
<td>1.5</td>
</tr>
</tbody>
</table>

While the factors of safety for Case IV and V are less than the minimum required, it is not considered to be significant because of the conservativeness of the assumptions and these cases are not considered critical for the dam (see paragraphs 3.7.4 and 3.7.5).

3.7 Discussion of Individual Cases.

3.7.1 Case I -

3.7.1.1 Upstream Slope - The tangent elevation of the slip surface was varied from 424' to 484' (see Figure 1A). The results indicate that the slip surface "walks out" due to the lack of a cohesion value for the materials. The significance is that the lowest factor of safety represents only a very shallow, noncritical failure surface. Therefore, for this analysis the minimum factor of safety chosen was for the first slip circle that was considered to be "significant" (i.e., that would encompass a significant portion of the embankment, where failure would endanger the structure). The factor of safety for this slip circle is 2.13. The water level was taken to be groundwater only and the elevation used is just above the embankment foundation line.
3.7.1.2 Downstream Slope - The tangent elevations for the slip circles was varied from 422' to 462' (see Figure 1B). The minimum factor of safety obtained was 1.65. "Walking out" was not encountered in this or any other subsequent downstream analysis.

3.7.2 Case II Sudden Drawdown From Maximum Pool - The sudden drawdown case assumes that steady seepage is occurring (the phreatic line is assumed to be horizontal) and the pool is drawn down from a maximum pool elevation of 520.4' to approximately ground level, 470' (see Figure 2). This is the most critical case for Onondaga Dam, which under flood conditions would experience rapid changes in pool elevation. Figures 7 and 8 show that the expected drawdown rate is approximately 80 hours from maximum pool to spillway crest elevation and then 11 days to drawdown the remainder of the pool to normal levels (i.e. no pool). The slip circle again exhibited "walk out" and a failure surface was chosen as in Case I at a point where failure would endanger the embankment. The minimum factor of safety obtained was 1.25.

3.7.3 Case III Sudden Drawdown for Spillway Crest Elevation - This case is the same as Case II. The minimum factor of safety obtained was 1.34 (see Figure 3).

3.7.4 Case IV - Partial Pool with Steady Seepage - The partial pool case examines the upstream slope stability for various pool levels. Steady seepage conditions are assumed and the phreatic line is assumed to be horizontal (see Figure 4). For each failure surface the pool elevation is varied and the minimum factor of safety is chosen. The minimum factor of safety obtained was 1.83 for the failure surface that would endanger the embankment.

3.7.5 Case V Steady Seepage with Maximum Storage Pool - It is unlikely that a condition of steady seepage would occur at the dam because of the rapid rise to and drawdown from maximum pool expected at the site (para. 4.6.2 above). The case was examined however, and a phreatic surface drawn (see Figure 5). The main portion of the dam consists of pervious materials. This portion of the embankment is approximately 300 times more pervious than the sloping impervious core (see Appendix B). The main portion of the dam would, therefore, drain freely to a level equaling the tailwater (see reference 9, Chapter 6). The tailwater at the dam is the result of backwater effects of flow downstream of the embankment (see Figure 7). The minimum factor of safety obtained for this case was 1.51.

3.7.6 Case VI - Steady Seepage with Surcharge Pool - The surcharge pool case assumes that there is a rapid increase in the pool height while the phreatic surface remains constant. In the case of Onondaga this would be a rapid increase from no pool, to maximum storage pool. The weight of the water would be added to that portion of the failure surface that it affects. The critical failure surface for the no pool case does not, however, intersect the upstream slope. An examination of failure surfaces that would intersect the upstream slope reveals relatively high factors of safety (1.8 - 3.9). It can be seen on Figure 6 that the effect of the additional water weight on these failure surfaces would be minimal. This case is, therefore, not critical for the dam. The minimum factor of safety would be the same as that at case I DS, 1.65.
3.7.7 Case VII Earthquake (Cases I, IV, and V) - ETL 1110-2-301 states that this case is no longer valid for embankment dams. It was, however, evaluated using the computer program and the critical slip circles for Cases I, IV, and V. It is included in the analysis for informational purposes only and is not supported by hand computation. For further information on the seismic stability of the embankment see paragraph 3.3 above and reference 7.

4. SPILLWAY STABILITY ANALYSIS

An evaluation of the stability of the side channel spillway was submitted after the 26 September 1978 Periodic Inspection of Onondaga Dam as an attachment to Period Inspection Report No. 3. It was approved by NCDED-T 1st Indorsement dated 25 March 1979. The analysis is extracted and attached as Attachment 1.

The spillway stability analysis examined the sliding stability of the concrete spillway using EM 1110-2-2200. The analysis examined the structure under a variety of loading and uplift conditions. Two separate failure modes were assumed: one at the concrete bedrock contact and the other along a plane through the rock below the spillway. The following Table 4.4.3 summarizes the results.
Table 4.4.3

Concrete - Rock Sliding Coefficient

<table>
<thead>
<tr>
<th>Case</th>
<th>Allowable</th>
<th>Calculated</th>
<th>Middle Third</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>0.65</td>
<td>0.41</td>
<td>Yes</td>
<td>1. Headwater at MAX Pool, 520.3'</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>2. Full Hydrostatic pressure against the upstream face.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>3. Tailwater at 497.5'.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>4. Uplift 100 percent HW at heel to 100 percent at toe.</td>
</tr>
<tr>
<td>II</td>
<td>0.65</td>
<td>0.43</td>
<td>Yes</td>
<td>1, 2, 3 as above.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>4. Uplift 100 percent HW at heel to 0 percent at toe.</td>
</tr>
<tr>
<td>III</td>
<td>0.65</td>
<td>0.23</td>
<td>Yes</td>
<td>1, 2, 4, as in Case I.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>3. TW at 504.5'.</td>
</tr>
</tbody>
</table>

Rock - Rock Sliding Coefficient

<table>
<thead>
<tr>
<th>Case</th>
<th>Allowable</th>
<th>Calculated</th>
<th>Middle Third</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>0.30</td>
<td>0.26</td>
<td>Yes</td>
<td>1, 2, 3, 4, as in Case I above.</td>
</tr>
<tr>
<td>II</td>
<td>0.30</td>
<td>NA</td>
<td></td>
<td>See Appendix F.</td>
</tr>
<tr>
<td>III</td>
<td>0.30</td>
<td>0.10</td>
<td>Yes</td>
<td>1, 2, 3, 4. Same as Case III above.</td>
</tr>
<tr>
<td>IV</td>
<td>0.30</td>
<td>0.28</td>
<td>No*</td>
<td>1, 2, 4. Same as Case I. 3 TW at 485.4'.</td>
</tr>
<tr>
<td>V</td>
<td>0.30</td>
<td>0.16</td>
<td>Yes</td>
<td>1. HW at 504.5'.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>2. Same as Case II.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>3. TW=0</td>
</tr>
</tbody>
</table>

*Resultant is .75 feet outside the middle third.

The report concluded that the calculated sliding friction was less than the allowable and the resultant was either within the middle or close to it.
5. SEISMIC STABILITY STUDY

In 1982, a study was conducted to determine the maximum earthquake that would effect the site and to assess the earthquake effects on the dam. The study was performed by Haley & Aldrich of New York under contract number DACW 49-81-D-0011 dated 13 October 1981. This report was submitted by the Buffalo District and approved by NCDEG-6 1st Indorsement dated 5 April 1983 subject to minor revisions. Attachment 2 is a copy of this report which includes the recommended revisions. The study included geological, seismological and subsurface investigations, and geotechnical engineering analyses. The study consisted of the following elements:

- An evaluation of the regional and local geology
- Performance of subsurface explorations and laboratory testing to further define the nature and density of soil deposits
- An evaluation of the regional and local tectonic history with respect to structural deformation including faults
- A review of historical regional seismicity
- The determination of the maximum earthquake that will affect the site
- An assessment of earthquake effects on the dam, including an evaluation of liquefaction potential of the subsurface soils

The report concluded that the dam is located in an area that can be described as being nearly aseismic. The maximum earthquake intensity expected is VI (Modified Mercalli Intensity) with a peak horizontal ground acceleration of 0.05g in rock and 0.06g in soil. The report went on to conclude that:

"...The embankment and foundation soils are not considered to be susceptible to liquification... Minor seismically-induced settlement of the embankment and subsoils may occur, but the settlement will not be detrimental to the performance of the structure."

6. SLOPE PROTECTION EVALUATION

4.1 General.

The upstream slope of Onondaga Dam is protected by a 36-inch thick dumped layer of riprap that was excavated from the spillway channel during construction of the dam. This stone is breaking up thereby reducing the protection that it affords the dam. Reduction in size is estimated to be more than 50 percent. See Figures 10 through 13.

The design gradation curve for the riprap is at Figure 14. The rationale behind this gradation specification is not apparent; however, Sherard (Reference 17) reports that in the mid 1940's it was commonly considered that a dumped riprap layer of 36 inches was satisfactory under any wave action. Therefore, it is assumed that the design was adequate.
An analysis using the current Corps criteria as outlined in EM 1110-2-2300 and ETL 1110-2-120 is necessary for comparison to the original design specifications. An evaluation must then be made to determine the effect of the breakup.

6.2 Wave Analysis.

A wave analysis for Onondaga Dam was performed and is at Appendix E. The recommended maximum wave height for Onondaga Dam is 2 feet.

6.3 Gradation requirements.

Using EM 1110-2-2300, and ETL 1110-2-120, a specific gravity of 2.65, an average value of the slope of the embankment, and a wave height of 2 feet, the following gradation is obtained:

\[
\begin{align*}
\text{Median rock size } W_A &= 27.5 \text{ lb.} \\
\text{Riprap Thickness } T &= 12 \text{ in.} \\
W_{\text{MAX}} &= 4 \times W_A = 110 \text{ lb.} \\
W_{\text{MIN}} &= W_A/8 = 3.4 \text{ lb.}
\end{align*}
\]

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<th>Limits of Stone Weight (lbs.)</th>
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<td>10</td>
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This information is plotted on Figure 14 for comparison with the original design specifications. The actual computations are in Appendix F.

7. CONCLUSION AND RECOMMENDATIONS.

7.1 The embankment, spillway, and foundation of Onondaga Dam have been determined to be statically stable and meet all current Corps criteria. No further analysis is recommended at this time.

7.2 The foundation and embankment soils are not considered to be susceptible to liquefaction and the dam will experience no reduction in its capacity after experiencing the maximum expected earthquake. No further analysis is recommended at this time.

7.3 The rip-rap slope protection original design specification gradation exceeds current Corps criteria. The extent to which the current significant rip rap breakup affects the slope protection is unknown. In order to make a more complete evaluation of current conditions, the existing gradation of the rip rap we need to be established and the effects of future breakup considered. In order to determine the degree of deterioration, in-place gradation test would need to be performed.
8. REFERENCES


NOTES:
1. FOR RATIONALE OF ADOPTED DESIGN VALUE SELECTION, SEE...
2. FOR COMPUTER INPUT FILES AND SAMPLE OUTPUT LISTINGS
TANGENT CENTER FACTOR OF SAFETY

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<td>160</td>
<td>780</td>
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* SELECTED MIN. FOS. OTHER FAILURE SURFACES REPRESENTS ONLY SHALLOW NON CRITICAL SURFACES

ADOPTED DESIGN DATA

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FAILURE SURFACE \( F = 2.13 \)

EMBANKMENT RANDOM FILL

GROUND WATER

FLUVIAL OVERBANK

DELTAIC DEPOSIT

LACUSTRINE

ONONDAGA DAM, NEW YORK

STABILITY ANALYSIS CASE 1

UPSTREAM SLOPE

STA. 6 + 02

U.S. ARMY ENGINEER DISTRICT BUFFALO

TO ACCOMPANY STABILITY ANALYSIS DATED MAY 1966

FIGURE 1A
TANGENT CENTER

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<td>462</td>
<td>700</td>
<td>-160</td>
<td>700</td>
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* Minimum

NOTES:
1. FOR RATIONALE OF ADOPTED DESIGN SEE APPENDIX D.
### ADOPTED DESIGN DATA

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* MINIMUM

---

For the rationale of adopted design values selection, see Appendix B. For listings of computer input files and sample output, see Appendix D.
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2. FOR COMPUTER INPUT FILES AND SAMPLE OUTPUT
TANGENT CENTER

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SELECTED MIN F.O.S., OTHER SURFACE REPRESENTS ONLY SHALLOW NON CRITICAL FAILURE SURFACE

ADOPTED DESIGN DATA

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ONONDAGA DAM, NEW YORK
STABILITY ANALYSIS, CASE 11
SUDDEN DRAWDOWN FROM MAXIMUM POOL
STA.6+02
U.S. ARMY ENGINEER DISTRICT BUFFALO
TO ACCOMPANY STABILITY ANALYSIS
DATED: MAY 1986

FIGURE 2
CENTER +120.580

POOL ELEVATION BEFORE DRAWDOWN
(Spillway Crest El.)

IMPERVIOUS ZONE

POOL ELEVATION AFTER DRAWDOWN

FAILURE SURFACE

FLUVIAL

DELTAIC

LACUSTRINE

CROSS SECTION STATION

NOTES:
1. FOR RATIONALE OF ADOPTED DESIGN VALUE SEL
2. FOR LISTINGS OF COMPUTER INPUT FILES AND S
ONONDAGA DAM, NEW YORK
STABILITY ANALYSIS, CASE III
SUDDEN DRAWDOWN FROM SPILLWAY CREST
STA. 6 + 02
US ARMY ENGINEER DISTRICT BUFFALO
TO ACCOMPANY STABILITY ANALYSIS
DATED: MAY 1966

SECTION STATION

ONALE OF ADOPTED DESIGN VALUE SELECTION, SEE APPENDIX B.
ININGS OF COMPUTER INPUT FILES AND SAMPLE OUTPUT, SEE APPENDIX D.
TANGENT CENTER POOL FACTOR OF EL. X Y EL. SAFETY

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SELECTED MIN F.O.S.

ADOPTED DESIGN DATA

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ONONDAGA DAM, NEW YORK
STABILITY ANALYSIS, CASE IV
PARTIAL POOL WITH STEADY SEEPAGE
STA. 6+02
U.S. ARMY ENGINEER DISTRICT BUFFALO
TO ACCOMPANY STABILITY ANALYSIS
DATED: MAY 1986

FIGURE 4
1. Tailwater is backwater from flow restrictions downstream of dam assume backwater.
2. Seepage line is near vertical in embankment because it is approximate 300 times.
3. For rationale of adopted design values, see Appendix B.
4. For listings of computer input files and sample output, see Appendix D.
**Selected Min. F.O.S.**

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**ADOPTED DESIGN DATA**

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**ONONDAGA DAM, NEW YORK STABILITY ANALYSIS CASE V STEADY SEEPAGE FROM MAXIMUM POOL STA.6+02**

**DATED: MAY 1986**

**FIGURE 5**
Effect of surcharge pool is minimal on these failure surfaces.

**NOTES:**

1. Factors of Safety (F) are prior to application of the surcharge pool.
2. For rationale of adopted design values, see Appendix B.
3. For listings of computer input files and sample output, see Appendix D.
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TAILWATER ELEVATION IN FEET ABOVE M.S.L.

W.S. (EXISTING CHANNEL CONDITIONS)

EL. 464.7 (H.W. MARK - 3/17/44)

NOTE:
1. CURVE APPLIES 600' BELOW CENTERLINE OF DAM.
2. TAILWATER IS EFFECTIVELY BACKWATER AND NOT DUE TO SEEPAGE THROUGH THE DAM.

FIGURE 7

ONONDAGA DAM, NEW YORK
TAILWATER RATING CURVE
U.S. ARMY ENGINEER DISTRICT, BUFFALO
TO ACCOMPANY STABILITY ANALYSIS DATED
ONONDAGA DAM, NEW YORK

DRAWDOWN CURVES

U.S. ARMY ENGINEER DISTRICT BUFFALO
TO ACCOMPANY STABILITY ANALYSIS DATED

FIGURE 9
Figure 12 - Riprap Size Reduction

Figure 13 - Riprap Size Reduction
WEIGHT OF STONES IN POUNDS

SPECIFIC GRAVITY OF ROCK = 2.65
* ASSUMING STONE SHAPE MIDWAY BETWEEN A SPHERE AND CUBE

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ONONDAGA DAM, NEW YORK
RIPRAP GRADATION CURVES
U.S. ARMY ENGINEER DISTRICT, BUFFALO
TO ACCOMPANY STABILITY ANALYSIS
DATED
REF: DESIGN ANALYSIS
FOR ONONDAGA
RESERVOIR (1945)

ONONDAGA DAM NEW YORK
PROJECT LOCATION
US ARMY ENGINEER DISTRICT BUFFALO
TO ACCOMPANY STABILITY ANALYSIS
DATED MAY 1986

PLATE 1
LEGEND

APPROXIMATE BOUNDARY OF PHYSIOGRAPHIC PROVINCES

MESOZOIC ERA

- CRETACEOUS (UPPER) CLAY, SAND AND GRAVEL (UNCONSOLIDATED BEDROCK)
- TRIASSIC (UPPER) RED SANDSTONE, SHALE AND CONGLOMERATE, INTRUDED BY PALISADES SILL

PALEOZOIC ERA

- PENNSYLVANIAN AND MISSISSIPPIAN CONGLOMERATE
- DEVONIAN (LATE UPPER) SHALE, SILTSTONE, SANDSTONE
- DEVONIAN (EARLY UPPER) SHALE, SILTSTONE, SANDSTONE
- DEVONIAN (LOWER AND MIDDLE) LIMESTONE OVERLAIN BY SHALE, SILTSTONE AND SANDSTONE
- SILURIAN DOLOSTONE, LIMESTONE SHALE, SALTBEDS, SANDSTONE, CONGLOMERATE IN SOUTHEASTERN PART OF STATE
- ORDOVICIAN, MAINLY SHALE AND SANDSTONE IN UPPER PART, LIMESTONE AND DOLOSTONE IN LOWER
- CAMBRIAN SANDSTONE AND QUARTZOSE DOLOSTONE
- PRECAMBRIAN, CAMBRIAN, ORDOVICIAN - INTENSELY FOLDED AND THRUST-FAULTED SHALE, PHYLLITE, SCHIST, GNEISS, GRANITE, MARBLE, SCHIST, MARBLE, DOLERITE, ON STATE ISLAND, ALSO SHALES, LIMESTONE AND MARBLE

PRECAMBRIAN

- METAMORPHIC AND IGNEOUS ROCKS, STRUCTURALLY COMPLEX

NOTES:

- BOUNDARIES OF PHYSIOGRAPHIC PROVINCES APPEARENTLY
- PLATE FROM ONONDAGA DAM SEISMIC STABILITY INVESTIGATION DATED NOV. 1962

ONONDAGA DAM, NEW YORK

PHYSIOGRAPHIC PROVINCES AND GENERAL BEDROCK GEOLOGY OF NEW YORK STATE

US ARMY ENGINEER DISTRICT BUFFALO
TO ACCOMPANY STABILITY ANALYSIS DATED MAY 1966

PLATE 3
NOTES
1. BEDROCK CONTACT LINES AND LOCATIONS OF QUATERNARY DEPOSITS ARE APPROXIMATE AND BASED ON GEOLOGIC LITERATURE REVIEW AND FIELD MAPPING BY USA OF NEW YORK PERSONNEL DURING THE FALL OF 1981.
2. CONTACT LINES FOR THE ONONDAGA LIMESTONE AND MANLIUS FORMATION ARE GENERALLY BURIED WITHIN QUATERNARY DEPOSITS AND ARE THEREFORE INFERRED FROM AVAILABLE GEOLOGIC INFORMATION AND OBSERVATIONS.
3. CONTACT LINES FOR UNITS ABOVE THE ONONDAGA LIMESTONE ARE EITHER OBSERVED OR CLOSELY APPROXIMATED AS MANY SMALLER EXPOSURES OF THESE UNITS EXIST ALONG THE VALLEY WALLS BETWEEN THE MAJOR BEDROCK EXPOSURES.
4. THE TOPS AND SIDES OF HILLS ARE GENERALLY BEDROCK OVERLAIN BY A MANTLE OF GLACIAL TILL.
5. BASE MAP OBTAINED FROM USGS QUADRANGLE SOUTH ONONDAGA NEW YORK.
6. PLATE FROM ONONDAGA DAM SEISMIC STABILITY INVESTIGATION DATED NOVEMBER 1982.
NOTES:

1. PLATE FROM ONONDAGA DAM SEISMIC STABILITY INVESTIGATION DATED NOVEMBER 1982.
2. FOR PROFILE A-A SEE PLATE 7.
3. FOR CROSS SECTION B-B SEE PLATE 8.
PERTINENT DATA
APPENDIX A
STABILITY ANALYSIS

U.S. Army Corps of Engineers, Buffalo District
1776 Niagara Street
Buffalo, NY
APPENDIX A
PERTINENT DATA
ONONDAGA DAM AND RESERVOIR

A1. GENERAL
Purpose - Flood Control
Drainage area above dam - 68.1 sq. mi.
Drainage area, U.S.G.S. gage (Dorwin Ave.) - 88.9 sq. mi.
Drainage area, mouth of Onondaga Creek - 108.9 sq. mi.

A2. DAM
Type - Rolled Earth
Length, feet - 1,782
Maximum height, feet - 67
Top width, feet - 25
Top elevation, feet above mean sea level - 526

A3. SPILLWAY
Type - Uncontrolled ogee, side channel overflow
Crest length, feet - 200
Crest elevation, feet above mean sea level - 504.5
Surcharge, design flood, feet - 15.8
Capacity at 15.8 feet surcharge - 48,500 cfs

A4. OUTLET
Type - Uncontrolled circular conduit
Number - One
Diameter, feet - 6.5
Length, feet - 329
Location - Under east (right) section of dam

A-1
A4. OUTLET (Cont'd)

Invert elevation at intake, feet - 457.0
Invert elevation at outlet, feet - 456.21
Discharge, pool at spillway crest elevation, cfs - 1,270
Minimum time required to empty reservoir from spillway crest elevation, no inflow - with assumed base flow of 2 cfs/sq. mile - 11 days

A5. RESERVOIR

Area, spillway crest elevation (504.5) - 910 acres
Capacity spillway crest elevation (504.5) - 18,200 acre feet
Area, 15.8 feet surcharge - 1,640 acres
Capacity 15.8 feet surcharge - 38,200 acres feet
ONONDAGA DAM, NY

SELECTION OF ANALYSIS
SOIL PARAMETERS
APPENDIX B
STABILITY ANALYSIS

U.S. Army Corps of Engineers, Buffalo District
1776 Niagara Street
Buffalo, NY
APPENDIX B

SELECTION OF ANALYSIS
SOIL PARAMETERS

B1. GENERAL

The parameters used in this stability analysis were based on the test data from the original 1945 Design Analysis (Reference 5) and a subsurface exploration program conducted for the seismic stability analysis done in 1982 (Reference 7). These two programs are discussed in more detail in subsequent paragraphs.

B2. 1945 DESIGN ANALYSIS EXPLORATION PROGRAM

The original design analysis exploration program was carried out in 1944 and 1945. It consisted primarily of 2-1/2-inch diameter holes and test pits at the dam site and at potential borrow areas. The boring log descriptions for holes in the vicinity of the analysis cross section are on Plate 3. The testing program consisted of classification, density, consolidation, direct shear, triaxial shear, permeability and compaction. The results of these tests are at Figures B1 thru B11 and they are summarized in Tables B1 through B3. The direct shear and triaxial tests were consolidated undrained tests (R tests). There is no consolidated drained (S) test data or unconsolidated undrained (Q) test data available for the analysis. Therefore, all strengths used in the analysis are R strengths. In those materials where cohesion was present, it was ignored. This was done to be conservative where cases required a composite strength of envelope (R and S) See Figures B12 and B13.

B3. 1982 EXPLORATION PROGRAM

In 1982, an exploration program was conducted to determine the seismic stability of the dam. The program consisted of three test borings from the dam crest. These borings provide the most recent data available on the dam. The laboratory testing consisted of natural water contents, Atterberg Limits, and grain size distribution. The only information obtained in this program that is relatable to strength is the blow counts (standard penetration test - SPT). The SPT data and boring descriptions are at Plate 7.

B4. DESCRIPTION OF SOILS AND PARAMETER SELECTION

B4.1 Riprap.

The rock used in the slope protection and toe was excavated from the spillway channel. A specific gravity of 2.65 (limestone) an angle of internal friction of 40°, and an average porosity for dumped riprap of 36 percent was assumed (Reference 12). This yields a unit weight of 105 pcf for this riprap.
B4.2 Filter Material.

The filter material was ignored in this analysis due to its similarity to the embankment material and its relatively small size.

B4.3 Impervious Material (Core).

The unit weight and internal angle of friction, 145 pcf and 34° respectively, were obtained from the original design analysis and are based on test results conducted on samples taken from borrow areas.

B4.4 Random Fill Embankment Materials.

The unit weight and angle of internal friction, 145 pcf and 36° respectively, were obtained from the original design analysis. The value of φ appears to be on the conservative side based on the blow counts obtained for the 1982 exploration program. The blow counts indicate that the material is compact to very compact. According to Bowles (Ref. 8) this indicates an angle of internal friction between 38° and 43° and Hough (Ref. 10) indicates that for a compact sand and gravel mixture or coarse sand, the angle of internal friction could be as high as 45°.

B4.5 Fluvial Overbank.

The value of 23° for the angle of internal friction is an average value obtained from the original design analysis. The values in the analysis vary from 19.5° to 32° for the sandy and clayey silts. The unit weight of 105 pcf was the result of modifying the value in the analysis by lowering from 110 pcf to be on the conservative side.

B4.6 Deltaic Deposit.

The value of 35° was assigned based on a range of values obtained in the original design analysis (34°-36°) and comparing them to typical values of φ based upon blow counts. The value of 35° is conservative. The unit weight of 119 also obtained from the original analysis values of 117-120 pcf.

B4.7 Lacustrine.

The values of 27° and 124 pcf were assigned based on the original analysis.

B4.8 Till.

No values were assigned. The glacial till was considered to be "firm base."

B5 SUMMARY

The values selected in each case are based on test values and blow count information and are considered to be conservative values.
TABLE 3-1

PHYSICAL PROPERTIES OF FOUNDATION MATERIALS

<table>
<thead>
<tr>
<th>Soil Classification</th>
<th>Direct Shear ( (\phi C) ) (t.s.f.)</th>
<th>Triax. Shear ( (\phi C) ) (t.s.f.)</th>
<th>Coeff. of Perm. (cm/sec x10(^{-4}))</th>
<th>Unit Wt. (p.c.f.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sandy GRAVEL</td>
<td>35° 15', 0.0</td>
<td>36° 30', 0.0</td>
<td>100-950</td>
<td>131 125</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>(450 Av.)</td>
</tr>
<tr>
<td>Uniform Medium SAND</td>
<td>32° 30', 0.0</td>
<td>36° 40', 0.0</td>
<td>1-10</td>
<td>135 108</td>
</tr>
<tr>
<td>Silty CLAY</td>
<td>16° 30', 0.35</td>
<td>0.0001</td>
<td>127 99</td>
<td></td>
</tr>
<tr>
<td>Silty SAND</td>
<td><strong>36° 00', 0.0</strong></td>
<td><strong>1</strong></td>
<td><strong>15</strong></td>
<td></td>
</tr>
<tr>
<td>With embedded</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gravel</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Silt-bound sandy GRAVEL</td>
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<td><strong>15</strong></td>
<td><strong>15</strong></td>
<td></td>
</tr>
<tr>
<td>B. LEFT ABUTMENT SLOPE</td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>Silty, sandy GRAVEL</td>
<td><strong>36° 00', 0.0</strong></td>
<td><strong>15</strong></td>
<td><strong>15</strong></td>
<td></td>
</tr>
<tr>
<td>Fine SAND</td>
<td><strong>36° 00', 0.0</strong></td>
<td><strong>15</strong></td>
<td><strong>15</strong></td>
<td></td>
</tr>
<tr>
<td>SAND &amp; SILT</td>
<td><strong>30° 00', 0.0</strong></td>
<td><strong>15</strong></td>
<td><strong>15</strong></td>
<td></td>
</tr>
<tr>
<td>Sandy SILT</td>
<td><strong>30° 00', 0.0</strong></td>
<td><strong>15</strong></td>
<td><strong>15</strong></td>
<td></td>
</tr>
<tr>
<td>Silty CLAY</td>
<td><strong>16° 40', 0.22</strong></td>
<td><strong>16° 00', 0.20</strong></td>
<td><strong>0.0001</strong></td>
<td>122 94</td>
</tr>
</tbody>
</table>

* Value assigned from tests on similar materials from the site.
** Undisturbed samples
### TABLE (CONTINUED)

#### PHYSICAL PROPERTIES OF FOUNDATION MATERIALS

<table>
<thead>
<tr>
<th>Soil Classification</th>
<th>Direct Shear C</th>
<th>Triax. Shear C</th>
<th>Coeff. of Perm. (cm/sec x 10^-4)</th>
<th>Unit Wt. (p.o.f.)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(t.s.f.)</td>
<td>(t.s.f.)</td>
<td></td>
<td>Wet</td>
</tr>
<tr>
<td>C. VALLEY FLOOR</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Silty CLAY (At Surface)</td>
<td>19° 30'</td>
<td>0.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Silty CLAY (At Depth)</td>
<td>26° 00'</td>
<td>0.02</td>
<td></td>
<td>112</td>
</tr>
<tr>
<td>Clayey SILT</td>
<td>**28° 10'</td>
<td>0.1</td>
<td>23° 10'</td>
<td>0.19</td>
</tr>
<tr>
<td>Sandy SILT</td>
<td>**32° 00'</td>
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<td>0.001</td>
<td>110</td>
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<tr>
<td>Fine to Coarse SAND</td>
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<td>37° 30'</td>
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<tr>
<td>Fine to Coarse SAND with embedded Gravel</td>
<td>34° 10'</td>
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<td>13-43</td>
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<tr>
<td>Coarse SAND with embedded Gravel</td>
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<tr>
<td>Silty GRAVEL</td>
<td>27°-31°</td>
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<td>0.3-5</td>
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</tr>
<tr>
<td>Sandy GRAVEL</td>
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<td>0.00</td>
<td>15-200</td>
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<tr>
<td>D. ALLUVIAL FAN</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Clayey Silt &amp; SAND</td>
<td>**22°-26°</td>
<td>0.15</td>
<td>0.2</td>
<td>116</td>
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<td>Silty, Sandy GRAVEL</td>
<td>25° 45'</td>
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<td>1.0</td>
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</tr>
<tr>
<td>Sandy GRAVEL</td>
<td>36° 00'</td>
<td>0.00</td>
<td>25-70</td>
<td>117-120</td>
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<tr>
<td>E. RIGHT ABUTMENT</td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Silty, Sandy GRAVEL</td>
<td>*36° 00'</td>
<td>0.00</td>
<td>1.0</td>
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* Value assigned from tests on similar materials from the site
** Undisturbed samples
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<th>Soil Classification</th>
<th>Direct Shear Compaction</th>
<th>Coeff. of Permeability</th>
<th>Max. Unit Wt. (p.c.f.)</th>
<th>Opt. W (Percent) (Dry Wt.)</th>
<th>Dry Unit Wt. (p.c.f.)</th>
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<td></td>
<td>$\phi$ &amp; $C$ (t.s.f.)</td>
<td>(cm./sec. x 10^-4)</td>
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<td></td>
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<tr>
<td>A. LEFT ABUTMENT</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>Sandy GRAVEL</td>
<td>36° 30′ 0.0 100-950 130</td>
<td>9 124</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>B. LEFT ABUTMENT SLOPE</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Silty Sandy GRAVEL</td>
<td>36° 00′ 0.0 1.0 127</td>
<td>10</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Uniform Fine SAND</td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SAND &amp; SILT</td>
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<td></td>
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<tr>
<td>Silty CLAY</td>
<td>16° 00′ 0.2 0.0001 94</td>
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<td></td>
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<td></td>
</tr>
<tr>
<td>C. ALLUVIAL FAN</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Clayey SILT &amp; SAND</td>
<td>22°-28° 0.15 0.2</td>
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<td></td>
<td></td>
<td>95</td>
</tr>
<tr>
<td>Silty, Sandy GRAVEL</td>
<td>36° 00′ 0.0 1.0 127 10</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sandy GRAVEL</td>
<td>36° 00′ 0.0 25-70 131</td>
<td>9</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>D. RIGHT ABUTMENT</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Silty, Sandy GRAVEL</td>
<td>36° 00′ 0.0 1.0 127 10</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* Value assigned from tests on similar materials from the site
<table>
<thead>
<tr>
<th>Soil Classification</th>
<th>Shear C (t.s.f.)</th>
<th>Coeff. of Permeability (cm/sec. x 10^-4)</th>
<th>Unit Wt. (p.c.f.)</th>
<th>Wet</th>
<th>Dry</th>
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<tbody>
<tr>
<td>A. Pervious Section of Dam Embankment</td>
<td></td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sandy Gravel</td>
<td>36° 00' 0.0</td>
<td>*900</td>
<td>300</td>
<td>145</td>
<td>140</td>
</tr>
<tr>
<td>B. Imperious Section of Dam Embankment</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Silty Sandy Gravel</td>
<td>34° 00' 0.0</td>
<td>*3.0</td>
<td>1.0</td>
<td>145</td>
<td>133</td>
</tr>
<tr>
<td>C. Foundation</td>
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<td></td>
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<td></td>
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</tr>
<tr>
<td>Silty Clay (at surface)</td>
<td>19° 30' 0.0</td>
<td></td>
<td></td>
<td>95</td>
<td></td>
</tr>
<tr>
<td>Silty Clay (at depth)</td>
<td>26° 00' 0.02</td>
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<td>95</td>
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<td>Fine to Coarse Sand</td>
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<td>*150</td>
<td>50</td>
<td>*100</td>
<td></td>
</tr>
<tr>
<td>Fine to Coarse Sand with embedded Gravel</td>
<td>36° 00' 0.0</td>
<td>*120</td>
<td>40</td>
<td>*100</td>
<td></td>
</tr>
<tr>
<td>Coarse Sand with embedded Gravel</td>
<td>36° 30' 0.0</td>
<td>*1500</td>
<td>500</td>
<td>*100</td>
<td></td>
</tr>
<tr>
<td>Silty Gravel</td>
<td>30° 00' 0.0</td>
<td>*15</td>
<td>5</td>
<td>*100</td>
<td></td>
</tr>
<tr>
<td>Sandy Gravel</td>
<td>36° 00' -0.0</td>
<td>*600</td>
<td>200</td>
<td>*100</td>
<td></td>
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</tbody>
</table>

* Value assumed
<table>
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<tr>
<th>Sample</th>
<th>Grain Size</th>
<th>Remarks</th>
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<tr>
<td>1</td>
<td>S. Bag 3</td>
<td>D.H. 18</td>
</tr>
<tr>
<td>2</td>
<td>L. Bag 4</td>
<td>B.P. 7</td>
</tr>
<tr>
<td>3</td>
<td>C. H. 2</td>
<td>D.H. 2</td>
</tr>
</tbody>
</table>

**Grain Size Distribution Curves**

- **Site:** Dam Site 2A
- **Representative Curves for Borrow Materials**
- **Plotter:** V.R.B.
- **Date:** 23-7-1946

**Figure 8-2**
DIRECT SHEAR CHARACTERISTICS

<table>
<thead>
<tr>
<th>SITE</th>
<th>CA</th>
<th>HOE, D.N. 23</th>
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<tbody>
<tr>
<td>SAMPLE</td>
<td>541C. 9</td>
<td>DEPTH 30-300 FT.</td>
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<tr>
<td>PLOTTED</td>
<td>A.R.A.</td>
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</tr>
<tr>
<td>REMARKS</td>
<td>Fine To Coarse Sand</td>
<td>Consolidated Submerged</td>
</tr>
</tbody>
</table>

C.R.O. P. 31-28

NORMAL LOAD (T) IN TONS PER SQ. FT.

PERCENT FINES BY WEIGHT

GRAVEL | SAND | SILT | CLAY

GRANULOMETRY GRAPH
Figure B-12. Design envelope for cases II and III

Figure B-13. Design envelope for cases IV, V, and VI
HAND COMPUTATIONS
APPENDIX C
STABILITY ANALYSIS

U.S. Army Corps of Engineers, Buffalo District
1776 Niagara Street
Buffalo, NY
THE SIMPLIFIED BISHOP'S METHOD ASSUMES THAT THE FORCES ACTING ON THE SIDES OF THE SLICE ARE HORIZONTAL. THEREFORE THEY HAVE ZERO RESULTANT IN THE VERTICAL DIRECTION AND CAN BE ELIMINATED BY SUMMING THE FORCES IN THE VERTICAL DIRECTION. THE FORCES actING on a typical slice are

\[ F = \frac{1}{1 + \sum_{i=1}^{n} W_i \sin \theta_i} \left[ \sum_{i=1}^{n} [\cos \theta_i (W_i - N_i \Delta x_i) + \Delta P_i] [1/M_i(\theta)] \right] \]

WHERE
- \( F \): FACTOR OF SAFETY
- \( C \): Cohesion
- \( W_i \): TOTAL WEIGHT
- \( \theta_i \): ANGLE OF WEDGE A-B
- \( i \): SLICE NUMBER
- \( n \): NUMBER OF SLICES
- \( M_i(\theta) \): NORMAL FORCE
- \( U_i \): WATER FORCES
- \( T \): RESISTING FORCES
- \( \phi \): ANGLE OF INTERNAL FRICTION (AT BASE OF SLICE)

NOTE: IN FOLLOWING TABLES THE TERM \((W_i - N_i \Delta x_i)\) IS ANNOTATED AS \( W'\) (THE EFFECTIVE WEIGHT) \( W_b \) AND \( W_s \) BOUNCING AND SATURATED WEIGHT RESPECTIVELY.
FOR ESTIMATE OF VALUATION THE ABOVE EQUATION IS
REDUCED TO TABULAR FORM. AS CAN BE NOTED, THE
FACTOR OF SAFETY APPEARS ON BOTH
SIDES OF THE EQUATION THEREFORE IT MUST
BE ASSUMED TO CALCULATE M / (b) AND THEN
THE CALCULATED FOS IS COMPARED WITH THE
ASSUMED VALUE. THIS IS DONE SEVERAL
TIMES UNTIL THE FOS IS "SEGRETEED".
THEN THEY CAN BE ADJUSTED TO OBTAIN
THE FOS.
<table>
<thead>
<tr>
<th>SLICE NO</th>
<th>AX (FT)</th>
<th>HT LS (FT)</th>
<th>HT RS (FT)</th>
<th>HT (FT)</th>
<th>AREA (FT^2)</th>
<th>Wl (KIPS)</th>
<th>SEL (IN)</th>
<th>TOT</th>
<th>( \Theta_L )</th>
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<tr>
<td>2</td>
<td>12</td>
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<td>6.5</td>
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<td>88</td>
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<td>72.4</td>
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\[ F_{2.0} = \frac{285.4}{136.3} = 2.09 \]
\[ F_{2.2} = \frac{288.5}{136.3} = 2.12 \]

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\[ F = \frac{20.0}{14.3} = 1.39 \]

\[ F = \frac{20.0}{13.7} = 1.44 \]

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*NOTE USED 5^2 = 25 INSTEAD OF 5^3 HAS NO EFFECT ON MELT.*

![c-q]
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$\theta_c$ is the slice angle in degrees, and $W_{2s+w_0}$ and $W_{2h+w_0}$ are weight components. $M_1(\theta)$ is the moment at angle $\theta$, and $W_{tan} = \frac{N_1(\theta)}{\theta}$ represents the tangential weight.

For $F = 10$, $F = \frac{124.6}{124.4} = 1.02$

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For \( F = 1.5 \):
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F = \frac{1280}{96} = 13.30
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For \( F = 1.0 \):
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For \( F = 1.25 \):
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F = \frac{124}{96} = 1.26
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128.8  \[81.5 \times 187.7 = 17.2 \]

C-14
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FR. F = 1.0 \quad F = \frac{167.2}{125.6} = 1.33

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**Diagram:**
- **CALC F**
- **ASSUMED F**

**Graph:**
- **F = 1.4**
- **Assumed F: 1.0 to 2.0**

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<td>F = 1.0</td>
<td>F = 1.0</td>
<td>F = 1.5</td>
</tr>
<tr>
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<td>22.6</td>
<td>77.2</td>
<td>28.5</td>
<td>8.3</td>
<td>11.4</td>
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<tr>
<td>2</td>
<td>22.0</td>
<td>77.2</td>
<td>27.8</td>
<td>9.6</td>
<td>12.2</td>
</tr>
<tr>
<td>3</td>
<td>24.2</td>
<td>77.2</td>
<td>27.0</td>
<td>1.0</td>
<td>12.4</td>
</tr>
<tr>
<td>4</td>
<td>23.7</td>
<td>77.2</td>
<td>27.5</td>
<td>1.01</td>
<td>11.1</td>
</tr>
<tr>
<td>5</td>
<td>25.0</td>
<td>48.8</td>
<td>42.2</td>
<td>1.03</td>
<td>1.08</td>
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<tr>
<td>6</td>
<td>7.4</td>
<td>48.8</td>
<td>10.7</td>
<td>1.02</td>
<td>1.04</td>
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<tr>
<td>7</td>
<td>7.8</td>
<td>48.8</td>
<td>11.8</td>
<td>0.97</td>
<td>1.06</td>
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<tr>
<td>8</td>
<td>4.4</td>
<td>48.8</td>
<td>5.7</td>
<td>0.95</td>
<td>0.91</td>
</tr>
<tr>
<td>9</td>
<td>4.4</td>
<td>48.8</td>
<td>4.1</td>
<td>0.95</td>
<td>0.91</td>
</tr>
<tr>
<td>10</td>
<td>-3.4</td>
<td>53.9</td>
<td>3.8</td>
<td>0.95</td>
<td>0.91</td>
</tr>
<tr>
<td>11</td>
<td>-5.8</td>
<td>45.2</td>
<td>2.4</td>
<td>0.95</td>
<td>0.91</td>
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<tr>
<td>12</td>
<td>-5.8</td>
<td>45.2</td>
<td>2.4</td>
<td>0.95</td>
<td>0.91</td>
</tr>
<tr>
<td>13</td>
<td>-3.5</td>
<td>48.8</td>
<td>1.0</td>
<td>0.95</td>
<td>0.91</td>
</tr>
</tbody>
</table>

\[ F = \frac{36.7}{25.1} = 1.44 \]
\[ 36.7 \times 1.44 = 52.71 \]
\[ \frac{52.71}{25.1} = 2.09 \]
\[ \frac{52.71}{36.7} = 1.42 \]

Assumed $F$  | Calcd $F/Asumed F$
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>1.24</td>
</tr>
<tr>
<td>1.5</td>
<td>0.91</td>
</tr>
<tr>
<td>2.0</td>
<td>0.72</td>
</tr>
</tbody>
</table>

\[ F = 1.37 \]

C - 18
STEADY SEEPAGE W/SHEWANCE POOL IS NOT APPLICABLE TO CHOKOMA DAM. THE CASE OF STEADY SEEPAGE WITH A POOL LOWER THAN MAX POOL YIELDS FAILURE SURFACES THAT DO NOT INTERSECT THE UPSTREAM SLOPE. SINCE THE EMBANKMENT IS 300 TIMES MORE ROMAN THAN THE IMPERVIOUS SECTION, CHANGES IN POOL &2 HAD A MINIMAL EFFECT ON THE DOWNSTREAM SLOPE AND THE CRITICAL FAILURE SURFACES.
COWICHAN BAY, BC

COMPUTER INPUT FILES
AND SAMPLE OUTPUT
APPENDIX D

U.S. Army Corps of Engineers, Buffalo District
1776 Elmgate Street
Buffalo, NY
COMPUTER INPUT FILES
AND SAMPLE OUTPUT

APPENDIX D

D1. This appendix contains a copy of all input data files used to compute the slope stability factor of safety in this analysis. Pages D2-6 are copies of the input instructions outlined in the User's Manual for WES Program 10009. Pages D7-16 are the input files and pages D17-49 are an example output for Case I Upstream Slope. The other output files are on file in the Geotechnical Section, Buffalo District.

D2. Input files are labeled according to the different cases set forth in EM 1110-2-1902.
<table>
<thead>
<tr>
<th>Variable Name</th>
<th>Definitions and Instructions for Executing Analyses</th>
<th>Reference No. Problem</th>
</tr>
</thead>
<tbody>
<tr>
<td>PROJE</td>
<td>Identification of your analysis</td>
<td></td>
</tr>
<tr>
<td>NGRID</td>
<td>NGRID = 1, Grid system used for calculation of a factor of safety</td>
<td></td>
</tr>
<tr>
<td></td>
<td>NGRID = 0, Slope analysis for a factor of safety</td>
<td></td>
</tr>
<tr>
<td></td>
<td>If NGRID = 0, then DEL, XBG, YBG, XEND,</td>
<td></td>
</tr>
<tr>
<td></td>
<td>YEND, TGLOWY, WL, and KOUTER = 0</td>
<td></td>
</tr>
<tr>
<td>DEL</td>
<td>Increment (feet) in grid system (positive number)</td>
<td></td>
</tr>
<tr>
<td>XBG</td>
<td>Abscissa of lower right grid point</td>
<td></td>
</tr>
<tr>
<td>YBG</td>
<td>Ordinate of lower right grid point</td>
<td></td>
</tr>
<tr>
<td>XEND</td>
<td>Abscissa of upper left grid point</td>
<td></td>
</tr>
<tr>
<td>YEND</td>
<td>Ordinate of upper left grid point</td>
<td></td>
</tr>
<tr>
<td>TGLOWY</td>
<td>Tangent elevation for base of circle</td>
<td></td>
</tr>
<tr>
<td>WL</td>
<td>For EOC, WL = groundwater el (may be fictitious value lower than the ground level)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>SD, WL = pool el (before drawdown)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Two-force polygon scheme, WL = drawdown pool el for one-force polygon scheme</td>
<td></td>
</tr>
<tr>
<td></td>
<td>SS, WL = tailwater el</td>
<td></td>
</tr>
<tr>
<td></td>
<td>PP, WL = 0</td>
<td></td>
</tr>
<tr>
<td>KOUTER</td>
<td>Number of embankment profile intersecting the groundwater level (WL); number of uppermost embankment profile for partial pool case</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Zero if WL is a fictitious value</td>
<td></td>
</tr>
<tr>
<td>KST</td>
<td>Number of soil types including firm base</td>
<td></td>
</tr>
<tr>
<td>KBASE</td>
<td>Number of last soil profile (firm base)</td>
<td></td>
</tr>
<tr>
<td>BETAU</td>
<td>Angle of earth forces acting on the sides of slices measured clockwise from the positive x axis</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Zero if downstream analysis only</td>
<td></td>
</tr>
<tr>
<td>BETAD</td>
<td>Angle of earth forces acting on the sides of slices measured counterclockwise from the negative x axis (US Analyses)</td>
<td></td>
</tr>
<tr>
<td>BMX</td>
<td>Selected maximum slice width; the program will locate slice boundaries at each break in the geometry of the embankment; additional boundaries are added so that the slices will have the selected BMX</td>
<td></td>
</tr>
<tr>
<td>NK B</td>
<td>Enter the number 1</td>
<td></td>
</tr>
<tr>
<td>SCIL</td>
<td>Name of soil type</td>
<td></td>
</tr>
<tr>
<td>KS</td>
<td>Number of soil type: KS = 1, first soil in the profile, KS = 2, second soil in profile, etc.</td>
<td></td>
</tr>
<tr>
<td>GAMMA(KS, 1)</td>
<td>Moist unit weight of soil, kips/cu ft</td>
<td></td>
</tr>
<tr>
<td>Variable Name</td>
<td>Definitions and Instructions for Executing Analyses</td>
<td>Reference</td>
</tr>
<tr>
<td>---------------</td>
<td>----------------------------------------------------</td>
<td>-----------</td>
</tr>
<tr>
<td>GAMAKS, 2)</td>
<td>Saturated unit weight of soil, kips/cu ft</td>
<td></td>
</tr>
<tr>
<td>QC2</td>
<td>Unit cohesion from the second segment of the Q strength envelope or equal to QC</td>
<td></td>
</tr>
<tr>
<td>QTC2</td>
<td>Tan θ from the second segment of the Q strength envelope or equal to QTC</td>
<td></td>
</tr>
<tr>
<td>QTG2</td>
<td>Unit cohesion from the first segment of the Q strength envelope, kips/sq ft</td>
<td></td>
</tr>
<tr>
<td>QC</td>
<td>Tan θ from the first segment of the Q strength envelope</td>
<td></td>
</tr>
<tr>
<td>QTG</td>
<td>Unit cohesion from the first segment of the Q strength envelope</td>
<td></td>
</tr>
<tr>
<td>QTC2</td>
<td>Unit cohesion from the first segment of the Q strength envelope, kips/sq ft</td>
<td></td>
</tr>
<tr>
<td>QTG</td>
<td>Unit cohesion from the first segment of the Q strength envelope</td>
<td></td>
</tr>
<tr>
<td>RC</td>
<td>Unit cohesion from the R strength envelope, kips/sq ft</td>
<td></td>
</tr>
<tr>
<td>RTG</td>
<td>Tangent θ from the R strength envelope</td>
<td></td>
</tr>
<tr>
<td>SC</td>
<td>Unit cohesion from the S strength envelope, kips/sq ft</td>
<td></td>
</tr>
<tr>
<td>STG</td>
<td>Tan θ from the S strength envelope</td>
<td></td>
</tr>
<tr>
<td>Note: Repeat data groups D and E (see Table 2) for each soil type except the firm base. Soil data are not entered for the firm base.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Variable Name</th>
<th>Definitions and Instructions for Executing Analyses</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>K</td>
<td>Number of embankment profile</td>
<td></td>
</tr>
<tr>
<td>KPS</td>
<td>Number of soil type immediately under above profile</td>
<td></td>
</tr>
<tr>
<td>NNI</td>
<td>Number of coordinate points required to define profile</td>
<td></td>
</tr>
<tr>
<td>XP, YP</td>
<td>The abscissa and ordinate of the first point on the uppermost embankment profile. Continue with as many points as needed to define the first embankment profile. Note: Repeat data groups F and G (see Table 2) to completely define all other profiles in the embankment from top to firm base</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Variable Name</th>
<th>Definitions and Instructions for Executing Analyses</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>NSLOP</td>
<td>Code number of slope analyzed</td>
<td></td>
</tr>
<tr>
<td>NSLOPE = 1, upstream slope</td>
<td></td>
<td></td>
</tr>
<tr>
<td>NSLOPE = 2, downstream slope</td>
<td></td>
<td></td>
</tr>
<tr>
<td>NCASE</td>
<td>Code number for case analyzed</td>
<td></td>
</tr>
<tr>
<td>NCASE = 1, end of construction</td>
<td></td>
<td></td>
</tr>
<tr>
<td>NCASE = 2, sudden drawdown</td>
<td></td>
<td></td>
</tr>
<tr>
<td>NCASE = 3, partial pool</td>
<td></td>
<td></td>
</tr>
<tr>
<td>NCASE = 4, steady seepage</td>
<td></td>
<td></td>
</tr>
<tr>
<td>NLEVEL</td>
<td>Code number for phreatic line in embankment</td>
<td></td>
</tr>
<tr>
<td>NLEVEL = 1, horizontal line</td>
<td></td>
<td></td>
</tr>
<tr>
<td>NLEVEL = 2, nonhorizontal line (steady seepage and sudden drawdown cases for a one force polygon scheme)</td>
<td></td>
<td></td>
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</table>
Table 1 (cont?)

<table>
<thead>
<tr>
<th>Variable Name</th>
<th>Definitions and Instructions for Executing Analyses</th>
<th>Fig.</th>
<th>Example No.</th>
<th>Problem</th>
</tr>
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<tbody>
<tr>
<td>NPORE</td>
<td>Code number for source of pore pressure&lt;br&gt;NPORE = 1, phreatic surface (horizontal or nonhorizontal)&lt;br&gt;NPORE = 2, other sources (generally a flow net)</td>
<td></td>
<td></td>
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</tr>
<tr>
<td>NBETA</td>
<td>Enter the number 1 or 3&lt;br&gt;NBETA = 1, fixed direction of all side earth forces (the direction will be given in data group C, Table 1)&lt;br&gt;NBETA = 3, vary direction of side earth forces (in embankment zone)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>EQCOE</td>
<td>Seismic coefficient for earthquake&lt;br&gt;EQCOE = 0, no earthquake effects</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>WLAFT</td>
<td>Drawdown pool elevation for sudden drawdown&lt;br&gt;WLAFT = 0, all other cases</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>KAFT</td>
<td>Code number of outer soil profile intersecting the water level (drawdown pool elevation, WLAFT) in the sudden drawdown case&lt;br&gt;KAFT = 0, all other cases</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>NW</td>
<td>Number of points required to define phreatic surface (NLEVEL = 2)&lt;br&gt;NW = 0, for horizontal phreatic surface (L. I. T. L. - 1)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>NWAFT</td>
<td>NWAFT = 1, sudden drawdown case after the drawdown with one force polygon scheme (the phreatic line must be nonhorizontal)&lt;br&gt;NWAFT = 0, all other cases</td>
<td></td>
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<tr>
<td>DESCRl</td>
<td>Name of case analyzed (30 spaces for input)&lt;br&gt;Note: Data group J (Table 2) describe the direction of the side earth forces by specifying zones of varying direction (NBETA = 3 must have been entered in data group H, Table 2).&lt;br&gt;OMIT data group J for all other cases.</td>
<td></td>
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<tr>
<td>ZONEXl</td>
<td>Abscissa of first boundary</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BEl</td>
<td>Direction of side earth forces in zone specified by ZONEXl</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ZONEX2</td>
<td>Abscissa of the second boundary</td>
<td></td>
<td></td>
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</tr>
<tr>
<td>BE2</td>
<td>Direction of side earth forces in zone specified by ZONEX2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>XW, YW</td>
<td>Abscissa and ordinate, respectively, of the first point to define a nonhorizontal phreatic surface. Always enter points from right to left. All other points required to fully define the nonhorizontal phreatic surface should follow. OMIT for a horizontal phreatic surface.&lt;br&gt;Note: Data group L (Table 2) is used only for a nongrid system calculation of a factor of safety.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>M</td>
<td>Identification number of a trial arc (positive nonzero number). Use only for a nongrid case (NGRID = 0).&lt;br&gt;OMIT for a grid system calculation of a factor of safety (NGRID = 1)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Variable Name</td>
<td>Definitions and Instructions for Executing Analyses</td>
<td>Reference</td>
<td></td>
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<tr>
<td>---------------</td>
<td>-----------------------------------------------------</td>
<td>-----------</td>
<td></td>
<td></td>
</tr>
<tr>
<td>XOT, YOT</td>
<td>Abscissa and ordinate, respectively, of the center of the trial arc ORT for a grid system calculation of a factor of safety (i.e., ORT = 1)</td>
<td></td>
<td></td>
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</tr>
<tr>
<td>XTOET, YTOET</td>
<td>Abscissa and ordinate of the exit point of the circle ORT or XTOET = 0 and YTOET = tangent elevation for the circle</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>WL</td>
<td>WL = groundwater level for the end of construction case ORT or pool elevation before drawdown for the sudden drawdown case with a two force polygon scheme (i.e., horizontal phreatic surface) ORT or drawdown pool elevation for the one force polygon scheme ORT or pool elevation or the number 9999 for the partial pool case (i.e., an actual pool elevation is entered, the analysis will be run for that pool elevation only; if the number 9999 is entered, the program will vary the pool level and search out the pool level which results in the lowest factor of safety for the particular circle being run)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ROUTER</td>
<td>Code number of outer soil profile intersecting the water level entered in WL above; or the code number of the uppermost embankment profile for the partial pool case; or if WL is a fictitious value, ROUTER = 0</td>
<td></td>
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</tr>
<tr>
<td>M</td>
<td>Enter the number -1</td>
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</tr>
<tr>
<td>Data Group</td>
<td>Line No. Series for Example Problems</td>
<td>Variables in Free-Field Input Data Files</td>
<td></td>
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<tr>
<td>------------</td>
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<td>-----------------------------------------</td>
<td></td>
<td></td>
</tr>
<tr>
<td>A</td>
<td>100-199</td>
<td>PROJE, NGRID, DEL, XBG, YBG, XEND, YEND, TGLOWY, WL, KOUTER</td>
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<td></td>
</tr>
<tr>
<td>B</td>
<td>200-299</td>
<td>KST, KBASE, BETAU, BETAD, BMAX, NKB</td>
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<td></td>
</tr>
<tr>
<td>C</td>
<td>300-399</td>
<td>SOIL</td>
<td></td>
<td></td>
</tr>
<tr>
<td>D**</td>
<td>400-499</td>
<td>KS, GAMA(KS, 1), GAMA(KS, 2), QC2, QTG2, QC, QTG, RC, RTG, SC, STG</td>
<td></td>
<td></td>
</tr>
<tr>
<td>E**</td>
<td></td>
<td>K, KPS, NNI</td>
<td></td>
<td></td>
</tr>
<tr>
<td>F</td>
<td>500-599</td>
<td>XP, YP</td>
<td></td>
<td></td>
</tr>
<tr>
<td>G</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>H**</td>
<td></td>
<td>NSLOP, NCASE, NLEVEL, NPOR, NBETA, EQCOE, WLAFT, KAFT, NW, NWAFT</td>
<td></td>
<td></td>
</tr>
<tr>
<td>I**</td>
<td>700-799</td>
<td>DESCHI</td>
<td></td>
<td></td>
</tr>
<tr>
<td>J</td>
<td>800-899</td>
<td>ZONEXI, BEI, ZONEX2, BE2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>K</td>
<td></td>
<td>XW, YW</td>
<td></td>
<td></td>
</tr>
<tr>
<td>L</td>
<td>1000</td>
<td>M, XOT, YOT, XTOET, YTOET, WL, KOUTER</td>
<td></td>
<td></td>
</tr>
<tr>
<td>M</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* Suggested line numbering for free field input is consistent in all illustrations for ease in locating input variables in the input instructions and example problems.

** Repeat data groups D and E for each soil type except the firm base. Soil data are not entered for the firm base.
OLD,GGKUS1

/LIST
100  ONONDAGA DAM STABILITY CASE I US
200  1 40 40 540 200 700 424 461 0
300  6 6 338 338 20 1
400  IMPERVIOUS ZONE
  402 1 .139 .145 0 .675 0 .675 0 .675 0 .675
  410  PERVIOUS FILL
  412 2 .143 .145 0 .727 0 .727 0 .727
  420  FLUVIAL OVERBANK
  422 3 .105 .105 0 .425 0 .425 0 .425
  430  DELTAIC DEPOSIT
  432 4 .119 .119 0 .7 0 .7 0 .7 0 .7
  440  LACUSTRINE
  442 5 .124 .124 0 .51 0 .51 0 .51 0 .51
  500  1 1 6
  502 -12 525 12 525 55 505 104 485 151 469 500 469
  510  2 2 8
  512 -168 464 -104 485 -53 505 -12 525 0 522 110 460 130 448 500 448
  520  3 3 7
  522 -500 464 -168 464 -122 460 110 460 118 455 130 448 500 448
  530  4 4 6
  533 -500 430 -140 430 0 455 118 455 130 448 500 448
  540  5 5 6
  542 -500 374 -260 373 -40 404 0 435 165 445 500 445
  550  6 6 2
  552 -500 345 500 345

/OULD,ADFF
/LIST
400 1,1,1,1,3,0,0,0,0,0,0
410 CASE I US
415 12 360 151 360
420 -1
OLD, GGD01

/LIST

100 ONGONDAGA DAM STABILITY CASE I DS
200 1 40 -240 580 -80 740 422 461 0
300 6 6 0 338 20 1
400 RANDOM PERVIOUS
402 1 .143 .145 0 .727 0 .727 0 .727
410 RIP RAP TOE
412 2 .105 .105 0 .839 0 .839 0 .839
420 RANDOM PERVIOUS
422 3 .143 .145 0 .727 0 .727 0 .727
430 FLUVIAL OVERBANK
432 4 .105 .105 0 .488 0 .488 0 .488 0 .488
440 DELTAIC
442 5 .119 .119 0 .7 0 .7 0 .7 0 .7
500 1 1 7
502 -181 464 -168 464 -104 485 -53 505 -12 525 12 525 151 469
510 2 2 4
512 -181 464 -168 464 -104 485 -53 505
520 3 3 4
522 -181 464 -156 446 -144 446 -53 505
530 4 4 6
532 -500 464 -181 464 -156 446 -144 446 -122 460 151 460
540 5 5 4
542 -500 440 -140 440 0 455 151 455
550 6 6 4
552 -500 374 -260 373 -40 404 500 404
/

OLD, ADF081

/LIST

400 2 1 1 1 3 0 0 0 0 0
410 CASE I DS
415 -168 360 -12 360
420 -1
/
ONONDAGA DAM STABILITY CASE II SDD FROM MAX POOL

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/LIST

/OLD,ADFS02

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### ONONDAGA DAM STABILITY CASE III SDD FROM SPILLWAY EL

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200 1 40 40 540 200 700 434 0 1
300 6 6 338 338 20 1
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402 1 .139 .145 0 .675 0 .675 0 .675 0 .675
410 FERVIOUS FILL
412 2 .143 .145 0 .727 0 .727 0 .727
420 FLUVIAL OVERBANK
422 3 .105 .105 0 .425 0 .425 0 .425 0 .425
430 DELTAIC DEPOSIT
432 4 .119 .119 0 .7 0 .7 0 .7 0 .7
440 LACUSTINE
442 5 .124 .124 0 .51 0 .51 0 .51
500 1 1 6
502 -12 525 12 525 53 505 104 485 151 469 500 469
510 2 2 8
512 -168 464 -104 485 -53 505 -12 525 0 522 110 460 130 448 500 448
520 3 3 7
522 -500 464 -168 464 -122 460 110 460 118 455 130 448 500 448
530 4 4 6
532 -500 440 -140 440 0 455 118 455 130 448 500 448
540 5 5 6
542 -500 374 -260 373 -40 404 0 435 165 445 500 445
550 6 6 2
552 -500 345 500 345

/OLD, ADFFP

/LIST
400 1 3 1 1 3 0 0 0 0 0 0
410 CASE IV PP (SS)
415 12 360 151 360
420 -1
OLD,GGKSS1

/List

100 ONONDAGA DAM STABILITY CASE V SS MAX POOL DS
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300 6 6 0 338 20 1
400 RANDOM PERVIOUS
402 1 .143 .145 0 .727 0 .727 0 .727
410 RIP RAP TOE
412 2 .105 .105 0 .839 0 .839 0 .839
420 RANDOM PERVIOUS
422 3 .143 .145 0 .727 0 .727 0 .727
430 FLUVIAL OVERBANK
432 4 .105 .105 0 .488 0 .488 0 .488
440 DELTAIC
442 5 .119 .119 0 .7 0 .7 0 .7
500 1 1 7
502 -181 464 -168 464 -104 485 -53 505 -12 525 12 525 151 469
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542 -500 440 -140 440 0 455 151 455
550 6 6 4
552 -500 374 -260 373 -40 404 500 404
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OLD,ADFSS1

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OLD,GGKEQIB

/LIST

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200 1 40 -160 620 -160 620 442 461 0
300 6 6 0 338 20 1
400 RANDOM PERVIOUS
402 1 .143 .145 0 .727 0 .727 0 .727 0 .727
410 RIP RAP TOE
412 2 .105 .105 0 .839 0 .839 0 .839
420 RANDOM PERVIOUS
422 3 .143 .145 0 .727 0 .727 0 .727 0 .727
430 FLUVIAL OVERBANK
432 4 .105 .105 0 .488 0 .488 0 .488 0 .488
440 DELTAIC
442 5 .119 .119 0 .7 0 .7 0 .7
500 1 1 7
502 -181 464 -168 464 -104 485 -53 505 -12 525 12 525 151 469
510 2 2 4
512 -181 464 -168 464 -104 485 -53 505
520 3 3 4
522 -181 464 -156 446 -144 446 -53 505
530 4 4 6
532 -500 446 -181 464 -156 446 -144 446 -122 460 151 460
540 5 5 4
542 -500 440 -140 440 0 455 151 455
550 6 6 4
552 -500 374 -260 373 -40 404 500 404
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OLD,ADFEQIB

/LIST

400 2 1 1 1 3 .05 0 0 0 0
410 CASE I DB EQ CASE
415 -168 360 -12 360
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/LIST
100 ONONDAGA DAM STABILITY CASE IV PP (SS) EQ LOAD
200 1 40 120 620 120 620 464 0 1
300 6 6 338 338 20 1
400 IMPERVIOUS ZONE
402 1 .139 .145 0 .675 0 .675 0 .675 0 .675
410 PERVIOUS FILL
412 2 .143 .145 0 .727 0 .727 0 .727 0 .727
420 FLUVIAL OVERBANK
422 3 .105 .105 0 .425 0 .425 0 .425 0 .425
430 DELTAIC DEPOSIT
432 4 .119 .119 0 .7 0 .7 0 .7 0 .7
440 LACUSTRINE
442 5 .124 .124 0 .51 0 .51 0 .51 0 .51
500 1 1 6
502 -12 525 12 525 53 505 104 485 151 469 500 469
510 2 2 8
512 -168 464 -104 485 -53 505 -12 525 0 522 110 460 130 448 500 448
520 3 3 7
522 -500 464 -169 464 -122 460 110 460 118 455 130 448 500 448
530 4 4 6
532 -500 440 -140 440 0 455 118 455 130 448 500 448
540 5 5 6
542 -500 374 -260 373 -40 404 0 435 165 445 500 445
550 6 6 2
552 -500 345 500 345

/OLD,ADFFPEQ
OLD, GGKSSEQ

LIST
100 ONONDAGA DAM STABILITY CASE V SS MAX POOL DS ED LOAD
200 1 40 -120 580 -120 580 442 475 1
200 6 6 0 338 20 1
400 RANDOM PREVIOUS
402 1 .143 .145 0 .727 0 .727 0 .727
410 RIP RAP TOE
412 2 .105 .105 0 .839 0 .839 0 .839
420 RANDOM PREVIOUS
422 3 .143 .145 0 .727 0 .727 0 .727
430 FLUVIAL OVERBANK
432 4 .105 .105 0 .488 0 .488 0 .488
440 DELTAIC
442 5 .119 .119 0 .7 0 .7 0 .7
500 1 1 7
502 -181 464 -168 464 -104 485 -53 505 -12 525 12 525 151 469
510 2 2 4
512 -181 464 -168 464 -104 485 -53 505
520 3 3 4
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530 4 4 6
532 -500 464 -181 464 -156 446 -144 446 -122 460 151 460
540 5 5 4
542 -500 440 -140 440 0 455 151 455
550 6 6 4
552 -500 374 -260 373 -40 404 500 404

OLD, ADFSSEQ

LIST
400 2 4 2 1 3 .05 0 0 5 0
410 CASE V SS MAX POOL DS ED LOAD
415 -168 360 -12 360
417 -500 475 12 475 12 515 17 520 500 520
420 -1
GET, CORPS/ON=CECEL.P
/BEGIN, CORPS, 10009

***************************************
* CORPS PROGRAM 1 10009 *
* VERSION 0 13/10/01 *
***************************************

INPUT, NAME OF BASIC DATA FILE
? DATA
PERM FILE GUSU1 COPIED TO LOCAL FILE TAPE1
PROJECT: OOKINAGA DAM STABILITY CASE I US
INPUT, NAME OF THE ARC DATA FILE
? DATA
PERM FILE ADHY COPIED TO LOCAL FILE TAPE1

### ARC 1 ###
ARC NO.= 1 CENTER(X,Y)= 00.00, 540.00 EXIT(X,Y)= 135.57, 474.25
RAd.= 116.00 LOWEST Y= 424.00 SLICE NO.= 19 ML= 441.00
CASE I US
FS=  7.942, E.C.* -.00

### ARC 2 ###
ARC NO.= 2 CENTER(X,Y)= 80.00, 540.00 EXIT(X,Y)= 171.73, 469.00
RAd.= 116.00 LOWEST Y= 424.00 SLICE NO.= 19 ML= 441.00
CASE I US
FS=  3.215, E.C.* -.01

### ARC 3 ###
ARC NO.= 3 CENTER(X,Y)= 120.00, 540.00 EXIT(X,Y)= 211.73, 469.00
RAd.= 116.00 LOWEST Y= 424.00 SLICE NO.= 21 ML= 441.00
CASE I US
FS=  2.005, E.C.* .01

### ARC 4 ###
ARC NO.= 4 CENTER(X,Y)= 160.00, 540.00 EXIT(X,Y)= 251.73, 469.00
RAd.= 116.00 LOWEST Y= 424.00 SLICE NO.= 20 ML= 441.00
CASE I US
FS=  3.765, E.C.* -.02
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FS = 0.340, E.C. = -0.00

FS = 6.700, E.C. = -0.00

FS = 3.095, E.C. = -0.05

FS = 2.400, E.C. = 0.07

FS = 2.942, E.C. = 0.05

FS = 3.549, E.C. = -0.04
ARC

ARC NO. 11 CENTER(X,Y)= 40.00, 620.00 EXIT(X,Y)= 164.96, 469.00
RAN= 196.00 LOWEST Y= 424.00 SLICE NO.= 23 ML= 461.00
CASE I US
FS= 7.153, E.C.= -.03

ARC

ARC NO. 12 CENTER(X,Y)= 80.00, 620.00 EXIT(X,Y)= 204.96, 469.00
RAN= 196.00 LOWEST Y= 424.00 SLICE NO.= 22 ML= 461.00
CASE I US
FS= 3.408, E.C.= .04

ARC

ARC NO. 13 CENTER(X,Y)= 120.00, 620.00 EXIT(X,Y)= 244.96, 469.00
RAN= 196.00 LOWEST Y= 424.00 SLICE NO.= 24 ML= 461.00
CASE I US
FS= 2.707, E.C.= .00

ARC

ARC NO. 14 CENTER(X,Y)= 160.00, 620.00 EXIT(X,Y)= 284.96, 469.00
RAN= 196.00 LOWEST Y= 424.00 SLICE NO.= 24 ML= 461.00
CASE I US
FS= 2.666, E.C.= .00

ARC

ARC NO. 15 CENTER(X,Y)= 200.00, 620.00 EXIT(X,Y)= 324.96, 469.00
RAN= 196.00 LOWEST Y= 424.00 SLICE NO.= 22 ML= 461.00
CASE I US
FS= 4.229, E.C.= -.01

ARC

ARC NO. 16 CENTER(X,Y)= 40.00, 660.00 EXIT(X,Y)= 178.62, 469.00
RAN= 236.00 LOWEST Y= 424.00 SLICE NO.= 25 ML= 461.00
CASE I US
FS= 7.009, E.C.= -.01
**Case 1**

- **ARC 17**
  - Center: $(X,Y) = 100.00, 660.00$
  - Exit: $(X,Y) = 210.62, 469.00$
  - Radius: 236.00
  - Lowest $Y$: 424.00
  - Slice No.: 24
  - ML: 461.00
  - FS: 3.849, E.C.: 0.00

- **ARC 18**
  - Center: $(X,Y) = 120.00, 660.00$
  - Exit: $(X,Y) = 290.62, 469.00$
  - Radius: 236.00
  - Lowest $Y$: 424.00
  - Slice No.: 26
  - ML: 461.00
  - FS: 2.955, E.C.: 0.08

- **ARC 19**
  - Center: $(X,Y) = 160.00, 660.00$
  - Exit: $(X,Y) = 290.62, 469.00$
  - Radius: 236.00
  - Lowest $Y$: 424.00
  - Slice No.: 25
  - ML: 461.00
  - FS: 2.707, E.C.: 0.00

- **ARC 20**
  - Center: $(X,Y) = 200.00, 660.00$
  - Exit: $(X,Y) = 330.62, 469.00$
  - Radius: 236.00
  - Lowest $Y$: 424.00
  - Slice No.: 25
  - ML: 461.00
  - FS: 3.444, E.C.: 0.04

Analyses above are stored in local file tape

- **ARC 21**
  - Center: $(X,Y) = 40.00, 700.00$
  - Exit: $(X,Y) = 191.05, 469.00$
  - Radius: 276.00
  - Lowest $Y$: 424.00
  - Slice No.: 26
  - ML: 461.00
  - FS: 8.745, E.C.: -0.05

- **ARC 22**
  - Center: $(X,Y) = 80.00, 700.00$
  - Exit: $(X,Y) = 231.05, 469.00$
  - Radius: 276.00
  - Lowest $Y$: 424.00
  - Slice No.: 27
  - ML: 461.00
  - FS: 4.279, E.C.: 0.01
* * * ARC 23 * *
ARC NO.= 23 CENTER(X,Y)= 120.00, 700.00 EXIT(X,Y)= 271.05, 469.00
RAB.= 274.00 LOWEST Y= 424.00 SLICE NO.= 26 ML= 461.00
CASE I US
FS= 3.222, E.C.= .07

* * * ARC 24 * *
ARC NO.= 24 CENTER(X,Y)= 160.00, 700.00 EXIT(X,Y)= 311.05, 469.00
RAB.= 274.00 LOWEST Y= 424.00 SLICE NO.= 26 ML= 461.00
CASE I US
FS= 2.878, E.C.= -.09

* * * ARC 25 * *
ARC NO.= 25 CENTER(X,Y)= 200.00, 700.00 EXIT(X,Y)= 351.05, 469.00
RAB.= 274.00 LOWEST Y= 424.00 SLICE NO.= 27 ML= 461.00
CASE I US
FS= 3.163, E.C.= -.01

LIST OF ARC/FS FOR LOWEST Y= 424.00, NO. OF ARC/FS
11/ 7.153, 12/ 3.430, 13/ 2.707, 14/ 2.664, 15/ 4.220, 16/ 7.840, 17/ 3.849, 18/ 2.925, 19/ 2.767, 20/ 3.444,
21/ 8.745, 22/ 4.293, 23/ 3.222, 24/ 2.878, 25/ 3.163,
THE MIN FS IS 2.480, AT CENTER NO. 8

ANALYSES ABOVE ARE STORED IN LOCAL FILE TAPERS
CODE: 1=NEW ARC FILE. 2=RENAME AN ARC. 3=MORIFY GRID. 4=STOP
? 3
READ NEW GRID, 0 VAR. ;
REL, X0G, Y0G, XEND, YEND, TOLNW, ML, XOUTER
? 40 40 500 200 700 434 461 0

* * * ARC 1 * *
ARC NO.= 1 CENTER(X,Y)= 40.00, 540.00 EXIT(X,Y)= 125.70, 477.61
RAB.= 186.00 LOWEST Y= 434.00 SLICE NO.= 17 ML= 461.00
CASE I US
FS= 7.272, E.C.= -.03
**ARC 2**

ARC NO.= 2  
CENTER(X,Y)= 80.00, 540.00  
EXIT(X,Y)= 158.71, 469.00  
RAB.= 106.00  
LOWEST Y= 434.00  
SLICE NO.= 10  
UL= 661.00  
CASE I US

FS= 3.154, E.C.= -.04

**ARC 3**

ARC NO.= 3  
CENTER(X,Y)= 120.00, 540.00  
EXIT(X,Y)= 198.71, 469.00  
RAB.= 106.00  
LOWEST Y= 434.00  
SLICE NO.= 10  
UL= 661.00  
CASE I US

FS= 2.643, E.C.= -.01

**ARC 4**

ARC NO.= 4  
CENTER(X,Y)= 160.00, 540.00  
EXIT(X,Y)= 238.71, 469.00  
RAB.= 106.00  
LOWEST Y= 434.00  
SLICE NO.= 17  
UL= 661.00  
CASE I US

FS= 3.815, E.C.= -.03

**ARC 5**

ARC NO.= 5  
CENTER(X,Y)= 200.00, 540.00  
EXIT(X,Y)= 278.71, 469.00  
RAB.= 106.00  
LOWEST Y= 434.00  
SLICE NO.= 13  
UL= 661.00  
CASE I US

FS= 12.342, E.C.= -.01

**ARC 6**

ARC NO.= 6  
CENTER(X,Y)= 40.00, 500.00  
EXIT(X,Y)= 129.31, 472.98  
RAB.= 146.00  
LOWEST Y= 434.00  
SLICE NO.= 19  
UL= 661.00  
CASE I US

FS= 6.483, E.C.= -.06

**ARC 7**

ARC NO.= 7  
CENTER(X,Y)= 80.00, 500.00  
EXIT(X,Y)= 174.04, 469.00  
RAB.= 146.00  
LOWEST Y= 434.00  
SLICE NO.= 19  
UL= 661.00  
CASE I US

FS= 2.715, E.C.= .02
# **ARC 0**

**ARC NO.**: 8  
**CENTER(X,Y)**: 120.00, 500.00  
**EXIT(X,Y)**: 214.84, 449.00  
**RAB.**: 146.00  
**LOWEST Y**: 434.00  
**SLICE NO.**: 21  
**ML.**: 461.00  
**CASE I US**

**FS**: 2.342, **E.C.**: .00

# **ARC 1**

**ARC NO.**: 9  
**CENTER(X,Y)**: 160.00, 500.00  
**EXIT(X,Y)**: 254.84, 449.00  
**RAB.**: 146.00  
**LOWEST Y**: 434.00  
**SLICE NO.**: 19  
**ML.**: 461.00  
**CASE I US**

**FS**: 3.047, **E.C.**: .02

# **ARC 2**

**ARC NO.**: 10  
**CENTER(X,Y)**: 200.00, 500.00  
**EXIT(X,Y)**: 294.84, 469.00  
**RAB.**: 146.00  
**LOWEST Y**: 434.00  
**SLICE NO.**: 15  
**ML.**: 461.00  
**CASE I US**

**FS**: 4.657, **E.C.**: -.01

# **ARC 3**

**ARC NO.**: 11  
**CENTER(X,Y)**: 40.00, 620.00  
**EXIT(X,Y)**: 149.37, 449.93  
**RAB.**: 186.00  
**LOWEST Y**: 434.00  
**SLICE NO.**: 19  
**ML.**: 461.00  
**CASE I US**

**FS**: 6.591, **E.C.**: -.10

# **ARC 4**

**ARC NO.**: 12  
**CENTER(X,Y)**: 80.00, 620.00  
**EXIT(X,Y)**: 186.00, 449.90  
**RAB.**: 186.00  
**LOWEST Y**: 434.00  
**SLICE NO.**: 20  
**ML.**: 461.00  
**CASE I US**

**FS**: 3.310, **E.C.**: .01

# **ARC 5**

**ARC NO.**: 13  
**CENTER(X,Y)**: 120.00, 620.00  
**EXIT(X,Y)**: 226.60, 469.00  
**RAB.**: 186.00  
**LOWEST Y**: 434.00  
**SLICE NO.**: 21  
**ML.**: 461.00  
**CASE I US**

**FS**: 2.499, **E.C.**: .01
No text content is provided in this image.
**ARC 20**

ARC NO.: 20  CENTER(X,Y): 200.00, 640.00  EXIT(X,Y): 320.00, 440.00  
RAB.: 226.00  LOWEST Y: 434.00  SLICE NO.: 23  ML.: 441.00  
CASE I US  
FS.: 3.131, E.C.: -.09

**ARC 21**

ARC NO.: 21  CENTER(X,Y): 40.00, 700.00  EXIT(X,Y): 171.89, 440.00  
RAB.: 266.00  LOWEST Y: 434.00  SLICE NO.: 24  ML.: 441.00  
CASE I US  
FS.: 7.798, E.C.: -.04

**ARC 22**

ARC NO.: 22  CENTER(X,Y): 80.00, 700.00  EXIT(X,Y): 211.89, 440.00  
RAB.: 266.00  LOWEST Y: 434.00  SLICE NO.: 23  ML.: 441.00  
CASE I US  
FS.: 4.016, E.C.: .01

**ARC 23**

ARC NO.: 23  CENTER(X,Y): 120.00, 700.00  EXIT(X,Y): 251.89, 440.00  
RAB.: 266.00  LOWEST Y: 434.00  SLICE NO.: 25  ML.: 441.00  
CASE I US  
FS.: 3.111, E.C.: .00

**ARC 24**

ARC NO.: 24  CENTER(X,Y): 160.00, 700.00  EXIT(X,Y): 291.89, 440.00  
RAB.: 266.00  LOWEST Y: 434.00  SLICE NO.: 25  ML.: 441.00  
CASE I US  
FS.: 2.742, E.C.: .00

**ARC 25**

ARC NO.: 25  CENTER(X,Y): 200.00, 700.00  EXIT(X,Y): 331.89, 440.00  
RAB.: 266.00  LOWEST Y: 434.00  SLICE NO.: 25  ML.: 441.00  
CASE I US  
FS.: 3.279, E.C.: .02
LIST OF ARC/FS FOR LOWEST T = 434.00, NO. OF ARC/FS
10/ 6.691, 11/ 3.310, 12/ 2.658, 13/ 4.786, 14/ 7.115, 15/ 3.653, 16/ 2.767, 17/ 2.621, 18/ 3.931,

THE MIN FS IS 2.362, AT CENTER NO. 8

ANALYSES ABOVE ARE STORED IN LOCAL FILE TAPE12
CODE: 1=NEW ARC FILE. 2=RERUN AN ARC. 3=MODIFY GRID. 4=STOP

? 3
READ NEW GRID, B VAR. 1
REL, SRC, YRC, XEND, XORD, YL, XL, ROUTER
? 4 40 540 200 700 444 461 0

1

ARC NO.= 1 CENTER(X,Y)= 40.00, 540.00 EXIT(X,Y)= 115.73, 401.01
RAB.= 96.00 LOWEST T= 444.00 SLICE NO.= 15 ML= 461.00
CASE 1 US
FS= 4.483, E.C.= -.02

2

ARC NO.= 2 CENTER(X,Y)= 80.00, 540.00 EXIT(X,Y)= 146.32, 470.59
RAB.= 96.00 LOWEST T= 444.00 SLICE NO.= 14 ML= 461.00
CASE 1 US
FS= 2.155, E.C.= -.01

3

ARC NO.= 3 CENTER(X,Y)= 120.00, 540.00 EXIT(X,Y)= 184.61, 449.00
RAB.= 96.00 LOWEST T= 444.00 SLICE NO.= 14 ML= 461.00
CASE 1 US
FS= 2.900, E.C.= -.02

4

ARC NO.= 4 CENTER(X,Y)= 160.00, 540.00 EXIT(X,Y)= 224.61, 449.00
RAB.= 96.00 LOWEST T= 444.00 SLICE NO.= 16 ML= 461.00
CASE 1 US
FS= 4.062, E.C.= -.00
* * * ARC 5 * *
ARC NO. = 5 CENTER (X, Y) = 200.00, 540.00 EXIT (X, Y) = 264.61, 469.00
RAH. = 96.00 LOWEST T = 444.00 SLICE NO. = 10 ML = 461.00
CASE I US
FS = 25.671, E.C. = -.04

* * * ARC 6 * *
ARC NO. = 6 CENTER (X, Y) = 40.00, 580.00 EXIT (X, Y) = 128.44, 476.60
RAH. = 136.00 LOWEST T = 444.00 SLICE NO. = 17 ML = 461.00
CASE I US
FS = 6.269, E.C. = -.01

* * * ARC 7 * *
ARC NO. = 7 CENTER (X, Y) = 80.00, 580.00 EXIT (X, Y) = 158.58, 469.00
RAH. = 136.00 LOWEST T = 444.00 SLICE NO. = 17 ML = 461.00
CASE I US
FS = 3.078, E.C. = .00

* * * ARC 8 * *
ARC NO. = 8 CENTER (X, Y) = 120.00, 580.00 EXIT (X, Y) = 198.58, 469.00
RAH. = 136.00 LOWEST T = 444.00 SLICE NO. = 19 ML = 461.00
CASE I US
FS = 2.507, E.C. = .01

* * * ARC 9 * *
ARC NO. = 9 CENTER (X, Y) = 160.00, 580.00 EXIT (X, Y) = 238.58, 469.00
RAH. = 136.00 LOWEST T = 444.00 SLICE NO. = 19 ML = 461.00
CASE I US
FS = 3.309, E.C. = -.01

* * * ARC 10 * *
ARC NO. = 10 CENTER (X, Y) = 200.00, 580.00 EXIT (X, Y) = 278.58, 469.00
RAH. = 136.00 LOWEST T = 444.00 SLICE NO. = 12 ML = 461.00
CASE I US
FS = 10.308, E.C. = -.03
ANALYSES ABOVE ARE STORED IN LOCAL FILE TAPE11
ARC 11
ARC NO.: 11 CENTER(X,Y) = 40.00, 400.00 EXIT(X,Y) = 127.42, 475.55
RAA = 176.00 LOWEST Y = 444.00 SLICE NO. = 18 LR = 461.00
CASE I US

FS = 6.719, E.C. = -.00

ARC 12
ARC NO.: 12 CENTER(X,Y) = 80.00, 420.00 EXIT(X,Y) = 120.42, 469.00
RAA = 176.00 LOWEST Y = 444.00 SLICE NO. = 18 LR = 461.00
CASE I US

FS = 3.443, E.C. = .04

ARC 13
ARC NO.: 13 CENTER(X,Y) = 120.00, 620.00 EXIT(X,Y) = 210.42, 461.00
RAA = 176.00 LOWEST Y = 444.00 SLICE NO. = 19 LR = 461.00
CASE I US

FS = 2.557, E.C. = .03

ARC 14
ARC NO.: 14 CENTER(X,Y) = 160.00, 620.00 EXIT(X,Y) = 250.42, 469.00
RAA = 176.00 LOWEST Y = 444.00 SLICE NO. = 19 LR = 461.00
CASE I US

FS = 2.994, E.C. = -.01

ARC 15
ARC NO.: 15 CENTER(X,Y) = 200.00, 420.00 EXIT(X,Y) = 290.42, 469.00
RAA = 176.00 LOWEST Y = 444.00 SLICE NO. = 13 LR = 461.00
CASE I US

FS = 6.514, E.C. = -.00

ARC 16
ARC NO.: 16 CENTER(X,Y) = 40.00, 460.00 EXIT(X,Y) = 144.78, 471.12
RAA = 216.00 LOWEST Y = 444.00 SLICE NO. = 19 LR = 461.00
CASE I US

FS = 7.320, E.C. = -.01
* * * ARC 17 * * *
ARC NO. = 17 CENTER(X,Y) = 00.00, 660.00 EXIT(X,Y) = 100.07, 469.00
RAD. = 216.00 LOWEST Y = 444.00 SLICE NO. = 19 ML. = 461.00
CASE I US
FS = 3.734, E.C.* .02

* * * ARC 18 * * *
ARC NO. = 18 CENTER(X,Y) = 120.00, 660.00 EXIT(X,Y) = 220.07, 469.00
RAD. = 216.00 LOWEST Y = 444.00 SLICE NO. = 20 ML. = 461.00
CASE I US
FS = 2.789, E.C.* .02

* * * ARC 19 * * *
ARC NO. = 19 CENTER(X,Y) = 140.00, 660.00 EXIT(X,Y) = 240.07, 469.00
RAD. = 216.00 LOWEST Y = 444.00 SLICE NO. = 22 ML. = 461.00
CASE I US
FS = 2.716, E.C.* .08

* * * ARC 20 * * *
ARC NO. = 20 CENTER(X,Y) = 200.00, 660.00 EXIT(X,Y) = 300.07, 469.00
RAD. = 216.00 LOWEST Y = 444.00 SLICE NO. = 19 ML. = 461.00
CASE I US
FS = 4.089, E.C.* -.02

* * * ARC 21 * * *
ARC NO. = 21 CENTER(X,Y) = 40.00, 700.00 EXIT(X,Y) = 130.41, 469.13
RAD. = 256.00 LOWEST Y = 444.00 SLICE NO. = 20 ML. = 461.00
CASE I US
FS = 7.975, E.C.* -.02

* * * ARC 22 * * *
ARC NO. = 22 CENTER(X,Y) = 90.00, 700.00 EXIT(X,Y) = 190.34, 469.00
RAD. = 256.00 LOWEST Y = 444.00 SLICE NO. = 22 ML. = 461.00
CASE I US
FS = 4.188, E.C.* .03
* * * ARC  22 * * *
ARC NO.=  23  CENTER(X,Y)=  120.00, 700.00  EXIT(X,Y)=  230.34, 449.00
RAB.=  234.00  LOWEST Y=  444.00  SLICE NO.=  20  ML=  461.00
CASE I US
FS=  3.044,  E.C.=  .02

* * * ARC  24 * * *
ARC NO.=  24  CENTER(X,Y)=  160.00, 700.00  EXIT(X,Y)=  270.34, 449.00
RAB.=  234.00  LOWEST Y=  444.00  SLICE NO.=  22  ML=  461.00
CASE I US
FS=  2.764,  E.C.=  .04

* * * ARC  25 * * *
ARC NO.=  25  CENTER(X,Y)=  200.00, 700.00  EXIT(X,Y)=  310.34, 449.00
RAB.=  234.00  LOWEST Y=  444.00  SLICE NO.=  22  ML=  461.00
CASE I US
FS=  3.941,  E.C.=  .02

LIST OF ARC/FS FOR LOWEST Y= 444.00, No. OF ARC/FS
11/ 6.710, 12/ 3.443, 13/ 2.557, 14/ 2.894, 15/ 6.514, 16/ 7.320, 17/ 3.734, 18/ 2.789, 19/ 2.716, 20/ 6.800,
21/ 7.975, 22/ 4.188, 23/ 3.044, 24/ 2.764, 25/ 3.941,
THE LOWEST FS IS 2.507, AT CENTER NO. B

ANALYSES ABOVE ARE STORED IN LOCAL FILE TAPE12
CORE: 1:NEW ARC FILE.  2:RENAME ARC.  3:MODIFY GRID.  4:STOP
? 3
READ NEW GRID, 0 VAR.:
REL, XCR, YCR, XDB, YDB, TDLNY, HL, ROUTER
? 40 40 540 200 700 454 461 0

* * * ARC  1 * * *
ARC NO.=  1  CENTER(X,Y)=  40.00, 540.00  EXIT(X,Y)=  105.64, 464.44
RAB.=  86.00  LOWEST Y=  454.00  SLICE NO.=  12  ML=  461.00
CASE I US
FS=  5.817,  E.C.=  -.00
THE RESULTS ARE NOT CORRECT—ERROR OF CLOSURE NOT CONVERGED DUE TO THE INTER SLICE TENSILE FORCE EXISTS.
* * * ARC 2 * * *
ARC NO.= 2 CENTER(X,Y)= 80.00, 540.00 EXIT(X,Y)= 125.48, 474.28
RAI.= 86.00 LOWEST Y= 454.00 SLICE NO.= 13 ML= 461.00
CASE I US
FS= 2.758, E.C.= -.00

* * * ARC 3 * * *
ARC NO.= 3 CENTER(X,Y)= 120.00, 540.00 EXIT(X,Y)= 168.53, 449.00
RAI.= 86.00 LOWEST Y= 454.00 SLICE NO.= 12 ML= 461.00
CASE I US
FS= 2.584, E.C.= -.01

* * * ARC 4 * * *
ARC NO.= 4 CENTER(X,Y)= 160.00, 540.00 EXIT(X,Y)= 208.53, 449.00
RAI.= 86.00 LOWEST Y= 454.00 SLICE NO.= 7 ML= 461.00
CASE I US
FS= 4.163, E.C.= -.04

* * * ARC 5 * * *
ARC NO.= 5 CENTER(X,Y)= 200.00, 540.00 EXIT(X,Y)= 248.53, 449.00
RAI.= 86.00 LOWEST Y= 454.00 SLICE NO.= 5 ML= 461.00
CASE I US
THE RESULTS ARE NOT CORRECT--ERROR OF CLOSURE NOT CONVERGED DUE TO THE INTER SLICE TENSILE FORCE EXISTS.
FS= 65.526, E.C.= -1.77
ANALYSES ABOVE ARE STORED IN LOCAL FILE TAPE11

* * * ARC 6 * * *
ARC NO.= 6 CENTER(X,Y)= 40.00, 580.00 EXIT(X,Y)= 117.28, 480.48
RAI.= 126.00 LOWEST Y= 454.00 SLICE NO.= 16 ML= 461.00
CASE I US
FS= 5.754, E.C.= -.01

* * * ARC 7 * * *
ARC NO.= 7 CENTER(X,Y)= 80.00, 580.00 EXIT(X,Y)= 143.71, 471.41
RAI.= 126.00 LOWEST Y= 454.00 SLICE NO.= 14 ML= 461.00
CASE I US
FS= 2.650, E.C.= -.01
*** ARC 8 ***
ARC NO.= 8 CENTER(X,Y)= 120.00, 500.00 EXIT(X,Y)= 179.42, 449.00
RAD.= 126.00 LOWEST Y= 454.00 SLICE NO.= 15 NL= 441.00
CASE 1 US
FS= 2.247, E.C.= .05

*** ARC 9 ***
ARC NO.= 9 CENTER(X,Y)= 160.00, 500.00 EXIT(X,Y)= 219.42, 449.00
RAD.= 126.00 LOWEST Y= 454.00 SLICE NO.= 10 NL= 441.00
CASE 1 US
FS= 3.378, E.C.= -.05

*** ARC 10 ***
ARC NO.= 10 CENTER(X,Y)= 200.00, 500.00 EXIT(X,Y)= 259.42, 449.00
RAD.= 126.00 LOWEST Y= 454.00 SLICE NO.= 8 NL= 441.00
CASE 1 US
FS= 32.698, E.C.= -.01

*** ARC 11 ***
ARC NO.= 11 CENTER(X,Y)= 40.00, 620.00 EXIT(X,Y)= 120.47, 477.69
RAD.= 166.00 LOWEST Y= 454.00 SLICE NO.= 17 NL= 441.00
CASE 1 US
FS= 6.172, E.C.= -.06

*** ARC 12 ***
ARC NO.= 12 CENTER(X,Y)= 80.00, 620.00 EXIT(X,Y)= 149.83, 469.40
RAD.= 166.00 LOWEST Y= 454.00 SLICE NO.= 15 NL= 441.00
CASE 1 US
FS= 2.990, E.C.= .06

*** ARC 13 ***
ARC NO.= 13 CENTER(X,Y)= 120.00, 620.00 EXIT(X,Y)= 188.84, 449.00
RAD.= 166.00 LOWEST Y= 454.00 SLICE NO.= 18 NL= 441.00
CASE 1 US
FS= 2.247, E.C.= .00
**CASE 1 US**

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<th>LOWEST Y</th>
<th>SLICE No.</th>
<th>ML.</th>
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**CASE 1 VS**

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**ARC 20**

ARC NO.: 20  CENTER(X,Y)= 200.00, 460.00  EXIT(X,Y)= 277.17, 469.00
RAB= 204.00  LOSTEST Y= 454.00  SLICE NO.= 12  UL= 461.00
CASE I US
F5= 4.856,  E.C.=-.04

**ARC 21**

ARC NO.: 21  CENTER(X,Y)= 40.00, 700.00  EXIT(X,Y)= 136.78, 473.84
RAB= 246.00  LOSTEST Y= 454.00  SLICE NO.= 17  UL= 461.00
CASE I US
F5= 7.172,  E.C.=-.04

**ARC 22**

ARC NO.: 22  CENTER(X,Y)= 80.00, 700.00  EXIT(X,Y)= 164.59, 469.00
RAB= 246.00  LOSTEST Y= 454.00  SLICE NO.= 19  UL= 461.00
CASE I US
F5= 3.657,  E.C.=.06

**ARC 23**

ARC NO.: 23  CENTER(X,Y)= 120.00, 700.00  EXIT(X,Y)= 204.59, 469.00
RAB= 246.00  LOSTEST Y= 454.00  SLICE NO.= 19  UL= 461.00
CASE I US
F5= 2.685,  E.C.=.00

**ARC 24**

ARC NO.: 24  CENTER(X,Y)= 160.00, 700.00  EXIT(X,Y)= 246.59, 469.00
RAB= 246.00  LOSTEST Y= 454.00  SLICE NO.= 19  UL= 461.00
CASE I US
F5= 2.660,  E.C.=-.05

**ARC 25**

ARC NO.: 25  CENTER(X,Y)= 200.00, 700.00  EXIT(X,Y)= 294.59, 469.00
RAB= 246.00  LOSTEST Y= 454.00  SLICE NO.= 14  UL= 461.00
CASE I US
F5= 4.846,  E.C.=-.01

ANALYZED ABOVE ARE STORED IN LOCAL FILE TAPE12

**LIST OF ARC/F5 FOR LOSTEST Y= 454.00, NO. OF ARC/F5**

| 1/5.817, 2/2.759, 3/2.397, 4/4.163, 5/5.324, 6/5.756, 7/2.459, 8/2.247, 9/3.378, 10/32 |
| 11/6.172, 12/2.709, 13/2.242, 14/3.030, 15/11.799, 16/4.722, 17/3.299, 18/2.6144, 19/2.697, 20/3 |
| 21/7.172, 22/3.607, 23/2.605, 24/2.608, 25/4.844,     |

**THE MIN F5 IS 2.242, AT CENTER NO. 13**
ANALYSES ABOVE ARE STORED IN LOCAL FILE TAPE13

CODE: 1:NEW ARC FILE, 2:REPLACE AN ARC, 3:HORSTIT GRID, 4:STOP

# ARCS

## ARC 1
- ARC NO.: 1
- CENTER(X,Y): 40.00, -500.00
- EXIT(X,Y): 105.72, -464.41
- RAB.: 116.00
- LOWEST Y: -464.00
- SLICE NO.: 11
- ML: -464.00
- CASE 1-US

FS: 5.383, E.C.: -.01

## ARC 2
- ARC NO.: 2
- CENTER(X,Y): 80.00, -500.00
- EXIT(X,Y): 130.99, -473.81
- RAB.: 116.00
- LOWEST Y: -464.00
- SLICE NO.: 12
- ML: -464.00
- CASE 1-US

FS: 2.685, E.C.: .00

## ARC 3
- ARC NO.: 3
- CENTER(X,Y): 120.00, -500.00
- EXIT(X,Y): 153.69, -469.00
- RAB.: 116.00
- LOWEST Y: -464.00
- SLICE NO.: 11
- ML: -464.00
- CASE 1-US

FS: 2.200, E.C.: .06

## ARC 4
- ARC NO.: 4
- CENTER(X,Y): 160.00, -500.00
- EXIT(X,Y): 193.69, -469.00
- RAB.: 116.00
- LOWEST Y: -464.00
- SLICE NO.: 8
- ML: -464.00
- CASE 1-US

FS: 2.952, E.C.: .00

## ARC 5
- ARC NO.: 5
- CENTER(X,Y): 200.00, -500.00
- EXIT(X,Y): 233.69, -469.00
- RAB.: 116.00
- LOWEST Y: -464.00
- SLICE NO.: 4
- ML: -464.00
- CASE 1-US

THE RESULTS ARE NOT CORRECT—ERROR OF CLOSURE NOT CONVERGED DUE TO THE INTER SLICE TENSILE FORCE EXISTS.

FS: 81.221, E.C.: -.25
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The results are not correct—error of closure not converged due to the inter slice tensile force exists.

FS= 3.294, E.C.= .00

FS= 2.254, E.C.= .05

FS= 2.244, E.C.= .02

FS= 3.667, E.C.= -.01

FS= 7.947, E.C.= -.01

FS= 3.764, E.C.= .02
* * * ARC  18  * * *
ARC No. 18 CENTER(X,Y) = 120.00, 700.00  EXIT(X,Y) = 168.32, 469.00
RAB = 236.00  LOWEST Y = 464.00  SLICE No. = 14  ML = 461.00
CASE I US
FS= 2.472,  E.C. = .03

* * * ARC  19  * * *
ARC No. 19 CENTER(X,Y) = 140.00, 700.00  EXIT(X,Y) = 208.32, 469.00
RAB = 236.00  LOWEST Y = 464.00  SLICE No. = 14  ML = 461.00
CASE I US
FS= 2.136,  E.C. = .00

* * * ARC  20  * * *
ARC No. 20 CENTER(X,Y) = 200.00, 700.00  EXIT(X,Y) = 248.32, 469.00
RAB = 236.00  LOWEST Y = 464.00  SLICE No. = 10  ML = 461.00
CASE I US
AT X = 147.84 THERE MAY EXIST A RAB BOUND., OR SLICE,CHECK GRAPHICALLY
AT X = 151.08 THERE MAY EXIST A RAB BOUND., OR SLICE,CHECK GRAPHICALLY

* * * ARC  21  * * *
ARC No. 21 CENTER(X,Y) = 40.00, 740.00  EXIT(X,Y) = 125.66, 477.63
RAB = 276.00  LOWEST Y = 464.00  SLICE No. = 16  ML = 461.00
CASE I US
FS= 8.720,  E.C. = -.04
ANALYSES ABOVE ARE STORED IN LOCAL FILE TAPE11

* * * ARC  22  * * *
ARC No. 22 CENTER(X,Y) = 80.00, 740.00  EXIT(X,Y) = 143.74, 471.47
RAB = 276.00  LOWEST Y = 464.00  SLICE No. = 15  ML = 461.00
CASE I US
FS= 4.204,  E.C. = .03

* * * ARC  23  * * *
ARC No. 23 CENTER(X,Y) = 120.00, 740.00  EXIT(X,Y) = 172.30, 469.00
RAB = 276.00  LOWEST Y = 464.00  SLICE No. = 16  ML = 461.00
CASE I US
FS= 2.722,  E.C. = .05
**OAK 240 mU., 24 CBM(X,Y) 160.0, 70.0
EXIT(XT) 212.30, 469.00
NO.= 17
LEST Y- 464.00
W.' 461.00
CAE I Us
FS- 2.204, E.C.=-.09

**OAK 25 CENTER(XY) 200.00, 740.00 EXIIIXY) 252.30, 469.00
R6.w 276.00
LOEST Y- 464.00
SLICE NO.- 12
W.' 461.00
CAE I Us
FS= 4.712, E.C.=-.02

LIST OF ARC/FS FOR LOWEST Y= 464.00, NO.OF ARC/FS
1/ 5.383, 2/ 2.665, 3/ 2.208, 4/ 2.953, 5/ 81.221, 6/ 6.349, 7/ 2.982, 8/ 2.127, 9/ 2.535, 10/ 000000,
11/ 7.139, 12/ 3.259, 13/ 2.254, 14/ 2.244, 15/ 31.407, 16/ 7.947, 17/ 3.766, 18/ 2.472, 19/ 2.134, 20/ 99.000,
21/ 8.720, 22/ 4.204, 23/ 2.727, 24/ 2.204, 25/ 4.712,
THE MIN FS IS 2.127, AT CENTER NO. B

ANALYSES ABOVE ARC STORED IN LOCAL FILE TAPE.12
CODE: 1=NEW ARC FILE, 2=RERUN AN ARC, 3=MODIFY GRID, 4=STOP
? 3
READ NEW GRID, 8 VAR.
? REL,INC, YBC, XDE, YDE, TSC. OY, IM, ROUTER
? 40 80 660 240 420 474 461 0

**ARC 1
ARC NO.= 1 CENTER(X,Y)= 80.00, 460.00 EXIT(X,Y)= 122.12, 479.83
RAB.= 186.00 LOWEST Y= 474.00 SLICE NO.= 11 W.' 461.00
CASE I US
FS= 2.940, E.C.= .04

**ARC 2
ARC NO.= 2 CENTER(X,Y)= 120.00, 460.00 EXIT(X,Y)= 134.62, 474.58
RAB.= 186.00 LOWEST Y= 474.00 SLICE NO.= 12 W.' 461.00
CASE I US
FS= 1.972, E.C.= .05
**Arc 9**
NO ANALYSIS, FAILURE ARC CANNOT BE FORMULATED
CENTER, (X,Y), AT 200.00 700.00

**Arc 10**
NO ANALYSIS, FAILURE ARC CANNOT BE FORMULATED
CENTER, (X,Y), AT 240.00 700.00

**Arc 11**
ARC NO. = 11  CENTER(X,Y) = 80.00, 740.00  EXIT(X,Y) = 125.03, 477.04
RAB = 246.00  LOWEST T = 474.00  SLICE NO. = 12  ML = 461.00
CASE I US
FS = 3.710, E.C. = .03

**Arc 12**
ARC NO. = 12  CENTER(X,Y) = 120.00, 740.00  EXIT(X,Y) = 125.04, 474.03
RAB = 246.00  LOWEST T = 474.00  SLICE NO. = 12  ML = 461.00
CASE I US
FS = 2.367, E.C. = -.03

**Arc 13**
ARC NO. = 13  CENTER(X,Y) = 160.00, 740.00  EXIT(X,Y) = 131.96, 475.48
RAB = 246.00  LOWEST T = 474.00  SLICE NO. = 9  ML = 461.00
CASE I US
FS = 1.493, E.C. = .00

**Arc 14**
NO ANALYSIS, FAILURE ARC CANNOT BE FORMULATED
CENTER, (X,Y), AT 200.00 740.00

**Arc 15**
NO ANALYSIS, FAILURE ARC CANNOT BE FORMULATED
CENTER, (X,Y), AT 240.00 740.00
* * * ARC 3 * * *
ARC NO. = 3 CENTER (X,Y) = 140.00, 440.00 EXIT (X,Y) = 128.34, 474.71
RAB. = 186.00 LONEST Y = 474.00 SLICE NO. = 4 ML. = 441.00
CASE 1 US
FS = 1.820, E.C. = -.02

* * * ARC 4 * * *
NO ANALYSIS, FAILURE ARC CANNOT BE FORMULATED
CENTER, (X,Y), AT 200.00 660.00

* * * ARC 5 * * *
NO ANALYSIS, FAILURE ARC CANNOT BE FORMULATED
CENTER, (X,Y), AT 200.00 660.00

* * * ARC 6 * * *
ARC NO. = 6 CENTER (X,Y) = 80.00, 700.00 EXIT (X,Y) = 123.75, 479.28
RAB. = 226.00 LONEST Y = 474.00 SLICE NO. = 11 ML. = 441.00
CASE 1 US
FS = 3.227, E.C. = 0.00

* * * ARC 7 * * *
ARC NO. = 7 CENTER (X,Y) = 120.00, 700.00 EXIT (X,Y) = 134.47, 474.49
RAB. = 226.00 LONEST Y = 474.00 SLICE NO. = 12 ML. = 441.00
CASE 1 US
FS = 2.133, E.C. = 0.00

* * * ARC 8 * * *
ARC NO. = 8 CENTER (X,Y) = 140.00, 700.00 EXIT (X,Y) = 130.72, 475.92
RAB. = 226.00 LONEST Y = 474.00 SLICE NO. = 7 ML. = 441.00
CASE 1 US
FS = 1.739, E.C. = .00
* * * ARC 16 * *
ARC NO. = 16 CENTER(X,Y) = 80.00, 780.00 EXIT(X,Y) = 126.07, 477.49
RAB. = 306.00 LOWEST Y = 474.00 SLICE NO. = 14 ML = 461.00
CASE I US
FS= 4.145, E.C. = .04

* * * ARC 17 * *
ARC NO. = 17 CENTER(X,Y) = 120.00, 780.00 EXIT(X,Y) = 135.20, 474.38
RAB. = 306.00 LOWEST Y = 474.00 SLICE NO. = 13 ML = 461.00
CASE I US
FS= 2.640, E.C. = .00

* * * ARC 18 * *
ARC NO. = 18 CENTER(X,Y) = 160.00, 780.00 EXIT(X,Y) = 125.74, 475.22
RAB. = 306.00 LOWEST Y = 474.00 SLICE NO. = 11 ML = 461.00
CASE I US
FS= 1.863, E.C. = -.08

* * * ARC 19 * *
NO ANALYSIS, FAILURE ARC CANNOT BE FORMULATED
CENTER, (X,Y), AT 200.00, 780.00

* * * ARC 20 * *
NO ANALYSIS, FAILURE ARC CANNOT BE FORMULATED
CENTER, (X,Y), AT 240.00, 780.00

* * * ARC 21 * *
ARC NO. = 21 CENTER(X,Y) = 80.00, 620.00 EXIT(X,Y) = 126.92, 477.20
RAB. = 346.00 LOWEST Y = 474.00 SLICE NO. = 14 ML = 461.00
CASE I US
FS= 4.540, E.C. = .01

* * * ARC 22 * *
ARC NO. = 22 CENTER(X,Y) = 120.00, 620.00 EXIT(X,Y) = 125.32, 474.34
RAB. = 346.00 LOWEST Y = 474.00 SLICE NO. = 13 ML = 461.00
CASE I US
FS= 2.896, E.C. = -.01
ARC 23

ARC NO.: 23  CENTER(X,Y) 160.00  740.00  EXIT(X,Y) 133.78  475.03
RAD.: 346.00  LOWEST T= 474.00  SLICE NO.: 12  NL= 661.00
CASE I US
FS= 2.042, E.C.= .03

ARC 24

NO ANALYSIS, FAILURE ARC CANNOT BE FORMULATED
CENTER,(X,Y), AT 200.00  820.00

ARC 25

NO ANALYSIS, FAILURE ARC CANNOT BE FORMULATED
CENTER, (X,Y), AT 240.00  820.00

LIST OF ARC/FS FOR LOWEST T= 474.00, NO.OS ARC/FS

21/ 4.540, 22/ 2.896, 23/ 2.042, 24/ 99.000, 25/ 99.000,

THE MIN FS IS 1.493, AT CENTER NO. 13

ANALYSES ABOVE ARE STORED IN LOCAL FILE TAPE11

CODE: 1=NEW ARC FILE, 2=RUN AN ARC, 3=MODIFY GRID, 4=STOP

3

READ NEW GRID, G VAR. 1
REL,ARC,XC,YES,YES,YES,ML,OUTER
7 40 159 240 200 464 461 0

ARC 1

ARC NO.: 1  CENTER(X,Y) 120.00  740.00  EXIT(X,Y) 105.73  484.40
RAD.: 256.00  LOWEST T= 484.00  SLICE NO.: 10  ML= 461.00
CASE I US
FS= 2.071, E.C.= .02

ANALYSES ABOVE ARE STORED IN LOCAL FILE TAPE11

ARC 2

NO ANALYSIS, FAILURE ARC CANNOT BE FORMULATED
CENTER, (X,Y), AT 160.00  740.00
**Arc 3**

No analysis. Failure arc cannot be formulated.

Center, (x, y), at: 200.00, 740.00

**Arc 4**

No analysis. Failure arc cannot be formulated.

Center, (x, y), at: 240.00, 740.00

**Arc 5**

Arc No.:  5  Center (x, y): 120.00, 760.00  Exit (x, y): 105.96, 684.33
RAE: 296.00  Lowest Y: 484.00  Slice No.: 11  UR: 1141.00  Case I US

FS: 2.320, E.C.: .01

**Arc 6**

Arc No.:  6  Center (x, y): 160.00, 780.00  Exit (x, y): 73.96, 498.78
RAE: 296.00  Lowest Y: 484.00  Slice No.: 6  UR: 1141.00  Case I US

FS: 1.444, E.C.: .01

**Arc 7**

No analysis. Failure arc cannot be formulated.

Center, (x, y), at: 200.00, 780.00

**Arc 8**

No analysis. Failure arc cannot be formulated.

Center, (x, y), at: 240.00, 780.00

**Arc 9**

Arc No.:  9  Center (x, y): 120.00, 829.00  Exit (x, y): 106.09, 684.29
RAE: 336.00  Lowest Y: 484.00  Slice No.: 11  UR: 1141.00  Case I US

FS: 2.575, E.C.: .01
**ARC** 10

ARC NO. 10
CENTER (X, Y) = 160.00, 620.00
EXIT (X, Y) = 84.04, 492.51
RAD. = 336.00
LOWEST Y = 484.00
SLICE NO. = 6
ML = 461.00
CASE I US

FS = 1.475, E.C = .03

**ARC** 11

NO ANALYSIS, FAILURE ARC CANNOT BE FORMULATED
CENTER (X, Y), AT 200.00 820.00

**ARC** 12

NO ANALYSIS, FAILURE ARC CANNOT BE FORMULATED
CENTER (X, Y), AT 240.00 820.00

LIST OF ARC/FS FOR LOWEST Y = 484.00, NO. OF ARC/FS
1/ 2.071, 2/99.000, 3/99.000, 4/99.000, 5/ 2.320, 6/ 1.464, 7/99.000, 8/99.000, 9/ 2.575, 10/ 1.475,
11/99.000, 12/99.000,

THE MIN FS IS 1.464, AT CENTER NO. 6

ANALYSES ABOVE ARE STORED IN LOCAL FILE TAPE12
CODE: 1=NEW ARC FILE, 2=REUN ARC, 3=MODIFY GRID, 4=STOP
? 3
READ NEW GRID, 8 VAR.
BEL, TDC, TDC, XEM, XEM, TDC, YL, YL, XL, XL
? 40 120 620 120 620 464 461 0

**ARC** 1

ARC NO. 1
CENTER (X, Y) = 120.00, 620.00
EXIT (X, Y) = 139.18, 469.00
RAD. = 156.00
LOWEST Y = 464.00
SLICE NO. = 14
ML = 461.00
CASE I US

FS = 2.127, E.C = -.06

LIST OF ARC/FS FOR LOWEST Y = 464.00, NO. OF ARC/FS
1/ 2.127,

THE MIN FS IS 2.127, AT CENTER NO. 1

ANALYSES ABOVE ARE STORED IN LOCAL FILE TAPE11
CODE: 1=NEW ARC FILE, 2=REUN ARC, 3=MODIFY GRID, 4=STOP
? 2
PERM FILE GS0081 Copied To LOCAL FILE TAPE1
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### PROFILE INPUTS

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**ERROR OF CLOSURE** = 16.26 TRIAL FS = 2.5000

**ERROR OF CLOSURE** = 12.32 TRIAL FS = 2.0000

**ERROR OF CLOSURE** = -0.06 TRIAL FS = 2.1265

---

**RESULTS FROM COMPOSITE FORCE POLYGON**

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<th>GLIC SOIL # BEVE'S STRENGTH # INT. SLI. F. x EFF. %</th>
<th>NORMAL MT-PORE</th>
</tr>
</thead>
<tbody>
<tr>
<td>INDEX CONE, TAN PHI, PUSHER, RESULT STRESS FORCE RESULT</td>
<td></td>
</tr>
<tr>
<td>NS L J KIP PHI RES KIP KIP KIP KIP KIP</td>
<td></td>
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<tr>
<td>1 1 1 0.00 .3 17.6 0.0 .2 .1 .42 .34</td>
<td></td>
</tr>
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<td>2 2 1 0.00 .3 10.9 .2 .8 .4 .97 .67</td>
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<tr>
<td>3 2 1 0.00 .3 10.9 .8 10.8 1.1 20.16 18.78</td>
<td></td>
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<tr>
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<tr>
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</tr>
<tr>
<td>6 2 1 0.00 .3 10.9 34.6 24.8 2.9 44.25 49.23</td>
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</tr>
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<td>7 2 1 0.00 .3 10.9 44.8 48.3 3.1 54.63 58.37</td>
<td></td>
</tr>
<tr>
<td>8 2 1 0.00 .3 10.9 48.5 45.4 3.2 54.73 54.62</td>
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</tr>
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<td>9 2 1 0.00 .3 10.9 45.4 36.4 3.2 51.75 50.95</td>
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<tr>
<td>10 1 1 0.00 .3 17.6 34.6 34.2 3.0 9.48 9.08</td>
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<tr>
<td>11 1 1 0.00 .3 17.6 34.2 22.0 2.7 42.96 29.03</td>
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</tr>
<tr>
<td>12 1 1 0.00 .3 17.6 22.0 9.6 2.1 32.00 26.97</td>
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<tr>
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<td></td>
</tr>
<tr>
<td>14 1 1 0.00 .3 17.6 .6 -.1 .1 1.19 1.07</td>
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**FP** = 2.137, E.C. = -0.6

**BISHOP SIMPLIFIED METHOD USING S STRENGTH FOR ALL CASES**

**CONVEX F(2.119431753225)""
ONONDAGA DAM, NY

WAVE ANALYSIS
APPENDIX R

U.S. Army Corps of Engineers, Buffalo District
1776 Niagara Street
Buffalo, NY
APPENDIX E
WAVE ANALYSIS
FOR ONONDAGA DAM*

E1. GENERAL

The wave analysis for Onondage Dam was accomplished using guidelines established in ETL 1110-2-221 - "Wave Runup and Wind Setup on Reservoir Embankments." A stage-frequency analysis using maximum peak and daily pool elevations for Onondage Dam was done to determine pool levels to be used in the design analysis.

E2. POOL STAGE FREQUENCY CURVES

Pool stage-frequency curves were developed using maximum peak (or instantaneous) pool elevations and maximum daily pool elevations for water years 1953 through 1983. Both the peak and daily pool elevations were ranked from highest to lowest and plotted on probability paper. The plotting positions of the data was determined using the Median Plotting Position Method. These curves can be found on Figure E1 and E2. The data used for the frequency curve can be found on Table E1. The instantaneous and daily pool stage-frequency curves were plotted together on Figure E3.

The stage-frequency curve used in this analysis was the curve developed using the daily data. Wave generation depends on wind speed and duration, thus using the daily stage-frequency curve would provide a stable pool level for wave generation. Using Figure E2, the 100-year daily pool elevation would be elevation 490.0 feet NGVD.

The Probable Maximum Flood (PMF) estimate was re-developed during the dam break analysis for Onondaga Dam. The instantaneous peak PMF pool elevation is elevation 519.0 feet NGVD. The pool behind the Onondaga Dam would be at or near elevation 519.0 feet NGVD for around 6 hours, so elevation 519.0 feet NGVD was used as the pool elevation in the wave analysis.

E3. MAXIMUM WINDS

The design wind and duration was developed using paragraph 3 of ETL 1110-2-221: Design Wind Velocity Curves. Using the regional winds statistics found on Figure 2 through 9 in ETL 1110-2-221, the following wind criteria for Onondaga Dam is applicable:

<table>
<thead>
<tr>
<th>Period</th>
<th>Winter</th>
<th>Spring</th>
<th>Summer</th>
<th>Fall</th>
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<tbody>
<tr>
<td>1 Minute</td>
<td>60 MPH</td>
<td>55 MPH</td>
<td>50 MPH</td>
<td>60 MPH</td>
</tr>
<tr>
<td>1 Hour</td>
<td>40 MPH</td>
<td>35 MPH</td>
<td>30 MPH</td>
<td>40 MPH</td>
</tr>
</tbody>
</table>

*Performed by a Hydrologic Investigations Sections, Buffalo District
Since most of the peak annual events occur in the late winter - early spring months (as seen in Table E1) it was decided to use the largest of the two wind statistics to develop the wind velocity duration curve.

Table E1 - Maximum Peak and Daily Ponding Elevation for Onondaga Dam
Date (peak/daily)

<table>
<thead>
<tr>
<th>Year</th>
<th>Peak</th>
<th>Daily</th>
<th>Date</th>
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<tbody>
<tr>
<td>1953</td>
<td>468.06</td>
<td>467.6</td>
<td>12-12/12-11</td>
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<tr>
<td>1954</td>
<td>472.07</td>
<td>471.9</td>
<td>5-4/5-4</td>
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<tr>
<td>1955</td>
<td>473.15</td>
<td>472.4</td>
<td>3-2/3-1</td>
</tr>
<tr>
<td>1956</td>
<td>476.96</td>
<td>471.9</td>
<td>3-9/3-8</td>
</tr>
<tr>
<td>1957</td>
<td>473.18</td>
<td>472.8</td>
<td>1-23/1-23</td>
</tr>
<tr>
<td>1958</td>
<td>471.9</td>
<td>471.7</td>
<td>4-8/4-7</td>
</tr>
<tr>
<td>1959</td>
<td>477.5</td>
<td>477.4</td>
<td>1-22/1-22</td>
</tr>
<tr>
<td>1960</td>
<td>485.9</td>
<td>485.1</td>
<td>4-1/4-1</td>
</tr>
<tr>
<td>1961</td>
<td>477.3</td>
<td>472.2</td>
<td>2-26/2-26</td>
</tr>
<tr>
<td>1962</td>
<td>471.0</td>
<td>470.4</td>
<td>3-13/3-12</td>
</tr>
<tr>
<td>1963</td>
<td>471.0</td>
<td>471.0</td>
<td>3-26/3-26</td>
</tr>
<tr>
<td>1964</td>
<td>478.2</td>
<td>476.9</td>
<td>3-6/3-5</td>
</tr>
<tr>
<td>1965</td>
<td>466.8</td>
<td>466.0</td>
<td>2-9/2-8</td>
</tr>
<tr>
<td>1966</td>
<td>471.1</td>
<td>471.1</td>
<td>2-14/2-14</td>
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<td>1967</td>
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<td>464.3</td>
<td>3-30/3-29</td>
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<tr>
<td>1968</td>
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<td>467.0</td>
<td>6-29/6-28</td>
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<tr>
<td>1969</td>
<td>467.10</td>
<td>467.02</td>
<td>2-2/2-2</td>
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<td>4-5/4-5</td>
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<td>1971</td>
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<td>12-8/12-8</td>
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<td>477.73</td>
<td>477.12</td>
<td>9-27/9-27</td>
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<tr>
<td>1976</td>
<td>474.99</td>
<td>474.10</td>
<td>4-16/4-17</td>
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<td>1977</td>
<td>472.89</td>
<td>472.58</td>
<td>3-14/3-14</td>
</tr>
<tr>
<td>1978</td>
<td>477.40</td>
<td>477.03</td>
<td>10-18/10-18</td>
</tr>
<tr>
<td>1979</td>
<td>483.80</td>
<td>482.96</td>
<td>3-6/3-7</td>
</tr>
<tr>
<td>1980</td>
<td>473.68</td>
<td>472.85</td>
<td>3-22/3-22</td>
</tr>
<tr>
<td>1981</td>
<td>469.16</td>
<td>468.43</td>
<td>2-12/2-12</td>
</tr>
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<td>1982</td>
<td>479.58</td>
<td>478.94</td>
<td>10-29/10-29</td>
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<td>1983</td>
<td>475.04</td>
<td>474.81</td>
<td>4-27/4-27</td>
</tr>
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</table>
The raw wind statistics so far developed must be expanded and modified to include larger durations and also to take into account the fact that wind travels faster over water than land. Using information provided in Paragraph 3 of ETL 1110-2-221, the raw data can be modified to produce the following statistics:

<table>
<thead>
<tr>
<th>Wind Duration (Hours)</th>
<th>Percent of 1 Hour Velocity (mph)</th>
<th>1 Hour Velocity (mph)</th>
<th>1 Hour Velocity (mph)</th>
</tr>
</thead>
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<tr>
<td>1</td>
<td>100</td>
<td>40</td>
<td>40</td>
</tr>
<tr>
<td>2</td>
<td>96</td>
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<td>37</td>
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</tr>
<tr>
<td>6</td>
<td>88</td>
<td>40</td>
<td>35</td>
</tr>
</tbody>
</table>

Before the wind velocity-duration curve can be adjusted to reflect the difference in wind speed over water to wind speed over land, the effective Fetch (Fe) must be calculated.

The effective Fetch (Fe) was calculated using the guidelines in paragraph 4 of ETL 1110-2-221, Effective Water Fetch (Fe) for Wave Generation. Wind generated waves are influenced by both the direction of the wind and the distance the wind blows over the surface of the reservoir or the fetch. Since inland reservoir's shorelines are generally narrower than open water, an effective Fetch (Fe) concept was used to compensate for the smaller waves found on reservoirs. The Fe adjustment is based on drawing radial lines from the dam embankment to various points on the reservoir shoreline. The radials are of equal adjustment and encompass an area of 45° on each side of the central radial. Five independent Fe calculations were done for the reservoir. The Fe resulting from these calculations ranged between 2,900 feet to 3,950 feet. Since maximum wave heights depend on the maximum effective fetch, a Fe of 3,950 feet was chosen for design purposes. The Fe calculations for the Fe of 3,950 feet can be found on Figure E4.

The Wind Velocity Ratio (velocity over water/velocity over land) for a fetch of 3,950 feet or a .75 mile is approximately 1.11. The wind velocity-duration curve was then adjusted for this ratio and is:

<table>
<thead>
<tr>
<th>Wind Duration</th>
<th>Wind Velocity Over Land</th>
<th>Wind Velocity Over Water</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Minute</td>
<td>60 MPH</td>
<td>67 MPH</td>
</tr>
<tr>
<td>1 Hour</td>
<td>40 MPH</td>
<td>44 MPH</td>
</tr>
<tr>
<td>2 Hours</td>
<td>38 MPH</td>
<td>42 MPH</td>
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<tr>
<td>3 Hours</td>
<td>37 MPH</td>
<td>41 MPH</td>
</tr>
<tr>
<td>4 Hours</td>
<td>36 MPH</td>
<td>40 MPH</td>
</tr>
<tr>
<td>6 Hours</td>
<td>35 MPH</td>
<td>39 MPH</td>
</tr>
</tbody>
</table>

This wind velocity-duration curve can be found on Figure E5.

The wind velocity and duration parameters that are needed to calculate wave height are found at the intersection of the regional wind velocity-curve developed above and the wind velocity duration curve for a .75 mile fetch of open water.
water. The wind velocity duration curve for the .75 mile fetch is calculated using Figure 11 from ETL 1110-2-221. This curve is as follows:

<table>
<thead>
<tr>
<th>Wind Duration</th>
<th>Wind Speed</th>
</tr>
</thead>
<tbody>
<tr>
<td>25 Minutes</td>
<td>13.5 MPH</td>
</tr>
<tr>
<td>20 Minutes</td>
<td>22 MPH</td>
</tr>
<tr>
<td>15 Minutes</td>
<td>43 MPH</td>
</tr>
</tbody>
</table>

This curve is then plotted on Figure E5. The intersection of this curve and the regional wind velocity - duration curves gives you the wave design parameters of wind velocity and duration for Onondaga Reservoir. The intersection of the two lines is at a wind velocity of 52 MPH and a wind duration of 14 minutes.

E4. WAVE HEIGHT

Using the design wind of 52 mph and a wind duration of 14 minutes, the design "significant wave" height \( (H_s) \) would be approximately 2 feet (using Figure 11 of ETL 1110-2-221). This wave height is for the deep water condition. To see if deep water conditions are prevalent, the criteria that depth of water be greater than 1/2 the wave length must be met. The wave length can be calculated using the equation:

\[
L = 5.12 (T)^2
\]

where: 
- \( L \) = wave length
- \( T \) = wave period.

The wave period can be calculated by using Figure 12 of ETL 1110-2-221. Using this figure, the wave period is approximately 2.6 seconds. The wave length would then be 35 feet. The average depth of the Onondaga Reservoir with a full pool is 20 feet. Thus \( 20 > 1/2 (35) \), deep water conditions are met. The average depth of the reservoir pool that is in the effective fetch range is probably greater than 20 feet. This is because the reservoir is generally deeper in the area near the dam than the areas in the upper part of the pool area. The maximum fetch length is around the dam area, not the upper pools of the reservoir (See Figure E4).

This wave height of 2 feet would be applicable over a range of reservoir pool levels. The shoreline prevalent in the maximum fetch area has relatively steep sides. Thus, an increase in pool elevation does not increase pool size in this area, thus the effective fetch would remain the same.

E5. WAVE RUNUP

The Shore Protection Manual defines wave runup as "The rush of water up a structure or beach on the breaking of a wave. Also, Uprush. The amount of runup is the vertical height above stillwater level that the rush of water reaches." The wave runup for Onondaga Dam was calculated by using the guidelines in Paragraph 5 - Wave Runup of ETL 1110-2-221.
The wave runup (vertical height) was calculated by using equation 2 of ETL 1110-2-221. This equation is:

\[ \frac{R_s}{H_s} = \left( \frac{0.4 + (H_s/35)^{1/2}}{\cot 0} \right) - 1 \]

Where:
- \( R_s \) = wave Runup
- \( H_s \) = wave height = 2 feet
- \( L_o \) = wave length = 35 feet
- \( \cot 0 \) = COt of angle of side slope of embankment = 1.5

\[ R_s/2 = (0.4 + (2/35)^{1/2} 1.5) - 1 = 1.32 \]

\[ R_s = 1.32(2) = 2.64 \text{ feet} \]

Since equation 2 uses the significant wave height \((H_s)\) in its calculation, the amount of wave runup is understated. This is due to the fact that 13 percent of the waves in the wave train will be higher than the significant wave height. To compensate for this, it is assumed (ETL 1110-2-221) that the wave heights higher than the significant wave height would increase wave runup by 50 percent. Thus, the maximum runup \((R_m)\) would be:

\[ R_m = 1.5 \times R_s = 1.5 \times (2.64) = 3.96 \text{ feet} \]

E6. WIND SETUP

The Shore Protection Manual defines wind setup as "The vertical rise in the still water level on the leeward side of a body of water caused by wind stresses on the surface of the water." The wind setup for Onondage Dam was calculated using Equation 3 of ETL 1110-2-221. This equation is:

\[ S = \frac{U^2 F}{1,400(D)} \]

Where:
- \( S \) = Wind Setup
- \( U \) = Design wind velocity = 52 MPH
- \( F \) = Fetch = \( 2 \times \text{Fe} \) = 1.50 miles
- \( D \) = Average water depth; for 100-year pool elevation = 15.1 feet
  for PMF pool elevation = 23 feet

Using the 100-year pool elevation, the wind setup would be .20 feet, using the PMF pool elevation, the wind setup would be .13 feet.

E7. DESIGN HEIGHTS

The maximum vertical distance embankment protection is required would be the sum of the stillwater pool elevation, the wind setup, and wave runup. For the 100-year pool event, this elevation would be elevation 494.2 feet NGVD. For the PMF event, the pool level would be elevation 523.13 feet NGVD.
ONONDAGA DAM, NY

SLOPE PROTECTION CALCULATIONS
APPENDIX F

U.S. Army Corps of Engineers, Buffalo District
1776 Niagara Street
Buffalo, NY
Computation of RipRap Design

A. Layer Thickness;

\[ h_s = \left( \frac{G_{s\text{move}} H_s}{2.65(42.4)^{1/3}} \right)^{1/3} \]

\[ G_{s\text{move}} = 2.65 \]

\[ H_s = 24 \]

\[ c_{\text{ota}} = 2.5 \]

\[ W_a = \frac{1}{4.37} \frac{(H_s)^3}{c_{\text{ota}}(g-1)^3} \]

\[ W_a = \frac{165.4(2)^3}{4.37(24)(8-1)} = 27.5 \text{ lb} \]

\[ W_{\text{max}} = 4W_a = 4(27.5 \text{ lb}) = 110 \text{ lb} \]

\[ W_{\text{min}} = \frac{W_a}{g} = \frac{27.5 \text{ lb}}{8} = 3.4 \text{ lb} \]

\[ T = \frac{20}{W_{\text{min}}} = \frac{20}{3.4 \text{ lb}} = 11 \text{ in.} \]
B. GRADATION:

T = 120

<table>
<thead>
<tr>
<th>Percent Below</th>
<th>Limits of Stone Weight (lbs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>86 - 35</td>
</tr>
<tr>
<td>50</td>
<td>26 - 17</td>
</tr>
<tr>
<td>15</td>
<td>13 - 5</td>
</tr>
<tr>
<td>10</td>
<td>4 - 12</td>
</tr>
</tbody>
</table>
UNANDAGA DAM, NY

SPILLWAY STABILITY ANALYSIS
ATTACHMENT NO. 1

U.S. Army Corps of Engineers, Buffalo District
1776 Niagara Street
Buffalo, NY
SUBJECT: Periodic Inspection Report No. 3, Onondaga Dam, Onondaga Creek, NY

DA, North Central Division, Corps of Engineers, 536 South Clark Street, Chicago, Illinois 60605

TO: District Engineer, Buffalo

1. The subject Periodic Inspection Report is approved.

2. Copies of the original 1945 stability analysis are not considered a satisfactory update of embankment stability in accordance with current standards. Recompute and resubmit this analysis prior to the next scheduled periodic inspection.

3. It will be important to monitor the observed seepage (page 4, paragraph 8b) during periods of high pool.

FOR THE DIVISION ENGINEER:

DONALD J. LEONARD
Acting Chief, Engineering Division

Copy furnished:
DAEN-CME-BB, w/cy of bsc and incl
NCBED-DM 5 March 1979

SUBJECT: Periodic Inspection Report No. 3, Onondaga Dam, Onondaga Creek, NY

Division Engineer, North Central
ATTN: NCBED-T

1. Request approval of the attached document.

2. Enclosed are five copies of the "Periodic Inspection Report No. 3, Onondaga Dam Onondaga Creek, NY" dated 9 November 1978.

DONALD M. LIDDELL
Chief, Engineering Division

Incl. as

NCBED-D
NCBED-DM
8. Spillway Analysis

The spillway is being analyzed under present design criteria, reference EM 1110-2-2200. The main change between the original design and the criteria outlined in this manual derives from uplift pressure requirements. The original design was based on uplift over 50% of the concrete-bedrock interface area. Presently, the EM requires the uplift pressure at any point under the structure will be tailwater pressure plus the pressure measured as an ordinate from tailwater to the hydraulic gradient between upper and lower pool. Uplift pressure will be considered as acting over 100% of the area upon which it impinges.

Where no provision for uplift reduction has been made, the hydraulic gradient will be assumed to vary as a straight line from headwater to tailwater."

There are two sections of the spillway being analyzed, at ELEV 485.4' and 461.5'. The section at 485.4' is the concrete and bedrock interface, while at 461.5' is a
The loading cases analyzed are as follows:

(A) Section at Elov 485.4 ft

Case I @ El 485.4

a) Water surface at Elev 520.3
b) Full hydrostatic pressure against upstream face
c) Effective tailwater at Elev 497.5
d) Uplift 100% headwater at the heel decreasing uniformly to 100% effective tailwater at the toe.

Case II @ El 485.4

a) Water surface at Elev. 520.3
b) Full hydrostatic pressure against upstream face
c) Effective tailwater at zero
d) Uplift 100% headwater at the heel decreasing uniformly to zero at the toe.

Case III @ El 485.4

a) Same as Case I @ 485.4' except effective tailwater is at El 504.5'.
(B) Sections at Elev. 461.5

**Case I** @ El. 461.5

1) Same as Case I at El. 461.5

**Case II** @ El. 461.5

- Original Case II is not considered applicable for an analysis at Elev. 461.5 because it was felt impossible for an condition of full headwater to exist with no effective tailwater. Case III below, assumptions was considered a sufficiently severe condition to cover the original assumption of Case I.

**Case III** @ El. 461.5

2) Same as Case I @ El. 461.5 except that effective tailwater Elev. at 504.5.

**Case IV** @ El. 461.5

3) Same as Case I @ El. 461.5 except that effective tailwater Elev. at 485.4.

**Case IV** @ El. 461.5:

a) water surface at Elev. 504.5
b) full hydrostatic pressure against upstream face.

3) No tailwater on spillway side.
4) Lift 100% headwater at the heel decreasing to zero at the toe.

**Sliding Coef**

1) Concrete to Rock = 0.65
2) Rock to Rock = 0.30.
Properties of Concrete wier section above El 485.4

\[ \text{Area} = \frac{\text{EM}_w}{\Sigma A} = \frac{2433.8}{482.2} = 5.04 \text{ ft} \]

\[ W_c = A \times 0.15\% = 482.2 \times 0.15\% = 72.32 \text{ kips} \]
Case I at EL 485.6'

\[ \Sigma M_c \text{ about H.C.} \]

<table>
<thead>
<tr>
<th>( V_k )</th>
<th>( V_m )</th>
<th>( \Sigma V )</th>
<th>( \Sigma H )</th>
<th>( \Sigma M_c )</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.9 l/s</td>
<td>4.9 l/s</td>
<td>19.9 l/s</td>
<td>19.9 l/s</td>
<td>19.9 l/s</td>
</tr>
<tr>
<td>3.37</td>
<td>3.37</td>
<td>9.34</td>
<td>9.34</td>
<td>9.34</td>
</tr>
<tr>
<td>2.09</td>
<td>2.09</td>
<td>6.27</td>
<td>6.27</td>
<td>6.27</td>
</tr>
<tr>
<td>1.33</td>
<td>1.33</td>
<td>4.09</td>
<td>4.09</td>
<td>4.09</td>
</tr>
<tr>
<td>0.86</td>
<td>0.86</td>
<td>2.58</td>
<td>2.58</td>
<td>2.58</td>
</tr>
<tr>
<td>0.47</td>
<td>0.47</td>
<td>1.43</td>
<td>1.43</td>
<td>1.43</td>
</tr>
</tbody>
</table>

Total: \( \Sigma V = 31.97 \) l/s, \( \Sigma H = 25.48 \) ft, \( \Sigma M_c = 734.71 \) ft-lb

\[ \bar{Q} = \frac{\Sigma M_c}{\Sigma V} = \frac{734.71}{31.97} = 22.98 \text{ ft} \]
\[
\frac{b}{3} = \frac{34.72}{3} = 11.57 \text{ ft.}
\]
\[
e = 5.63 \text{ ft} < e_{\text{max}} = 5.79 \text{ ft} \quad \text{okay}
\]

Resultant is 0.17 ft within the middle third of base.

**Sliding Coef (without Rails)**
\[
\frac{Eh}{2V} = \frac{25.48}{31.97} = 0.797 > 0.65 \quad \text{NG}
\]

**Sliding Coef (with Rail) using monolith #1 with 5 Rails (=70 ASCE)**
\[
F_i = \frac{5.2\text{in.}}{2} \times \frac{12 \text{in.}^2}{33.13 \text{ ft}} = 12.26 \text{ kips}
\]
\[
C_{\text{eff}} = \frac{H_i - F_i}{V} = \frac{25.48 - 12.26}{31.97}
\]
\[
\left[ \text{Sliding Coef} = 0.41 \right] < 0.65 \quad \text{okay}
\]

Max. Ftn. Pressue = \[
\frac{34.72^2}{34.72^2} \left( 1 + \frac{6(5.63)}{34.72} \right) = 182 \text{ kips}
\]

Min. Ftn. Pressure = \[
\frac{34.72^2}{34.72^2} \left( 1 - \frac{6(5.63)}{34.72} \right) = 0.025 \text{ kips}
\]
Subject: 
Computation of: 
Computed by: 
Checked by: 
Date: 

Case II
Elev. 485.4 ft

\[ \begin{align*}
\Sigma M_C & = \Sigma M_H \quad \text{(Sum of Moments about Point C)} \\
V_1 & = 7.28 \quad \Sigma E & = 5.01 \quad E H & = 21.91 \\
V_2 & = 15.41 \quad \Sigma h & = 11.57 \quad E H' & = 114.45 \\
V_3 & = 37.87 \quad \Sigma h & = 10.86 \quad \Sigma h' & = 9.55 \\
H_4 & = 11.40 \quad \Sigma h & = 6.37 \quad 160.11 \\
H_5 & = 72.42 \\
\end{align*} \]

\[ \Sigma H = 30.26 \quad \Sigma M = 950.93 \]

\[ \bar{y} = \frac{\Sigma M}{\Sigma E} = \frac{950.93}{41.73} = 22.79 \text{ ft} \]
\[ \frac{6}{3} = 2 \]

\[ E = 5.43 \text{ ft} = E_{\text{max}} = 5.79 \quad \text{(okay)} \]

Resultant is 0.36 ft within the middle third of base.

**Sliding Coef (without Rails).**

\[ \frac{\Sigma H}{\Sigma V} = \frac{44.72}{44.72} = 0.785 > 0.65 \quad \text{N.G.} \]

**Sliding Coef with Rails.**

\[ F_i = (\text{see } P_2) = 12.26 \text{ kN} \]

\[ \text{Sliding Coef} = \frac{H_2 - F_i}{\frac{V}{6}} = \frac{50.26 - 12.26}{44.72} \]

\[ \left[ \text{Sliding Coef} = 0.43 \right] < 0.65 \quad \text{okay} \]

**Max Ftn. Pressure**

\[ \frac{44.72}{34.72} \left( 1 + \frac{6.54}{34.72} \right) = 0.33 \text{ kN/ft} \]

**Min Ftn. Pressure**

\[ \frac{44.72}{34.72} \left( 1 - \frac{6.54}{34.72} \right) = 0.075 \text{ kN/ft} \]
\[ \begin{align*}
\text{Case III, Elev. 485.4 ft.} \\
\text{HW, EL 520.3 ft.} \\
\text{TW, EL 504.5 ft.} \\
\end{align*} \]

\[ \begin{align*}
\text{\( V_1 \) at Point \( \text{C} \)} & & \text{EV} & & \Sigma H & & \Sigma M & & \Sigma M_2 \\
V_1 & = & \text{53.95 ft.} & & 72.68 & & 3.01 & & 21.91 \\
V_2 & = & \left( \text{53.95} \times 19.1 \right) - 30.94 & & 72.32 & & 17.72 & & 104.85 \\
V_3 & = & \left( \text{53.95} \times 19.1 \right) - 0.0625 & & 8.17 & & 11.57 & & 439.16 \\
H_3 & = & -\frac{1}{2} \times 19.1 \times 34.72 \times 0.0625 & & -20.72 & & 2.15 & & -71.67 \\
H_2 & = & -\frac{1}{2} \times 19.1 \times 19.1 \times 0.0625 & & -11.40 & & 9.55 & & 180.11 \\
H_1 & = & -\frac{1}{2} \times 19.1 \times 19.1 \times 0.0625 & & -11.40 & & 6.37 & & -72.67 \\
\text{Totals} & & \text{EV} = 29.18 & & \Sigma H = 18.86 & & \Sigma M = 543.49 \\
\end{align*} \]

\[ \frac{\Sigma M_2}{\Sigma V} = \frac{543.49}{29.18} = 18.63 \text{ ft} \]
\[ b = 11.57 \text{ ft.} \]
\[ c = 11.27 \text{ ft.} \quad \text{and} \quad \text{cure} = 5.79 \text{ ft.} \]

Resultant is 4.52 \text{ ft.} within the middle third of base.

Sliding Coef (without Rails)
\[ \frac{FH}{E} = \frac{12.86}{29.78} \approx 0.45 \approx 0.45 \]

Sliding Coef (with Rails)
\[ F_r = \text{sum of} \cdot c = 12.26 \text{ k/ft} \]
\[ \text{Coef} = \frac{H_r - F_r}{V_{in}} = \frac{18.86 - 12.26}{29.18} \]

Sliding Coef = 0.23 < 0.65

Max Flt. Pressure = \[ \frac{29.18}{34.72} \cdot (1 + \frac{6(1.27)}{34.72}) = 1.02 \text{ k/ft} \]

Min Flt. Pressure = \[ \frac{39.18}{34.72} \cdot (1 - \frac{6(1.27)}{34.72}) = 0.65 \text{ k/ft} \]
Investigation of conditions at ELEV 4615' F
Case 1 at ELEV 4615'
\[ \Sigma_{\text{load}} \quad \text{about} \quad Pt. \ H \]

<table>
<thead>
<tr>
<th>ZV</th>
<th>ZH</th>
<th>ma</th>
<th>ZH \sigma</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.28</td>
<td>3.01</td>
<td>21.91</td>
<td></td>
</tr>
<tr>
<td>72.32</td>
<td>15.41</td>
<td>1114.45</td>
<td></td>
</tr>
<tr>
<td>3.27</td>
<td>31.75</td>
<td>107.00</td>
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</tr>
<tr>
<td>9.04</td>
<td>49.70</td>
<td>367.43</td>
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</tr>
<tr>
<td>124.17</td>
<td>17.36</td>
<td>1160.18</td>
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<tr>
<td>38.79</td>
<td>828.95</td>
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</tr>
<tr>
<td>8.93</td>
<td>42.68</td>
<td>381.33</td>
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</tr>
<tr>
<td>85.24</td>
<td>45.56</td>
<td>123.44</td>
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<tr>
<td>82.30</td>
<td>31.11</td>
<td>163.33</td>
<td></td>
</tr>
<tr>
<td>42.46</td>
<td>21.50</td>
<td>92.49</td>
<td></td>
</tr>
<tr>
<td>57.78</td>
<td>14.33</td>
<td>88.59</td>
<td></td>
</tr>
<tr>
<td>-40.50</td>
<td>15.00</td>
<td>48.00</td>
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</tbody>
</table>

Total \( \Sigma \text{V} = 108.57 \quad \Sigma \text{H} = 59.74 \quad \Sigma \text{H} \sigma = 3270.51 \)

\[ \bar{f} = \frac{\Sigma_{\text{H}}}{\Sigma \text{V}} = \frac{3270.51}{108.57} = 30.12 \text{ ft.} \]

\[ \bar{z} = \frac{46.67}{3} = 15.56 \text{ ft.} \]

\[ c = 6.80' < C_{\text{max}} = 7.78 \text{ ft} \]

Resultant is 0.98' within the middle third of base.

Sliding Coeff (without Rails):

\[ \text{Coeff} = \frac{\Sigma_{\text{H}}}{\Sigma \text{V}} = \frac{59.74}{108.57} = \]

\[ \text{Coeff} = 0.55 \times 0.30 = 0.17 \text{ N.G} \]

Sliding Coeff (with Rails) (more 1/4 #6)

\[ F_0 = (3 \text{ each}) \times 6.81 \text{ in.} \times 12 \times 4 \text{ in.} \]

\[ F_0 = 31.87 \text{ kips} \]
\[
\text{Cof} = \frac{H_1 - H_2}{H_1} = \frac{59.74 - 31.87}{108.57} = 0.26
\]

\text{Sliding Cof} = 0.26 \leq 0.30 \quad \text{OK!}

\text{Max Fm. Pressure} = \frac{108.57}{46.67} \left(1 + \frac{6 (6.8)}{46.67}\right) = 4.36 \text{ ksf}

\text{Min Fm. Pressure} = \frac{108.57}{46.67} \left(1 - \frac{6 (6.8)}{46.67}\right) = 0.293 \text{ ksf}
Case III of Elev. 461.5 ft.
\[
\begin{align*}
&\text{\(\Sigma M\)} \quad \text{about } A \quad \Sigma V \\
&V_1: \quad \text{see \(P_1\)} \quad 12 \quad \Sigma V \quad \Sigma H \quad m \quad \Sigma M_w \\
&V_2: \quad 3 \quad \Sigma V \quad 21.41 \quad 21.41 \\
&V_3: \quad \left(3.12 \times 2.25\right) \left(0.31 \times 2.25\right) \times 0.0625 = 4.01 \quad 4.01 \\
&V_4: \quad 14.1 \times 14.15 \times 0.0625 = 14.17 \quad 14.17 \\
&V_5: \quad 5 \quad \Sigma V \quad 124.47 \quad 124.47 \\
&V_6: \quad 8 \quad \Sigma V \quad 19.1 \quad 19.1 \\
&V_7: \quad -85.76 \quad 85.76 \\
&V_8: -0.83 \times 0.83 \times 0.0625 = -0.6271 \\
&V_9: \quad 53.78 \quad 53.78 \\
&V_{10}: -0.83 \times 0.83 \times 0.0625 = -0.6278 \\
&\text{Total} \quad \Sigma V = 109.23 \quad \Sigma H = 42.46 \quad \Sigma M = 296.03 \\

&\frac{\Sigma M}{\Sigma V} = \frac{2960.34}{109.23} = 27.10 \text{ ft} \\

&\frac{k}{3} = 15.56 \text{ ft} \quad \epsilon = 3.76 \text{ ft} < \epsilon_{\text{max}} = 7.77 \text{ ft} \quad \text{OK} \\

&\text{Resultant is 4.0 ft within the middle third of base.} \\

&\text{Sliding Coef. (without rails)} \\
&\text{Coef.} = \frac{\Sigma H}{\Sigma V} = \frac{42.46}{109.23} \\
&\text{Coef.} = 0.389 \quad k = 0.30 \quad \text{NG}.
\end{align*}
\]
Sliding Coef. (with Rail):

\[ \tau = (50 \times 10^{2} + 12) = 31.87 \text{ kPa} \]

\[ \text{Coef.} = \frac{1.75 \tau}{V_{0}} = \frac{42.46 - 31.87}{109.23} \]

\[ \text{Sliding Coef.} = 0.10 < 0.30 \quad \text{kPa} \]

Max Friction Pressure:

\[ \frac{109.23}{46.67 + \left(1 - \frac{6(3.26)}{46.67}\right)} = 3.47 \text{ kPa} \]

Min Friction Pressure:

\[ \frac{109.23}{46.67 + \left(1 + \frac{6(3.26)}{46.67}\right)} = 1.21 \text{ kPa} \]
Case III at ELEV. 461.5 ft.

DISREGARD SEE REANALYSIS AT END OF THIS APP.
\[ \bar{y} = \frac{\Sigma M_y}{\Sigma V} = \frac{327.48}{113.81} = 2.87 \text{ ft} \]

\[ \bar{z} = 15.56' \quad [2 \cdot 8.53 > \text{max} = 7.78] \quad \text{N.G.} \]

Resultant is 0.75 ft outside the middle third of base. N.G.

Sliding Coeff (with out Reil's)

\[ \text{Coeff} = \frac{\Sigma H}{\Sigma V} = \frac{82.39}{113.81} \]

\[ \text{Sliding Coeff} = 0.724 \neq 0.30 \quad \text{N.G.} \]
Sliding Coef. (with Boils)

\[ \text{Coeff.} = \frac{H_e - F_0}{V_e} = \frac{82.39 \text{ kips}}{113.81 \text{ kips}} = 0.72 \]

\[ \text{Sliding Coef} = 0.44 \] exceeds 0.30 N.G.

Max Ftn. Pressure = \[ \frac{113.81}{46.67} \left( 1 + \frac{67 (8.53)}{46.67} \right) = 5.11 \text{ ksf} \]

Min Ftn. Pressure = \[ \frac{113.81}{46.67} \left( 1 - \frac{67 (8.53)}{46.67} \right) = -0.24 \text{ ksf} \]

Min Ftn. Pressure is -0.24 ksf which indicates a "tension zone".

Case III shows instability in all areas of analysis.
1) Resultant falls out of middle 1/3 of base.
2) Sliding Coef. = 0.30.
3) Tension zone in Ftn.

Case III is re-analysed with a tension zone to see if this would result in the resultant force falling within the middle 1/3 of the compression zone. Analysis was as follows (Fig #20 thru Fig #2).
Case IV. Analysis of tension zone.

\[ \Sigma M_a \text{ about } PHA \]

\[ \Sigma V \]

\[ \text{ma} \]

\[ \Sigma M \]

\[ \nu_2 = \frac{1}{2} \times 58.8 \times (46.67 - x) - 0.025 = (85.75 - 1.84X) \cdot \left[ x + \frac{1}{4} (46.67 - x) \right] = 7.23X^2 - 25.55X + 163.3 \]

\[ \nu_2 = \frac{1}{2} \times 73.9 \times (46.67 - x) - 0.0625 = (83.75 - 0.75X) \cdot \left[ x + \frac{1}{4} (46.67 - x) \right] = 7.23X^2 - 11.72X + 163.9 \]

\[ \nu_2 = 58.8 \times (X) - 0.025 \]

\[ = 3.075X \cdot \left( \frac{X}{4} \right) = 1.84X^2 \]

\[ \Sigma V = (120.4 + 109X) \]

\[ \Sigma M = 0.36X^2 + 16.83X + 241.8 \]

From \( Py = 12 \times 18 \):

\[ \begin{array}{c|c|c|c|c}
\nu_2 & \Sigma V & \Sigma H & \text{ma} & \Sigma M (m) \\
------------------- & ------------------- & ------------------- & ------------ & ------------------- \\
72.68 & 7.28 & 3.01 & 21.91 & \\
124.47 & 72.32 & 15.71 & 1114.45 & \\
21.42 & 124.47 & 17.36 & 2140.63 & \\
8.93 & 21.42 & 38.70 & 828.95 & \\
42.46 & 8.93 & 42.68 & 381.13 & \\
57.78 & 42.46 & 21.50 & 912.69 & \\
-17.85 & 57.78 & 14.33 & 828.18 & \\
7.97 & -17.85 & 7.97 & -192.21 & \\
\end{array} \]

\[ \Sigma V = 234.42 \]

\[ \Sigma H = 52.39k \]

\[ \Sigma M = 6106.12 \times 4.4 \]
\[ \bar{F} = \frac{\Sigma M}{\Sigma V} \]

Resistive location is given by \( \bar{F} \). The max. range of \( \bar{F} \) would be the outside limit of the kern point. Therefore \( \bar{F} = x + \frac{2}{3}(46.67-x) \).

\[ \bar{F} = \frac{\Sigma M}{\Sigma V} = \left[ x + \frac{2}{3}(46.67-x) \right] \]

\[ \Sigma M = 6106.12 - (0.36x^2 + 16.83x + 24181) \]
\[ \Sigma V = 234.42 - (120.6 + 1085x) \]

Substituting \( x \) solving for \( x \)

\( x = 7.0 \) ft tension zone.

Checking analysis with a 7.0 ft tension zone, to see if resistant will fall within the middle \( \frac{1}{3} \) of compression zone.

[Diagram of forces and points labeled A, V₁₀, V₉, V₈, and V₇ with dimensions and directions indicated.]
\begin{tabular}{|c|c|c|c|}
\hline
\textbf{V} & \textbf{E.V.} & \textbf{E.H.} & \textbf{EM(A)} \\
\hline
\text{V} & \text{see pg. 12} & 7.58 & 3.07 & 21.91 \\
\text{V} & 72.32 & 15.41 & 114.45 \\
\text{V} & 124.47 & 17.56 & 2160.82 \\
\text{V} & 21.42 & 58.70 & 828.95 \\
\text{V} & 8.93 & 42.68 & 381.13 \\
\text{V} & \frac{1}{2}(588 + 39.67 - 0.0225) & -72.89 & 20.22 & -1473.91 \\
\text{V} & \frac{1}{2}(239 + 39.67 - 0.0225) & -24.67 & 33.45 & 991.12 \\
\text{V} & 588 + 70 - 0.0225 & -25.76 & 5.60 & 90.04 \\
\text{H} & \text{see pg. 12} & 42.46 & 21.50 & 9.12.89 \\
\text{H} & 57.78 & 14.33 & 828.18 \\
\text{H} & -17.85 & 7.97 & -142.21 \\
\hline
\end{tabular}

\text{\textit{Total}} \quad \text{\textbf{EM}} = 3551.05 k \text{N}

\text{\textbf{\textit{f}} = \frac{\text{EM}}{\text{\textbf{E.V} = 106.17 k}} = \frac{3551.05 k}{106.17 k} = 33.45 \text{ ft}}

\text{\textbf{\textit{f}} = \frac{3}{5} \text{ (compression) = 39.67}} \quad \text{\textit{f}} = 13.22 \quad \text{\textit{f}} \leq \text{\textit{f}}_{\text{max} = 61.61 \text{ ft}}

\text{\textit{f}} = 61.61 \text{ ft} \leq \text{\textit{f}}_{\text{max}} \quad \text{\textit{f}} \leq \text{\textit{f}}_{\text{max}} \quad \text{\textit{f}} \leq \text{\textit{f}}_{\text{max}} \quad \text{\textit{f}} \leq \text{\textit{f}}_{\text{max}}

\text{\textit{f}} \text{ Resultant falls within the middle \textbf{3} of the compression zone of base.}

\text{\textbf{Sliding Coef. \ (with Rails): There are 13 Rails}}

\text{\textbf{Coef} = \frac{H_{w} - F_{k}}{V_{w}} = \frac{82.39 - 34.87}{106.17}

\text{\textit{Sliding Coef} = 0.48} \geq 0.30 \text{ for Rock to Rock}

\text{\textit{No, good!!}}
Calculate the required amount of steel so that sliding coeff. = 0.30.

\[
\frac{H - F}{V} = 0.30
\]

\[
F = H - 0.30 V
\]

\[
F = 82.39^t - 0.3(106.17) = 50.54 \text{ kips}
\]

Knowing that:

\[
F = \left[\frac{\# \text{ Rails} \times \text{Area/Rail}}{\text{length of rail}}\right] \times \text{shear strength of Rail.}
\]

\[
= \left[\frac{N \times A}{L}\right] S
\]

solving for \( N = \frac{F \cdot L}{A \cdot S} \)

Number of Rails = \( N = \frac{(50.54 \text{ kips}) (3.33 \text{ ft})}{6.81 \text{ in}^2 \times 12 \frac{\text{in}}{\text{ft}}^2} = 20.6 \)

or \( \underline{use \ 21 \ Rails \ - \ (HSC \ \# \ 70)} \)

\[ \{ \text{there were 13 rails provided, therefore additional 8 rails are needed, if there has to be a sliding coeff. = 0.30.} \]
Case V at E elevation 461.5 ft.

H.W. EL 504.5' (crest)

N.W. EL 496.5' 11.95'

H2

V1, V2, V3, V4, V5, V6, V7, V8
Subject: Computation of

<table>
<thead>
<tr>
<th>Coeff.</th>
<th>( \Sigma M_a )</th>
<th>( \frac{\Sigma M_a}{\Sigma V} )</th>
<th>( \Sigma H )</th>
<th>( m_a )</th>
<th>( \Sigma M_a )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( V_1 )</td>
<td>( \frac{1}{2} \times 10 \times 10 \times 0.0625 ) = 3.13</td>
<td>3.13</td>
<td>3.33</td>
<td>10.43</td>
<td></td>
</tr>
<tr>
<td>( V_2 )</td>
<td>( \frac{5}{2} \times 9 \times 12 ) = 272.32</td>
<td>272.32</td>
<td>15.41</td>
<td>111.44.45</td>
<td></td>
</tr>
<tr>
<td>( V_3 )</td>
<td>( \frac{1}{2} \times 14 \times 14 ) = 144.47</td>
<td>144.47</td>
<td>17.36</td>
<td>216.01.22</td>
<td></td>
</tr>
<tr>
<td>( V_4 )</td>
<td>( \frac{1}{2} \times 14 \times 14 ) = 144.47</td>
<td>144.47</td>
<td>17.36</td>
<td>216.01.22</td>
<td></td>
</tr>
<tr>
<td>( V_5 )</td>
<td>( \frac{1}{2} \times 14 \times 14 \times 0.0625 \times 0.12 ) = -62.71</td>
<td>-62.71</td>
<td>15.56</td>
<td>-97.77</td>
<td></td>
</tr>
<tr>
<td>( V_6 )</td>
<td>( \frac{1}{2} \times 14 \times 14 \times 0.0625 \times 0.12 ) = -62.71</td>
<td>-62.71</td>
<td>15.56</td>
<td>-97.77</td>
<td></td>
</tr>
</tbody>
</table>

\( \Sigma V = 158.63 \) \( m_a = 57.78 \)

\( \Sigma M_a = 3962.06 \)

\( \frac{\Sigma M_a}{\Sigma V} = 25.01 \text{ ft.} \)

\( \frac{\Sigma M_a}{\Sigma V} = 15.56 \text{ ft} \)

\( E = 1.48 \text{ ft} < E_{\text{max}} = 7.78 \text{ ft} \) Okay

Resultant is 6.11 ft. within the middle third of base.

Sliding Coef (without rails)

\( \text{Coeff} = \frac{\Sigma H}{\Sigma V} = \frac{57.78}{158.63} \)

Sliding Coef = 0.364 * 0.3 N.G.
Sliding Coef. (with Rails)

\[
\text{Coef} = \frac{V_0 - F_0}{V_0} = \frac{57.78 - 34.82}{158.63}
\]

\[
\left(\text{Sliding Coef} = 0.16\right) < 0.30 \quad \text{OK}
\]

Max Ftn. Pressure = \(\frac{158.63}{46.67} \times \left(1 + \frac{6(48)}{46.67}\right)\) = 4.13 k/sf

Min Ftn. Pressure = \(\frac{158.63}{46.67} \times \left(1 - \frac{6(168)}{46.67}\right)\) = 2.66 k/sf.
C. Conclusion

The stability analysis of Onondaga Dam's spillway structure resulted in stability for all cases investigated except for the conditions under Case II. Case II investigation was for the following loading conditions:

a) analysis at Elev. 461.5 ft.
b) full design headwater at Elev. 520.3 ft.
c) tailwater elevation of 485.6 ft.
d) full uplift pressure over base area (100% headwater varying to 100% tailwater).
e) full hydrostatic pressure against upstream face.

The results of the analysis under Case II shows that the resultant force falls within the middle one-third of the compression zone, with a seven foot tension zone at the heel of the cross-section. The analysis also revealed a failure in sliding, with the actual sliding coefficient (0.48) to be in excess of the allowable design value (0.30).
Based on the above analysis, Buffalo District proposes to provide additional anchorage to ensure stability in the monoliths that are affected by the loading and design criteria of Case II. Request from NCDEED-T, either concurrence on the above analysis and proposed action or guidance as to the relaxation of design criteria.
I CASE II EVALUATED AT ELEV. 461.5'

a. WATER SURFACE AT ELEV 520.3'
b. FULL HYDROSTATIC PRESSURES AGAINST UPSTREAM FACE.
c. EFFECTIVE TAILWATER AT ELEV 485.4'
d. UPLIFT, 100% HEADWATER AT THE NEEL DECREASING UNIFORMLY TO 100% EFFECTIVE TAILWATER AT THE TOE.
Subject: Denvera Dam

Computation of...

Composed by
Checked by 2/26/70
Date 2/26/70

<table>
<thead>
<tr>
<th>X</th>
<th>X</th>
<th>X</th>
<th>X</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>4</td>
<td>5</td>
<td>6</td>
<td>7</td>
</tr>
<tr>
<td>8</td>
<td>9</td>
<td>10</td>
<td>11</td>
</tr>
</tbody>
</table>

\[
\begin{array}{c|c|c|c|c}
\text{H} & \text{A} & \text{N} & \text{X} & \text{X} \\
\hline
\text{X} & \text{X} & \text{X} & \text{X} & \text{X} \\
\end{array}
\]

Location of Resultant:

\[
\bar{y} = \frac{X \cdot A \cdot N}{X \cdot X \cdot X} = 31.8^\circ
\]

Base = 46.67 = 15.56'

\[
\cos \theta = 0.53 \Rightarrow \text{Error} = \frac{15.56}{2} = 7.78^\circ
\]

Resultant is outside of minor

\[
\frac{1}{2} \text{ of Base}
\]

Tension Zone - Uplift Diagram
\[ \sum M = 0 \]

\[ \sum M = 0.36x^2 + 14.83x + 2418.11 \]

\[ \sum M = 6106.12 \quad \text{k-ft} \]

\[ \frac{\sum M}{\sum V} \quad \text{THE LOCATION OF THE RESULTANT} \]

\[ \bar{x} = x + \frac{3}{2} (46.67 - x) \]

\[ \frac{\sum M}{\sum V} = x + \frac{3}{2} (46.67 - x) \]

\[ \sum V = 234.42 - (100.6 + 1085.1) \]

Solving for \( x \):

\[ x = 7.04 \quad \text{ft} \]

Selected:

\[ x = 7.04 \quad \text{ft} \]

\[ V_a = -27.83 \quad \text{k}\]

\[ V_b = -29.69 \quad \text{k}\]

\[ V_c = -26.76 \quad \text{k}\]
PERCENTAGE OF BASE IN COMPRESSION

\[ \text{PERCENTAGE} = \left( \frac{46.67 - 2}{46.67} \right) \times 100 = 85\% > 75\% \text{ MIN (OK)} \]

RESISTANCE FORCE FOR PILES - ASCE 70

\[ F_{\text{pile}} = \frac{13.8 \times 1.61 \times 12}{33.33 + \text{friction}} = 31.87 \text{ kips/ft of pile} \]

RESISTANT FORCE FOR ANCHOR BARS - 1/4 in.

(SEE ATTACHED DRAWING)

1. THERE ARE THREE HOLE OF ANCHOR BARS
2. THERE ARE EIGHT BARS/ROW.
3. BARS ARE INCLINED AT 71° TO THE VERTICAL.

\[ F_{\text{anchor}} = \left[ \frac{3.1415 \times 8 \times \text{area}}{33.33 + \text{friction}} \right] \times 20 \text{ kips/ft} \cos 19° \]

\[ F_{\text{anchor}} = 21.28 \text{ kips/ft of anchor} \]

SLIDING COEFFICIENT

DURING THE CONSTRUCTION OPERATIONS, ROUGHNESS ALONG A HORIZONTAL PLANE IN THE SUPPORTING ROCK AND REVERSED IN THE PROCESS OF BLASTING ROCK TO EXCAVATE FOR THE KEY AT THE HEEL OF THE GRAVITY SECTION, A LAYER OF ROCK APPROXIMATELY 10 FT. LONG, 20 FT. WIDE, AND 4 FT. THICK, ACQUIRED TOWARD THE SPILLWAY SHOULDER UP TO ABOUT ONE FOOT SLIDING ON A PLANE AT APPROXIMATELY ELEV. 465.4.

IT WAS DECIDED TO CLEAN OFF THE ROCK.
TO BLEW 485'4", CONSTRUCT THE GRANULAR CONCRETE SPILLWAY WEIR SECTION, WITH BASE AT THIS ELEVATION AND ELIMINATE THE KEY TO ELIMINATE THE POSSIBILITY OF SLIDING ON SIMILAR PLANES OF WEAKNESS BELOW ELEV. 485'4". IT WAS DECIDED TO STRENGTHEN THE ROCK SUPPORTING THE CONCRETE WEIR BY THE FOLLOWING MEASURES:

a) TO GROUT STEEL RODS EXTENDING FROM THE CONCRETE SLAB FACING INTO THE ROCK SUPPORTING THE WEIR A DISTANCE NECESSARY TO ENGAGE A MASS OF ROCK SUFFICIENT TO PROVIDE STABILITY AGAINST OVERTURNING.

b) TO GROUT VERTICALLY H-STEEL BEAMS IN HOLES DRILLED THROUGH THE ROCK BENEATH THE CONCRETE WEIR SECTION TO RESIST THAT PART OF THE HORIZONTAL FORCES TENDING TO CAUSE SLIDING NOT RESISTED BY THE SLIDING FRICTION OF THE ROCK.

c) THE COEFFICIENT OF SLIDING FRICTION OF 0.3 IS TO BE USED DUE TO THE HIGHLY FRICTIONED ROCK, BOTH HORIZONTALLY AND VERTICALLY AND THE PRESENCE OF SEAMS WITHIN THE ROCK MASS.

\[
\text{Sliding Coefficient} = \frac{2H - \text{Fman} - \text{Ffric}}{2v}
\]

\[
= \frac{58.39^2 - 31.87^2 - 21.28^2}{0.17^2} = 21.24^2
\]

\[
\text{Sliding Coefficient} = 0.275 < \text{Allowable 0.3}\text{(OK)}
\]
Summary of Conclusion:

This report summarizes the analysis of the structural section. The following conclusions are drawn based on the analysis:

1. The maximum compressive stress at the section is [value].
2. The safety factor is [value].
3. The design checks were performed using the appropriate codes and standards.

Maximum Compression Stress:

\[ \sigma = \frac{P}{A} \]

where:
- \( P \) is the applied load
- \( A \) is the cross-sectional area

The results show that the section meets the design requirements.