MISSISSIPPI RIVER
DAVENPORT, IOWA

PHASE II

GENERAL DESIGN MEMORANDUM
LOCAL FLOOD PROTECTION

VOLUME 2 OF 2
APPENDICES
(REVISED)

FEBRUARY 1982

US Army Corps of Engineers
Rock Island District

MISSISSIPPI RIVER
DAVENPORT, IOWA

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LOCAL FLOOD PROTECTION

VOLUME 2 OF 2

APPENDICES

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US ARMY ENGINEER DISTRICT, ROCK ISLAND
CORPS OF ENGINEERS
CLOCK TOWER BUILDING
ROCK ISLAND, ILLINOIS 61201

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MISSISSIPPI RIVER
DAVENPORT, IOWA

LOCAL FLOOD PROTECTION
PHASE II
GENERAL DESIGN MEMORANDUM

APPENDIX A
HYDROLOGY AND HYDRAULICS
(REVISED)

US ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
CLOCK TOWER BUILDING
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PHASE II
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APPENDIX A
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APPENDIX A
HYDROLOGY AND HYDRAULICS

1. General

1.1 Purpose. The purpose of this supplement is to provide a basis for the approval of the interior flood prevention plan for the proposed flood control project at Davenport, Iowa. The proposed line of protection is shown on plate A-1.1 and the Plan of Improvement is shown on plate A-1.2. The Rock Island District submitted a Phase II GDM to NCD in October 1979. Subsequent review resulted in the requirement for further interior flood prevention analyses. This report is a summary of those analyses.

1.2 Scope. This supplement discusses the procedures and results of hydrologic and hydraulic investigations associated with the selection of an interior flooding prevention plan. The plan involves selection of numerous improvements including interceptor sewers, gatewells, culverts, and pump stations to evacuate trapped interior runoff. Physically, the project line of protection extends along the Mississippi River from near Locks and Dam No. 15 downstream to Highway I-280. Levees and channel improvements along Black Hawk Creek, proposed in the Phase I GDM, have been dropped from the proposed plan because of a lack of economic feasibility. The improvements along Black Hawk Creek are independent of the proposed project.

1.3 Project Site. The city of Davenport, Iowa, population 100,000, is part of the Iowa-Illinois Quad Cities which has an aggregate population of 350,000. Like the other Quad Cities, and many other cities, Davenport began as a "river town." It expanded early into the level plains above the river valley, but Davenport's manufacturing and commercial heart remained in the flood plain along the Mississippi River. Recent development of outlying shopping centers has diverted much traffic from the downtown. Nevertheless, this area remains vital to Davenport despite the fact that it is susceptible to flood disruption. Hence the demand for this project.

2. Climatology. Climatological data is available at National Weather Service Station at the Moline Airport and at Locks and Dam 15 at Davenport, Iowa. The climate is generally temperate with hot summers and cold winters. The bulk of the precipitation occurs during spring and summer. Mean monthly climatological data is presented on table A-2. Rainfall data--presented on plates A-2.1 through A-2.14--include rainfall envelope, seasonal rainfall

and flow distribution, daily rainfalls during closures, important hourly storms, and storms of various frequencies. In addition, plate A-2.15--taken from US Weather Bureau Technical Paper No. 40 (1961)--portrays monthly rainfall probabilities. The monthly probabilities are then integrated to derive a monthly probability of receiving intense rainfall. This computation is summarized on plate A-2.16.

3. Hydrology

3.1 General - Mississippi River. The drainage area of the Mississippi River at Davenport is 88,500 square miles, plus an additional 10,700 miles added with the confluence of the Rock River at the downstream end of the project. The Rock River, because of its relatively smaller drainage area, does not cause flooding by itself at Davenport, but may aggravate flood problems on the Mississippi River with its contribution of extra flow. Davenport is also the site of Locks and Dam No. 15, which is located at the upstream tie of the project.

3.2 Floods of Record. The Davenport gage (since 1934 called the Locks and Dam No. 15 tailwater gage) was established 1 January 1879. Several flood flow measurements have been made at Davenport, but for the most part flows are taken to be similar to those at Clinton, Iowa, where the drainage area is almost the same, and where flow is regularly measured. The 10 largest floods of record (thru 1975) are tabulated on table A-1. Flood hydrographs for the years 1951-1975 are shown on plates A-3.1 and A-3.2.

3.3 Flow Duration and Rating Curves. The discharge rating curve for the gage at Locks and Dam No. 15 tailwater (river mile 482.9) is shown on plate A-3.3. Stage-duration curve at Lock and Dam No. 15 is shown on plate A-3.4. Elevation-duration profiles in the project area are shown on plate A-3.5.

3.4 Flow Frequency. The peak discharge-frequency curve for the above gage is shown on plate A-3.6. This curve was based on the report, "Generalized, Regionalized Flow Frequency Study on the Mississippi River from Guttenberg, Iowa, to Hamburg Bay, Illinois," (March 1966, approved by Office, Chief of Engineers on 13 May 1966). This curve does not reflect Water Resources Council Bulletin No. 17A, "Guidelines for Determining Flood Flow Frequency," since formulation of this project predates this March 1976 reference.

3.5 Water Surface Profiles. Flow frequency is discussed immediately above. Using the thus-derived 0.5 percent annual exceedence (200 year) event flow of 389,000 cfs, backwater profiles for levee height determination were accomplished using the HEC-2 computer model. In addition, channel and valley cross sections, bridge geometry, and roughness coefficients comprise the input needed for the backwater model. Channel data was obtained from sounding surveys and channel inspector reports. Overbank data were obtained from topographic maps having 1- or 2-foot contour intervals and a horizontal scale of 1-inch equals 200 feet. Bridge geometry was obtained from construction plans. The sections were

typically 0.3 to 0.5 miles apart except at bridges where they were closer. Roughness coefficients of 0.027 for the channel and 0.065 for the overbank were obtained from verification of the 1965 flood. Expansion and contraction coefficients were 0.3 and 0.1, respectively. These values were regarded as appropriate for a river of this size (0.5 to 1.0 miles across), especially since the river is doing very little expanding and contracting. Starting water surface elevations for the Mississippi River were taken from a 1966 study performed by the Corps of Engineers. That 1966 study was performed in conjunction with development of Mississippi River profiles. The above described backwater analysis substantially verified these 1966 profiles. In the downtown area of Davenport, the two profiles were within 0.15 feet of each other. Along Credit Island, departures increased to an average of 0.3 feet with a maximum of 0.4 feet. Levee profiles and affected structures are to be built to the higher (backwater) elevations. Levee freeboard is nominally 3.0 feet except in the upstream reach where the freeboard is increased to 4.0 feet in accordance with paragraph 3c of Civil Works Engineer Bulletin 54-14 and paragraph 12a of EM 110-2-1601 (1 July 1970), Hydraulic Design of Flood Control Channels. Freeboard selection is described in paragraph 3.8. Profiles of the levee design grade and of both the 1966 study and HEC-2 backwater 0.5 per annual exceedence (200 year) event flood profiles are shown on plate A-3.7. On the same plate is plotted the Standard Project Flood which is described immediately below.

3.6 Standard Project Flood. The flow of the Standard Project Flood for the Mississippi River from Dubuque, Iowa, to Hannibal, Missouri, is derived on plate A-3.8 by comparing estimates of maximum events for locations in the Mississippi River drainage area. The maximum discharges of record were plotted against drainage area on log-log paper for important gaging stations on the Mississippi River from Mankato, Minnesota, to St. Louis, Missouri. A straight line envelope was drawn forming the maximum discharge of record. The estimated Stanadard Project Flood for both Mankato and St. Louis was plotted on the same grid. A line was drawn parallel to the maximum discharge of record line to closely fit the plotted points. This line is the Standard Project Flood envelope line. Intersecting this envelope line with the drainage area of the Mississippi River at Davenport, Iowa, the discharge of the Standard Project Flood at Davenport is thus directly estimated to be approximately 465,000 cfs. This flow is 52 percent greater than the flood of record, and 17 percent greater than the selected design flood. The Standard Project Flood profile was computed using the HEC-2 backwater model described above and is plotted on plate A-3.7.

3.7 Effect of Project on Other Quad-City Projects. The proposed Davenport Local Flood Protection project would reduce the freeboard to a minor extent on the existing Rock Island project. The 50-, 100-, and 200-year flood profiles on the Mississippi River incorporate confined conditions, including flood control projects at all of the Quad Cities. However, the proposed Davenport encroaches upon this confined condition in that the Credit Island Slough was included within the line of protection to serve as a ponding area for Black Hawk Creek. This encroachment increases the stage of the 200-year flood by 0.04 to 0.13 foot, thus

reducing the freeboard of the existing Rock Island Project by a relatively small amount. Had the Davenport line of protection also included all of Credit Island, the loss of freeboard of the Rock Island project would have been substantially greater.

3.8 Freeboard Design. Freeboard was designed in accordance with Civil Works Engineering Bulletin 54-14 and EM 1110-2-160: Hydraulic Design of Flood Control Channels. Generally, freeboard is provided to ensure that the desired degree of protection will not be reduced by unexpected factors such as embankment settlement, restriction of the channel, unusual wave action, trash accumulation at bridges, or shifting rating curves. At Davenport, the adjoining Mississippi River channel is approximately 1/2-mile wide. Channel structures within the project area include three bridges and Lock and Dam No. 15. Backwater analyses using the HEC-2 computer model indicate minor (less than 0.1 foot) losses at the bridges and a loss of 0.7 foot at the lock and dam. All losses closely agree with observed losses. Trash accumulation upon these structures has not been a problem, even during the 1965 flood which was 4 feet higher than any event then on record. Lock and Dam No. 15 is by far the most susceptible to trash accumulation, but a rail-mounted crane has access to the entire dam. A study of past floods was conducted to investigate the possibility of shifting rating curves. The Mississippi River - which is founded on rock through the project area - was found to be stable. Wave wash was found to be limited because the river in the project reach is curved, reducing wave fetch to 2.9 miles and to negligible amounts in the direction of prevailing winds. Riprap is to be provided to protect the levee against wave wash in affected reaches. Settlement and consolidation is expected to be fairly high, i.e., 7 to 20 inches as described in appendix B, but this is to be compensated by overbuilding the embankment 8 to 10 percent. It is expected that 70 to 80 percent of the settlement will occur in the first year. Since the entire project is urban in nature, a freeboard of 3.0 is selected in accordance with paragraph 12(3) and consideration of the above. In accordance with same reference and Civil Works Engineering Bulletin 54-14, additional (1.0 foot) freeboard is provided in the upper reach, ending below the Centennial Bridge. These results are consistent with the existing Rock Island, Illinois, line of protection across the river.

4. Occurrence of Interior Flooding

4.1 General. High Mississippi River stages after construction of the project will result in closure of most outlets of the city storm sewer system. Closure of 1 to 2 months duration is likely during major floods. Such floods may be expected only infrequently, say once every 15 years. The effects of past flooding have been serious; the downtown area has undergone deterioration. Development and redevelopment has been stunted. A major employer has threatened to relocate (and has since closed down part of its operations). In addition to extensive industrial and commercial disruption, several hundred homes have been flooded.

4.2 Project Conditions. Without adequate interior flood prevention facilities, many of these same problems caused by high stages on the

Mississippi River will occur despite a line of protection. Normal rains occurring during long closures will result in prolonged traffic disruption, water on the first floor of scores of homes, extensive basement flooding, inundation of hundreds of acres, and flooding of large areas adjacent to the downtown area. Above normal rains would result in levels of structural damage unexpected in a protected area.

4.3 Construction of the project will result in a new, quite important condition: sudden arrival of runoff. Before construction of the project, there was several weeks warning that heavy snows in Wisconsin and Minnesota posed a flood threat. During the flood, the river stages rise typically a half foot per day. City street crews close affected streets and parking lots as rising waters take hours to cross yards of pavement. However, during blocked-gravity conditions, runoff would arrive very rapidly. In both Main and Gaines Street areas, runoff would arrive in large quantities in 30 to 40 minutes. At Credit Island Slough, major rises would occur in approximately 3 to 4 hours. Operation of the project would necessarily account for these short action times.

5. Interior Drainage Subareas

5.1 General. Drainage areas will be discussed as they pertain to each pump station for ease of continuity to the reader. As will be seen, the areas served by the three pump stations are rigidly delineated by existing pressure sewers that the city constructed in late 1960's in anticipation of the project. The presence of these pressure sewers prevents the continuity of interceptor storm sewers. Overland flow to another pump station might provide some relief, but would not evacuate low-lying sewers, which--full of water--would interfere with building drainage. Moreover, effective overland flow is largely precluded by rail switchyards. Drainage areas are shown on plates A-4.1 and A-1.2. Summary of unit hydrograph input by Clark's method is shown on plate A-4.2. Derivation of input parameters is discussed below in paragraphs 6.1, 7.1, and 8.1. Pump stations (and their associated areas) are discussed immediately below.

5.1.1 Drainage Area - Main Street Pump Station. Runoff from Area 1, shown on plate A-4.1, drains through several storm sewers which empty directly into the Mississippi River. During high stages on the Mississippi River, these storm sewers will be blocked with the closure of gatewells. Storm runoff will be gathered by interceptor sewers and evacuated by a pump station at Main Street. Area 1 consists of 108 acres and is entirely urban. Area 1 is mostly level, extensively sewered, and 20-25 percent impervious. The low-lying portion of Area 1 contains 700 parking spaces. Loss of this parking sharply curtails business in the nearby downtown area of Davenport since there is simply no substitute parking within reasonable distance. Minor stages of interior flooding in Area 1 result in closure of mainline tracks of the C.M.St.P.&P. Railroad. Closed at the same elevation is River Drive, which is discussed below.

5.1.2 Drainage Area -- Gaines Street Pump Station. The Gaines Street pump station serves Area 2, 360 acres. In addition, during heavy rains coincident with blocked gravity drainage, the Gaines Street pump

station must also evacuate runoff that exceeds the capacity of pressure sewers serving Areas 3B and 3C. These areas are discussed below in paragraph 5.1.4. Areas 3B and 3C are hydrologically homogeneous with Area 2 in that they are generally upland areas of moderate slope, are sewered, and contain approximately 15-20 percent impervious surfaces. The low-lying portion of Area 2--that where ponding occurs--contains only one through street, numerous railroad tracks, and parking. However, the street is River Drive, the city's principal along river traffic artery; the trackage is mostly abandoned--occupying optimally located land which would be developed except for its flood proneness; and the parking is vital to the commercial and service functions of the adjacent downtown. Monetary damages due to interior flooding would not be great from even moderate storms, but disruption will be extensive, and the sight of expanses of water behind the project levee will certainly cause problems in public perception.

5.1.3 Drainage Area - Credit Island Pump Station. The Credit Island pump station evacuates quantities of interior runoff hugely greater than either of the Main or Gaines Street pump stations discussed above. Those pump stations serve drainage areas, respectively, of 108 and 360 acres; the Credit Island pump station serves 7,860 acres. Most of the area is contained in Area 5, which is the watershed of Black Hawk Creek, 6,700 acres. Area 5 is mostly agricultural, hilly, and is relatively steep. Area 4 is half rural and half urban, and is drained by McManus Creek. Area 4A is entirely urban, mostly level, and approximately 10 percent impervious due to rooftops and paving. The low-lying portions of these areas--where ponded runoff becomes a problem--are extensive. Interior flooding in Areas 4 and 4A closes River Drive and Mainline C.M.St.P.&P. Railroad tracks and inundates parking and access to several commercial firms. In Area 5, the same interior flooding blocks access to several buildings along the Credit Island Slough and causes structural damage. The most important effects are indirect; high stages cause backwater up Black Hawk Creek which seats flapgates on sewers draining the Garden Addition, discussed below in section 9. Subsequent rainfall would flood several dozen homes.

5.1.4 Drainage Areas--Pressure Sewers. During both gravity drainage and blocked gravity drainage, runoff from Areas 3A, 3B, 3C, and 3D are drained directly to the Mississippi River by pressure sewers. These areas are entirely urban, contain bluff portions, and have no low-lying portions. As mentioned above, during heavy rainfall, runoff will exceed the pressure sewer capacities and will flow to low-lying portions of Area 2.

5.2 Impacts--No-Pumping. Providing no pumping generally renders the entire project ineffective except for major floods. According to a Period of Record Analysis based upon 100 years of data, there would be little or no reduction of interior stages except during major floods. This conclusion is qualified and quantified immediately below.

5.2.1 Main Street and Gaines Street Pump Stations. Before proceeding, it will be noted that several considerations are called up here without prior reference. These include the Period of Record Analysis and several

hydrologic-hydraulic parameters. These will be discussed below in detail. Hence, several observations are be made now and will be evolved later with technical detail. First, at the 10 percent annual exceedence (10-year) event, the levee project without pumping will reduce peak stages in the interior area by 2.0 feet, as shown on plate A-5.1. It is not until the 4.0 percent annual exceedence (25-year) event, that the net reduction in the unmodified river stage exceeds 3.0 feet in the interior area. Beyond this point, reductions increase rapidly; up to this point, however, the project has had only minor effect.

5.2.2 Credit Island Pump Station. At the Credit Island pump station, as shown on plate A-5.3, construction of a levee without pumping lowers the peak stage in the interior less than a foot at the 10 percent annual exceedence event (10-year flood). It is not until a 3.0 percent annual exceedence event (33-year flood) that does the difference between the project without pumping and the flooding river stage exceeds 2.0 feet. Not only does the project without pumping fail to significantly reduce interior stages for less than major floods, but high interior stages endure for 2 months.

5.3 Impacts of Storm Water Ponding. Ponded storm water impacts upon public perception, transportation, social, environmental, and other concerns. The impacts upon all of these areas are greatly exacerbated by the long durations of ponding. It is one thing to briefly flood a major street; it is quite another to permit that same street to be flooded for several weeks. Accordingly, information from the Period of Record Analyses referenced above has been compiled to derive ponding duration frequences for the Main-Gaines Street areas and at Credit Island Slough, shown on plates A-5.3 and A-5.4, respectively.

5.3.1 Public Perception. Floods on the Mississippi River usually last several weeks, are typically derived from snowmelt in Minnesota and Wisconsin, and usually crest in the Quad Cities in early May. This time of year is also marked by spring thunderstorms. Since major plant growth has not begun, runoff may be high. In addition to rainfall, the long flood may mean significant volumes of seepage. Allowing rainfall and seepage to accumulate would mean closure of a major street, cessation of rail service on mainline trackage, loss of parking, and the presence of 90-120 acres of water standing for many weeks adjacent to the levee adjacent to the downtown. At the other end of the project, the combination of high interior stages and rainfall would flood the Garden Addition housing again--just as before the project. In fact, public perception might be that the entire project is functioning in this manner.

5.3.2 Transportation. Several impacts occur at a moderate interior stage of El. 560.4--which is a 11.0 percent annual exceedence (9-year) event according to the Period of Record Analysis referred to above and discussed below in section 6.5. These include the closure of River Drive (the major along--river artery), flooding out of siding and mainline tracks of the C.M.St.P.&P. Railroad, inundation of 960 parking places, and blockage of access to several industrial concerns. These impacts, while associated with relatively minor stage-damages, become serious with

prolonged duration. Loss of parking for a day may mean a delayed shopping trip, but the loss of parking for three weeks means a loss of business. Thus the downtown area of Davenport is affected importantly--though indirectly, simply by loss of parking, since there is no substitute parking available. Similarly, with River Drive closed because of ponded waters, traffic would be necessarily routed through the downtown causing further disruption. Prolonged loss of rail service and access would likewise add to commercial and industrial disruption. The city experiences these problems during floods without a project.

5.3.3 Downtown Development. The city of Davenport is urgently concerned about the future of its downtown. In fact, the protection of the downtown and elimination of flood-associated disruption is certainly the major motive of the city's pursuit of the project. However, even with the project, the city may find development difficult--if adequate pumping is not provided. This is because without pumping, the 100-year peak interior stage still floods large areas of level land near the line of protection, as illustrated on plate A-5.5.

5.3.4 Economic Impacts. Construction of the project, but not providing pumping of interior runoff, will result in 38 percent of preventable damages being "left on the table" for all events up to the 50-year flood. The closure of River Drive and other impacts describe above in para 5.3.2, might occur only once every 15 years, but the levee begins its protection only at a 7- or 8-year (12.5 percent annual exceedence) event.

5.3.5 Other Impacts. The prolonged presence of large areas of ponded water is likely to have several other impacts, including increased infiltration into sanitary sewers, odor problems, and possible contamination (and dispersal) from point pollution sources. Long reaches of runoff ponded adjacent to the levee would hinder the flood fight; for example, if a pressure boil developed, it could go undetected in its early stages by the already standing water. The ponded waters would delay and hinder movement of work force and material at a critical moment.

5.3.6 Major Flood - Scenario. Up to this point, impacts have been attributed only to floods of 2 percent annual exceedance (50-year) events and less. However, it would be illustrative to consider the impacts of a 0.5 percent annual exceedence (200-year) flood. This would be a full test of the project, but the project would be expected to perform reasonably well. The first step is to construct a hypothetical 0.5 percent annual exceedence (200-year) flood in order to establish the length of gate closure. This construction, shown on plate A-5.6, was derived using a 0.5 foot/day rise, 2 days at crest, and a 0.4 foot/day fall. These parameters are consistent with past floods; the 1965 flood hydrograph is also plotted on plate A-5.6 for comparison. The resulting gate closure is 34 days near Gaines Street and 46 days at Credit Island. April and May rainfall average 3.02 and 3.87 inches, respectively. Runoff of 1 inch per week would not be unreasonable. With computed seepage and uniform runoff, interior stages rise steadily, as shown on plate A-5.7. The material consequences are discussed below.

5.3.6.1 Impacts - Main Street and Gaines Street Areas. Several impacts may be immediately observed. Closed for 22 days are River Drive, switchyards, the C.M.St.P.&P. mainline tracks, 700 parking spaces, and the Municipal Stadium. As mentioned above, closure of these facilities seriously curtails the commercial activities of the adjacent downtown area of Davenport. Traffic from River Drive must be routed through the downtown; would-be shoppers and employees have no place to park; transfer of goods is impeded. During these 3 weeks, primary access to Oscar Mayer & Company, a major employer, is blocked, and the same firm must pump its lower levels to keep functioning. In the area adjacent to the downtown, there would be 90 to 120 acres under water.

5.3.6.2 Impacts--Credit Island Area. In the Credit Island Slough, interior stages start at lower levels than at Gaines Street area and rise more slowly. Until elevation 560.5 is reached, Credit Island Slough may receive overland from the Gaines Street area. However, above El. 560.5, i.e., after approximately 24 days of closure, this flow is reversed: water from the overflowing Credit Island Slough is backing into the downtown. The impacts described there are thus worsened. Added to the losses described above is the municipal garage, which was the headquarters of the flood fight. However, by this time, in the lower portion of the project, 200 homes in the Garden Addition would have been affected. Throughout the city, interior flooding would have spread to more than 300 acres.

Interior Drainage Analysis

The impacts described above warrant further investigation of pumping. Accordingly, analysis of interior flood prevention will begin with the description of hydrologic characteristics of the subareas and continue with description of existing sewers, gravity design, and pump station selection. For ease of reading, related material will be kept together as much as possible; each area will be described and its associated pump station evolved in the same section.

This section necessarily contains much descriptive material. Key pump capacity selection paragraphs are 6.6.6 and 6.6.7 (at Gaines Street), 7.3.3 and 7.4 (at Main Street), and 8.6.3 and 8.7 (at Credit Island).

GAINES STREET PUMP STATION

6. General.

Area 2, which is served by the Gaines Street Pump Station, will be considered ahead of Area 1. This is because, while the two areas are adjacent and share ponding at higher ponding elevations, Area 2 is much the larger and its problems dominate. Therefore, the pump stations for Areas 1 and 2 will be seen to have similar criteria; it is logical to begin with Area 2 since its criteria will largely suffice for Area 1.

6.1 Area 2 Hydraulic Characteristics. As discussed above in paragraph 5.1.2, Area 2 is urban, contains approximately 15-20 percent impervious surfaces, and is sewered. Drainage area limits were determined using terrain contours and city sewer maps. Contours were used to determine area-capacity curves, shown on plate A-6.1. Above El. 561.0, the area is poorly defined, with ponding slowly "leaking" overland toward Credit Island Slough. Capacity includes 1.0 acre-feet of substreet storage in existing sewers. The sewers would tend to accelerate runoff, but observation during rainfall indicate that runoff ponds at generally undersized inlets for several minutes. This fact was incorporated into the development of the unit hydrographs. Input for derivation of unit hydrographs according to Clark's Method is portrayed on plate A-4.2. Six-hour rainfall series were applied; the resulting hydrographs are tabulated on plate A-6.2. The drainage system in Area 2 is portrayed on a sewer schematic; gravity drainage on plate A-6.3, and blocked-gravity on plate A-6.4. Discussion of analysis of capacity of existing sewers is described below in paragraph 6.1.1. As mentioned above in paragraph 5.1.2, Area 2 receives runoff also from Area 3C when runoff during heavy rains exceeds the capacity of the Marquette Street pressure sewer. Discussion of this pressure sewer follows below in paragraph 6.1.2 and seepage in paragraph 6.1.3.

6.1.1 Gravity Outlets. Rating curves for the existing gravity outlets in Area 2 are shown on plates A-6.5.1 thru A-6.5.4. The ratings were computed in accordance with EM 1110-345-284 (14 August 1964). The tailwater of all of the gravity outlets was deeply submerged; outlet control was the design flow condition. Two tailwaters, at 50 percent and the 2 percent duration, were examined. The 2 percent tailwater corresponds to conditions concurrent with the 1973 flood. The nomograph on Plate I-9 of the above-referenced EM was employed in the solution of the equation:

$$H = \frac{2.52(1+Ke)}{D4} + \frac{466.2n^2L}{D16/3}\frac{Q^2}{10}$$

Where H = head in feet, Ke = entrance loss coefficient, D = pipe diameter in feet, n = Manning's roughness coefficient, L = pipe length in feet, and Q = discharge rate in cfs. The roughness coefficient was taken to be 0.012 and pipes were typically 150-200 feet long. Inlet loss coefficients were taken to be 0.5, best descriptive of socket ends of reinforced concrete pipe in manholes.

6.1.2 Pressure Sewer. Area 3B runoff that exceeds the capacity of the pressure sewer from Marquette Street flows into Area 2, and hence must be picked up by the Gaines Street Pump Station. The capacity of this pressure sewer--a 66" RCP at the point of pressurization on West 4th Street--was analyzed in accordance with EM 1110-345-284 (14 August 1964). Preliminary computations indicate that velocities at capacity will be approximately 8 feet per second (fps). Therefore, the capacity of the Marquette Street pressure sewer was estimated by testing if there was sufficient head to maintain eight fps. Available static head is the difference between the top of the protection and the 25-year flood stage (selected here as a representative condition), i.e. El. 571.9 minus El. 562.9 or 9.0 feet. Flowing full under pressure, the barrel friction loss is 6.17 feet, exit loss is 0.5 feet, manhole losses are 1.0 feet, and additional transitional losses are estimated to be 0.8 feet. There is no entrance loss since the water is already in the sewer and already has velocity at the point of pressurization. Total losses are then 8.47 feet, which is less than the available head of 9.0 feet. Therefore, 8 fps is taken to be a reasonable velocity; capacity is directly computed to be 180 cfs, the product of pipe area and the velocity. Runoff in excess of the 180 cfs capacity of the pressure sewer "spills" downhill to the pump station. This spillage increases sharply with increasing rates of runoff and is part of the Area 2 hydrographs tabulated on plate A-6.2. A profile of the Marquette Street pressure is shown on plate A-6.2.1.

6.1.3 Seepage. Maximum seepage into Area 2 is 1,400 gpm at 12.6 feet head. Seepage begins at El. 559.0 and increases linearly with increasing head. This rate is not high, but during long floods, the total volume would be significant. For example, during the hypothetical 200-year flood constructed on plate A-5.6, seepage totaled 70 acre-feet.

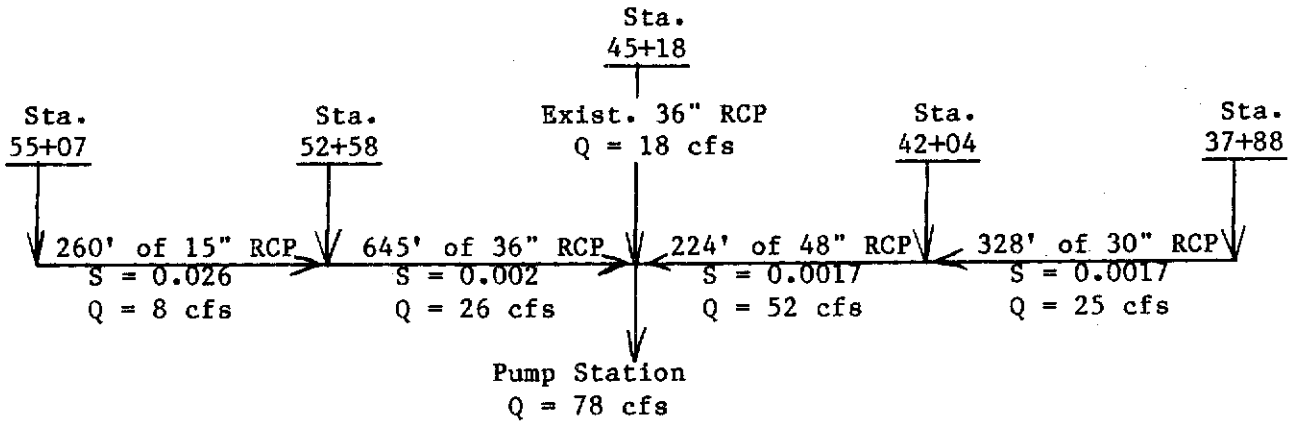
6.2 Storm Sewers

6.2.1 Existing Situation. Runoff from Area 2 drains to the Mississippi River via several storm sewers. Rating curves for these sewers are contained on plates A-6.5.1 to A-6.5.4. A composite rating is portrayed in plate A-6.5.6. During heavy rains, runoff accumulates in the streets and ponds at inlets.

6.2.2 Proposed - Gravity. The existing city storm sewer system will not be affected by the project. Existing gravity capacity will be supplemented by openings in the floodwall at Scott, Western, and Gaines Streets. The existing gravity capacity was examined using the rating curves on plates A-6.5.1 through A-6.5.4. The river level was assumed to be at El. 555.0 (similar to the major storm events of 1973) and a 100-year storm as routed through secondary ponding available to the areas served by the Gaines Street pump station. The maximum stage would be elevation 560.7 at Gaines Street, and River Drive would be closed for 1 hour. Minor street flooding and ponding has been observed to occur during heavy rains under existing gravity conditions. Gravity ratings for the 100-year event is shown on plate A-6.5.1.

6.2.3 Proposed - Blocked Gravity (Interceptors). During high Mississippi River stages, the existing storm sewers will be blocked by the closure of gatewells to be constructed as part of the project. To provide positive drainage, an interceptor sewer is proposed. The capacity of the intercepting sewer was designed to at least equal the capacity of the proposed pump station, discussed below. Design head was measured from basin inverts since surcharge of the existing sewer system is not an infrequent occurrence. A Manning's roughness coefficient "n" of 0.012 was used. The sewer was assumed to be flowing full. Layout, slopes, lengths, flows, and inflows are portrayed in the schematic below. The interceptor will not add to the capacity of the existing storm sewer system during gravity conditions. The profile of the interceptor is shown on Plates 19, 20, and 21 of the main report.

INTERCEPTOR SCHEMATIC
Gaines Street Pump Station
(Blocked Gravity Drainage Only)



6.3 Performance Criteria--Interior Flooding Control. Performance specifications for the interior flood control plan must be chosen carefully. These specifications--or criteria--form a basis for judging the adequacy of any proposed plan. If the interior flood control design criteria are not reasonable, then it is hardly likely that the selected design will be reasonable. Therefore, before initiating hydrologic design of the pump station and other facilities, these criteria will now be carefully evolved and selected.

6.3.1 Range of Runoff Events Criteria. It is unnecessary and perhaps unadvisable to select a specific runoff event with which to estimate pumping requirements. Various factors are considered in selecting a range of runoff. From plate A-3.5, it is seen that the gate closure elevation of 557.0 corresponds to an elevation-duration of 1.5 percent. The coincident probability of receiving a 1.0-year storm during a 1.5 percent elevation-duration closure is once in nearly 70 years. Therefore, the blocked-gravity equivalent event (storm) that would have a 0.5 percent change of annual exceedence (i.e., 200 years) equals a 2.85-year annual rainfall event. In the same vein, it can be observed on plates A-3.1 and A-3.2 that major floods on the upper Mississippi River

typically occur between late April and June. From plate A-2.16, it can be seen that during these months, the occurrence of significant rainfall is twice as likely as average. Therefore, doubling the above probabilities extends the upward range to a 5.7-year storm. These considerations are not intended to be precise computations, but rather merely an indication of a suitable range of runoff. These considerations indicate that runoff derived from storm magnitudes ranging from 1 to 5 years appear to be reasonable.

6.3.2 Maximum Allowable Stage Criteria. While there appears to be no distinct low elevation at which ponding must urgently be prevented, examination of stage-damages do indicate a limiting elevation. Examination of plate A-6.6 points toward El. 561.0 as a limiting elevation. At El. 561.0, stage damages increase abruptly; in the next foot of rise, damages rise 130 percent. However, before El. 561.0, serious disruption has already occurred; River Drive was closed at a elevation (El. 560.4), but since the duration of closure is the key consideration, this time aspect is discussed separately in the next paragraph. For a Maximum Allowable Stage, use El. 561.0.

6.3.3 Allowable Interior Stage-Duration Criteria. It is desired to limit durations to avoid to interrupting auto traffic for excessive periods. Traffic disruption is not gradual; a stage versus traffic disruption plot would resemble stair steps, with each step occurring as another street is closed. In this case, we need consider only two "steps." The first step occurs at El. 560.4 when River Drive is closed and the second at El. 564.0 when Second Street is closed. A curve, unquantified of course, is shown on plate A-6.7. However, since the second step requires 6 inches of accumulated runoff, only the first will be considered. Only 1.2 inches of ponded runoff closes River Drive. Therefore, El. 560.4 is the one street closure stage at which ponding durations shall be limited. The length of the allowable ponding durations is discussed in the next paragraph.

6.3.4 Ponding Durations Criteria. Maximum durations of ponding are almost entirely a matter of judgement since intangible factors dominate their choice.

6.3.4.1 River Drive Closure Duration. River Drive is Davenport's major along-river traffic artery: a major thoroughfare, four lane open access, and also Highway US 61. It would be often regarded as poor practice to allow such a route to be closed for more than one hour. However, alternate routes are available--although with major inconvenience during rush hours. Accumulated delays of 15 to 45 minutes at intersections would be typical, depending upon the time of day. Also, alternate routing of traffic would cause commercial disruption. However, as mentioned above, the Period of Record Analysis indicates that closure would be infrequent, say once in 15 years. In view of this infrequency, a maximum closure of 3 hours was selected. A shorter closure was not felt necessary because closure (of 3 hours) and rush hour would coincide only 50 percent of the time during weekdays. On the other hand, it was felt that closures much longer than 3 hours would disparage public perception of the project.

6.3.4.2 Time to Evacuate Ponding. The second stage-duration consideration is the maximum time to evacuate ponding after receipt of runoff. While the closure of River Drive duration is a single value, in this case a range of drawdown times is appropriate since a range of runoff is being considered. Since drainage of parking lots, commercial storage (i.e., lumber yards), and railage, the maximum time to draw ponding back down within normal pump cycling stages (El. 557.0) is tentatively selected to range between 6 and 12 hours. This range is regarded as tentative since the actual selection of pump station capacity may include trade-offs between criteria to attain optimal results.

6.3.5 Pump Station Operating Elevations. Layout of interceptor sewers in Area 2 dictated a sump elevation of El. 543.5 at the Gaines Street Pump Station. Derivation of gate closure, the pump activation stage, is straight forward. Indeed, the top of the existing Davenport "sea wall" is approximately El. 558.0. Closure was thus set 1.0 foot lower to allow for untimely rise in the river. Closure above El. 557.0 would risk unnecessary damage; closure below El. 557.0 would serve no advantage. Below El. 557.0 there is only pipe storage. To provide 10 feet of pump cycling and still maintain adequate pump intake submergence, the minimum sump level pump cutoff was set at El. 547.0. This selection will permit utilization of approximately one-half acre-feet of storage.

6.4 Pump Station Costs. New costs were prepared. It is believed that these costs represent a more realistic appraisal of future experience.

6.4.1 Total first costs are a synopsis of recent bid experience.

6.4.2 It was observed that energy and demand costs at a particular site will tend to be constant; no matter what the size of pump station, the same volume of runoff must be pumped. Furthermore, the city of Davenport is served by the Iowa-Illinois Gas and Electric Company which does not include demand (i.e., excess facilities) charges in its monthly billing. (The minimum monthly bill is presently only $4.00.)

6.4.3 Annual maintenance costs were based upon actual costs experienced at the city of Clinton. This Mississippi River community, 30 miles upstream of Davenport, has had four Corps pump stations in service for several years. Clinton's pumping situation resembles Davenport's in that it has a similar frequency of flooding and a similar low incidence of blocked gravity conditions.

6.4.4 Annual replacement costs were based on replacing pumps and motors every 33 years.

6.5 Methods of Pump Capacity Design. The Gaines Street Pump Station was analyzed (as were all the pump stations) using three methods: the Period of Record (POR) analysis, the Total Probability Method (IPM), and the Impact Trapezoid Method (ITM). These methods are described immediately below.

6.5.1 Period of Record (POR) Analysis. In the Period of Record (POR) analysis, the designer derives a partial duration series of peak-interior stages in the protected interior area that would have occurred if the project had been existent during the entire record of rainfall and river stage events. This record should be preferably 50 years long and may be transferred from a nearby location. In performing the POR analysis, the designer develops synthetic runoff hydrographs from the record of rainfall events. The runoff hydrographs are routed through peak interior stages, are then computed using a variety of different pump station capacities during blocked-gravity conditions. The resulting peak interior stages are then ranked and assigned plotting positions for a frequency curve. At Davenport, 101 years of record are incorporated into the anlaysis; it is expected that this length of record will "average out" the affects of rare and unusual events. At Davenport, gravity events were not considered since there is no attempt at claiming benefits from any gravity redesign. To do so would require redesign of the city sewerage system, an undertaking which is outside the scope of this project.

6.5.2 Total Probability Method (TPM). A second approach is the Total Probability Method (TPM). This approach uses hypothetical interior runoff events combined with an independent stage-duration curve for the river in solving the total probability theorem:

$$P(A) = \sum_{i=1}^{n} P(A/B_i)\, P(B_i)$$

where $B_1, B_2 \ldots B_n$ is a set of mutually exclusive, collectively exhaustive events.

$P(A)$ = the probability of exceeding a given pond elevation

$P(A/B_i)$ = the probability of exceeding a given pond elevation if the river is at an elevation (i).

$P(B_i)$ = the probability the river is at the specific elevation, letting i take on the full range of values, 1 thru n.

This approach has the advantage of requiring less observed data at a specific site as well as a rather straightforward analytical method which can be programmed. There are shortcomings, however, in the accurate development of local runoff hydrographs of sufficient duration to be certain that the proper total runoff volume is included. This is particularly true during long periods of high river stages and small or zero pump capacity. In addition to this, confidence in the results can only be obtained by comparing solutions with a Period of Record Analysis--thus increasing the time and manpower of the total study process. In this study, the Total Probability Method analysis was performed using the

Rock Island District, Corps of Engineers' computer program INDRN. This program is designed to analyze the effects of a flood hydrograph plus seepage routed through the ponding area for a series of river stages and pumping capacities. The computed outflow in the program includes either gravity or pumped discharge, depending upon input gate closure, interior pond and river elevations.

6.5.3 Impact Trapezoid Method. A third method offers the opportunity of incorporating the results of the POR and TPM (or any other method for that matter) into a single comprehensive pump selection framework. The Impact Trapezoid Method is not so much a novel analytical procedure as it is a graphical technique for displaying the impacts of various pump station capacities upon several design and performance criteria selected by the designer. The Gaines Street pump station is used as an explanatory example (at the risk of duplicating material brought out in later paragraphs). The Impact Trapezoid is constructed upon a routing graph, Maximum Ponding Elevation Vs. Pump Capacity at Various Runoff Events. An example is plate A-6.8. Selected impacting criteria are then added, resulting in plate A-6.9, and, in fact, a wider range of parameters has been supplied so that the reader may see the effect of different selections. (More discussion of these lines is in paragraph 6.6.5). Plate A-6.9 has become rather complex; therefore, plate A-6.10 is supplied which contains only the selected criteria. Generally, these criteria may include various limiting elevations and allowable drawdown times. At the Gaines Street (Area 2) pump station, the selected criteria included the range of hypothetical storms, the maximum length of closure at River Drive, and the maximum time to evacuate ponded runoff. The display of plotted criteria will usually form a closed region, a trapezoid, wherein there is a defined range of pump station capacities that meet all of the selected criteria. The Impact Traezoid is shown on plate A-6.10. Final selection of the pump station capacity is made within this defined range, usually with the assistance of other methods, e.g. TPM and POR analyses. Application of this method to the Gaines Street pump station is discussed in detail in paragraphs 6.6.5 and 6.6.6.

6.6 Summary of Pump Station Analyses. Pumping requirements for Area 2 were analyzed by all three of the methods discussed above. Considerable weight was given to the results of the Period of Record Analysis since the record incorporated 101 years of data. It will be seen that the Period of Record Analysis arrived at pump station capacities 20 percent larger than the Impact Trapezoid Method at all three sites in Davenport. These results are considered to be quite comparable. On the other hand, at all three sites, the Total Probability Method indicated benefits less than two percent of those computed by the Period of Record Analysis. Before taking up discussion of the results of these analyses, the site-specific procedures necessary to apply the method will be discussed.

6.6.1 Application of the Period of Record (POR). The Period of Record (POR) analysis required several preliminary activities. While there are 101 years of record, hourly rainfall is available only in the last 28 years. A method was needed to incorporate the entire record into the analysis or else a significant result would not be possible. Therefore, a method for extension was developed and then applied first to only the

last 28 years of record. The same 28 years of record were analyzed with the detail permitted by the record of hourly rainfall. The results of the two analyses were compared; the analysis using hourly rainfall indicated stage damages 29 percent greater than the analysis using daily rainfall. These results are interpreted as being comparable; therefore, the method used for incorporating the entire record into the POR analysis was judged to be satisfactory. This method included the following procedures:

a. Infiltration equals 30 percent.

b. Total seepage is the product of the length (in days) of closure and the mean seepage rate.

c. The selected maximum interior stage during any interior flood is the lower of (1) that resulting from accumulated runoff and seepage plus 1.0 feet, or (2) that resulting from the peak river stage.

While these procedures may introduce some imprecision of computation, the limitations of the data probably preclude greater accuracy. Compared with the more detailed analysis of the 28-year record, it appears that the above procedures understate damages. Rainfall and stage data are shown on table A-4 and the zero-pumping elevation-frequency curve is shown on plate A-6.11.

6.6.2 Pump Selection by Period of Record Analysis. Pumps of 5,000, 10,000, 20,000, 40,000 and 60,000 gpm were examined and elevation-frequency curves for each were derived. These elevation-frequency data were incorporated into a full economic analysis, described in detail in section 2 of the Economic Appendix (F). The optimum thus derived is a station capacity of 43,000 gpm. The Impact Trapezoid Method is used below to examine this solution within the context of site specific performance criteria; final pump slection is described in paragraph 6.6.7.

6.6.3 Application of the Total Probability Method. Analysis of the Total Probability Method (TPM) was accomplished using the Corps of Engineer's program INDRN. Several inputs are required. These include stage-damage curves, elevation-capacity curves, seepage rates, and gravity ratings. During simulation of blocked-gravity conditions, interior stages were allowed to exceed the outside river stage by an unreasonably high value before gates were assumed to be opened. This was to prevent excessive switching back and forth between pumping and gravity flow. This procedure overestimates benefits, but this problem could be looked at in greater detail if the situation warranted. Otherwise, the pump station was operated in accordance with the Pump Operation Schedule on Plate A-6.12.

6.6.4 Pump Selection by the Total Probability Method (TPM). As discussed above in paragraph 6.6, the Total Probability Method (TPM) was unsuccessful in this case; computed benefits were two orders of magnitude smaller than were those computed by the Period of Record Analysis. And as discussed in the paragraph immediately above, even these benefits were overestimated in application of the method. Since the results were so different from those obtained by the Period of Record Analysis (which incorporates 101 years of record), the TPM will be omitted from further consideration.

6.6.5 Application of Impact Trapezoid Method. The Impact Trapezoid Method requires three straightforward steps. First, several criteria are selected. These may be tangible or intangible. Second, each selected criteria is graphically portrayed on a pump routings plate. For example, if a certain peak interior stage is selected as a desired maximum, then the impact of this criteria would be represented by a horizontal line drawn upon the flood routings plate at that elevation. As discussed above, when all of the criteria are sketched on the routings plate, the resulting lines and curves often form a closed region, usually a trapezoid. This completes the second step. Third, from within the closed region, the optimum capacity of the proposed pump facility is selected. Pump selection thus entails logical consideration of the desired criteria. The criteria for the Gaines Street pump station, discussed above in section 6.3, are listed below for convenience.

a. Maximum Allowable Stage = El. 561.0

b. Allowable Interior Stage Duration:

(1) Three hours at El. 560.4 (River Drive closure)

(2) Six to 12 hours drawdown to El. 557.0

c. Runoff = 1 to 5 years

6.6.6 Pump Selection by the Impact Trapezoid Method (ITM). The Impact Trapezoid Method is a graphical rather than an analytical procedure; it results in a pump selection range rather than an optimized value. However, a single value solution may be obtained by careful weighing of previously selected criteria (listed immediately above). Selection is not straightforward; trade-offs between criteria are necessary. In this instance, the Impact Trapezoid, plotted on plate A-6.10, indicates that pump station capacities over a range of 17,000 gpm to 60,000 gpm of least partially meet all selection criteria.

Would the least capacity, 17,000 gpm, work? Yes, if only the 1-year runoff event is considered. But beyond that event, there are problems. At the 2 year, River Drive is closed for 3 hours--which satisfies selected criteria, but ponding drawdown requires almost 20 hours. While this is not outrageous, pump selection is highly sensitive to further increases in runoff. At the 5-year event, the River Drive closure duration zooms to 8 hours, and the peak elevation rises to El. 561.0, which is the limiting elevation because of high damages. Furthermore, ponding drawdown requires almost 28 hours, permitting excessive risk of stacked runoff events. Because of this extreme sensitivity to runoff above the 2-year level, it is felt that 17,000 gpm pumping capacity is inadequate.

Pursuing this line of reasoning, one can again refer to plate A-6.10 and note the lowest pumping capacity that meets all criteria at the 5-year level is 40,000 gpm. (In this point, it is seen that three of the five criteria fall nearly upon a triple intersection. Whatever the appropriateness of the criteria, they are consistent (albeit inadvertently) in

their influence upon the pump selection.) Could a lesser capacity be used? Yes; dropping back to 35,000 gpm, there is a slight compromise of the criteria to limit closure of River Drive, i.e., 4 hours, as can be seen on plate A-6.9. Other criteria are satisfied. Can even a lower capacity be used? Probably not. Closure of River Drive is increased to nearly 6 hours, and ponding drawdown exceeds 12 hours by a minor amount. However, while all criteria are still met at the 2-year level, again, in the upper range of runoff events, maintenance of selected criteria is deteriorating. Lower pump capacities need not be considered.

In summarizing this rather long section, a recap would be convenient.

* The pump selection range was from 17,000 gpm to 60,000 gpm.

* 17,000 gpm capacity is sensitive to runoff events above the 1-year level.

* 40,000 gpm is the lowest pumping capacity that meets all selected criteria at the higher range of selected runoff events.

* 35,000 gpm still substantially meets all criteria, although River Drive closure duration increases to 4 hours at the 5-year runoff event.

* 30,000 gpm is less successful in meeting selected design criteria; River Drive closure increases unacceptably to nearly 6 hours.

Therefore, by a process of elimination via a careful consideration of selected design and performance criteria via the Impact Trapezoid Method, one arrives at a pumping capacity of 35,000 gpm. This capacity may be regarded as the minimum capacity that meets all of the selected criteria.

6.6.7 Final Pump Selection. At this point, the results of the Period of Record Analysis and the Impact Trapezoid Method may be brought together. The Impact Trapezoid Method (ITM) indicates that all selected performance criteria may be met with a pumping capacity of 35,000 gpm. However, the Period of Record Analysis indicates that a selection of 43,000--approximately 20 percent greater--is economically optimal. This difference is not a problem: interpretation of these results indicates that while a 35,000 gpm pump capacity is adequate at this site, a larger 43,000 gpm pump capacity would produce a worthwhile increase in benefits. On plate A-6.10 it is also noted that 43,000 gpm pump capacity is near the center of the Impact Trapezoid. Thus, this selection is reasonable under both the POR and ITM methods. Select 43,000 gpm.

6.7 Examination of Selected Pump Capacity. Examine the pump station under a 100-year and Standard Project Floods. Plate A-6.9 indicates that a 100-year storm, a 240 percent overload, would result in a stage of El. 561.6 with River Drive blocked for over 12 hours. Referring to the stage-damage curve on plate A-6.6, it is seen that economic losses are $550,000. This amount includes damage to vehicles in low-lying parking lots and other sudden inundation effect as discussed above in paragraph 5.

While such an event would be a major economic loss, no loss of life would be threatened. A Standard Project Storm involving some 13.8 inches of rain cannot be considered as an isolated effect only in the area protected by the Gaines Street pump station; ponding there would merge with flooding from the Credit Island Slough. The maximum stage would be El. 563.5 with disruption above El. 560.5 in excess of 4 days, and $1,460,000 stage damages in the upper downtown. However, again, no loss of life would be threatened.

6.8 Effects of the Pump Station. Short term effects of the pump station are displayed on plate A-6.13. Long-term effects would include increased land values and increased use of the low-lying areas. This would result in an increased liability to damages. The effects of the pump station cannot be entirely separated from that of the levee because increased public confidence will lead to increased use of the protected area. The pump station is simply a minimum service to assure that the project performs as intended.

6.9 Temporary Pumping. The possibility of providing temporary pumping was examined, since by pumping out of five manholes, the construction of the interceptors and pump station could be avoided. However, temporary pumping was found to be infeasible since there would be little time in which to mobilize. Significant runoff could arrive in as little as one-half hour. By virtue of this fast runoff and rapid ponding of large areas, there would be very little start-up time. Once ponding was rising in the streets, it would be difficult to reach and service the pumps. Reliance would therefore be placed on all five manhole pumps operating nearly instantly and dependably under conditions of frequent start-up and shutdown reflecting inflexibility of operation. This scenario, perhaps occurring in early morning hours during a rainstorm, seems unsuitable for a downtown commercial and industrial area.

6.10 Operation Schedule. The operation schedule for the Gaines Street Pump Station has been largely determined during the derivation of performance criteria for the Impact Trapezoid. Gate closure and pump activation level is set at El. 557.0 and the sump level for pump cutoff is set at El. 547.0 by the interceptor elevation. The operation schedule is shown on plate A-6.12.

MAIN STREET (AREA 1) PUMP STATION

7. General.

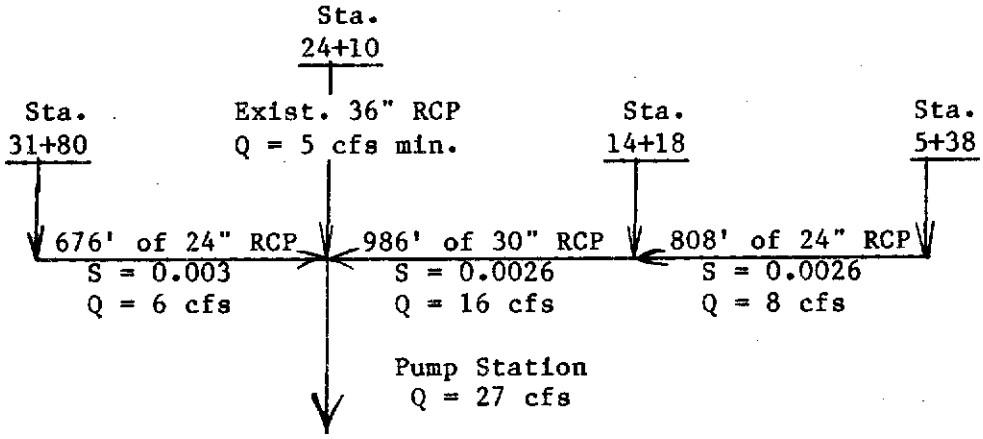
As discussed above, the Main Street (Area 1) Pump Station is being studied after Area 2. This is because Area 2 is much the larger, its problems dominant, and, as will be seen, its design and performance criteria identical. Since much of the supporting data for this analysis was derived similarly to that of Area 2, discussion of that derivation will not be considered pertinant unless exceptional. It will be seen that the Main Street pump station is a minimally-sized facility.

7.1 Area 1--Hydraulic Characteristics. To recap discussion in paragraph 5.1.1, it is recalled that Area 1 is relatively small (108 acres), 20-25 percent impervious, mostly level, highly sewered, and entirely urban. Area-capacity curves are shown on plate A-7.1. A unit hydrograph for Area 1 is shown on plate A-7.2. Various storm hydrographs are tabulated on plate A-7.2.1. Discussion of the development of the unit hydrograph proceeds below in paragraph 7.1.2. Area 1 is drained by nine gravity outlets emptying into the Mississippi River. During proposed blocked-gravity conditions, these existing outlets will be closed with gatewells and their flow picked up in an interceptor sewer (shown on plates 20 and 21 of the main report). Schematics on plates A-7.3 and A-7.4 portray storm sewerage under gravity and blocked-gravity conditions, respectively. Not shown on the schematics is the 66-inch RCP pressure sewer under Iowa street from Area 3D, shown on plate A-4.1. There is no runoff from Area 3D into Area 1.

7.1.1 Gravity Outlets. The eight gravity outlets in Area 1 were rated at 10 and 50 percent Mississippi River elevation durations. The resulting rating curves are shown on plates A-7.5.1 thru A-7.5.9. The ratings were performed in accordance with EM 1110-345-284 (14 Aug 64), identical to the method described above in paragraph 6.1.1. No change to the existing sewer (gravity) system is contemplated. A composite gravity rating at 50, 10, and 1 percent Mississsippi elevation durations is portrayed on plate A-7.6.

7.1.2 Interceptors. Interceptors will be discussed at this point to keep related material together despite the fact that the pump station has not yet been sized; the capacity of the interceptor is to be designed to at least equal the capacity of the pump station. An interceptor is necessary to provide positive evacuation of nine existing storm sewers during periods of prolonged blocked-gravity drainage. Design head was measured from basin inverts since surcharge of the existing sewer system is not an infrequent occurrence. A Manning's roughness coefficient "n" of 0.012 was used. The pipe was assumed to be flowing full since the outlets are deeply submerged. Layout, slopes, lengths, and flows are portrayed in the schematic below. Due to layout, the interceptor will not add to the capacity of the existing storm sewer system during gravity conditions. The profile of the interceptor is shown on plates 20 and 21 of the main report.

INTERCEPTOR SCHEMATIC
Gaines Street Pump Station
(Blocked Gravity Drainage Only)



7.1.3 Unit Hydrograph. Clark's method was used to obtain the unit hydrograph. The Time of Concentraction (Tc) was selected to be 30 minutes to account for ponding at the inlets which generally have less capacity than the storm sewers. Clark's "R" was selected to be 12 minutes to reflect a medium amount of transport storage. The unit hydrograph is shown on plate A-7.2. Other sources of inflow include seepage which is discussed in the next paragraph.

7.1.4 Seepage. Seepage into Area 1 is negligible. Through-seepage is virtually eliminated by use of floodwalls and by impervious levees downstream. Underseepage from the riverbed is largely prevented by the existing seawall. Geotechnical discussion of seepage follows in Appendix B of this report.

7.2 Design and Performance Criteria - Interior Flood Control. The design and performance criteria at the Main Street (Area 1) Pump Station will be the same as those used at the Gaines Street (Area 2) Pump Station. Several considerations require this. First, both are positioned at the same elevation atop the Davenport "seawall" (actually, an earth retention structure forming an artificial riverbank). Hence both are similarly affected by the Mississippi River, experiencing identical elevation-durations and the same flood flows. Second, both stations are to be positioned in adjacent low-lying areas that are relatively level and are about the same in elevation. In fact, the division between Area 1 and Area 2 disappears above El. 560.4 (which is the same elevation at which River Drive closes). Thus, at ponding stages above El. 560.4, the two ponding areas are connected. The two areas may have been served by the same pump station except for the fact that an interceptor could not be continued through Ripley Street because of an existing large pressure sewer. Overland flows to Area 2 could eventually relieve much of the ponded interior runoff in Area 1, but would not provide evacuation of the several storm sewers during prolonged periods of blocked gravity conditions. Third, since Area 2 is 2.7 times as large as Area 1, pump performance (or lack of it) would tend to influence Area 1 rather than

the other way around. Any performance criteria selected independently for Area 1 would tend to default to those of Area 2. Because of the accumulated similarities of the two areas, because of their spartial interdependence, and because the Main Street facility in a sense supplements the Gaines Street Pump Station, selection of different design and performance criteria seem inappropriate. It is convenient to list those criteria from paragraph 6.6.5:

a. Maximum allowable stage: El. 561.0.

b. Allowable Interior Stage Duration:

(1) Three hours at El. 560.4 (River Drive closure).

(2) Six to 12 hours drawdown to El. 557.0.

c. Runoff: 1 to 5 years.

7.3 Pump Capacity Analysis. From this point, it is proper to procede immediately to the pump capacity analysis. Descriptions of the three methods employed are described in paragraphs 6.5 to 6.5.3 and will not be repeated here.

7.3.1 Period of Record (POR) Analysis. The Period of Record (POR) Analysis at the Main Street Pump Station was very similar in application to that at Area 2. The Maximum Elevation Vs. Frequency curve for zero pumping is shown on plate A-7.9. In addition, pumps of 5,000, 10,000, and 20,000 gpm were examined and elevation-frequency curves were computed for each, also plotted on plate A-7.9. Influencing the latter curves is overflow from Area 2 with a 43,000 pump station capacity at approximately a 0.01 percent annual exceedence (100 year) event. In the drawing of the elevation-frequency curves for the various sizes of pumps, those data points were more heavily weighted that would tend to minimize apparent benefits and that would plot with some consistency against the zero pumping curve which incorporates less modified data. Economic analysis (discussed fully in appendix F) of these elevation-frequency data for the zero, 5,000, 10,000, and 20,000 gpm pumps indicate optimization at 12,000 gpm. Final selection is discussed below in paragraph 7.4.

7.3.2 Total Probability Method (TPM). The Total Probability Method (TPM) at Main Street was accomplished very similarly for Area 2, using the Corps of Engineers program INDRN. Input is discussed in paragraph 6.6.3. Benefits computed by TPM were approximately two percent of those computed by the Period of Record Analysis. As concluded above in respect to Area 2, since these results differ so much from those obtained incorporating 101 years of record, the TPM will not be included for further consideration.

7.3.3 Impact Trapezoid Method (ITM). The procedure for constructing the Impact Trapezoid has been discussed in detail in paragraphs 6.6.5 and 6.6.6. Accordingly, flood routing (maximum elevation vs. pump capacity) curves are shown on plate A-7.10. To this plate are added the graphical representation of the design and performance (selection) criteria, which

are listed at the close of paragraph 7.4. The result is plate A-7.11, which also includes expanded parameters so that one can see the effect of criteria different from those selected. The Impact Trapezoid is constructed on plate A-7.12, and several superfluous lines have been deleted. A glance at plate A-7.12 indicates that:

(1) the limiting elevation of El. 561.0 has no influence on pump selection

(2) likewise, the El. 560.5 (closure of River Drive) has no influence on pump selection, and

(3) the only important criteria is the range of runoff events and the allowable time to evacuate ponding.

This outcome is very similar to that of Area 2, which is not surprising. It is further noted that pumping in Area 1, ranging from 3,700 gpm to 10,000 gpm, will meet the lower ranges of the selection criteria. Pumps of 6,300 to 10,000 gpm will meet all criteria.

7.4 Pump Selection. It is recalled that the POR analysis resulted in a selection of 12,000 gpm, which is slightly past the upper end of the selection range discussed immediately above. This is a similar outcome to Gaines Street (Area 2) where the POR analysis arrived at a capacity approximately 20 percent greater than that suggested by the Impact Trapezoid Method. This is interpreted similarly to Gaines Street (Area 2): a 10,000 gpm will meet selected performance criteria, but the larger 12,000 gpm pump will achieve optimal economic results. Secect 12,000 gpm.

7.5 Temporary Pumping. In Area 1, temporary pumping was dismissed as a reasonable possibility since pumping at nine locations would be required.

7.6 Examination of Selected Capacity. The effects of the selected pump capacity at the Main Street Pump Station are similar to those in Area 2. For the 100-year and the Standard Project Storm (SPS) during blocked gravity conditions, the Area 1 ponding would merge with Area 2 (and then with Credit Island Harbor at the SPS). The effects in Area 1 at these low frequency (of occurrence) events are not separable from those in Area 2 which have been discussed in 6.7. It is repeated that while damage would be high, there appears to be no threat to life. Under an SPS, ponding in parking lots would be a maximum of 4.0 feet deep, but the upper 2.0 feet of that depth would arrive gradually over a 12-hour period. In Area 1, these depths and gradual rises do not appear to be life threatening.

7.7 Operation Schedule. The operation schedule for the Main Street Pump Station has already been partially determined in the derivation of the selection criteria. That is, the gate closure and pump activation stages equal El. 557.0. Likewise, the other operational parameters are similar to those of the Gaines Street Pump Station. Interceptor layout fixes the sump at El. 543.0. From that point, allowing four feet of intake submergence indicate a pump shutoff at El. 547.0 and 10.0 feet of cycling between El. 547.0 and El. 557.0. The operation schedule is shown on plate A-7.13.

CREDIT ISLAND PUMP STATION

8. General. The Credit Island Pump Station will entail a design quite different in scope and consideration from the two facilities previously discussed. The drainage area is 20 times larger; the ponding area contains 240 acres of permanent slough; pond drawdown is measured in terms of days rather than hours. The Credit Island Pump Station will be responsible for evacuating the flows of Black Hawk Creek (8.4 square miles) and for evacuation of a few hundred acres of urban runoff. The Credit Island Pump Station serves Areas 4, 4A, and 5, which are shown on plates A-1.2 and A-4.1.

8.1 Hydraulic Characteristics. Hydraulic and hydrologic characteristics to be considered in this section are area-capacity curves, hydrographs, and seepage.

8.1.1 Ponding. Limited secondary ponding is available in Areas 4 and 4A; area-capacity curves are shown on plates A-8.2 and A-8.3. Primary ponding is available in the Credit Island Slough which contains a very large amount of storage. To account for future conditions (i.e., filling) and to limit right-of-way purchases, the following assumptions have been made:

a. Below El. 554.0. Regard as permanently "wet" and protected by the Section 404 permit program.

b. El. 554.0 to El. 558.0. Areas 100 percent filled.

c. El. 558.0 to El. 560.0. Areas 50 percent filled.

d. Above El. 560.0. Elevation-area curve to increase at same rate as the previous increment, plus one-half of the incremental areas of the other subareas, 4A through 1. This procedure reflects two conditions:

(1) all subareas begin to merge above El. 560.0, and

(2) the majority of runoff originates in Area 5.

These assumptions are conservative under present conditions, but are an attempt to reflect future conditions. The resulting area-capacity curve is shown on plate A-8.1.

8.1.2 Inflows-Hydrographs. The inflows into the Credit Island Slough are a complex mixture of urban and rural runoff, regulated flow, and flows "spilling" past pressure sewer inlets.

8.1.2.1 Area 5 - Runoff - Upper Portion, Black Hawk Creek. Black Hawk Creek includes modification by the I-280 Lake System, best shown on plate A-1.2. Reference is made at this point to a consultant's study "Feasibility Report: Utilization of I-280 Highway Embankment as an Impounding Structure to Provide a 95 Acre Lake Development; Volumes I and II." The discharge from the lakes were determined by assuming the

lakes full at start, computing storage volumes above the various ungated spillways, and then routing various runoffs through the system. The routings were accomplished for each lake and the combined flow of the system was routed through the final spillway into Black Hawk Creek.

8.1.2.2 Area 5 - Runoff - Lower Portion, Black Hawk Creek. The Clark method of determining the unit hydrograph was used. Values for the Time of Concentration (Tc) and Clark's "R" were based on local experience. Input for the unit hydrographs is summarized on plate A-4.2.

8.1.2.3 Area 4 Runoff. Area 4 runoff consists of McManus Creek which is to be directed into the Credit Island Slough only during times of blocked-gravity drainage. Construction of a separate pump station is unfeasible since there is almost no ponding in the vicinity. Area 4 is an hour-glass shaped area which would tend to produce a double peaked hydrograph. Conservatively, this effect was ignored. The unit hydrograph was derived using Clark's method, with values for Tc and Clark's "R," modified for time of travel which is influenced by sewers and gutters. Input for the unit hydrograph is summarized on plate A-4.2.

8.1.2.4 Area 3A - "Spillage." Area 3A does not contribute to the blocked-gravity inflow since the Division Street pressure sewer conveys its flow directly to the Mississippi River. However, under conditions of heavy rainfall, runoff may exceed the capacity of the pressure sewer. Excess runoff will bypass the inlets of the pressure sewer and will "spill" into Area 4A and eventually into the Credit Island Slough. The capacity of the pressure sewer is estimated below in paragraph 8.2.2 to be 510 cfs for the twin 72-inch RCP's. Therefore, Area 3A runoff in excess of 510 cfs becomes "spillage." The Area 3A hydrograph was determined using Clark's method, with adjustments to input to reflect the largely urban nature of the watershed. Input for the unit hydrograph is summarized on plate A-4.2.

8.1.2.5 Area 4A Runoff. Area 4A is generally flat: parking lots, the city cemetery, and the former dump site. Even with future development, it is expected that runoff entry into sewers will be delayed by sluggish overland flow. These factors were considered in deriving "Tc" and "R" for use in Clark's method, which are summarized on plate A-4.2. Area 4A plus "spillage" from 3A constitute a very small percentage of the total blocked-gravity discharge into the Credit Island Slough.

8.1.2.6 Composite Unit Hydrograph. Runoff from Areas 4A, 4, and 5 (lower) may routinely be brought together and combined into a single inflow hydrograph at the mouth of Black Hawk Creek. However, it was decided to give the "spillage" from Area 3A and the Lake I-280 discharge a little different treatment to simplify runoff computation. The unit hydrograph was computed by first deriving a 5-year storm runoff hydrograph (which included all sources of flow except seepage), and then dividing the ordinates of this 5-year storm hydrograph by the 5-year rainfall excesses. The unit hydrograph thus derived is on plate A-8.4. This procedure introduces volumetric errors of plus 6.5 percent error in the 1.0 year and plus 3.7 percent error in the 2.0-year storm hydrograph. The 10-year

storm volume will be understated by 2.5 percent and the 100-year storm by 12 percent. Peak flows will be less affected since the stream regulation affects only the recession limb of the hydrograph. Various storm hydrographs are portrayed on plate A-8.4.1.

8.1.3 Seepage. Seepage into the Credit Island Slough is moderate. Rates of through-seepage and under-seepage will not be distinguished. Seepage directly into the ponding area (the Credit Island Slough) is estimated to 126 gpm per foot of head. Other seepage is an additional 211 gpm per foot of head. Determination of these rates of seepage is explained in the geotechnical discussion, appendix B of this report.

8.2 Existing Drainage Appurtenances. The existing gravity drainage system is portrayed on a storm drainage schematic, plate A-8.5. There will be no proposed changes to the existing gravity system. The proposed blocked-gravity schematic is shown on plate A-8.6.

8.2.1 Existing Gravity Outlets--Howell and Sturdevant Streets. Presently, there are two existing gravity outlets entering the Mississippi River from Howell and Sturdevant Streets. Rating of these storm sewers were accomplished in accordance with EM 1110-345-284 (14 Aug 64), as described above in paragraph 6.1.1. The ratings were determined for 50 and 1 percent Mississippi River elevation-durations. These ratings are shown on plates A-8.7 and A-8.8.

8.2.2 Division Street Pressure Sewer. The Division Street pressure sewer was installed by the city in 1968 in preparation of the project. The capacity of this existing pressure sewer was estimated on the basis of blocked-gravity conditions occurring during a 4 percent annual exceedence (25 year) flood on the Mississippi River. Procedures in EM 1110-345-284 (14 Aug 64) were employed. Preliminary computations indicated that velocities in the 72-inch RCP will be about 9 fps at capacity. This velocity would provide a total discharge of 510 cfs. Therefore, the capacity was estimated by testing the system to check if there is sufficient available head to conduct flows at 9 fps. The static head is the difference between the top of freeboard (El. 571.7) and the 25-year flood (El. 562.85), or 8.85 feet. However, at the point of pressurization the flow is not static but has a flowing full velocity of 9.0 fps or an incoming velocity head of 1.26 feet. This head is added to the static head to arrive at 9.91 feet total available head. Losses include barrel friction in the 2,360 feet of sewer between Second Avenue (where pressurization begins) of 7.25 feet and an outlet loss (at 9 fps) of 1.26 feet. After allowing a 0.2-foot loss at each of five manholes and 0.5 feet for transition losses, computed head losses total 10.01 feet versus 9.91 feet available. Therefore, 9 fps does not seem unreasonable. Capacity at 9 fps equals 255 cfs in each 72-inch RCP or say 510 cfs for both. A profile of the Division Street pressure sewer is shown on plate A-8.4.2. As discussed above in paragraph 8.1.2.4, runoff in Area 3A in excess of this capacity "spills"--runs over land where it ponds in low-lying areas well below the level of the lowest pressure sewer inlets. This spillage must be conveyed eventually to McManus Creek and then to the Credit Island Slough during conditions of blocked-gravity drainage. It is noted here

that the Division Street pressure sewer is not yet fully pressurized to the top of protection by the city. Other city obligations are discussed in section 10.

8.3 Proposed Drainage Appurtenances--Blocked Gravity.

8.3.1 Interceptors from Area 4 and 4A. The 72-inch RCP storm sewer from Sturdevant Street is currently somewhat oversized due to the construction of the Division Street pressure sewer which removed most of its contributing drainage area. In Areas 4 and 4A, it is desired to evacuate interior drainage reasonably quickly because of a long sag in River Drive, a major traffic artery. A 2-year storm during blocked gravity will produce a peak of 64 cfs, but a 30-inch RCP at a 0.0025 slope carrying 22 cfs will permit reopening of River Drive in one hour. Alignment of the proposed interceptor sewer is moved inland in order to pass under the Division Street pressure sewer. In order to obtain a reasonable hydraulic slope, it was necessary to locate the invert of the interceptor above that of the Sturdevant Street sewer. This practice is permissable in this case because of the low velocities in the 72-inch RCP (2-3 fps) and because flow will be diverted through the interceptor only during blocked-gravity drainage condition, 2 percent of the time. The next leg of the interceptor extends from the Howell Street sewer to McManus Creek, which in turn is diverted into Credit Island Slough only during blocked-gravity drainage. The outlet invert of the interceptor is thus the intersecting elevation with McManus Creek, or rather its diversion ditch, discussed below in paragraph 8.3.4. Peak velocities in the existing Howell Street sewer are 4-5 fps. Available head along the entire interceptor was the difference between El. 560.0 (River Drive inlets) and El. 555.0 (Credit Island Slough level). The flow length is about 2,100 feet. The hydraulic slope is thus 0.0024. After head losses, a 72-inch RCP was selected. The minimum slope at which the 72-inch RCP will carry the incoming 135 cfs is S = 0.001. A flow schematic is shown following paragraph 8.3.5. A plan and profile of this interceptor is shown on plates 13 through 16 of the main report.

8.3.2 Howell Street Gatewell. The existing Howell Street sewer (60-inch RCP at outlet) is to be provided with a gatewell and slide gate shown on Plate 15 of the main report. The outlet elevation is to be El. 545.14. Gate closure at the Howell Street gatewell is to be at El. 555.9. This elevation corresponds on profile (plate A-3.5) to an El. 555.0 at the Credit Island Slough. Should closure be effected at a higher elevation, river water would back up the sewer and flow down the interceptor toward the slough. In the event of runoff, the interceptor would lack desired capacity. Pressurization of the Howell Street sewer was considered, but found to be fruitless since very little drainage area could be removed from draining into the Credit Island Slough.

8.3.3 McManus Creek Outlet. During gravity conditions (98 percent of the time), McManus Creek will be discharged directly into the Mississippi River via a twin 10' x 10' RCB passing through the line of protection. This structure matches an existing structure that conveys flow under River Drive. It is noted that McManus Creek here has a

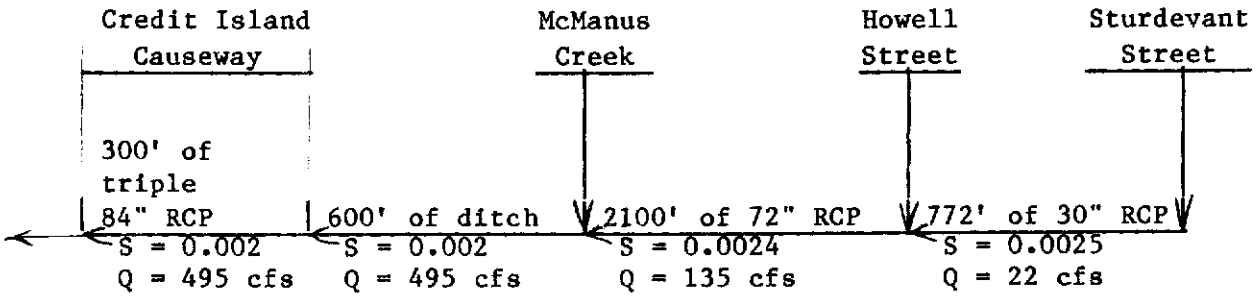
relatively steep slope of 1.6 percent. Thus, submergence of the proposed structure is 5.5 feet greater than the existing, but local terrain provides more than compensating confinement of the headwater to assure equivalent capacity.

8.3.4 McManus Creek Diversion. During blocked-gravity conditions, the gatewell on the twin 10' x 10' RCB described above is closed. McManus Creek is then shunted toward Credit Island Slough through a ditch along the landside of the levee. In addition to the flow of McManus Creek, the ditch must also carry the flows of the interceptor that outfalls from Howell Street. For blocked-gravity conditions, the flow from McManus Creek is considered to be limited to the capacity of the existing 6' x 10' RCB conduit indicated on plate A-1.1. It is also desirable to limit the water surface at the diversion to El. 558.0 to prevent excessive reduction of the capacity of the Howell Street interceptor, or even reverse flow. This required a ditch of a 20-foot bottom width with 3 on 1 side slopes. (Consideration was given to permanently diverting McManus Creek into Black Hawk Creek, but this was rejected because of the possibility of pollutants from the large urban watershed and because of poor flow characteristics during more severe storms.)

8.3.5 McManus Diversion - Under the Credit Island Causeway. The McManus Diversion must be conveyed through the embankment of the Credit Island Causeway in order to outfall into the Credit Island Slough, as shown on plate 13 of the main report. Conveyance of 495 cfs required a triple 84 RCP plus with localized dredging in the Credit Island Slough in order to form a flowway. Computations are summarized immediately below.

INTERCEPTOR SCHEMATIC
Credit Island Slough

(Blocked Gravity Flow Only)



8.3.6 Credit Island Slough Inlet - Gravity. During low water, the Credit Island Slough is subject to low oxygen and resulting stagnation because of inadequate inflow. This condition would largely offset the potential environmental benefits accruable to dredging of the slough during levee construction and to the separation of the slough from high water (and high sedimentation). In sizing the gravity inlet, available head across the slough is the fall of the Mississippi River from the

upstream end to the downstream end. Available head thus changes with the flow (and change of profile slope) of the Mississippi River. The inlet was regarded as a submerged culvert and was sized and rated in accordance with EM 1110-345-284 (14 Aug 64). The selection was a twin 8' x 8' RCB, and its rating is shown on plate A-8.9. This size of inlet was provided in order to assure a rapid rate of water turnover in the slough; a slough turnover-duration curve is portrayed on plate A-8.10. It is noted on plate A-8.10 that 88 percent of the time, turnover occurs in less than 3 days and turnover in excess of 4 days occurs only 1 percent of the time.

8.3.7 Credit Island Slough Outlet - Gravity. The outlet of Credit Island Slough is to be a 40-foot wide concrete framed opening with a sill at El. 838.0. Routings at 50 percent and 10 percent duration were simply a solution of the head losses along the slough and out the 40-feet wide, 70-feet long outlet. It was found that virtually all losses occurred at the outlet itself. Loss coefficients for this short structure were 0.5 $V^2/2g$ at its inlet, 0.1 $V^2/2g$ along its length, and 1.0 $V^2/2g$ for the exit loss. Ratings were then simply a solution of the equation $H = KV^2/2g$ where H = head loss, K is the sum of the loss coefficients, V is the average velocity, and g is gravitational constant, 32 ft/sec/sec. These ratings at the 50 and 10 percent Mississippi River elevation duration are plotted on plate A-8.11. Routing of the 100-year event is portrayed on plate A-11.1.

8.4 Performance and Design Criteria - Interior Flood Control. Performance and design (selection) criteria at the Credit Island Slough will be similar in content to those at the Main and Gaines Street pump stations. However, in addition, criteria must also specify the utilization of a very large volume of storage that may be thoughtfully used to reduce pumping requirements and permit flexibility in operation.

8.4.1 Range of Runoff Criteria. Conditions affecting the selection of a range in runoff events to be used as a design criteria in choosing pumping capacity are similar to Gaines Street. Thus, the content of paragraph 6.3.1 is largely applicable. Credit Island does have 2.0 percent gate closure duration rather than a 1.0 percent as at Gaines Street Pump Station. This factor by itself would lend weight toward selection of a higher runoff range, but there is a great deal more storage at Credit Island. For example, 1.0 inch of runoff with no pumping may cause no damage at Credit Island whereas 1.0 inch of confined runoff at Gaines Street would cause $140,000 in damage and at least moderate disruption. No quantification of these factors will be attempted; the runoff range of 1 to 5 years will be used.

8.4.2 Minimum Pond Elevation. The minimum sump elevation should be set as low as possible to maximize storage without abbrogating environmental considerations. Project enhancement of aquatic life within the Credit Island Slough is desired. Lowering the slough below seasonal depths should be avoided as a general criteria. Referring to plate A-8.1, it is noted that at El. 552.0, the depth above El. 546.0 is 6.0 feet and that the amount of useful storage below El. 552.0 decreases rapidly. Therefore, select El. 552.0. If further storage is required, lower elevations may then be considered along with their deleterious effects.

8.4.3 Gate Closure Elevation. The gate closure elevation is to be set at the highest elevation which does not cause excessive nuisance. An examination of elevations around the slough indicate that S. Concord Avenue closes in places at El. 553.5. However, with inconvenience, all existing buildings can be reached by approaching along Concord from the opposite direction. Above El. 555.0, buildings begin to be inaccessible to vehicles, causing much greater inconvenience. Therefore, allow the closure elevation to be placed at El. 555.0. Referring to plate A-3.5, it is seen that this elevation corresponds to a 2 percent elevation duration.

8.4.4 Maximum Desirable Elevation. The maximum allowable ponding elevation is dependent upon that elevation at which damages either become excessive or begin to drastically increase. Refer to the damage-elevation curve on plate A-8.12. Two points must be considered. First, at El. 559.0, rate of damages begins to increase rapidly, and River Drive is closed downstream of Sturdevant Street. Drainage problems would result in Subareas 4 and 4A as this high water level reduces capacity of the interceptor sewer. Second, at El. 557.0 the point of moderate nuisance in the slough area is crossed as some structures are damaged. With increased residential development, this condition would become even less acceptable. Therefore, unless other considerations place an upward pressure upon this limiting elevation, use El. 557.0.

8.4.5 Allowable Initial Drawdown Time. The storage between the Gate Closure Elevation (El. 555.0) and the Minimum Pond Elevation (El. 552.0) amounts to 1.8 inches of runoff. However, to be useful, this storage must be available when it is needed. A small pump capacity requiring 10 days to accomplish this initial drawdown would clearly be too small to make available this storage; quite probably runoff would have arrived before then. On the other hand, it is uneconomical to provide pumping sufficient to accomplish drawdown in one day; usually the Mississippi River does not rise so quickly that the gates could not be opened and the high Credit Island Slough be simply dumped into the river via gravity. Therefore, selection of an Allowable Initial Drawdown Time consists of discerning how long it typically takes the Mississippi River to rise from the Gate Closure Elevation (El. 555.0) to the Maximum Desirable Elevation (El. 557.0). Research of records of past floods suggests that at these stages a moderately rapidly rising Mississippi would make this rise in 2-1/2 days. Thus, 2-1/2 days is selected as the Allowable Initial Drawdown Time.

This criteria is incorporated into the storm routings (Maximum Pond Elevation Vs. Pump Capacity) on plate A-6.13. The resulting family of curves on plate A-6.13 are prepared with 2-1/2 days of advance pumping after gate closure at El. 555.0. It is noted on this plate that it requires approximately 68,000 gpm to accomplish this initial drawdown. The curves abruptly change slope at this capacity. At less than 68,000 gpm, runoff is received into a Credit Island Slough which has not yet been drawn to El. 552.0. Above 68,000 gpm, the slough is maintaining at El. 552.0 when runoff arrives. This criteria will not be represented on the routings plate except by this break-slope in the curves. Also shown on the same plate are (dotted) extensions of the routing curves as if initial drawdown to El. 552.0 had already been accomplished.

8.4.6 Maximum Runoff Removal Time. Next, consider the time required to remove runoff, i.e., to pump the slough back down to El. 552.0, after the initial drawdown has been accomplished. This time is expected to be at least as great as the initial drawdown. The most important factor in selecting this time is a reasonable expected reoccurrence of significant runoff. Four days was at first considered since the maximum duration of storms presented in the weather bureau Technical Paper No. 40 was 96 hours, and would by implication define a close limit. However, perusal of rainfall data indicates that significant runoff cannot be assumed to be separated by 4 days. Data suggest that 3 days in the late spring months would be a more representative value. This value (3 days) was selected, with a range of 2 to 4 days to be considered. The sensitivity of this selection is reduced by the fact that the city can take measures to reduce damages if it is seen that a major weather front is coming at a time when the slough is vulnerable to further runoff.

8.5 Routings. Pump routings were performed using the area-capacity curve and the operating elevations discussed above. That is, the gates were closed at El. 555.0, and the slough pumped down to El. 552.0 to await runoff. It was also required to show the effect of an initial drawdown time limited to 2-1/2 days. Routings are summarized on plate 8.13. The same routing summary is shown on plate A-8.14, to which has been added a family of equal-hour drawdown (to El. 552) curves at 2, 3, 4, 6, and 9 days. Since the average length of closure approaches 7 days, the first 2-1/2 days after closure comprises a large percentage of the actual closure times.

8.6 Pump Capacity Analyses. At Credit Island, the pump capacity requirements were analyzed by three methods: (1) the Period of Record (POR) anlaysis, (2) the Total Probability Method (TPM), and (3) Impact Trapezoid Method. All three of the methods have been described in detail above in section 6.5.

8.6.1 Period of Record (POR) Analysis. The Period of Record (POR) Analysis at Credit Island was similar to that at Gaines Street except for seepage, which is discussed immediately below.

8.6.1.1 Seepage. As recalled from paragraph 8.1.3 above, seepage is divided into two rates: pond seepage and levee seepage. Seepage into the pond (Credit Island Slough) begins with gate closure at El. 555.0. However, levee seepage does not begin until the Mississippi River exceeds the average ground elevation of El. 559.0. (A rationale for a summary seepage computation is contained below in prargraph 8.6.4.) The average rates of seepage were taken to be one-half of the peak rate. Duration of the pond seepage is simply the duration of closure excluding those days near end of the closure when the slough exceeds El. 555.0. Negative seepage was not considered. Duration of levee seepage was taken to be a lesser value: the difference between the peak river stage and the average ground (El. 559.0) times 4.5. This incorporates an average river rise of 0.5 feet/day and a fall of 0.4 feet/day, i.e., $1/0.5 + 1/0.4 = 4.5$. The computed volume is added to runoff in determining the peak inferior stage. Thus, seepage is automatically incorporated into the POR Analysis.

8.6.1.2 POR Results. The results of the Period of Record (POR) Analysis are plotted on plate A-8.15. These results, when incorporated into an economic analysis (discussed in section 2 of appendix F), indicate an optimum capacity of 100,000 gpm. This capacity is larger than that indicated by the Impact Trapezoid Method discussed below in paragraph 8.6.3. Final selection of pumping capacity if discussed in paragraph 8.7, where the differing results of the two methods are synthesized.

8.6.2 Total Probability Method. The results of the Total Probability Analysis (TPM) were similar to those obtained above for the Main and Gaines Street Pump Stations. Benefits were very small: much different results than the Period of Record analysis which incorporates 101 years of record.

8.6.3 Impact Trapezoid Analysis. Capacity determination of the Credit Island Slough Pump Station was accomplished with the help of an Impact Trapezoid Analysis, utilizing the considerable ponding volume available. Criteria discussed under section 8.4 is now displayed:

a. Runoff Range = 1 to 5 years

b. Minimum Pond Elevation = El. 552.0

c. Gate Closure Elevation = El. 555.0

d. Maximum Desirable Elevation = El. 557.0

e. Allowable Initial Drawdown Time = 2-1/2 days

f. Maximum Runoff Removal Time = 2-4 days

As discussed in paragraph 8.5, hypothetical storm routings are contained in plate A-8.13. To plate A-8.14 has been added the above criteria plus expanded ranges of the same criteria in order that the reader may appraise the effect of different selections. As a result, plate A-8.14 has become rather complex. Deleting all curves except the above criteria, one arrives at the somewhat abbreviated plate A-8.16 upon which is displayed the Impact Trapezoid. Examination of plate A-8.16 indicates the following:

a. The Maximum Desirable Elevation (El. 557.0) has very little influence upon pump selection.

b. The controlling criteria is the runoff range (1 to 5 years) and the maximum runoff removal (2-4 days).

c. The pump capacities included in the Impact Trapezoid range from 54,000 to 125,000 gpm.

Selection from within this Impact Trapezoid on plate A-8.16 is forthright: 70,000 gpm is the lowest capacity that meets all criteria. The 5-year storm peak just equals El. 557.0. At capacities below 70,000 gpm, compliance with criteria deteriorate very rapidly. For example, a 20 percent reduction in capacity from 70,000 gpm (i.e., to 56,000 gpm) results in a 0.6 foot rise (to El. 557.8) in the peak pond and an increase from 4 to 6 days in runoff drawdown at the 5-year level of runoff. In contrast, increasing pumping capacity improves performance against selected criteria only sluggishly. For example, to lower the peak pond elevation 0.6 feet, the pump capacity must be increased 60 percent. That is, response to selected performance criteria is highly sensitive to lower selection and insensitive to higher. Final selection is discussed in paragraph 8.7.

8.6.4 Seepage Allowance. In contract to the POR Analysis which incorporates seepage, an allowance for seepage must be added to the results of the Impact Trapezoid Method. As discussed in paragraph 8.1.3, "pond" seepage (measured from El. 555.0) is 126 gpm per foot of head "levee" seepage (measured from El. 559.0) is 211 gpm for foot of head. Often it is best practice to construct a seepage hydrograph and incorporate this inflow into the pump station analysis. However, on the Upper Mississippi River, the seepage hydrograph would have a 30-50 day duration while the runoff and its pumping would endure 3 or 4 days. Use of seepage rate varying in time would achieve very little. Therefore, an average rate will be selected. The average seepage rate will be taken to be rate of seepage occurring midway between the average ground and the 0.5 percent annual exceedence (200 year) flood. The midpoint elevation is thus El. 563.25. The average seepage rate, directly computed, is 1,940 gpm, or say, 2,000 gpm.

8.7 Pump Selection. The above results will now be summarized. Computed above is an average seepage rate of approximately 2,000 gpm. Consideration of selected performance criteria via the Impact Trapezoid Method indicates that a capacity of 70,000 gpm (plus 2,000 gpm for seepage) is required for adequate performance of the project. However, economic analysis based upon the Period of Record (POR) analysis indicates that a 100,000 gpm capacity (which includes seepage) is economically optimal. This is only fair agreement between the two methods. While the Impact Trapezoid Method indicates that a 72,000 gpm facility will meet performance criteria, economic analysis via the POR Analysis indicates that worthwhile benefits may be obtained by increasing pumping capacity to 100,000 gpm. In this sense, the two methods are compatible. A capacity of 100,000 gpm is selected.

8.8 Effects of the Pump Station. The construction and operation of the pump station in the Credit Island Slough will have various effects which should be summarized at this point before progressing further. First, the presently flood-susceptible lands will increase in value, and land use will tend to change. The change will be a trend toward increased industrialization and development in this area which is well served by highway, railway, and river transportation. Secondly, there will be several environmental effects, many of which are positive. The isolation of the slough during high water will greatly reduce the amount of sediment

deposited in the slough. During low water, recirculation via the twin 8' x 8' box through the Credit Island causeway will enhance water quality. Presently, the slough is subject to large sediment deposition during floods and to low oxygen and shallow water during the warm summer months. A Slough Turnover versus Duration is shown on plate A-8.10. Proposed dredging of the slough will deepen it and upgrade the habitat for fish, likely leading to improved fishing in the slough, and thus stimulating the use of the adjoining park on Credit Island. An important indirect effect of the pump station in the Credit Island Slough will be improved interior drainage in the Garden Addition. This will be discussed below. Tangible and intangible effects of the pump station are portrayed on plate A-8.18.

9. Garden Addition Effects and Recommendations. The majority of residential interior flooding in the project area occurs in the Garden Addition, primarily because of blocked interior drainage. Presently, high stages on the Mississippi River drown the mouth of Blackhawk Creek causing closure of flapgates on storm sewers which drain the Garden Addition. This closure occurs at El. 555.0, is relatively frequent, and during major floods, endures for 6 to 8 weeks. During these long closures, the Garden Addition is extremely vulnerable to rainfall, since there is no storm drainage. With the operation of the proposed Credit Island Slough pump station, these durations would be reduced, from 6 to 8 weeks to 2 or 3 days. Nevertheless, it is reasonable that for a densely developed residential area subject to interior drainage problems, a minimum of 1.5 inches in 1 hour of runoff should be handled without serious damage. There are existing pumping facilities at two locations (3,200 gpm each) as shown on plate A-9.1 with routings as shown on the same plate. Routings indicate that 1-1/2 inches of runoff will produce a peak stage of 557.2 which will put water against 75 homes, and will flood streets serving 60 more homes for more than 2-1/2 days. These are unacceptable stages and durations. Therefore, it is recommended that an additional 6,000 gpm of pumping be provided. This will accomplish a slight reduction in the peak to El. 557.0 but will reduce the durations to about 16 hours. Due to the small size of the unit(s), it is recommended that they be purchased, installed, and operated by the city.

10. Nahant Marsh. The hydrologic aspects of the marsh will be directly affected by the project. While the drainage basin of the marsh will continue to feed runoff to the marsh, periodic flooding by the Mississippi River will be largely ended by the proposed project since the marsh is within the protected area. Preservation of the marsh was the subject of a special post authorization report and is further discussed in Appendix H, Environment and Cultural Resources.

11. City Obligations. Obligations by the city include completing the pressurizing of sewers on Division, Marquette, and Ripley Streets. Due to the small size of facilities required, it is suggested that the city purchase, install, and operate minor pumping facilities of 6,000 gpm at the Garden Addition.

For the convenience of the reader, plate numbers are keyed to the writeup. For example, all plate numbers pertaining to the Main Street Pump Station are prefixed a "7." and are referenced in section "7." which discusses the Main Street Pump Station.

TABLE A-1

Ten Largest Floods of Record - Mississippi River
At Davenport, Iowa (1)

Order of Magnitude	Peak Discharge c.f.s.	Date of Crest	Duration of Flow Above 170,000 c.f.s. (2)	Crest Stage L&D #15 Tailwater (ft.)
1	307,000	28 Apr 65	16 Apr-12 May	22.5
2	242,000	26 Apr 69	18 Apr- 6 May	19.2
3	216,200 (4)	9 May 75	2 May-19 May	19.16
4	228,000	26 Mar 73	17 Mar-20 May (3)	18.8
5	227,500	27-28 Apr 52	18 Apr- 7 May	18.6
6	221,500	26-27 Apr 51	18 Apr- 9 May	18.2
7	211,000	15 Apr 67	11-22 Apr	17.4
8	189,000	29 Jun 44	28 Jun- 1 Jul	16.4
9	175,900	15 May 54	13-17 May	15.5
10	173,000	31 Mar 45	31 Mar- 1 Apr	15.0

(1) Flood record from L&D 15 TW gage - gage zero = 542.50 m.s.l.

(2) Flood stage at L&D 15 TW is 15.0 ft. with a corresponding discharge of 170,000 c.f.s.

(3) Highest of 3 flood peaks that occurred between 17 March and 20 May 1973.

(4) Discharge affected by Rock River.

Table A-2 Climatological Data

Davenport , Iowa

Station	Length of Record Years	Average Monthly Precipitation in Inches												Avg Annual Precip Inches	Max Yearly Precip Inches	Min Yearly Precip Inches
		Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec			
Davenport (L&D #15)	99	1.56	1.39	2.38	3.11	3.82	4.25	3.67	3.52	3.57	2.40	1.92	1.58	33.17	49.61	17.33
Moline	39	1.59	1.25	2.43	3.59	3.85	4.31	4.20	3.44	3.87	2.67	2.04	1.58	34.82	51.72	20.20
		Normal Monthly Precipitation in Inches*														
Davenport (L&D #15)		1.56	1.32	2.53	3.34	3.59	4.53	3.49	3.58	3.80	2.32	2.11	1.71			
Moline		1.61	1.35	2.39	3.17	3.80	4.37	3.26	3.53	3.25	2.46	1.95	1.65			

		Mean Monthly Temperatures												Avg Yearly Temp	Max Temp	Min Temp
		Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec			
Davenport (L&D #15)	97	22.3	25.8	36.6	50.4	61.6	71.1	76.0	73.8	66.0	54.5	39.4	27.2	50.4	111	-27
Moline	39	22.2	26.5	35.9	50.2	61.1	71.2	75.2	73.2	65.1	54.2	39.0	26.9	50.1	100	-26

		Mean Monthly Snowfall in Inches												Mean Yearly Snowfall Inches
		Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	
Davenport (L&D #15)	76	7.4	6.3	5.4	0.8	T	T	0	T	0	0.2	1.7	5.4	27.2
Moline	37	7.3	5.3	6.4	0.7	0	0	0	0	0	T	1.4	6.9	28.0

*Corresponds to climatological normals established by U. S. Weather Bureau for period 1931-1960.

TABLE A-3

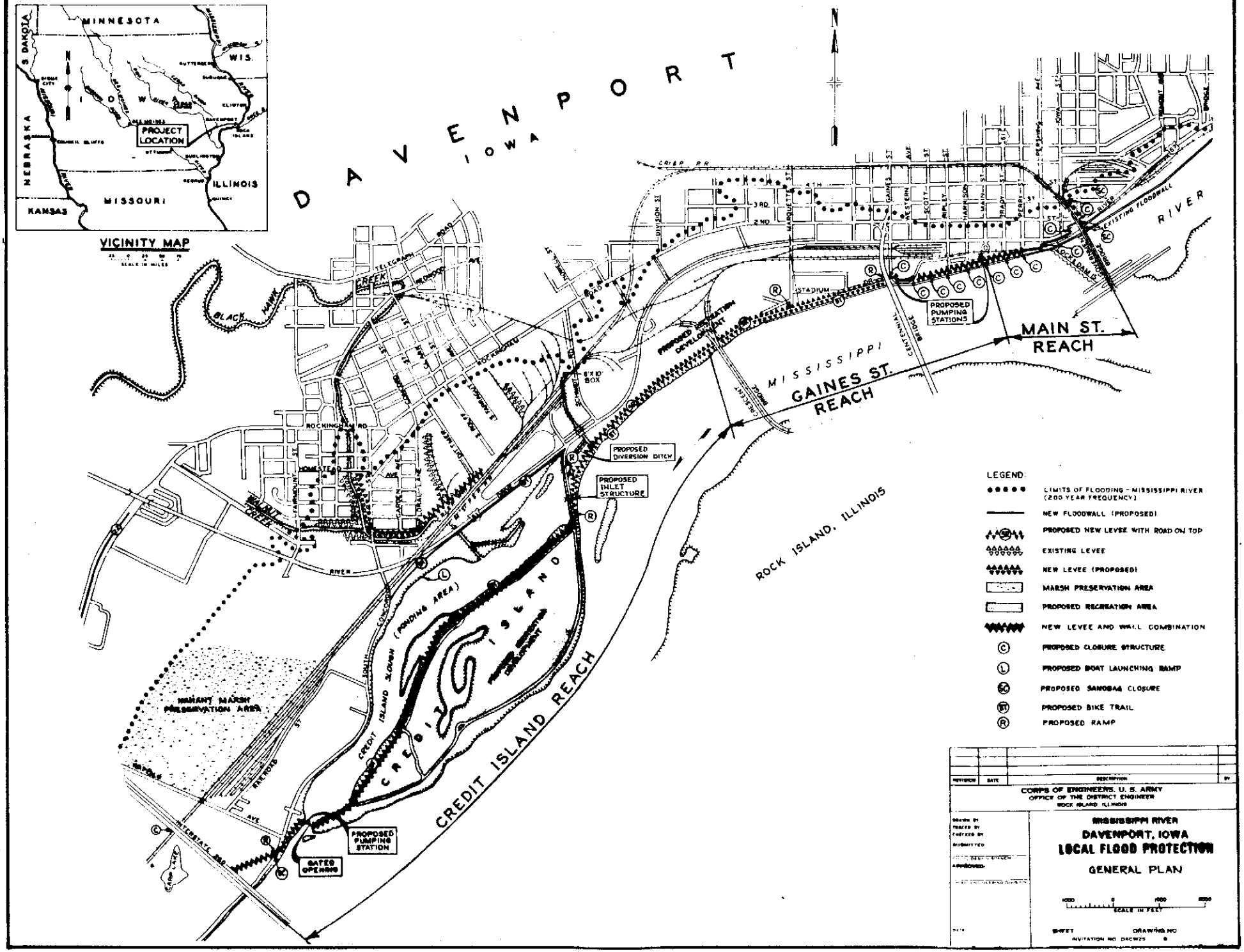
Davenport, Iowa
INTERIOR DRAINAGE SUMMARY

Reach	Sub-Area	Drainage Areas (Acres)	Gravity Design Flow (c.f.s.)	Gravity Design Structure	Blocked Gravity Flow (c.f.s)	Blocked Gravity Design Structure
II	1A *	181	Not affected.	None, except gatewells on existing outlets.	250	Preliminary consideration pending receipt of city plans for area. 9,000 g.p.m. pump station (assuming adequate ponding).
	1B	41	Not affected.	None	250	
	2A1	275	Not affected	None	250	
I	2A2	48	Not affected	None	560	Flows transferred to 2B by 48"/60" RCB.
	2B	115	Not affected	None	560	35,000 g.p.m. pump station to serve sub-areas 2A2, 2B, & 3A.
	3A *	335	Not affected	None	560	
III	3B *	996	Not affected	None	2200	Small portion of area transferred to 2B by 36" RCP.
	4	960	Facilities to match upstream existing.	Twin 10'x10' RCB with Gatewell.	2200	Flows transferred to Area 5 by diversion ditch.
	5	6700	Not affected	40' wide navigation gate at downstream end of slough.	2200	70,000 g.p.m. pump station to serve areas 3B, 4, & 5, using Credit Island Slough as a ponding area.

* Served by pressure sewer.

TABLE 4
Hydrologic Data
High Flow Periods
Mississippi River at Davenport, Iowa

Year	Date of Crest	Peak Flow 1,000 c.f.s.	#15 T.W. Elev. (m.s.l.)	River Elev. At Gaine St.	Total Days Closure	Rate of Max. Seepage AF/Day	Total Rainfall (Inches)	Max. Daily Rainfall	Estimated Interior 0 Pump	Peak Stage 40,000 GPM
1875	29-30 Apr	107	557.2	556.9	13	-	1.75	0.66	557.9	-
1876	7-10 May	165	557.1	556.8	31	-	3.71	2.19	558.0	-
1876	30 May - 2 Jun	165	557.1	556.8	12	-	2.09	1.85	558.6	-
1880	25 Jun	250	561.6	561.3	20	1.10	2.96	1.20	561.3	558.1
1881	12 May	174	557.6	557.3	9	-	0.34	8.31	557.4	-
1881	26-27 Oct	237	561.0	560.7	35	0.82	9.61	2.29	561.7	559.3
1882	22 Apr	186	558.3	558.0	13	-	1.82	0.86	559.0	557.5
1883	1 May	177	557.8	557.5	17	-	3.01	0.91	557.3	-
1884	27-28 Mar	144	556.0	555.7	2	-	0.17	0.17	-	-
1884	27-29 Sep	144	556.0	555.7	3	-	0.94	0.53	558.0	-
1884	18 Oct	146	556.0	555.7	3	-	0.02	0.02	557.0	-
1885	1-2 May	157	556.7	556.4	8	-	1.75	1.15	-	-
1887	28 Apr	149	556.3	556.0	5	-	0.32	0.23	557.0	-
1888	14-16 May	248	561.5	561.2	32	1.06	11.37	1.81	562.2	558.6
1890	28-29 Apr	142	555.8	555.5	1	-	0	-	-	-
1892	26 Jun	238	561.0	560.7	45	0.82	13.39	4.36	561.7	559.6
1893	23 Apr	153	556.5	556.2	11	-	2.14	0.06	-	-
1893	12-13 May	174	557.6	557.3	23	-	1.93	0.64	558.3	-
1894	30-31 May	157	556.7	556.4	8	-	0	-	-	-
1896	28-29 May	148	556.2	555.9	5	-	0.43	0.43	-	-
1897	17-18 Apr	198	558.9	558.8	23	-	2.44	1.03	559.6	557.6
1899	24-27 Jun	149	556.8	556.5	7	-	0.17	0.13	-	-
1900	20 Oct	142	555.8	555.5	1	-	0	-	-	-
1903	4-8 Jun	165	557.1	556.8	14	-	3.73	2.03	557.8	-
1903	29-30 Sep	176	557.7	557.4	9	-	1.21	0.44	558.4	-
1905	21-22 Jun	172	557.6	557.3	10	-	1.32	0.94	558.3	-
1906	22-25 Apr	169	557.4	557.1	18	-	0.67	0.46	558.1	-
1907	11-13 Apr	171	557.4	557.1	11	-	0.13	0.11	558.1	-
1916	5 May	195	558.8	558.5	32	-	4.40	1.11	559.5	558.0
1916	8-10 Jun	146	556.0	555.7	6	-	0.51	0.39	-	-
1917	21 Apr	142	555.8	555.5	3	-	0.02	0.02	-	-
1919	25 Apr	166	557.2	556.9	11	-	1.66	0.78	557.9	-
1919	5-6 May	140	555.7	555.4	7	-	0.56	0.56	-	-
1920	9 Apr	222	560.2	559.9	15	0.43	3.27	1.02	560.9	557.4
1922	23 Apr	212	559.7	559.4	15	0.19	0.76	0.70	558.9	557.45
1929	2-3 Apr	146	556.0	555.7	3	-	1.12	1.08	-	-
1929	19-20 Apr	146	556.0	555.7	7	-	3.32	1.72	-	-
1938	23 Sep	169	557.4	557.1	7	-	0	-	557.0	-
1939	9 Apr	146	556.0	557.1	1	-	0.53	0.53	-	-
1942	13 Jun	170	557.4	557.1	9	-	3.63	2.43	558.4	-
1943	16-17 Apr	154	556.5	556.2	8	-	0.02	-	-	-
1943	13 Jun	143	555.9	555.6	3	-	0.08	0.08	-	-
1943	30 Jun	159	557.8	557.5	15	-	0.50	0.22	557.5	-
1944	26 May	142	555.8	555.5	2	-	0.41	0.41	-	-
1944	28 Jun	169	557.4	557.1	10	-	0.70	0.29	558.4	-
1945	29 Mar	164	557.1	556.8	11	-	1.06	0.33	558.1	-
1946	28 Mar	145	556.0	555.7	4	-	0	0.00	-	-
1951	26-27 Apr	221	560.2	559.9	24	0.43	5.71	1.72	560.9	558.8
1952	27 Apr	225	560.4	560.1	22	0.47	1.03	0.35	559.5	556.8
1954	14 May	176	557.7	557.4	8	-	0.03	0.03	557.0	-
1960	18 May	151	556.4	556.1	4	-	1.65	0.75	-	-
1961	2 Apr	143	555.9	555.6	2	-	0	0.00	-	-
1965	28 Apr	307	564.5	564.2	29	2.50	4.42	2.26	562.2	-
1967	14 Apr	201	559.0	558.7	14	-	2.55	0.69	559.7	557.0
1969	25 Apr	230	560.6	560.3	22	0.624	1.99	0.69	560.6	557.0
1971	24 Apr	138	556.8	556.5	7	-	1.43	1.42	-	-
1972	2 May	153	556.5	556.2	6	-	0.09	0.00	-	-
1973	25 Mar	207	559.5	559.2	17	0.96	3.24	1.95	560.6	558.9
1973	24 Apr	162	557.0	556.7	8	-	7.43	4.26	-	556.6
1973	13 May	174	557.6	557.3	11	-	0.08	0.04	560.2	-
1974	24 Jun	158	556.8	556.5	5	-	4.08	2.69	-	-
1975	9 May	214	559.8	559.5	18	0.24	1.21	0.62	559.6	557.0

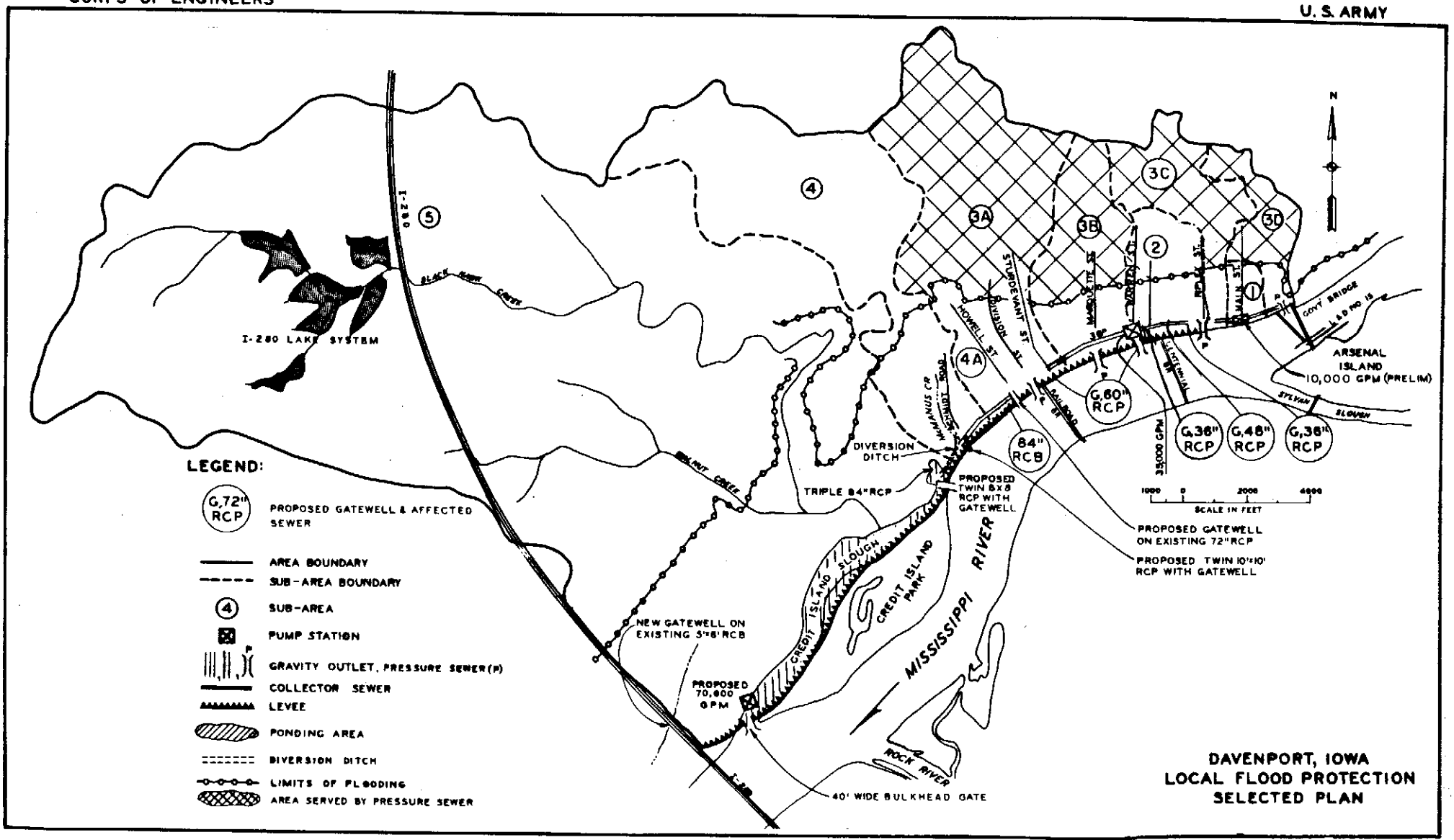


DAVENPORT
IOWA
MINNESOTA
S. DAKOTA
WIS.
NEBRASKA
KANSAS
MISSOURI
ILLINOIS
PROJECT LOCATION
VICINITY MAP
SCALE IN MILES
BLACK HAWK CREEK
MISSISSIPPI RIVER
ROCK ISLAND, ILLINOIS
MAIN ST. REACH
GAINES ST. REACH
CREDIT ISLAND REACH
CREDIT ISLAND
CREDIT ISLAND SLOUGH (PONDING AREA)
HAHANT MARSH PRESERVATION AREA
PROPOSED PUMPING STATIONS
PROPOSED DIVERSION DITCH
PROPOSED INLET STRUCTURE
PROPOSED PUMPING STATION
GATED OPENING
EXISTING FLOODWALL
PROPOSED RECREATION DEVELOPMENT
8'X10' BOX
LEGEND:
LIMITS OF FLOODING - MISSISSIPPI RIVER (200 YEAR FREQUENCY)
NEW FLOODWALL (PROPOSED)
PROPOSED NEW LEVEE WITH ROAD ON TOP
EXISTING LEVEE
NEW LEVEE (PROPOSED)
MARSH PRESERVATION AREA
PROPOSED RECREATION AREA
NEW LEVEE AND WALL COMBINATION
PROPOSED CLOSURE STRUCTURE
PROPOSED BOAT LAUNCHING RAMP
PROPOSED SANDBAG CLOSURE
PROPOSED BIKE TRAIL
PROPOSED RAMP
CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
ROCK ISLAND ILLINOIS
MISSISSIPPI RIVER
DAVENPORT, IOWA
LOCAL FLOOD PROTECTION
GENERAL PLAN
SCALE IN FEET
SHEET
DRAWING NO
INVITATION NO DACW25

PLATE A-1.1

CORPS OF ENGINEERS

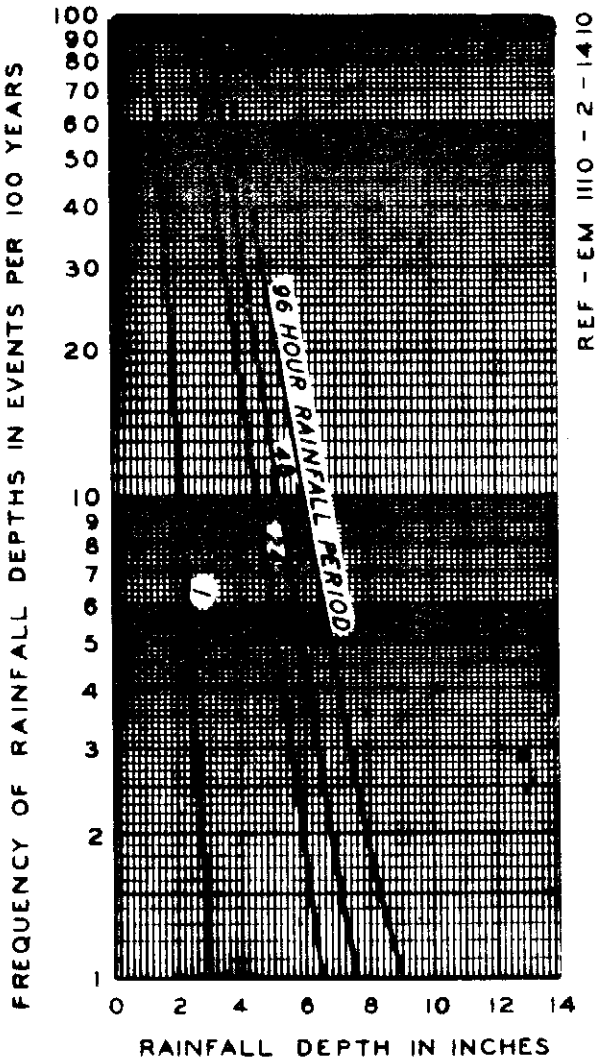
U. S. ARMY



DAVENPORT, IOWA
LOCAL FLOOD PROTECTION
SELECTED PLAN

PLATE A-1,2

CORPS OF ENGINEERS



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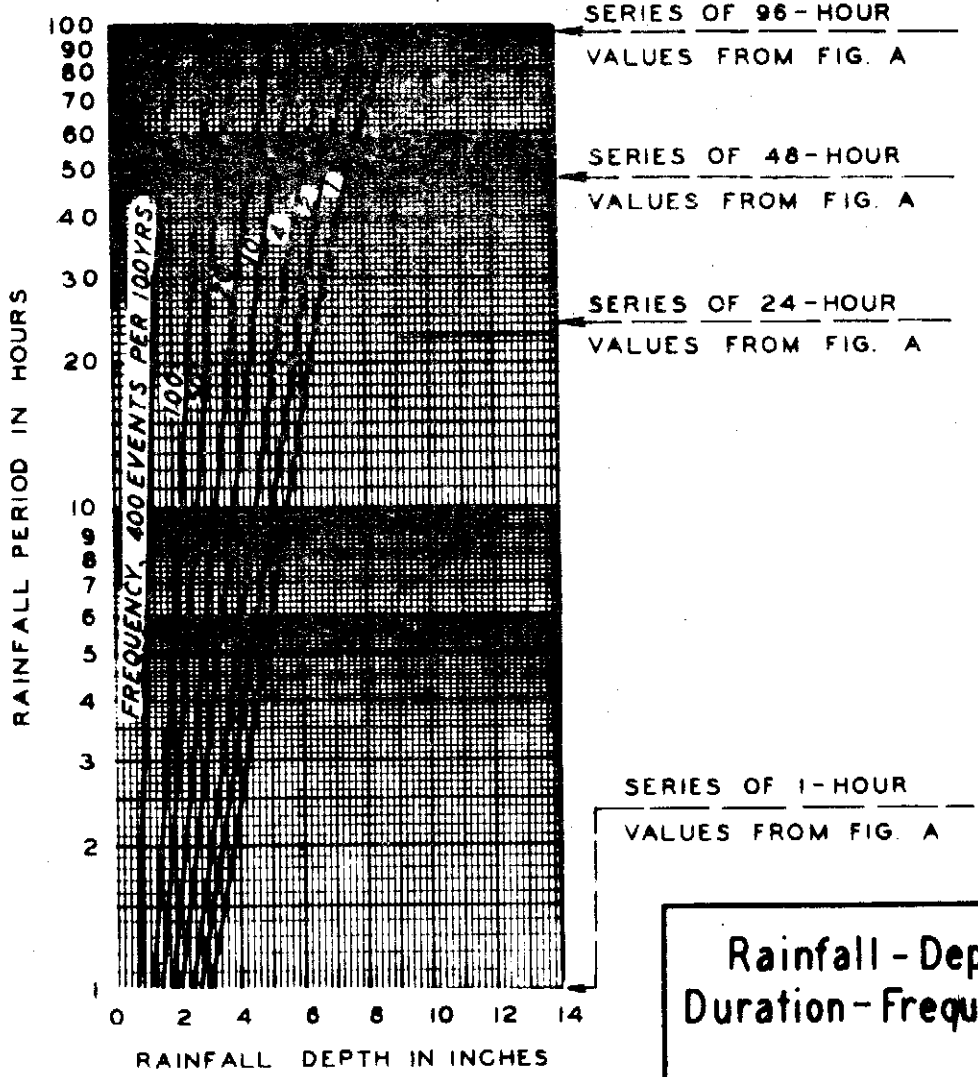
PARAMETERS ARE RAINFALL PERIODS IN HOURS

FIG. A



PARAMETERS ARE IN EVENTS PER 100 YEARS

FIG. B

Rainfall - Depth Duration - Frequency

Relations at Davenport, Iowa

U.S. ARMY

PLATE A-2.1

1

TABULATION OF DATA CORRESPONDING TO ENVELOPING CURVES
OF ACCUMULATIVE RAINFALL-DURATION - FREQUENCY RELATIONS

DAVENPORT, IOWA

Line Ref	Max Rainfall Duration in Hrs.	Rainfall in inches depth corresponding to various average frequencies and durations in hours. Average Exceedence Interval, in Years					
		1	2	5	10	50	100
1	2						
1	1	1.26	1.50	1.91	2.19	2.75	3.02
2	2	1.57	1.86	2.35	2.68	3.44	3.78
3	3	1.71	2.09	2.60	2.96	3.85	4.20
4	4	1.82	2.21	2.78	3.18	4.13	4.49
5	5	1.92	2.31	2.91	3.33	4.34	4.73
6	6	2.00	2.40	3.04	3.46	4.52	4.92
7	12	2.28	2.78	3.47	3.99	5.20	5.65
8	18	2.42	2.99	3.73	4.29	5.60	6.09
9	24	2.55	3.15	3.90	4.50	5.91	6.40
10	48	2.95	3.60	4.49	5.20	6.79	7.52
11	72	3.20	3.89	4.90	5.66	7.48	8.34
12	96	3.40	4.10	5.20	6.02	8.02	9.00
Rainfall by one-hour increments during maximum 6-hour accumulations							
13	0-1	1.26	1.50	1.91	2.19	2.75	3.02
14	1-2	0.31	0.36	0.44	0.49	0.69	0.76
15	2-3	0.14	0.23	0.25	0.28	0.41	0.42
16	3-4	0.11	0.12	0.18	0.22	0.28	0.29
17	4-5	0.10	0.10	0.13	0.15	0.21	0.24
18	5-6	0.08	0.09	0.13	0.13	0.18	0.19
Rainfall by 6- hour increments during maximum 24 hour accumulation							
19	0-6	2.00	2.40	3.04	3.46	4.52	4.92
20	6-12	0.28	0.38	0.43	0.53	0.68	0.73
21	12-18	0.14	0.21	0.26	0.30	0.40	0.44
22	18-24	0.13	0.16	0.17	0.21	0.31	0.31
Rainfall by 24 hour increments during maximum 96-hour accumulation							
23	0 - 24	2.55	3.15	3.90	4.50	5.91	6.40
24	24 - 48	0.40	0.45	0.59	0.70	0.88	1.12
25	48 - 72	0.25	0.29	0.41	0.46	0.69	0.82
26	72 - 96	0.20	0.21	0.30	0.36	0.54	0.66

HYPOTHETICAL HYETOGRAPH OF RAINFALL AND RAINFALL-EXCESS

CORRESPONDING TO 1 YEAR FREQUENCY EVENTS

Line Ref.	Hypothetical flood time (Acum. hours to end of Prd.)	Hypothetical hyetograph: 24 Hr. Inc. of rainfall		6 Hr. Inc. of rainfall		1 Hr. Inc. of rainfall		Accum. rainfall inches	Estimated rainfall excess (Re) and assumed effective duration: % surface runoff	F av In. per hr	Re in time Inc.	Acum. Re Inch	Assumed Time of Re, in Clock Hrs.
		order of mag.	Inches	order of mag.	Inches	order of mag.	Inches						
1	2	3	4	5	6	7	8	9	10	11	12	13	14
1	24	4	.28										
2	48	3	.27										
3	72	2	.40										
4	78			4	.15								
5	84			3	.20								
6	90			2	.30								
7	91					6	.05		0.0	0.05	0	0	
8	92	1				5	.10		(50)	0.05	.05	0.05	
9	93			1		4	.14		(64)	0.05	.09	0.14	
10	94					3	.16		(69)	0.05	.11	0.25	
11	95					2	.26		(81)	0.05	.21	0.46	
12	96		2.65		2.00	1	1.29		(96)	0.05	1.24	1.70	

PLATE A-2.3

HYPOTHETICAL HYETOGRAPH OF RAINFALL AND RAINFALL-EXCESS
CORRESPONDING TO 2 YEAR FREQUENCY EVENTS

Line Ref.	Hypothetical flood time (Acum hours to end of Prd.	Hypothetical hyetograph: 24 Hr. increment of rainfall		6 Hr. Inc. of rainfall		1 Hr. Inc. of rainfall		Accum rainfall inches	Estimated rainfall excess (Re) and assumed effective duration: % surface runoff	F av In. per Hr	R_e in time Inc.	Acum. R_e Inch	Assumed Time of Re, in Clock Hrs.
		order of mag.	Inches	order of mag.	Inches	order of mag.	Inches						
1	2	3	4	5	6	7	8	9	10	11	12	13	14
1	24	4	.23										
2	48	3	.35										
3	72	2	.48										
4	78			4	.18								
5	84			3	.22								
6	90			2	.35								
7	91					6	.10		(50)	0.05	0.05	0.05	
8	92	1		1		5	.10		(50)	0.05	0.05	0.10	
9	93					4	.15		(67)	0.05	0.10	0.20	
10	94					3	.20		(75)	0.05	0.15	0.35	
11	95					2	.34		(85)	0.05	0.29	0.64	
12	96		3.15		2.40	1	1.51		(97)	0.05	1.46	2.10	

PLATE A-2.4

HYPOTHETICAL HYETOGRAPH OF RAINFALL AND RAINFALL-EXCESS
CORRESPONDING TO 5 YEAR FREQUENCY EVENTS

Line Ref.	Hypothetical flood time (Acum hours to end of Prd.	Hypothetical hyetograph: 24 Hr. increment of rainfall		6 Hr. Inc. of rainfall		1 Hr. Inc. of rainfall		Accum rainfall inches	Estimated rainfall excess (Re) and assumed effective duration: % surface runoff	F av In. per Hr	R_e in time Inc.	Acum. R_e Inch	Assumed Time of Re, in Clock Hrs.
		order of mag.	Inches	order of mag.	Inches	order of mag.	Inches						
1	2	3	4	5	6	7	8	9	10	11	12	13	14
1	24	4	.37										
2	48	3	.55										
3	72	2	.60										
4	78			4	.25								
5	84			3	.30								
6	90			2	.45								
7	91					6	.10		(50)	0.05	0.05	0.05	
8	92	1		1		5	.14		(64)	0.05	0.09	0.14	
9	93					4	.13		(62)	0.05	0.08	0.22	
10	94					3	.23		(78)	0.05	0.18	0.40	
11	95					2	.46		(89)	0.05	0.41	0.81	
12	96		3.95		2.95	1	1.89		(97)	0.05	1.84	2.65	

PLATE A-2.5

HYPOTHETICAL HYETOGRAPH OF RAINFALL AND RAINFALL-EXCESS

CORRESPONDING TO 10 YEAR FREQUENCY EVENTS

Line Ref.	Hypothetical flood time (Acum. hours to end of Prd.)	Hypothetical hyetograph: 24 Hr. Inc. of rainfall		6 Hr. Inc. of rainfall		1 Hr. Inc. of rainfall		Accum. rainfall inches	Estimated rainfall excess (Re) and assumed effective duration				
		order of mag.	Inches	order of mag.	Inches	order of mag.	Inches		% surface runoff	F av In. per hr	Re in time Inc.	Acum. Re Inch	Assumed Time of Re, in Clock Hrs.
1	2	3	4	5	6	7	8	9	10	11	12	13	14
1	24	4	.44										
2	48	3	.51										
3	72	2	.72										
4	78			4	.27								
5	84			3	.31								
6	90			2	.55								
7	91					6	.11		(55)	0.05	0.06	0.06	
8	92	1				5	.14		(64)	0.05	0.09	0.15	
9	93			1		4	.20		(75)	0.05	0.15	0.30	
10	94					3	.29		(83)	0.05	0.24	0.54	
11	95					2	.45		(89)	0.05	0.40	0.94	
12	96		4.48		3.35	1	2.16		(98)	0.05	2.11	3.05	

PLATE A-2.6

HYPOTHETICAL HYETOGRAPH OF RAINFALL AND RAINFALL-EXCESS

CORRESPONDING TO 25 YEAR FREQUENCY EVENTS

Line Ref.	Hypothetical flood time (Acum. hours to end of Prd.)	Hypothetical hyetograph						Accum. rainfall inches	Estimated rainfall excess (Re) and assumed effective duration				
		24 Hr. Inc. of rainfall		6 Hr. Inc. of rainfall		1 Hr. Inc. of rainfall			% surface runoff	F av In. per hr	Re in time Inc.	Acum. Re Inch	Assumed Time of Re, in Clock Hrs.
		order of mag.	Inches	order of mag.	Inches	order of mag.	Inches						
1	2	3	4	5	6	7	8	9	10	11	12	13	14
1	24	4	.81										
2	48	3	.80										
3	72	2	.94										
4	78			4	.30								
5	84			3	.40								
6	90			2	.56								
7	91					6	.14		(64)	0.05	0.09	0.09	
8	92	1				5	.16		(69)	0.05	0.11	0.20	
9	93			1		4	.20		(75)	0.05	0.15	0.35	
10	94					3	.29		(83)	0.05	0.24	0.59	
11	95					2	.50		(90)	0.05	0.45	1.04	
12	96		5.05		3.79	1	2.50		(98)	0.05	2.45	3.49	

PLATE A-2.7

HYPOTHETICAL HYETOGRAPH OF RAINFALL AND RAINFALL-EXCESS

CORRESPONDING TO 50YEAR FREQUENCY EVENTS

Line Ref.	Hypothetical flood time (Acum. hours to end of Prd.)	Hypothetical hyetograph						Accum. rainfall inches	Estimated rainfall excess (Re) and assumed effective duration				
		24 Hr. Inc. of rainfall		6 Hr. Inc. of rainfall		1 Hr. Inc. of rainfall			% surface runoff	F av In. per hr	Re in time Inc.	Acum. Re Inch	Assumed Time of Re, in Clock Hrs.
		order of mag.	Inches	order of mag.	Inches	order of mag.	Inches						
1	2	3	4	5	6	7	8	9	10	11	12	13	14
1	24	4	.85										
2	48	3	.80										
3	72	2	1.03										
4	78			4	.36								
5	84			3	.41								
6	90			2	.69								
7	91					6	.13		(62)	0.05	0.08	0.08	
8	92	1				5	.19		(74)	0.05	0.14	0.22	
9	93			1		4	.25		(80)	0.05	0.20	0.42	
10	94					3	.34		(85)	0.05	0.29	0.71	
11	95					2	.55		(91)	0.05	0.50	1.21	
12	96		5.67		4.21	1	2.75		(98)	0.05	2.70	3.91	

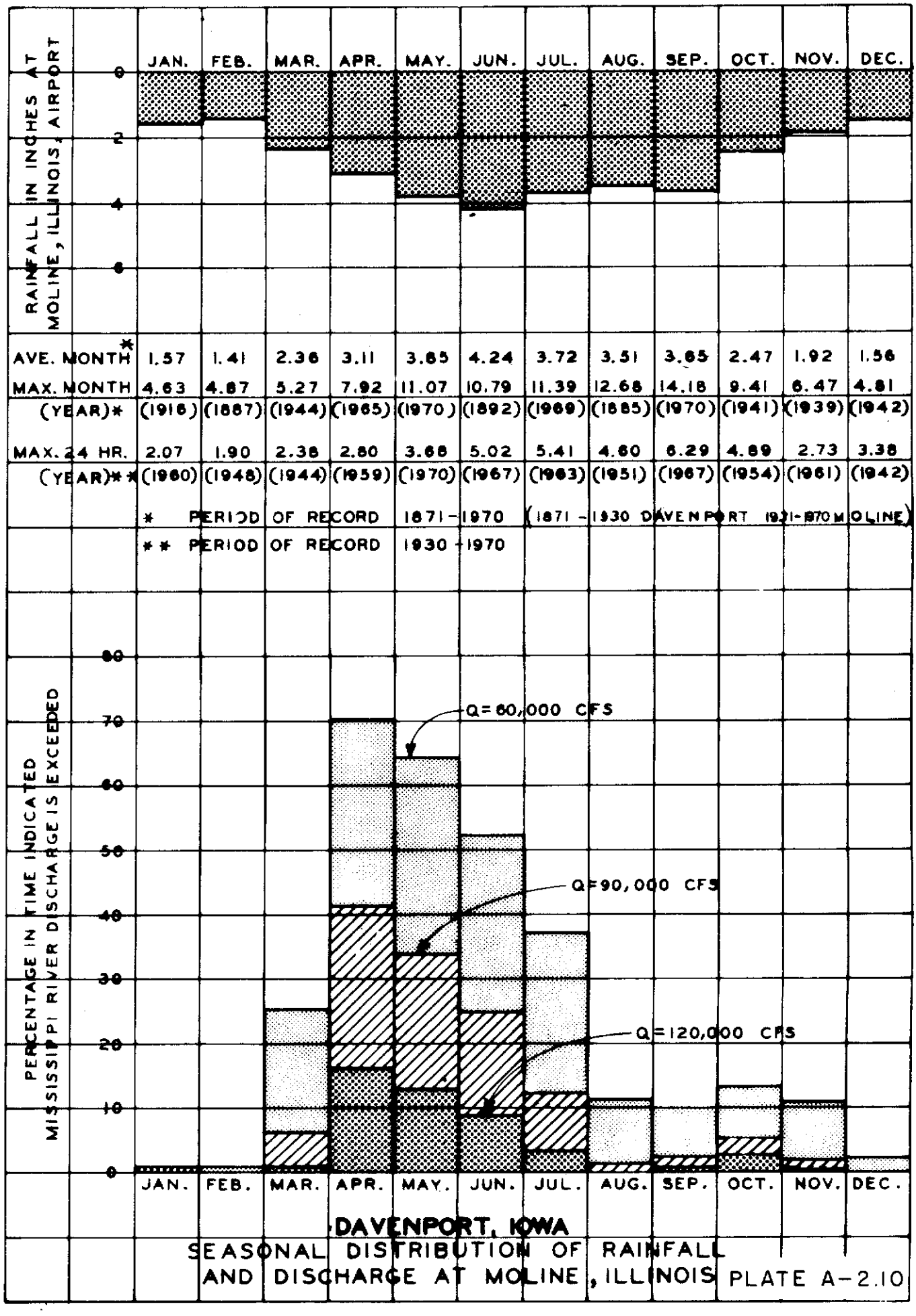
PLATE A-2.8

HYPOTHETICAL HYETOGRAPH OF RAINFALL AND RAINFALL-EXCESS

CORRESPONDING TO 100 YEAR FREQUENCY EVENTS

Line Ref.	Hypothetical flood time (Acum. hours to end of Prd.)	Hypothetical hyetograph						Accum. rainfall inches	Estimated rainfall excess (Re) and assumed effective duration				
		24 Hr. Inc. of rainfall		6 Hr. Inc. of rainfall		1 Hr. Inc. of rainfall			% surface runoff	F av In. per hr	Re in time Inc.	Acum. Re Inch	Assumed Time of Re, in Clock Hrs.
		order of mag.	Inches	order of mag.	Inches	order of mag.	Inches						
1	2	3	4	5	6	7	8	9	10	11	12	13	14
1	24	4	.90										
2	48	3	.85										
3	72	2	1.09										
4	78			4	.41								
5	84			3	.49								
6	90			2	.73								
7	91					6	0.19		(74)	0.05	0.14	0.14	
8	92	1				5	0.23		(78)	0.05	0.18	0.32	
9	93			1		4	0.29		(83)	0.05	0.24	0.56	
10	94					3	0.37		(86)	0.05	0.32	0.88	
11	95					2	0.70		(93)	0.05	0.65	1.53	
12	96		6.41		4.78	1	3.00		(98)	0.05	2.95	4.48	

PLATE A-2.9



DAVENPORT, IOWA

SEASONAL DISTRIBUTION OF RAINFALL AND DISCHARGE AT MOLINE, ILLINOIS

PLATE A-2.10

Davenport, Iowa

Daily Rainfall During Blocked Gravity Conditions
Based L/D #15 T. W., Davenport, Iowa

Period of Record: 1940 to 1975

Date	Max Elevation(2)	Dates w/elevation 566.5	No. of Days	Daily Precipitation (1) (Inches)	Total Precip (Inches)
28 & 29 Apr 1951	18.3	15 Apr to 14 May	30	.45 .11 .54 .62 .01 .09 .22 .07 .09 .71 1.07 1.72 .01	5.71
				.03 .16 .46 .06 .81 .57 .03 .16 .10 .67 2.19 .77	6.01
28 April 1952	18.63	10 Apr to 10 May	31	.01 .40 .21 .01 .03 .27 .18 .55 .06	1.72
				.30 .08 .49 .03 .01 .15 .28 .03 .01 .09 .16 .51 .13	2.27
28 April 1965	22.48	12 Apr to 15 May	34	1.34 .09 .01 .26 2.26 .39 .43 .07 .34 0.01 .76 .23	6.19
				.05 .35 .45 1.22 .65 .20 .43 .40 .76 .35	4.86
15 April 1967	17.4	9 Apr to 25 Apr	17	.08 .44 .69 .02 .15 .61 .43 .13 .15	2.70
				.47 .25 .65 .50 .50 .54 .13	3.04
27 April 1969	19.3	15 Apr to 13 May	29	.30 .69 .01 .30 .08 .29 .14 .47 .01 .44 .25	2.98
				.37 .02 .73 .03 .17 .25 .15 .30 .75 .02 .10 .15 .05 .17	3.26

Plate A-2.11

Davenport, Iowa

Daily Rainfall During Blocked Gravity Conditions
Based L/D #15 T. W., Davenport, Iowa

Period of Record: 1940 to 1975

Date	Max Elevation(2)	Dates w/elevation 566.5	No. of Days	Daily Precipitation (1) (Inches	Total Precip (Inches)
23 Mar 1973	18.84	15 Mar to 6 Apr	23	.17 .02 .85 .05 .03 1.95 .27 .09 .12	3.55
				.07 .11 .28 .11 .05 1.05 .10 .10 .10	1.97
25 & 26 Apr 1973	18.5	18 Apr to 23 May	36	.01 2.37 4.26 .07 .09 .61 2.08 .06 .31	
				.66 .38 .02 .04 .01 .01 .04 .36 .01	11.39
				.05 2.00 1.95 .30 1.10 2.08 .55 .75 .20	
				.03 .01 .32	9.34
6 & 7 June 1973	14.4	4 June to 10 June	7	.71 .05	.76
				.70 .20 .05	.95
9 May 1975	19.16	29 Apr to 21 May	23		
				.03 .02 .13 .15 .18 .29	.80

(1) For each event the top row is precipitation recorded at Moline APWD, Moline, Illinois; The bottom row is precipitation recorded at L/D No. 15, Davenport, Iowa.

(2) Lock and Dam #15 T. W. Gage zero = 542.5

19

Davenport, Iowa

Hourly Rainfall for Day of Max.
Precipitation During Blocked Gravity Conditions

Period of Record: 1940 to 1975

Recorded Rainfall for 24 April 1965

Rainfall in Inches

Hour	APWD Moline	L/D 15 Davenport
1	.81	.30
2	.42	.18
3	.28	.27
4	.13	.17
5	.07	.04
6	.10	.04
7		
8		.01
9		
10		
11		.22
12	.14	
13	.05	.13
14	.04	.05
15	.05	.04
16	.01	
17		
18		
19		
20		
21		.02
22	.14	.17
23	.01	.01
24	.01	.01
TOTAL	2.26	1.66

PLATE A-2.13

20

Davenport, Iowa

Hourly Rainfall for Day of Max.
Precipitation During Blocked Gravity Conditions

Period of Record: 1940 to 1975

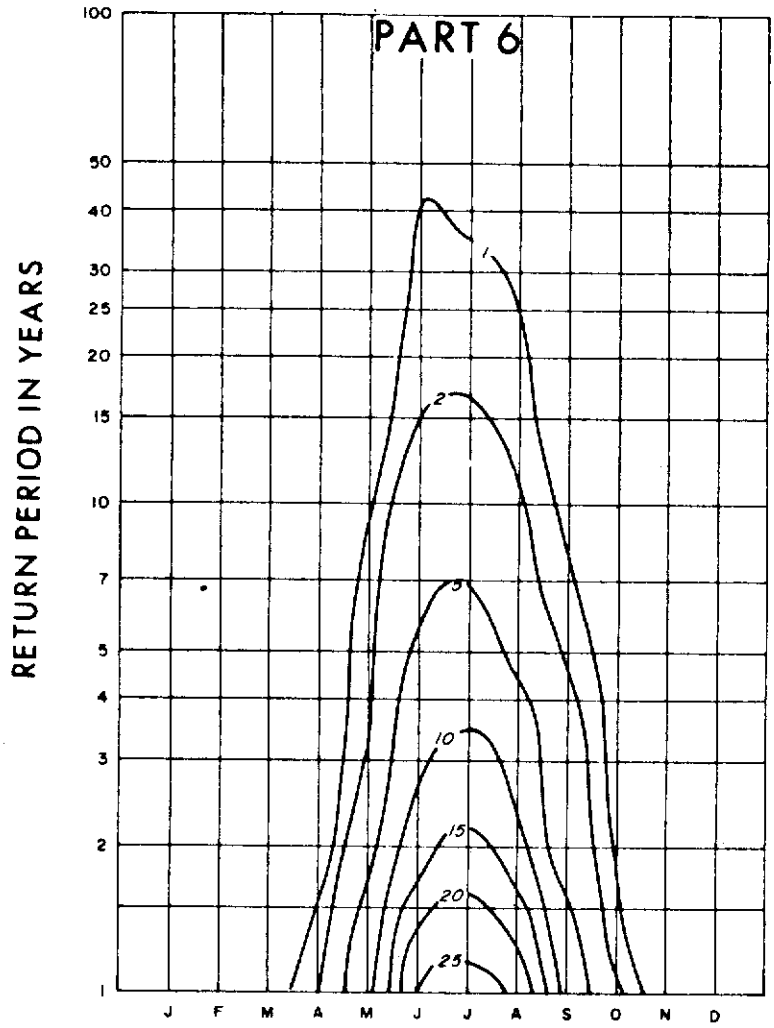
Recorded Rainfall for 20 & 21 April 1973

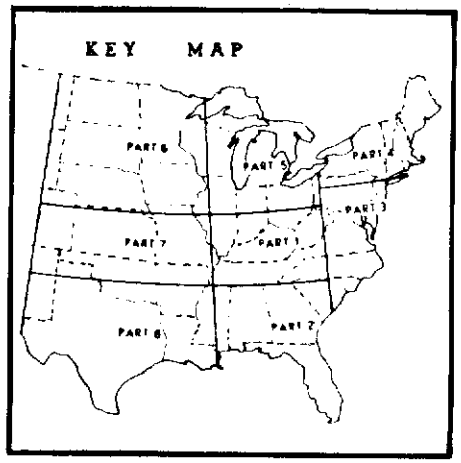
Rainfall in Inches

Hour	APWB Moline	L/D 15 Davenport
1	.57	.2
2	.01	.1
3	.49	.3
4	.53	.2
5	.56	.2
6	.26	
7	.34	.5
8	.27	
9	.01	
10		
11		
12		
13		
14	.03	.1
15	.30	.1
16	.07	.1
17	.12	
18		
19		
20		.1
21		
22	.22	
23	.65	.3
24	1.38	.5
TOTAL	5.81	2.7

PLATE A-2.14

SEASONAL PROBABILITY OF INTENSE 1-HOUR RAINFALL





PROBABILITY IN PERCENT OF OBTAINING A RAINFALL IN ANY MONTH OF A PAR — TICULAR YEAR EQUAL TO OR EXCEEDING THE RETURN PERIOD VALUES TAKEN FROM THE ISOPLUVIAL MAPS.

REVISION	DATE	DESCRIPTION	BY

CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
ROCK ISLAND, ILLINOIS

DRAWN BY
TRACED BY
CHECKED BY
SUBMITTED
CHIEF, DESIGN BRANCH
RECOMMENDED
CHIEF, ENGINEERING DIVISION
APPROVED
COL, CORPS OF ENGINEERS
DISTRICT ENGINEER
ROCK ISLAND DISTRICT
DATE

MISSISSIPPI RIVER
DAVENPORT, IOWA
LOCAL FLOOD PROTECTION
MONTHLY RAINFALL PROBABILITIES
SCALE AS SHOWN

SHEET
DRAWING NO
INVITATION NO DACW25

PLATE A-- 2.15

NO. 340-20 DIETZGEN GRAPH PAPER
20 X 20 PER INCH

DIETZGEN CORPORATION
MADE IN U.S.A.

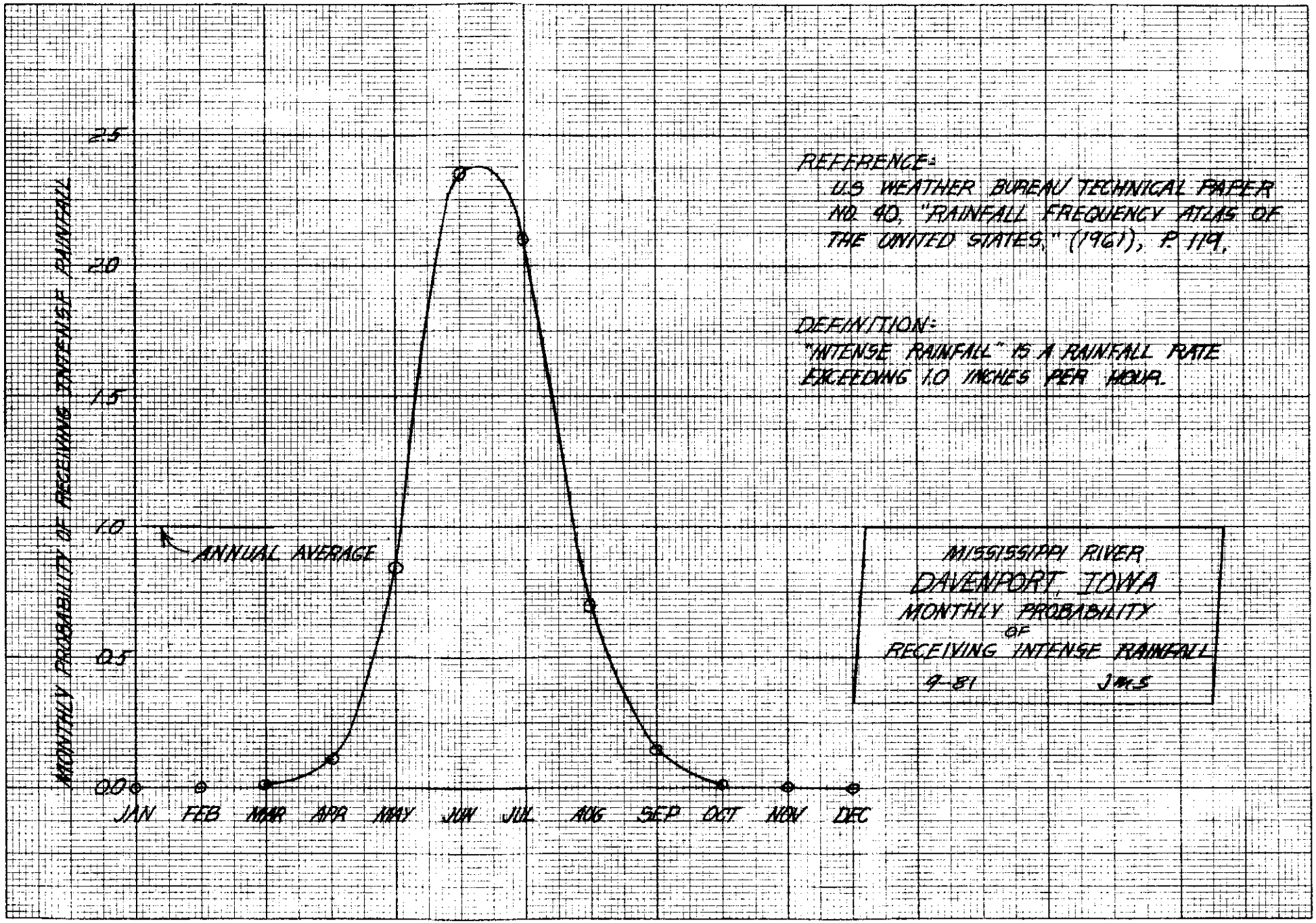
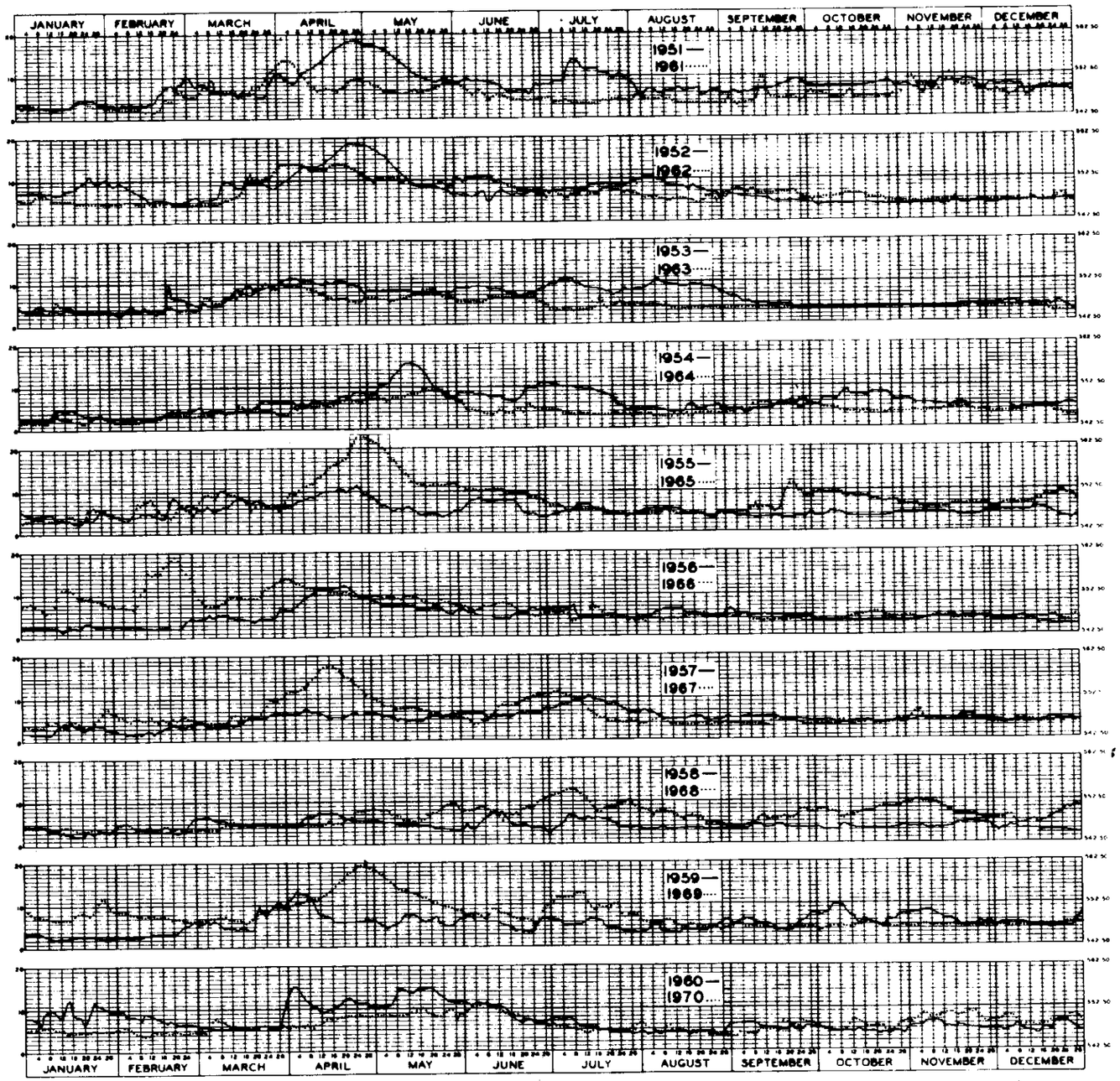


PLATE A-2.16



NOTE
ZERO OF GAGE EL 542.50

HYDROGRAPHS
DAM 15 TAILWATER

REVISION	DATE	DESCRIPTION	BY
1	4 MAY 70	ADDED 1969 HYDROGRAPH	

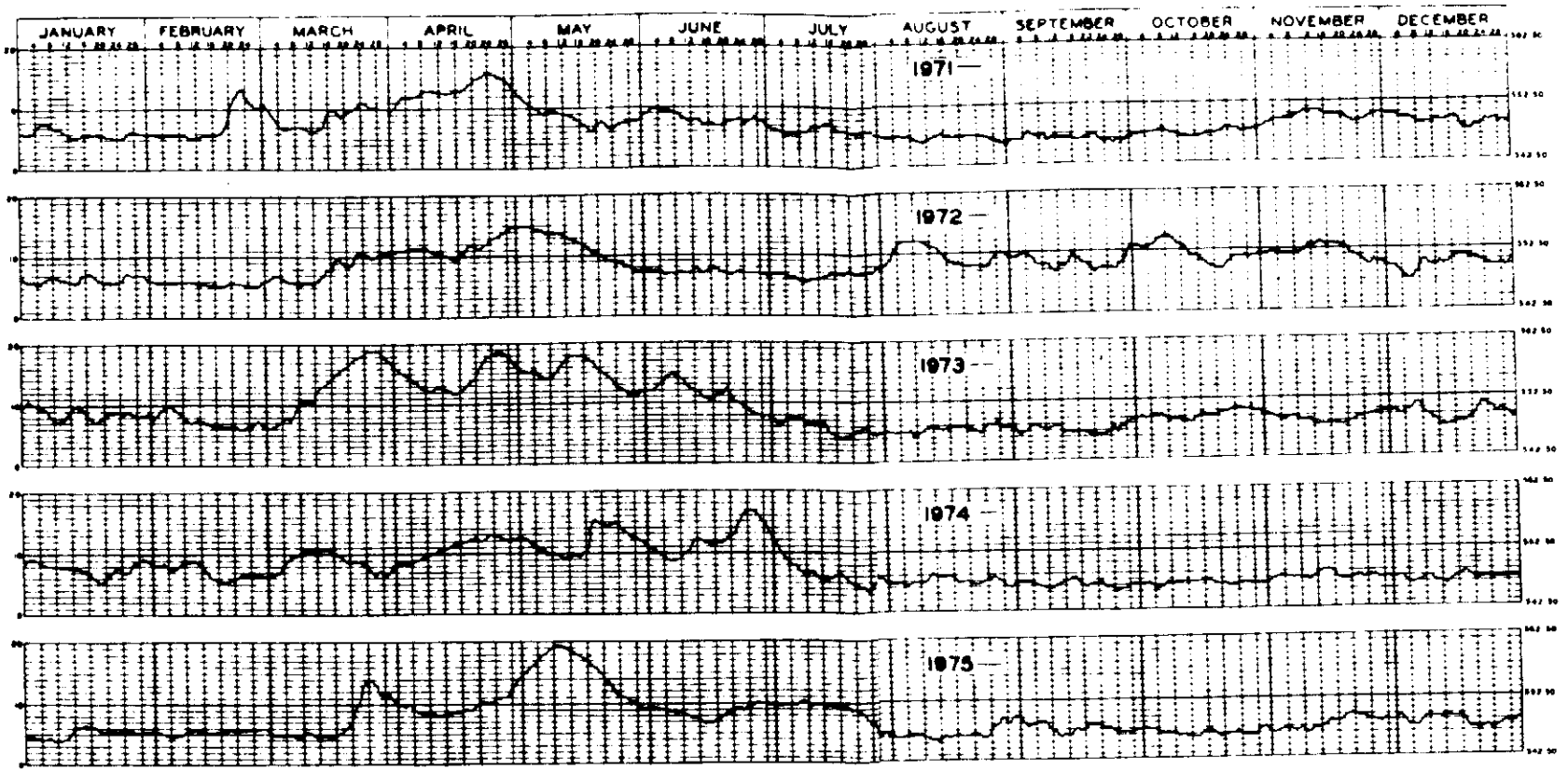
CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
ROCK ISLAND, ILLINOIS

DRAWN BY
TRACED BY
CHECKED BY
SUBMITTED
CHIEF HYDRAULIC SECTION
RECOMMENDED
CHIEF ENGINEERING BR.
APPROVED
CHIEF, ENGINEERING DIV.
DATE 17 APRIL 1969

MISSISSIPPI RIVER
DAVENPORT, IOWA
LOCAL FLOOD PROTECTION
HYDRAULIC DATA
DAM NO 15

SHEET 3 OF 32 DRAWING NO. RI-1
NATION NO. 38CW23 7 B 0027

PLATE A-3.1



HYDROGRAPHS
DAM 15 TAILWATER

NOTE
ZERO OF GAGE EL. 542.50

CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
ROCK ISLAND, ILLINOIS

MISSISSIPPI RIVER
DAVENPORT, IOWA
LOCAL FLOOD PROTECTION
HYDRAULIC DATA
DAM NO. 15

SHEET
DRAWING NO.

PLATE A-32

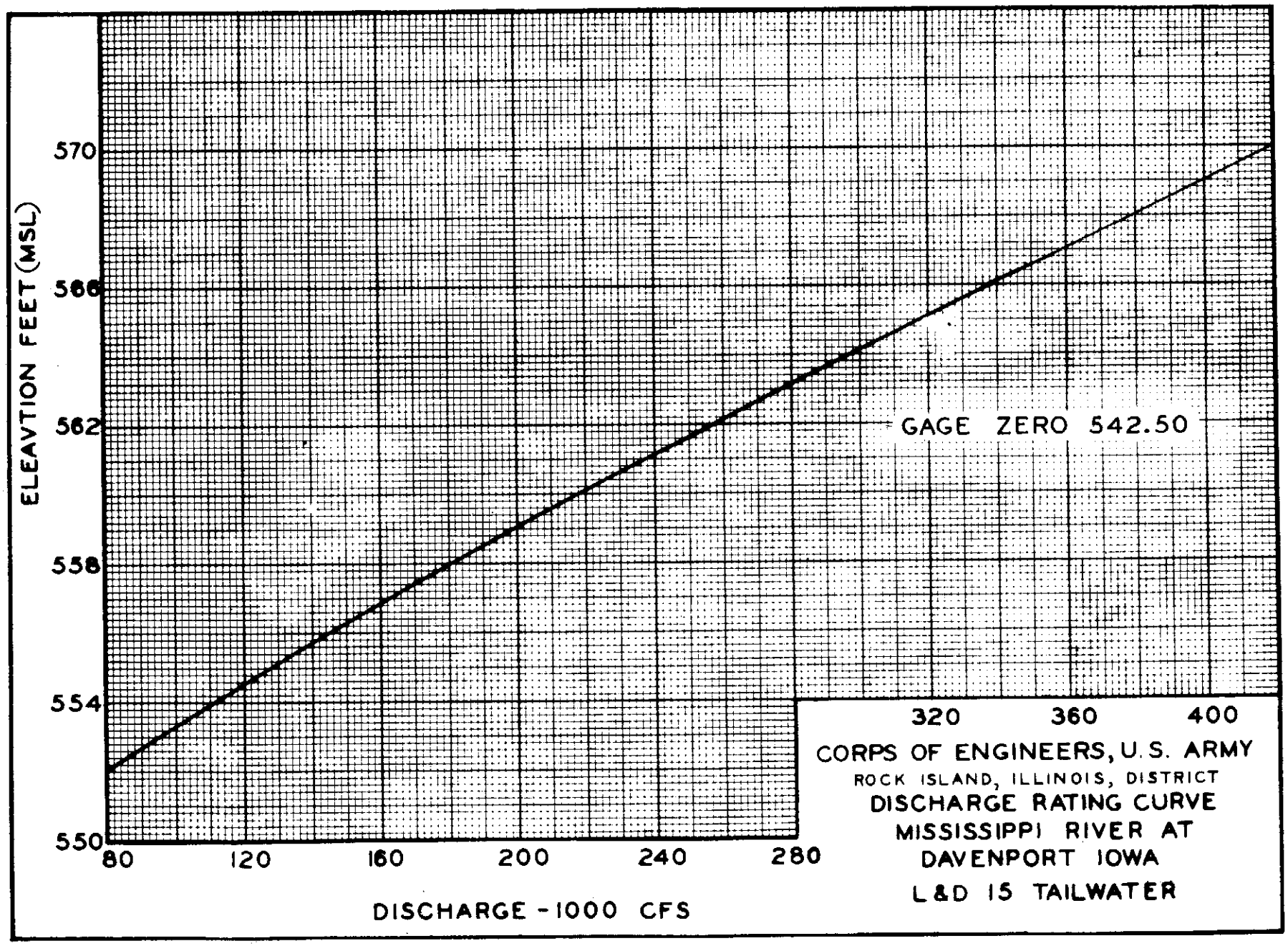

ELEAVTION FEET (MSL)
570
566
562
558
554
550
80
120
160
200
240
280
320
360
400
GAGE ZERO 542.50
DISCHARGE - 1000 CFS
CORPS OF ENGINEERS, U.S. ARMY
ROCK ISLAND, ILLINOIS, DISTRICT
DISCHARGE RATING CURVE
MISSISSIPPI RIVER AT
DAVENPORT IOWA
L&D 15 TAILWATER

PLATE A-3.3

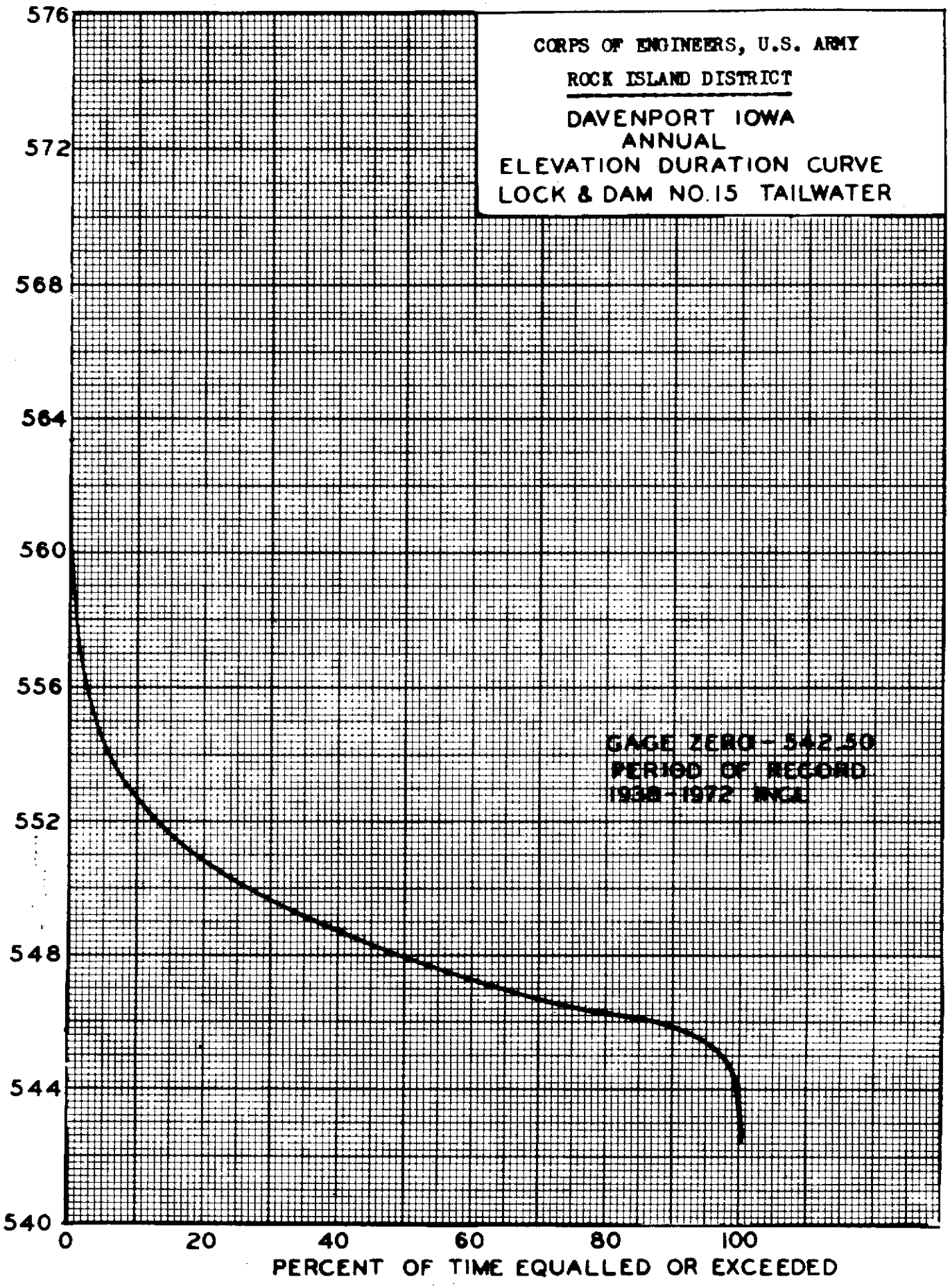
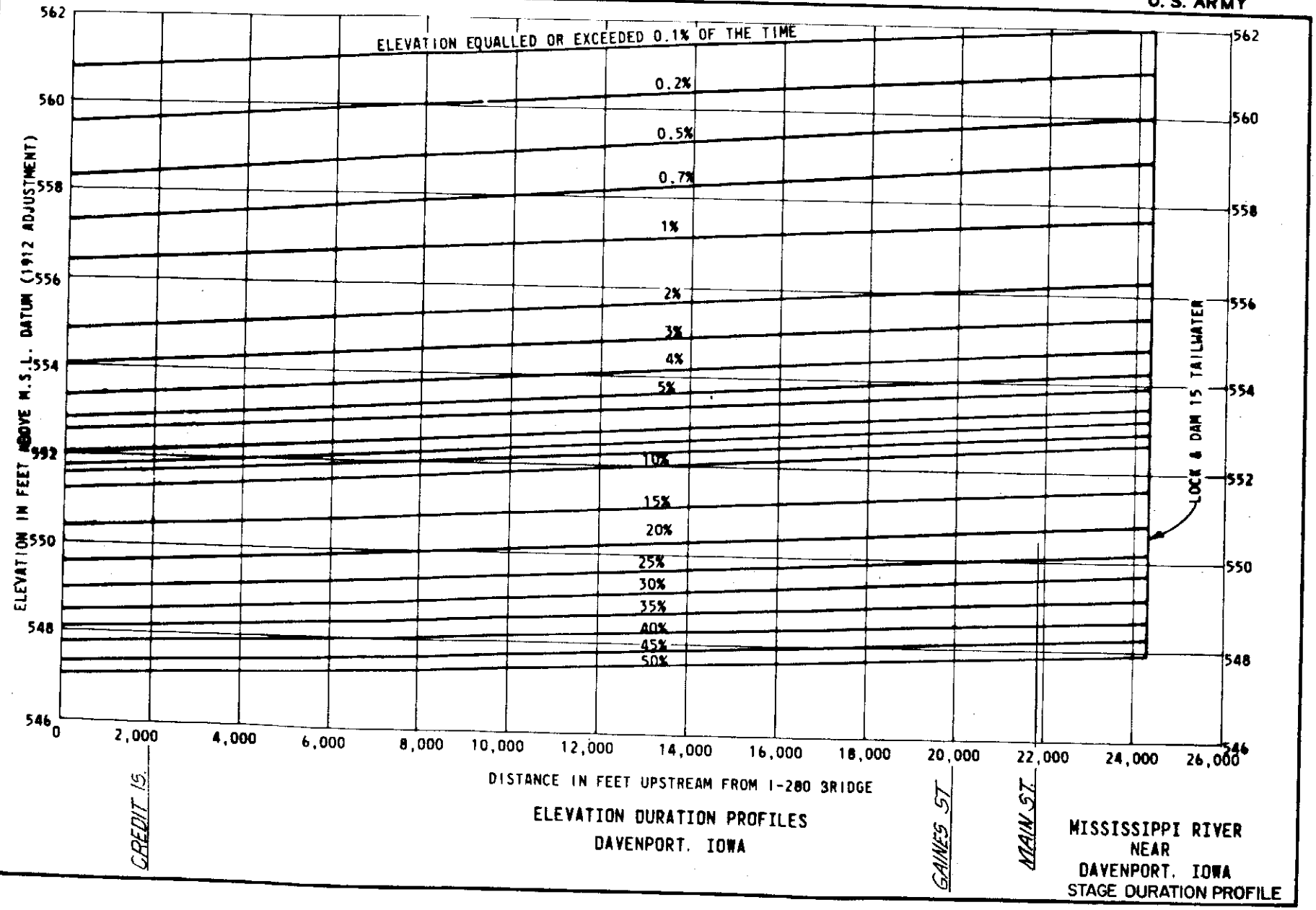

CORPS OF ENGINEERS, U.S. ARMY
ROCK ISLAND DISTRICT
DAVENPORT IOWA
ANNUAL
ELEVATION DURATION CURVE
LOCK & DAM NO. 15 TAILWATER
GAGE ZERO - 542.50
PERIOD OF RECORD
1938-1972 INCL
576
572
568
564
560
556
552
548
544
540
0
20
40
60
80
100
PERCENT OF TIME EQUALLED OR EXCEEDED

PLATE A-3.4

CORPS OF ENGINEERS

U. S. ARMY



ELEVATION DURATION PROFILES
DAVENPORT, IOWA

MISSISSIPPI RIVER
NEAR
DAVENPORT, IOWA
STAGE DURATION PROFILE

PLATE A-3.5

PLATE A-35

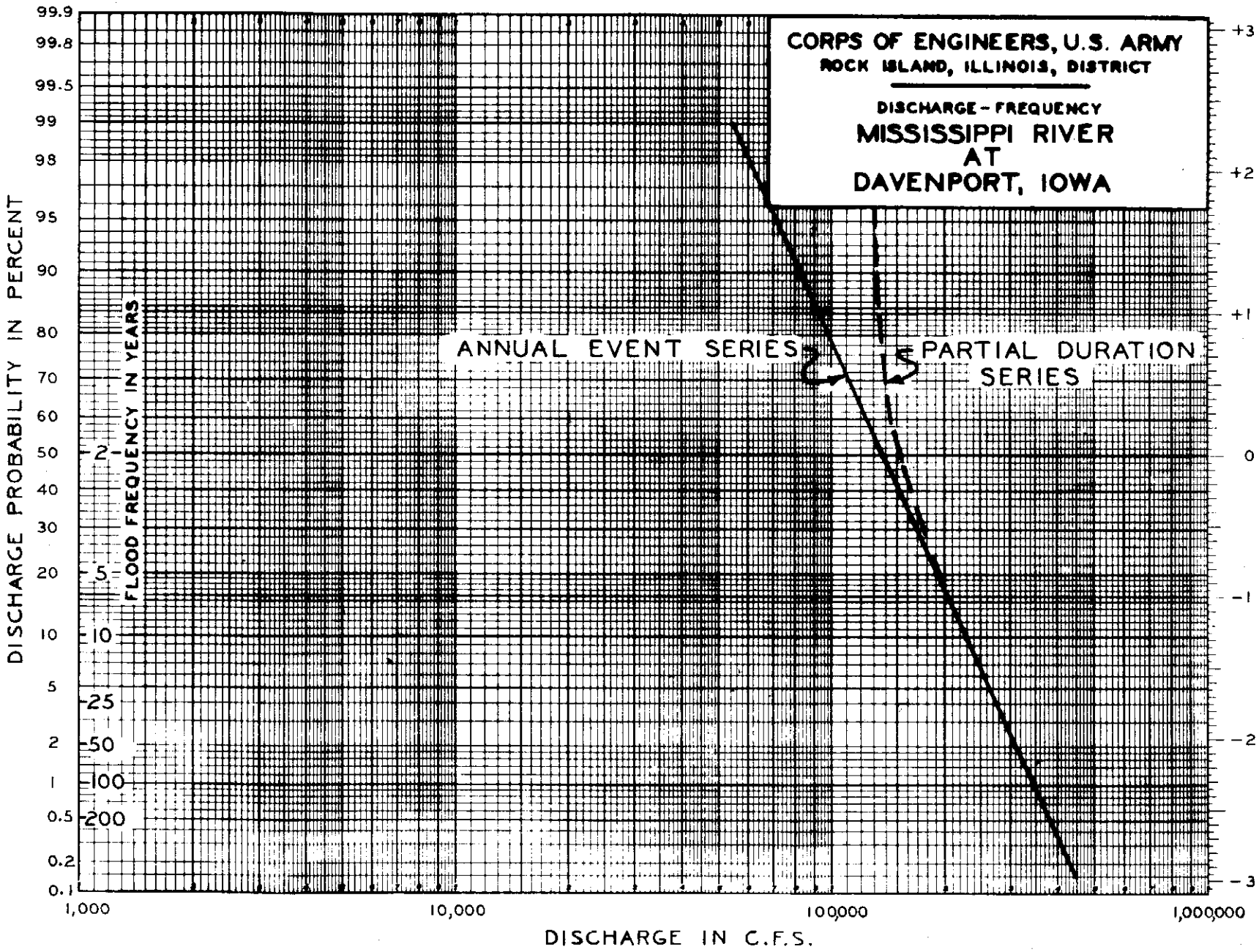
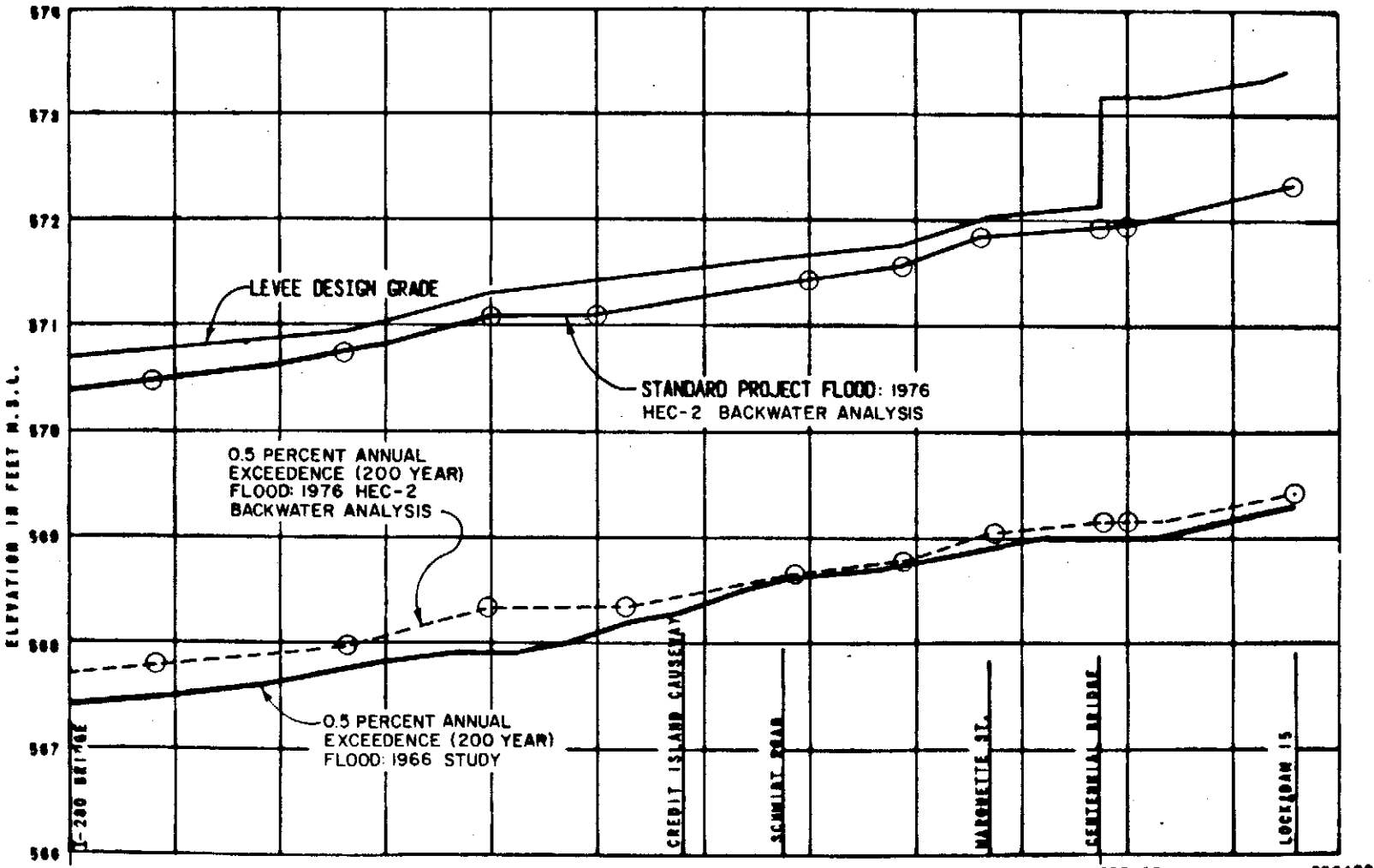

CORPS OF ENGINEERS, U.S. ARMY
ROCK ISLAND, ILLINOIS, DISTRICT
DISCHARGE - FREQUENCY
MISSISSIPPI RIVER
AT
DAVENPORT, IOWA
ANNUAL EVENT SERIES
PARTIAL DURATION SERIES
DISCHARGE PROBABILITY IN PERCENT
99.9
99.8
99.5
99
98
95
90
80
70
60
50
40
30
20
10
5
2
1
0.5
0.2
0.1
FLOOD FREQUENCY IN YEARS
2
5
10
25
50
100
200
+3
+2
+1
0
-1
-2
-3
1,000
10,000
100,000
1,000,000
DISCHARGE IN C.F.S.

PLATE A-3.6

CORPS OF ENGINEERS

U.S. ARMY

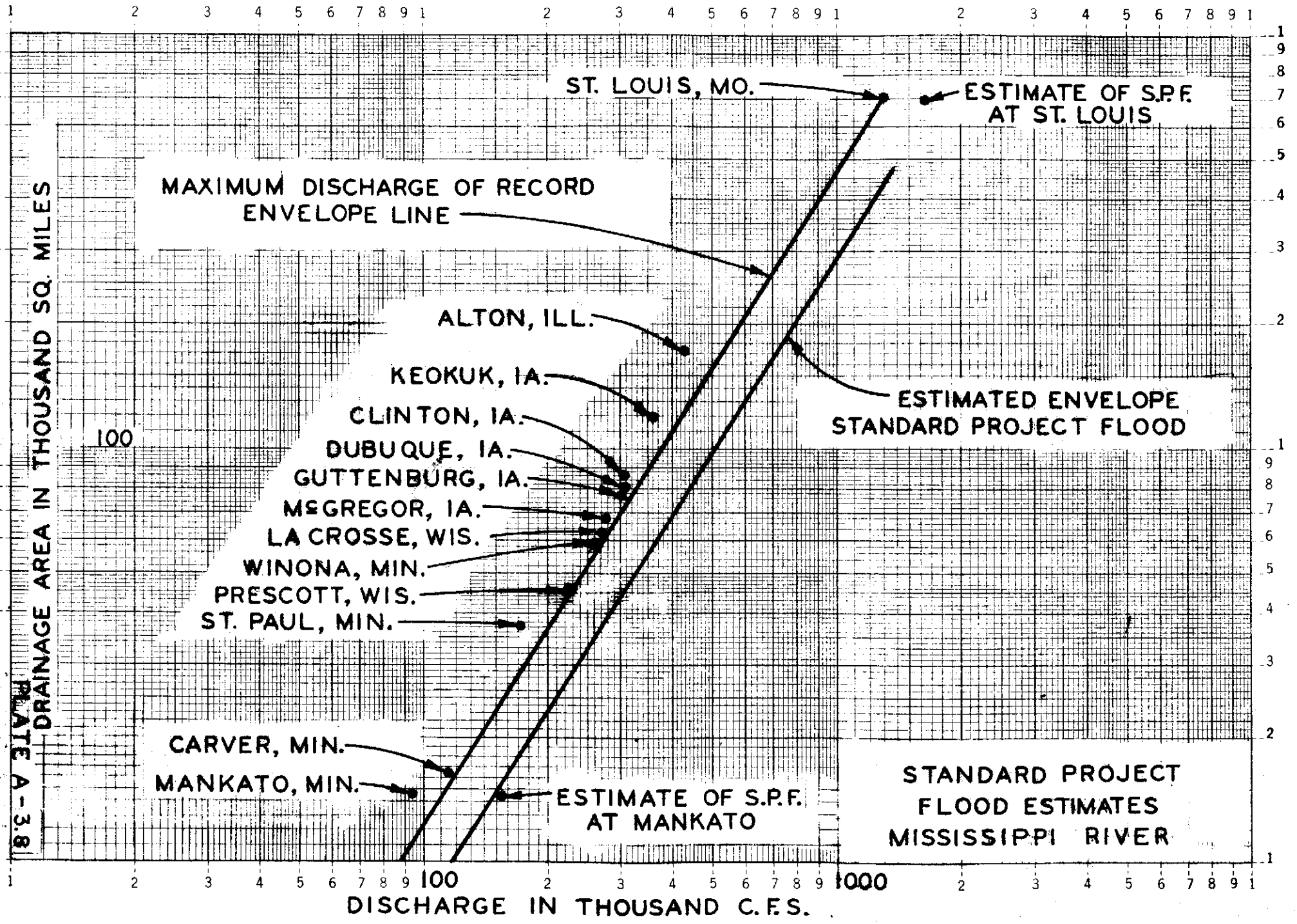


MISSISSIPPI RIVER

DAVENPORT, IOWA

WATER SURFACE PROFILES

PLATE A-27


ST. LOUIS, MO.
ESTIMATE OF S.P.F. AT ST. LOUIS
MAXIMUM DISCHARGE OF RECORD ENVELOPE LINE
ALTON, ILL.
KEOKUK, IA.
CLINTON, IA.
DUBUQUE, IA.
GUTTENBURG, IA.
McGREGOR, IA.
LA CROSSE, WIS.
WINONA, MIN.
PRESCOTT, WIS.
ST. PAUL, MIN.
ESTIMATED ENVELOPE STANDARD PROJECT FLOOD
CARVER, MIN.
MANKATO, MIN.
ESTIMATE OF S.P.F. AT MANKATO
STANDARD PROJECT FLOOD ESTIMATES MISSISSIPPI RIVER
DRAINAGE AREA IN THOUSAND SQ. MILES
100
1000
DISCHARGE IN THOUSAND C.F.S.
PLATE A-3.8

INTERIOR FLOOD RELIEF	
PUMP STATION	DRAINAGE AREA(S)
MAIN STREET	1
GAINES STREET	2
CREDIT ISLAND SLOUGH	4,4A,5

LEGEND:

- DRAINAGE AREA BOUNDARY
- SUB-AREA BOUNDARY
- AREAS SERVED BY EXISTING PRESSURE SEWERS

DAVENPORT

ROCKINGHAM

NAHANT MARSH DRAINAGE

CREDIT ISLAND SLOUGH

CREDIT ISLAND PARK

3A 3B 3C 3D 4 4A

1 2

MISSISSIPPI RIVER
DAVENPORT, IOWA
LOCAL FLOOD PROTECTION

DRAINAGE AREAS

1000 0 2000 4000 6000
SCALE IN FEET

PLATE A-4.1

SUMMARY

UNIT HYDROGRAPHS

INPUT FOR CLARK'S METHOD

TIME OF CONCENTRATION

Sub-Area	Drainage Area (Acres)	Time Incr. (Minutes)	Overland Travel Time (Minutes)	Inlet Time (Minutes)	Tc Comp (Minutes)	Tc Used (Minutes)	Factors Influencing Tc	Clark's "R" Used Minutes	Factors Influencing "R"
1	108	5	12	5	17	30			
2	360	5	15±	5	19.5	30	Reflect overland flow in low lying areas to the Gaines P.S.	10	Use R = 1/3 Tc
3A	682	5	33	10	43	30	Upland area drained by existing pressure sewer	10	
3B	192	5	17	15	32	30	Upland area drained by existing pressure sewer	15	Narrow upper basin
3C	175	5	22	10	32	30	Upland area drained by existing pressure sewer	10	
3D	181	5	13	5	18	30	Upland area drained by existing pressure sewer	15	
4	960	5				90	Average travel time = 3.1 f.p.s.	75	Basin is hour-glass shaped
5 Excluding I-280 Lakes	3,712	15				120	Average travel time = 2.87 f.p.s.	90	Reflect local experience
I-280 Lakes	1,892	-					Regulated flow: added manually to Area 5		
Garden Add.	156	5				30		20	Urban basin of low relief

PLATE A-42

NO. 340-L310 DIETZGEN GRAPH PAPER
SEMI-LOGARITHMIC
3 CYCLES X 10 DIVISIONS PER INCH

DIETZGEN CORPORATION
MADE IN U.S.A.

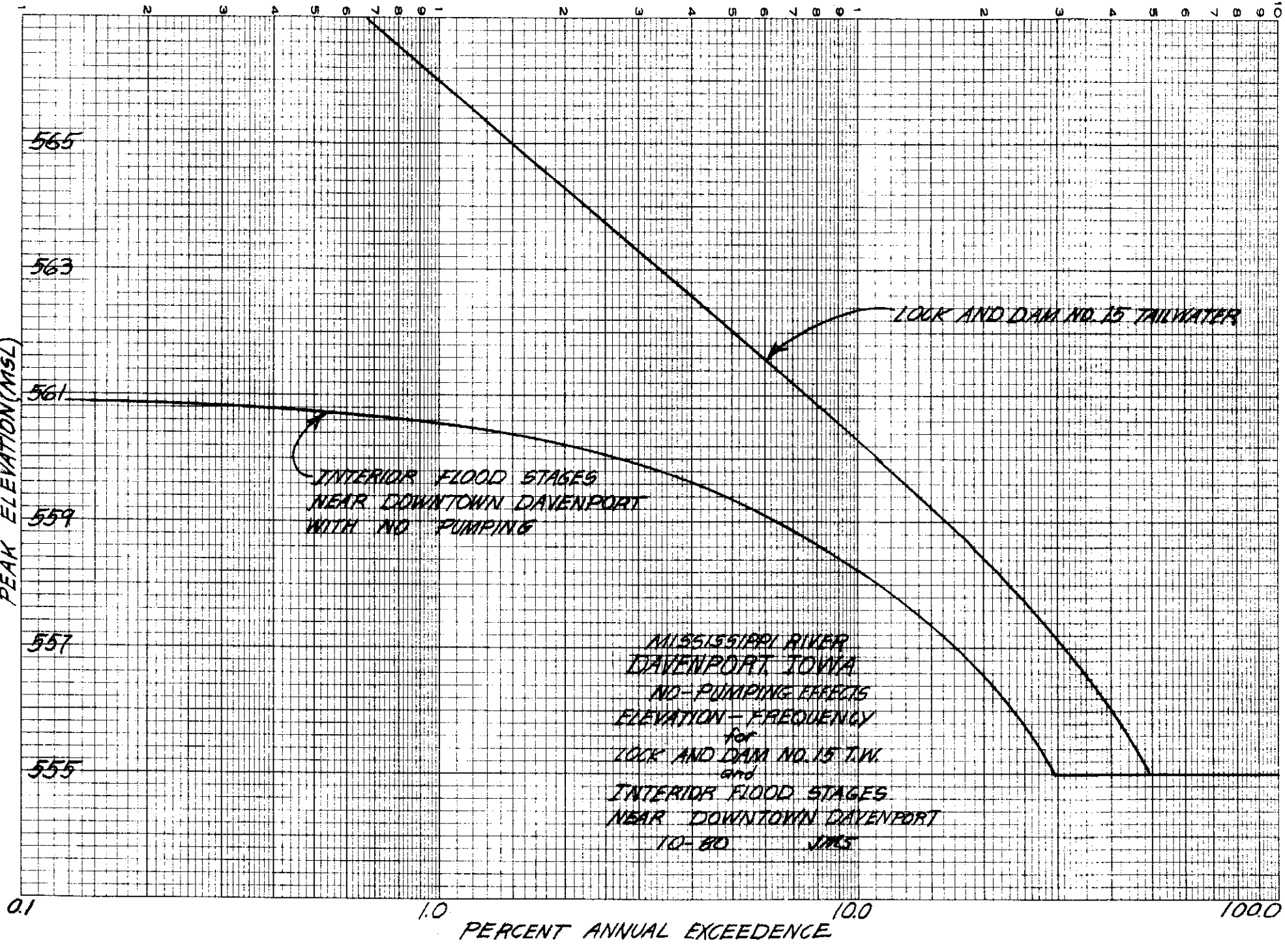


PLATE A-51

NO. 340-L310 DIETZGEN GRAPH PAPER
SEMI-LOGARITHMIC
3 CYCLES X 10 DIVISIONS PER INCH

DIETZGEN CORPORATION
MADE IN U.S.A.

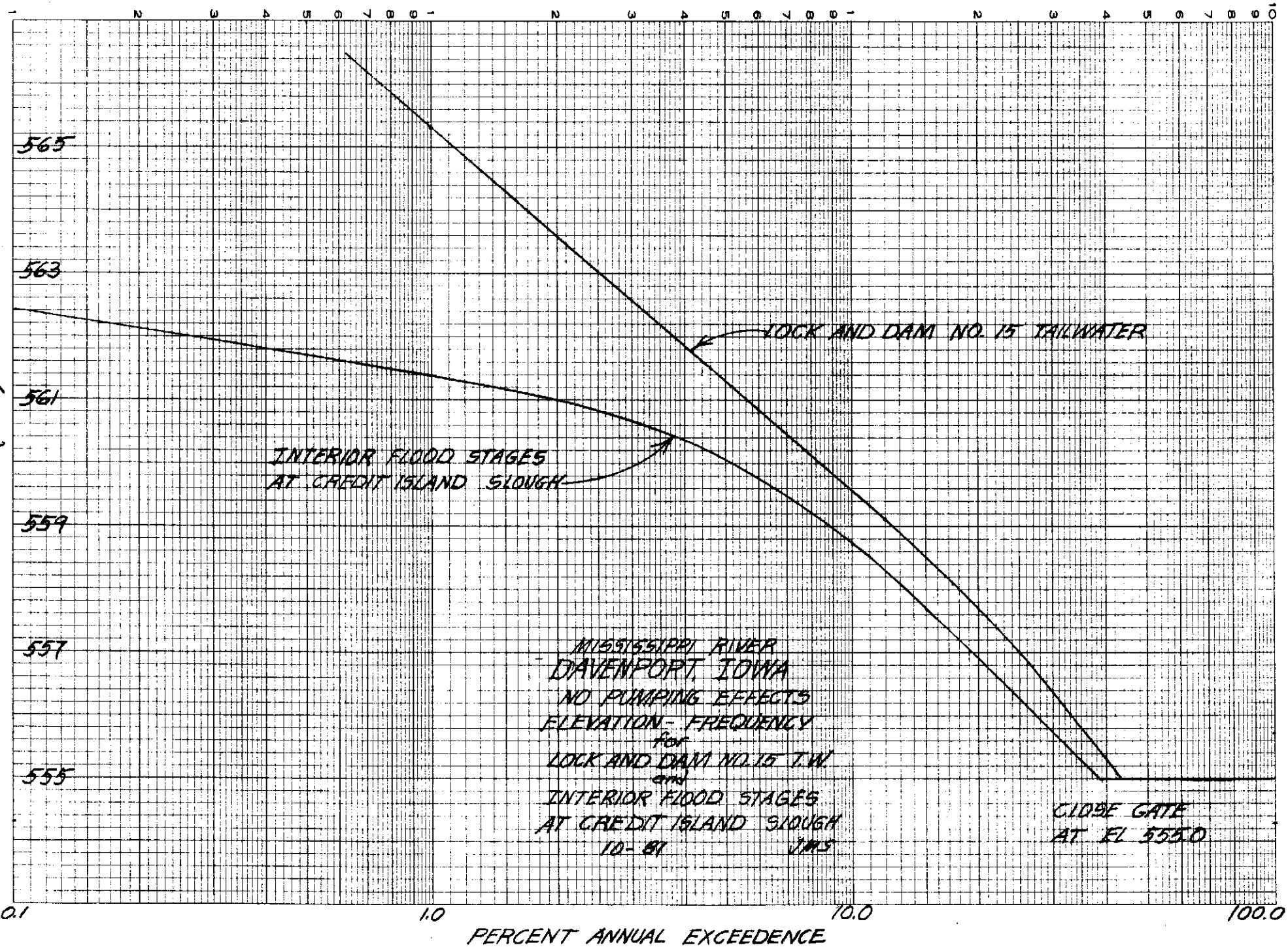


PLATE A-52

NO. 340-L310 DIETZGEN GRAPH PAPER
SEMI-LOGARITHMIC
3 CYCLES X 10 DIVISIONS PER INCH

DIETZGEN CORPORATION
MADE IN U.S.A.

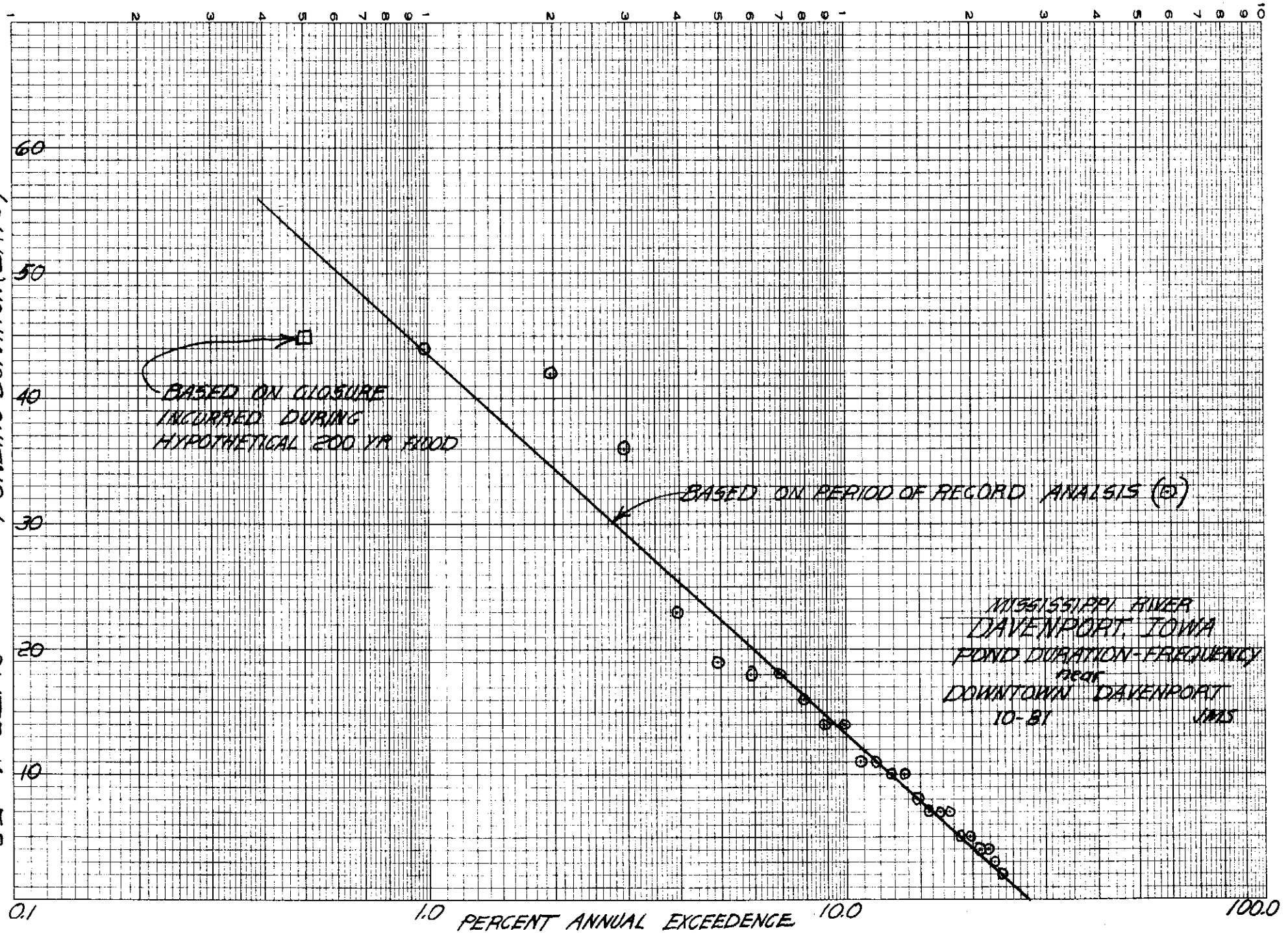


PLATE A-5.3

NO. 340-L310 DIETZGEN GRAPH PAPER
SEMI-LOGARITHMIC
3 CYCLES X 10 DIVISIONS PER INCH

DIETZGEN CORPORATION
MADE IN U.S.A.

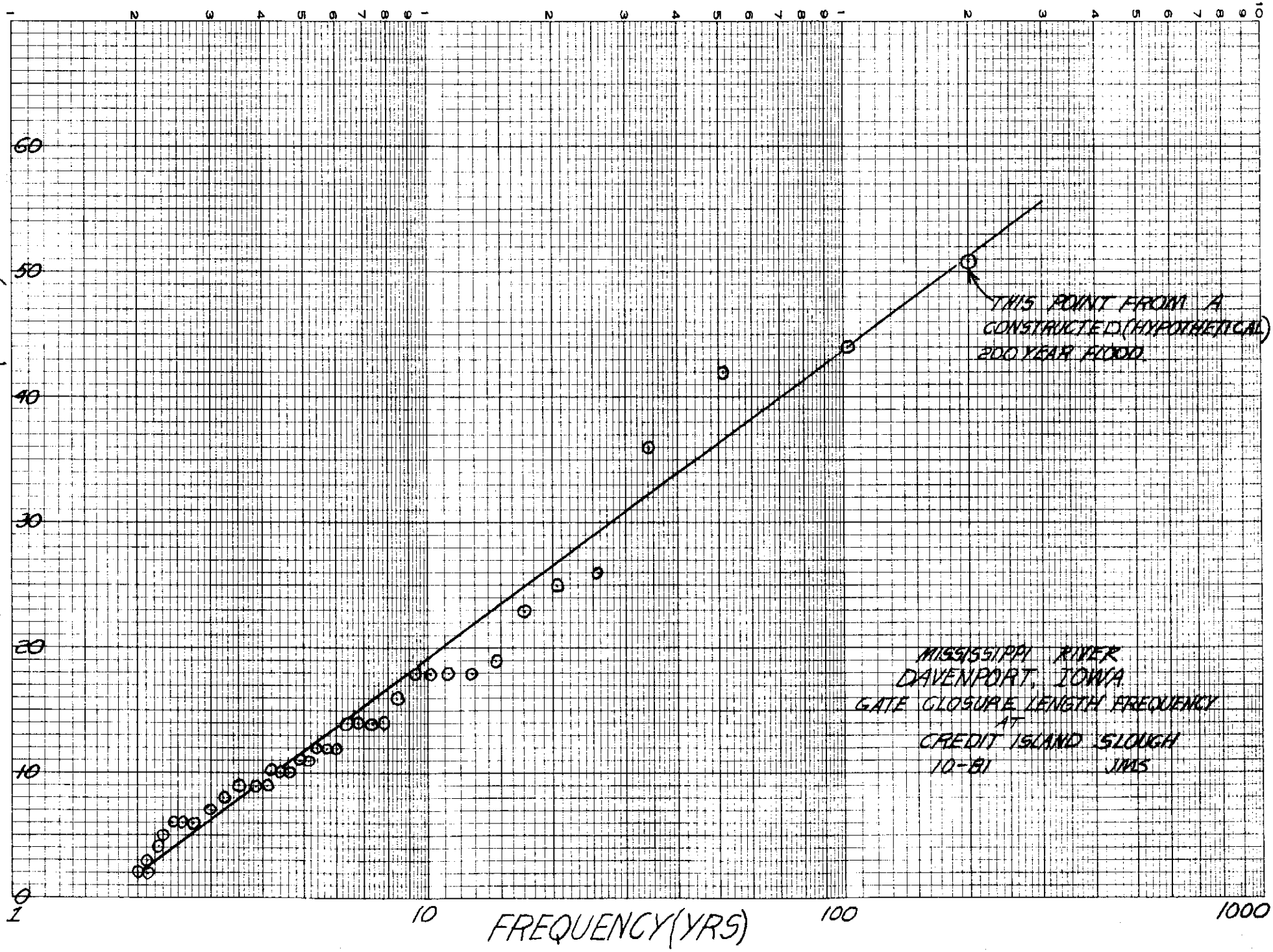


PLATE A-54

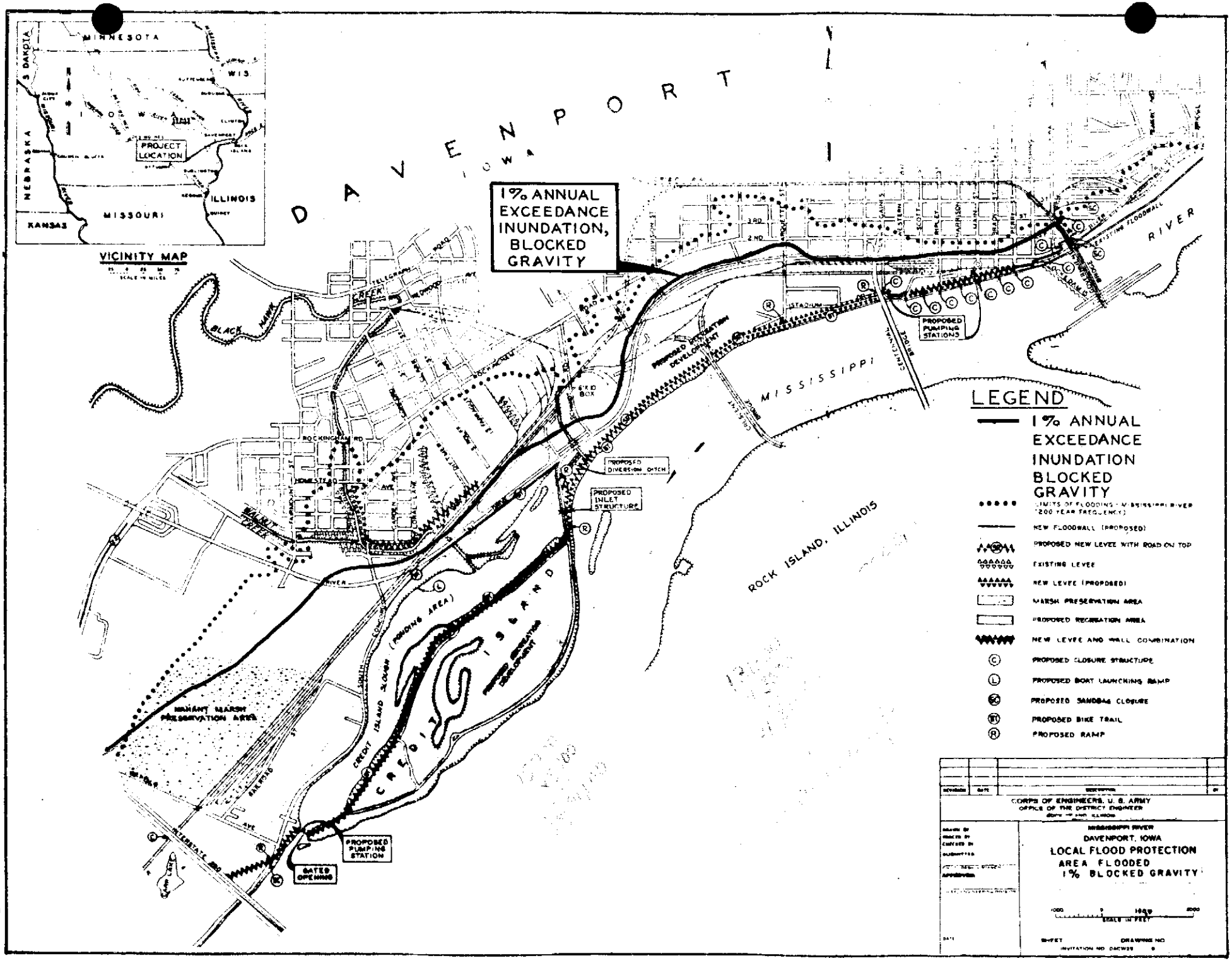

MINNESOTA
S DAKOTA
WIS
NEBRASKA
PROJECT LOCATION
ILLINOIS
MISSOURI
KANSAS
VICINITY MAP
DAVENPORT
IOWA
1% ANNUAL EXCEEDANCE INUNDATION, BLOCKED GRAVITY
BLACK HAWK CREEK
MISSISSIPPI RIVER
ROCK ISLAND, ILLINOIS
PROPOSED PUMPING STATIONS
PROPOSED DIVERSION DITCH
PROPOSED INLET STRUCTURE
PROPOSED PUMPING STATION
GATED OPENINGS
CREDIT ISLAND SLOUGH (PONDING AREA)
MARSH PRESERVATION AREA
LEGEND
1% ANNUAL EXCEEDANCE INUNDATION BLOCKED GRAVITY
LIMITS OF FLOODING - MISSISSIPPI RIVER (200 YEAR FREQUENCY)
NEW FLOODWALL (PROPOSED)
PROPOSED NEW LEVEE WITH ROAD ON TOP
EXISTING LEVEE
NEW LEVEE (PROPOSED)
MARSH PRESERVATION AREA
PROPOSED RECREATION AREA
NEW LEVEE AND WALL COMBINATION
PROPOSED CLOSURE STRUCTURE
PROPOSED BOAT LAUNCHING RAMP
PROPOSED SANDBAG CLOSURE
PROPOSED BIKE TRAIL
PROPOSED RAMP
CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
MISSISSIPPI RIVER
DAVENPORT, IOWA
LOCAL FLOOD PROTECTION
AREA FLOODED
1% BLOCKED GRAVITY
SCALE IN FEET

PLATE A-55

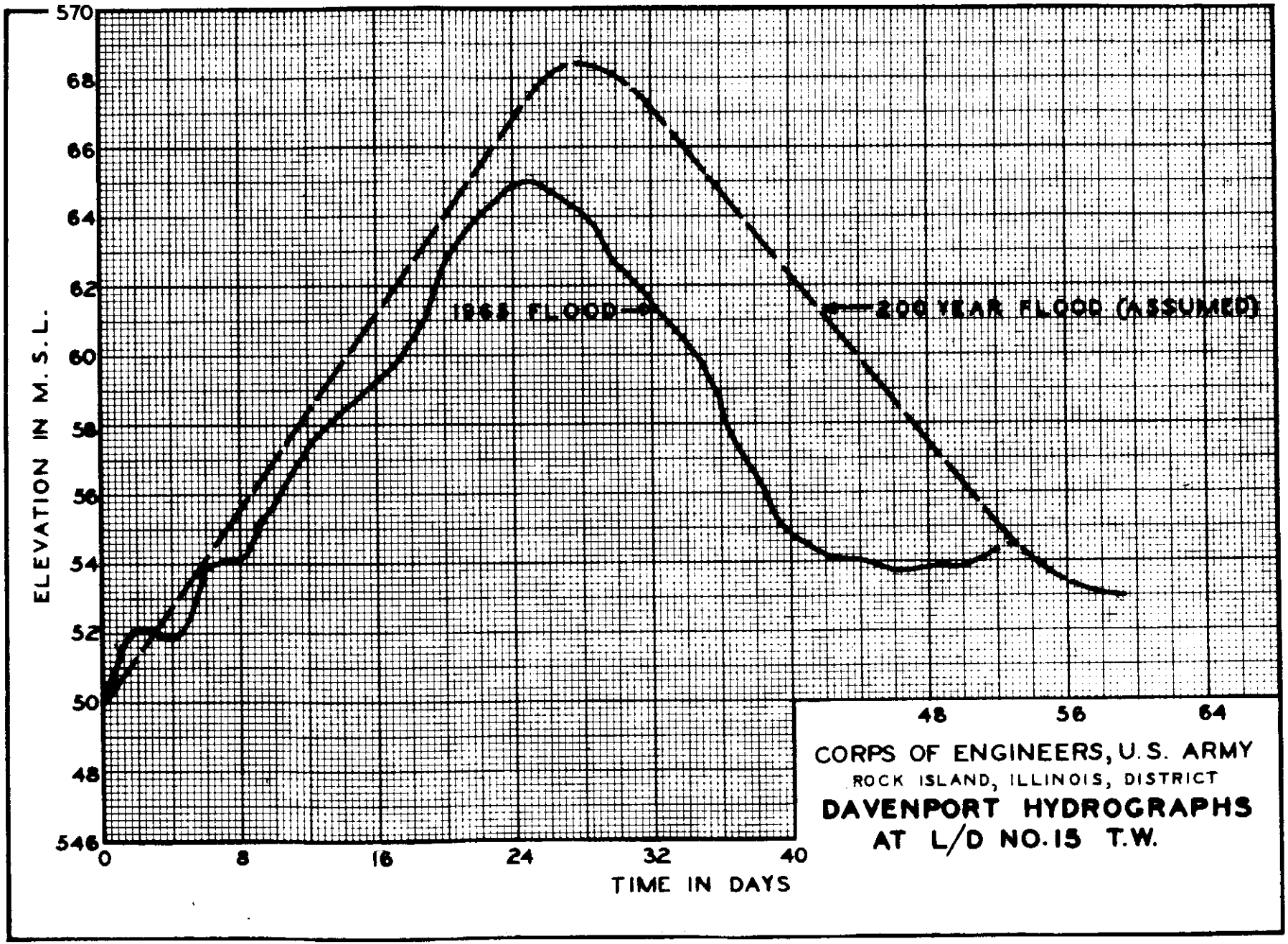

570
68
66
64
62
60
58
56
54
52
50
48
546
ELEVATION IN M.S.L.
1965 FLOOD
200 YEAR FLOOD (ASSUMED)
0
8
16
24
32
40
48
56
64
TIME IN DAYS
CORPS OF ENGINEERS, U.S. ARMY
ROCK ISLAND, ILLINOIS, DISTRICT
DAVENPORT HYDROGRAPHS
AT L/D NO. 15 T.W.

PLATE A-5.6

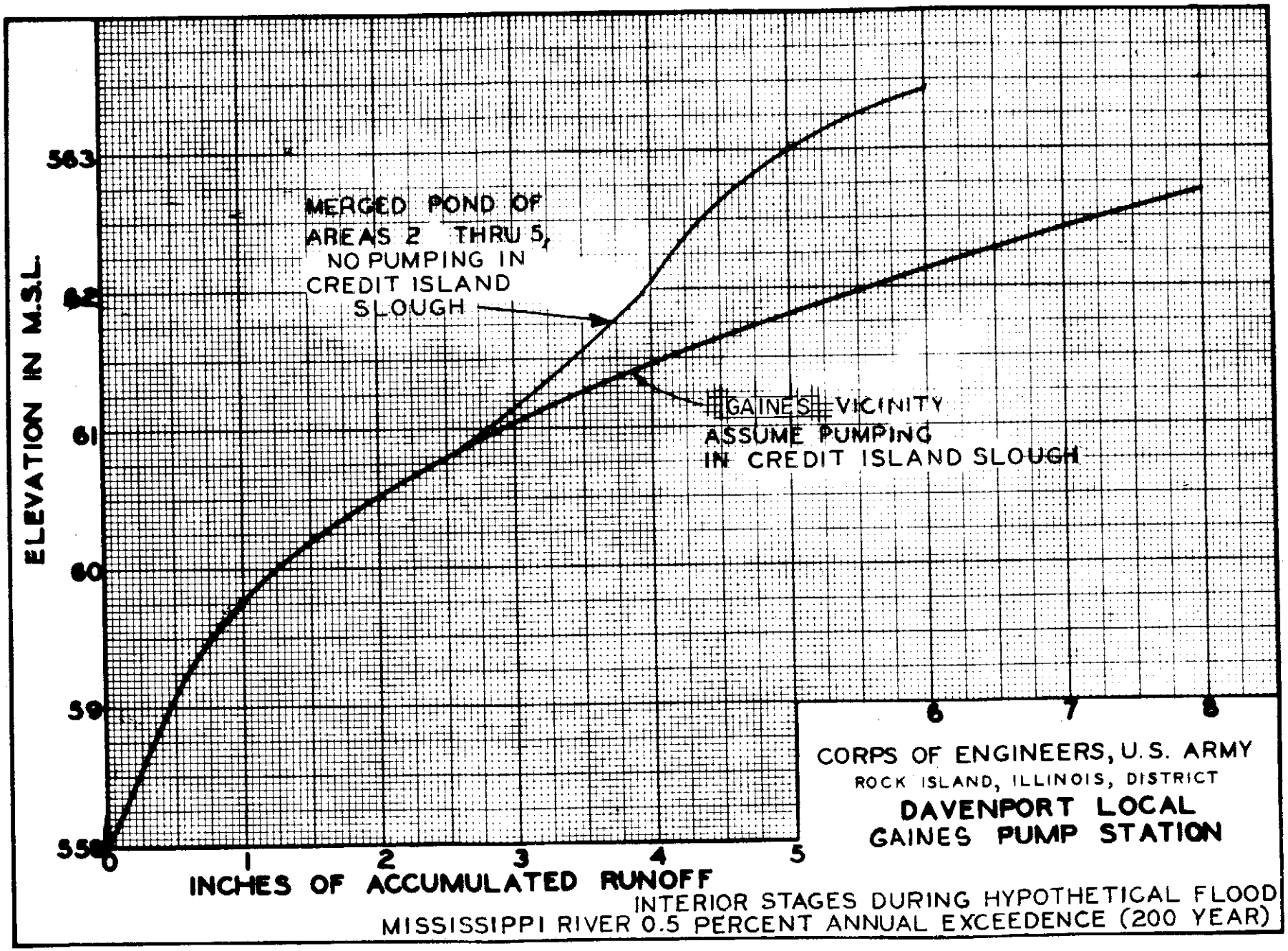

ELEVATION IN M.S.L.
63
62
61
60
59
55
0
1
2
3
4
5
6
7
8
INCHES OF ACCUMULATED RUNOFF
MERGED POND OF AREAS 2 THRU 5, NO PUMPING IN CREDIT ISLAND SLOUGH
GAINES VICINITY ASSUME PUMPING IN CREDIT ISLAND SLOUGH
CORPS OF ENGINEERS, U.S. ARMY
ROCK ISLAND, ILLINOIS, DISTRICT
DAVENPORT LOCAL
GAINES PUMP STATION
INTERIOR STAGES DURING HYPOTHETICAL FLOOD
MISSISSIPPI RIVER 0.5 PERCENT ANNUAL EXCEEDENCE (200 YEAR)

PLATE A-5.7

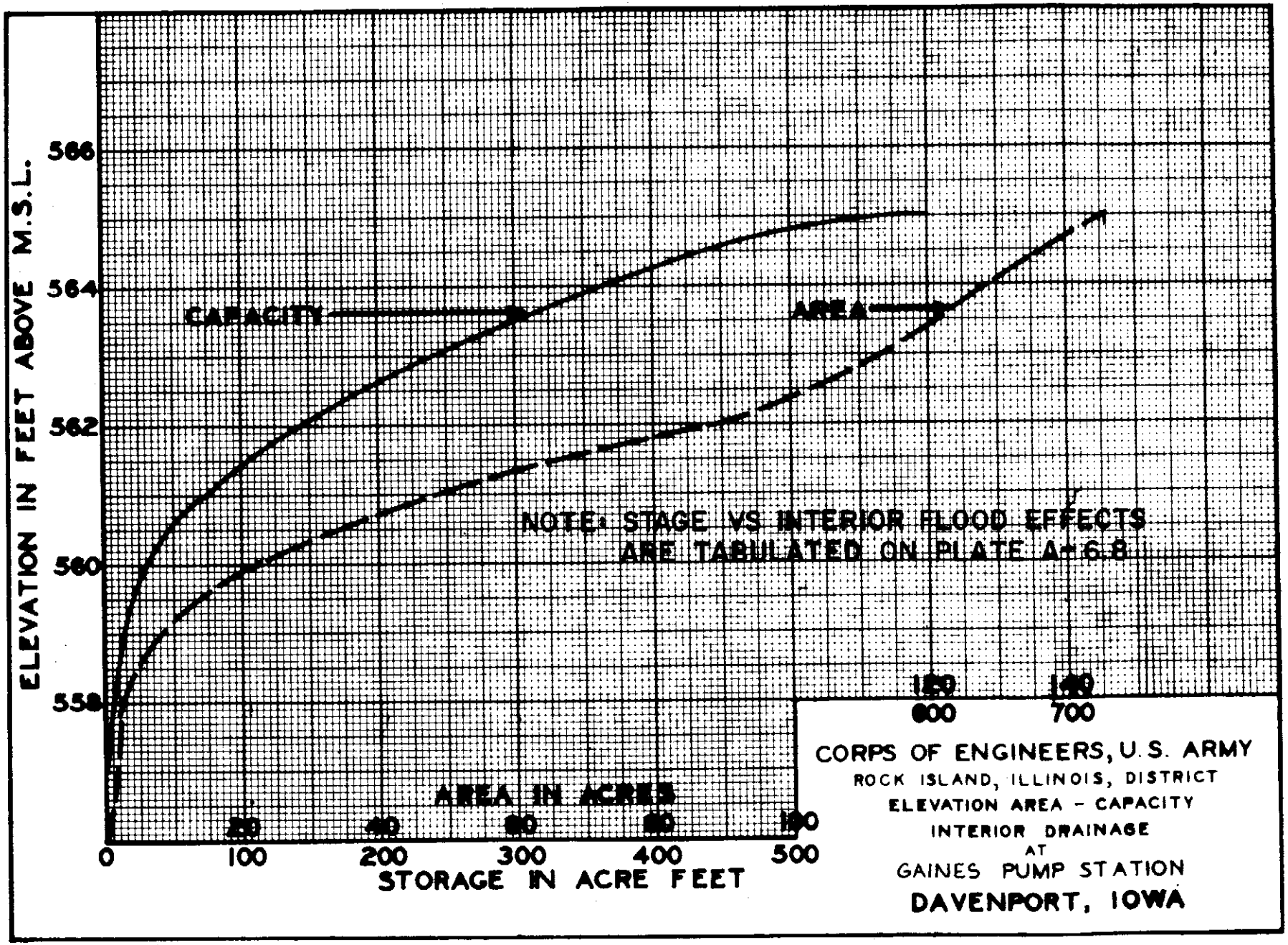

ELEVATION IN FEET ABOVE M.S.L.
566
564
562
560
558
CAPACITY
AREA
NOTE: STAGE VS INTERIOR FLOOD EFFECTS ARE TABULATED ON PLATE A-6.8
AREA IN ACRES
0
100
200
300
400
500
STORAGE IN ACRE FEET
CORPS OF ENGINEERS, U.S. ARMY
ROCK ISLAND, ILLINOIS, DISTRICT
ELEVATION AREA - CAPACITY
INTERIOR DRAINAGE
AT
GAINES PUMP STATION
DAVENPORT, IOWA

PLATE A-6.1

COMBINED RUNOFF (CFS) HYDROGRAPHS
GAINES PUMP STATION

(INCLUDES EFFECT OF PRESSURE SEWERS)

Time in 15 Min. Periods	FREQUENCY IN YRS. 1	2	5	10	25	50	100
0	1	1	0	1	1	1	1
1	6	6	0	6	12	12	18
2	11	11	1	13	23	23	35
3	15	15	6	20	29	27	47
4	20	19	11	26	34	31	55
5	26	20	15	28	36	38	58
6	28	17	20	29	38	46	60
7	30	17	26	31	41	50	65
8	34	21	28	35	43	53	71
9	36	26	30	42	45	60	78
10	38	30	34	49	50	68	84
11	41	35	35	55	55	72	87
12	46	41	32	60	60	76	90
13	56	44	30	70	71	85	111
14	67	50	32	81	81	96	116
15	73	55	41	86	86	102	127
16	111	62	52	93	94	114	165
17	234	76	58	112	118	138	208
18	395	93	71	133	145	163	227
19	458	101	100	143	158	179	307
20	409	144	129	201	230	256	656
21	86	281	143	404	492	561	1107
22	40	451	197	633	878	984	1337
23	19	568	367	840	1071	1201	1296
24	9	553	629	871	1043	1171	840
25	4	328	728	627	677	755	274
26	2	124	739	269	207	236	74
27	1	37	454	53	62	68	17
28	0	8	155	12	14	15	3
29		2	46	2	3	3	3
30		2	11	2	3	3	3
31		1	2	2	3	3	2
32		1	2	2	3	2	1
33		0	2	1	2	1	0
34			1	0	2	0	
35			0		1		
36					0		

Plate A-6.2

NO. 340-20 DIETZGEN GRAPH PAPER
20 X 20 PER INCH
DIETZGEN CORPORATION
MADE IN U.S.A.

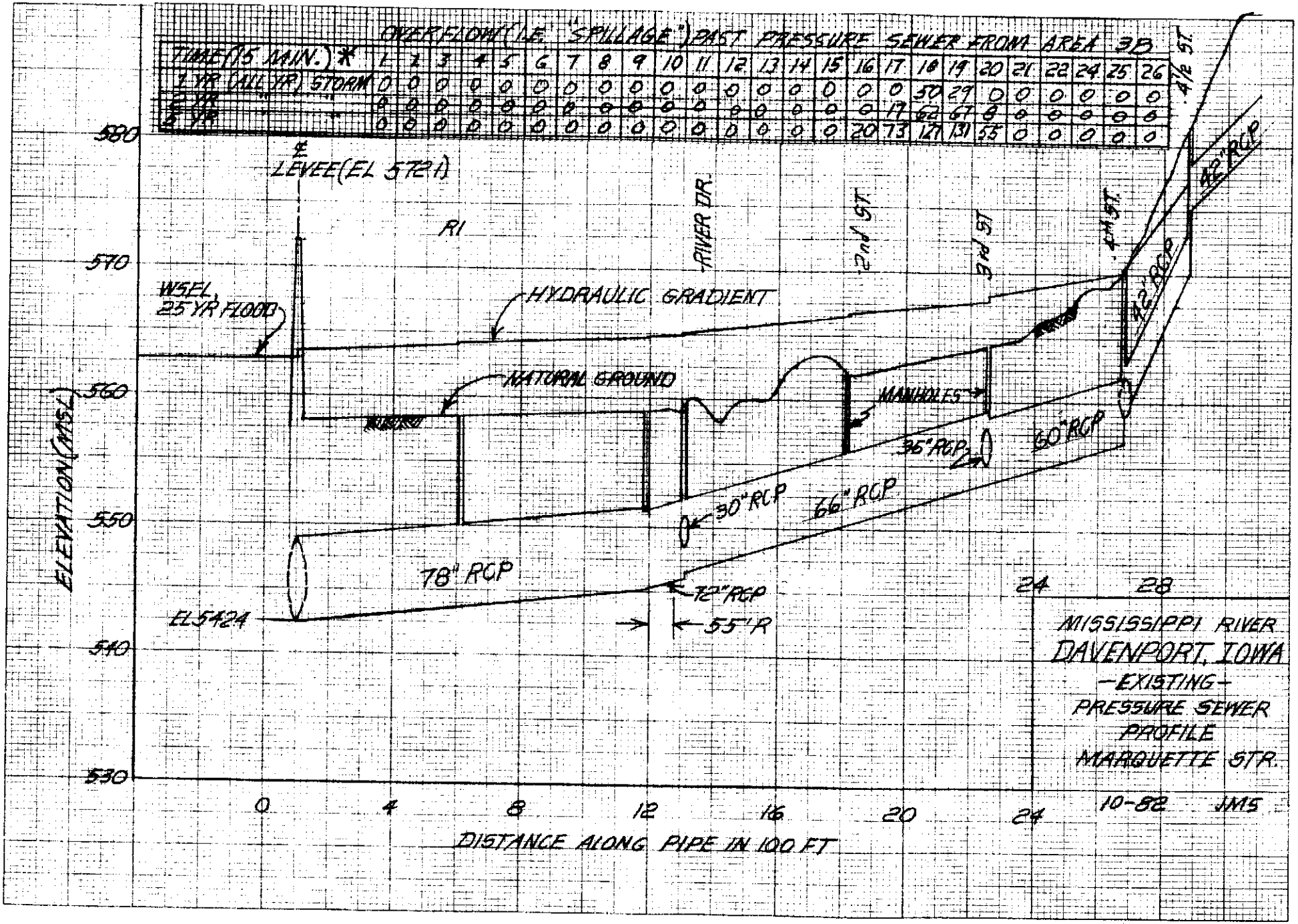


PLATE A-6.21

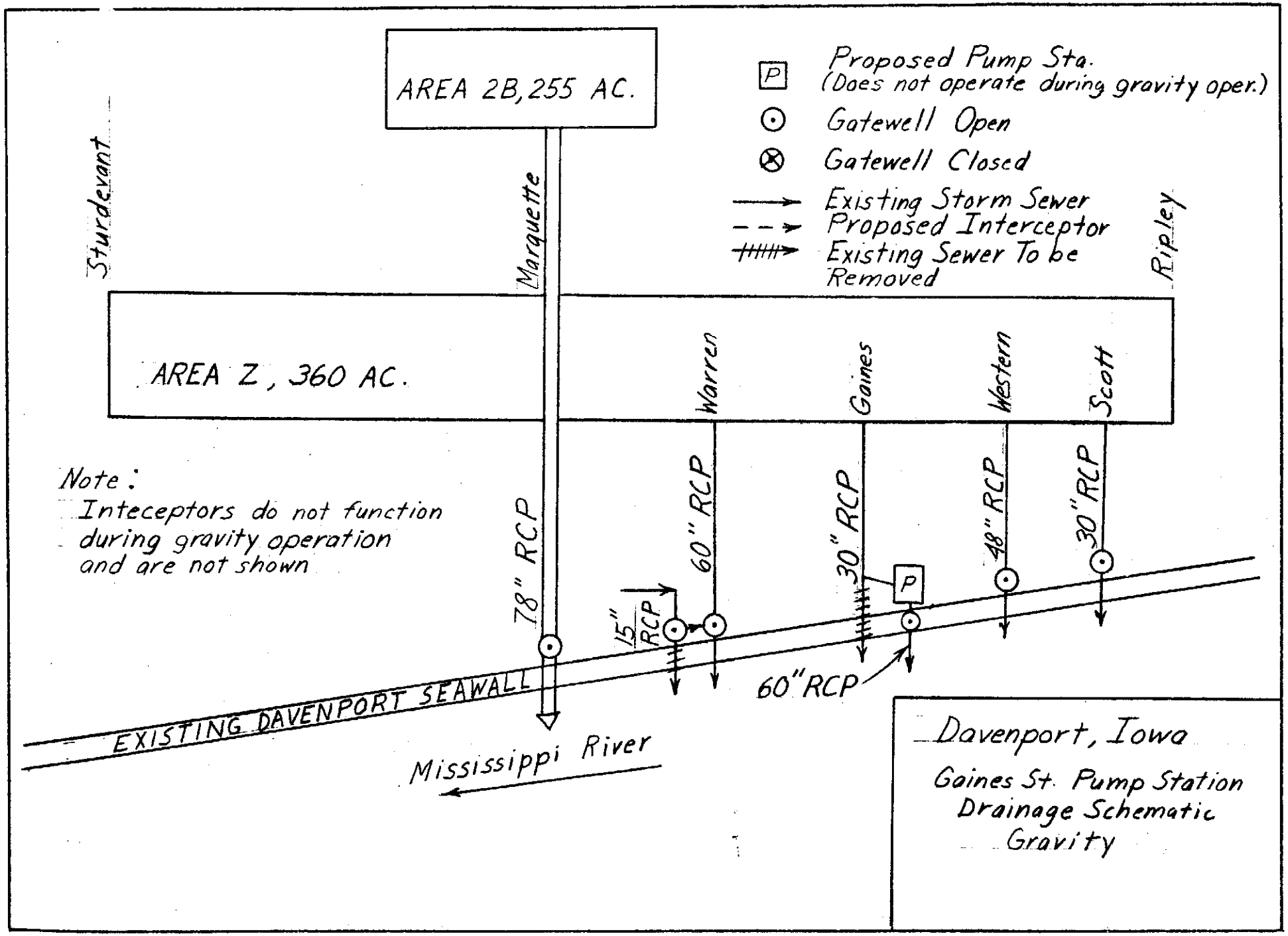

AREA 2B, 255 AC.
AREA Z, 360 AC.
Sturdevant
Marquette
Warren
Gaines
Western
Scott
Ripley
P Proposed Pump Sta. (Does not operate during gravity oper.)
Gatewell Open
Gatewell Closed
Existing Storm Sewer
Proposed Interceptor
Existing Sewer To be Removed
Note: Inteceptors do not function during gravity operation and are not shown
78" RCP
15" RCP
60" RCP
30" RCP
48" RCP
30" RCP
60" RCP
EXISTING DAVENPORT SEAWALL
Mississippi River
Davenport, Iowa
Gaines St. Pump Station
Drainage Schematic
Gravity

Plate A-6.3

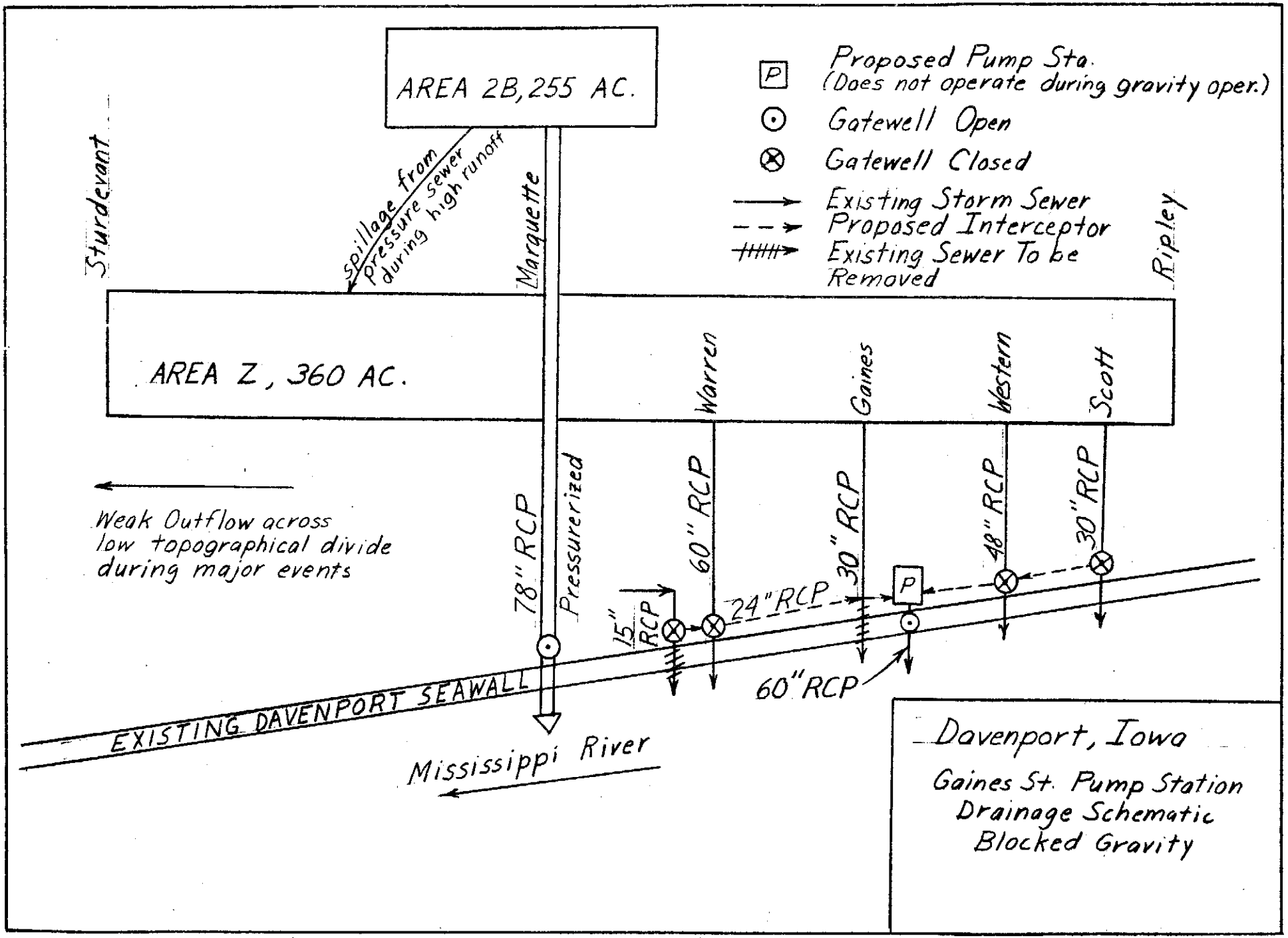

AREA 2B, 255 AC.
AREA Z, 360 AC.
Sturdevant
Marquette
Ripley
Warren
Gaines
Western
Scott
Spillage from pressure sewer during high runoff
Proposed Pump Sta. (Does not operate during gravity oper.)
Gatewell Open
Gatewell Closed
Existing Storm Sewer
Proposed Interceptor
Existing Sewer To be Removed
Weak Outflow across low topographical divide during major events
78" RCP
Pressurized
60" RCP
15" RCP
24" RCP
30" RCP
48" RCP
30" RCP
60" RCP
P
EXISTING DAVENPORT SEAWALL
Mississippi River
Davenport, Iowa
Gaines St. Pump Station
Drainage Schematic
Blocked Gravity

Plate A-6.4

NO. 340-20 DIETZGEN GRAPH PAPER
20 X 20 PER INCH

DIETZGEN CORPORATION
MADE IN U.S.A.

REV 1-82

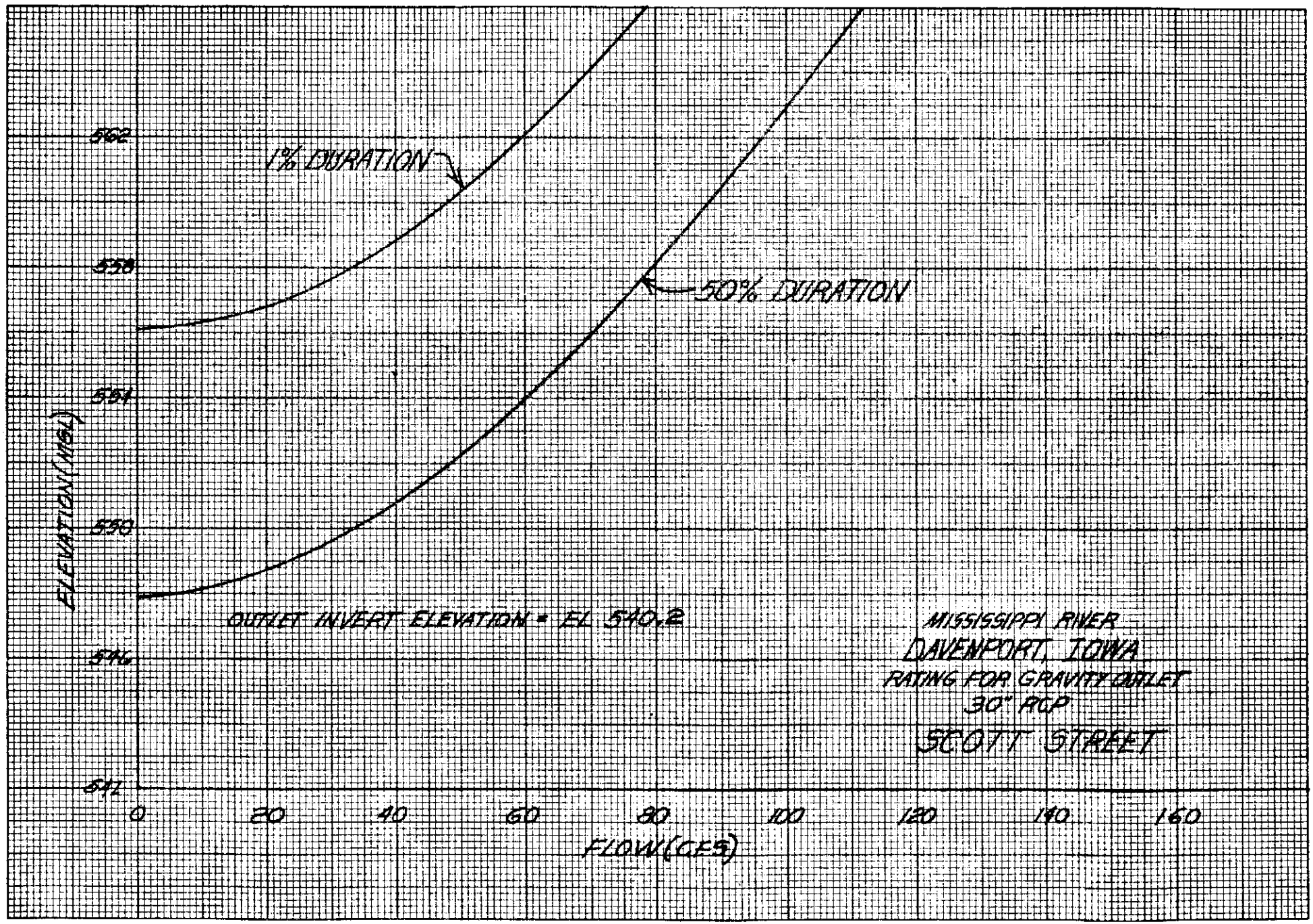


PLATE A-6.51

NO. 340-20 DIETZGEN GRAPH PAPER
20 X 20 PER INCH
DIETZGEN CORPORATION
MADE IN U.S.A.

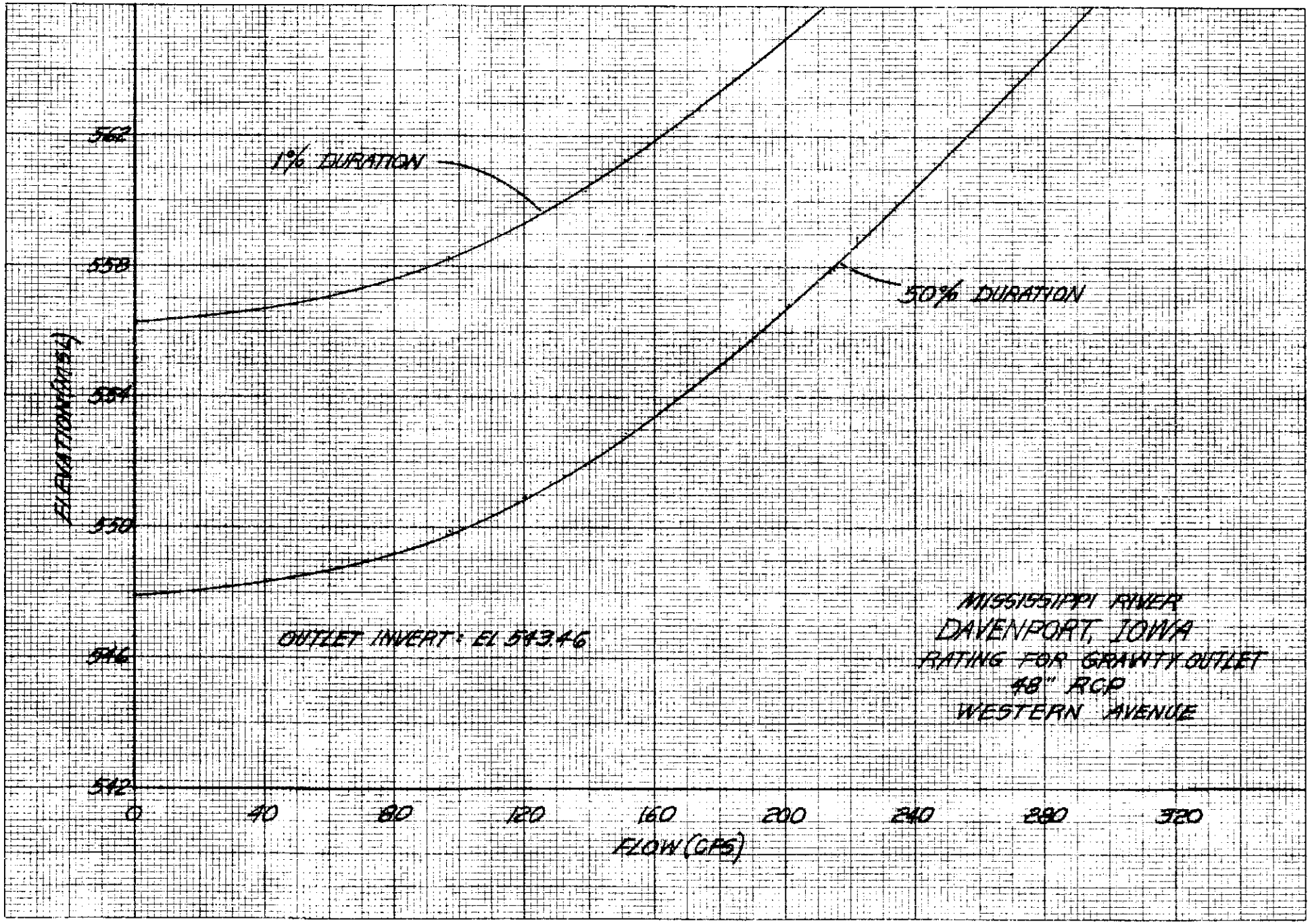


PLATE A-6.52

NO. 340-20 DIETZGEN GRAPH PAPER
20 X 20 PER INCH

DIETZGEN CORPORATION
MADE IN U.S.A.

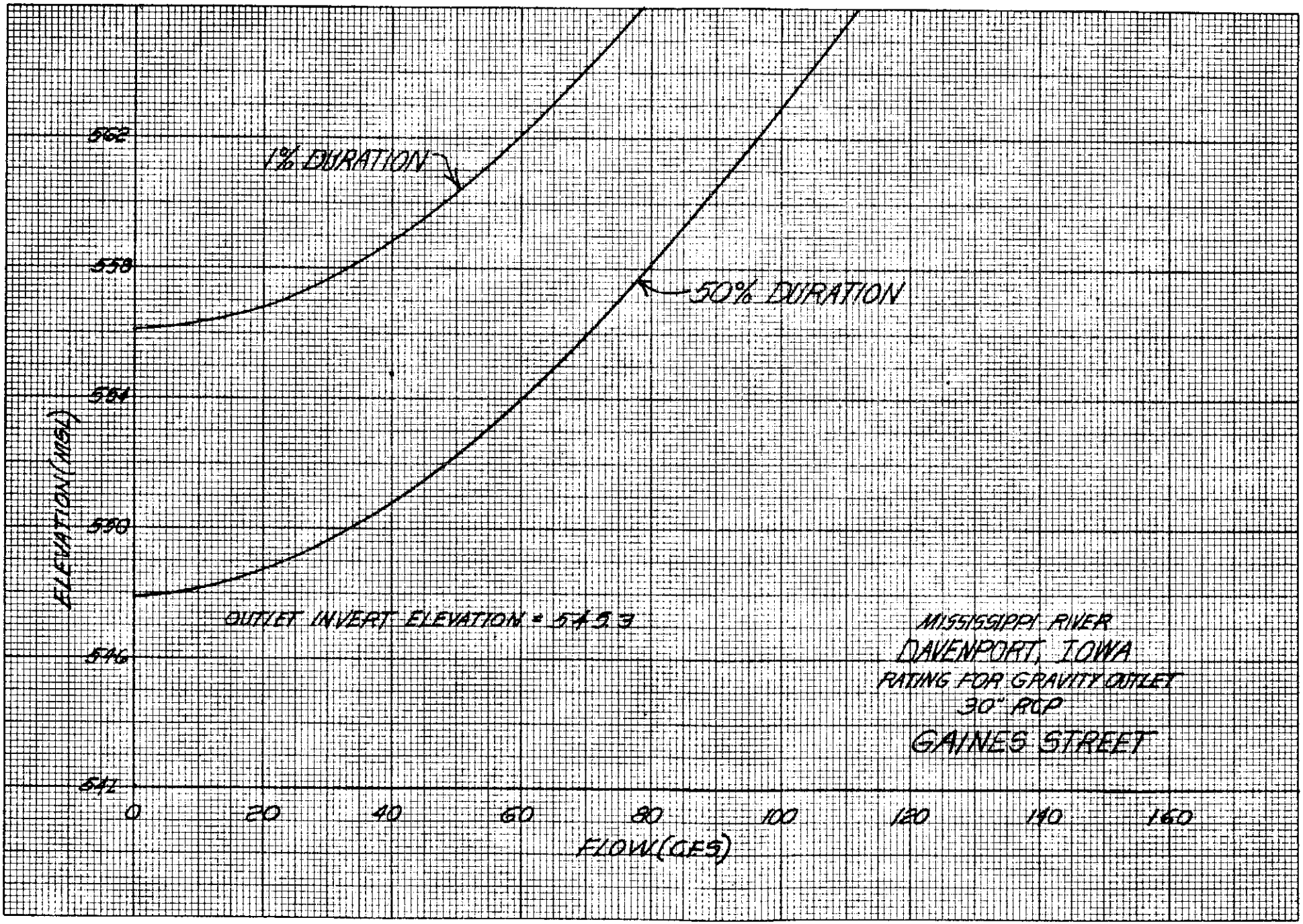


PLATE A-6.5.3

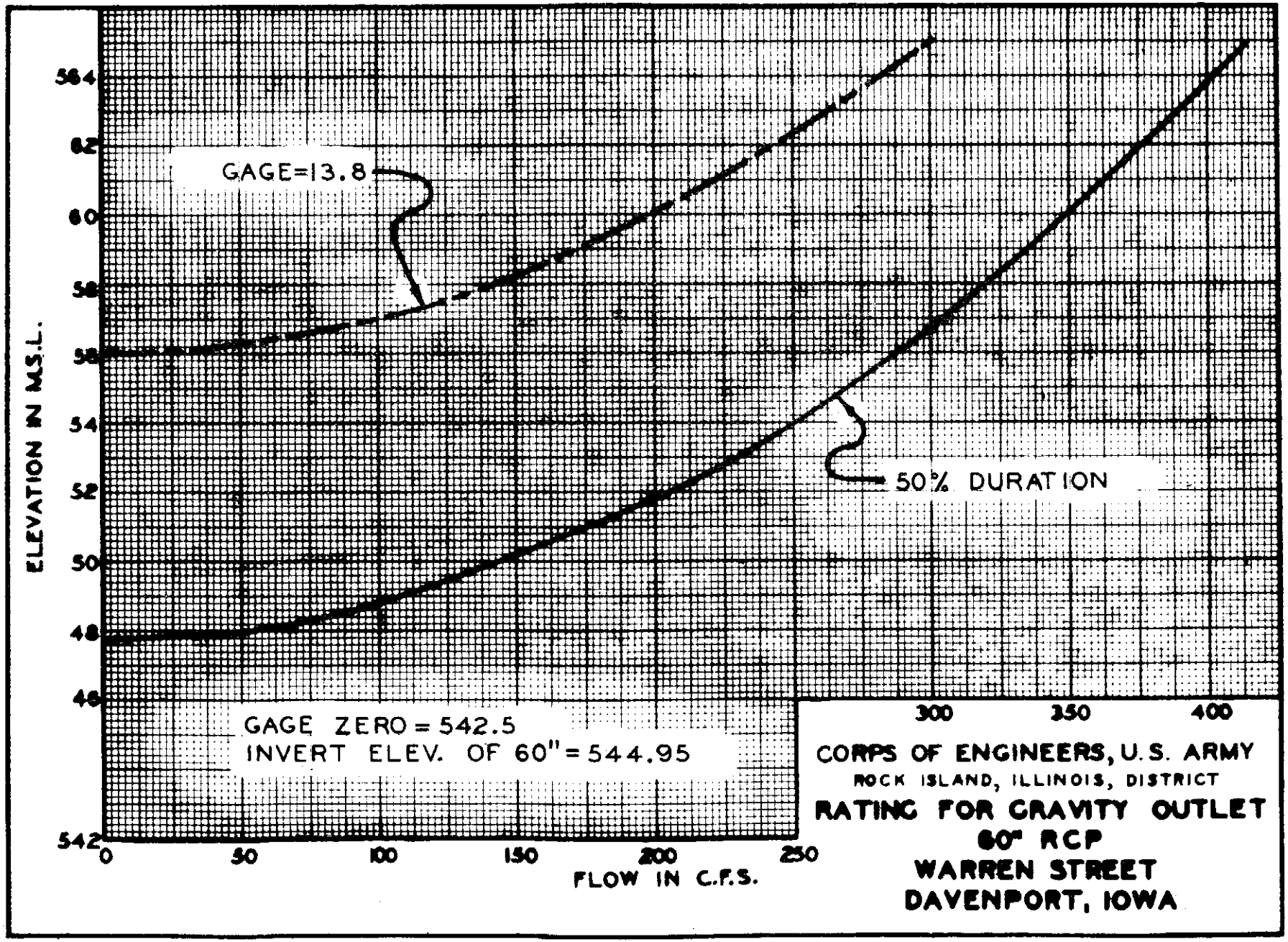

ELEVATION IN M.S.L.
564
62
60
58
56
54
52
50
48
46
542
0
50
100
150
200
250
300
350
400
FLOW IN C.F.S.
GAGE=13.8
50% DURATION
GAGE ZERO = 542.5
INVERT ELEV. OF 60" = 544.95
CORPS OF ENGINEERS, U.S. ARMY
ROCK ISLAND, ILLINOIS, DISTRICT
RATING FOR GRAVITY OUTLET
60" RCP
WARREN STREET
DAVENPORT, IOWA

PLATE A-6.5.4

NO. 340-20 DIETZGEN GRAPH PAPER
20 X 20 PER INCH

DIETZGEN CORPORATION
MADE IN U.S.A.

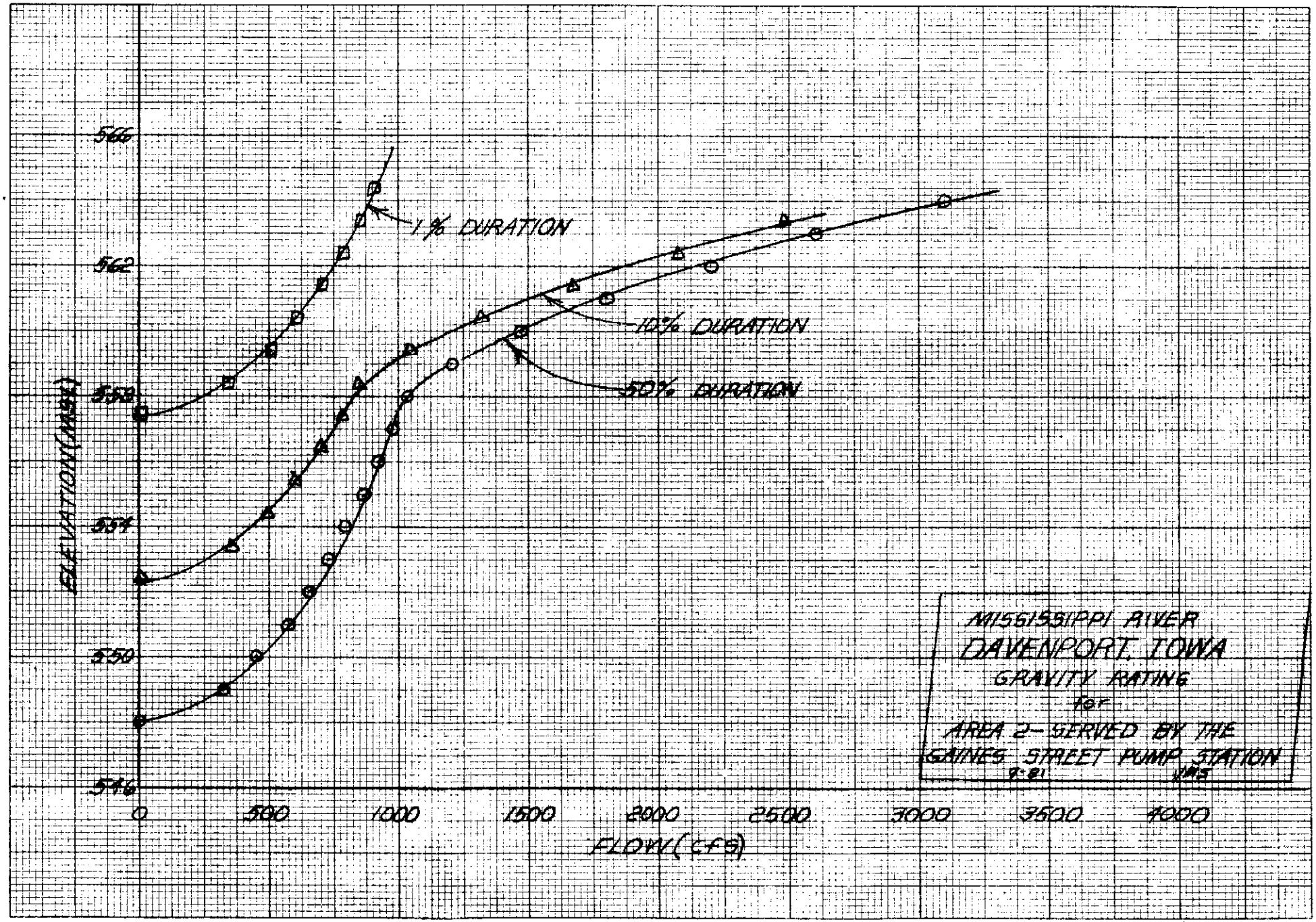


PLATE A-6.5.5

NO. 340-20 DIETZGEN GRAPH PAPER
20 X 20 PER INCH
DIETZGEN CORPORATION
MADE IN U.S.A.

=1506

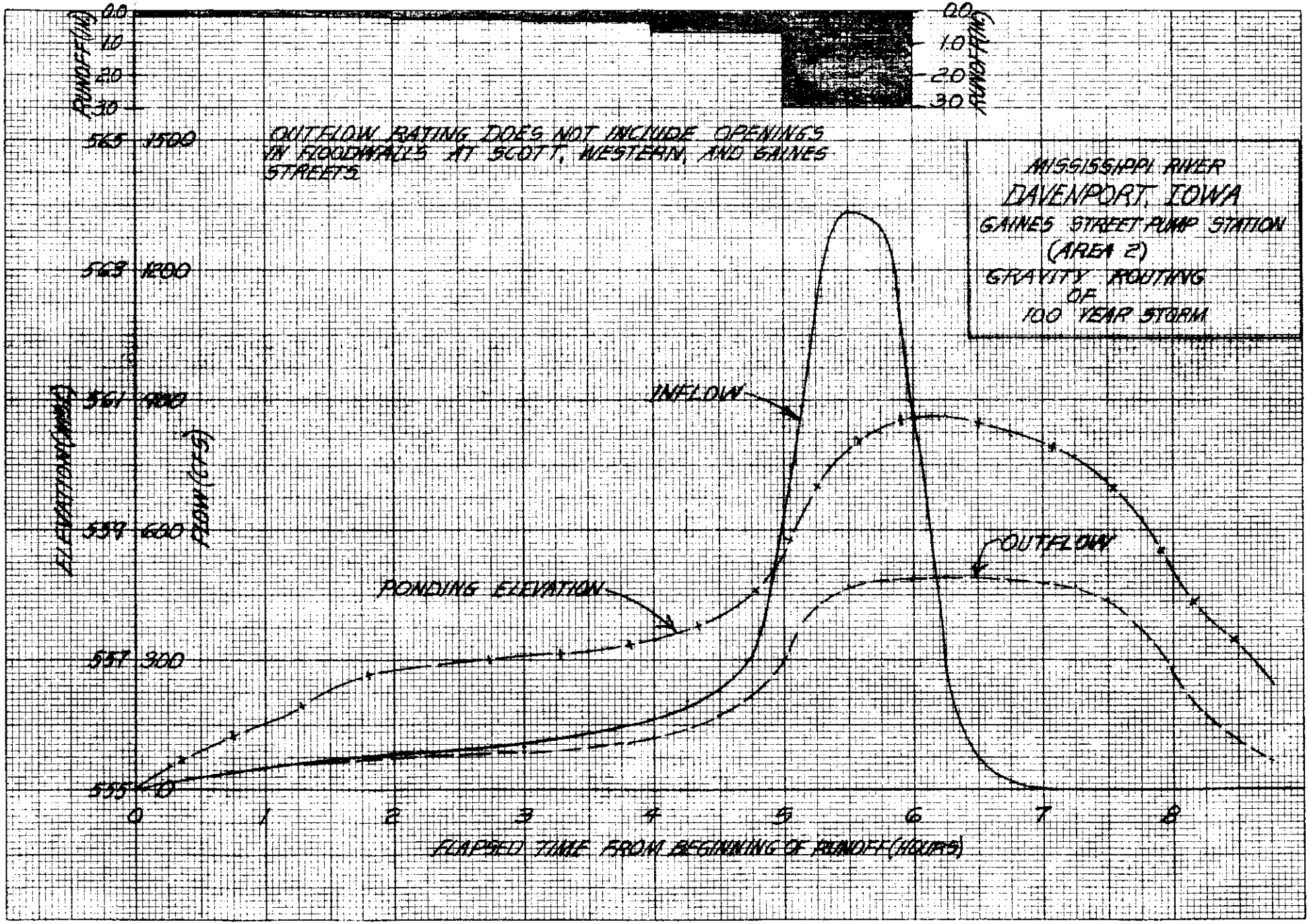


PLATE A-6.5.6.

NO. 340-20 DIETZGEN GRAPH PAPER
20 X 20 PER INCH

DIETZGEN CORPORATION
MADE IN U.S.A.

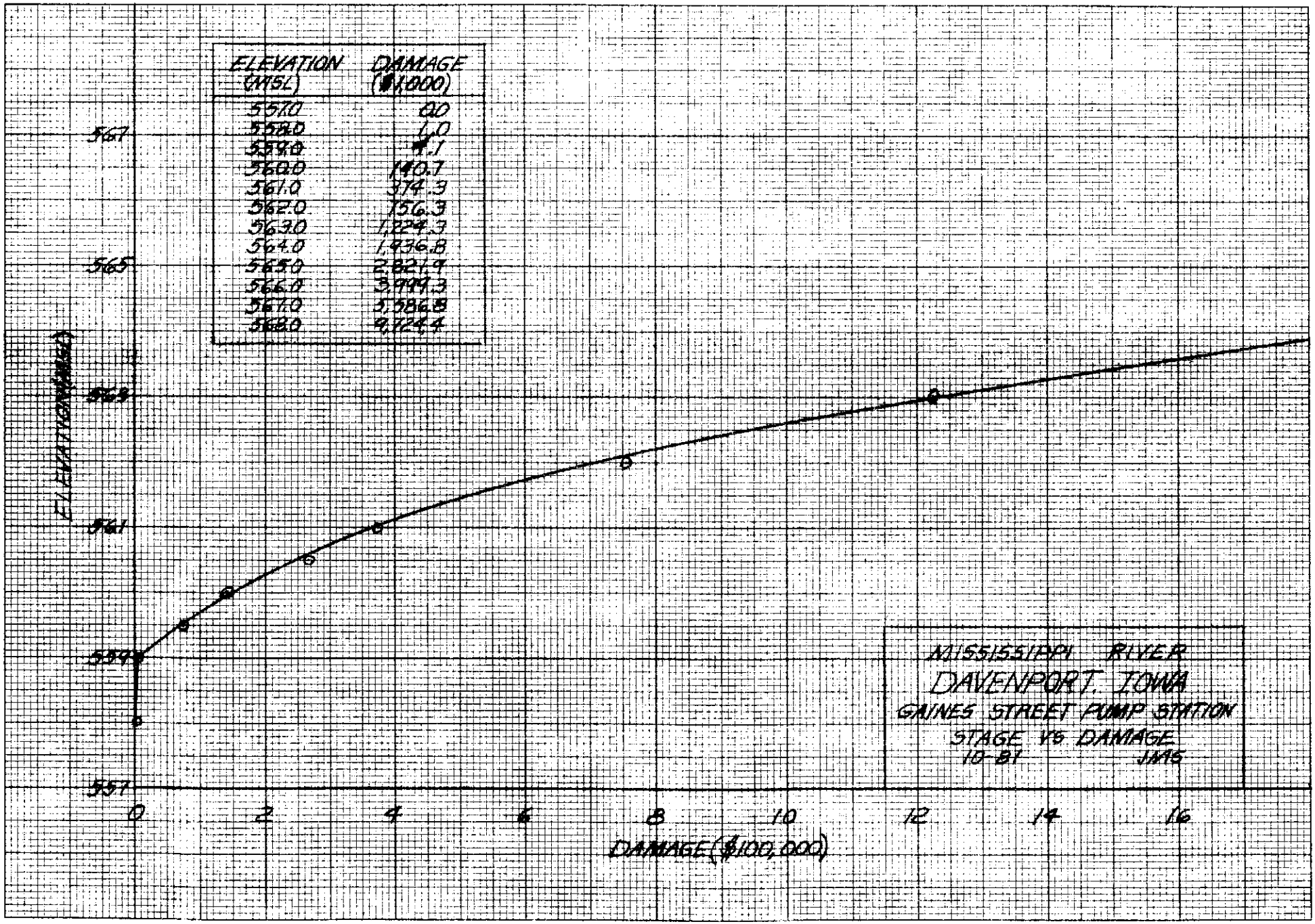


PLATE A-6.6

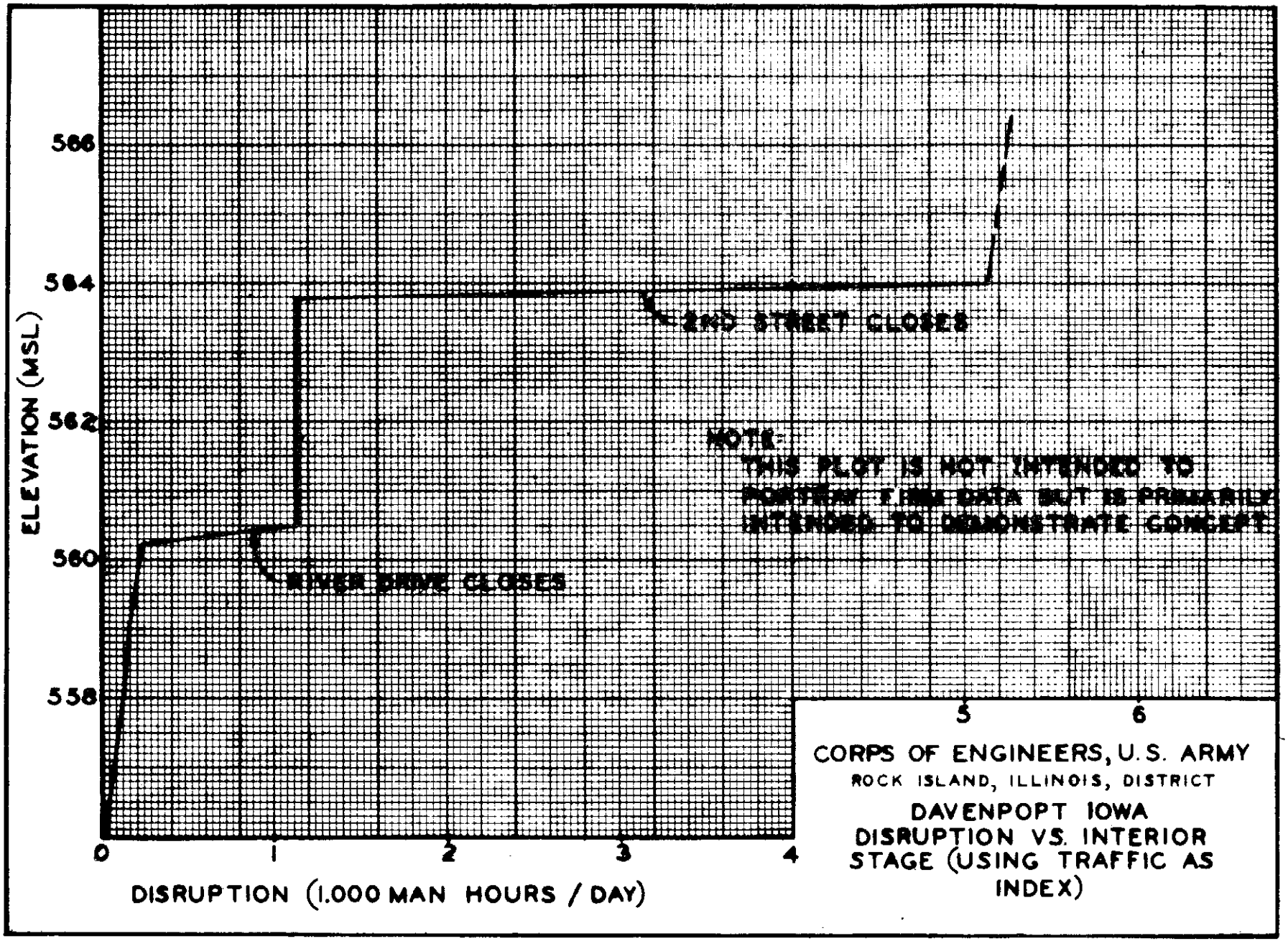

566
564
562
560
558
ELEVATION (MSL)
2ND STREET CLOSES
RIVER DRIVE CLOSES
NOTE:
THIS PLOT IS NOT INTENDED TO
PORTRAY FIRM DATA BUT IS PRIMARILY
INTENDED TO DEMONSTRATE CONCEPT
0
1
2
3
4
5
6
DISRUPTION (1.000 MAN HOURS / DAY)
CORPS OF ENGINEERS, U.S. ARMY
ROCK ISLAND, ILLINOIS, DISTRICT
DAVENPOPT IOWA
DISRUPTION VS. INTERIOR
STAGE (USING TRAFFIC AS
INDEX)

PLATE A-6.7

NO. 340-20 DIETZGEN GRAPH PAPER
20 X 20 PER INCH
DIETZGEN CORPORATION

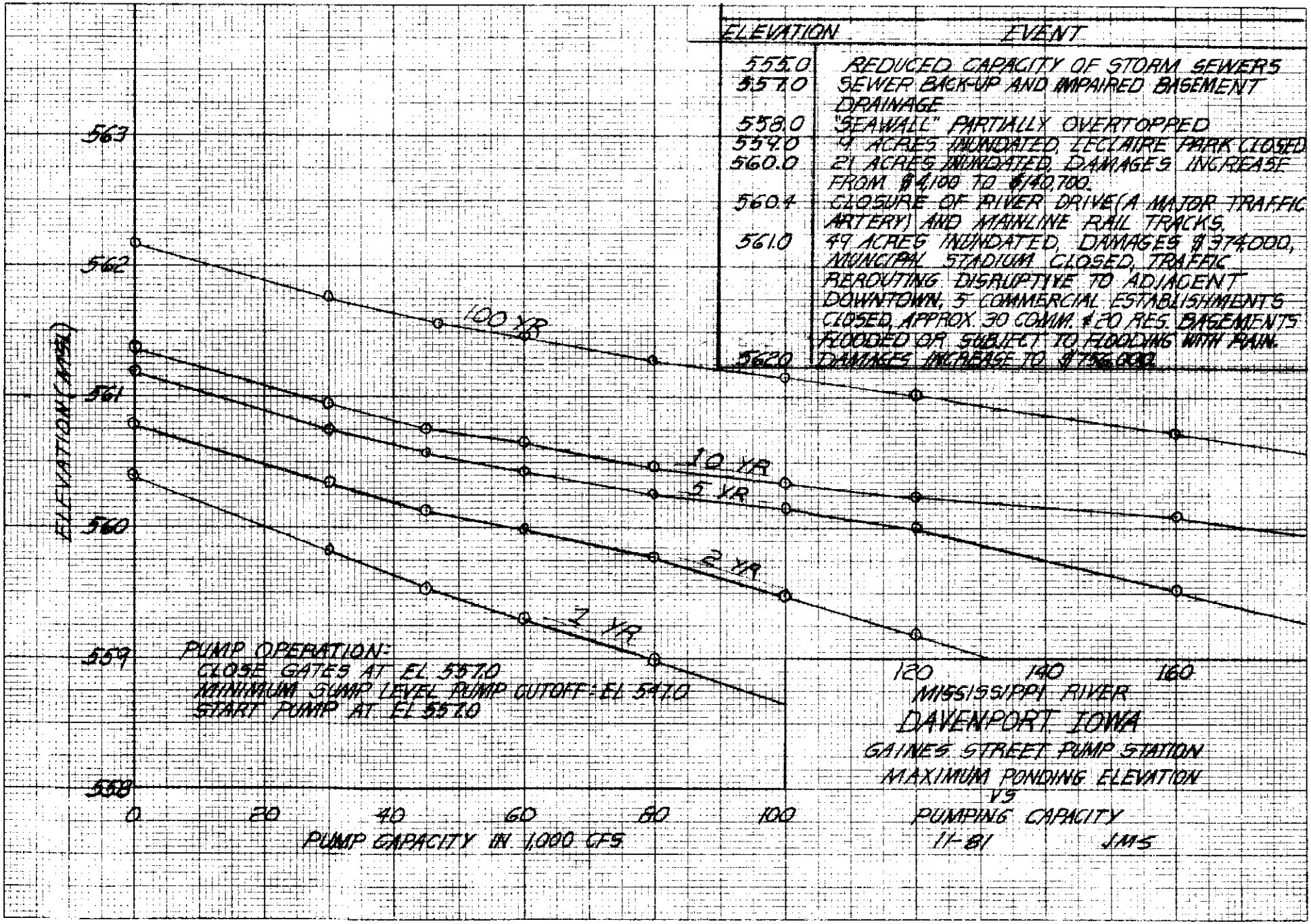


PLATE A-68

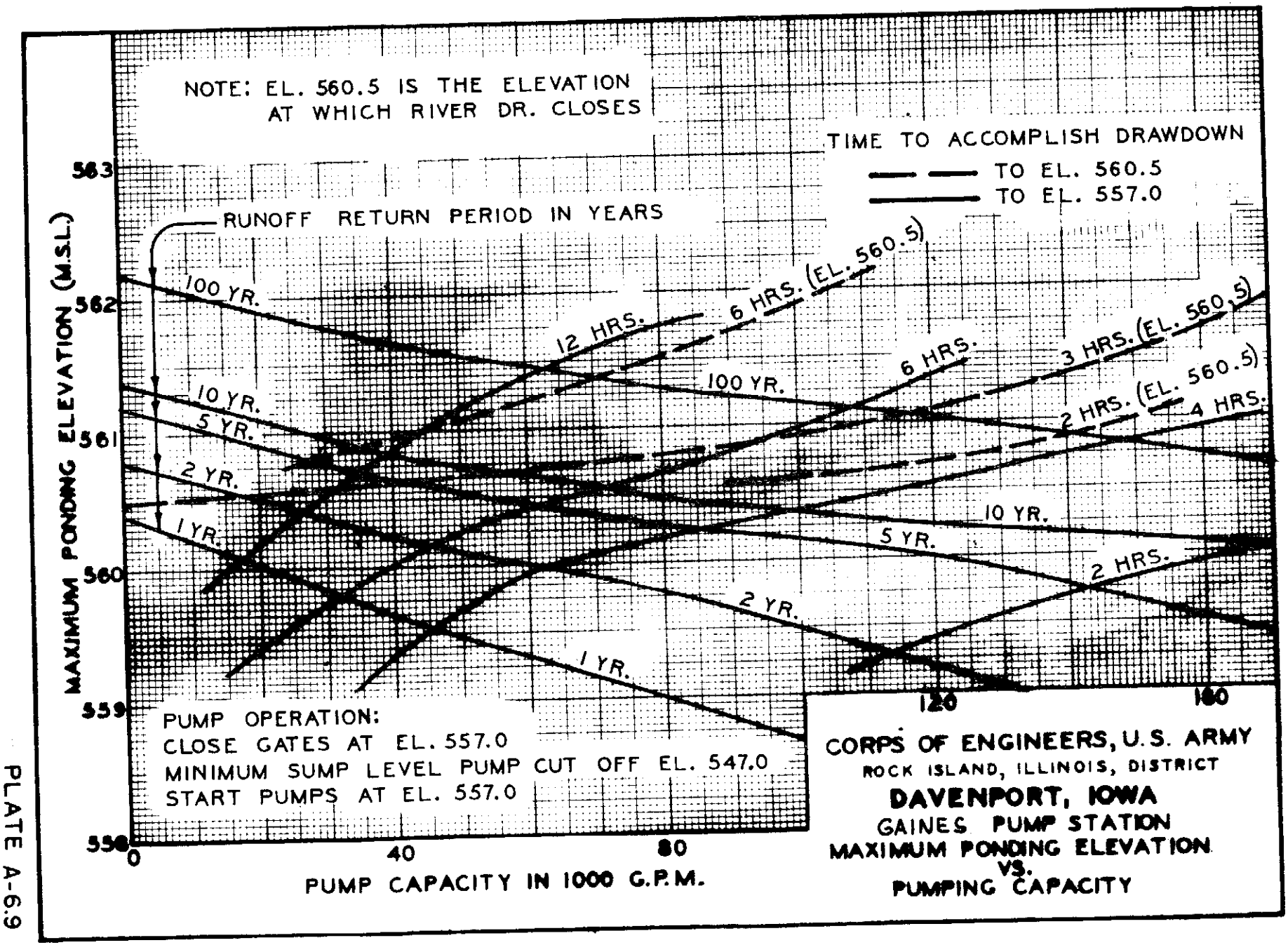

NOTE: EL. 560.5 IS THE ELEVATION AT WHICH RIVER DR. CLOSES
TIME TO ACCOMPLISH DRAWDOWN
TO EL. 560.5
TO EL. 557.0
RUNOFF RETURN PERIOD IN YEARS
100 YR.
10 YR.
5 YR.
2 YR.
1 YR.
12 HRS.
6 HRS. (EL. 560.5)
6 HRS.
3 HRS. (EL. 560.5)
2 HRS. (EL. 560.5)
4 HRS.
2 HRS.
MAXIMUM PONDING ELEVATION (M.S.L.)
563
562
561
560
559
558
0
40
80
120
160
PUMP CAPACITY IN 1000 G.P.M.
PUMP OPERATION:
CLOSE GATES AT EL. 557.0
MINIMUM SUMP LEVEL PUMP CUT OFF EL. 547.0
START PUMPS AT EL. 557.0
CORPS OF ENGINEERS, U.S. ARMY
ROCK ISLAND, ILLINOIS, DISTRICT
DAVENPORT, IOWA
GAINES PUMP STATION
MAXIMUM PONDING ELEVATION
VS.
PUMPING CAPACITY

PLATE A-6.9

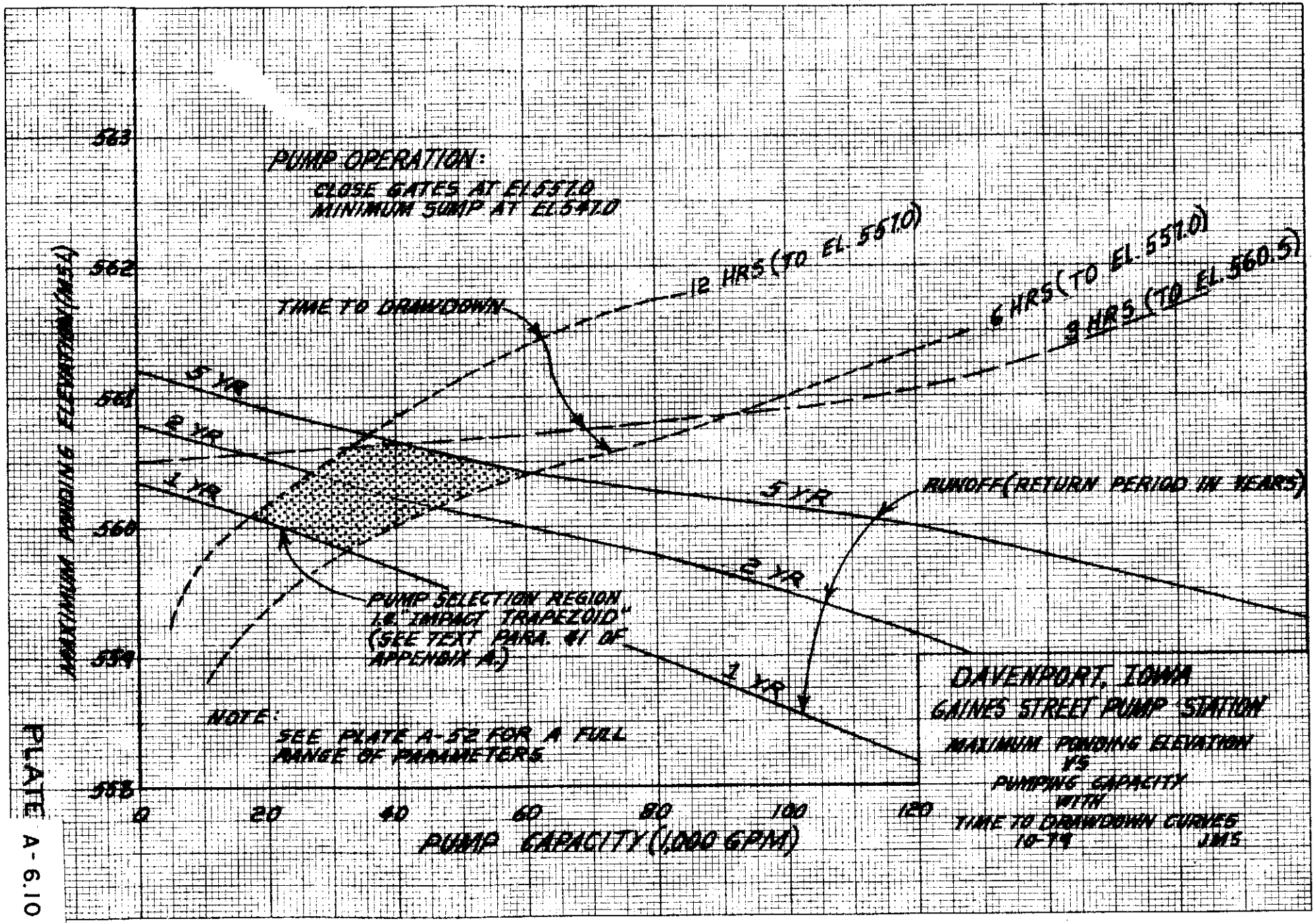

PUMP OPERATION:
CLOSE GATES AT EL 557.0
MINIMUM SUMP AT EL 547.0
TIME TO DRAWDOWN
12 HRS (TO EL. 551.0)
6 HRS (TO EL. 551.0)
3 HRS (TO EL. 560.5)
5 YR
2 YR
1 YR
RUNOFF (RETURN PERIOD IN YEARS)
PUMP SELECTION REGION
I.E. "IMPACT TRAPEZOID"
(SEE TEXT PARA. 41 OF APPENDIX A)
NOTE:
SEE PLATE A-52 FOR A FULL RANGE OF PARAMETERS
MAXIMUM PONDING ELEVATION (FEET)
563
562
561
560
559
558
0
20
40
60
80
100
120
PUMP CAPACITY (1,000 GPM)
DAVENPORT, IOWA
GAINES STREET PUMP STATION
MAXIMUM PONDING ELEVATION
VS
PUMPING CAPACITY
WITH
TIME TO DRAWDOWN CURVES
10-79 JMS

PLATE A-6.10

NO. 340-L310 DIETZGEN GRAPH PAPER
SEMI-LOGARITHMIC
3 CYCLES X 10 DIVISIONS PER INCH
DIETZGEN CORPORATION
MADE IN U.S.A.

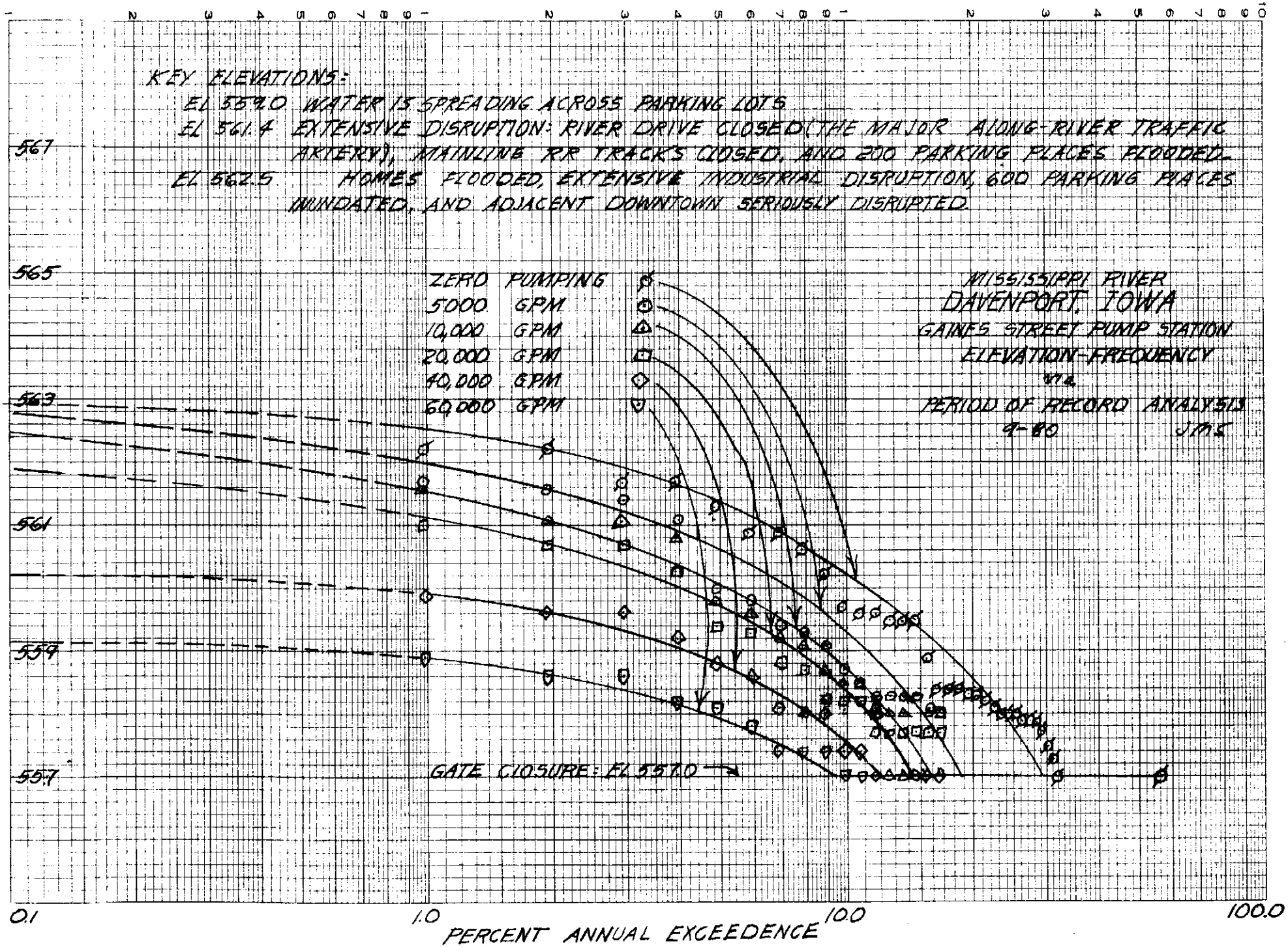


PLATE A-6.11

Schedule No.	Conditions that Determine Operations: River Stage in ft. m.s.l. at Davenport Gage — Prevailing on day of Operation	Conditions that Determine Operations: River Stage in ft. m.s.l. at Davenport Gage — Highest Forecast	Operation of Gaines Street Pump Station: Gravity Outlet	Operation of Gaines Street Pump Station: Pumping Operation	Key Elevations: Lowest Gate Closing River Stage	Key Elevations: Sump Level for Pump Activation	Key Elevations: Sump Level for Pump Cut-off
1	2	3	4	5	6	7	8
1	Below 557.0	Below 557.0	Fully open	None	557.0	---	---
2	Above 559.0	Above 559.0	Fully closed except when sump exceeds river by 1.0 feet.	Pump sump down to El. 547.0, and operate as long as gates are closed.	557.0	557.0	547.0

NOTE:
The sump levels and river stages correspond to the Davenport gage: Lock and Dam No. 15 Tailwater.

Plate A-612

TABLE 4-A

PUMP STATION REVIEW

PROJECT: Davenport, Iowa | STATION LOCATION: Gaines Street | CLOSURE/PUMPDOWN: 557.0/547.0 | DATE: June 1976

TEST	TANGIBLE EFFECTS					INTANGIBLE EFFECTS								
						Transportation		Industrial/Commercial			Social		Aesthetic Environmental	Health/ Safety
Event	Pump Capacity 1000 g.p.m.	Peak Stage m.s.l.	Time to Drawdown	Damages $1,000	Item Reference Paragraph	Disruption	Time 4/	Disruption	Time 4/	Homes Affected	Disruption	Time 4/	Degree 5/	Degree 5/
200-Year Flood 1/	0	561.4	37+ days 3/	42.0	12	Severe	10+ days	Mod-Sev	5 days	-	Light	-	Mod-Sev	Mod-Sev
200-Year Flood 2/	0	563.5	37+ days 3/	90.0	12	Severe	14+ days	Severe	10+ days	-	Moderate	-	Severe	Mod-Sev
Selected Pump 1 Yr	35.0	559.7	6 hrs	27.0	14, 15	None	0 hrs	None	0 hrs					
2 Yr	35.0	560.3	9 hrs	31.5		Moderate	0 hrs	None	0 hrs					
5 Yr	35.0	560.7	12 hrs	34.5		Moderate	4 hrs	Light	3 hrs					
100 Yr	35.0	561.7	24+ hrs	46.0		Severe	19 hrs	Severe	12 hrs	-	Light		Light	Light
20% Selected Pump 1 Yr	7.0	560.3	22 hrs	31.5	14, 15, 21	Light	0 hrs	Light	0 hrs					
2 Yr	7.0	560.7	33 hrs	34.5		Severe	23 hrs	Light	0 hrs					
5 Yr	7.0	561.1	48 hrs	38.0		Severe	45 hrs	Moderate	3 hrs					
100 Yr	7.0	562.1	130+ hrs	53.5		Severe	90+ hrs	Severe	60 hrs	-	Light		Moderate	Moderate
20% Selected Pump 1 Yr	70.0	559.2	3 hrs	18.5	14, 15	None	0 hrs	None	0 hrs					
2 Yr	70.0	559.9	4 hrs	28.5		None	0 hrs	None	0 hrs					
5 Yr	70.0	560.4	5 hrs	32.5		None	0 hrs	None	0 hrs					
100 Yr	70.0	561.4	11 hrs	41.5		Mod-Sev	6 hrs	Mod-Sev	2 hrs	-	Light		Light	Light

NOTES:

1/ 200-year flood event is the expected interior drainage experience during the project design flood with probable closure duration and "typical" runoff, assuming pumping at Credit Island Slough.

2/ Same as 1/ but assume no pumping at the Davenport Harbor, and subsequent merging of pumping areas.

3/ Natural recession in case of zero pumping.

4/ Does not include cleanup time.

5/ Severity of indicated items is dependent upon duration.

SEVERITY FACTORS

ITEM	"C" Elev	FACTORS
Transportation	560.5	Importance and size route.
Industrial/ Commercial	561.5	Size facility and/or community importance.
Social	5/	Extent of disruption of people's lives.
Aesthetic/ Environmental	5/	To be coordinated with others.
Health/Safety	5/	To be coordinated with others.

ROCK ISLAND DISTRICT

MISSISSIPPI RIVER

DAVENPORT, IOWA

GAINES STREET PUMP STATION

PLATE A-6.13

NO. 340-20 DIETZGEN GRAPH PAPER
20 X 20 PER INCH

DIETZGEN CORPORATION
MADE IN U.S.A.

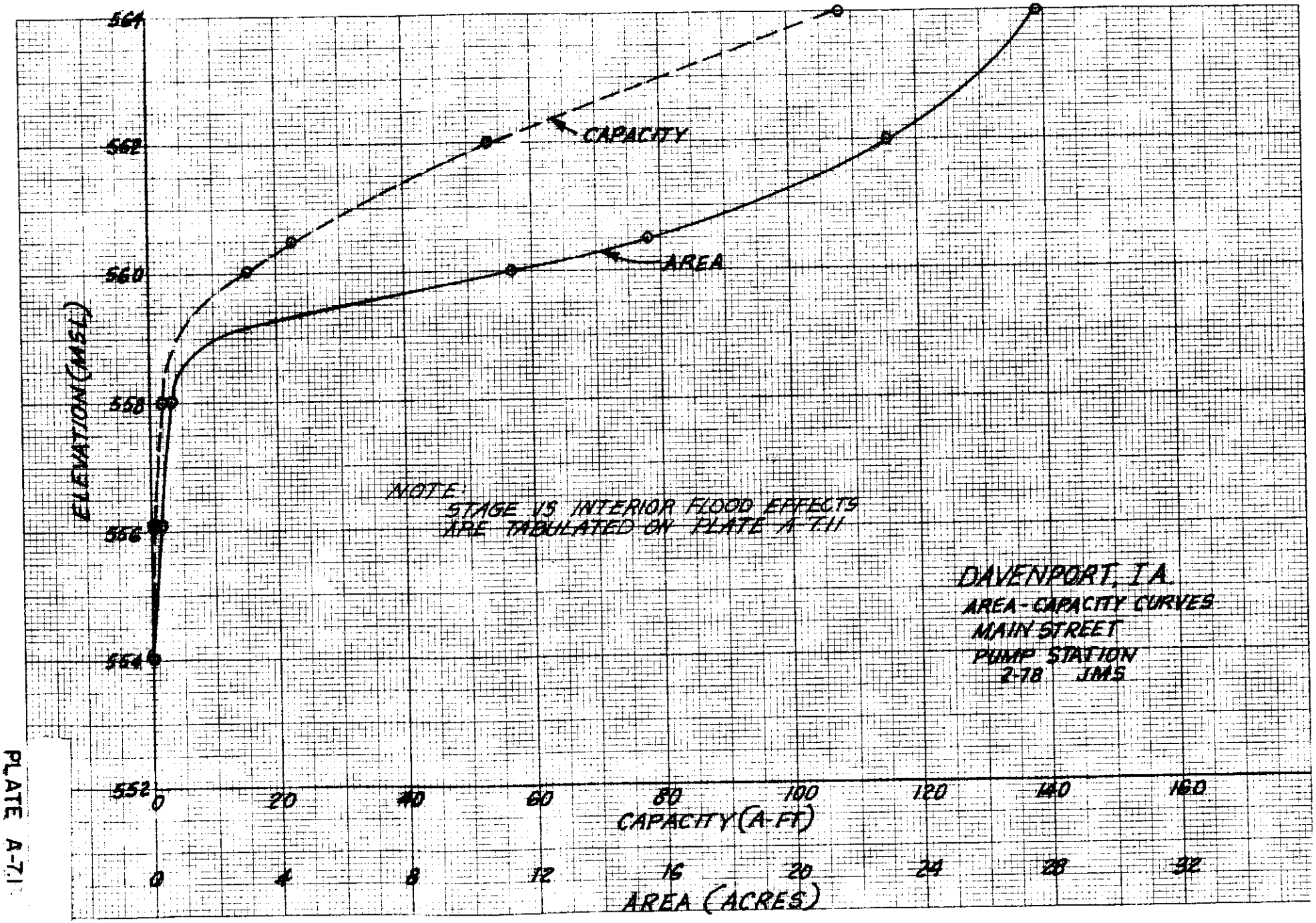


PLATE A-7.1

NO. 340-20 DIETZGEN GRAPH PAPER
20 X 20 PER INCH
DIETZGEN CORPORATION
MADE IN U.S.A.

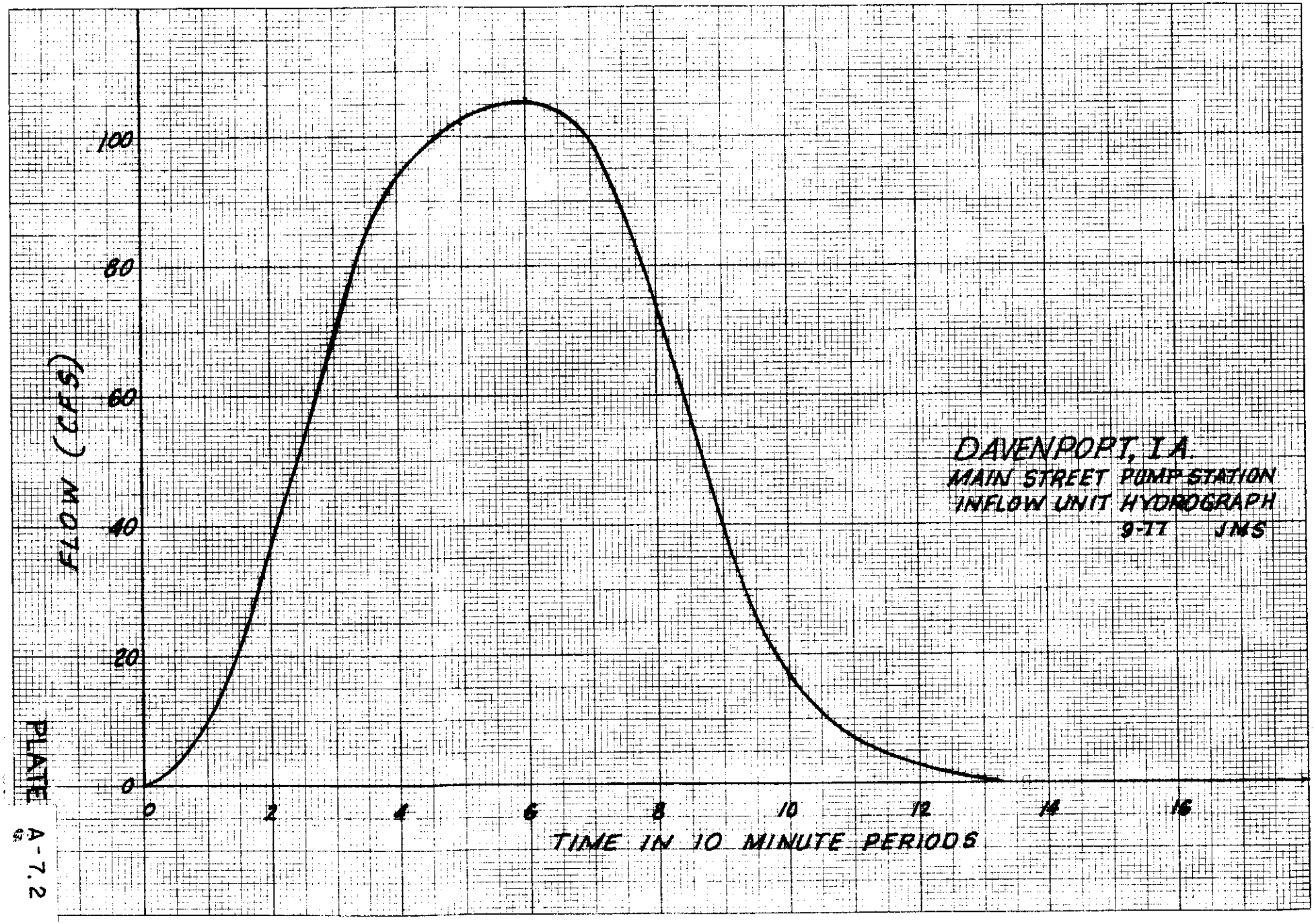


PLATE A-7.2

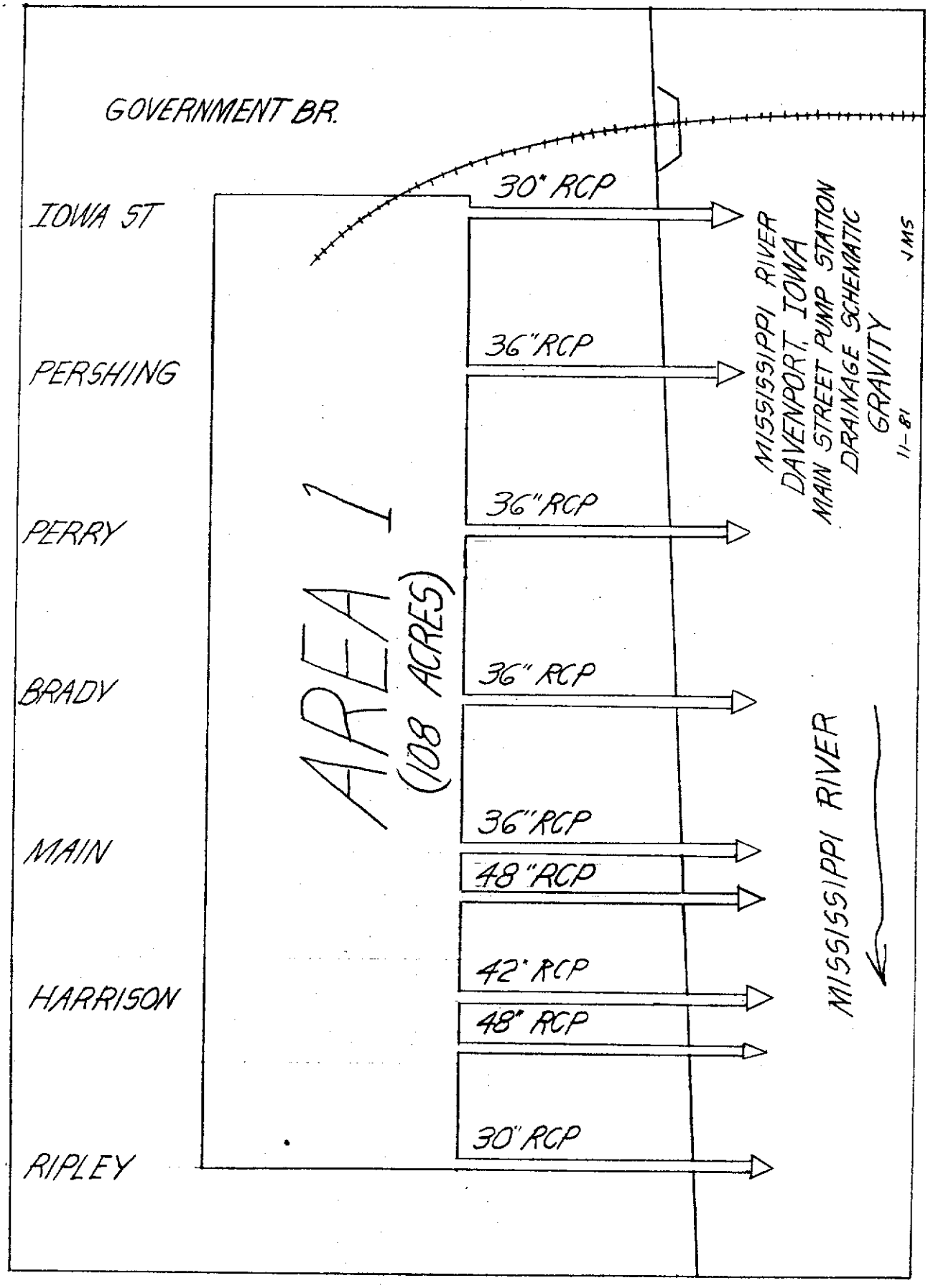

GOVERNMENT BR.
IOWA ST
PERSHING
PERRY
BRADY
MAIN
HARRISON
RIPLEY
30" RCP
36" RCP
36" RCP
36" RCP
36" RCP
48" RCP
42" RCP
48" RCP
30" RCP
AREA 1
(108 ACRES)
MISSISSIPPI RIVER
DAVENPORT, IOWA
MAIN STREET PUMP STATION
DRAINAGE SCHEMATIC
GRAVITY
11-81
JMS
MISSISSIPPI RIVER

PLATE A-7.3

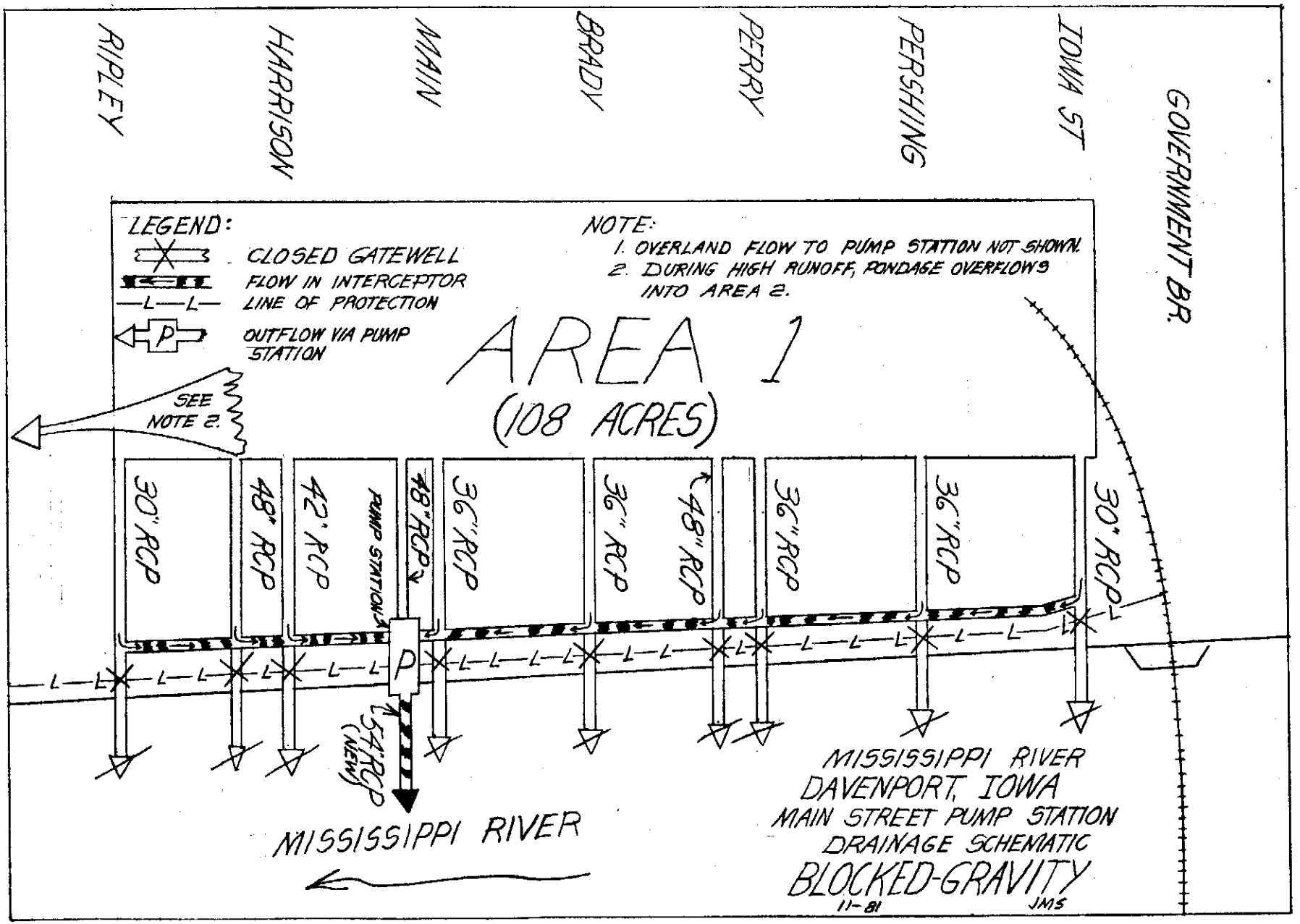

RIPLEY
HARRISON
MAIN
BRADY
PERRY
PERSHING
IOWA ST
GOVERNMENT BR.
LEGEND:
CLOSED GATEWELL
FLOW IN INTERCEPTOR
LINE OF PROTECTION
OUTFLOW VIA PUMP STATION
NOTE:
1. OVERLAND FLOW TO PUMP STATION NOT SHOWN.
2. DURING HIGH RUNOFF, PONDAGE OVERFLOWS INTO AREA 2.
AREA 1
(108 ACRES)
SEE NOTE 2.
30" RCP
48" RCP
42" RCP
PUMP STATION
48" RCP
36" RCP
36" RCP
48" RCP
36" RCP
36" RCP
30" RCP
54" RCP (NEW)
MISSISSIPPI RIVER
MISSISSIPPI RIVER
DAVENPORT, IOWA
MAIN STREET PUMP STATION
DRAINAGE SCHEMATIC
BLOCKED-GRAVITY
11-81
JMS

PLATE A-7.4

NO. 340-20 DIETZGEN GRAPH PAPER
20 X 20 PER INCH
DIETZGEN CORPORATION
MADE IN U.S.A.

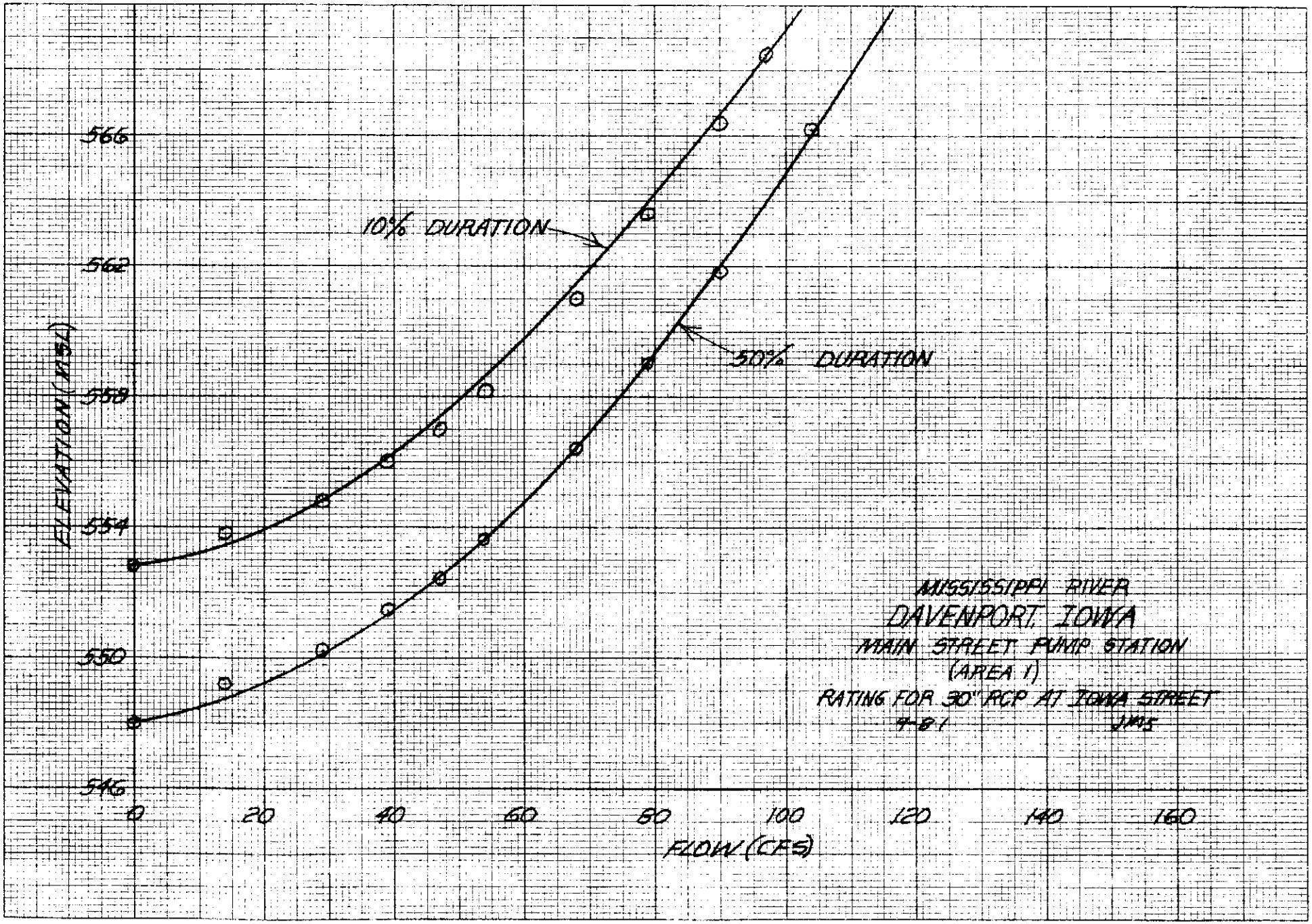


PLATE A-7.51

NO. 340-20 DIETZGEN GRAPH PAPER
20 X 20 PER INCH
DIETZGEN CORPORATION
MADE IN U.S.A.

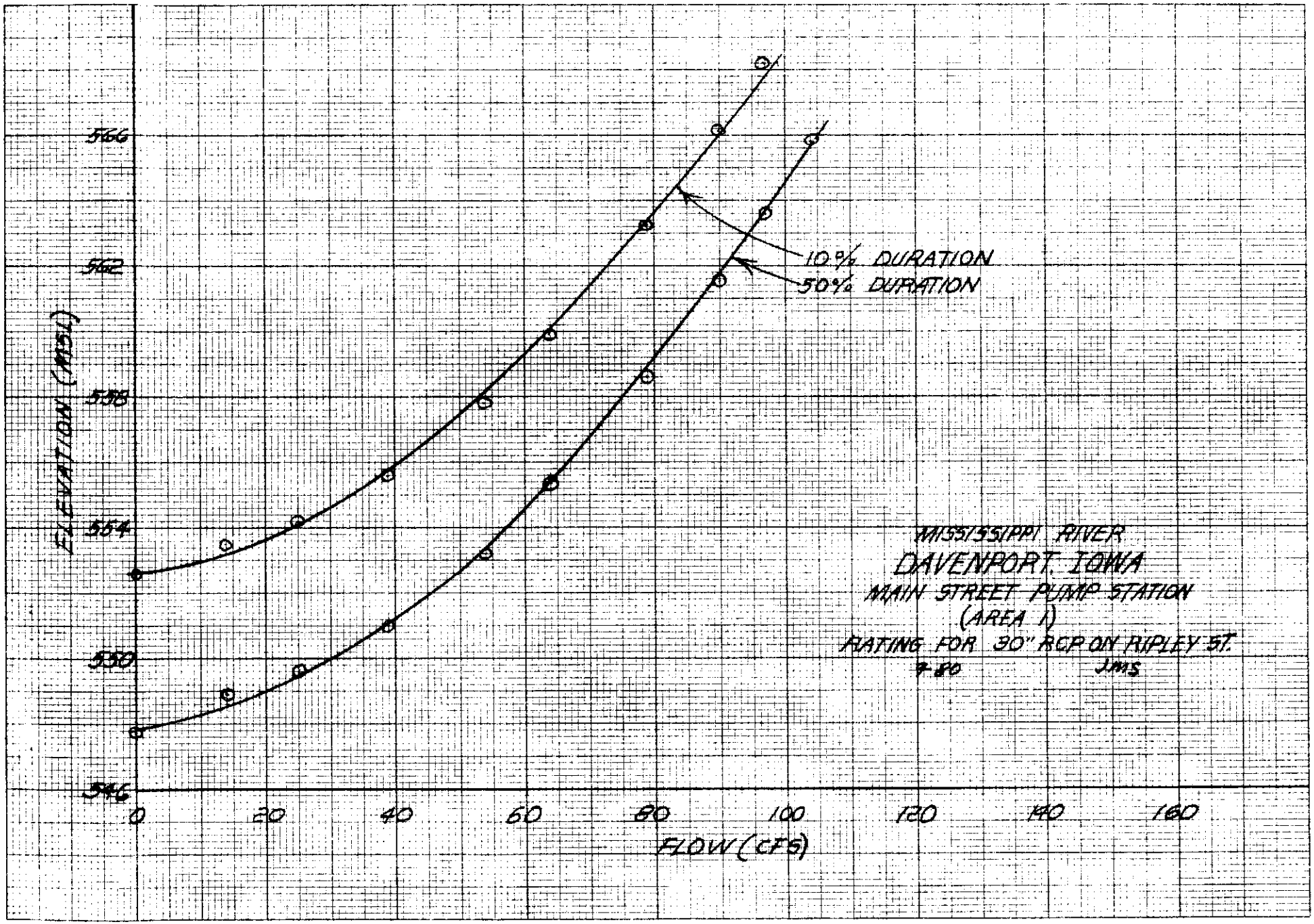


PLATE A-7.52

NO. 340-20 DIETZGEN GRAPH PAPER
20 X 20 PER INCH

DIETZGEN CORPORATION
MADE IN U.S.A.

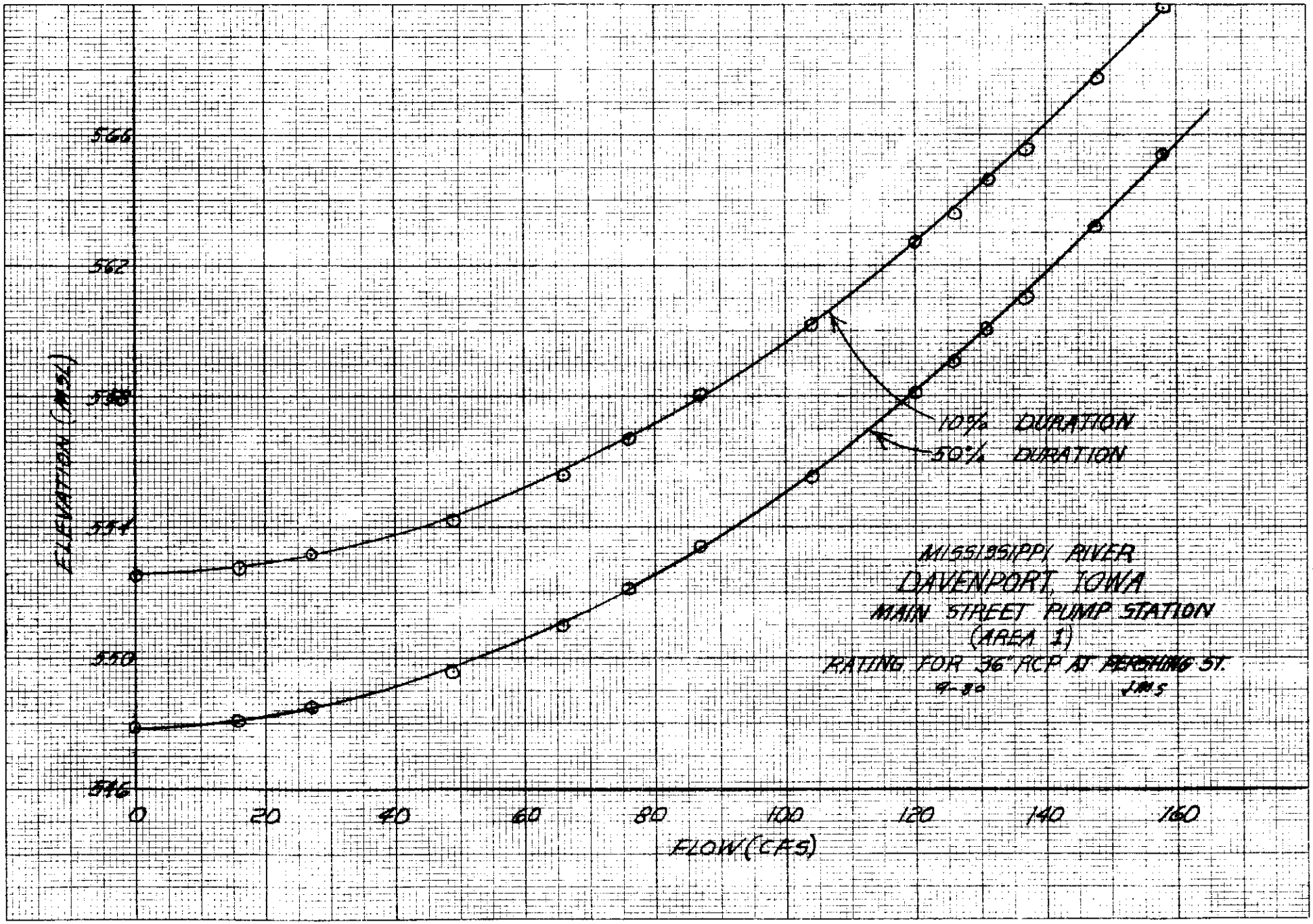


PLATE A-7.53

NO. 340-20 DIETZGEN GRAPH PAPER
20 X 20 PER INCH
DIETZGEN CORPORATION
MADE IN U.S.A.

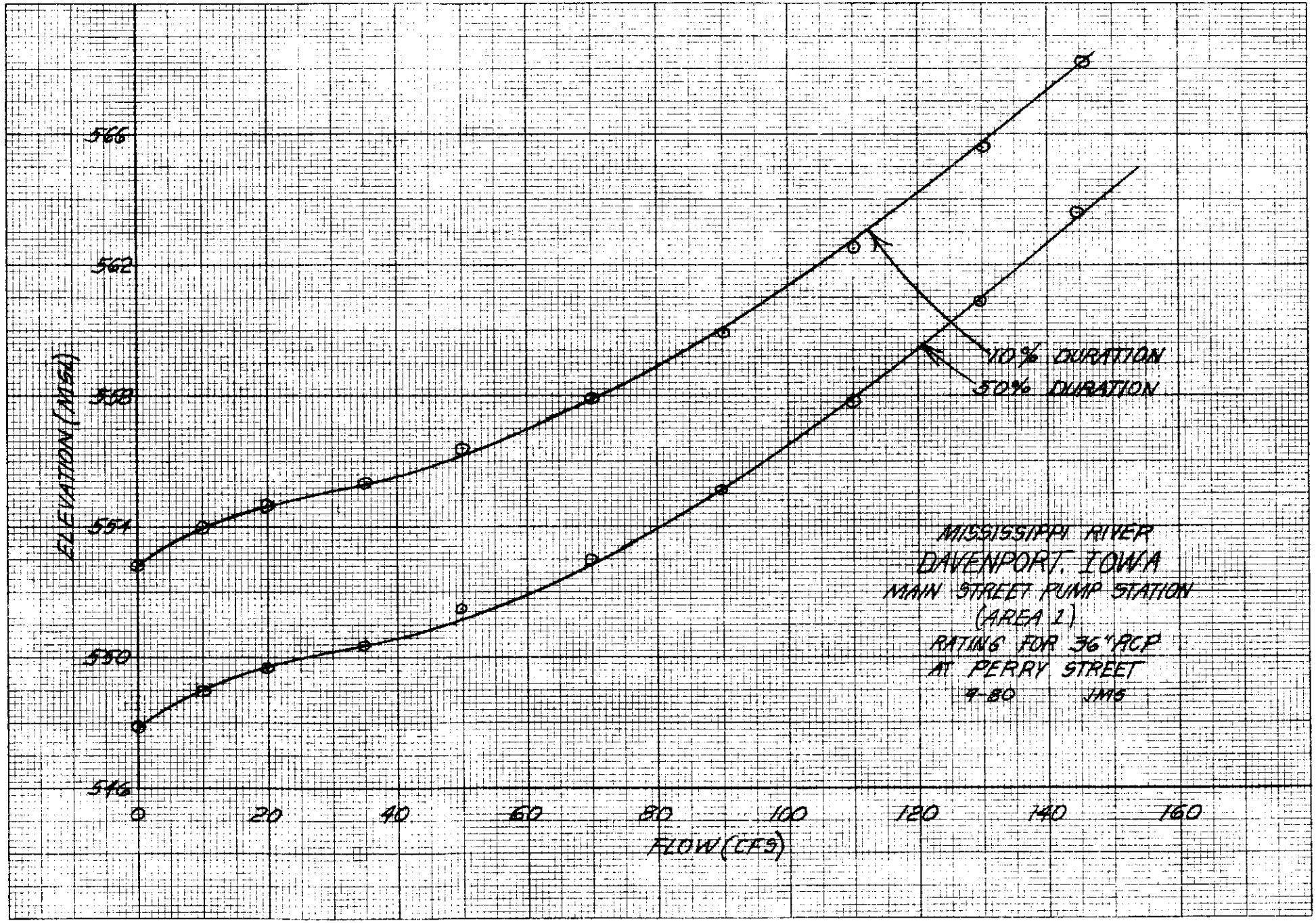


PLATE A-754

NO. 340-20 DIETZGEN GRAPH PAPER
20 X 20 PER INCH
DIETZGEN CORPORATION
MADE IN U.S.A.

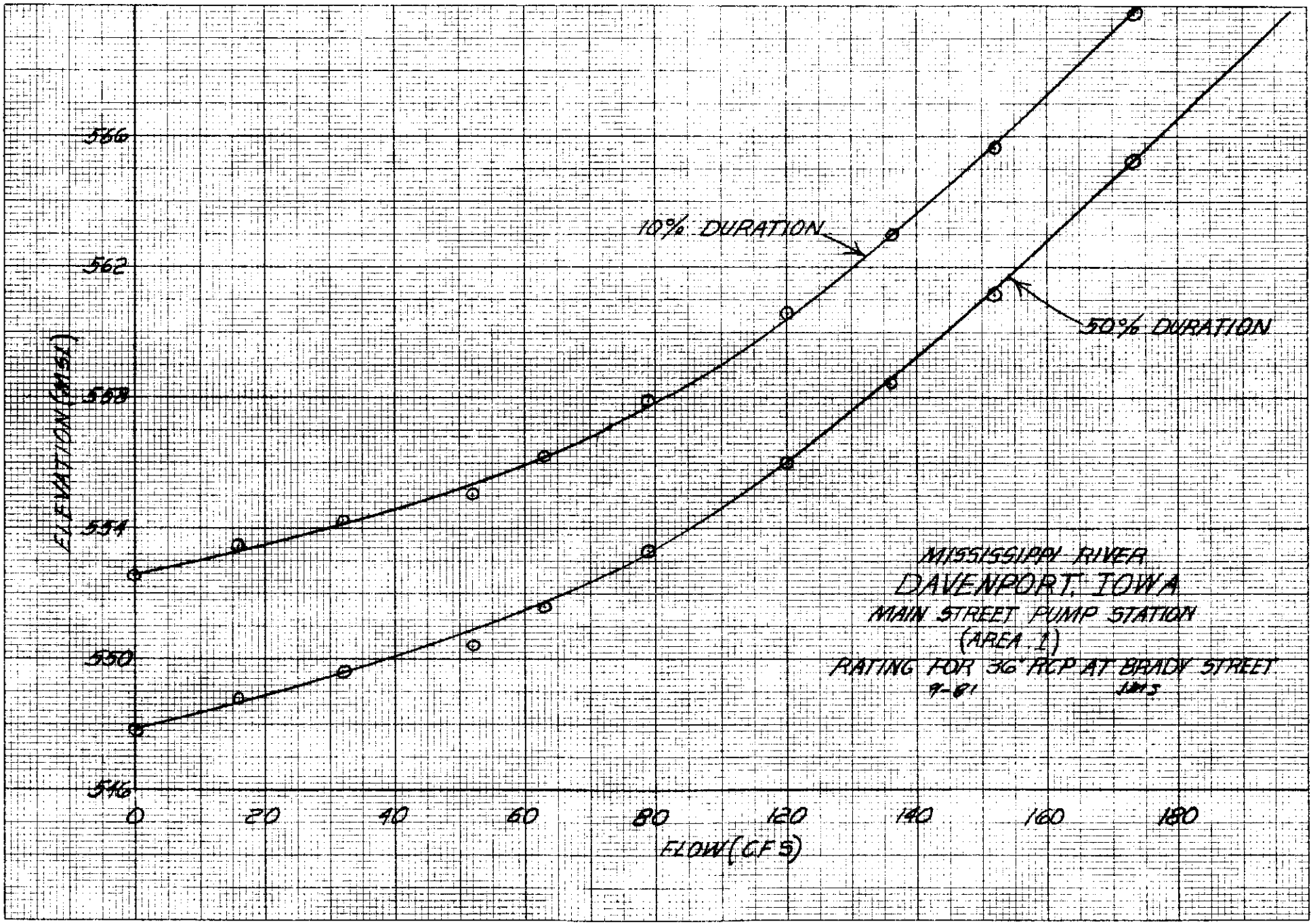


PLATE A-7.55

NO. 340-20 DIETZGEN GRAPH PAPER
20 X 20 PER INCH

DIETZGEN CORPORATION
MADE IN U.S.A.

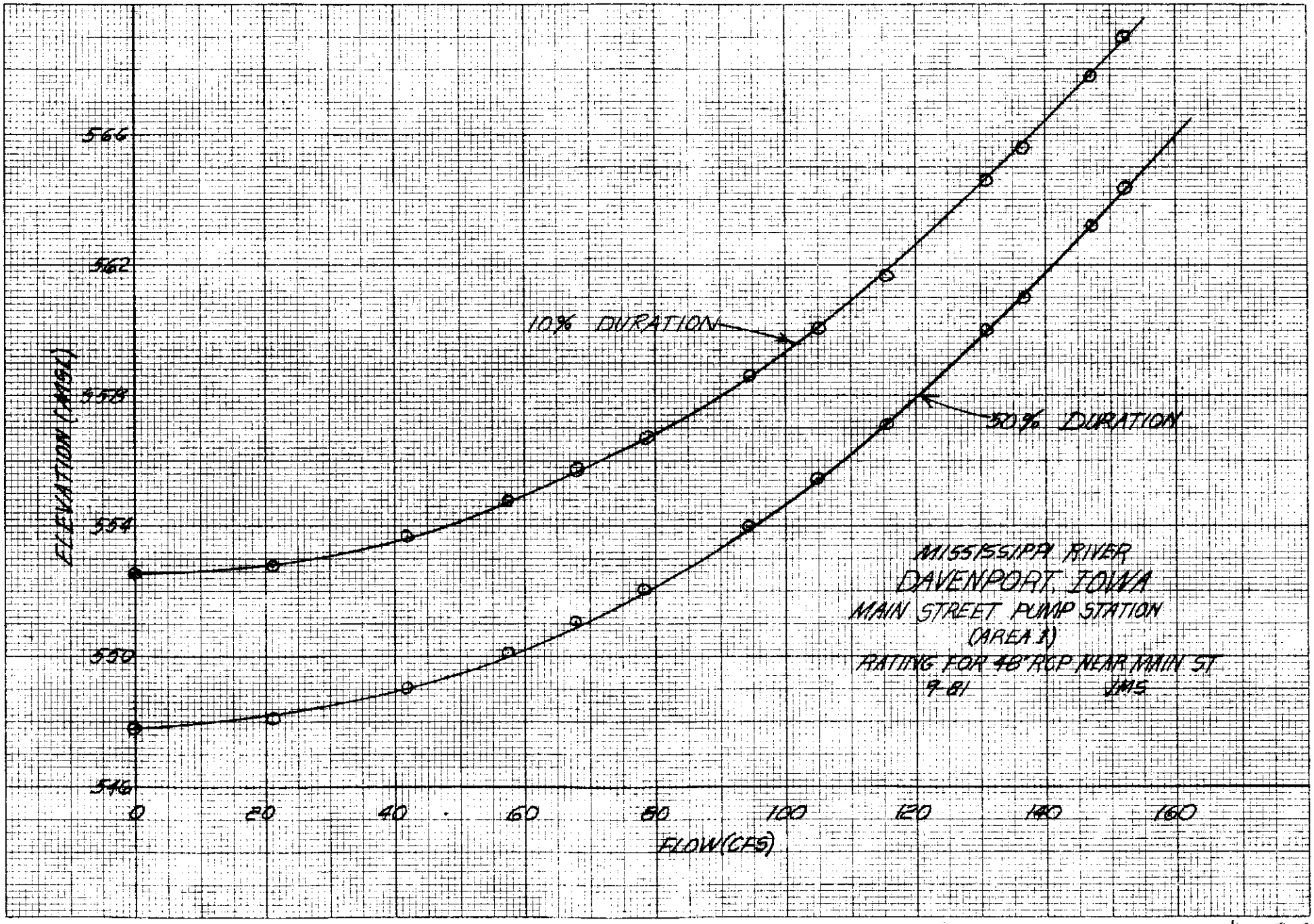


PLATE A-756

NO. 340-20 DIETZGEN GRAPH PAPER
20 X 20 PER INCH

DIETZGEN CORPORATION
MADE IN U.S.A.

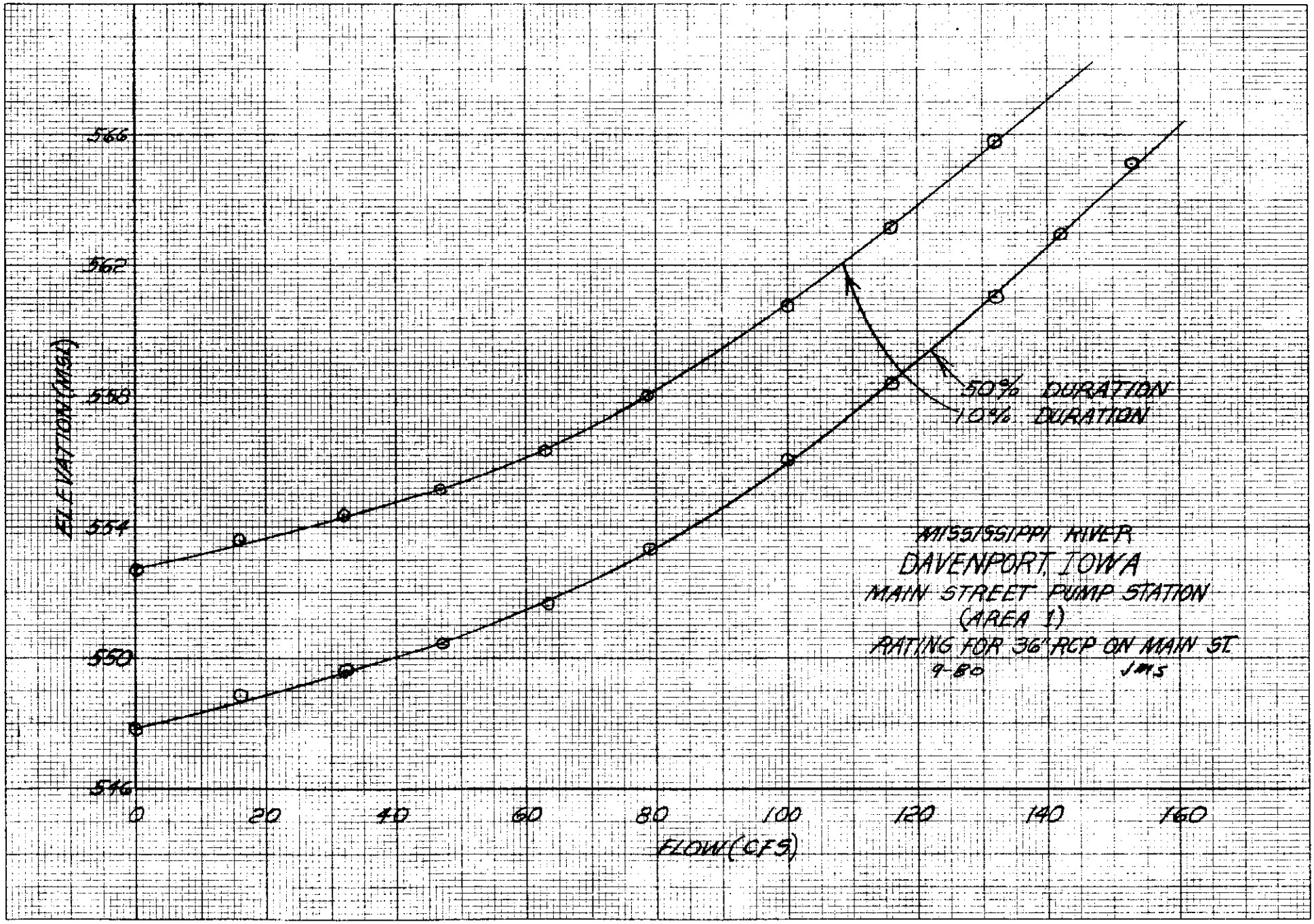


PLATE A-7.5.7

NO. 340-20 DIETZGEN GRAPH PAPER
20 X 20 PER INCH
DIETZGEN CORPORATION
MADE IN U.S.A.

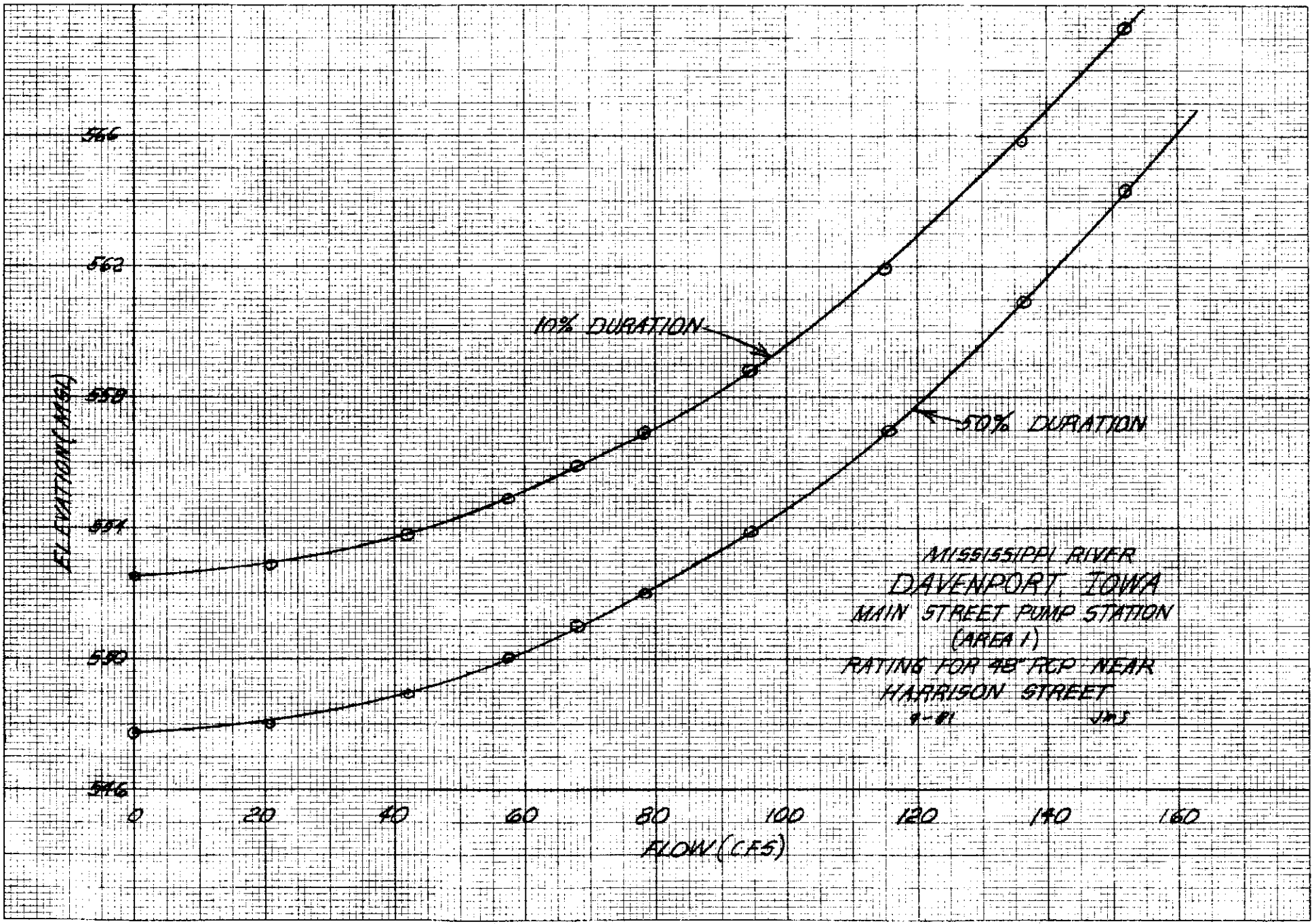


PLATE A-7.58

NO. 340-20 DIETZGEN GRAPH PAPER
20 X 20 PER INCH

DIETZGEN CORPORATION
MADE IN U.S.A.

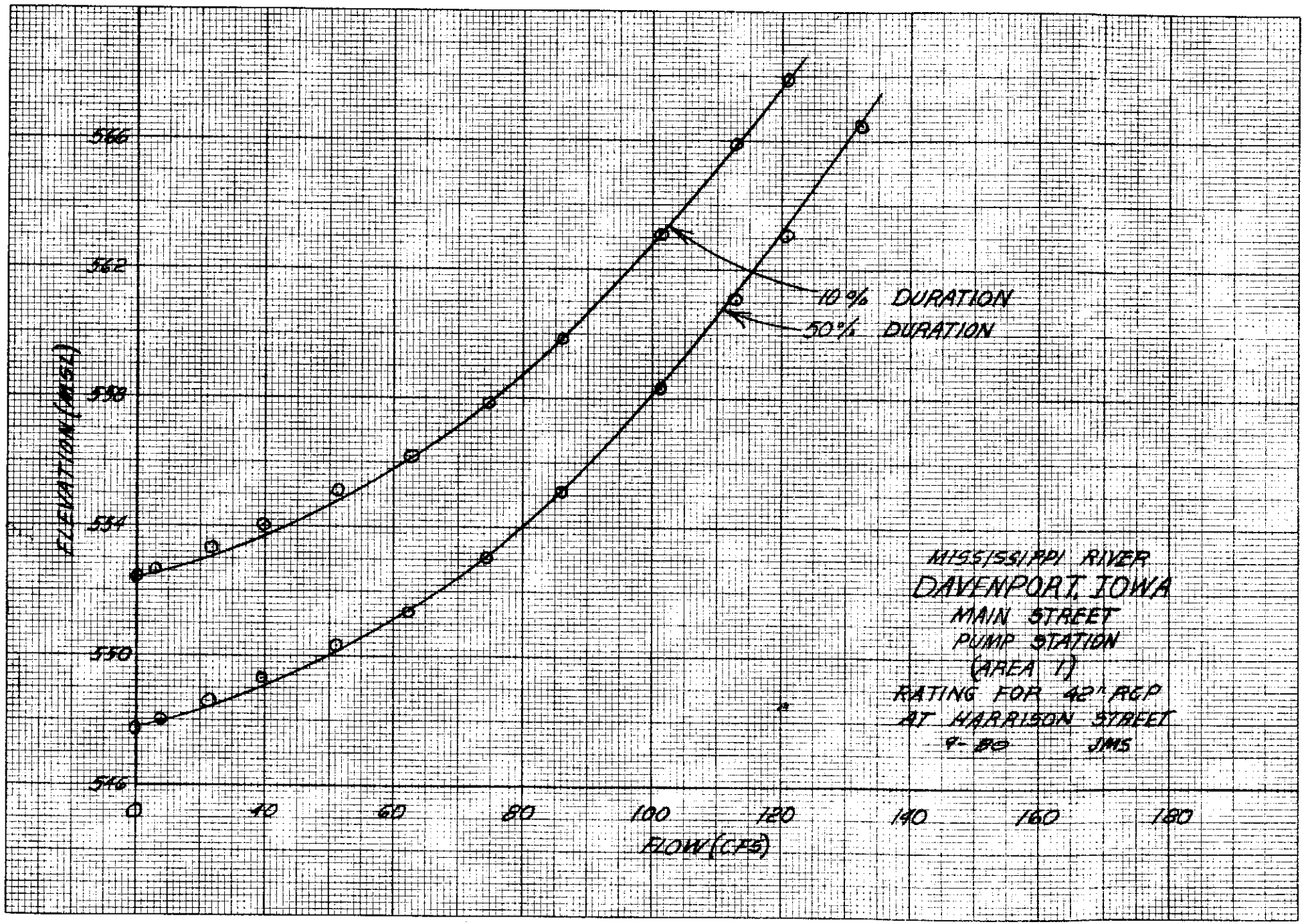
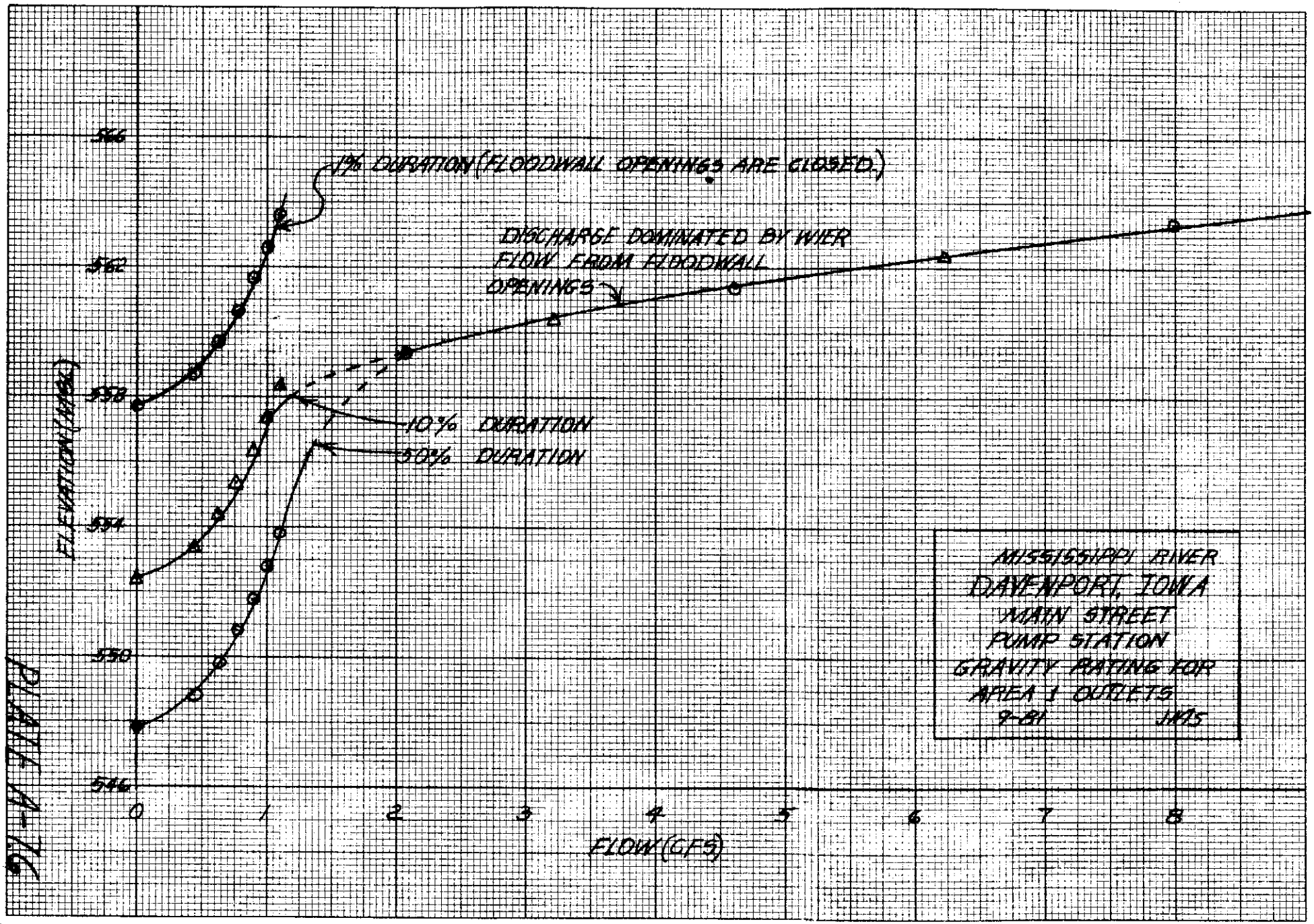


PLATE 7.9

NO. 340-20 DIETZGEN GRAPH PAPER
20 X 20 PER INCH

DIETZGEN CORPORATION
MADE IN U.S.A.



NO. 340-20 DIETZGEN GRAPH PAPER
20 X 20 PER INCH

DIETZGEN CORPORATION
MADE IN U.S.A.

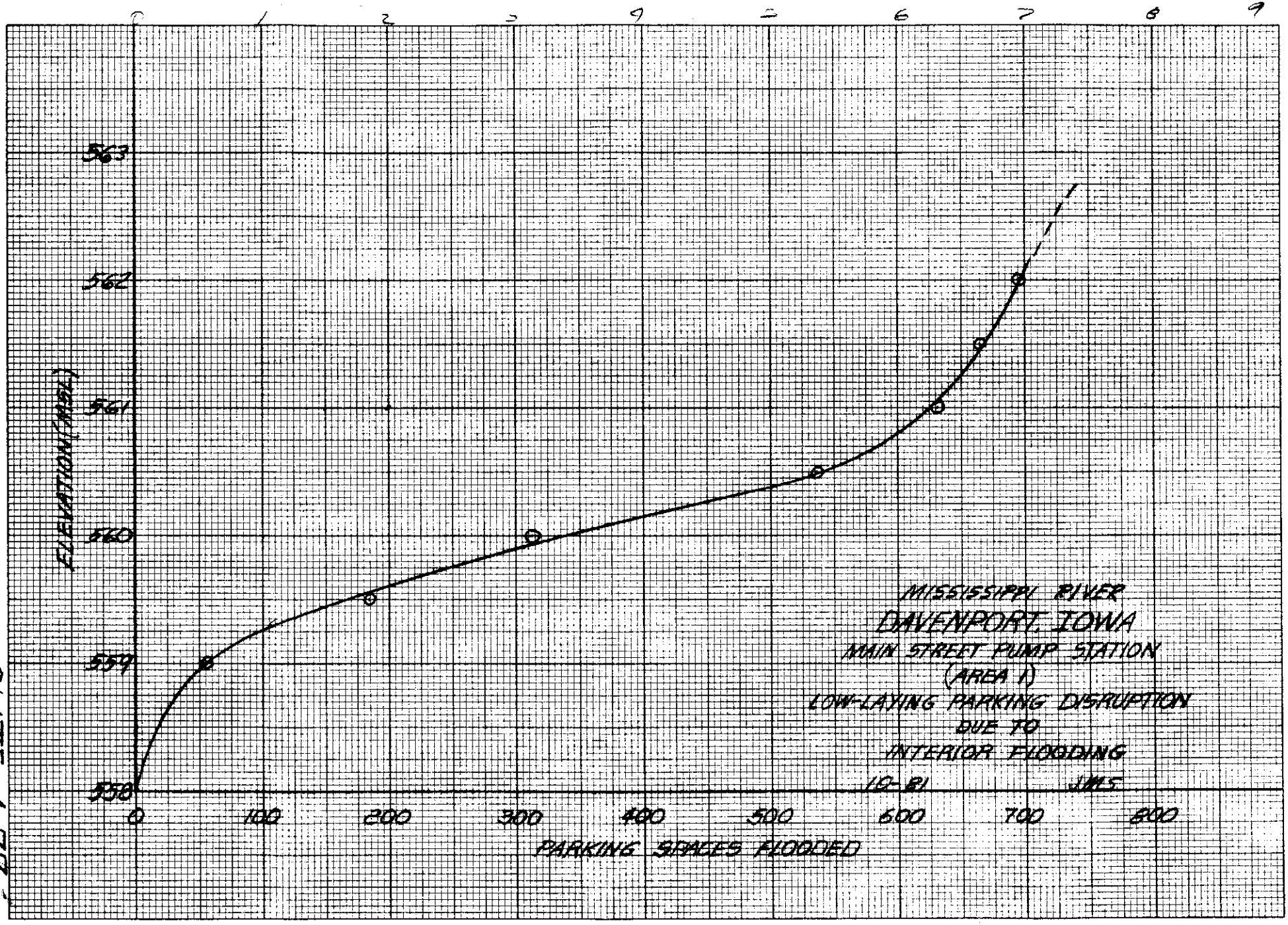


PLATE A-7.2

NO. 340-20 DIETZGEN GRAPH PAPER
20 X 20 PER INCH

DIETZGEN CORPORATION
MADE IN U.S.A.

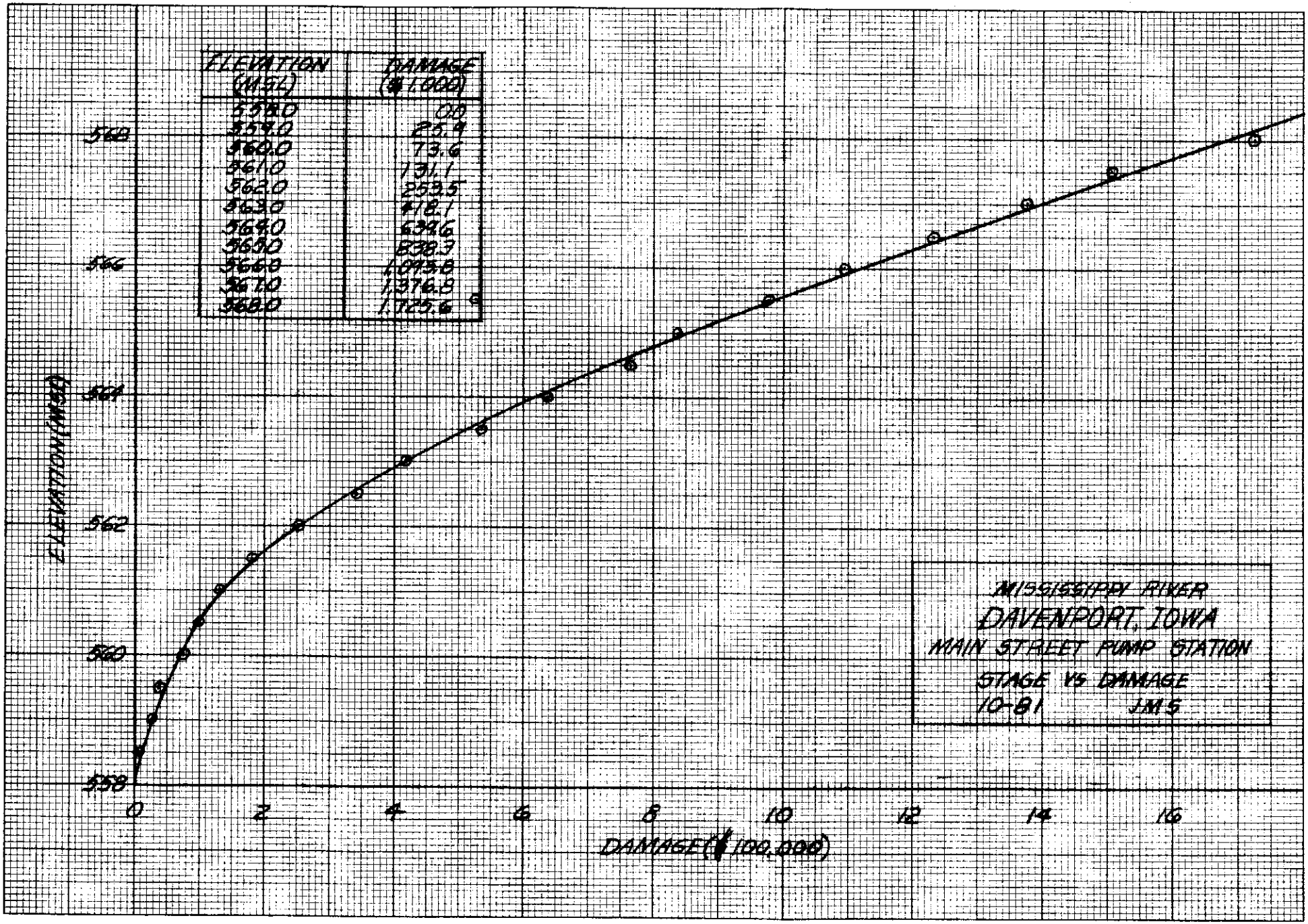


PLATE A-7.8

10-5-81 MAIN STREET, 1.0' GRAVITY SURCHARGE

NO. 340-L310 DIETZGEN GRAPH PAPER
SEMI-LOGARITHMIC
3 CYCLES X 10 DIVISIONS PER INCH

DIETZGEN CORPORATION
MADE IN U.S.A.

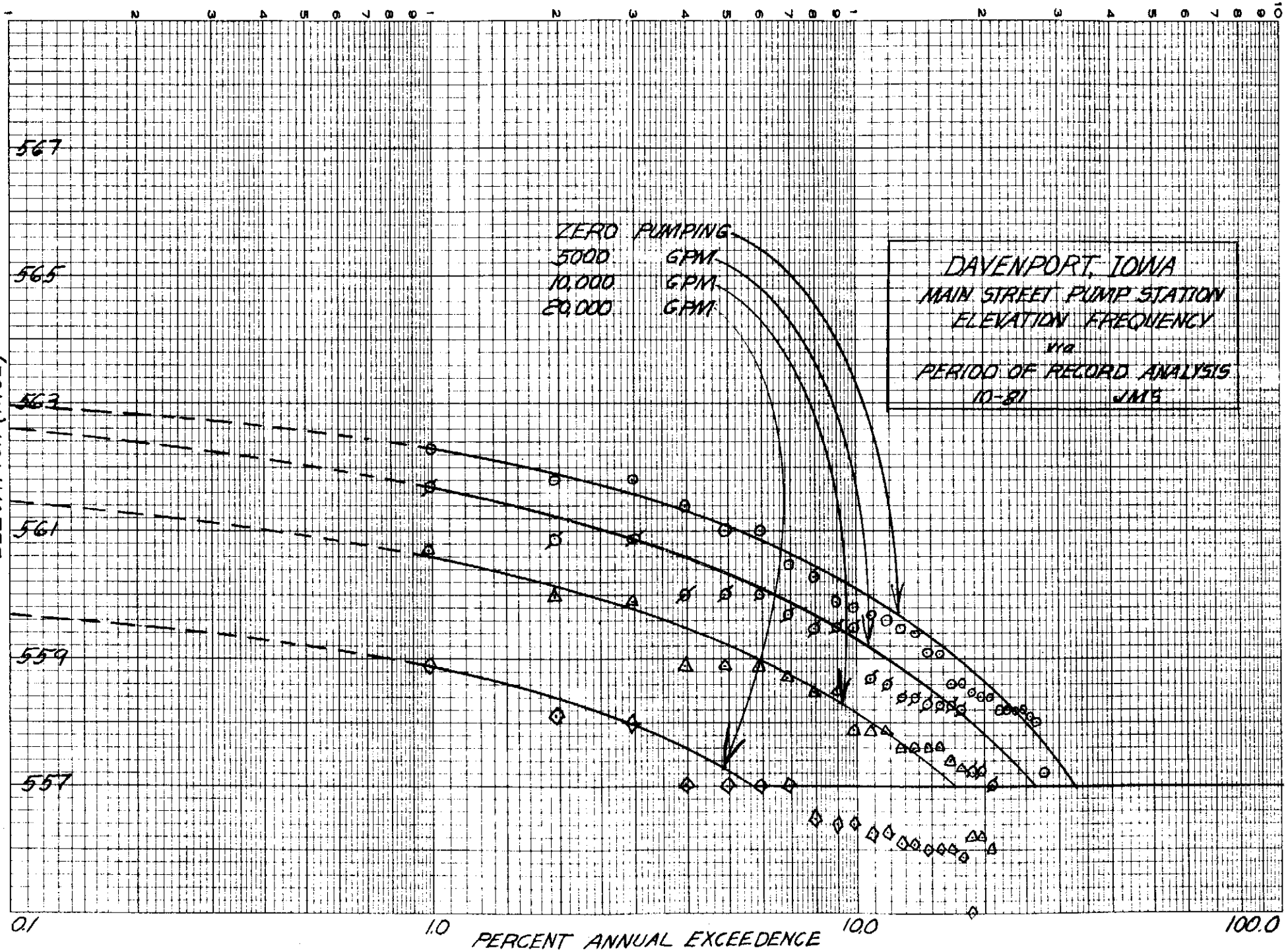


PLATE A-79

NO. 340-20 DIETZGEN GRAPH PAPER
20 X 20 PER INCH

DIETZGEN CORPORATION
MADE IN U.S.A.

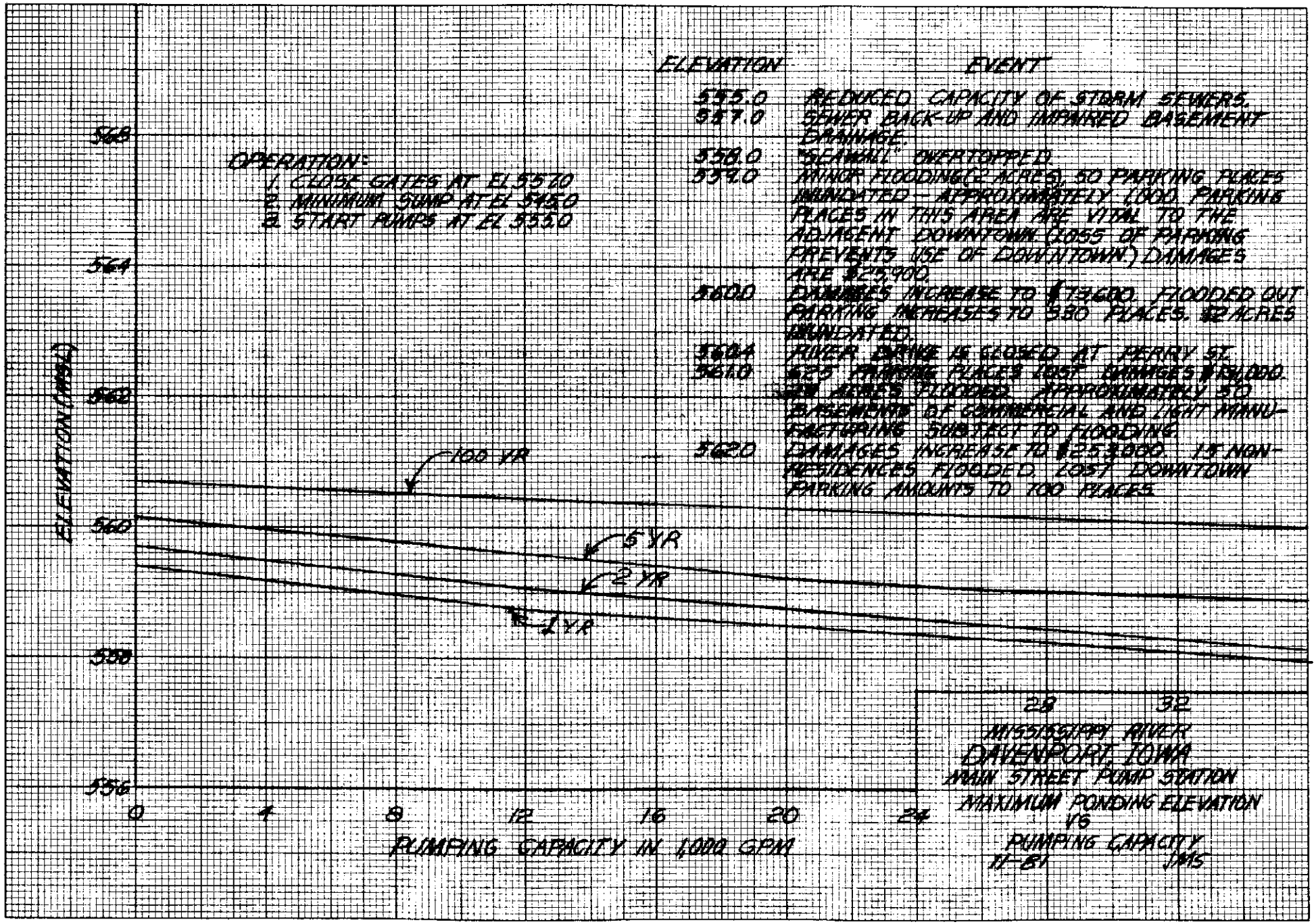


PLATE A-710

NO. 340-20 DIETZGEN GRAPH PAPER
20 X 20 PER INCH

DIETZGEN CORPORATION
MADE IN U.S.A.

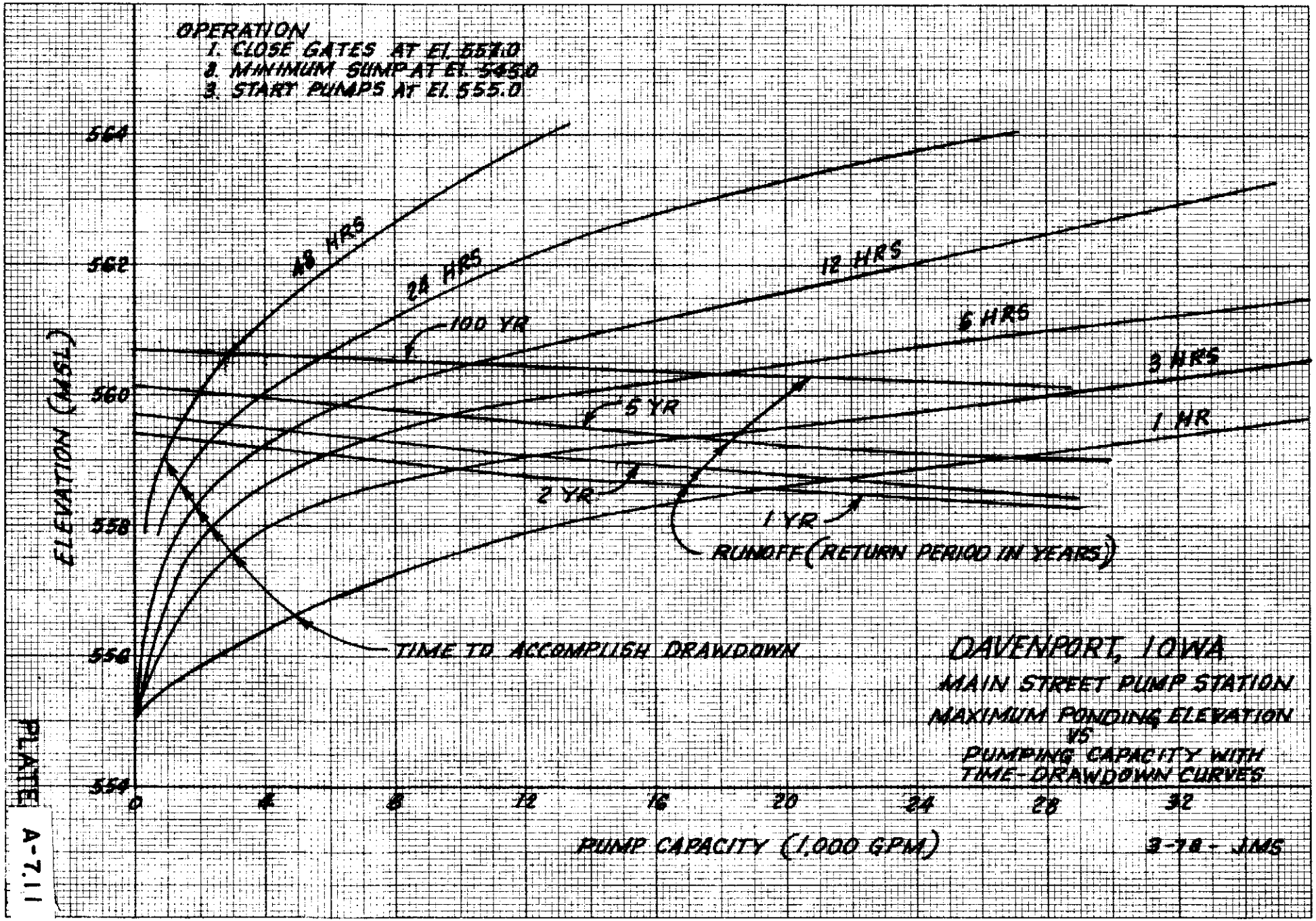


PLATE A-7.11

NO. 340-20 DIETZGEN GRAPH PAPER
20 X 20 PER INCH

DIETZGEN CORPORATION
MADE IN U.S.A.

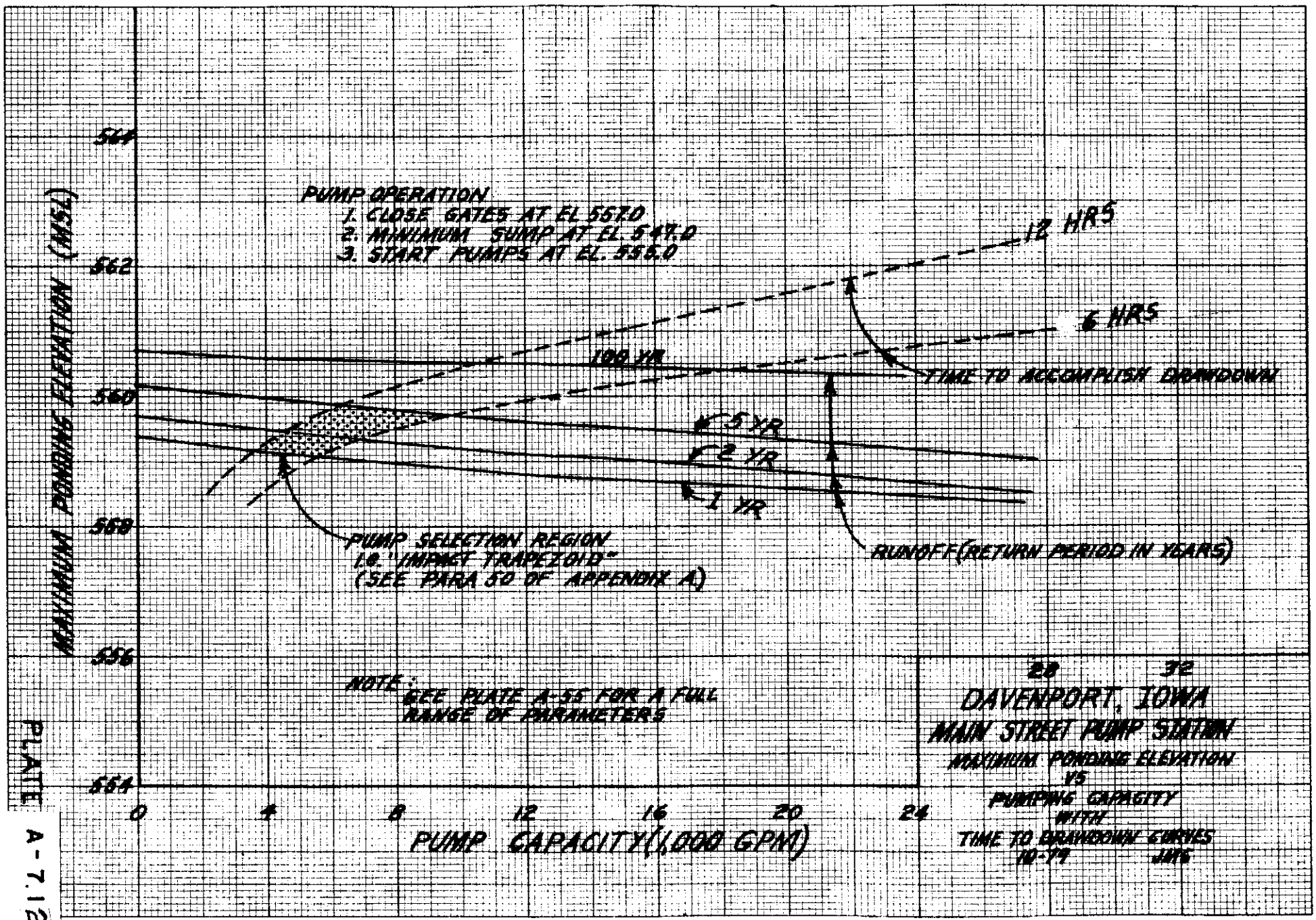


PLATE A-7.12

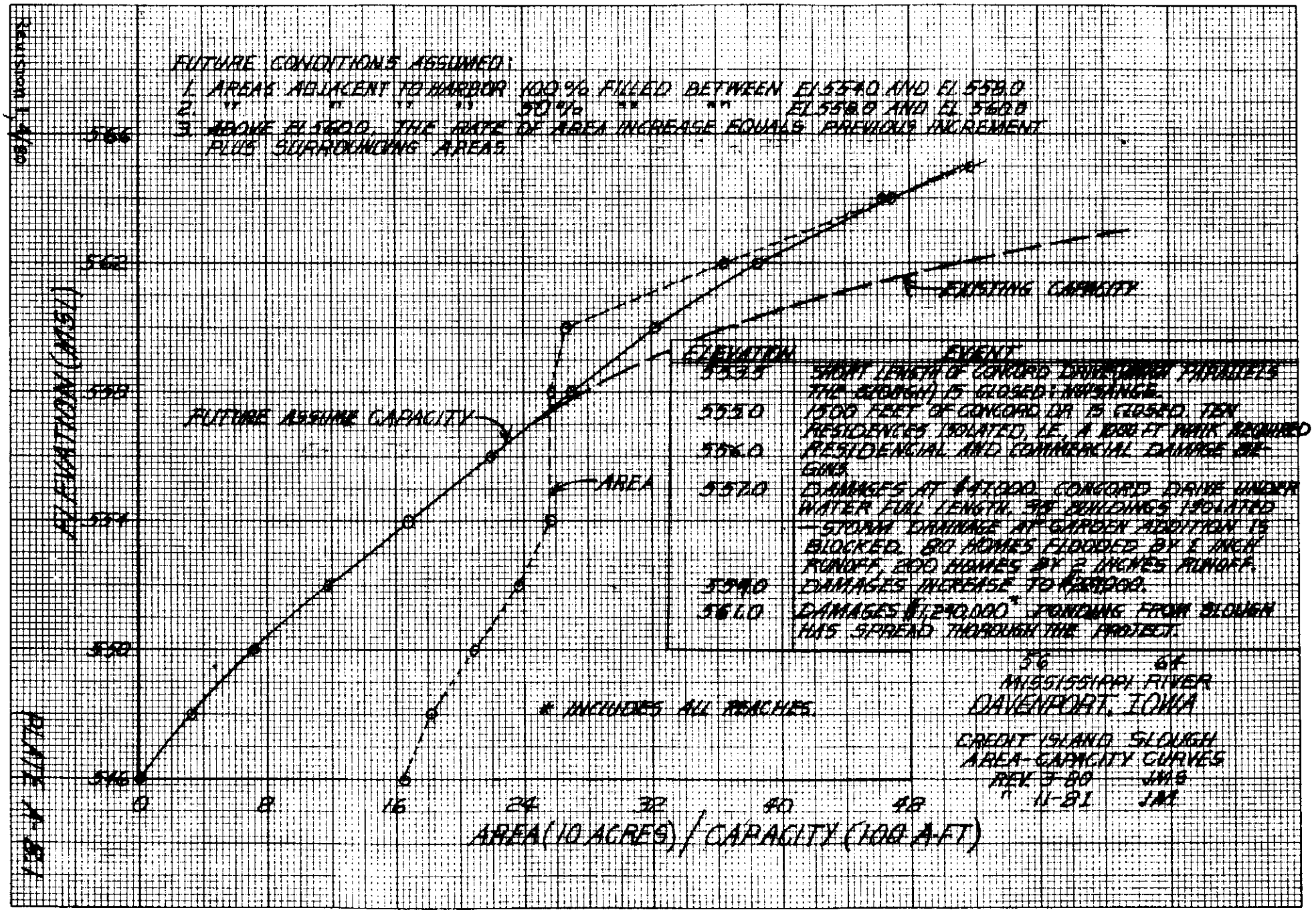
Schedule No	Conditions that Determine Operations: River Stage in ft.m.s.l. at Davenport Gage		Operation of Main Street Pump Station		Key Elevations		
	Prevailing on day of Operation	Highest Forecast	Gravity Outlet	Pumping Operation	Lowest Gate Closing River Stage	Sump Level for Pump Activation	Sump Level for Pump Cut-off
1	2	3	4	5	6	7	8
1	Below 557.0	Below 557.0	Fully open	None	557.0	---	---
2	Above 559.0	Above 559.0	Fully open except when sump exceeds river by 1.0 ft.	Pump sump down to El. 547.0, and operate as long as gates are closed.	557.0	557.0	547.0

NOTE: The sump levels and river stages correspond to the Davenport gage: Lock and Dam No. 15 Tailwater.

PLATE A-7.13

NO. 340-20 DIETZGEN GRAPH PAPER
20 X 20 PER INCH

DIETZGEN CORPORATION
MADE IN U.S.A.



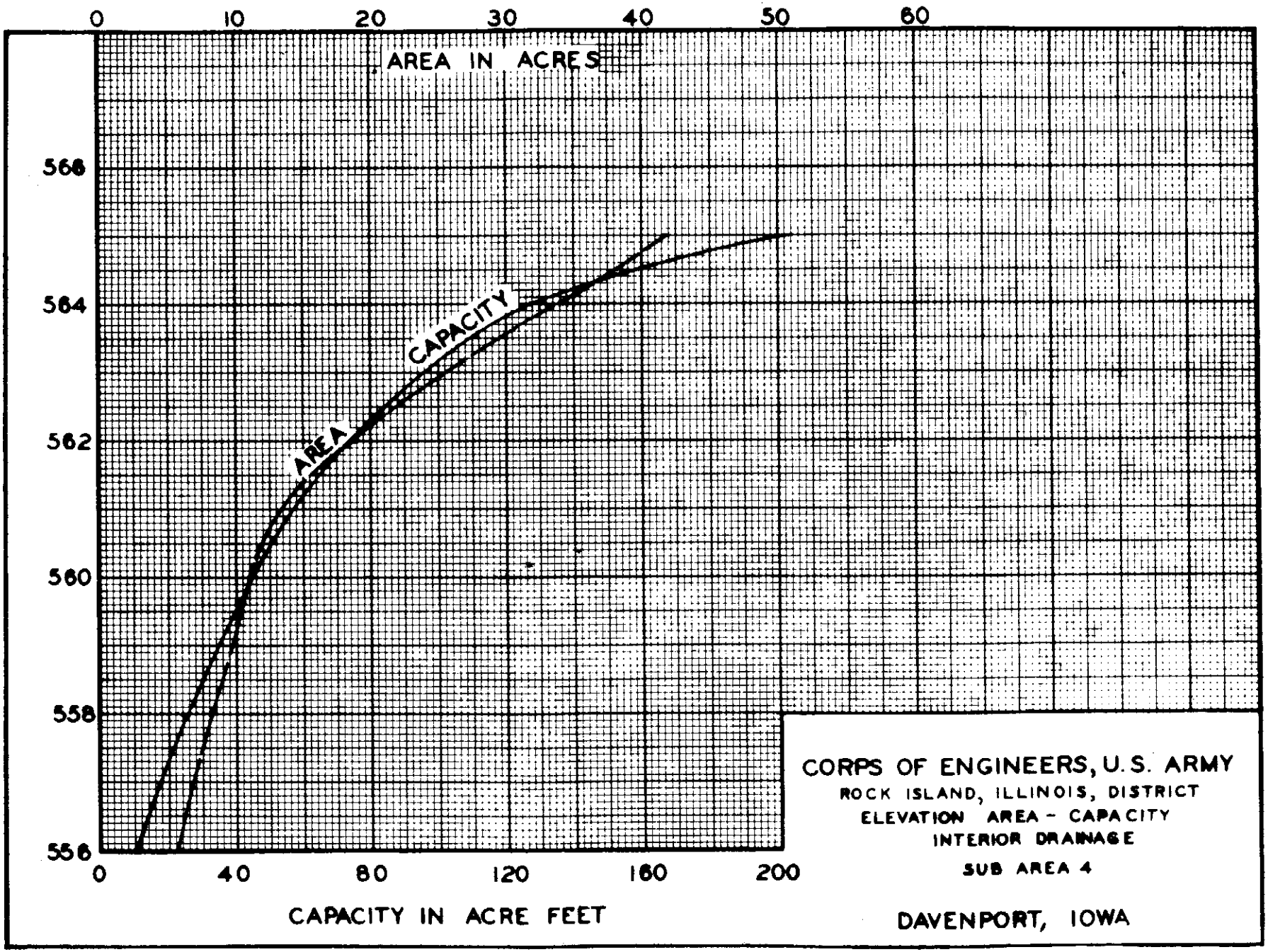

0
10
20
30
40
50
60
AREA IN ACRES
568
564
562
560
558
556
CAPACITY
AREA
0
40
80
120
160
200
CAPACITY IN ACRE FEET
CORPS OF ENGINEERS, U.S. ARMY
ROCK ISLAND, ILLINOIS, DISTRICT
ELEVATION AREA - CAPACITY
INTERIOR DRAINAGE
SUB AREA 4
DAVENPORT, IOWA

PLATE A-8.2

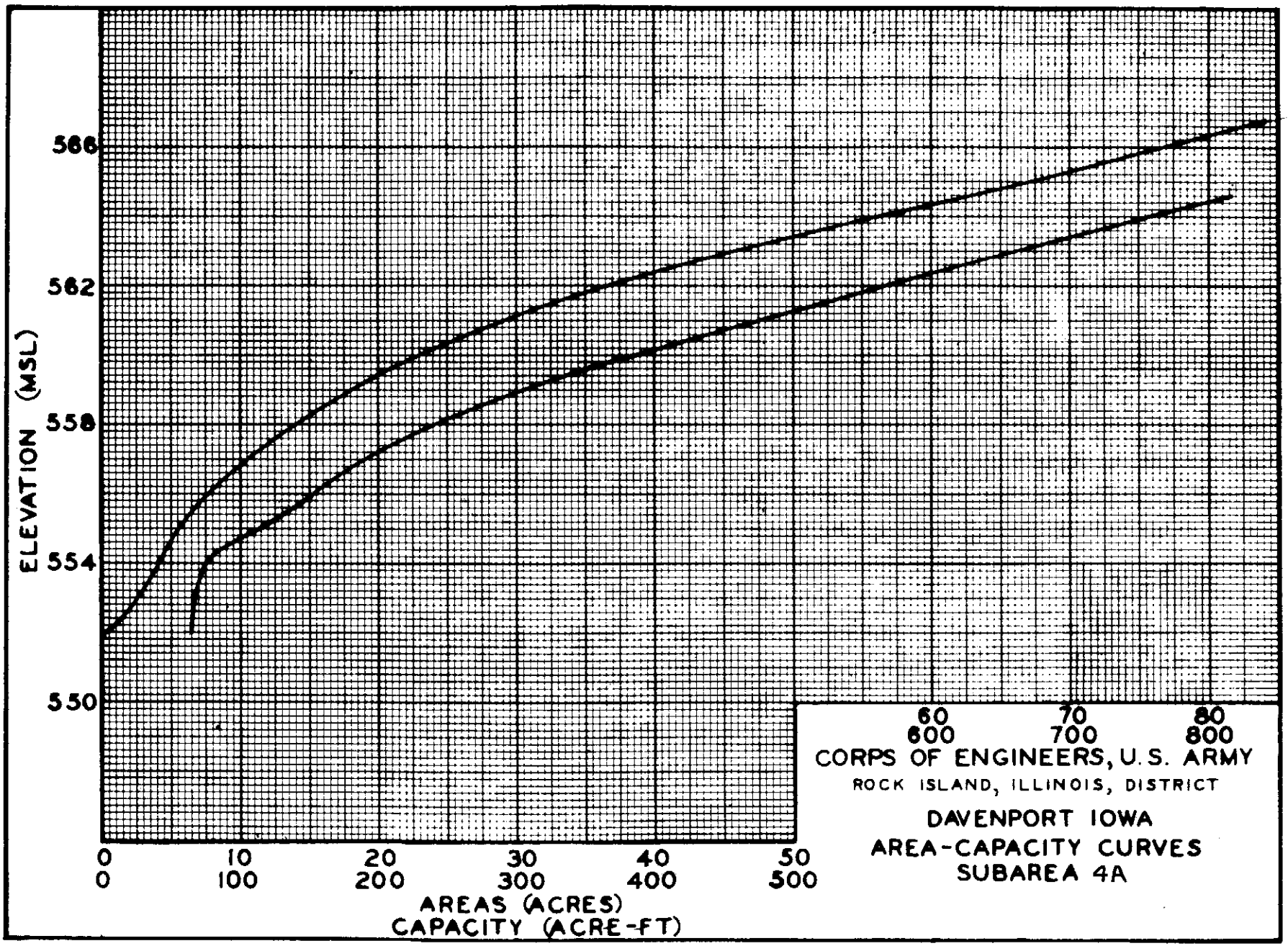

ELEVATION (MSL)
566
562
558
554
550
0
0
10
100
20
200
30
300
40
400
50
500
AREAS (ACRES)
CAPACITY (ACRE-FT)
60
600
70
700
80
800
CORPS OF ENGINEERS, U.S. ARMY
ROCK ISLAND, ILLINOIS, DISTRICT
DAVENPORT IOWA
AREA-CAPACITY CURVES
SUBAREA 4A

PLATE A-8.3

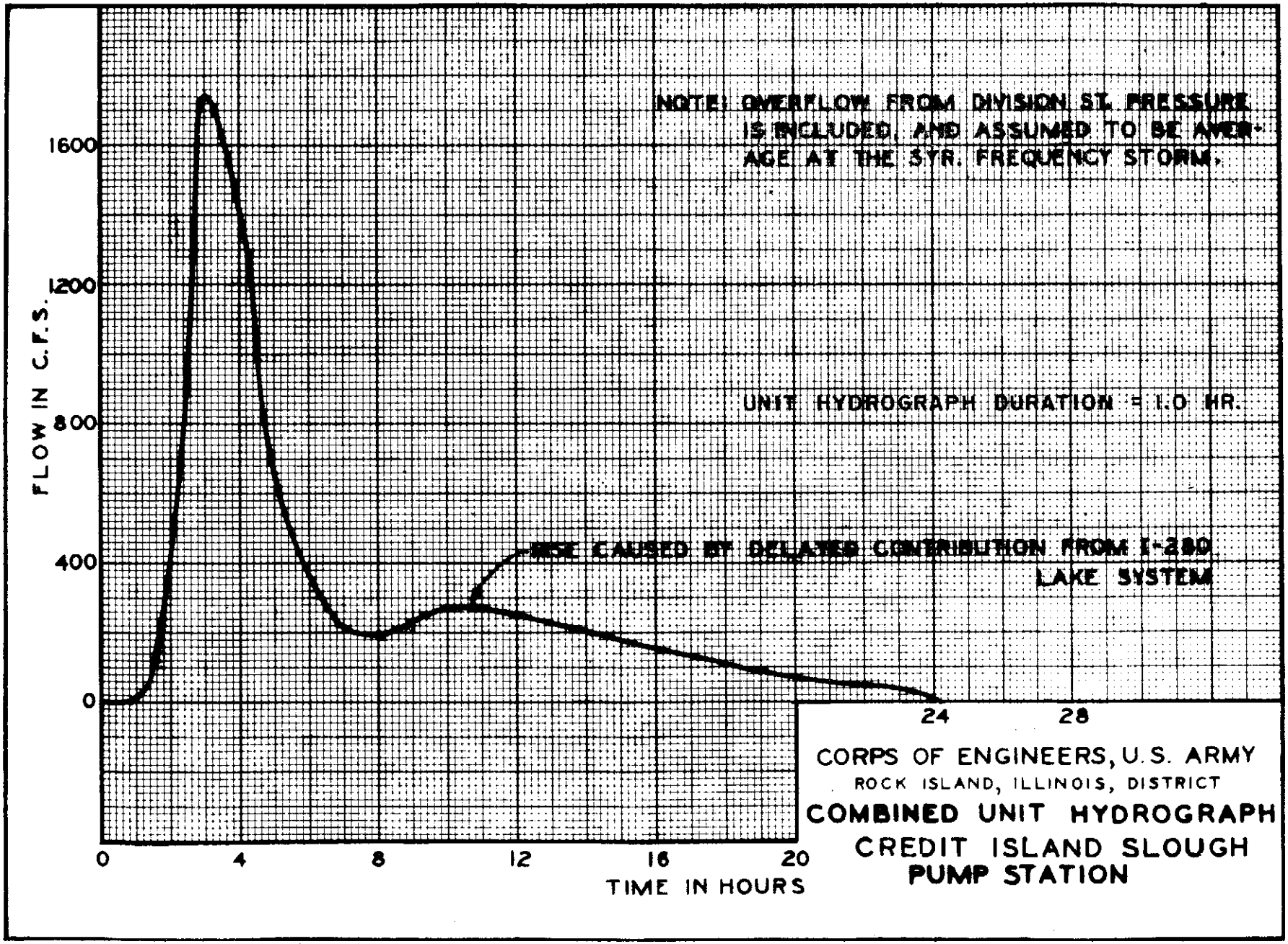

NOTE: OVERFLOW FROM DIVISION ST. PRESSURE IS INCLUDED, AND ASSUMED TO BE AVERAGE AT THE 5YR. FREQUENCY STORM.
UNIT HYDROGRAPH DURATION = 1.0 HR.
RISE CAUSED BY DELAYED CONTRIBUTION FROM I-280 LAKE SYSTEM
FLOW IN C.F.S.
1600
1200
800
400
0
0
4
8
12
16
20
24
28
TIME IN HOURS
CORPS OF ENGINEERS, U.S. ARMY
ROCK ISLAND, ILLINOIS, DISTRICT
COMBINED UNIT HYDROGRAPH
CREDIT ISLAND SLOUGH
PUMP STATION

PLATE A-8.4

Combined Runoff (in c.f.s.) Hydrographs
Credit Island Pump Station

Time in 60-Min Period	Frequency in Years							
0	1	2	5	10	25	50	100	SPF
1	1	1	1	1	1	1	1	1
2	23	25	26	32	42	37	65	142
3	129	132	139	147	208	205	329	717
4	279	289	299	311	388	450	623	1,411
5	451	461	512	540	588	784	915	2,055
6	1,176	590	697	699	947	1,144	1,371	2,989
7	2,571	1,492	1,921	2,162	2,461	2,737	3,211	7,000
8	1,954	3,106	3,958	4,481	5,153	5,714	6,415	13,985
9	955	2,345	2,974	3,386	3,909	4,326	4,819	10,500
10	539	1,154	1,462	1,668	1,926	2,135	2,395	5,221
11	368	654	830	954	1,099	1,224	1,388	3,025
12	351	455	571	662	757	851	977	2,130
13	410	436	549	636	839	815	826	1,810
14	462	506	640	736	941	942	1,081	2,360
15	447	567	717	823	909	1,053	1,199	2,613
16	408	547	693	794	827	1,015	1,154	2,520
17	365	498	630	722	739	924	1,049	2,280
18	323	454	593	645	654	935	986	2,149
19	284	394	499	541	573	730	828	1,810
20	249	346	428	501	502	640	725	1,581
21	214	460	384	439	431	560	634	1,380
22	172	209	265	376	347	480	542	1,175
23	138	168	213	302	347	387	437	953
24	112	168	172	244	229	311	351	765
25	92	136	141	197	226	252	284	620
26	75	111	115	161	184	205	231	504
27	59	90	91	131	150	167	187	407
28	43	71	66	103	118	132	147	320
29	28	52	42	75	86	95	106	231
30	14	33	21	48	55	31	68	146
31	1	16	10	24	27	30	34	74
32		8	3	11	13	15	16	34
33		2	2	3	3	4	4	9
34		1	2	3	3	4	4	8
35			1	2	2	3	3	7
36				1	2	2	3	6
37					1	2	2	5
38						1	2	4
39							1	3
40								2
41								1
42								
43								
44								
45								
46								
47								
48								
49								
50								

NO. 340-20 DIETZGEN GRAPH PAPER
20 X 20 PER INCH

DIETZGEN CORPORATION
MADE IN U.S.A.

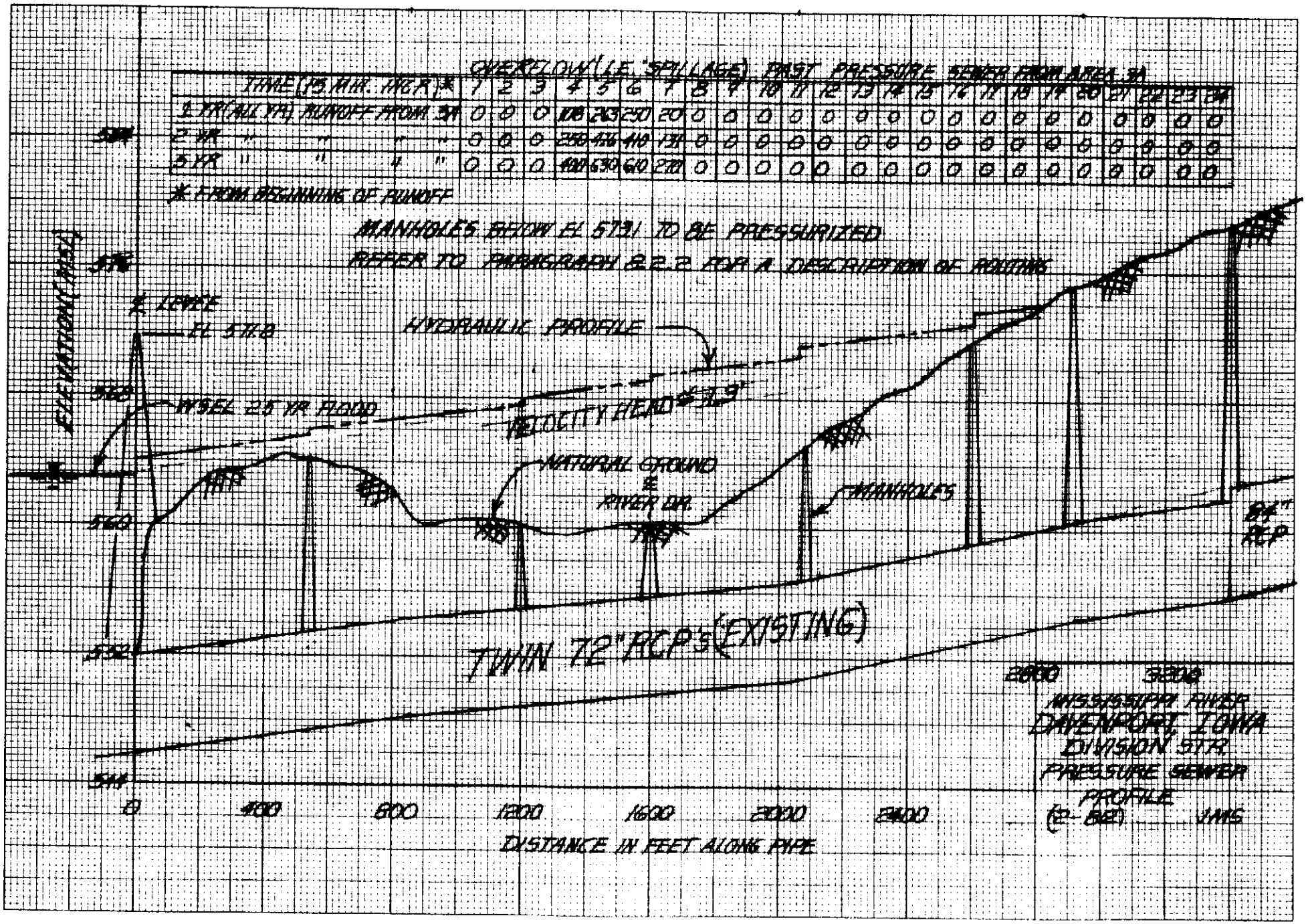


PLATE A-842

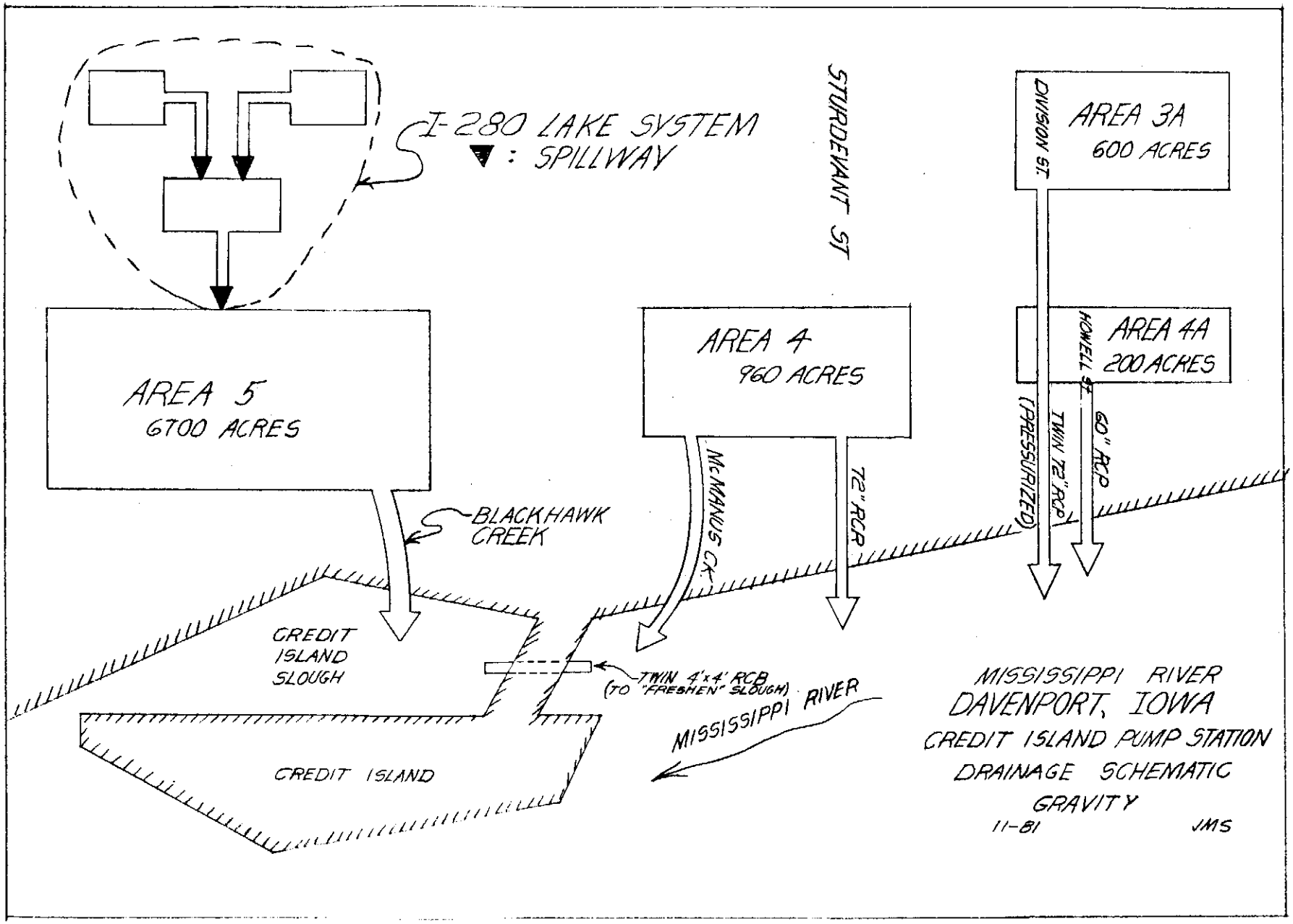

I-280 LAKE SYSTEM
▼ : SPILLWAY
STURDEVANT ST
DIVISION ST.
AREA 3A
600 ACRES
AREA 5
6700 ACRES
AREA 4
960 ACRES
HOWELL ST
AREA 4A
200 ACRES
(PRESSURIZED)
TWIN 72" RCP
60" RCP
72" RCP
McMANUS CK
BLACKHAWK CREEK
CREDIT ISLAND SLOUGH
TWIN 4'x4' RCB (TO "FRESHMEN" SLOUGH)
MISSISSIPPI RIVER
CREDIT ISLAND
MISSISSIPPI RIVER
DAVENPORT, IOWA
CREDIT ISLAND PUMP STATION
DRAINAGE SCHEMATIC
GRAVITY
11-81
JMS

PLATE A-8.5

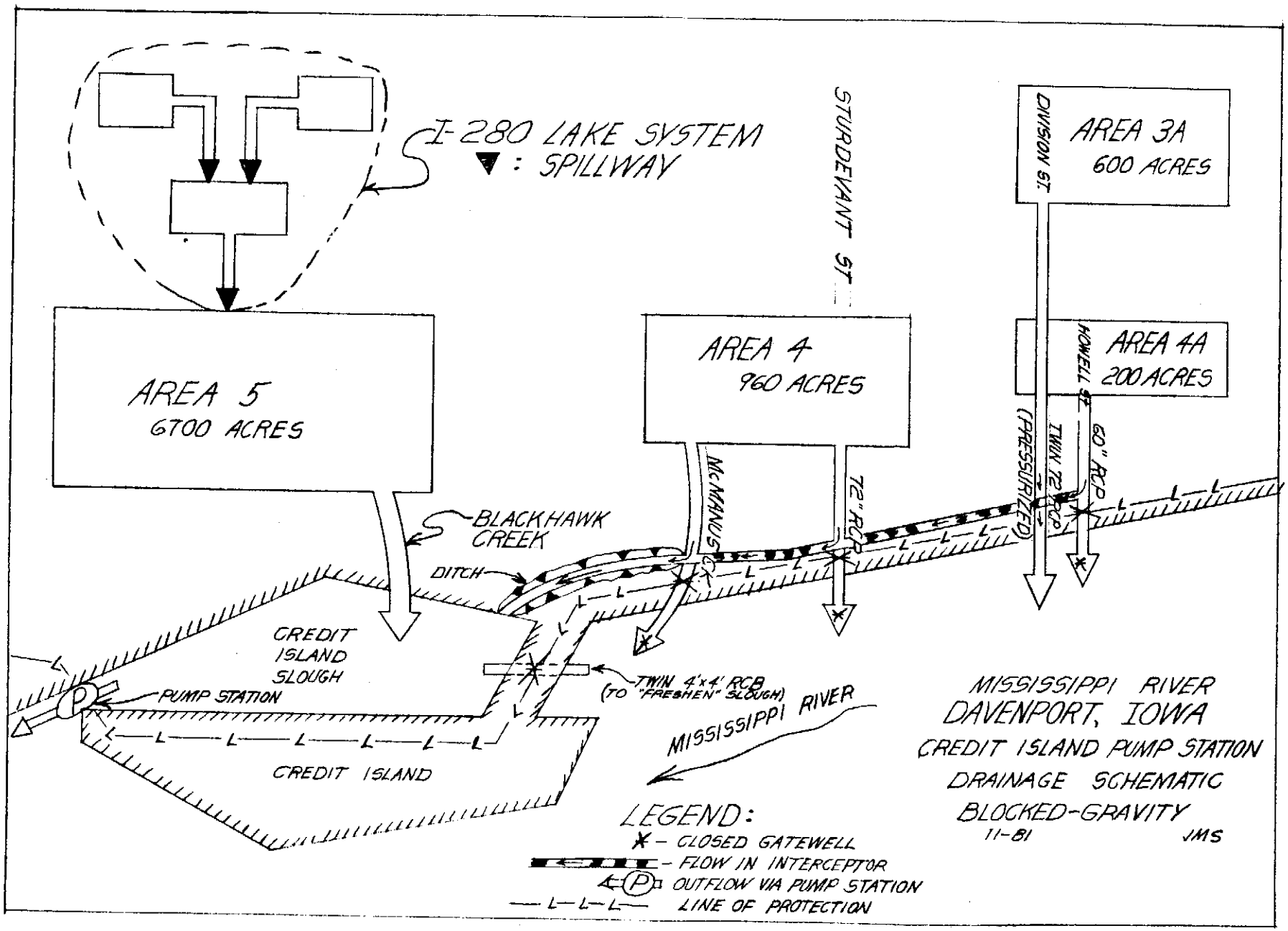

I-280 LAKE SYSTEM
▼ : SPILLWAY
STURDEVANT ST.
DIVISION ST.
AREA 3A
600 ACRES
AREA 4A
200 ACRES
HOWELL ST.
AREA 4
960 ACRES
AREA 5
6700 ACRES
BLACKHAWK CREEK
DITCH
McMANUS
72" RCP
(PRESSURIZED)
TWIN 72" RCP
60" RCP
CREDIT ISLAND SLOUGH
PUMP STATION
TWIN 4'x4' RCB
(TO "FRESHEN" SLOUGH)
MISSISSIPPI RIVER
CREDIT ISLAND
MISSISSIPPI RIVER
DAVENPORT, IOWA
CREDIT ISLAND PUMP STATION
DRAINAGE SCHEMATIC
BLOCKED-GRAVITY
11-81
JMS
LEGEND:
✱ - CLOSED GATEWELL
- FLOW IN INTERCEPTOR
(P) OUTFLOW VIA PUMP STATION
L-L-L LINE OF PROTECTION

PLATE A-86

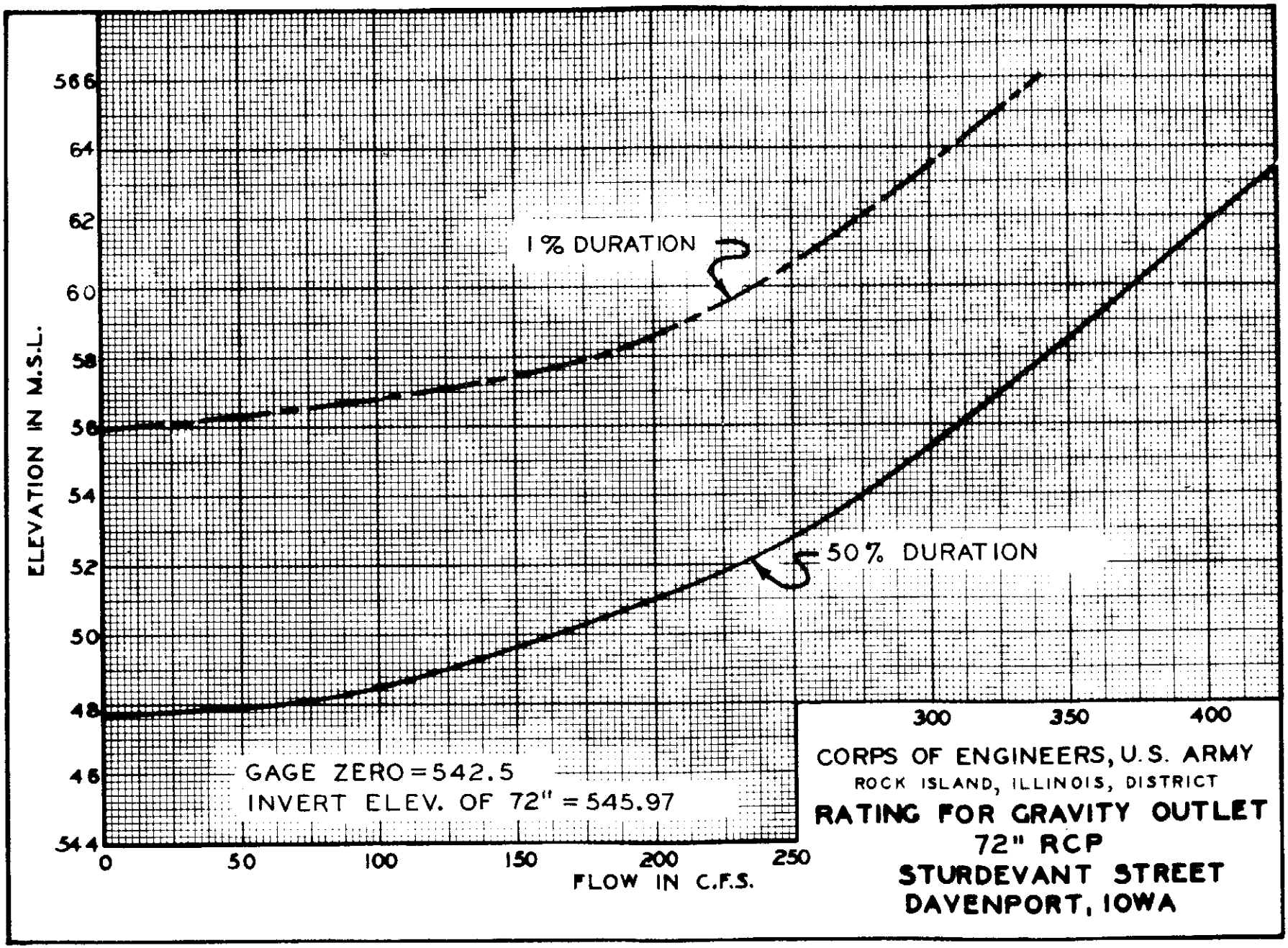

ELEVATION IN M.S.L.
566
64
62
60
58
56
54
52
50
48
46
544
1% DURATION
50% DURATION
GAGE ZERO=542.5
INVERT ELEV. OF 72" = 545.97
0
50
100
150
200
250
300
350
400
FLOW IN C.F.S.
CORPS OF ENGINEERS, U.S. ARMY
ROCK ISLAND, ILLINOIS, DISTRICT
RATING FOR GRAVITY OUTLET
72" RCP
STURDEVANT STREET
DAVENPORT, IOWA

PLATE A-8.7

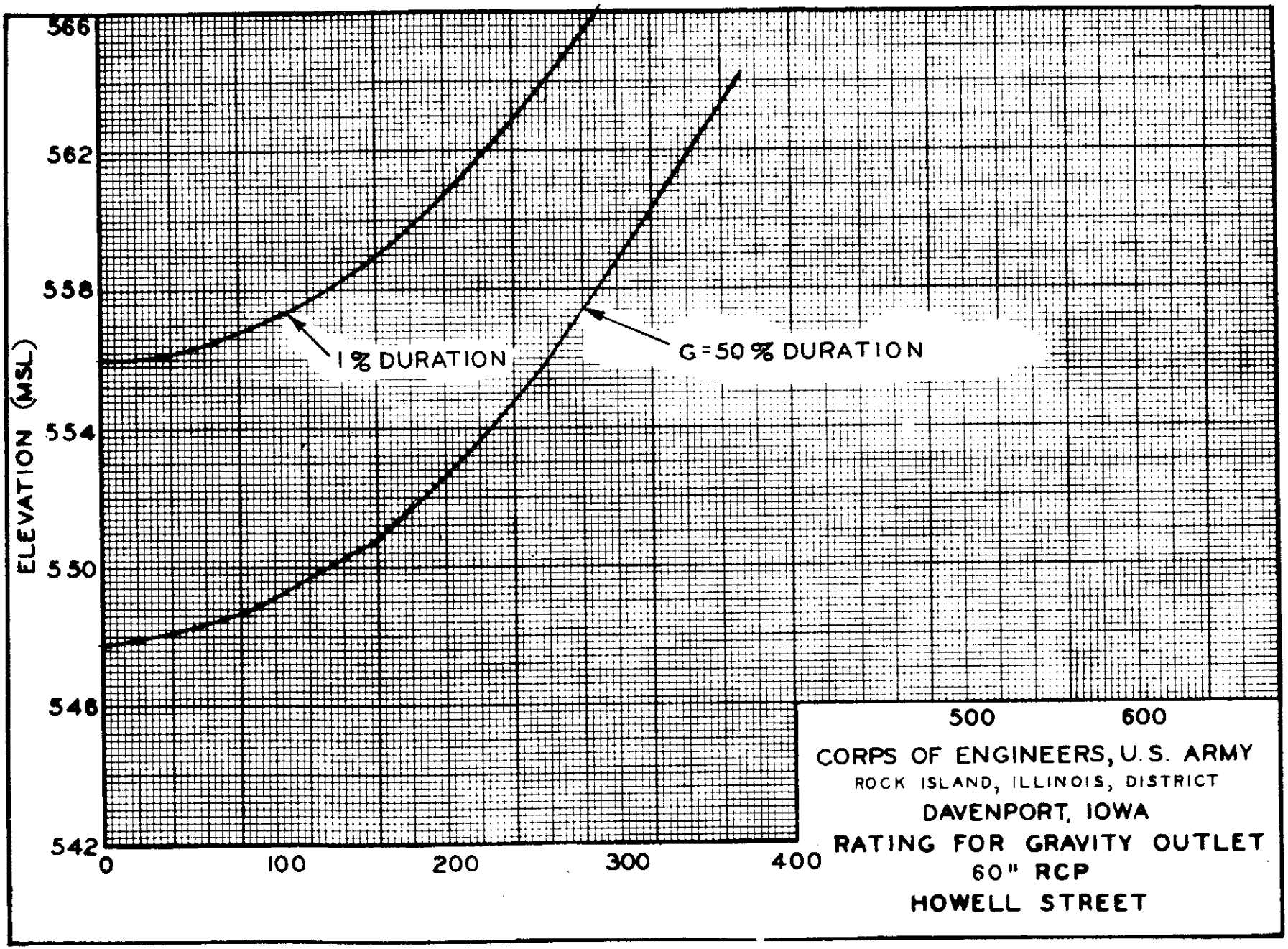
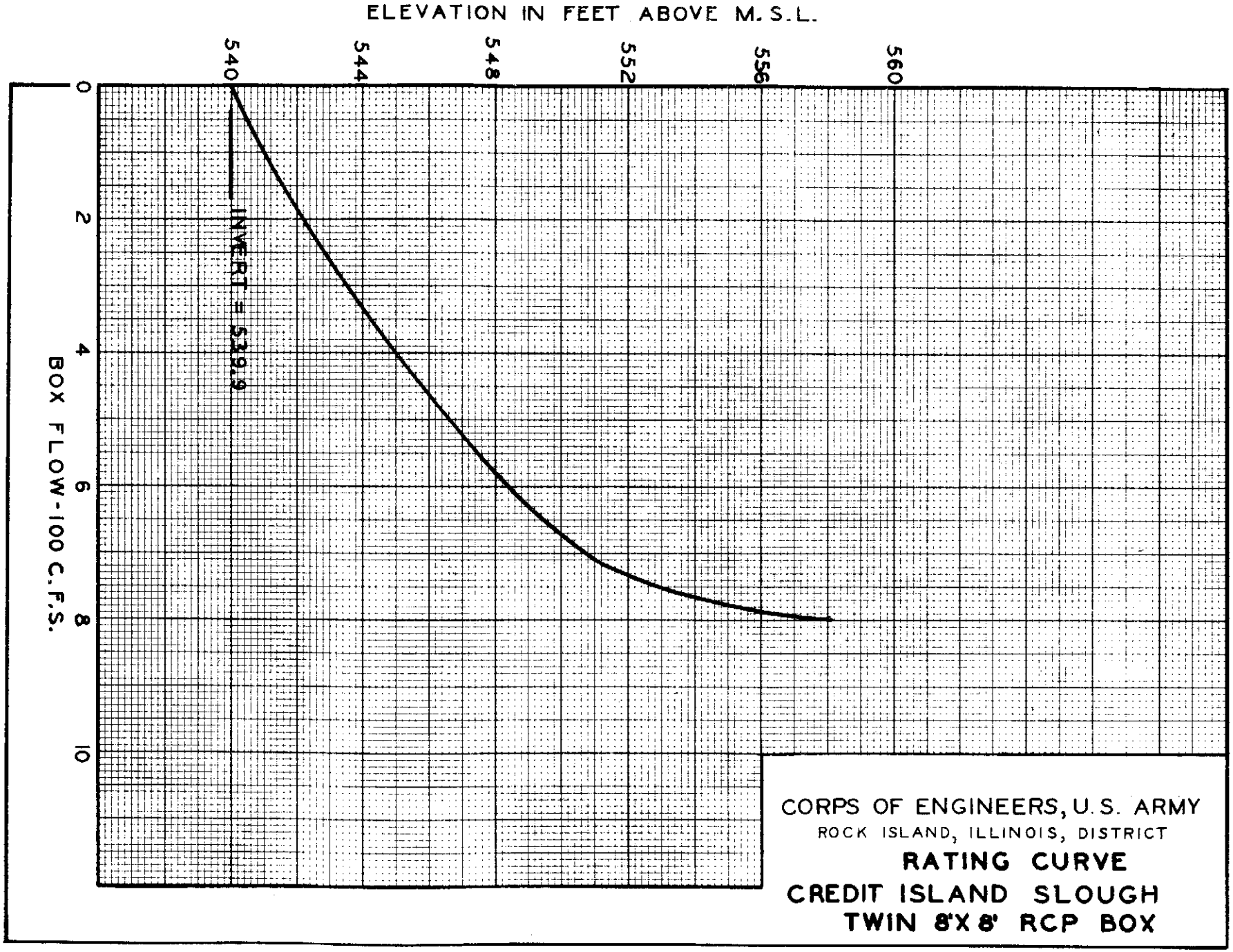

566
562
558
554
550
546
542
ELEVATION (MSL)
0
100
200
300
400
500
600
1% DURATION
G=50% DURATION
CORPS OF ENGINEERS, U.S. ARMY
ROCK ISLAND, ILLINOIS, DISTRICT
DAVENPORT, IOWA
RATING FOR GRAVITY OUTLET
60" RCP
HOWELL STREET

PLATE A-8.8



CORPS OF ENGINEERS, U.S. ARMY
ROCK ISLAND, ILLINOIS, DISTRICT

RATING CURVE
CREDIT ISLAND SLOUGH
TWIN 8'X 8' RCP BOX

PLATE A-8.9

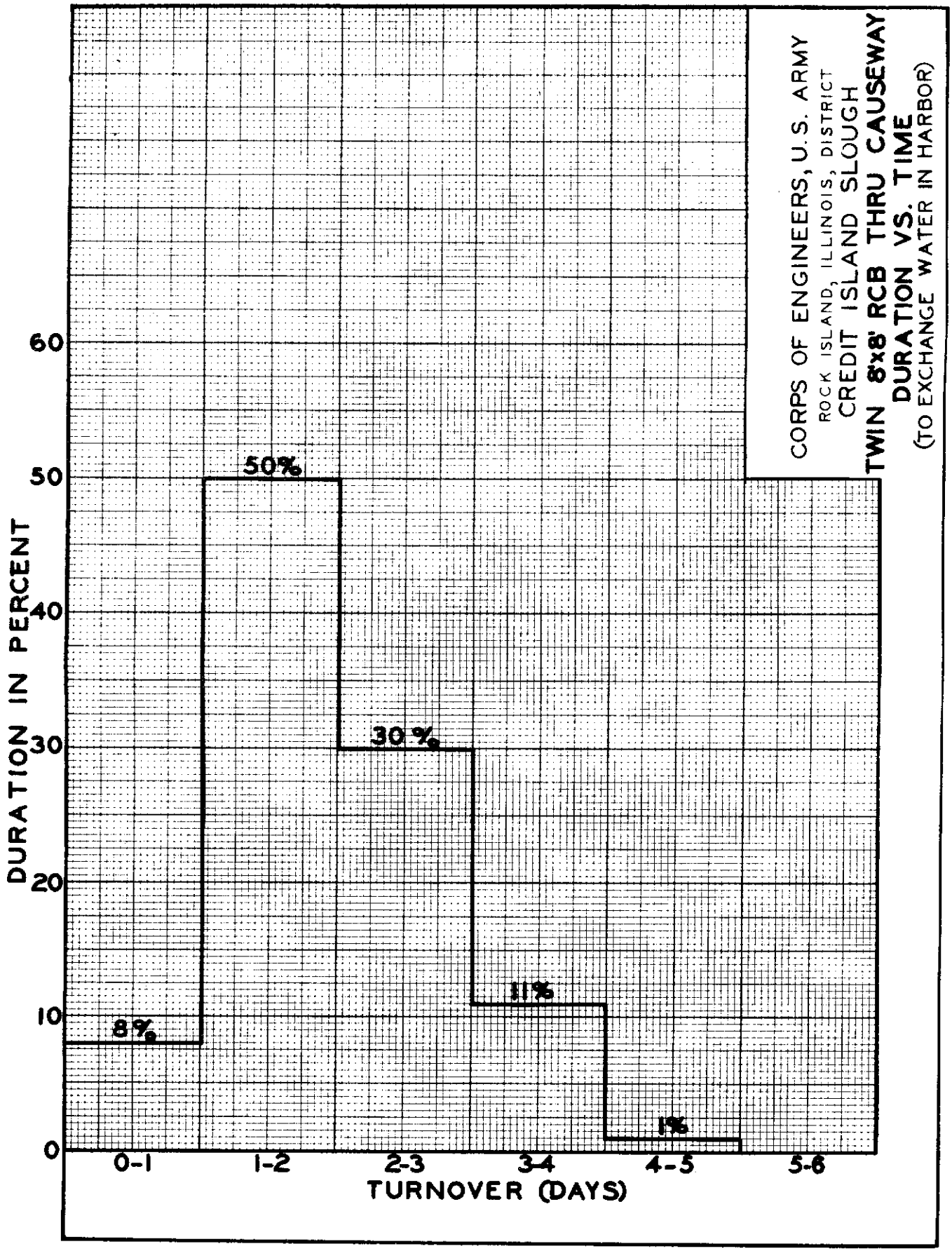

CORPS OF ENGINEERS, U.S. ARMY
ROCK ISLAND, ILLINOIS, DISTRICT
CREDIT ISLAND SLOUGH
TWIN 8'x8' RCB THRU CAUSEWAY
DURATION VS. TIME
(TO EXCHANGE WATER IN HARBOR)
DURATION IN PERCENT
0
10
20
30
40
50
60
50%
30%
11%
8%
1%
0-1
1-2
2-3
3-4
4-5
5-6
TURNOVER (DAYS)

PLATE A-8.10

NO. 340-20 DIETZGEN GRAPH PAPER
20 X 20 PER INCH

DIETZGEN CORPORATION
MADE IN U.S.A.

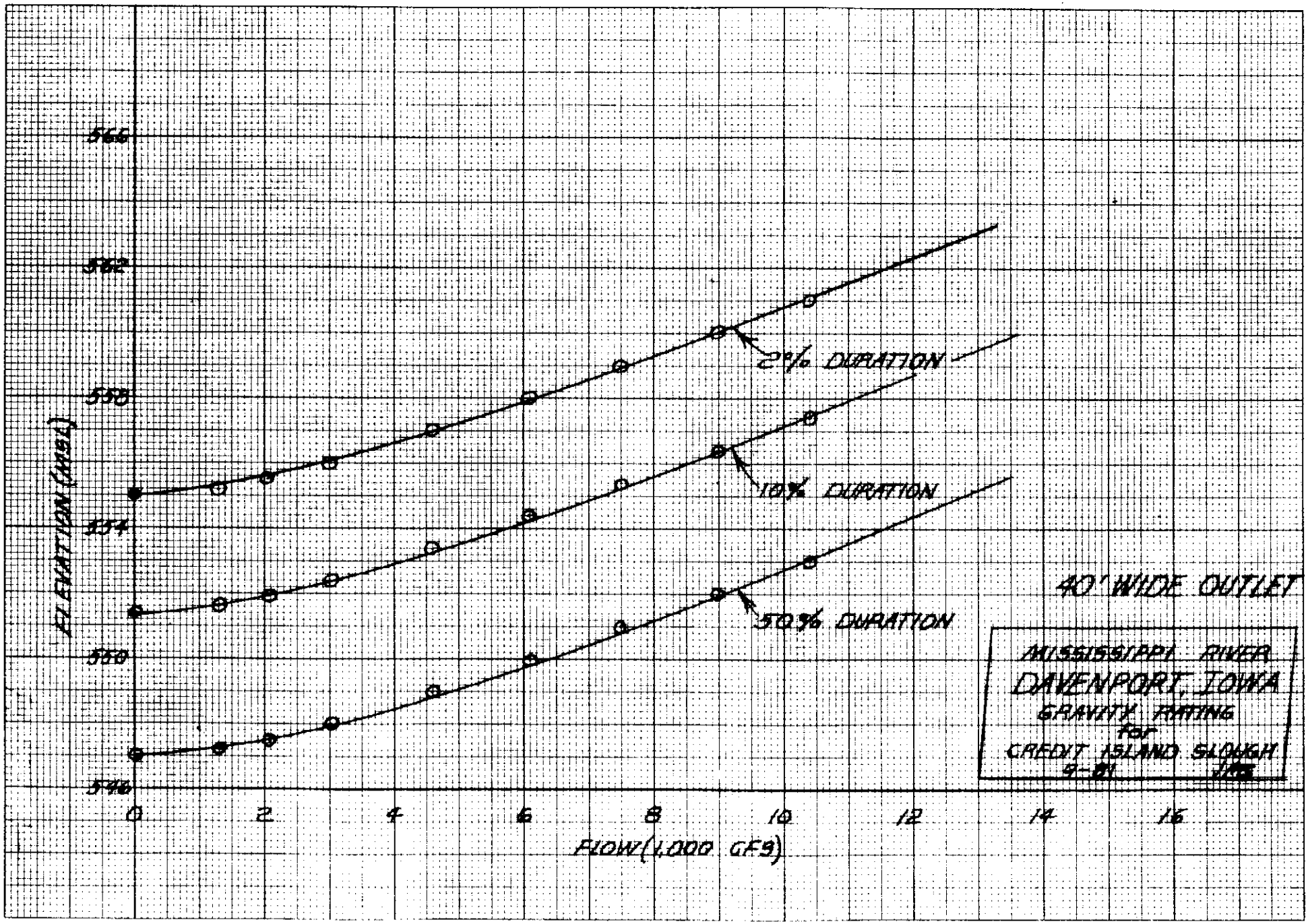


PLATE A-8.11

NO. 340-20 DIETZGEN GRAPH PAPER
20 X 20 PER INCH

DIETZGEN CORPORATION
MADE IN U. S. A.

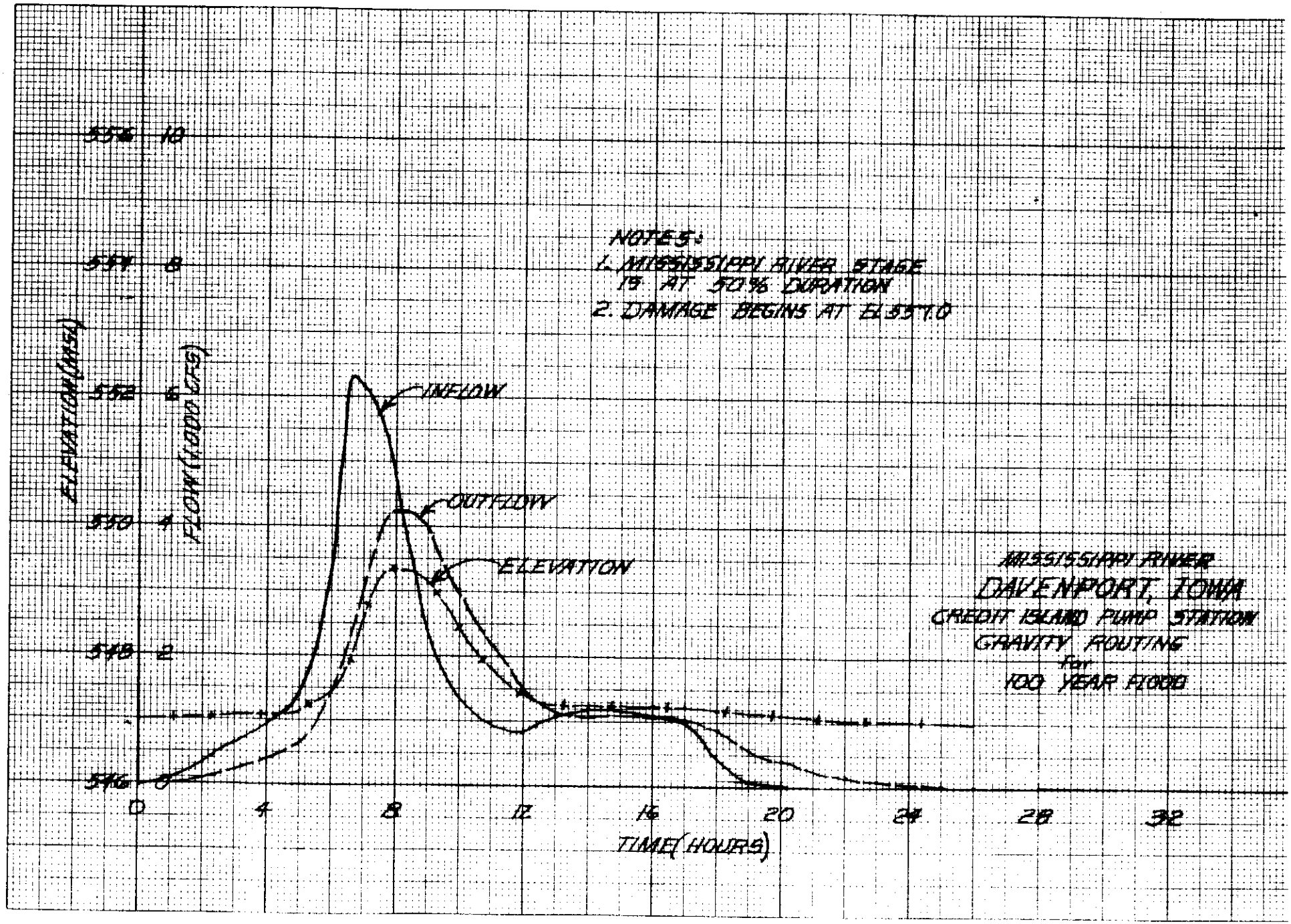


PLATE A-8.11.1

NO. 341120 DIETZGEN GRAPH PAPER
20 X 20 PER INCH

DIETZGEN CORPORATION
MADE IN U.S.A.

ELEVATION (MSL)	DAMAGE ($1,000)
556.0	0
557.0	47.0
558.0	112.7
559.0	227.5
560.0	400.6
561.0	748.2
562.0	1,425.0
563.0	2,280.3
564.0	3,332.2
565.0	4,973.0
566.0	6,200.6
567.0	8,611.1
568.0	12,098.0

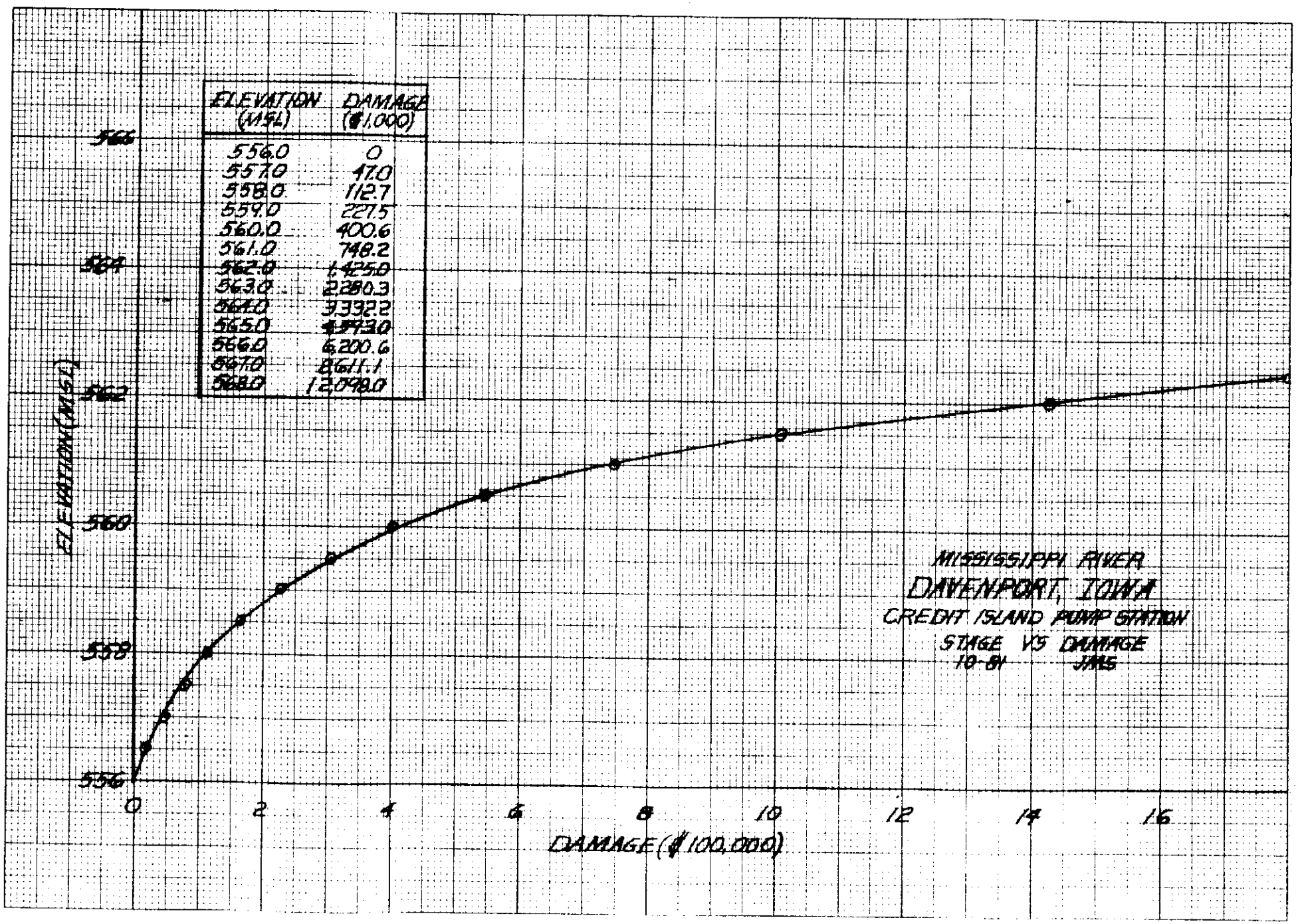


PLATE A-8.12

NO. 340-20 DIETZGEN GRAPH PAPER
20 X 20 PER INCH

DIETZGEN CORPORATION
MADE IN U.S.A.

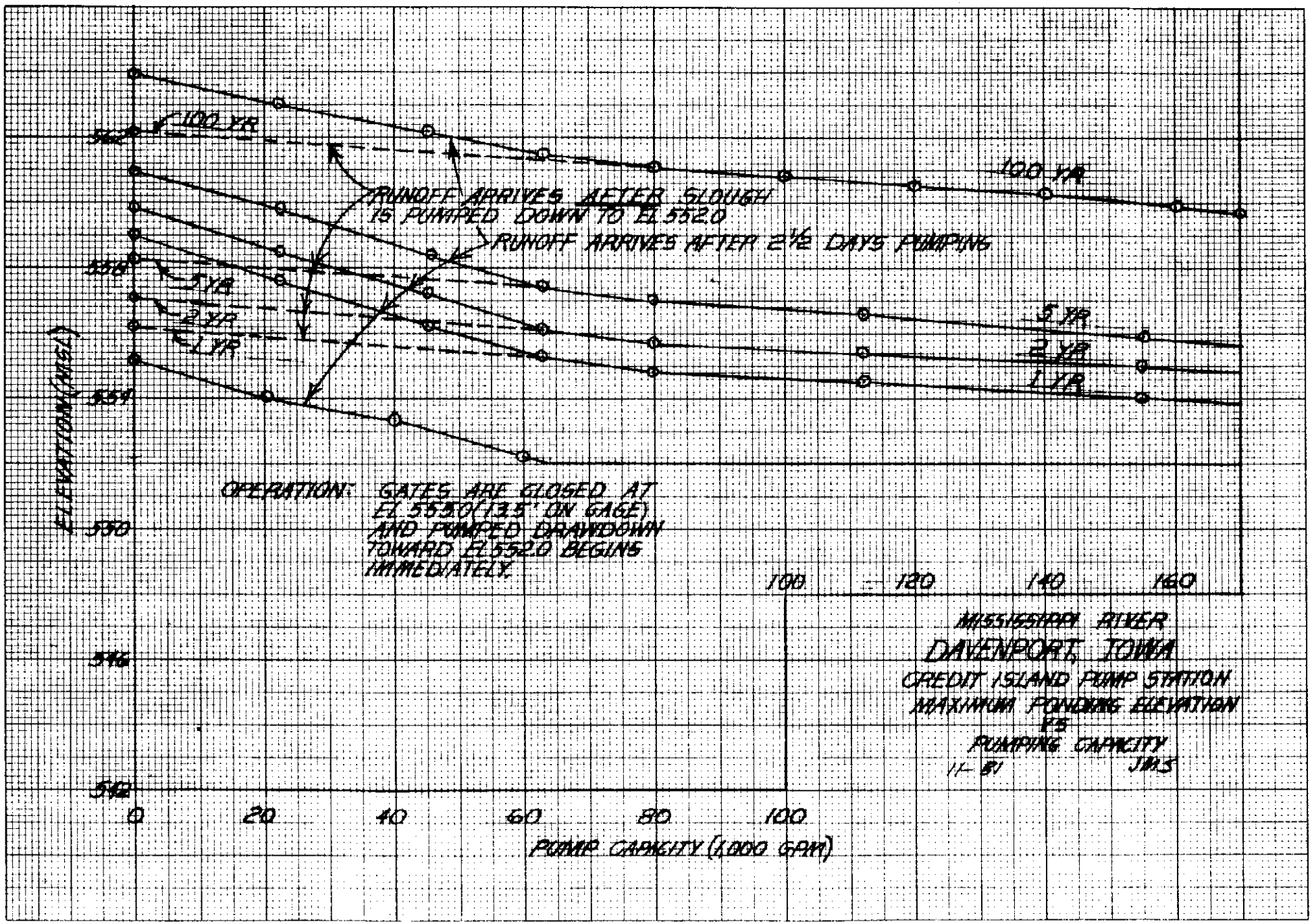


PLATE A-8.13

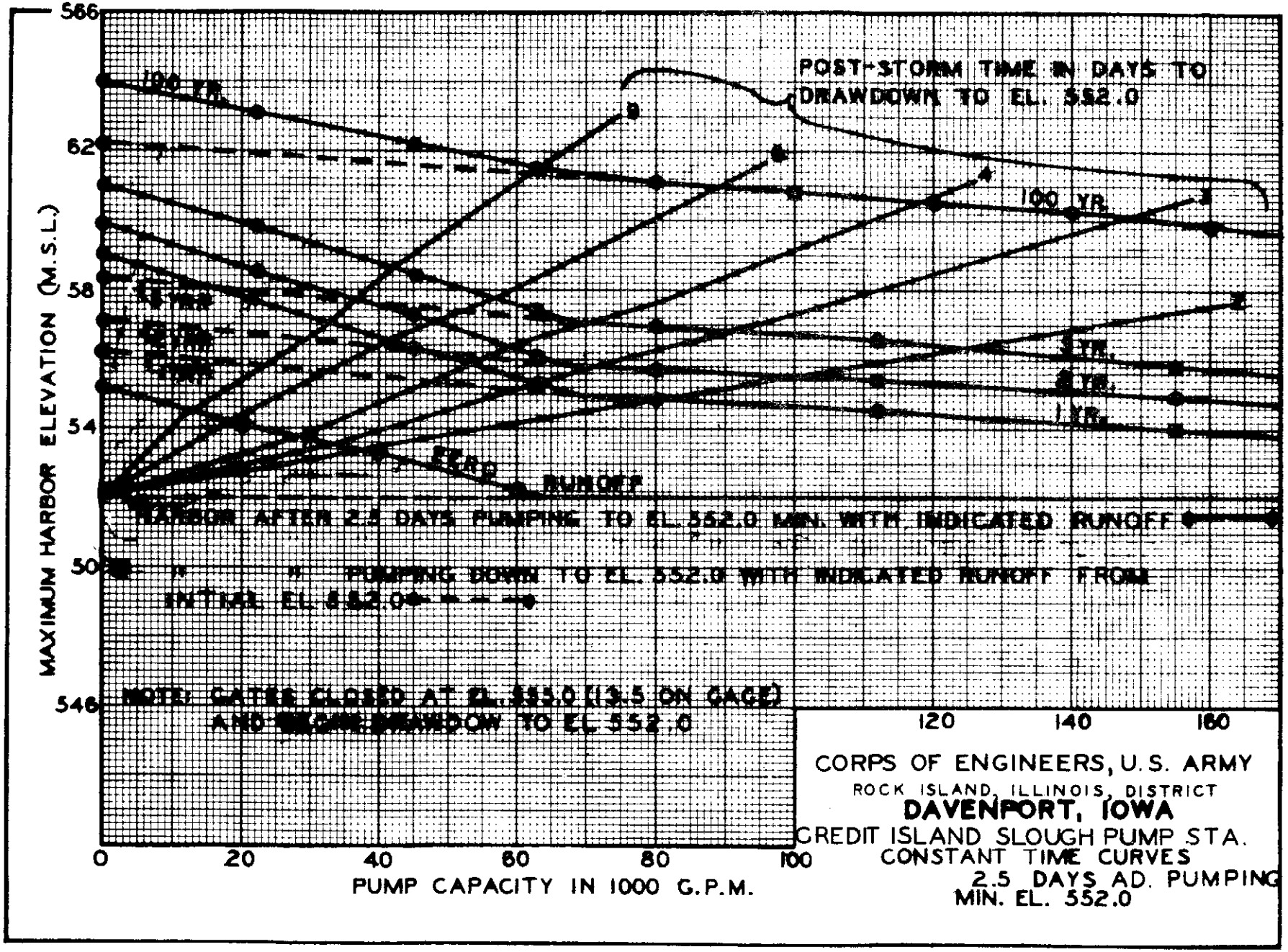

MAXIMUM HARBOR ELEVATION (M.S.L.)
566
62
58
54
50
546
0
20
40
60
80
100
120
140
160
PUMP CAPACITY IN 1000 G.P.M.
POST-STORM TIME IN DAYS TO DRAWDOWN TO EL. 552.0
100 YR.
100 YR.
5 YR.
2 YR.
1 YR.
ZERO RUNOFF
HARBOR AFTER 2.5 DAYS PUMPING TO EL. 552.0 MIN. WITH INDICATED RUNOFF
PUMPING DOWN TO EL. 552.0 WITH INDICATED RUNOFF FROM INITIAL EL. 552.0
NOTE: GATES CLOSED AT EL. 553.0 (13.5 ON GAGE) AND [illegible] DRAWDOWN TO EL. 552.0
CORPS OF ENGINEERS, U.S. ARMY
ROCK ISLAND, ILLINOIS, DISTRICT
DAVENPORT, IOWA
CREDIT ISLAND SLOUGH PUMP STA.
CONSTANT TIME CURVES
2.5 DAYS AD. PUMPING
MIN. EL. 552.0

PLATE A-8.14

FINISH STAGE EVENTS I = 0.6, Q0 = 560 AF 4-27-81

NO. 340-L310 DIETZGEN GRAPH PAPER
SEMI-LOGARITHMIC
3 CYCLES X 10 DIVISIONS PER INCH

DIETZGEN CORPORATION
MADE IN U.S.A.

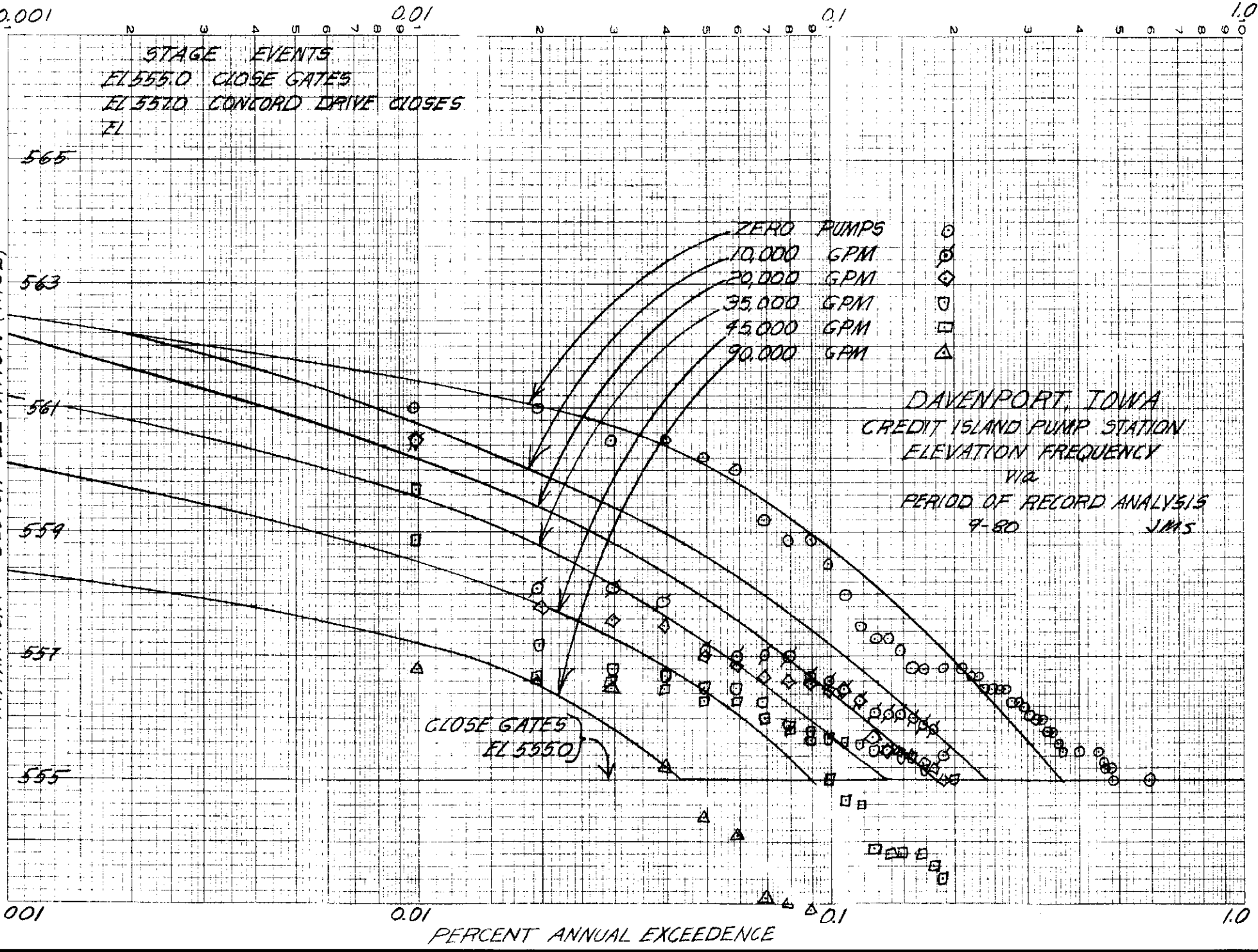


PLATE A-8.15

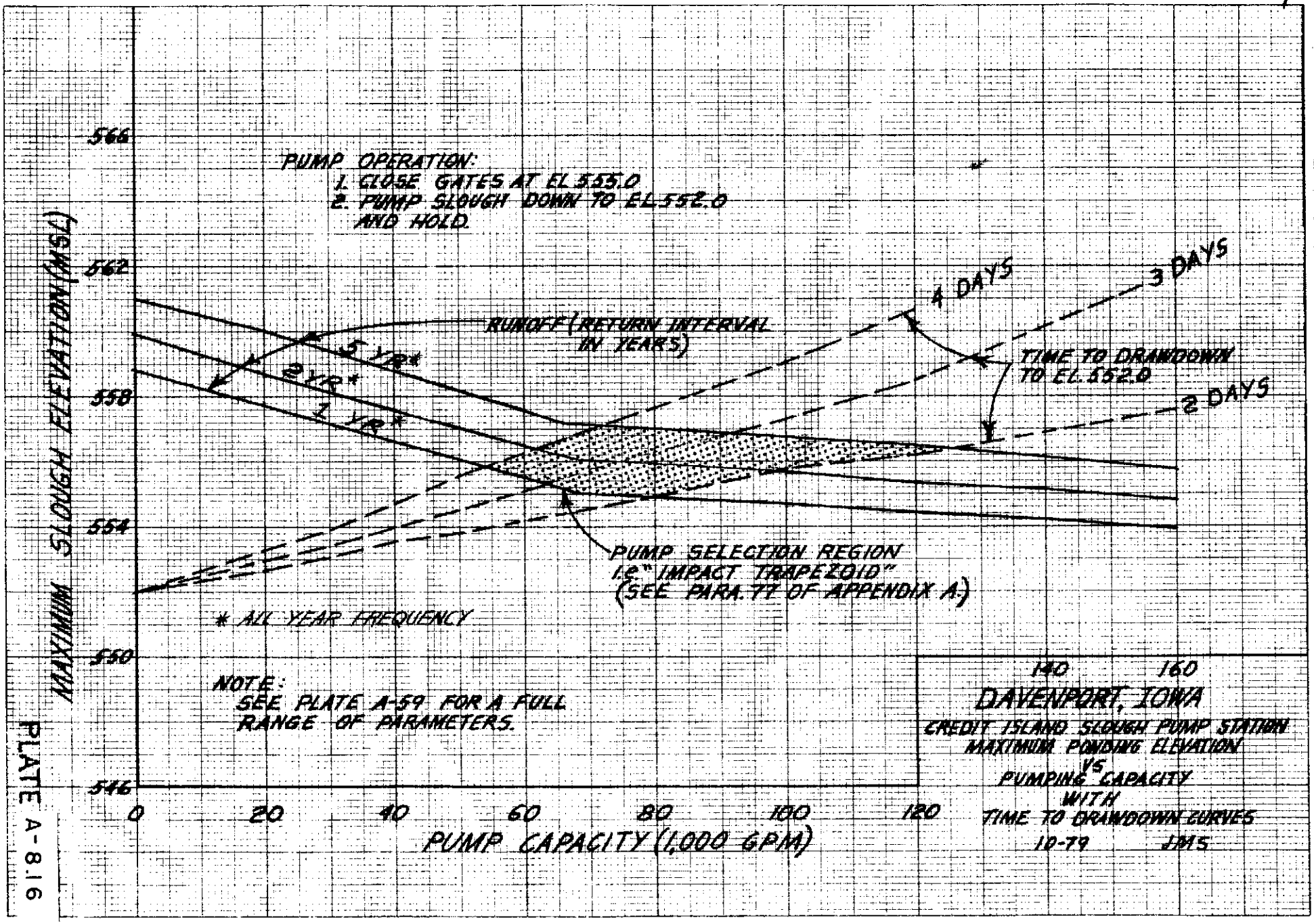

PUMP OPERATION:
1. CLOSE GATES AT EL 555.0
2. PUMP SLOUGH DOWN TO EL 552.0 AND HOLD.
RUNOFF (RETURN INTERVAL IN YEARS)
5 YR*
2 YR*
1 YR*
4 DAYS
3 DAYS
2 DAYS
TIME TO DRAWDOWN TO EL 552.0
PUMP SELECTION REGION
I.E." IMPACT TRAPEZOID"
(SEE PARA. 77 OF APPENDIX A.)
* ALL YEAR FREQUENCY
NOTE:
SEE PLATE A-59 FOR A FULL RANGE OF PARAMETERS.
MAXIMUM SLOUGH ELEVATION (MSL)
566
562
558
554
550
546
0
20
40
60
80
100
120
140
160
PUMP CAPACITY (1,000 GPM)
DAVENPORT, IOWA
CREDIT ISLAND SLOUGH PUMP STATION
MAXIMUM PONDING ELEVATION
VS
PUMPING CAPACITY
WITH
TIME TO DRAWDOWN CURVES
10-79 JD15

PLATE A-8.16

Schedule No	Conditions that Determine Operations: River Stage in ft m.s.l. at Davenport Gage		Operation of Credit Island Slough Pump Station		Key Elevations		
	Prevailing on Day of Operation	Highest Forecast	Gravity Outlet	Pumping Operation	Lowest Gate Closing River Stage	Sump Level for Pump Activation	Sump Level for Pump Cut-off
1	2	3	4	5	6	7	8
1	Below 555.0	Below 555.0	Fully Open	None	555.0	---	---
2	Above 555.0	Below 557.0	Closed except when harbor exceeds river by 0.2 ft.	Pump as long as gates are closed	555.0	555.0	555.0
3	Above 555.0	Above 557.0	Closed except when harbor exceeds rising river by 0.4 ft. or falling river by 0.2 ft.	Pump to draw harbor down to El. 552.0. Operate till cut-off is reached	---	555.0	552.0
4	Receding River, 555.0 to 553.0	Falling below 553.0	Closed except when harbor exceeds river by 0.2 ft.	Cease pumping and allow harbor to rise until gates are open.	552.0	None	None

NOTE:
The sump levels correspond to the elevations at this station while river stages correspond to the gage at Lock and Dam No. 15 tailwater. Gage zero = El. 542.5. Station elevations are 1.0 below gage on profile.

PLATE A-8.17

PUMP STATION REVIEW

PROJECT: Davenport, Iowa STATION LOCATION: Credit Island Slough CLOSURE/PUMPDOWN: 555.0/552.0 DATE: June 1976

TEST		TANGIBLE EFFECTS			INTANGIBLE EFFECTS									
Event	Pump Capacity 1000 g.p.m.	Peak Stage m.s.l.	Time to Drawdown (Days) 2/	Damages $1,000	Item Reference Paragraph	Transportation Disruption	Transportation Time 3/ (Days)	Industrial/ Commercial Disruption	Industrial/ Commercial Time 3/ (Days)	Social Homes Affected	Social Disruption	Social Time 3/ (Days)	Aesthetic/ Environmental Degree	Health/ Safety Degree
200-Yr Flood 1/	0	563.2	39 ±	770.0 ±	31	Severe	10 +	Severe	10 ±	75-200	Mod.-Sev.	10-20	Moderate	Poss. Sev.
Selected Pump 1 Yr	70.0	555.0	2.5	0	31, 37, 39	None	0	None	0	0	- -	-	None	None
2 "	70.0	555.8	3	<1		Vry Little	1	None	0	0	- -	-	None	None
5 "	70.0	557.2	4	1.7		Little	2	Little	-	75	Light	2	Little	None
100 "	70.0	561.3	8	410.0		Severe	10	Mod.-Sev.	9	100	Moderate	4	Moderate	Moderate
20% Selected Pump 1 Yr	14.0	558.1	25	3.4	31, 37	Little	-	Little	-	75	Light	12	None	None
2 "	14.0	559.0	27	17.0		Moderate	-	Little	-	75	Moderate	13	None	None
5 "	14.0	560.3	34	200.0		Severe	4	Moderate	-	75-200	Mod.-Sev.	18	Little	Little-Mod.
100	14.0	563.4	39 ±	840.0		Severe	39 ±	Severe	39 ±	200 +	Severe	39 ±	Moderate	Mod.-Sev.
200% Selected Pump 1 Yr	140.0	554.2	<1	0	31, 37	None	0	None	-	0	- -	-	None	None
2 "	140.0	555.1	1	<1		None	0	None	-	0	- -	-	None	None
5 "	140.0	556.3	<2	<1		Little	-	None	-	10	Light	0	None	None
100 "	140.0	560.3	3	200		Little	<1	Little	<1	75	Light-Mod.	2	Little	Very Little

Appendix A

NOTES:

1/ 200-year flood event is the expected interior drainage experience during the project design flood with probable closure duration and "typical" runoff.

2/ Natural recession in case of zero pumping.

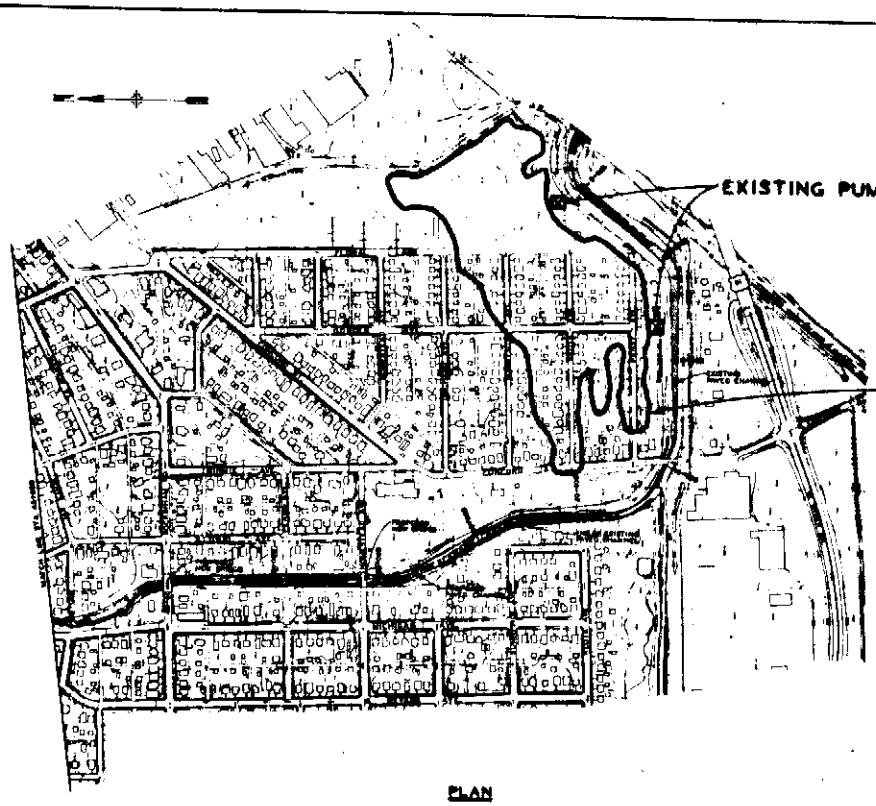
3/ Does not include cleanup time.

4/ Severity of indicated items is dependent upon duration.

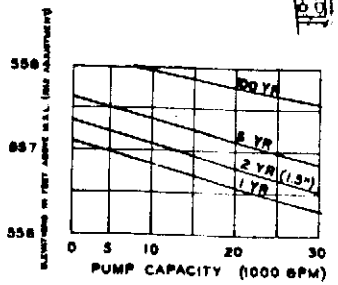
SEVERITY FACTORS		
ITEM	"C" ELEV.	FACTORS
Transportation	559.9	Importance and size route.
Industrial/ Commercial	560.5	Size facility and/or community importance.
Social	N/A 4/	Extent of disruption of people's lives.
Aesthetic/ Environmental	N/A 4/	To be coordinated with others.
Health/Safety	N/A 4/	To be coordinated with others.

ROCK ISLAND DISTRICT
MISSISSIPPI RIVER
Davenport, Iowa
CREDIT ISLAND
SLOUGH PUMP STATION REVIEW

PLATE A-8.18



PLAN



NOTE:

1. BLOCKED GRAVITY CONDITIONS OCCUR WHEN BLACKHAWK CREEK EXCEEDS EL. 555.0 BY EITHER HIGH STAGES OR BY BACK UP FROM THE CREDIT ISLAND SLOUGH.
2. EXISTING PUMPING TOTALS APPROX 6400 GPM.

CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
ROCK ISLAND, ILLINOIS

MISSISSIPPI RIVER
DAVENPORT, IOWA
LOCAL FLOOD PROTECTION
GARDEN ADDITION
INTERIOR DRAINAGE

PLATE A-9.1

PLATE A-9.1

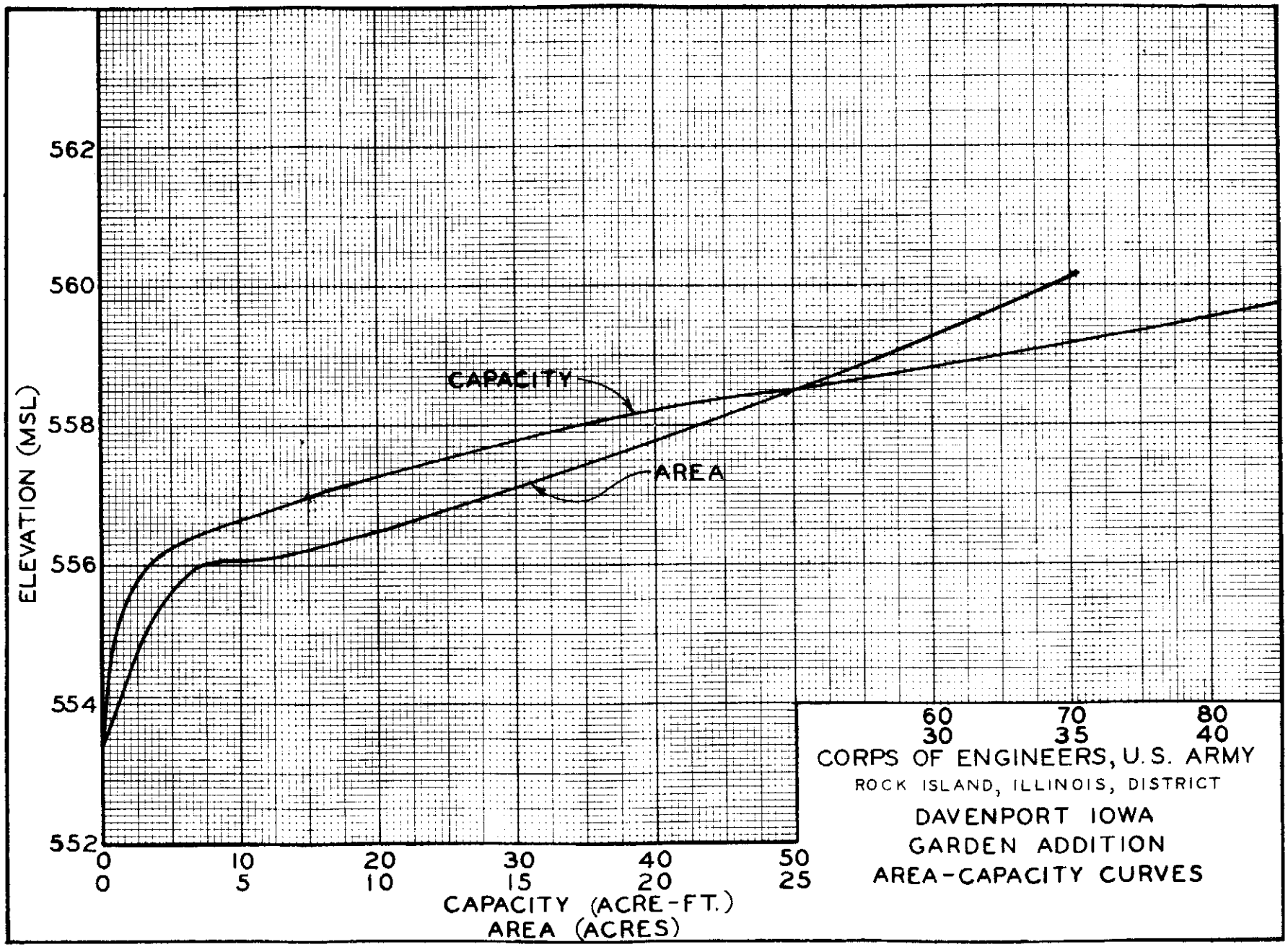

562
560
558
556
554
552
ELEVATION (MSL)
CAPACITY
AREA
0
0
10
5
20
10
30
15
40
20
50
25
CAPACITY (ACRE-FT.)
AREA (ACRES)
60
30
70
35
80
40
CORPS OF ENGINEERS, U.S. ARMY
ROCK ISLAND, ILLINOIS, DISTRICT
DAVENPORT IOWA
GARDEN ADDITION
AREA-CAPACITY CURVES

PLATE A-9.2

MISSISSIPPI RIVER
DAVENPORT, IOWA

LOCAL FLOOD PROTECTION
PHASE II
GENERAL DESIGN MEMORANDUM

APPENDIX B
GEOTECHNICAL DESIGN
(REVISED)

US ARMY ENGINEER DISTRICT, ROCK ISLAND
CORPS OF ENGINEERS
CLOCK TOWER BUILDING
ROCK ISLAND, ILLINOIS 61201

MISSISSIPPI RIVER
DAVENPORT, IOWA
LOCAL FLOOD PROTECTION

PHASE II
GENERAL DESIGN MEMORANDUM

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GEOTECHNICAL DESIGN

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Geology

1. Davenport lies at the downstream end of the reach of the Mississippi River known in former times as the Rock Island Rapids. Through the reach of the project area the riverbed is in a rock channel with Devonian limestone at the upper end and Pennsylvanian shales at the lower end. Rocks of the Mississippian system are absent in the area. Both the Devonian and Pennsylvanian rocks present irregular erosion surfaces.

2. Bedrock in the bluff is covered with deposits of probably three glacial invasions, the Nebraskan, Kansan, and Illinoisan with the most recent being the Illinoisan. The glacial till deposited by these three glacial invasions are covered by a thick deposit of loess. The gently sloping bluff rises to a height of about 150 feet above the Mississippi River.

3. The flood plain in Davenport varies in width from a few hundred feet at the upstream end of the project are, at the Government Bridge, to about a mile at the lower end, near the Interstate Highway 280 Bridge. The soils of flood plain are mainly alluvium, underlain by sand and gravelly sand strata, thin or non-existent in the downstream portions of the project area becoming a bit thicker in the upstream areas.

4. Blackhawk Creek is the only tributary stream entering the flood plain of the project area. Upstream from the bluff line it has a winding course, but having a fairly steep gradient, Blackhawk Creek has not developed a significant flood plain. Its course across the Mississippi River flood plain is also winding, flanked in some reaches by levees. It discharges into the part of the river known variously as Credit Island Slough, Davenport Harbor, or the Harbor of Refuge.

5. Ravines and gullies cut into the bluff are usually occupied by roads. Drainage from these is carried to the river in storm sewers.

Subsurface Explorations

6. Some 40+ drill borings were taken in 1974 and 1975 along (approximately 500 to 1,000 feet intervals) and adjacent to the line of the proposed flood control structures. Rotary drill borings were generally taken to rock. Several of these drill borings were taken in the river to determine

foundation characteristics and to locate possible hydraulic sand borrow areas. These 40+ drill borings were supplemented for this study by several drill borings previously made by others for an investigation for a sewer line. Also, 20 hand-augered borings (4-inch) were taken in 1974 and 1975 in Credit Island Slough. These auger borings were located in the field by Foundation and Materials Branch soils technicians with the purpose of defining areas of generalized soil conditions as well as areas of specific concern for underseepage, slope stability, settlement, and borrow. Generally, the auger borings penetrated the cohesive alluvial soils, and were extended into the cohesionless soil by means of driving a thin wall pipe (1-1/2 inches) until refusal or for a distance of 5 feet. Additionally, 22 hand-augered borings, taken in 1960 and 1965 in Credit Island Slough for an investigation for a small boat harbor, and some 20 miscellaneous drill and hand-augered borings and jettings, taken at various times for a variety of purposes, were used in the present investigation to determine foundation soil properties and to locate possible borrow sites. The locations of all borings are shown on the plan and profile sheets, Plates 3 through 28, and on the soil boring location plan, Plate 32. The logs of the borings are shown on Plates 33 through 38. Further explanation of the boring details are shown in the boring legend on Plate 34.

Soil Tests

7. Soil samples in the drill and hand-augered borings were taken generally at 2-foot intervals of depth or at perceptible stratum changes. Visual classification was made of all soil samples. Natural moisture content was determined on essentially all impervious soil samples. Atterberg limits were obtained on selected samples. Blow-count pentration tests (2-inch o.d.) were determined at random intervals in the drill borings. Gradation tests were run on representative samples of cohesionless soils. The natural moisture contents, Atterberg limits, D_{10} sizes, and blow-counts are shown on the boring logs. The plasticity chart is shown on Plate B-1. Gradation curves and effective grain sizes (D_{10}) sizes are shown on Plate B-3.

8. Consolidation and triaxial "Q" shear strength tests (unconsolidated, undrained tests) were run on selected samples. The results of the consolidation test are presented on Plates B-11 through B-13. The results of the shear strength tests are shown on Plates B-4 through B-10.

Foundation Materials

9. Downstream from the above described reach, the flood plain materials are those naturally deposited by the river. Depth to rock in this reach varies from 7 feet to 20 feet. The alluvial materials are mostly lean clay and clayey sand. This impervious material overlying sand and/or rock varies in thickness from about 5 feet to about 14 feet. The sand stratum thickness, if present, varies up to about 8 feet. In the slough at the downstream end of Credit Island there appears to be a maximum thickness of about 5 feet of sand on shale rock, overlain by a minimum of approximately

4 feet of soft organic fat clay. In a 3,500 foot reach on Credit Island only clay was found above rock, with the shale rock being at an average depth of about 10 feet.

10. In the upstream 1.5 miles of the project area almost all borings show some fill materials. The distance that the river has been encroached upon increases from perhaps one hundred to two hundred feet at the Government Bridge to more than a quarter of a mile just upstream of Credit Island. Inspection of a map made at the turn of the century shows that a large embayment has been filled, engulfing three small islands in the process. Until just a few years ago, much of this area was the city's only garbage dump.

11. Depth to rock in the approximately upstream 1.5 miles of the project rech (Ripley Street to the vicinity of the old sewage treatment plant) varies from about 25 feet to more than 50 feet. The materials are generally lean and sandy clays containing broken concrete, rock, bricks, cinders, etc. There are some areas, however, in this 1.5 mile reach that contain fairly clean sand.

12. Borings in the slough adjacent to Credit Island show from 1 foot to 12 feet of silt and/or clay, an average being approximately 6 feet. The materials are mainly soft organic fat clay with some lean and medium clays. Most of the borings in the slough were hand-augered and terminated in cohesionless material (after 5 foot length of thin wall pipe driven) or at refusal in what was considered at or near bedrock. Three representative drill borings made in the slough in 1975 verified the previous assumption regarding bedrock surface elevations. See logs of borings D-32, D-33, and D-34, Plate 37.

Proposed Embankments

13. Levees to protect Davenport, Iowa, will be constructed of hydraulic sandfill taken from the Mississippi River with an impervious material face both riverside and landside. The hydraulic sandfill embankment construction procedures will be the same as those used on prior projects within the District, i.e., Fulton, Clinton, Muscatine, etc. All levee slopes will be shaped to IV on 3H riverside and IV on 4H landside.

Dredged material from Credit Island slough was considered for use as impervious material to cover the riverward slope of the sand core of the embankment and as semi-compacted impervious material to cover the landward slope. The dredged material was judged to be unsuitable as an impervious material cover for the riverward slope but can be used on the landward slope, if the material is removed by dragline from the slough. Borrow material suitable for use as compacted impervious riverward slope cover will be acquired from the excavation of Blackhawk Creek and from one of the bluffs near the project area with the city of Davenport, Iowa, providing the sites. Borrow areas will be tested prior to completion of plans and specifications and incorporated into them. In general, most of the embankment sections will be covered with a 3-foot thick compacted impervious blanket on the riverward slope and a 1.5 foot thick impervious

blanket on the landward slope. The cover on the landward side should provide sufficient thickness for growth of vegetation. No compaction of this material will be required. All inspection trenches will be required to be filled with compacted impervious materials.

Levee Sections

14. Interstate 280 Bridge to Davenport Harbor Closure - Station 245+20 to 256+40. In this reach the levee will have a maximum height of 20.5 feet and a crown width of 10 feet. The Mississippi River slope (IV on 3H) will be covered with 3 feet of compacted impervious material beneath 6 inches of bedding material and an 18 inch thick riprap blanket. The landward slope (IV on 4H) will be covered with 1.5 feet of impervious material and then seeded for erosion protection. For typical levee section of this reach, see Plate 30.

15. Credit Island Slough Closure - Station 221+00 to 245+20. The levee section will reach an approximate height of 27 feet depending on the required foundation excavation in this area. Excavation for the embankment will be required to bedrock in this reach. The embankment core, constructed of hydraulic sandfill, will be covered with 3-feet of compacted impervious material on the Mississippi River slope (IV on 3H) and 1.5 feet of impervious material on the Davenport Harbor slope (IV on 4H). The compacted impervious material on the riverward slope will be covered with an 6-inch thick blanket of bedding material and an 18-inch thick layer of riprap. The compacted impervious material can only be placed above the waterline. A clay berm will be required to be placed at the toe of riverward slope and tied into the compacted impervious cover. The crown width will be 12 feet in this reach. For typical levee section of this reach, see Plate 30.

16. Credit Island Reach - Station 144+00 to 221+00. In this reach the levee will have a maximum height of 26.0 feet. The crown width will be 12 feet from Station 144+00 to 200+00 and 10 feet from Station 200+00 to 221+00. The Mississippi River slope will be IV on 3H and the Credit

Island slope IV on 4H. Compacted impervious material cover the hydraulic sandfill core on the riverward slope and over the crown. The thickness will range from 3 feet thick on the riverward slope and 2.5 feet thick over the crown. A 1.5 foot thick impervious material blanket will be required on the Credit Island Slough slope. Both slopes will be seeded for erosion protection, no riprap will be placed in this reach. An inspection trench will be placed beneath the compacted impervious embankment material of the riverward slope. The trench should be excavated to impervious materials (approximate elevation 550.0±) and filled with compacted impervious material. For typical levee section of this reach, see Plate 30.

17. Credit Island Causeway - Station 125+00 to 144+00. The crown width will be 12 feet between Station 134+00 to 144+00 and increased to 30 feet between stations 125+00 to 134+00. Maximum levee height will be approximately 26 feet within this reach. The embankment will be hydraulic sandfill covered with compacted impervious material on the riverward slope and

impervious material on the landward slope. The thickness of compacted impervious material will be 3 feet on the IV to 3H riverward slope and 1.5 feet of impervious on IV to 4H Credit Island Slough side. Slope protection on the river slope will consist of 18 inches of riprap over 6 inches of bedding materials placed on the compacted impervious material. The slough slope will be seeded. For typical levee section of this reach, see Plate 30.

18. Davenport Shore Reach.

a. Station 98+00 to 125+00. The crown width will be 30 feet with IV on 3H riverward slope and IV on 4H landward slope. Maximum levee height for this reach will be approximately 25 feet. A 15 foot wide berm at elevation 561.0 on the riverside slope will provide a bench for a bicycle path through this reach. The bench at elevation 561.0 was required for stability of this reach and the bicycle path added to provide recreational benefits attributed to the project. Impervious material will cover the sandfill embankment as in the prior reach. Slope protection on the riverslope will consist of 18 inches of riprap over 6 inches of bedding materials placed on the compacted impervious material. The landward slope will be seeded. For typical levee section of this reach, see Plate 30.

b. Station 75+00 to 98+00. The existing slope in the area of the former garbage dump will be left as is, except to provide an adequate foundation for the scenic road. A 1½ foot thick layer of impervious material dredged from Credit Island Slough will be placed on the slope and over the entire former dump area to provide adequate material for vegetation growth. Borings will be taken for station 75+00 to 80+00 along the proposed alignment before completion of plans and specifications or a deeper than usual inspection trench excavated. No unusual conditions are anticipated.

c. Station 46+00 to 75+00. This reach will have a crown width of 30 feet with a riverside slope of IV on 3H and a landside slope of IV on 4H. The maximum embankment height in this reach will be about 15 feet. Compacted impervious material will cover the hydraulic sandfill on the riverside slope and it will be 3 feet thick. Impervious material will be placed on the landward slope and it will be 1.5 feet thick. The riverside layer of impervious material will be tied into the impervious riverside blanket extending to the existing seawall. An inspection trench will be located at the riverside toe. Slope protection for the riverside slope will consist of 18 inches of riprap over 6 inches of bedding materials placed on the compacted impervious material. The landside slope will be seeded to prevent erosion. For typical levee section of this reach, see Plate 30.

Levee Foundations

19. Interstate 280 Bridge to Davenport Harbor Closure - Station 245+20 to 256+40. Sufficient thickness of clay exists in the foundation to provide an adequate seal for underseepage control. The clay is of

sufficient strength to provide adequate support for the proposed embankment in this reach. The average water content for the CL-CH material is between 20 and 30, indicating a cohesion of 700-1000 p.s.f. Beneath the clay and directly over the bedrock lies a sand strata approximately 2 to 6 feet thick. The foundations for levees will be stripped of vegetation and other unsatisfactory materials.

20. Credit Island Slough Closure - Station 221+00 to 245+20. The foundation clay can be separated into two distinct layers. That above elevation 543.0 is a lean medium plastic clay, below this and generally to elevation 536.0 the clay exhibits high plasticity. Atterberg limits illustrating this difference are shown on Plate B-2. Shear test results from boring D-3i (see plate B-6) show that the lower soft clay is of insufficient strength (cohesion of approximately 170 p.s.f.) to support the proposed embankment closure section. At elevation 540.0 in boring D-11, the high moisture content indicates a soft clay condition. However, the Atterberg limits indicate the same origin as the above clay higher strength. The high moisture content in this one sample may be a reflection of the wood found in the strata. Below the soft clay an intermittant sand strata lies directly on shale rock. Due to the low shear strengths of the clay foundation, all materials will be excavated to rock and the proposed embankment placed directly on the shale rock. See Plate 30.

21. Credit Island Reach.

a. Station 190+00 to 221+00. Throughout this reach lean clay and/or clayey sand lies beneath the riverside slope of the embankment and directly upon shale rock. The clay is of sufficient strength to support the proposed embankment. This material is classified as a lean clay (CL) with an average water content of 15 percent, indicating a cohesive strength greater than 700 p.s.f. However, toward the slough, the soft alluvial clays will have to be removed by excavation and/or by displacement tion trench located beneath the riverside slope's compacted impervious blanket will be excavated to impervious materials and backfilled with compacted impervious materials to prevent underseepage in the upper pervious stratum. The foundations for levees will be stripped of vegetation and other unsatisfactory materials.

b. Station 144+00 to 190+00. In this reach a sand strata lies directly upon the shale rock. The sand strata ranges in thickness from two to four feet. This condition is revealed in borings D-6 through D-9. (See boring logs on Plate 33 and 34). Beneath the riverward slope of the embankment, the sand is overlain by stratas of lean clay, clayey silt, and silty sand. Boring D-10 indicated 11+ feet of sandy silt. These materials appear to be of sufficient strength to adequately support the proposed embankment. On the Credit Island Slough slope, the soft alluvial clays will have to be removed by excavation and/or displaced by the hydraulic sand embankment during construction. An inspection/cut off trench will be required beneath the riverward slope's compacted impervious material blanket excavated to impermeable material to prevent underseepage in the upper pervious materials. The foundation for levees will be stripped of vegetation and other unsatisfactory materials.

22. Credit Island Causeway.

a. Station 134+00 to 144+00. The clay foundation consists of lean to fat clay separated by a 5-foot thick strata of sand at station 143+00. Borings D-12 and AA-12, Plate 34, both indicate soft clay at elevations 537.0 and 541.0, respectively. The soil shear strengths (lean clay with indicated strength of 400 p.s.f. and fat clay with indicated strength between 450 and 550 p.s.f.) in combination with IV on 3H and IV on 4H slopes will be adequate to insure stability of the section. The soft clay at the base of embankment will have to be removed by excavation and/or by displacement by the hydraulic sand embankment during construction. A combination inspection/cut off trench, as in previous reach, will be required to prevent underseepage in the upper pervious soils. The foundation for embankments will be stripped of vegetation and other unsatisfactory materials.

b. Station 130+50 to 134+00. The foundation consists of lean to fat clay with some sandy clay present, underwhich there is a 1-foot thick strata of sand directly over bedrock. The clay foundation, in this short reach over the existing road embankment, has adequate strength to support the proposed embankments, as indicated in Boring D-13, Plate 34. Any soft material encountered at either toe will have to be removed by excavation and/or by displacement with the hydraulic sandfill embankment during construction. The foundation for the embankment will be stripped of vegetation and any other unsatisfactory materials.

c. Station 125+00 to 130+50. A 3-foot thick strata of sand lies above shale rock and is overlain by 13 feet of clay. The clay in this reach is of sufficient strength to support the proposed embankment. The average cohesion for the material in this reach is assumed to be 450 p.s.f. as indicated by the borings. Any soft material found during construction of the embankments will have to be removed by excavation and/or by displacment with hydraulic sandfill. The foundation for the levees will be stripped of vegetation and any other unsatisfactory materials.

23. Davenport Shore Reach.

a. Station 98+00 to 125+00. Approximately between Stations 110+00 to 125+00 and 98+00 to 102+00, pervious material exists beneath elevation 540.0 to bedrock. The limestone bedrock tends to raise (between Stations 102+00 to 110+00) to elevation 543.0 as indicated in Boring D-16, see Plate 34. In this reach from station 102+00 to 110+00, no sand is present. Above this a thick strata of clay extends to surface. Silt is more predominant beneath the landward side of the levee as shown in Borings L-13 and L-14, Plate 34.

Soft organic clays, rubble and debris at the surface will be removed to provide adequate slope stability. An inspection/cutoff trench will be located under the riverside slope's impervious material cover to prevent underseepage in the upper pervious fill materials.

b. Station 75+00 to 96+00. This reach is the former garbage dump for the city of Davenport, Iowa. Pervious sand strata lie between bedrock and surficial clays. The land topography is high and wide in this reach and will provide adequate protection against underseepage.

c. Station 46+00 to 75+00. Borings D-22 through D-24, Plate 35 indicate a minimum 4-foot thick impervious strata of lean clay - sandy clay at the surface. Areas, such as shown in Boring 13-E, Plate 35, where debris and rubble are found at or near the surface will be required to be removed. If the material extends to great of a depth, the first two feet will be removed and replaced by a two foot layer of impervious material. Generally in this reach there exists a 5-foot strata of broken rock, sand, and gravel beginning at elevation 541.5 and extending to bedrock. Above this strata is approximately 18 feet of lean clays, sandy clays and clayey sands. An inspection/cutoff trench will be located beneath the riverside slope's impervious material cover to prevent underseepage in the upper pervious materials. The foundation for the embankments will be stripped of vegetation and any other unsatisfactory materials.

Foundations for Other Structures

24. Pumping Stations, Gatewells and Closure Structures: Foundation considerations for pumping plants and related drainage structures are contained in Appendix C, Design Analysis. Construction dewatering will be the responsibility of the contractor. A contractor proposal for the method offered to accomplish the dewatering job will be required for each individual structure and will subsequently be reviewed and approved/disapproved by the District. No problems or unusual conditions are expected.

25. Walls: Any existing miscellaneous fill material at I-walls will be excavated and replaced with compacted impervious fill. I-walls will be founded on sheet piling driven through the compacted impervious backfill into the clay alluvium or into the underlying sand or to rock. The slopes of the riverward embankment will be IV on 3H. Design analysis for the walls are contained in Appendix C, Design Analysis.

26. Blackhawk Creek Levees: Flank levees along the creek will be repaired and/or brought up to grade to provide adequate protection against flooding. Due to the height of the levee in this reach (5'±) no detailed slope stability analysis were determined to be necessary since existing levees are in some areas higher than those proposed. The levees in this area will be repaired or raised using compacted impervious material. Soil design parameters used for the Blackhawk Creek gabion wall will be verified before construction.

Slope Stability

27. Critical Sections: A review of all the levee sections throughout the project revealed stations 113+00 and 143+00 as the most critical with respect to riverside and landside slope stability. These sections were selected because of the embankment height and/or thickness and consistency

of the clay foundation. Typical sections of these two stations are shown on Plate 31. The analyses were performed to insure that the criteria requiring a minimum factor of safety of 1.30 was satisfied. In general thin layers of soft clay will be removed from the foundation by excavation and/or by displace-ment by the hydraulic sandfill embankment during construction. This will be the case under the landward slopes of the Credit Island and Credit Island Causeway reaches where soft clays are in Credit Island Slough and in the vicinity of station 103+00 to 107+00 (Davenport Shore Reach).

28. Design Parameters: Undisturbed shear strength test data for foundation soils in the project were available. See Plates B-4 through B-10 for results. From these tests a shear strength of 400 p.s.f. for the foundations clays was assumed throughout the project area. Also undisturbed shear strength data from unconfined compression and unconsolidated-undrained (Q) triaxial tests from various projects in the Rock Island District were reviewed for the purpose of determining a correlation between undrained shear strength and water content. This data was analyzed for the three principal cohesive soil types (CL, CL-CH, and CH) that exist within the project area. Although the plots of the individual strengths versus water content vary, a generalized trend between cohesion and water content was determined by averaging shear strengths for individual water contents. These relationships are shown for CL, CL-CH, and CH soils on Plates B-16, B-17, and B-18, respectively, and together for comparison purposes on Plate B-19. Curves were drawn through the averaged data. Comparisons between the assumed 400 p.s.f. shear strength and shear strengths obtained from the generalized curves were made. In most instances, the shear strengths obtained from the generalized plots were at or higher than the 400 p.s.f. assumed. Usual conservation values for the embankment soil strengths were made. Assumptions made regarding unit weights and shear strengths of the foundation are given on slope stability Plates B-20 through B-25.

29. Method of Analysis. The circular arc slope stability analysis, in accordance with EM 1110-2-1902 dated April 1970, was determined the best suited for use in performing the required stability analysis for the project. In the circular arc stability analysis, it was assumed that the elevation of the potential sliding surface would be tangent between the low strength soil overlying the pervious founda-tion or rock. The tangent elevations can be found on stability Plates B-20 through B-25. Generally, each section was analyzed by searching for the minimum factor of safety. Normally, 15 to 20 trial circles were analyzed before the one with the minimum factor of safety was determined. RID computer program #74124A was used in performing the analysis with a hand check made of the computer results for the circular arc with the minimum factor of safety. Both the landside and riverside slopes of the selected levee stations were analyzed for their stability for the end of construction condition. Other stability cases were assumed not to be as critical as the end of construction condition since the clay faced sand embankment will be subjected to water for relatively short periods of time, thus not allowing the steady seepage or intermediate river stage conditions to develop. River stages also are not anticipated to drop fast enough for the sudden drawdown condition to exist.

30. Results of Slope Stability Analysis. A minimum factor of safety of 1.3 was considered essential because of the nonuniformity of the foundation soils and the availability of a minimum amount of shear strength data for this project.

a. Station 113+00. The section was analyzed for stability of the IV on 3H riverside slope with a 15 foot wide bench at elevation 561.0 and for stability of the IV on 4H landside slope. Both were analyzed for the end of construction condition. The riverside slope is 25.5 feet above the riverward toe and the landside slope is 18.5 above the landward toe. A 9-16 foot thickness of CH, CL-CH clay exists under the levee. The moisture content of these clays range from 38 to 55 percent. Shear strength data in this area (Boring D-35) and from the generalized shear strength versus water content plots, the lowest Q strength is determined to be 400 p.s.f. A recent soft sedimentary clay deposits exists from station 103+00 to 107+00 and it will be advantageous to call for removal of this unsantisfactory soft sedimentary foundation material in this reach. The results, shown on Plate B-20, indicate a factor of safety of 1.30 with no seismic force and 1.04 with 0.05 seismic force for the riverside slope. The landside slope's factor of safety, shown on plate B-21, with no seismic force is 1.52 and with 0.05 seismic force is 1.18.

b. Station 143+00. Both the IV on 3H riverside slope and the IV on 4H landside slope were analyzed for stability for the end of construction condition. The riverside slope is 21.3 feet above the riverward toe and the landside slope is 27.2 feet above the landward toe. The levee foundation consists of CL clays from existing ground surface to elevation 543 with an average water content of 32 percent. From elevation 543 to 530, there exists a 5-foot thick strata of sand. Below elevation 538 and extending to rock (elevation 529) CL clays are again encountered with an average water content 23 percent. The lowest Q strength was assumed to be 400 p.s.f. This compares favorably with the strength obtained from the strength versus water content plot for 32% and the 400 p.s.f. estimate is somewhat lower than the strength obtained using 23 percent water content. Slope stability analyses were performed for both shallow and deep failures in this reach (circle tangent to elevation 543 top of sand and tangent to elevation 529 top of rock). The results shown on Plate B-22 indicate a factory of safety of 1.48 without any seismic force and 1.23 with 0.05 seismic force for a shallow failure of the riverside slope. For a deep failure of the riverside slope, the factor of safety is 1.55 with no seismic force and 1.23 with 0.05 seismic force and the results are shown on Plate B-23. The results shown on Plate B-24. reveal a factor of safety of 1.88 with zero seismic force and 1.51 with 0.05 seismic force for a shallow failure of the landside slope. Shown on Plate B-25 are results for a deep failure of the landside slope, the factor of safety is 1.33 without any seismic force and 1.05 with 0.05 seismic force.

Settlement and Consolidation

31. Critical Sections. Two typical levee and foundation sections were analyzed for foundation settlement caused by the additional load of the new embankments. The sections investigated correspond to those reaches

analyzed for slope stability and are stations 113+00 and 143+00. These sections were selected because of embankment height and thickness of compressible clay foundation.

32. Design Parameters.

a. Consolidation test data for the foundation soils in the project data was available (see Plates B-11 through B-13). The e versus log p plot was used in determining changes in void ratio for changes in overburden pressure due to construction of the new embankment.

b. No decrease in vertical pressure with depth was assumed.

33. Method of Analysis.

a. Estimates of foundation settlements were computed using the following formula.

$$H = \frac{12H\ (e_1 - e_2)}{(1+e)}$$

where: H = estimated settlement in inches
H = thickness of compressible layer in feet
e^1 = void ratio at existing overburden pressure
e^1 = void ratio at existing overburden plus new embankment pressure

b. Settlements were computed for each stratum at its midpoint and summed to give the total settlement. It was assumed any compression of the foundation sands would occur concurrently with the construction of the new embankment.

34. Estimated and Time Rate of Settlements.

a. The estimated settlement for the two levee stations are summarized as follows:

Station 113+00 17 inches at Levee C/L
Station 113+00 9 inches at 561 bench C/L
Station 143+00 20 inches at Levee C/L

For actual settlement computations, see plates B-26 and B-27. It is estimated that maximum settlement will be on the order of 17-20 inches depending on height of embankment and thickness of compressible strata.

b. A time-rate of settlement curve was plotted using time curves obtained from the consolidation test data for this project. It is estimated that approximately 75 to 80 percent of the total consolidation will have taken place in the first year after completion of construction of the new embankment. See plates B-28 and B-29 for time rate of consolidation computations and curve.

c. To compensate for foundation settlement, the embankment needs to be overbuilt by 8 percent of the new embankment height throughout the entire reach of the critical sections. Any settlement of the hydraulic sandfill embankment is assumed to occur concurrently with construction and need not be compensated for. For plans and specifications a shrinkage allowance of 10 percent will be used.

35. Through Seepage. Seepage is not expected through the compacted impervious face on the Mississippi River slope and no problems of major concern are anticipated. For the Credit Island Slough closure, the clay berm and clay face will prevent any seepage. A differential head is not expected to occur that will cause seepage from the land-side to the river-side slope.

Underseepage

36. General. Underseepage control measures for the Davenport project are based on a study of thickness and characteristics of the impervious stratum and the pervious substratum. The longitudinal profile of the subsurface conditions is shown on Plates B-14 and B-15. In general, underseepage will be controlled by a compacted impervious inspection/cutoff trench and/or berms, or sections with a base width of 10H or greater.

37. Design of Seepage Berms.

a. The underseepage and berm analysis followed the criteria in "Relief Well Design," Civil Works Engineer Bulletin 55-11, dated 28 June 1955 and Waterways Experiment Station publication Technical Memorandum 3-424 "Investigation of Underseepage and its Control, Lower Mississippi River Levees," dated October 1956 and Engineering Manual 1110-2-1913 "Design and Construction of Levees," dated 31 March 1978. A "Report on Conference on Underseepage for Agricultural Levees," NCRGT letter to OCE, Rock Island District, dated 11 October 1960 and "Tentative Criteria for Use of Underseepage Control Measures on Agricultural Levees," Office, Chief of Engineers, dated 3 June 1958, established empirical limitations on such items as maximum length of berms and permeability ratios. A further modification of this criteria resulted from the "Minutes of Geotechnical Conference" held at RID on 29-30 April 1976. This method-ology of the underseepage berm analysis is the result of continuing documentation of performance of existing levees during high water in 1965 through 1975 from Dubuque, Iowa, to Hamburg, Illinois, by personnel of the Foundations and Materials Branch. This documentation includes periods when the river rose to the top of the existing levees while the levee performed satisfactorily with respect to underseepage and through seepage. Methodology was further established in Supplement No. 1 to Design Memorandum No. 1 for Fulton, Illinois, Local Flood Protection Project, Draft Dec 76. The nomenclature used in this section of the report is on Plates B-30 through B-34.

b. Design values used in determining the need for and designing of seepage berms for levees are given on the Summary of Underseepage and Berm Analysis plate. A standard analytical berm design for underseepage was not considered applicable because of the variability of the alluvial or surface soils. The interbedded strata of clay, clayey sand, silt, and sand are of such varied nature that it is not realistic to assume any continuous blanket conditions. However, to obtain an estimate of what may occur and to provide some basis for requiring or not requiring underseepage berms, stratification assumptions were made and an analysis performed. Permeability ratios are the same as those used for previous projects within the District. Further documentation for the use of these ratios will be presented upon publication of RID "Mississippi River Underseepage Study" currently being prepared by Waterways Experiment Station. No problems have arisen in the past using these design values and none are anticipated for the Davenport project.

38. Reaches and Required Treatment. The treatment required to control underseepage in the project area is not extensive. Typical sections of the proposed levee and existing foundation are shown for each reach on Plates B-14 and B-15. The borings used within each section to determine existing foundations are listed. Analysis were performed assuming Credit Island Slough would be dredged, no nearer than 100 feet to the landside toe. This restriction will provide adequate underseepage control from Station 221+00 to 134+00 (reaches adjacent to Credit Island Slough). Also for reaches from 221+00 to 134+00, the inspection trench will be shifted to the levee backslope, if at time of construction the water level in the slough is low enough that this change can be accomplished without extensive dewatering requirements. Computed factors of safety given in the following paragraphs (38a through 38l) are factors of safety for uplift (heave) of the landside blanket. Minimum recommended factor of safety as given in EM 1110-2-1903 for this condition (uplift of landside blanket) is 1.6. In some reaches, the factor of safety computed is less than the recommended minimum and no underseepage control is provided. The decision not to provide underseepage control is based on previous experience in similar situations within the District and engineering judgment since, as stated previously, there is a high variability of the alluvial and surface soils making a standard theoretical approach not completely applicable.

a. Station 256+40 to 245+20. The less than 5-foot thick natural sand aquifer and about 11 feet landside blanket thickness are sufficient to control underseepage for a head of 18.3 feet. Factors of safety for this reach is 2.16.

b. Station 245+20 to 221+00. Due to the soft foundation clays, all materials will be excavated to rock. (To prevent underseepage care will be taken to place an impervious berm on the riverside toe to provide a positive cutoff against underseepage.)

c. Station 221+00 to 190+00. No aquifer is shown in the borings within this reach. No underseepage analysis or control required. An inspection trench under the riverside slope will be provided in this reach.

d. Station 190+00 to 170+00. This section was analyzed with two different assumptions regarding entrance conditions. Assuming an open entrance at the lagoon on Credit Island the factor of safety was 1.29 and assuming a blocked entrance (ie, thinner riverside blanket) at the lagoon the factor of safety was 1.12. Since the factor of safety for the worst condition was low, engineering judgment was used to determine underseepage control. Consequently, a berm is not proposed for the 5-foot thick aquifer, 20.9 foot head, and 3.5 feet of landside blanket thickness. An inspection trench will be required underneath the riverside slope. Factor of safety for this reach with the slough dredged is 1.14 (blocked entrance at lagoon) and 1.31 (open entrance at lagoon).

e. Station 170+00 to 160+00. The less than 5-foot thick natural sand aquifer and 6-foot landside blanket thickness are enough to control underseepage for a head of 20.0 feet. Factor of safety for this reach is 1.54 or 1.67 with the slough dredged. An inspection trench beneath the riverward slope will be required.

f. Station 160+00 to 150+00. In this reach an inspection trench beneath the riverward slope will be required. The less than 3-foot thick natural sand aquifer and approximately 5-foot thick landside blanket are enough to control underseepage for a head of 24.6 feet. Factor of safety determined for this reach is 1.62 whether the slough is dredged or not.

g. Station 150+00 to 144+00. The factor of safety for this reach was determined to be 1.40, engineering judgement was used to obtain needed underseepage control. Correspondingly, a berm is not proposed for the 3-foot thick natural sand aquifer, 6-foot thick landside blanket, and 23.7 foot head. An inspection trench underneath the riverward slope will be required in this reach. The factor of safety for this reach is 1.48 with the slough dredged.

h. Station 144+00 to 134+00. Engineering judgement was again used to determine the required underseepage control since the factor of safety for this reach is 1.33 or 1.44 with the slough dredged. The combination of the less than 3 feet natural sand aquifer thickness, the 8-foot thick landside blanket and 25.9 foot head was determined to be adequate for underseepage control. An inspection trench beneath the riverward slope will be required.

i. Station 134+00 to 125+00. The less than 2-foot thick natural sand aquifer and about 12.5 foot landside blanket thickness are sufficient to control underseepage for a head of 19.8 feet. Factor of safety for this reach is 2.62. An inspection trench will be required beneath the riverside slope.

j. Station 125+00 to 117+00. The factor of safety for this reach was determined to be 1.91. Underseepage will be controlled by the thick landside blanket (about 14 feet) and the less than 7-foot thick natural sand aquifer for a head of 18.4 feet. An inspection trench will be required beneath the riverside slope.

k. Station 117+00 to 70+00. The base width of the proposed new embankment being greater than 10H, the natural topography being high (area of former garbage dump) and the thick landside blanket will provide sufficient protection against underseepage in this reach. An inspection trench will be required beneath the riverside slope of the proposed embankment.

l. Station 70+00 to 46+00. The 5-foot thick natural sand aquifer and about 19-foot thick landside blanket are sufficient to control underseepage for a head of 11.4 feet. An inspection trench will be required beneath the riverside slope. The factor of safety for this reach is 2.93.

39. Slope Protection. Reaches of levee exposed to wavewash by the Mississippi River will be protected by an 18-inch thick layer of riprap placed on a 6-inch crushed stone/sand bedding. For a IV on 3H slope and up to 3-foot waves, seldom exceeded in this reach of the river, the following gradation of riprap is to be used.

Percent Lighter by Weight	Weight of Pieces in Pounds
100	200-400
50	90-180
10	15-30

The gradation was determined in accordance with Plate II of EM-1110-2-2300 and will accomodate the maximum size stone for the 18 inch thickness of riprap planned. The design gradation is similar to that used in Clinton, Iowa, Design Memorandum No. 1, Supplement No. 2. All other slopes where no riprap protection is planned will be seeded. A gradation curve for the riprap protection is shown on Plate B-35.

Availability of Construction Materials

40. Pervious Borrow. The hydraulic sandfill borrow required for the project will be located along the Mississippi River and adjacent to Credit Island. No problems are anticipated in obtaining the required amount of sandfill for the project for the river.

41. Impervious Borrow. The impervious material required to cover the riverside slopes of the proposed new embankments will be obtained from one of the bluffs near the project and from the excavation of Blackhawk Creek. Portions of the dredged material from Credit Island Slough suitable for use as impervious material will be used on the landside face of the new embankment. Availability of impervious material should not present a problem.

42. Protection Stone. Suitable stone for production of riprap and bedding material can be obtained at the sources listed below.

Quarry	Owner	Ledges Acceptable
Linwood, Iowa Quarry	Linwood Stone Products Co. Davenport, Iowa	Gower stone
LeClaire, Iowa	LeClaire Quarries	Unweathered stone
Shaffton, Iowa Quarry	Alpha Crushed Stone Co. Marion, Iowa	Iowa State Highway Commission ledges 15 through 18

43. Concrete Materials. A separate design memorandum is being prepared in accordance with guidelines given in EM 1110-2-200 for concrete materials investigations. This design memorandum will be used as the basis for all upcoming construction projects in the Quad Cities Metropolitan area. Scheduled completion date is May 1982.

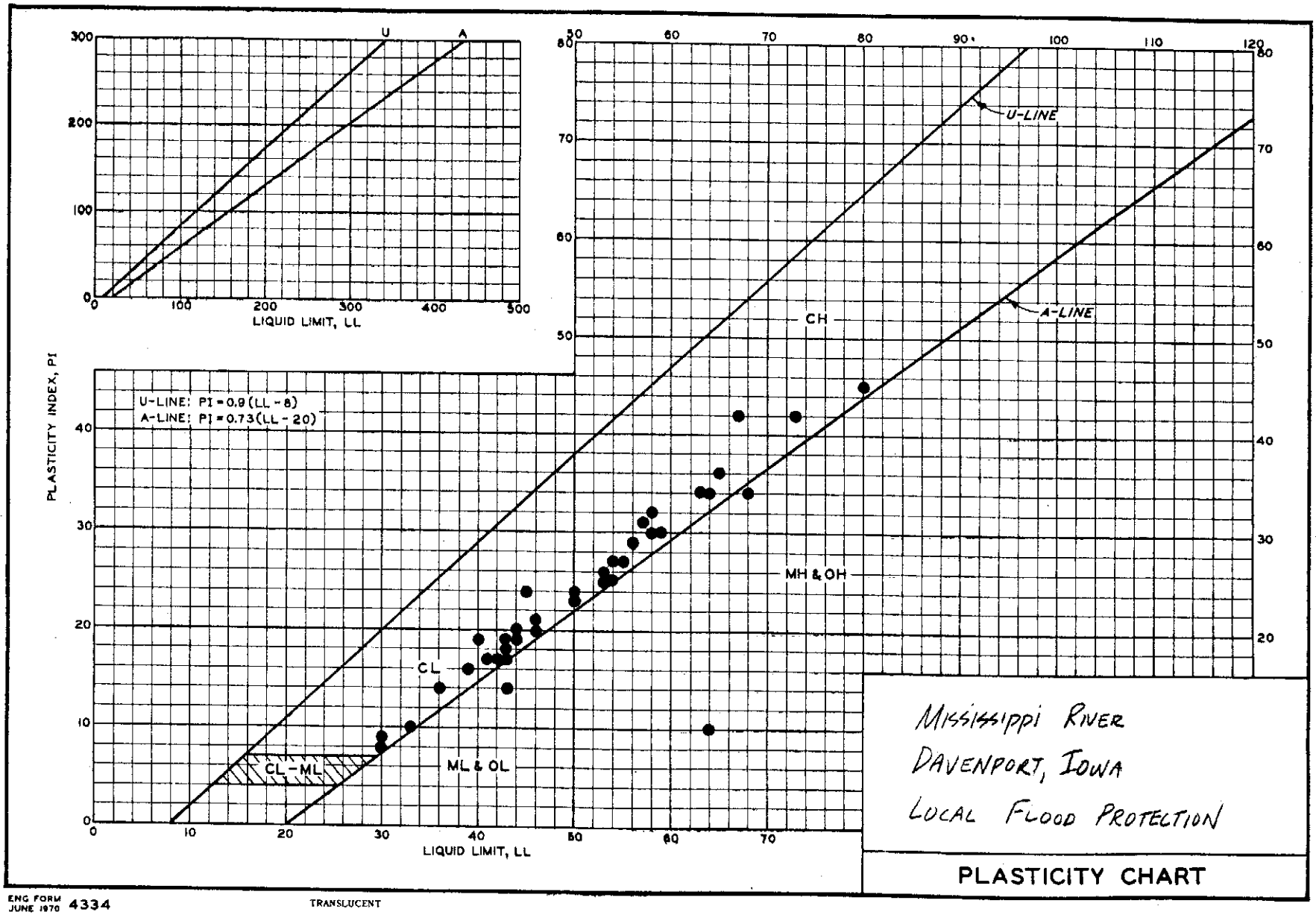
Flood History

44. General. The city of Davenport, Iowa, and its surrounding area are subject to flooding from Blackhawk Creek and Mississippi River backwater. Presently there is no flood protection system that protects the city of Davenport, particularly the highly commercial areas along the river and the central business district (CBD) area.

45. 1965 Flood. The record crest flooded the area extending the Davenport Water Works to the lower city limits. Included in this area are large industrial plants, wholesale houses, warehouses, commercial and business establishments, municipal stadium, parks, disposal plant and residential units. The entire area was inundated to various depths. Practically all of the business establishments riverward of Second Street were affected by basement flooding. Railroads and streets paralleling the river were flooded, closing down automobile and railroad traffic. Extensive emergency protective work was performed at essentially all the industrial and commercial plants. The Credit Island Golf Course and park were damaged so severely, they were required to be closed the remainder of the year. Residential units in the Garden Addition, near the mouth of Blackhawk Creek and lower reaches of the city were seriously affected. Damages occurred to 416 dwellings, causing 1318 persons to evacuate. Total amount of damages incurred by residents and businesses due to the 1965 flood was $3,824,000.

46. 1969 Flood. The river stage was about 3.2 feet lower than that of the record stage of April 1965. Operation Foresight (an early warning to area residents that flooding was expected) was implemented throughout the Rock Island District and Public Law 99 funds used to help prevent flooding. Protective work was done on essentially all commercial and industrial plants and residences along the river preventing approximately $100,000 worth of damage. Davenport area flooding was not a major problem due to the success of Operation Foresight and river stage levels being lower than predicted.

47. 1973 Flood. Main stem flooding was not a major problem in the Davenport area during the flood of 1973. Extensive emergency protective work was done at essentially all commercial and industrial plants along the river preventing an estimated $700,000 worth of damage. The area of Davenport approximately from 2nd Avenue riverward was inundated twice during the March-April 1973 period due to the Mississippi River cresting twice during this period. The river stage was about 3.7 feet lower than that of the record stage which occurred during April, 1965. Total damages incurred by businesses and residents due to the 1973 flood was estimated at $750,000.



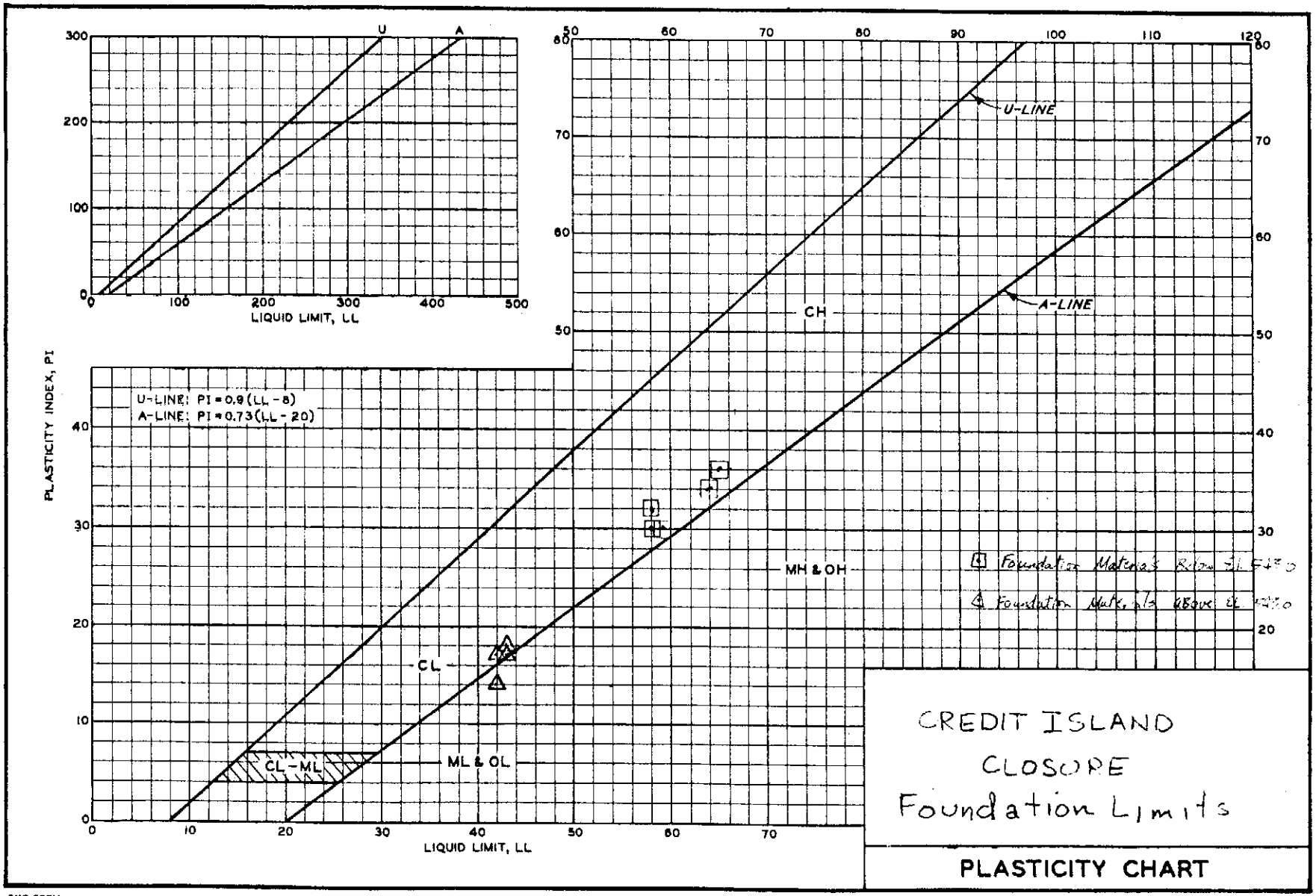
Mississippi River
Davenport, Iowa
Local Flood Protection

PLASTICITY CHART

PLATE B-1

ENG FORM 4334
JUNE 1970

TRANSLUCENT



CREDIT ISLAND

CLOSURE

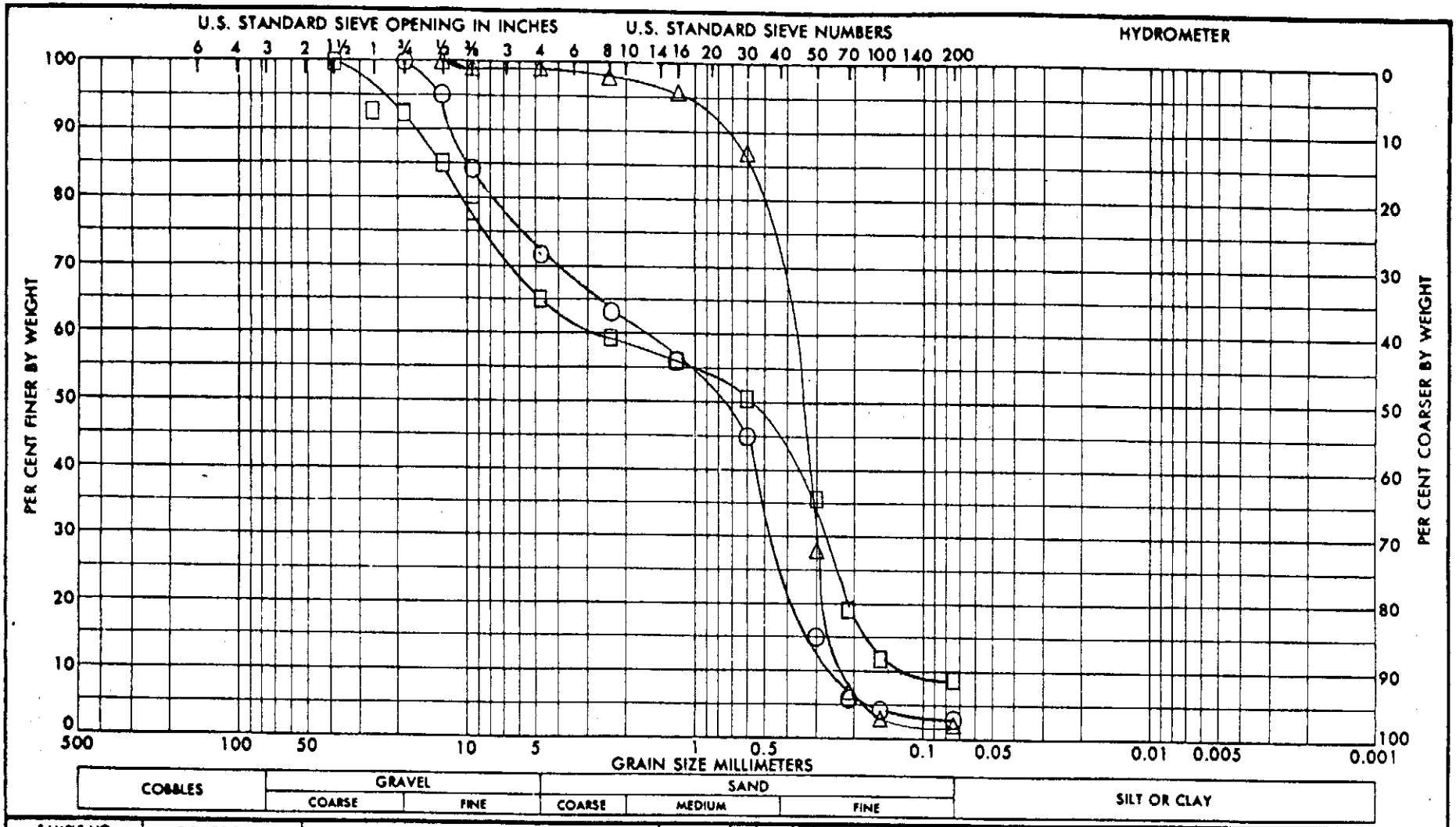
Foundation Limits

PLASTICITY CHART

PLATE B-2

ENG FORM 4334
JUNE 1970

TRANSLUCENT



SAMPLE NO.	ELEV OR DEPTH	CLASSIFICATION	NAT W%	LL	PL	PI
⊡ D21-8	18.5-19.5	SP-SC Gr clayey gravelly med. to fine sand				0.13
⊙ D22-12	29.5-30.5	SP G gravelly coarse to fine sand				0.26
△ D27-6	12.0	SP Br med. to fine sand				0.24

GRADATION CURVES

PROJECT Davenport, LOCAL PROTECTION

AREA

BORING NO.

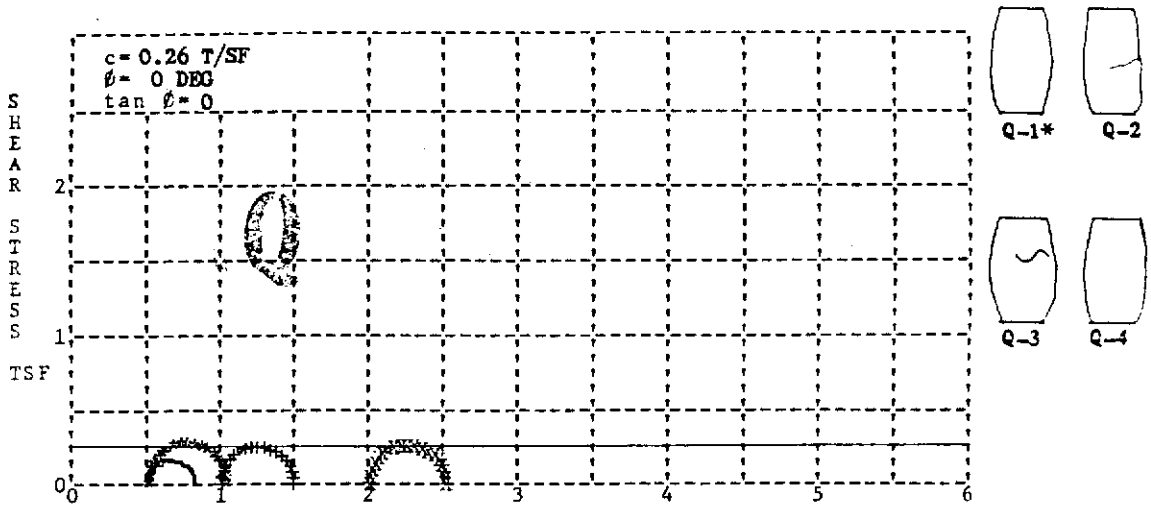
DATE November, 1974

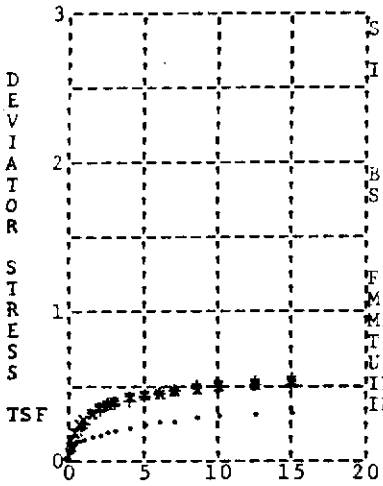
ENG FORM 2087
1 MAY 63

REPLACES WES FORM NO. 1241, SEP 1962, WHICH IS OBSOLETE.

U.S. GOVERNMENT PRINTING OFFICE : 1963 OF—700-128

PLATE B-3





SPECIMEN NUMBER		1	2	3	4
INITIAL	WATER CONTENT,%	38.8	38.7	38.7	41.1
	DRY DENSITY,PCF	78.5	79.3	78.6	76.2
	SATURATION,%	92	93	92	92
	VOID RATIO	1.13	1.11	1.13	1.20
BEFORE SHEAR	WATER CONTENT,%	38.7	38.3	38.4	40.8
	DRY DENSITY,PCF	80.2	79.3	78.6	76.4
	SATURATION,%	95	93	91	92
	VOID RATIO	1.09	1.11	1.13	1.19
FINAL BACK PRESSURE,TSF		.00	.00	.00	.00
MINOR PRINCIPAL STRESS,TSF		.50	1.00	2.00	.50
MAX DEVIATOR STRESS,TSF		.32	.51	.53	.54
TIME TO MAX DEV,MIN		33	30	30	29
ULT DEVIATOR STRESS,TSF		.32	.51	.53	.54
INITIAL DIAMETER,IN		1.392	1.388	1.392	1.406
INITIAL HEIGHT,IN		2.980	2.974	2.989	2.954

CONTROLLED-STRAIN TEST

DESCRIPTION OF SPECIMENS: **Lean clay, CL**

LL **46** PL **22** PI **24** Gs= **2.68** TYPE SPECIMEN: UNDISTURBED TYPE TEST Q

REMARKS: MACHINE PRINT OUT
FORMAT AFTER ENG FORM 2089

Questionable "before shear" dry density necessitated a rerun at 0.5 T/SF. Dark gray lean clay, with some fine hair roots. Plastic structure, soft consistency, sl. sensitive. Med. toughness at PL, dull shine, sluggish shake reaction.

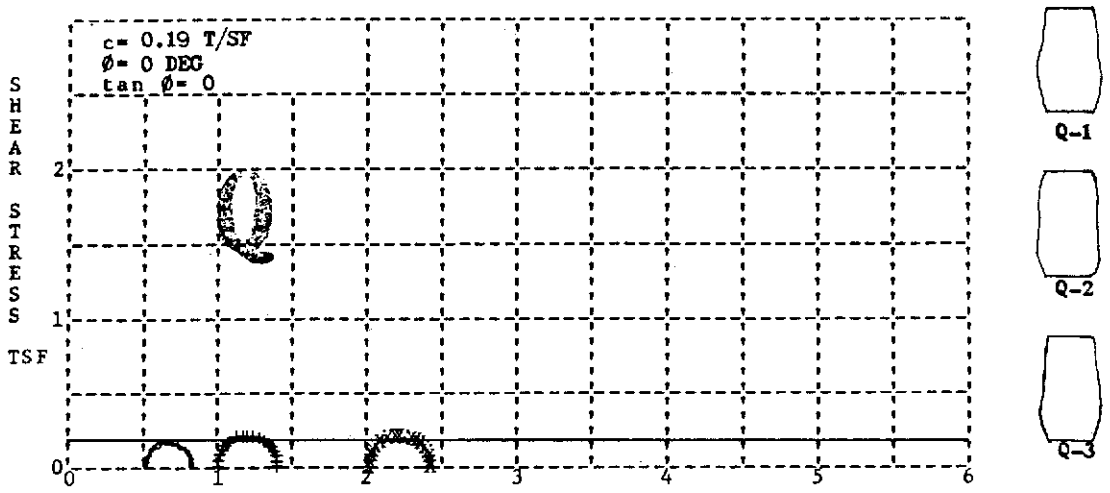
PROJECT: **Davenport, Iowa (Local)**
NCR-IA-75-B36-ED

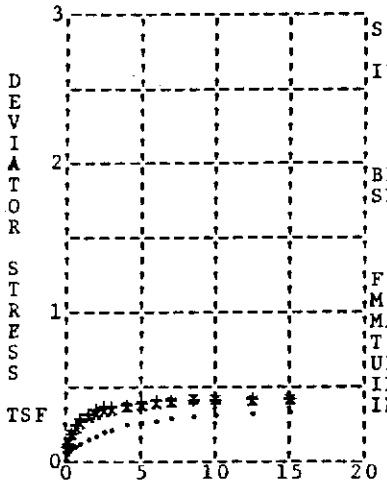
BORING NO: **D-5-1** SAMPLE NO: **1**

DEPTH: **5.0 - 7.4**

MRD LAB NO: **74/362** DATE **5 DEC 1974**

TRIAXIAL COMPRESSION TEST REPORT
FIGURE 1





SPECIMEN NUMBER		1	2	3
INITIAL	WATER CONTENT,%	43.4	43.8	44.9
	DRY DENSITY,PCF	74.1	75.0	73.4
	SATURATION,%	93	96	95
	VOID RATIO	1.24	1.21	1.26
BEFORE SHEAR	WATER CONTENT,%	43.1	43.3	44.5
	DRY DENSITY,PCF	74.5	75.0	73.4
	SATURATION,%	93	95	94
	VOID RATIO	1.23	1.21	1.26
FINAL BACK PRESSURE,TSF		.00	.00	.00
MINOR PRINCIPAL STRESS,TSF		.50	1.00	2.00
MAX DEVIATOR STRESS,TSF		.32	.41	.42
TIME TO MAX DEV,MIN		30	26	20
ULT DEVIATOR STRESS,TSF		.32	.41	.41
INITIAL DIAMETER,IN		1.391	1.392	1.406
INITIAL HEIGHT,IN		2.985	2.981	2.953

CONTROLLED-STRAIN TEST

DESCRIPTION OF SPECIMENS: **Lean clay, CL**

LL **46** PL **22** PI **24** Gs= 2.66 TYPE SPECIMEN: UNDISTURBED TYPE TEST Q

REMARKS: MACHINE PRINT OUT
FORMAT AFTER ENG FORM 2089

Gray lean clay with 3/4" sand seams at a approx. 5" to 7" centers within sample. A 2" layer of relatively undecayed bark and large roots was located near top. Plastic structure, soft consistency, slightly sensitive. Med. toughness at PL, dull shine, sluggish shake reaction.

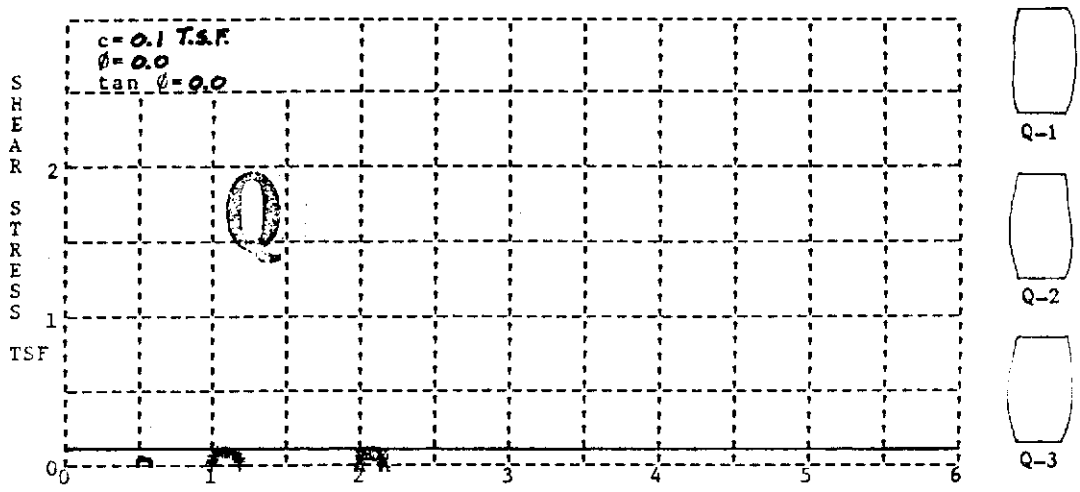
PROJECT: **Davenport, Iowa (Local)**
NCR-IA-75-B36-ED

BORING NO: **D-5-1** SAMPLE NO: **2**

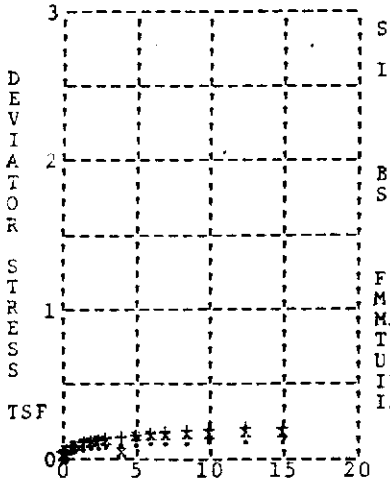
DEPTH: **7.5 - 9.9**

MRD LAB NO: **74/362** DATE **5 DEC 1974**

TRIAXIAL COMPRESSION TEST REPORT
FIGURE 3



NORMAL STRESS, TSF



AXIAL STRAIN, %

SPECIMEN NUMBER		1	2	3
INITIAL	WATER CONTENT, %	53.4	45.2	48.6
	DRY DENSITY, PCF	65.4	73.8	69.4
	SATURATION, %	93	97	93
	VOID RATIO	1.53	1.24	1.38
BEFORE SHEAR	WATER CONTENT, %	53.1	45.1	48.3
	DRY DENSITY, PCF	65.4	73.8	69.4
	SATURATION, %	92	96	93
	VOID RATIO	1.53	1.24	1.38
FINAL BACK PRESSURE, TSF		.00	.00	.00
MINOR PRINCIPAL STRESS, TSF		.50	1.00	2.00
MAX DEVIATOR STRESS, TSF		.11	.21	.19
TIME TO MAX DEV, MIN		32	30	37
ULT DEVIATOR STRESS, TSF		.11	.21	.19
INITIAL DIAMETER, IN		2.820	2.760	2.800
INITIAL HEIGHT, IN		5.960	6.190	5.930

CONTROLLED-STRAIN TEST

DESCRIPTION OF SPECIMENS:

LL 45 PL 23 PI 22 Gs= 2.65 TYPE SPECIMEN: UNDISTURBED TYPE TEST Q

REMARKS: MACHINE PRINT OUT
FORMAT AFTER ENG FORM 2089

Torvane 0.12 T/SF.
Dark gray clay with random gravel sizes. Plastic structure, soft consistency, slightly sensitive. Medium toughness at PL, gloss shine, no shake reaction.

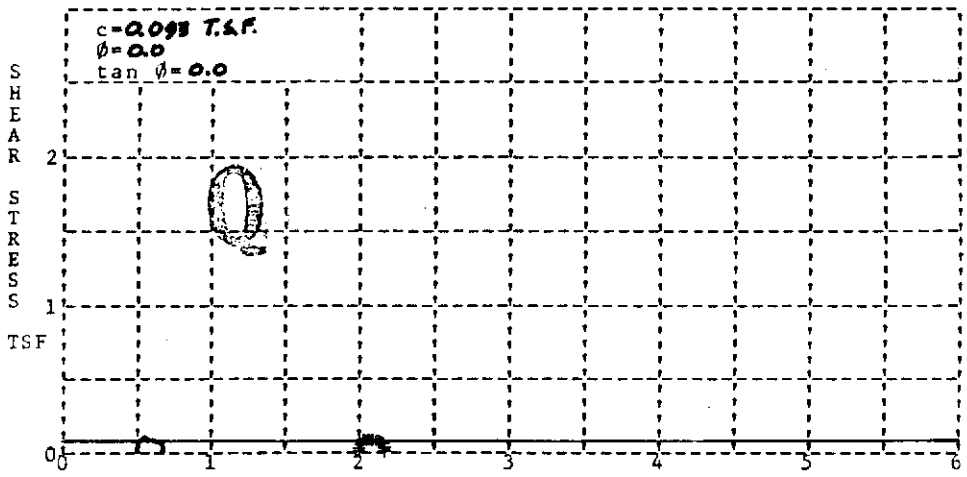
PROJECT: Davenport Local NCR-IA-76-BO1-ED-F

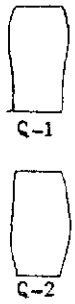
BORING NO: D-31 SAMPLE NO: 1

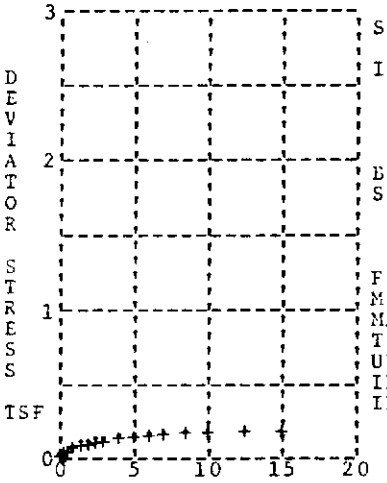
DEPTH: 10.8 - 13.0

MRD LAB NO: 75/189 DATE 11 NOV 1975

TRIAXIAL COMPRESSION TEST REPORT
FIGURE 1







SPECIMEN NUMBER		1	2
INITIAL	WATER CONTENT,%	62.1	64.1
	DRY DENSITY,PCF	61.7	60.9
	SATURATION,%	98	99
	VOID RATIO	1.69	1.72
BEFORE SHEAR	WATER CONTENT,%	62.0	64.1
	DRY DENSITY,PCF	61.7	60.9
	SATURATION,%	98	99
	VOID RATIO	1.69	1.72
FINAL BACK PRESSURE,TSF		.00	.00
MINOR PRINCIPAL STRESS,TSF		.50	2.00
MAX DEVIATOR STRESS,TSF		.19	.19
TIME TO MAX DEV,MIN		33	26
ULT DEVIATOR STRESS,TSF		.19	.18
INITIAL DIAMETER,IN		2.789	2.800
INITIAL HEIGHT,IN		6.340	6.180

CONTROLLED-STRAIN TEST

DESCRIPTION OF SPECIMENS:

LL 66 PL 33 PI 33 Gs= 2.66 TYPE SPECIMEN: UNDISTURBED TYPE TEST Q

REMARKS: MACHINE PRINT OUT
FORMAT AFTER ENG FORM 2089

Torvane 0.10 T/SF.
Gray clay with random sand particles. Plastic structure, soft consistency, slightly sensitive. Medium toughness at PL, dull shine, no shake reaction.

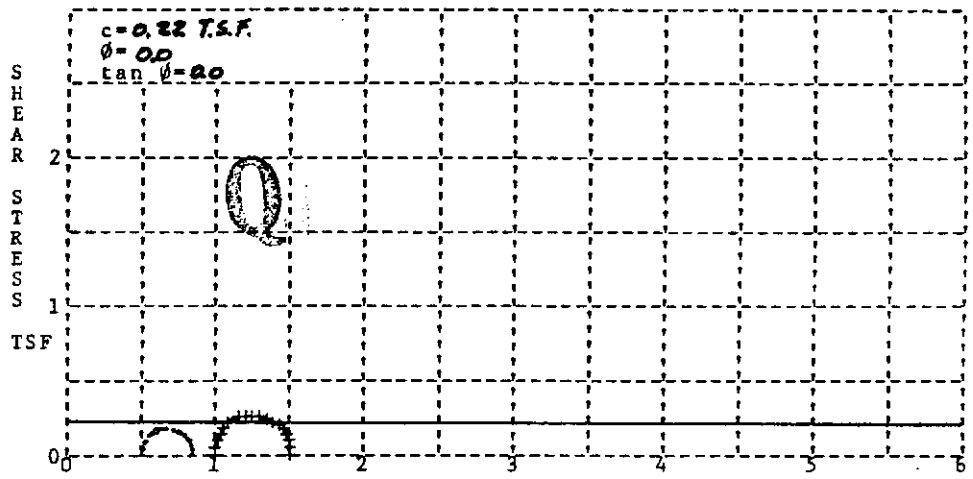
PROJECT: Davenport Local NCR-IA-76-B01-ED

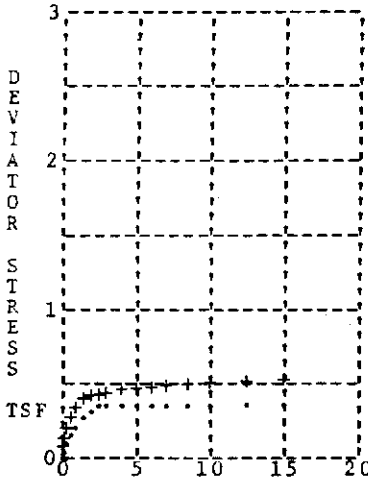
BORING NO: D-31 SAMPLE NO: 3

DEPTH: 13.3 - 15.6

MRD LAB NO: 75/189 DATE 11 NOV 1975

TRIAXIAL COMPRESSION TEST REPORT
FIGURE 2





SPECIMEN NUMBER		1	2
INITIAL	WATER CONTENT, %	39.2	37.7
	DRY DENSITY, PCF	77.1	82.2
	SATURATION, %	92	100+
	VOID RATIO	1.12	.99
BEFORE SHEAR	WATER CONTENT, %	39.0	37.6
	DRY DENSITY, PCF	77.2	82.9
	SATURATION, %	92	100+
	VOID RATIO	1.12	.97
FINAL BACK PRESSURE, TSF		.00	.00
MINOR PRINCIPAL STRESS, TSF		.50	1.00
MAX DEVIATOR STRESS, TSF		.36	.53
TIME TO MAX DEV, MIN		30	25
ULT DEVIATOR STRESS, TSF		.36	.53
INITIAL DIAMETER, IN		2.840	2.790
INITIAL HEIGHT, IN		6.050	5.810

CONTROLLED-STRAIN TEST

DESCRIPTION OF SPECIMENS:

LL 54 PL 27 PI 27 Gs= 2.62 TYPE SPECIMEN: UNDISTURBED TYPE TEST Q

REMARKS: MACHINE PRINT OUT
FORMAT AFTER ENG FORM 2089
Torvane 0.20 T/SF.
Gray clay with traces of fine sand and silt. Definite organic odor. Plastic structure, soft consistency, slightly sensitive. Medium toughness at PL, dull shine, sluggish shake reaction. Unable to trim third test specimen.

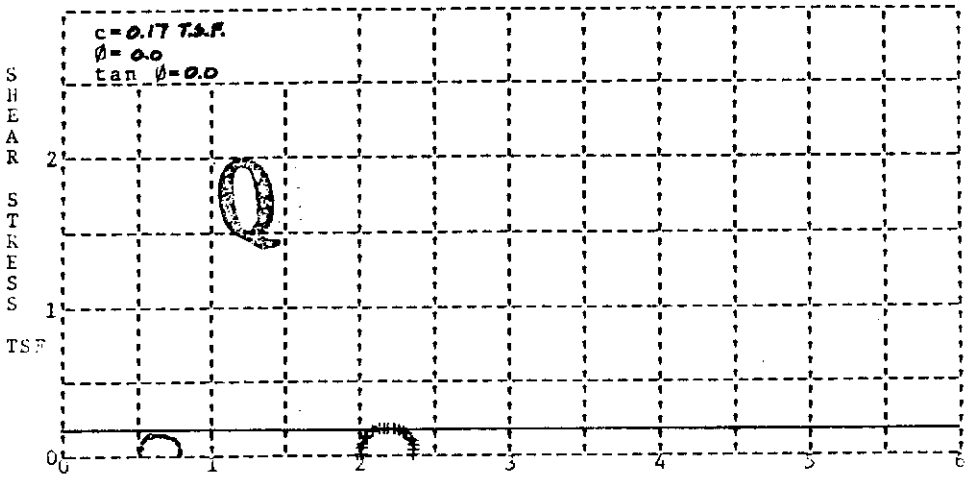
PROJECT: Davenport Local NCR-IA-76-B01-ED-F

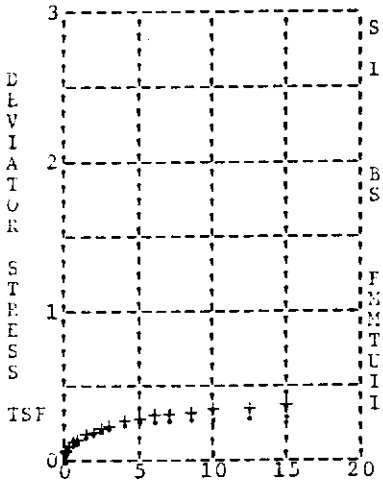
BORING NO: D-35 SAMPLE NO: 1

DEPTH: 4.2 - 6.6

MRD LAB NO: 75/189 DATE 11 NOV 1975

TRIAXIAL COMPRESSION TEST REPORT
FIGURE 3





SPECIMEN NUMBER		1	2
INITIAL	WATER CONTENT, %	41.6	41.0
	DRY DENSITY, PCF	76.8	81.1
	SATURATION, %	97	100+
	VOID RATIO	1.12	1.01
BEFORE SHEAR	WATER CONTENT, %	41.4	40.8
	DRY DENSITY, PCF	76.8	81.1
	SATURATION, %	96	100+
	VOID RATIO	1.12	1.01
FINAL BACK PRESSURE, TSF		.00	.00
MINOR PRINCIPAL STRESS, TSF		.50	2.00
MAX DEVIATOR STRESS, TSF		.29	.37
TIME TO MAX DEV, MIN		30	30
ULT DEVIATOR STRESS, TSF		.29	.37
INITIAL DIAMETER, IN.		2.820	2.760
INITIAL HEIGHT, IN.		6.310	6.140

CONTROLLED-STRAIN TEST

DESCRIPTION OF SPECIMENS:

LL 43 PL 23 PI 20 Gs= 2.61 TYPE SPECIMEN: UNDISTURBED TYPE TEST Q

REMARKS: MACHINE PRINT OUT FORMAT AFTER ENG FORM 2089

Torvane 0.22 T/SF.

Dark gray clay, trace of fine sand. Brittle structure, soft consistency, slightly sensitive. Medium toughness at PL, dull shine, sluggish shake reaction.

PROJECT: Davenport Local NCR-IA-76-B01-ED-F

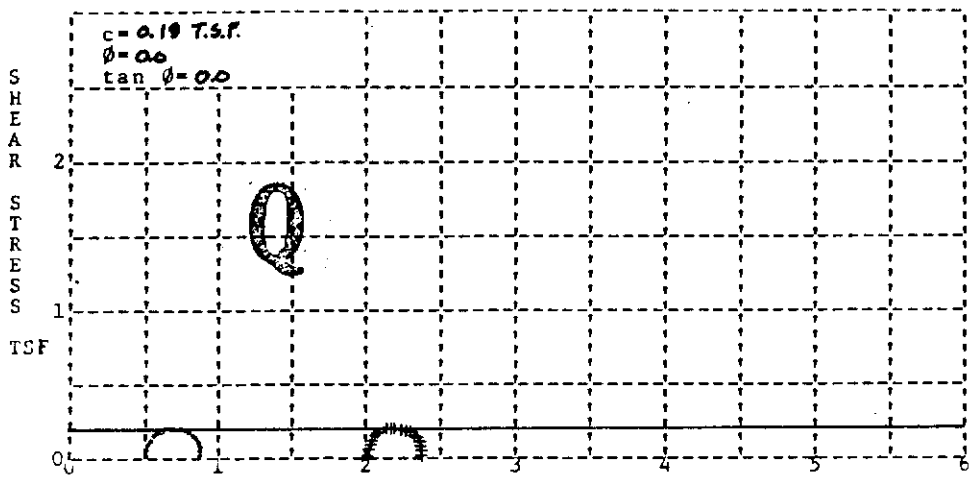
BORING NO: D-35 SAMPLE NO: 3

DEPTH: 6.7 - 9.2

MRD LAB NO:75/189 DATE 11 NOV 1975

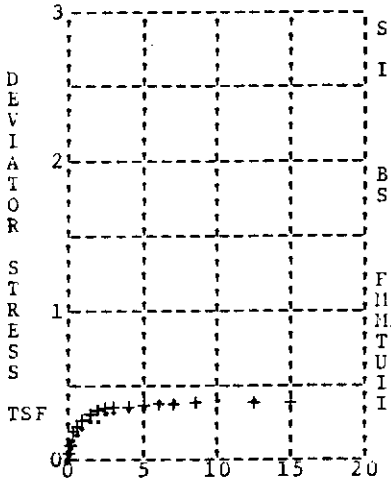
TRIAXIAL COMPRESSION TEST REPORT

FIGURE 4



Q-1

Q-2



SPECIMEN NUMBER		1	2
INITIAL	WATER CONTENT, %	48.2	55.1
	DRY DENSITY, PCF	70.1	65.3
	SATURATION, %	95	96
	VOID RATIO	1.32	1.49
BEFORE SHEAR	WATER CONTENT, %	48.1	55.0
	DRY DENSITY, PCF	70.5	67.0
	SATURATION, %	96	100+
	VOID RATIO	1.31	1.43
FINAL BACK PRESSURE, TSF		.00	.00
MINOR PRINCIPAL STRESS, TSF		.50	2.00
MAX DEVIATOR STRESS, TSF		.39	.38
TIME TO MAX DEV, MIN		31	25
ULT DEVIATOR STRESS, TSF		.39	.38
INITIAL DIAMETER, IN.		2.790	2.790
INITIAL HEIGHT, IN.		6.270	6.160

CONTROLLED-STRAIN TEST

DESCRIPTION OF SPECIMENS:

LL 60 PL 30 PI 30 Gs= 2.61 TYPE SPECIMEN: UNDISTURBED TYPE TEST Q

REMARKS: MACHINE PRINT OUT FORMAT AFTER ENG FORM 2089

Torvane 0.27 T/SF.

Gray clay with trace of silt. Brittle structure, soft consistency, slightly sensitive. Medium toughness at PL, dull shine, no shake reaction.

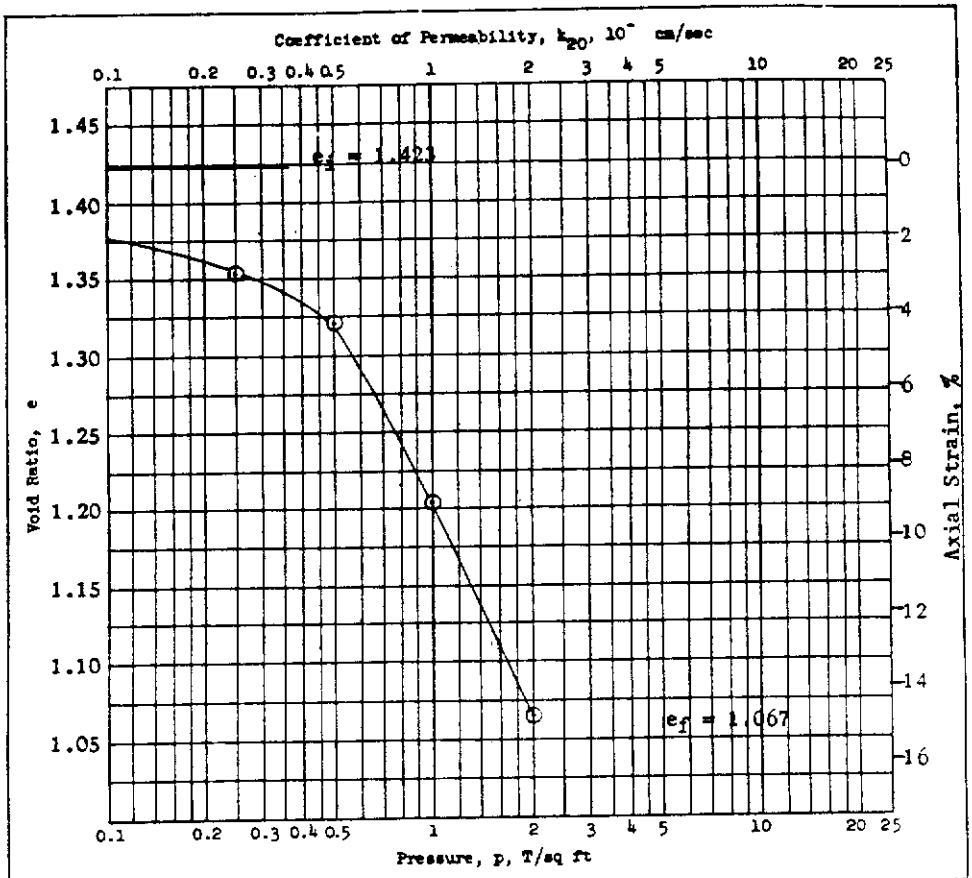
PROJECT: Davenport Local NCR-IA-76-B01-ED-F

BORING NO: D-35 SAMPLE NO: 5

DEPTH: 9.3 - 11.5

MRD LAB NO: 75/189 DATE 11 NOV 1975

TRIAXIAL COMPRESSION TEST REPORT
FIGURE 5

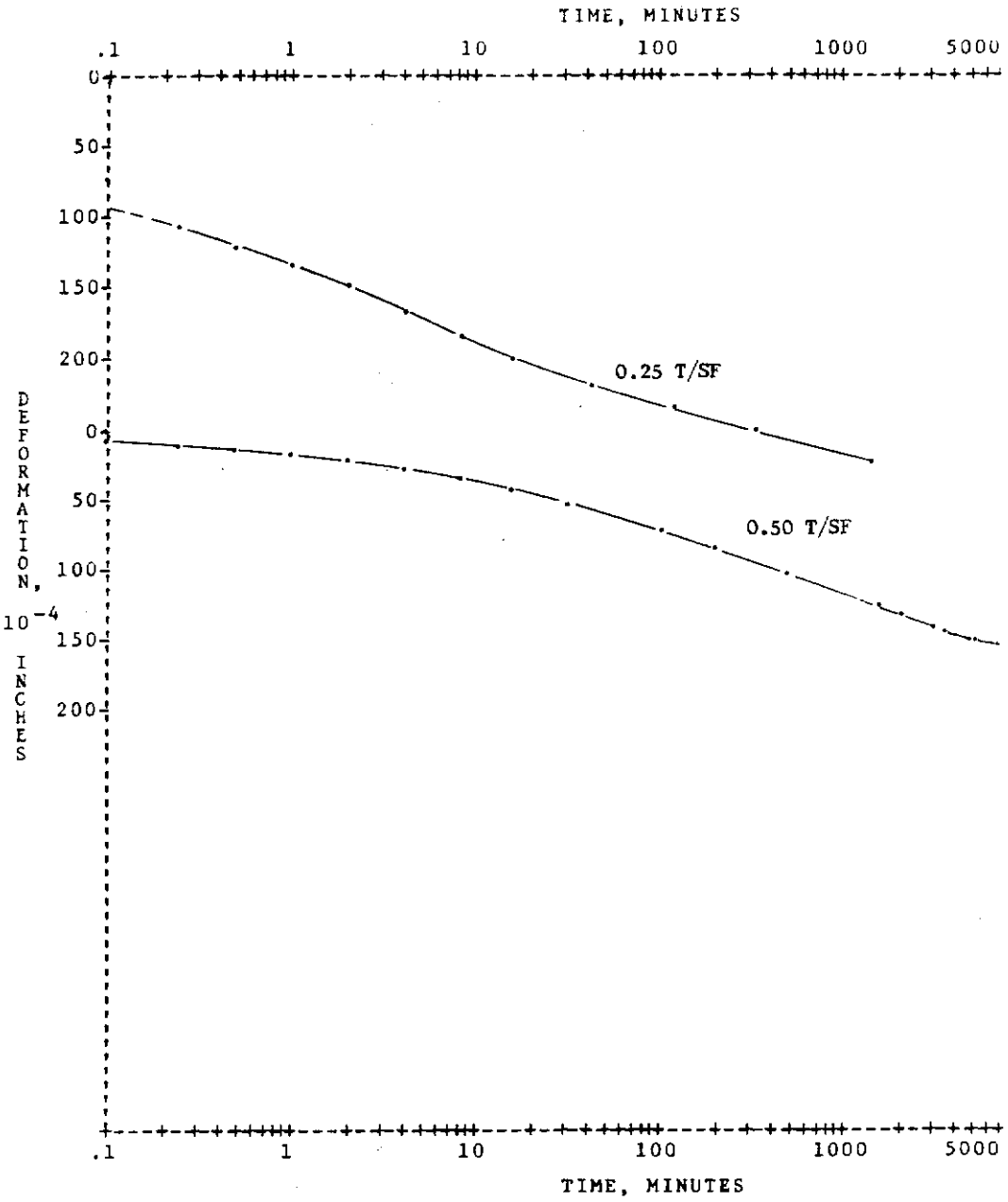


Type of Specimen: Undisturbed		Before Test		After Test	
Diam 4.28 in.	Ht 1.00 in.	Water Content, w_o	51.8 %	w_f	40.9 %
Overburden Pressure, Po T/sq ft		Void Ratio, e_o	1.423	e_f	1.067
Preconsol. Pressure, p_c T/sq ft		Saturation, S_o	95 %	S_f	100 %
Compression Index, C_c		Dry Density, γ_d	67.2 lb/ft^3		
Classification		k_{20} at e_o =	× 10^{-} cm/sec		
LL 60	G_s 2.61	Project: Davenport Local NCR-IA-76-B01-ED-F			
PL 30	D_{10}	MRD Lab. No. 75/189			
Remarks		Area			
		Boring No. D-35		Sample No. 5	
		Depth 9.3 - 11.5		Date 11 NOV 1975	
		CONSOLIDATION TEST REPORT			

ENG FORM 2090 1 MAY 63 PREVIOUS EDITIONS ARE OBSOLETE.

Figure 6

PLATE B-11



PROJECT: Davenport Local NCR-IA-76-B01-ED-F

MRD LABORATORY NO: 75/189

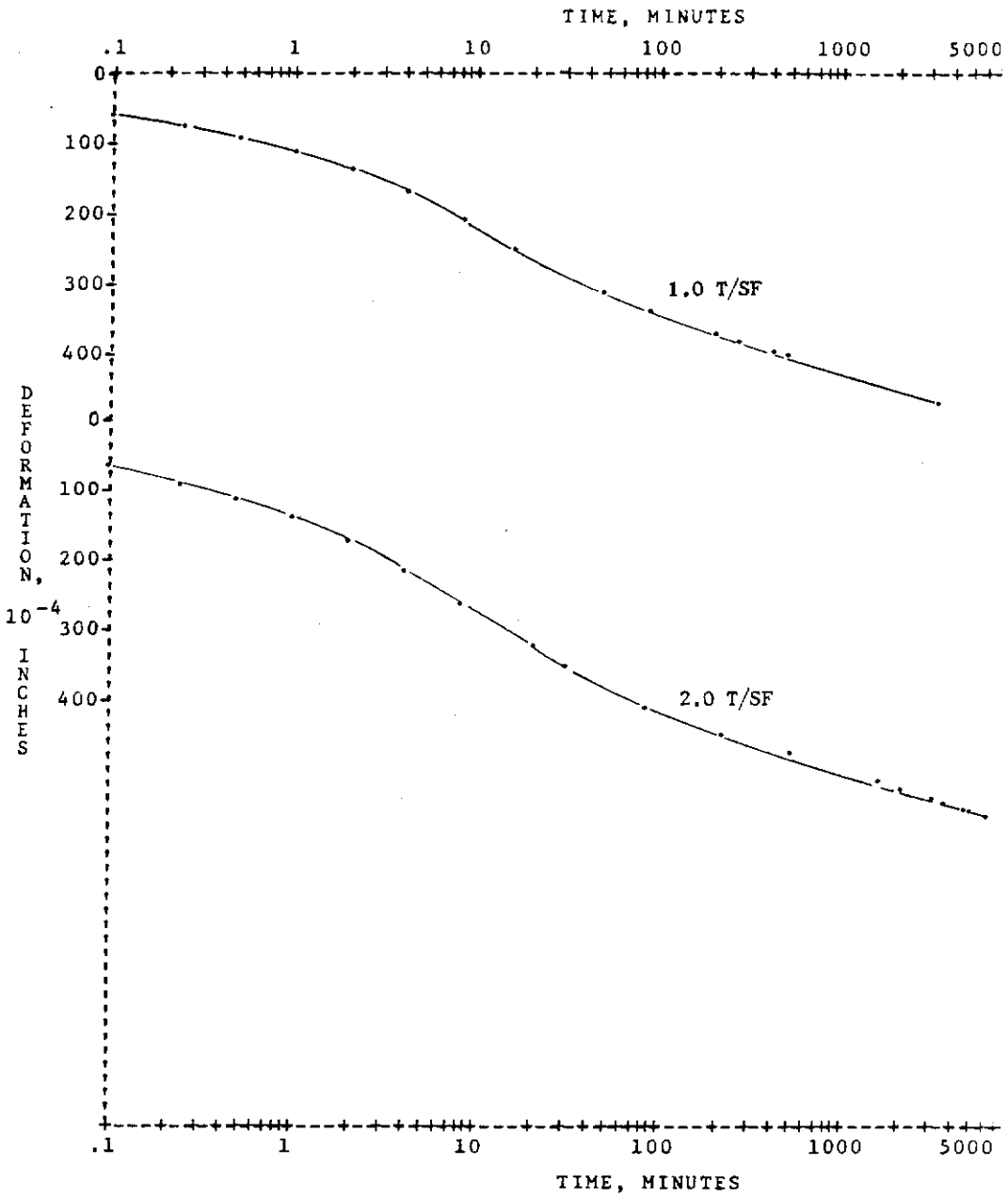
BORING NO: D-35 SAMPLE NO: 5 DEPTH: 9.3 - 11.5 DATE: 11 NOV 1975

CONSOLIDATION TEST -- TIME CURVES

MACHINE PRINT OUT
FORMAT AFTER ENG FORM 2088

FIGURE 7

PLATE B-12



PROJECT: Davenport Local NCR-IA-76-B01-ED-F

MRD LABORATORY NO: 75/189

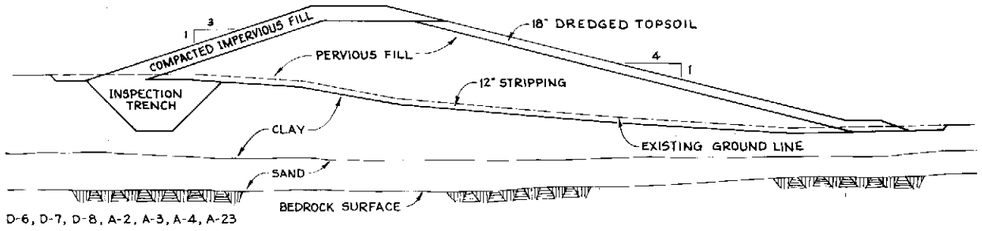
BORING NO: D-35 SAMPLE NO: 5 DEPTH: 9.3 - 11.5 DATE: 11 NOV 1975

CONSOLIDATION TEST -- TIME CURVES

MACHINE PRINT OUT
FORMAT AFTER ENG FORM 2088

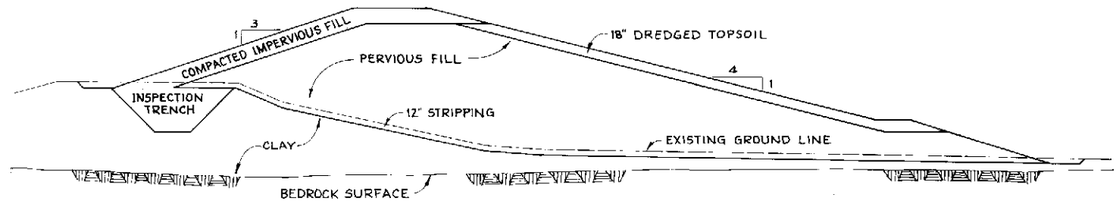
FIGURE 8

PLATE B-13



D-6, D-7, D-8, A-2, A-3, A-4, A-23

STA. 190+00 TO STA. 170+00



D-2, D-3, D-4, D-5, AA-1, AA-18

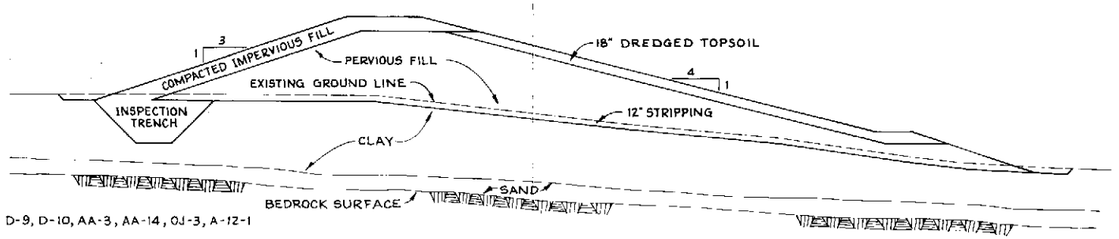
STA. 221+00 TO STA. 190+00

AA-20, L-1, D-1, D-S, D-S-1, D-31

STA. 245+20 TO STA. 221+00

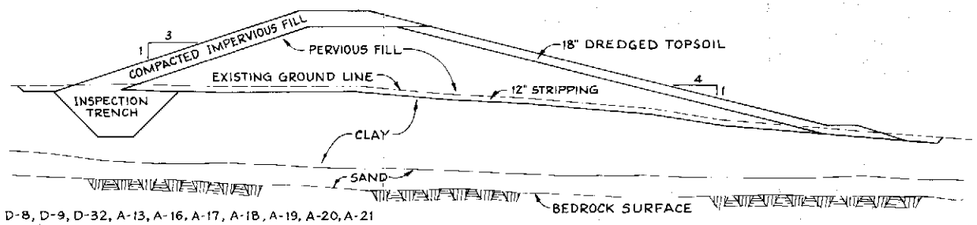
I-1, I-2, I-3, I-4, D-18, D-19

STA. 256+40 TO STA. 245+20



D-9, D-10, AA-3, AA-14, OJ-3, A-12-1

STA. 160+00 TO STA. 150+00



D-8, D-9, D-32, A-13, A-16, A-17, A-18, A-19, A-20, A-21

STA. 170+00 TO STA. 160+00

REVISION	DATE	DESCRIPTION	BY

CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
ROCK ISLAND, ILLINOIS

DRAWN BY:
TRACED BY:
CHECKED BY:
SUBMITTED:
CHIEF, DESIGN BRANCH
APPROVED:
CHIEF, ENGINEERING DIVISION
DATE:

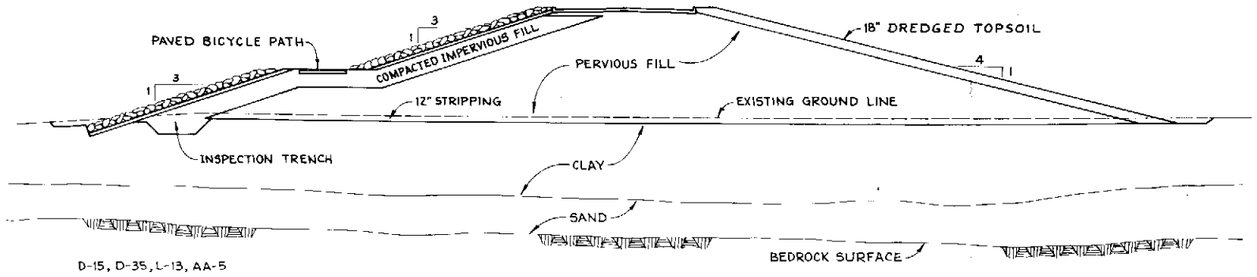
MISSISSIPPI RIVER
DAVENPORT, IOWA
LOCAL FLOOD PROTECTION

TYPICAL LEVEE SECTIONS AND SUB-SURFACE CONDITIONS I

NO SCALE

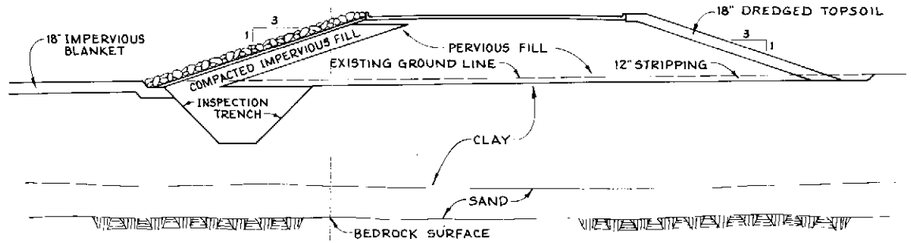
SHEET DRAWING NO

INVITATION NO. DACW25 - -8-



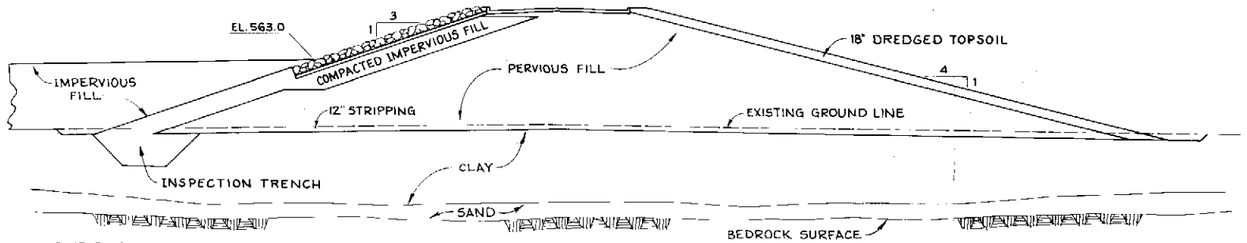
D-15, D-35, L-13, AA-5

STA. 125+00 TO STA. 117+00



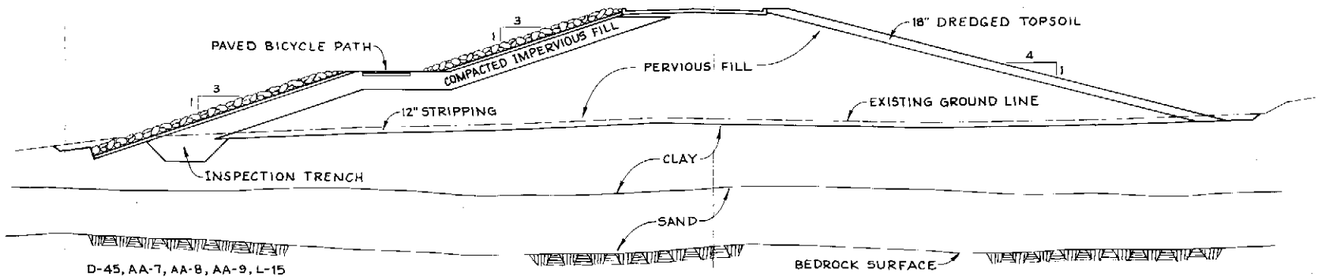
D-23, D-24, 5, 6, 7

STA. 75+00 TO STA. 46+00



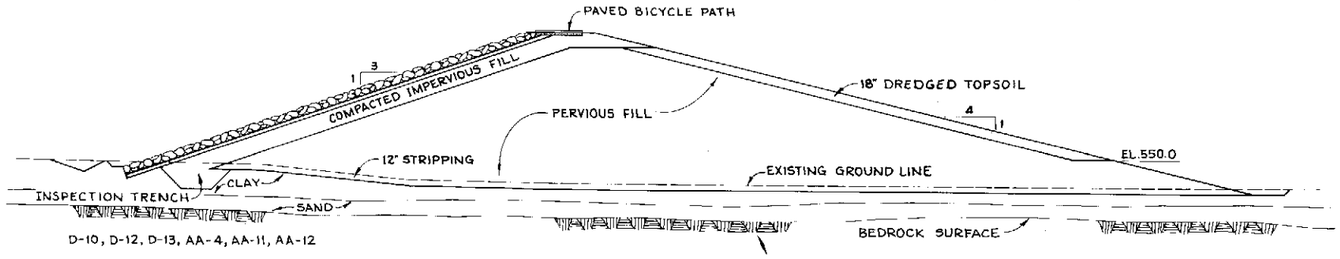
D-13, D-14

STA. 134+00 TO STA. 125+00



D-45, AA-7, AA-8, AA-9, L-15

STA. 117+00 TO STA. 98+00



D-10, D-12, D-13, AA-4, AA-11, AA-12

STA. 150+00 TO STA. 134+00

REVISION	DATE	DESCRIPTION	BY

CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
ROCK ISLAND, ILLINOIS

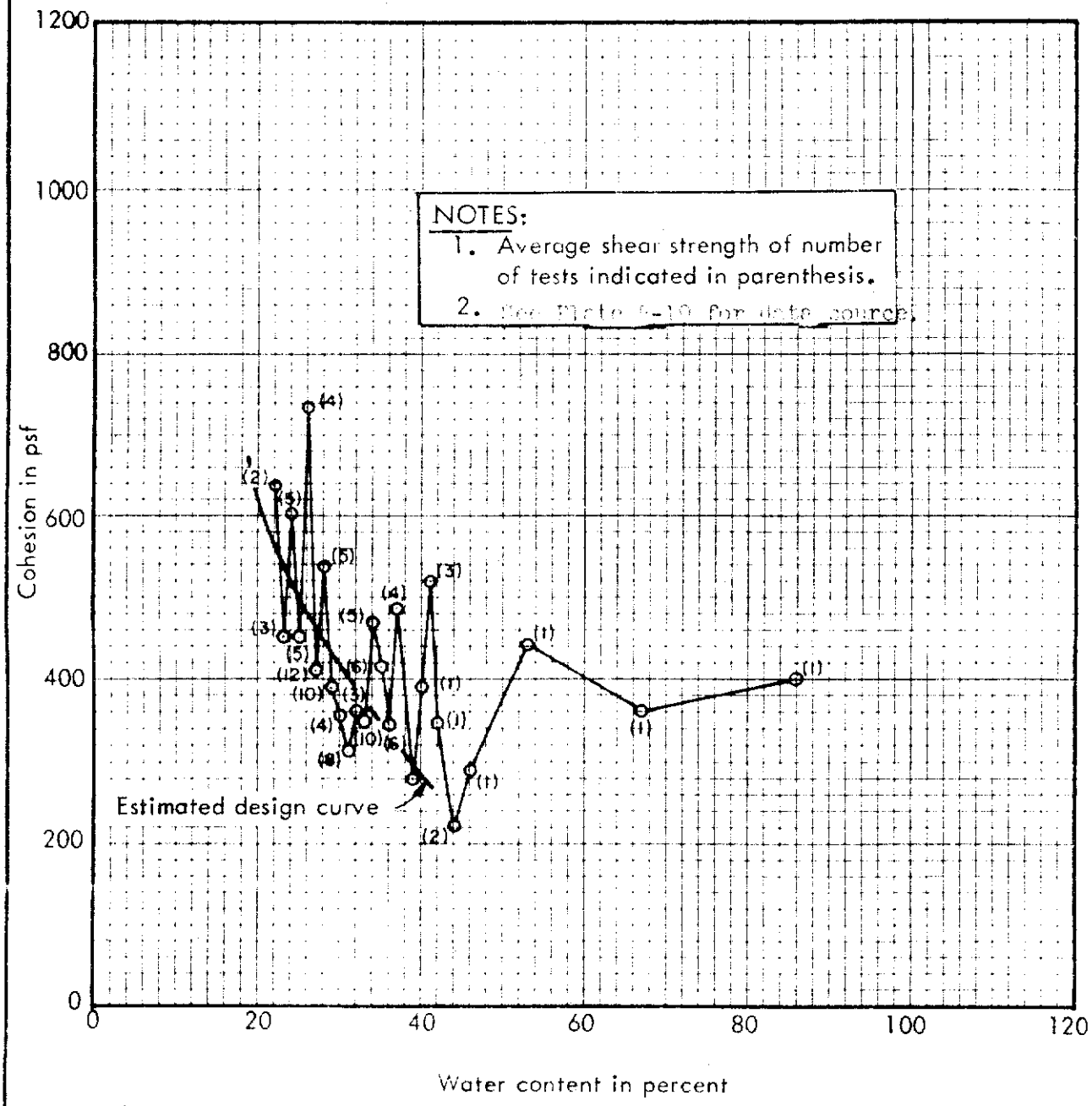
DRAWN BY:
TRACED BY:
CHECKED BY:
SUBMITTED:
CHIEF, DESIGN BRANCH
APPROVED:
CHIEF, ENGINEERING DIVISION
DATE:

MISSISSIPPI RIVER
DAVENPORT, IOWA
LOCAL FLOOD PROTECTION

TYPICAL LEVEE SECTIONS AND SUB-SURFACE CONDITIONS
II

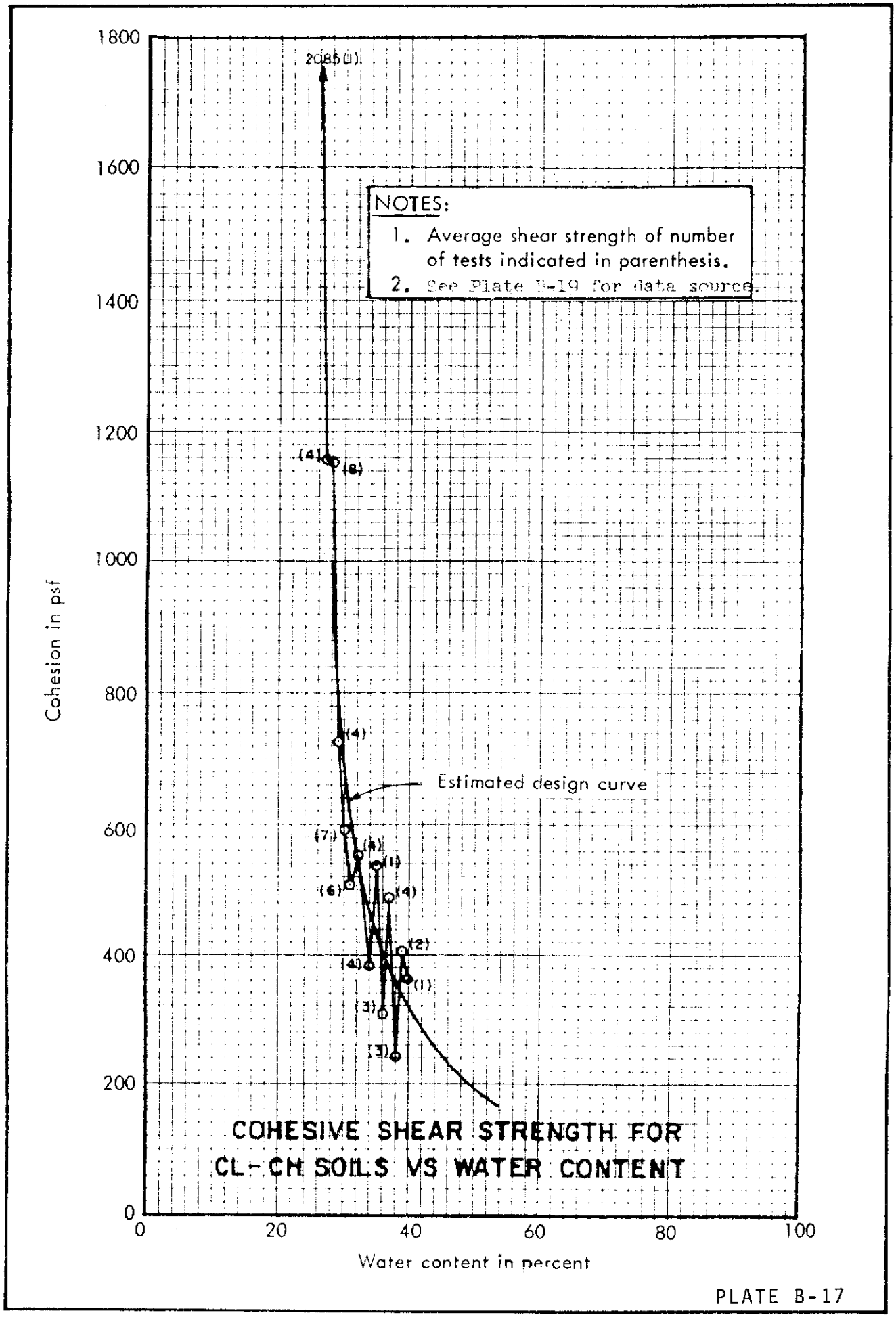
NO SCALE

SHEET DRAWING NO
INVITATION NO. DACW25- -B-



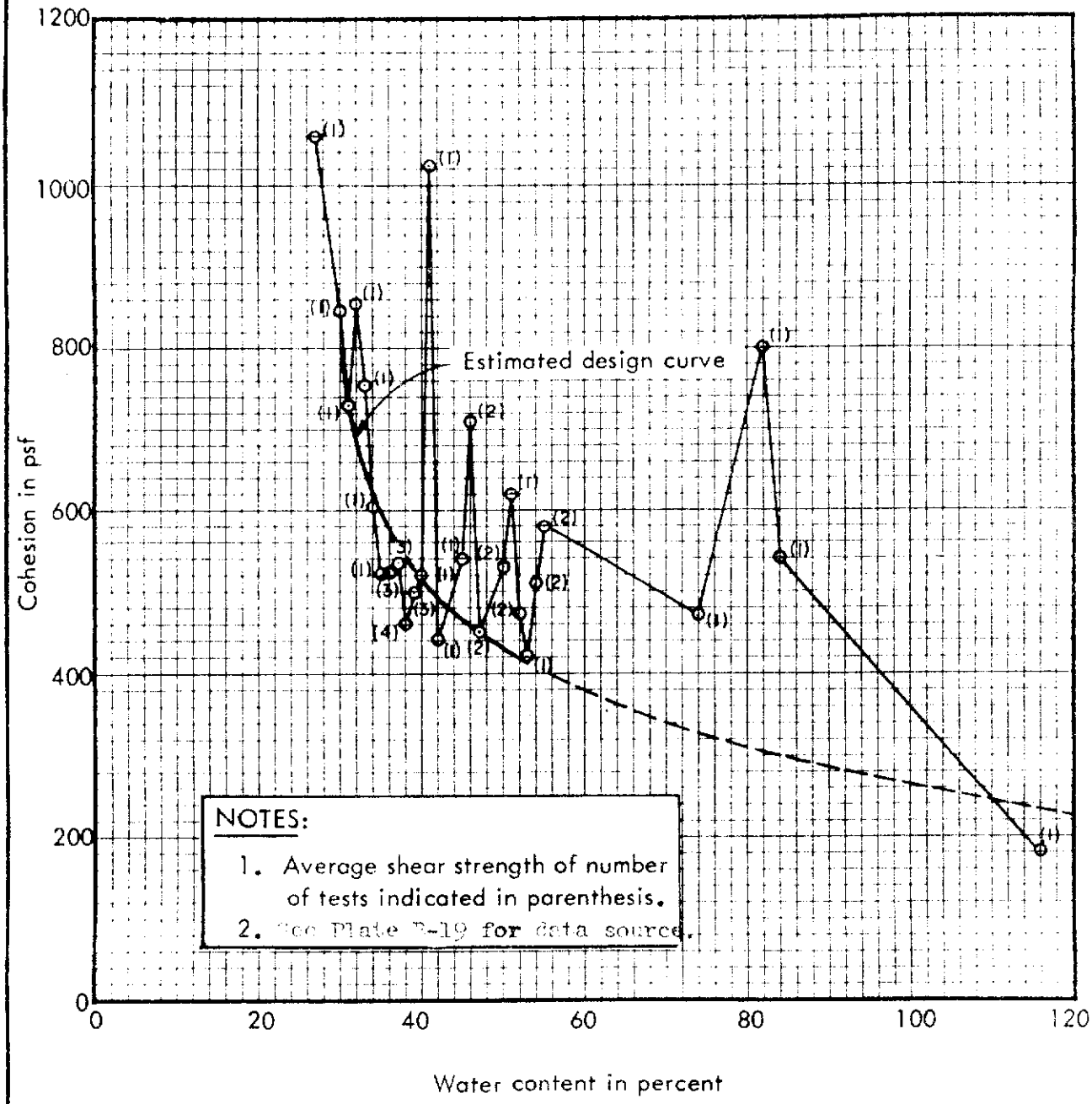
COHESIVE SHEAR STRENGTH
FOR CL SOILS VS
WATER CONTENT

PLATE B-16



COHESIVE SHEAR STRENGTH FOR CL-CH SOILS VS WATER CONTENT

PLATE B-17

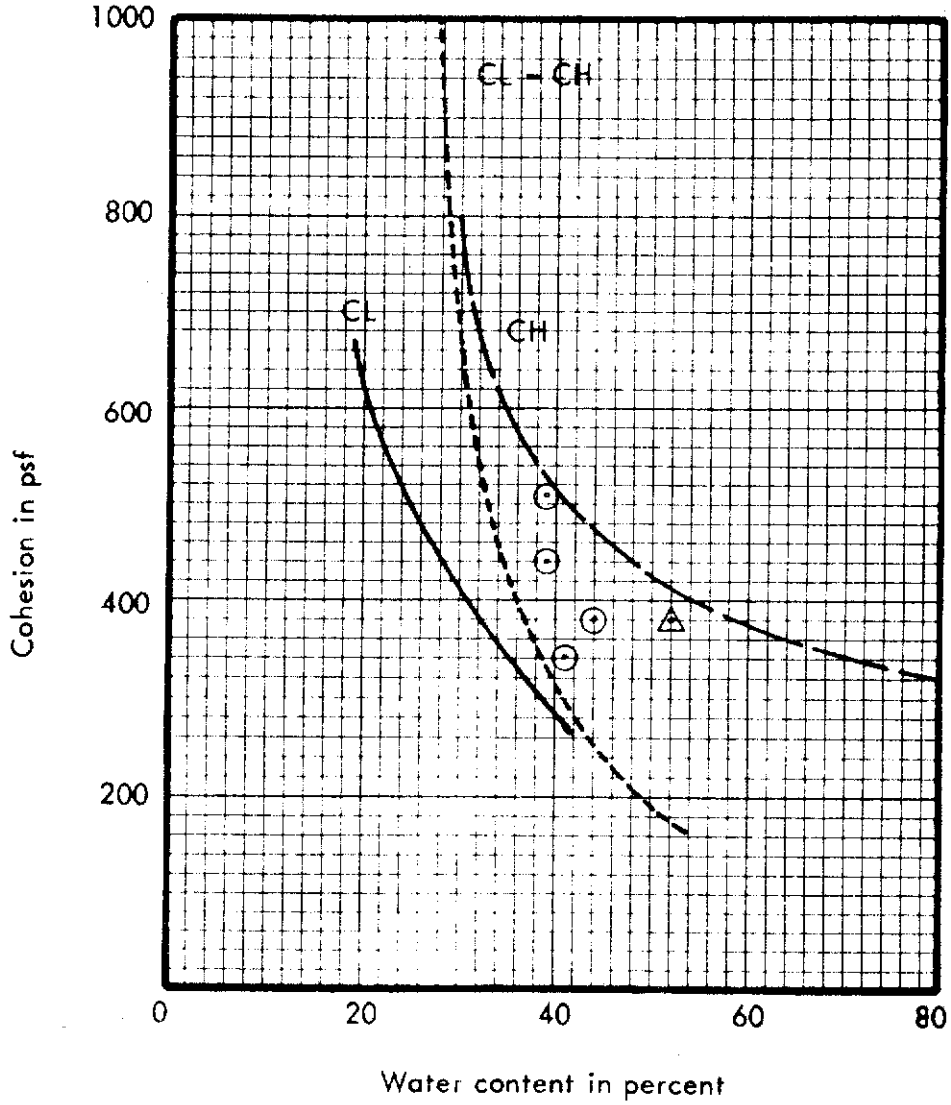


COHESIVE SHEAR STRENGTH
FOR CH SOILS VS
WATER CONTENT

PLATE B-18

⊙ - DAVENPORT (CL-CH)

△ - DAVENPORT (CH)

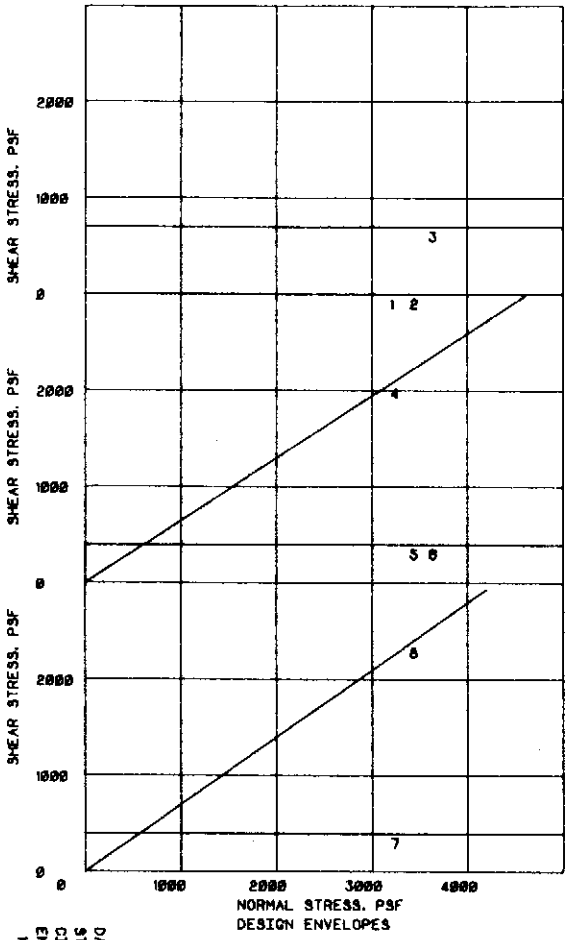


NOTE:

1. Cohesive shear strength curves derived from shear strength data supplied by Rock Island District for Mississippi River alluvial soils in the District, and shown on Plates B-16, B-17 and B-18.

COHESIVE SHEAR STRENGTH VS WATER CONTENT

PLATE B-19

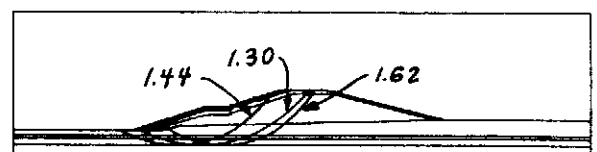


MATERIAL		SOIL WT. LBS./CUFT		SHEAR STRENGTH							
				Q		R		S		(R-S)/2	
		MOIST	SAT.	PHI DEGREES	COHESION PSF	PHI DEGREES	COHESION PSF	PHI DEGREES	COHESION PSF	PHI DEGREES	COHESION PSF
RIPRAP 18-INCH BLANKET	1	100.00	100.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
BEDDING MATERIAL 6-INCH BLANKET	2	120.00	120.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
COMPACTED IMPER EMBA EL 546 TO EL 571.6	3	125.00	125.00	0.00	700.00	0.00	0.00	0.00	0.00	0.00	0.00
HYDRAULIC SAND FILL EL 546 TO EL 571.6	4	120.00	120.00	33.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
CLAY FOUNDATION EL 543 TO EXIS GRND	5	110.00	110.00	0.00	400.00	0.00	0.00	0.00	0.00	0.00	0.00
CLAY FOUNDATION EL 541 TO EL 543	6	110.00	110.00	0.00	400.00	0.00	0.00	0.00	0.00	0.00	0.00
CLAY FOUNDATION EL 537 TO EL 541	7	110.00	110.00	0.00	400.00	0.00	0.00	0.00	0.00	0.00	0.00
SAND FOUNDATION EL 537 TO LIMESTONE	8	135.00	135.00	35.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

CIRCLE FAILURE SURFACE
RESULTS BY CYBER-175
COMPUTER PROGRAM 741-VV-F424A
TANGENT TO ELEV 537.00
TRIAL ARCS

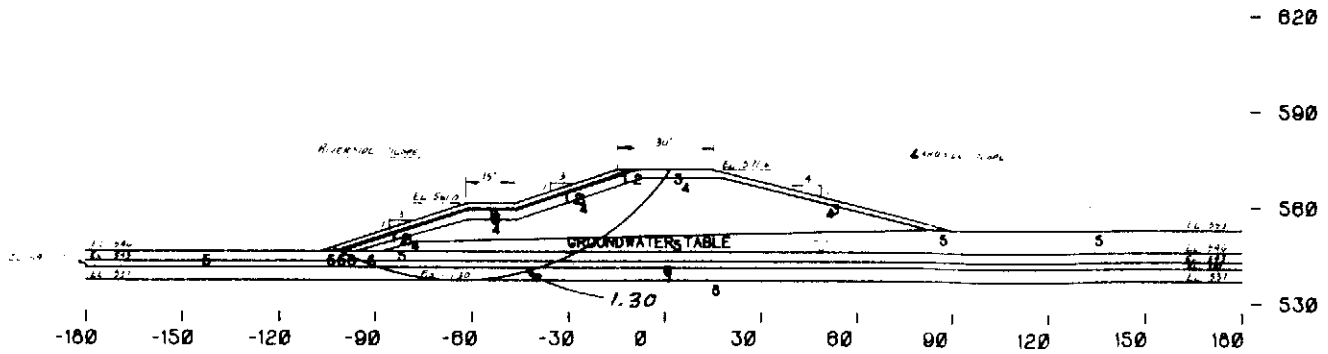
RADIUS OF CIRCLE	CENTER OF CIRCLE DISTANCE FROM ℄	CENTER OF CIRCLE ELEV	F.S. Q Y= 0.00	F.S. Q Y=0.05
80.00	-64.00	617.00	1.30	1.04
70.00	-79.00	607.00	1.44	
63.00	-50.00	600.00	1.62	
63.00	-60.00	600.00	1.42	
95.00	-59.00	632.00	1.33	
85.00	-63.00	622.00	1.30	

CRITICAL FAILURE SURFACE



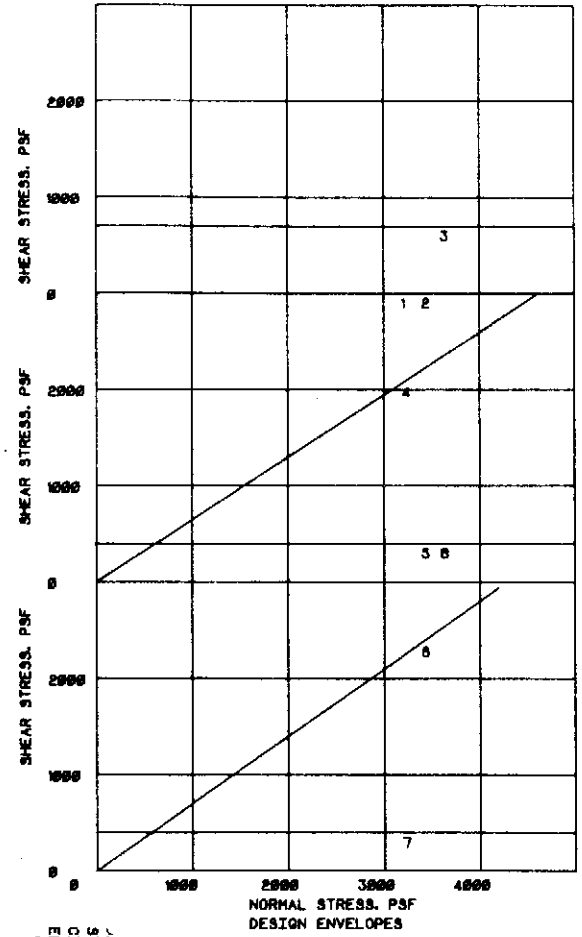
RANGE OF TRIAL FAILURE SURFACES

NOTES
1- ANALYSES WERE RUN ACCORDING TO EM 1110-2-1902 DATED APRIL 1970
2- THE SIDE EARTH FORCE DIRECTION WAS TAKEN AS THE AVERAGE OF THE EMBANKMENT SLOPES IMMEDIATELY ADJACENT TO THE SLICE INTERFACE
3- PSI-SIESMIC COEFFICIENT USED IN ANALYSES



DAVENPORT, LOCAL PROTECTION
STA. 113+00 RIVERSIDE SLOPE
CIRCLE SLOPE STABILITY ANALYSIS
END OF CONSTRUCTION CONDITION
10 MAY 1978 GJM

PLATE B-20

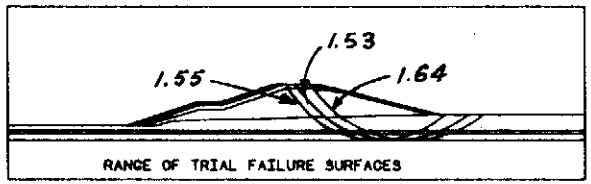


MATERIAL		SOIL WT. LBS./CUFT		SHEAR STRENGTH Q		R		S		(R+S)/2	
		MOIST	SAT.	PHI DEGREES	COHESION PSF	PHI DEGREES	COHESION PSF	PHI DEGREES	COHESION PSF	PHI DEGREES	COHESION PSF
RIPRAP 18-INCH BLANKET	1	150.00	150.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
BEDDING MATERIAL 6-INCH BLANKET	2	120.00	120.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
COMPACTED IMPER EMBA EL 546 TO EL 571.6	3	125.00	125.00	0.00	700.00	0.00	0.00	0.00	0.00	0.00	0.00
HYDRAULIC SAND FILL EL 546 TO EL 571.6	4	120.00	120.00	33.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
CLAY FOUNDATION EL 543 TO EXIS GRND	5	110.00	110.00	0.00	400.00	0.00	0.00	0.00	0.00	0.00	0.00
CLAY FOUNDATION EL 541 TO EL 543	6	110.00	110.00	0.00	400.00	0.00	0.00	0.00	0.00	0.00	0.00
CLAY FOUNDATION EL 537 TO EL 541	7	110.00	110.00	0.00	400.00	0.00	0.00	0.00	0.00	0.00	0.00
SAND FOUNDATION EL 537 TO LIMESTONE	8	135.00	135.00	35.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

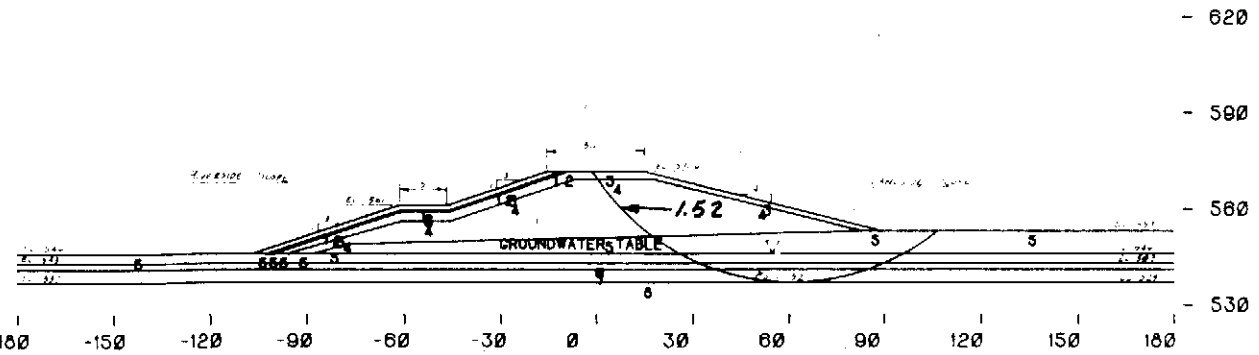
CIRCLE FAILURE SURFACE
RESULTS BY CYBER-175
COMPUTER PROGRAM 741-VV-F424A
TANGENT TO ELEV 537.00
TRIAL ARCS

RADIUS OF CIRCLE	CENTER OF CIRCLE DISTANCE FROM Q	CENTER OF CIRCLE ELEV	F.S. Q Q= 0.00	F.S. Q Y=.905
73.00	61.00	610.00	1.52	1.10
64.00	58.00	601.00	1.55	
72.00	71.00	609.00	1.64	
53.00	50.00	590.00	1.57	
63.00	61.00	620.00	1.57	
63.00	60.00	600.00	1.53	

CRITICAL FAILURE SURFACE

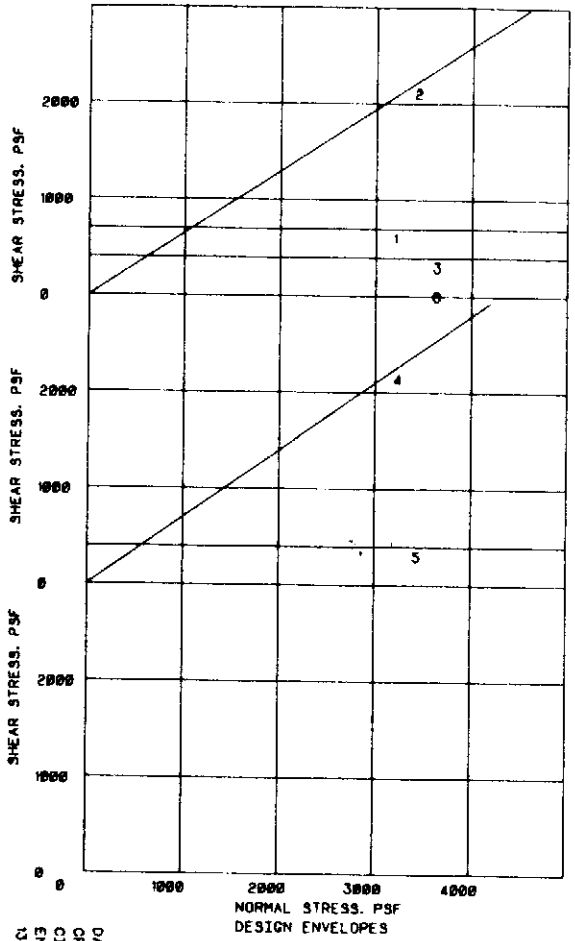


NOTES
1- ANALYSES WERE RUN ACCORDING TO EM 1110-2-1902 DATED APRIL 1978
2- THE SIDE EARTH FORCE DIRECTION WAS TAKEN AS THE AVERAGE OF THE EMBANKMENT SLOPES IMMEDIATELY ADJACENT TO THE SLICE INTERFACE
3- PSI-SIESMIC COEFFICIENT USED IN ANALYSES



DAVENPORT, LOCAL PROTECTION
STA. 113+00 LANDSIDE SLOPE
CIRCLE SLOPE STABILITY ANALYSIS
END OF CONSTRUCTION CONDITION
@ MAY 1978 C.W.

PLATE B-21



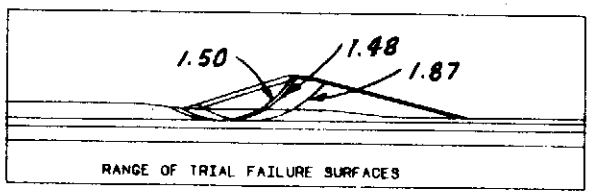
DESIGN ENVELOPES

MATERIAL		SOIL WT. LBS./CUFT		SHEAR STRENGTH Q		R		S		(R+S)/2	
		MOIST	SAT	PHI DEGREES	COHESION PSF	PHI DEGREES	COHESION PSF	PHI DEGREES	COHESION PSF	PHI DEGREES	COHESION PSF
COMPACTED IMPER EMBA EL 571.3 TO EL 546	1	125.00	125.00	0.00	700.00	0.00	0.00	0.00	0.00	0.00	0.00
HYDRAULIC SAN FILL EL 566.6 TO EL 546	2	120.00	120.00	33.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
CLAY FOUNDATION EL 553 TO EL 543	3	110.00	110.00	0.00	400.00	0.00	0.00	0.00	0.00	0.00	0.00
SAND FOUNDATION EL 543 TO EL 538	4	125.00	125.00	35.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
CLAY FOUNDATION EL 538 TO EL 529	5	110.00	110.00	0.00	400.00	0.00	0.00	0.00	0.00	0.00	0.00
ROCK - LIMESTONE BE LOW EL 529	6	150.00	150.00	45.00	3000.00	0.00	0.00	0.00	0.00	0.00	0.00

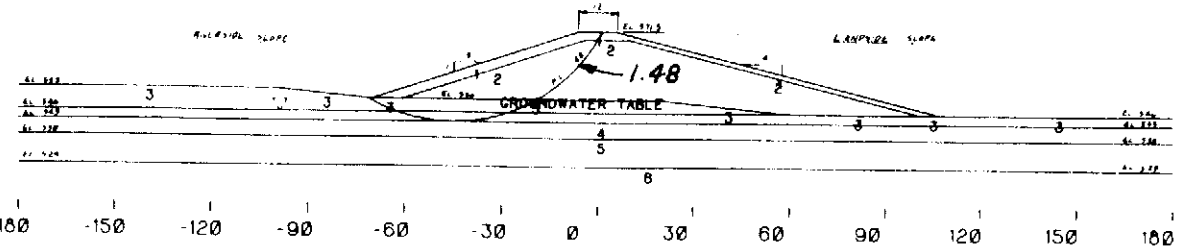
CIRCLE FAILURE SURFACE
RESULTS BY CYBER-175
COMPUTER PROGRAM 741-VV-F424A
TANGENT TO ELEV 543.00
TRIAL ARCS

RADIUS OF CIRCLE	CENTER OF CIRCLE DISTANCE FROM Q	CENTER OF CIRCLE ELEV	F.S. Φ= 0.00	F.S. Q Y=0.05
52.00	-45.00	595.00	1.46	1.23
57.00	-50.00	600.00	1.59	
57.00	-30.00	600.00	1.67	
47.00	-45.00	590.00	1.58	
67.00	-40.00	610.00	1.54	
57.00	-40.00	600.00	1.52	

CRITICAL FAILURE SURFACE



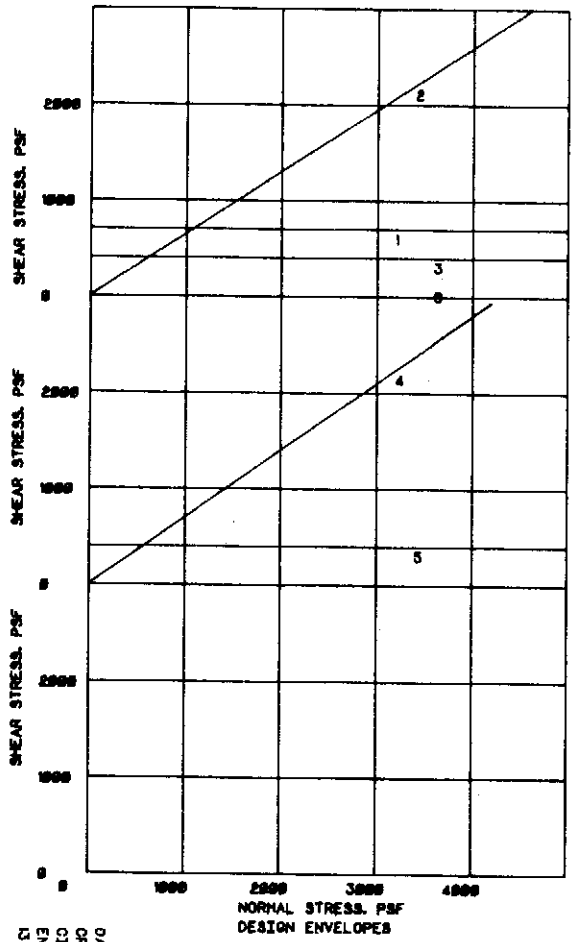
NOTES
1- ANALYSES WERE RUN ACCORDING TO EM 1110-2-1902 DATED APRIL 1970
2- THE SIDE EARTH FORCE DIRECTION WAS TAKEN AS THE AVERAGE OF THE EMBANKMENT SLOPES IMMEDIATELY ADJACENT TO THE SLICE INTERFACE
3- PSI-SIESMIC COEFFICIENT USED IN ANALYSES



RIVERSIDE SLOPE
SHALLOW FAILURE

DAVENPORT, LOCAL PROTECTION
CREDIT ISLAND CAUSEWAY STA. 143+00
CIRCLE SLOPE STABILITY ANALYSIS
END OF CONSTRUCTION CONDITION
13 APRIL 1976 CJM

PLATE B-22

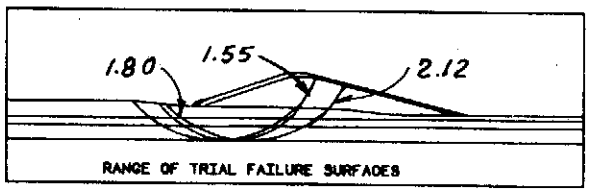


MATERIAL		SOIL WT. LBS./CUFT		SHEAR STRENGTH							
				Q		R		S		(R+S)/2	
		MOIST	SAT.	PHI DEGREES	COHESION PSF	PHI DEGREES	COHESION PSF	PHI DEGREES	COHESION PSF	PHI DEGREES	COHESION PSF
COMPACTED IMPER EMBA EL. 5713 TO EL. 546	1	125.00	125.00	0.00	700.00	0.00	0.00	0.00	0.00	0.00	0.00
HYDRAULIC SAN FILL EL. 586.5 TO EL. 546	2	120.00	120.00	33.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
CLAY FOUNDATION EL. 553 TO EL. 543	3	110.00	110.00	0.00	400.00	0.00	0.00	0.00	0.00	0.00	0.00
SAND FOUNDATION EL. 543 TO EL 536	4	125.00	125.00	35.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
CLAY FOUNDATION EL. 536 TO EL. 529	5	110.00	110.00	0.00	400.00	0.00	0.00	0.00	0.00	0.00	0.00
ROCK - LIMESTONE BELOW EL. 529	6	150.00	150.00	45.00	3000.00	0.00	0.00	0.00	0.00	0.00	0.00

CIRCLE FAILURE SURFACE
RESULTS BY CYBER-175
COMPUTER PROGRAM 741-VV-F424A
TANGENT TO ELEV 529.00
TRIAL ARCS

RADIUS OF CIRCLE	CENTER OF CIRCLE DISTANCE FROM Q	CENTER OF CIRCLE ELEV	F.S. Q ϕ= 0.00	F.S. Q Y=0.05
55.00	-41.00	564.00	1.55	1.23
60.00	-50.00	595.00	1.80	
71.00	-30.00	606.00	2.12	
50.00	-40.00	570.00	2.01	
61.00	-40.00	610.00	1.82	
73.00	-30.00	602.00	2.11	

CRITICAL FAILURE SURFACE



NOTES
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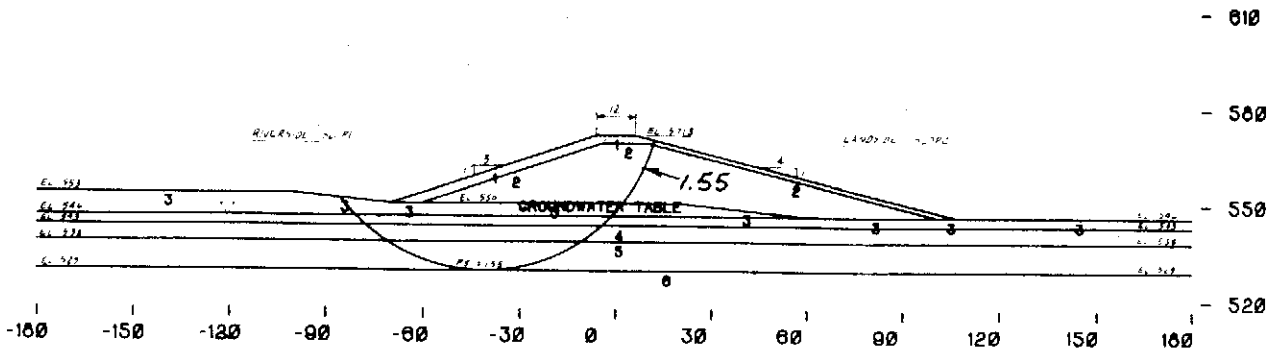
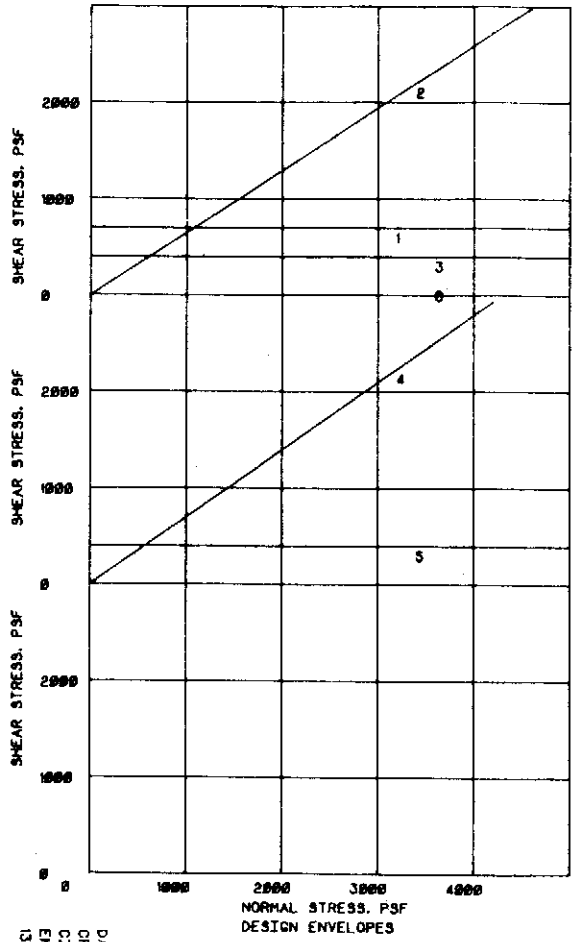
2- THE SIDE EARTH FORCE DIRECTION WAS TAKEN AS THE AVERAGE OF THE EMBANKMENT SLOPES IMMEDIATELY ADJACENT TO THE SLICE INTERFACE
3- PSI-SIESMIC COEFFICIENT USED IN ANALYSES



RIVERSIDE SLOPE
DEEP FAILURE

DAVENPORT, LOCAL PROTECTION
CREDIT ISLAND CAUSEWAY STA. 143+00
CIRCLE SLOPE STABILITY ANALYSIS
END OF CONSTRUCTION CONDITION
13 APRIL 1978 GJM

PLATE B-23

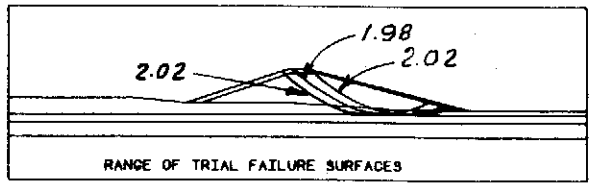


MATERIAL		SOIL WT. LBS./CUFT		SHEAR STRENGTH Q		R		S		(R+S)/2	
		MOIST	SAT.	PHI DEGREES	COHESION PSF	PHI DEGREES	COHESION PSF	PHI DEGREES	COHESION PSF	PHI DEGREES	COHESION PSF
COMPACTED IMPER EMBA EL 571.3 TO EL 548	1	125.00	125.00	0.00	700.00	0.00	0.00	0.00	0.00	0.00	0.00
HYDRAULIC SAN FILL EL 566.6 TO EL 548	2	120.00	120.00	33.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
CLAY FOUNDATION EL 583 TO EL 543	3	110.00	110.00	0.00	400.00	0.00	0.00	0.00	0.00	0.00	0.00
SAND FOUNDATION EL 543 TO EL 536	4	125.00	125.00	35.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
CLAY FOUNDATION EL. 536 TO EL 520	5	110.00	110.00	0.00	400.00	0.00	0.00	0.00	0.00	0.00	0.00
ROCK LIMESTONE BE LOW EL 520	6	150.00	150.00	45.00	3000.00	0.00	0.00	0.00	0.00	0.00	0.00

CIRCLE FAILURE SURFACE
RESULTS BY CYBER-175
COMPUTER PROGRAM 741-VV-F424A
TANGENT TO ELEV 543.00
TRIAL ARCS

RADIUS OF CIRCLE	CENTER OF CIRCLE DISTANCE FROM ℄	CENTER OF CIRCLE ELEV	F.S. Q ℄= 0.00	F.S. Q Y= 0.05
66.00	60.00	620.00	1.66	1.51
70.00	46.00	610.00	2.02	
74.00	79.00	617.00	2.02	
57.00	55.00	600.00	2.04	
66.00	56.00	633.00	1.66	
70.00	57.00	622.00	1.90	

FOR FINAL FAILURE SURFACE



NOTES
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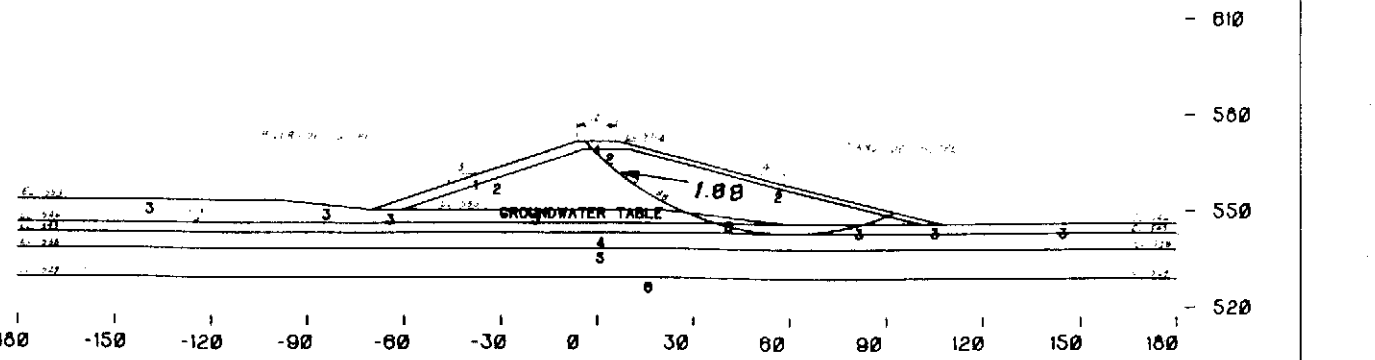
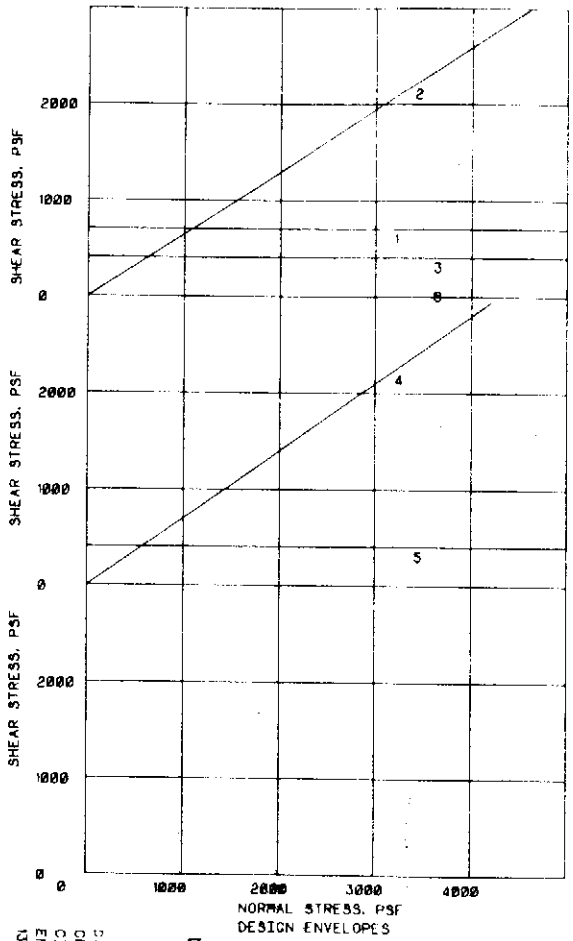
2- THE SIDE EARTH FORCE DIRECTION WAS TAKEN AS THE AVERAGE OF THE EMBANKMENT SLOPES IMMEDIATELY ADJACENT TO THE SLICE INTERFACE
3- PSI-SIESMIC COEFFICIENT USED IN ANALYSES



LANDSIDE SLOPE
SHALLOW FAILURE

DAVENPORT, LOCAL PROTECTION
CREDIT ISLAND CAUSEWAY STA. 143+00
CIRCLE SLOPE STABILITY ANALYSIS
END OF CONSTRUCTION CONDITION
13 APRIL 1978 CJM

PLATE B-24



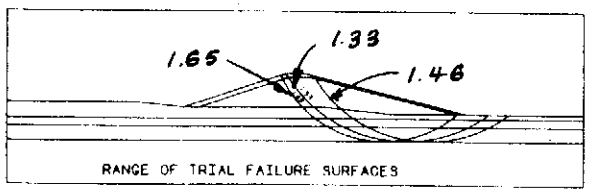
DESIGN ENVELOPES

MATERIAL		SOIL WT. LBS./CU.FT.		SHEAR STRENGTH Q		R		S		(R+S)/2	
		MOIST	SAT.	PHI DEGREES	COHESION PSF	PHI DEGREES	COHESION PSF	PHI DEGREES	COHESION PSF	PHI DEGREES	COHESION PSF
COMPACTED IMPER EMBA EL 571.3 TO EL 548	1	125.00	125.00	0.00	700.00	0.00	0.00	0.00	0.00	0.00	0.00
HYDRAULIC SAN FILL EL 566.6 TO EL 548	2	120.00	120.00	33.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
CLAY FOUNDATION EL 553 TO EL 543	3	110.00	110.00	0.00	400.00	0.00	0.00	0.00	0.00	0.00	0.00
SAND FOUNDATION EL 543 TO EL 536	4	125.00	125.00	35.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
CLAY FOUNDATION EL 536 TO EL 520	5	110.00	110.00	0.00	400.00	0.00	0.00	0.00	0.00	0.00	0.00
ROCK - LIMESTONE BE LOW EL 520	6	150.00	150.00	45.00	3000.00	0.00	0.00	0.00	0.00	0.00	0.00

CIRCLE FAILURE SURFACE
RESULTS BY CYBER-175
COMPUTER PROGRAM 741-VV-F424A
TANGENT TO ELEV 520.00
TRIAL ARCS

RADIUS OF CIRCLE	CENTER OF CIRCLE DISTANCE FROM ℄	CENTER OF CIRCLE ELEV	F.S. Q ψ = 0.00	F.S. Q ψ = .006
66.00	69.00	617.00	1.33	1.05
71.00	55.00	600.00	1.65	
63.00	63.00	612.00	1.46	
70.00	77.00	599.00	1.42	
97.00	66.00	626.00	1.42	
75.00	64.00	604.00	1.43	

CRITICAL FAILURE SURFACE



RANGE OF TRIAL FAILURE SURFACES

NOTES
1- ANALYSES WERE RUN ACCORDING TO EM 1110-2-1902 DATED APRIL 1970
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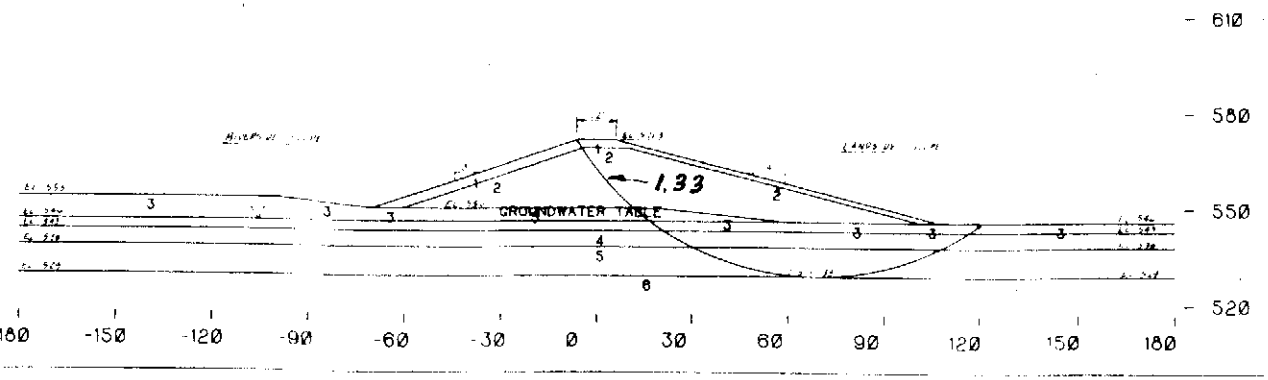
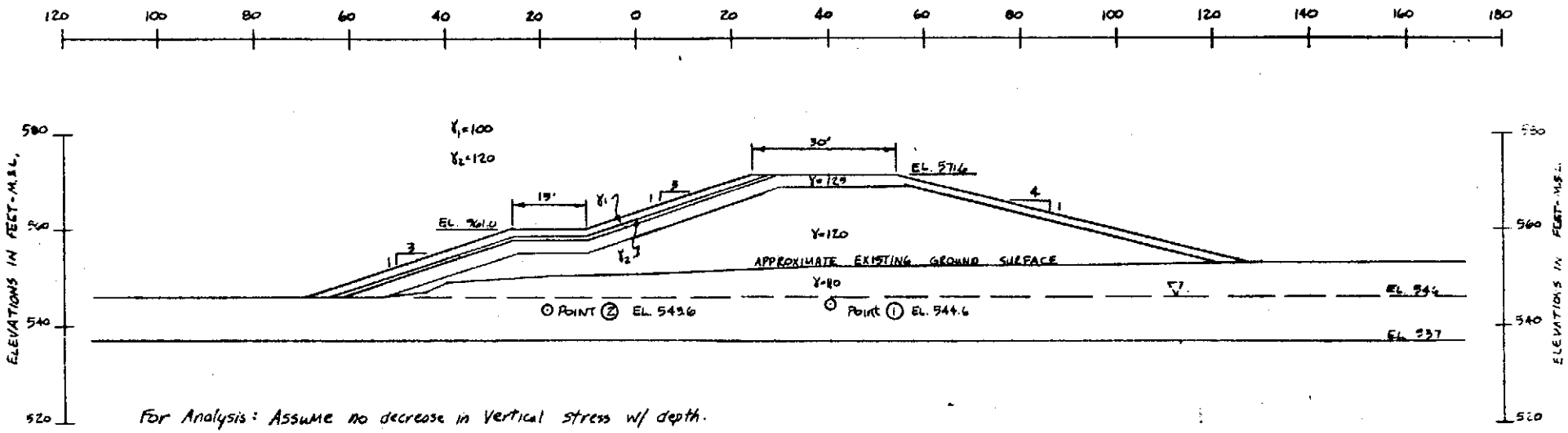
3- PSI-SIESMIC COEFFICIENT USED IN ANALYSES



LANDSIDE SLOPE
DEEP FAILURE

DAVENPORT, LOCAL PROTECTION
CREDIT ISLAND CAUSEWAY STA. 143+00
CIRCLE SLOPE STABILITY ANALYSIS
END OF CONSTRUCTION CONDITION
13 APRIL 1976 GJM

PLATE B-25



For Analysis: Assume no decrease in Vertical stress w/ depth.

C_c = Compression Index = $(e_1-e_2)/(\log_{10} P_2 - \log_{10} P_1) = (1.205-0.762)/(\log_{10} 10 - \log_{10} 1) = 0.443$

For Point ①

$\bar{P}_o = (552.3-546)(110) + (546-544.6)(110-62.4) = 760$ P.S.F. $= 0.380$ T.S.F.

$\Delta\bar{P} = (2.5)(125) + (571.6-552.3-2.9)(120) = 2329$ P.S.F. $= 1.16$ T.S.F.

Assume e log p curve from Boring D-35 is applicable.

$P_1 = \bar{P}_o = 0.380$ T.S.F. $e_1 = 1.335$ $H = 15.3'$

$P_2 = \bar{P}_o + \Delta\bar{P} = 1.540$ T.S.F. $e_2 = 1.120$

$$\Delta H = \frac{12\,H(e_1-e_2)}{1+e_1} = \frac{(12)(15.3)(1.335-1.120)}{1+1.335} \approx 17''$$

For Point ②

$\bar{P}_o = (550.2-546)(110) + (546-436)(110-62.4) = 576$ P.S.F. $= 0.288$ T.S.F.

$\Delta\bar{P} = (1.5)(100) + (0.9)(120) + (3)(125) + (58)(120) = 1281$ P.S.F. $= 0.641$ T.S.F.

Assume e log p curve from Boring D-35 is applicable.

$P_1 = \bar{P}_o = 0.288$ T.S.F. $C_1 = 1.350$ $H = 13.2'$

$P_2 = \bar{P}_o + \Delta\bar{P} = 0.929$ T.S.F. $e_2 = 1.215$

$$\frac{\Delta H}{1+e_1} = \frac{12\,H(e_1-e_2)}{1+e_1} = \frac{(12)(13.2)(1.350-1.215)}{1+1.350} \approx 9''$$

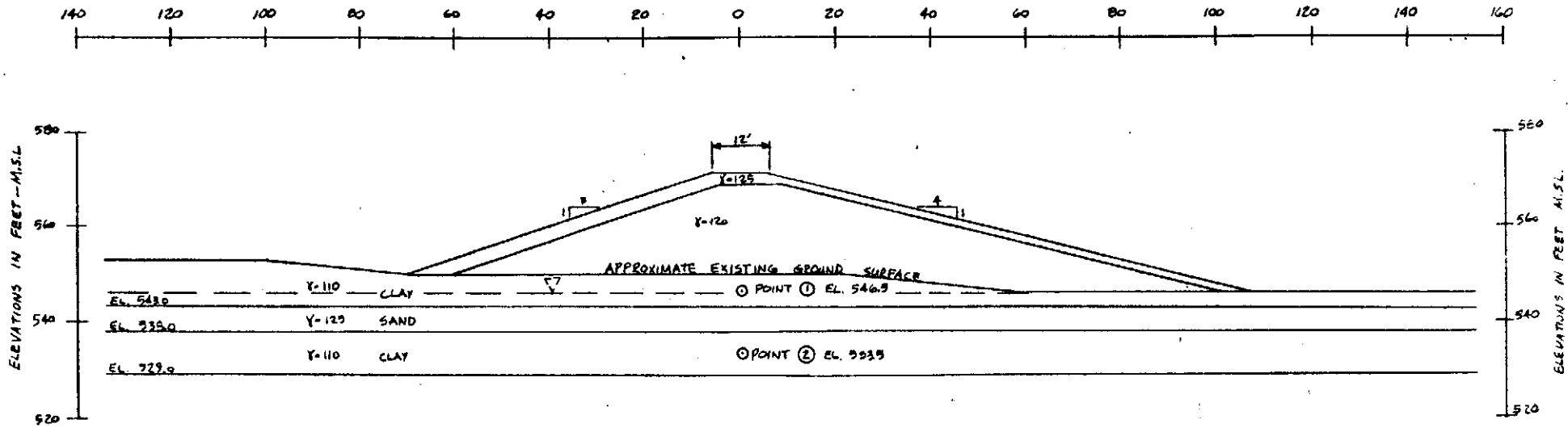
DAVENPORT, LOCAL PROTECTION

SETTLEMENT ANALYSIS

DAVENPORT SHORE REACH

STATION 113-00 ±

PLATE B-26



FOR ANALYSIS: Assume no decrease in Vertical Stress w/ depth.
Assume e log p curve from Boring D-35 is Applicable.

For Point ①

$\bar{P}_o = (3.5 \times 110) = 385$ P.S.F. $= 0.193$ T.S.F.

$\Delta\bar{P} = (2.5 \times 125) + (1.20)(188) = 2569$ P.S.F. $= 1.285$ T.S.F.

$P_1 = \bar{P}_o = 0.193$ T.S.F. $\quad e_1 = 1.375$

$P_2 = \bar{P}_o + \Delta\bar{P} = 1.478$ T.S.F. $\quad e_2 = 1.120$ $\quad H = 7'$

$$\Delta H = \frac{12H(e_1 - e_2)}{1+e_1} = \frac{(12 \times 7 \times 1.375 - 1.120)}{1 + 1.375} = 9''$$

For Point ②

$\bar{P}_o = (4 \times 110) + (3 \times 110 - 62.4) + (5 \times 125 - 62.4) + (4.5 \times 110 - 62.4) = 1110$ P.S.F. $= 0.556$ T.S.F.

$\Delta\bar{P} = 1.285$ T.S.F. (PREVIOUSLY COMPUTED)

$P_1 = \bar{P}_o = 0.555$ T.S.F. $\quad e_1 = 1.308$

$P_2 = \bar{P}_o + \Delta\bar{P} = 1.840$ T.S.F. $\quad e_2 = 1.080$ $\quad H = 9'$

$$\Delta H = \frac{12H(e_1 - e_2)}{1+e_1} = \frac{(12 \times 9 \times 1.309 - 1.080)}{1 + 1.309} = 10.7''$$

TOTAL SETTLEMENT = 19.7"

DAVENPORT, LOCAL PROTECTION
SETTLEMENT ANALYSIS
CREDIT ISLAND CAUSEWAY REACH
STATION 143+00 ±

PLATE B-27

Subject: DAVENPORT, LOCAL PROTECTION | Date: 12 APRIL 78

Computed by: G.J.M. | Checked by: | Sheet 1 of 2

TIME RATE OF CONSOLIDATION CURVE

SAMPLE No. 5 BOR. D-5

FOR LOAD of 1 T/ft²

$D_{50} = (340+60)/2 \cdot 200 \times 10^{-4} = 0.0200$ $t_{50} = 7$ min

AVG HEIGHT OF SAMPLE = 1 - 0.02 = 0.98" = 2H

$C_{v_1} = \frac{TH^2}{t_{50}} = \frac{(0.197)(.98/2)^2}{7.0} = 0.006757$ in²/min. $T = 0.197$ for t_{50}

FOR LOAD OF 2 T/ft²

$D_{50} = (435+65/2) \cdot 250 \times 10^{-4} = 0.0250$ $t_{50} = 7$ min

Avg. HEIGHT OF SAMPLE = 1 - 0.025 = 0.975" = 2H

$C_{v_2} = \frac{TH^2}{t_{50}} = \frac{(0.197)(.975/2)^2}{7.0} = 0.006688$ in²/min $T = 0.197$ for t_{50}

$C_{v_1} = (0.006757)(1/144)(525600) = 24.66$ ft²/yr.

$C_{v_2} = (0.006689)(1/144)(525600) = 24.41$ ft²/yr.

Use $C_v = (24.41 + 24.66)/2 = 24.54$ ft²/yr. since load range approx. 1.5 T/ft²

PLATE B-28

DAVENPORT, LOCAL PROTECTION

12 APRIL 1978

Computed by G.J.M.

Checked by

Sheet 2 of 2

$t = \frac{T^*(d/2)^2}{C_v}$ Assuming double drainage

$d = 15$ Approx. Avg. Clay depth.

$C_v = 24.54\ ft^2/yr$

	T^*	t_{yrs}
10%	0.008	0.02
20%	0.031	0.07
30%	0.071	0.16
40%	0.126	0.29
50%	0.197	0.45
60%	0.287	0.66
70%	0.403	0.92
80%	0.567	1.30
90%	0.848	1.94
100%	∞	∞

*T values obtained from "Soil Engineering" by Spangler and Hardy.

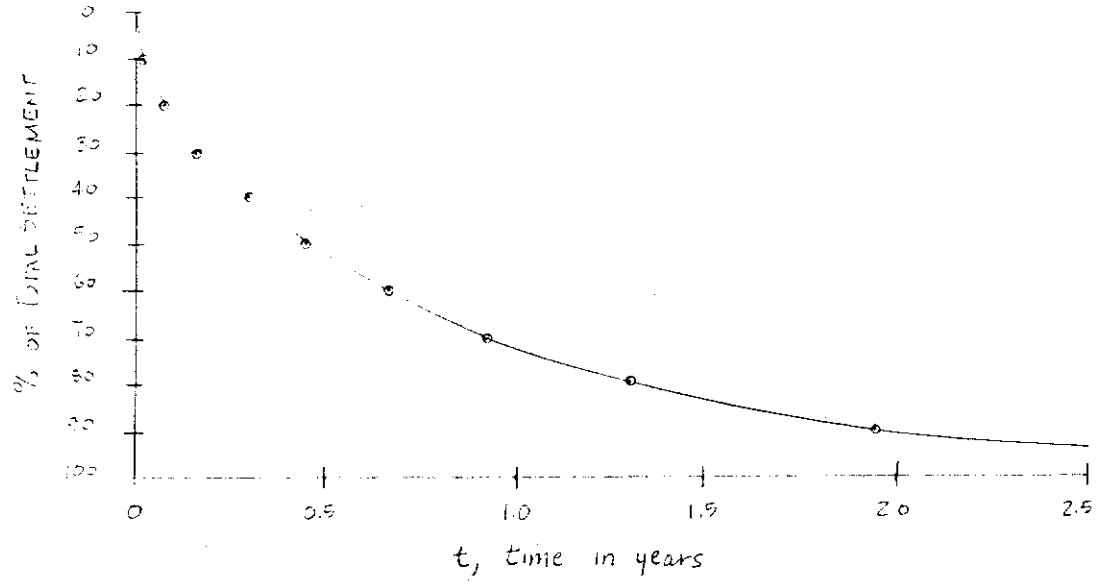


PLATE B-29

NCR Form

PROJECT Davenport, Local Protection Date 10 May 1978

Computed by GJM Checked by DHP

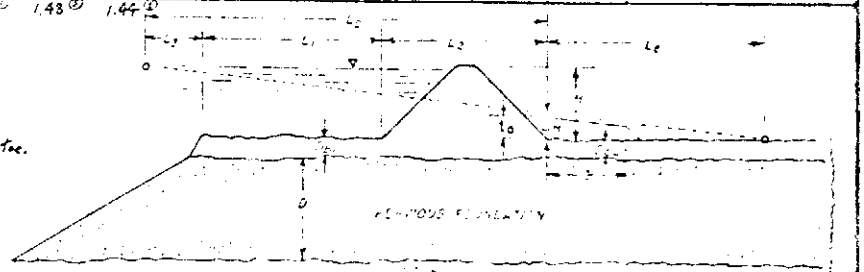
SUMMARY
UNDERSEEPAGE ANALYSIS DATA
BERM ANALYSIS Sheet 1 of 1

STATIONS		256+60 to 245+20	245+20 to 221+00	221+00 to 190+00	190+00 ② to 170+00	190+00 ① to 170+00	170+00 to 160+00	160+00 to 150+00	150+00 to 144+00	144+00 to 134+00	134+00 to 125+00	125+00 to 117+00	117+00 to 70+00	70+00 to 46+00
LEVEE DESIGN GRADE		570.5	EXCAVATE TO BEDROCK AND PROVIDE POSITIVE CUTOFF	NO AQUIFER PRESENT IN THIS REACH, SHALE OVERLAIN CLAY.	571.1	571.1	571.1	571.2	571.2	571.3	571.5	571.5	TOPOGRAPHY IN FARMER GARAGE DRAINAGE AREA SHOWS CONTROL UNDERSEEPAGE. BASE WIDTH GREATER THAN 100' IN THIS REACH AND MAY BE LESS	572.0
ELEV L.S. TOE OF TAILWATER		554.2			550.2	550.2	551.1	546.4	547.5	545.4	551.6	553.1		560.6
GROSS HEAD	H	18.3			20.9	20.9	20.0	24.6	23.7	25.9	19.9	18.4		11.4
TRANS. PERVIOUS FOUNDATION	D	3.0			5.0	5.0	3.5	2.6	3.0	2.8	1.8	4.6		5.0
TRANS. BLANKET THICKNESS L.S.	D_{BL}	10.7			3.5	3.5	6.0	4.4	6.0	8.0	12.2	13.7		18.6
TRANS. BLANKET THICKNESS R.S.	D_R	11.5			6.0	12.5	14.3	14.4	12.6	9.2	12.3	12.4		17.6
NAT. BLANKET LENGTH L.S.	L_L ③	∞			∞ ④	∞ ⑤	∞ ④	∞ ④	∞ ④	∞ ④	∞	∞		∞
NAT. BLANKET LENGTH R.S.	L_R	600'			2200'	350	2200'	1300'	700'	100'	100'	100'		50'
K_f/K_b LANDSIDE	A	200			100	100	200	100	200	200	200	200		200
K_f/K_b RIVERSIDE	B	800			800	800	800	800	800	800	800	800		800
C_L = 1/4 D_{BL} D	C_L	80.1			41.8	41.8	64.8	33.8	60.0	66.9	66.5	134.5		136.4
C_R = 1/3 D_{BL} D	C_R	166.1			154.9	223.6	200.1	173.0	173.9	143.6	123.1	256.9		265.3
$L_1 = C_R - 100$ ft L_R/C_R	L_1	165.9			154.9	204.9	200.1	173.0	173.8	86.5	84.6	95.2		49.4
L_3 = 0.4 A	L_3	1.3			2.2	2.2	1.5	1.1	1.3	1.2	0.8	2.9		2.2
L_2 = BASE WIDTH LEVEE	L_2 ⑥	142.8			130.6	130.6	128.2	150.6	155.0	184.0	179.0	174.3		101.2
$L_e = L_1 + L_2 + L_3$	L_e	310.0			287.7	337.7	329.8	324.7	330.1	271.7	264.4	272.9		152.8
$L_o = C_L(\frac{K_f}{K_b})$ for $\frac{L_L}{C_L}$	L_o	80.1			41.8	41.8	64.8	33.8	60.0	66.9	66.5	134.5		136.4
$L_s = L_e$		390.1			329.5	379.5	394.6	358.5	390.1	338.6	330.9	407.4		289.2
$\Delta H = H - 0.8 C_L/L_s + L_e$	ΔH	3.76			2.65	2.30	3.90	2.30	3.64	5.11	3.98	6.07		5.37
F.S. = 0.85 $C_{BL}/\Delta H$	F.S.	2.16			1.12	1.29	1.54	1.62	1.40	1.33	2.62	1.91		2.93
PROBABLE CONTROL		NONE			NONE	NONE	NONE	NONE	NONE	NONE	NONE	NONE		NONE
BERM WIDTH, B = $\Delta H - L_o$	B	—	—	—	—	—	—	—	—	—	—	—	—	—
COMPUTED $t_o = \frac{3\Delta H - K_f/K_b C_L}{K_f/K_b + 3}$		—	—	—	0.39	0.20	—	—	0.10	0.37	—	—	—	—
SELECTED t_o	t_o	—	—	—	—	—	—	—	—	—	—	—	—	—

NOK 1.14 ② 1.3 ⑤ 1.67 ① 1.62 ① 1.43 ⑤ 1.44 ①

REMARKS:

① Base width from average of project cross sections for reach analyzed.
② Assuming blocked entrance at lake on Credit Island Park.
③ Assuming open entrance at lake on Credit Island Park.
④ Credit Island Slough not dredged out. If dredged, limit of 100 beyond landside toe.
⑤ Factor of Safety if Credit Island Slough is dredged within 100 of proposed levee landside toe.



TRANSFORMED SOIL SECTION

PLATE B-30

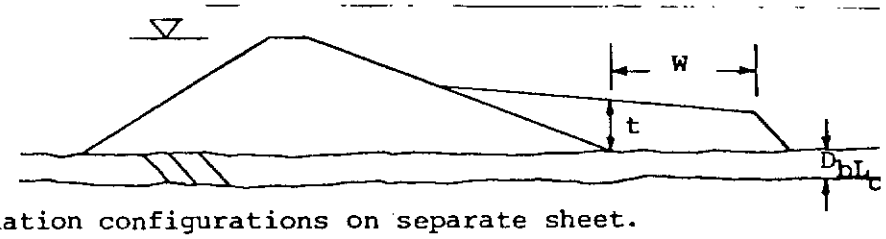
30 April 1976

A Method to Estimate the Thickness and Width of Sand Berms for Sand Levees

		Notes*	Stations
	Elev. top of levee		
	Elev. landside toe (tailwater)		
H	Gross head		
L_2	Base width of levee	(a)	
W_o	$10H - L_2$	(b)	
D	Trans. thickness perv. found.	(c)	
L_L	Landside blanket - width	(d)	
D_{bL}	- thickness	(e)	
D_{bL_c}	- cover	(f)	
A	- perm. ratio	(g)	
C_L	$\sqrt{A \cdot D_{bL} \cdot D}$		
L_R	Riverside blanket - width	(h)	
D_{bR}	- thickness	(i)	
B	- perm. ratio	(j)	
C_R	$\sqrt{B \cdot D_{bR} \cdot D}$		
L_3	Entrance: $0.44D$	(k)	
$L_{l(o)}$	Riverside: $C_R \cdot (\tanh L_R/C_R)$ (or)	(l)	
$L_{l(x)}$	$C_R/(\tanh L_R/C_R)$	(m)	
$L_{e(o)}$	Landside: $C_L \cdot (\tanh L_L/C_L)$ (or)	(n)	
$L_{e(x)}$	$C_L/(\tanh L_L/C_L)$	(o)	
$L_s + L_e$	Total: $L_2 + L_3 + L_l + L_e$		
ΔH	$H \cdot L_e/(L_s + L_e)$		
F.S.	$(\gamma_{bL} \cdot D_{bL_c})/(\Delta H \cdot \gamma_W)$	(p)	
t_o	$(1.5\gamma_W \cdot \Delta H - D_{bL_c} \cdot \gamma_{bL})/(1.5\gamma_W + \gamma_{bL})$	(q)	
Probable Control		(r)	
t	Selected berm thickness	(s)	
W	Selected berm width	(t)	

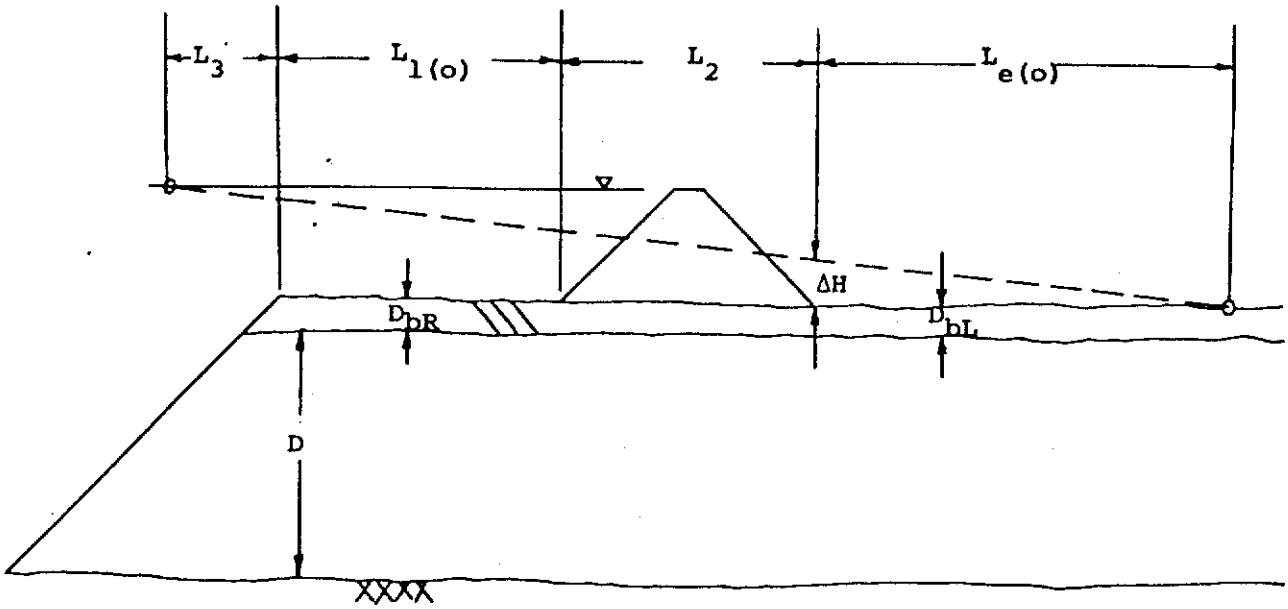
All dimensions in feet.

*Notes on separate sheet.

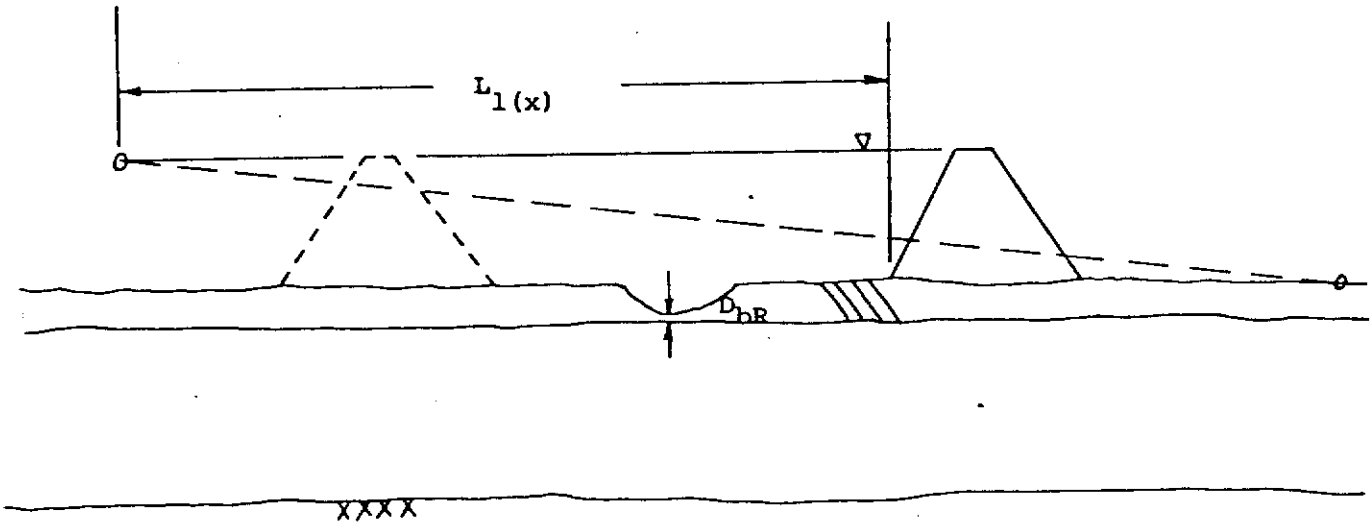


Sketch of levee and foundation configurations on separate sheet.

A Method to Estimate the Thickness and Width of Sand Berms for Sand Levees



Open entrance - Infinite L.S. blanket



Blocked entrance - Infinite L.S. blanket

TRANSFORMED SECTIONS

A Method to Estimate the Thickness and Width
of Sand Berms for Sand Levees

Notes:

(a) Measured from cross section at elev. of landside toe.

(b) Computed berm width. If base width equals or exceeds 10H no berm required.

(c) Procedure and computation on separate sheets.

(d) If L_L is less than 100 ft. provide berm and/or fill depressions to elev. of landside toe for a distance of 100 ft. beyond toe of levee or berm.

(e) If D_{bL} is zero provide berm as for (d). Transformed thickness if blanket includes semipervious soils. Use 0.5 of natural thickness for ML, 0.1 for SM, 0.0 for sand and 1.0 for impervious.

(f) Transformed thickness as for (e) except, when impervious blanket is overlain with pervious or semipervious soils, then the natural total thickness is used for the overlying soils.

(g) For D_{bL} equal to or less than 4, A = 100: for D_{bL} equal to or greater than 5, A = 200, = K_f/K_{bL}.

(h) For parallel diversion levees with blocked entrance, use 1/2 distance between riverside toes of levees.

(i) Transformed thickness as for (e).

(j) For D_{bR} equal to or less than 4, B = 400: for D_{bR} equal to or greater than 5, B = 800, = K_f/K_{bR}.

(k) For parallel diversion levees with clay in thalweg, use L_3 = 0.

(l) Use $L_{1(o)}$ for an open (o) entrance.

(m) Use $L_{1(x)}$ for blocked (x) entrance.

(n) Use $L_{e(o)}$ for finite open (o) exit or infinite blanket.

(o) Use $L_{e(x)}$ for blocked (x) exit.

(p) γ_{bL} is submerged unit weight of landside blanket cover and berm (use 53 p.c.f.). γ_W is 62.4 p.c.f.

(q) Computed berm thickness for a factor of safety of 1.5 at landside toe of levee.

(r) For factor of safety computed at the landside toe of the levee, a berm is not required if F.S. greater than 1.5. Berm required if F.S. is equal to or less than 1.0. If F.S. greater than 1.0 and less than 1.5, and if computed berm thickness (q) is greater than 2.0, berm required. If F.S. greater than 1.0 and less than 1.5, and if computed berm thickness (q) is less than 2.0, judgment will determine whether a berm is required.

(s) Minimum 3.0 ft.

(t) Minimum 30 ft.

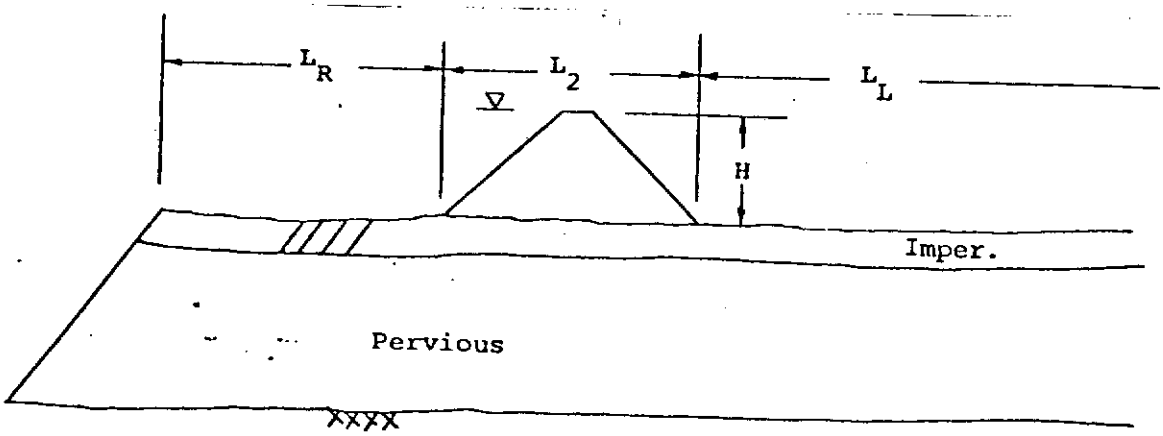


PLATE B-33

NATURAL SECTION

30 April 1976

Procedure for Transformation of Pervious Foundation

$$D = \sqrt{\Sigma(d_{(n)}K_{h(n)}) \; \Sigma \; (d_{(n)}/K_{v(n)})}$$

$$K_f = \sqrt{\Sigma(d_{(n)}K_{h(n)})/\Sigma \; (d_{(n)}/K_{v(n)})}$$

where

d (1,2,3....n) = increment (1,2,3....n) of depth of pervious strata

K_h (1,2,3....n) = horizontal permeability for corresponding increment (1,2,3....n) of pervious strata*

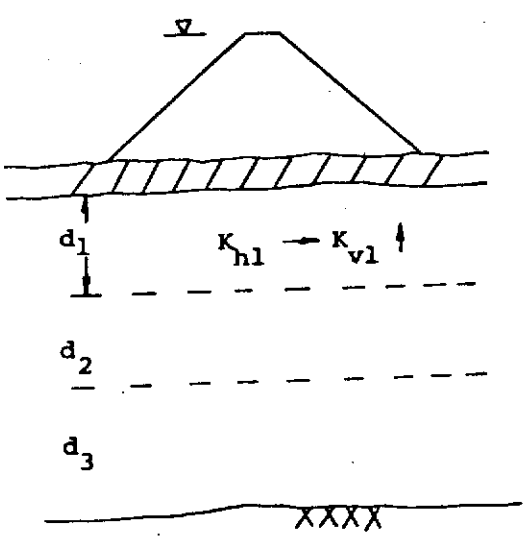
K_v (1,2,3....n) = vertical permeability for corresponding increment (1,2,3....n) of pervious strata

K_h (1,2,3....n)/K_v (1,2,3....n) = 1 if $D_{10} > 0.30$

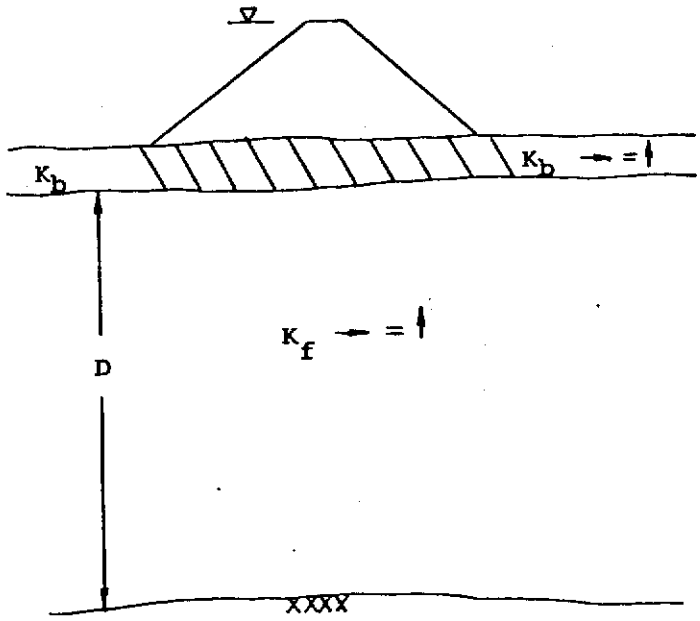
= 2 if $0.20 < D_{10} \leq 0.30$

= 3 if $D_{10} < 0.20$

*The value of K_h is estimated from the D_{10} size and the graph shown in Transactions of A.S.C.E., Vol. 126, 1961, Part 1, p. 1449, Figure 12.

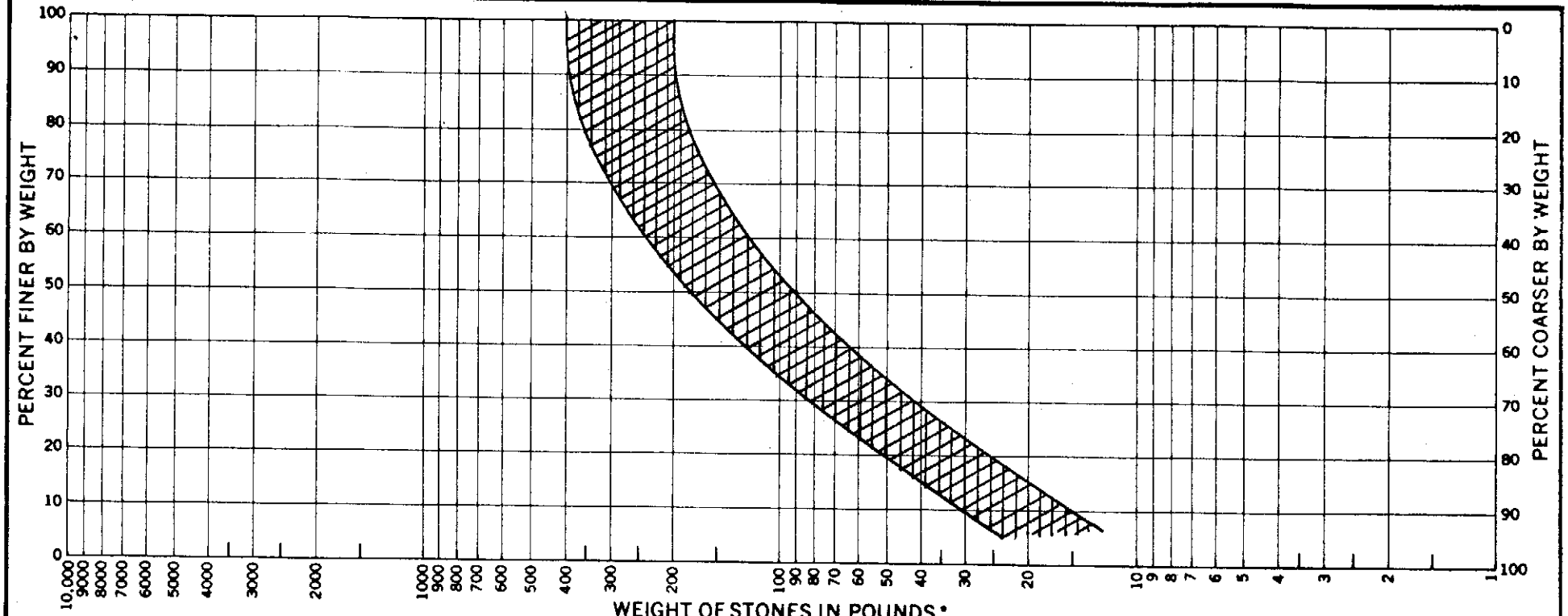


NATURAL



TRANSFORMED

PLATE B-34



18" thick riprap layer.

PROJECT DAVENPORT, LOCAL PROTECTION

AREA RIVERSIDE SLOPE PROTECTION

DATE

RIPRAP GRADATION CURVES

PLATE B-35

ENG FORM 4055
APR 67

MISSISSIPPI RIVER
DAVENPORT, IOWA
LOCAL FLOOD PROTECTION

PHASE II
GENERAL DESIGN MEMORANDUM

APPENDIX C

DESIGN ANALYSES
(REVISED)

US ARMY ENGINEER DISTRICT, ROCK ISLAND
CORPS OF ENGINEERS
CLOCK TOWER BUILDING
ROCK ISLAND, ILLINOIS 61201

MISSISSIPPI RIVER
DAVENPORT, IOWA

LOCAL FLOOD PROTECTION

PHASE II
GENERAL DESIGN MEMORANDUM

APPENDIX C
DESIGN ANALYSES

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PLATES

Plate No.	Subject
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2	Cap Floodwall Design
3	T-Type Floodwall Design
4	Folding Floodwall Design
5	Panel and Strut Closure Structure Design
6	Panel and Post Closure Structure Design
7	Slough Closure Structure Design
8	Slough Closure Bridge Design
9	Blackhawk Creek Bicycle Bridge Design

MISSISSIPPI RIVER
DAVENPORT, IOWA

LOCAL FLOOD PROTECTION
PHASE II
GENERAL DESIGN MEMORANDUM
APPENDIX C

DESIGN ANALYSES

I - GENERAL

1. General. This appendix describes the methods used in designing the structures required for the Davenport, Iowa, Local Flood Protection project. Supporting information is included, such as design criteria, basic data and assumptions, loads and loading conditions, and typical design computations and stability analyses. Each structure has been analyzed for stability under various loading conditions, and principle members have been analyzed to have stresses below maximum allowable values. Complete analysis of all details will be undertaken in final design.

II - DESIGN CRITERIA

2. Working stresses. Working stresses used in the design conform to those specified in Engineering Manual 1110-1-2101, "Working Stresses for Structural Design," dated November 1963, including changes 1 and 2. Loading conditions, design assumptions, and other criteria are based on the applicable parts of the following references:

a. EM 1110-2-2000, "Standard Practice for Concrete," including changes 1 and 2, dated November 1971.

b. EM 1110-2-2103, "Details of Steel Reinforcement for Cast in Place Concrete," dated May 1971.

c. EM 1110-2-2400, "Structural Design of Spillways and Outlet Works," dated November 1964.

d. EM 1110-2-2501, "Wall Design, Floodwalls," including changes 1 thru 3, dated June 1962.

e. EM 1110-2-2502, "Retaining Walls," including changes 1 thru 3, dated January 1965.

f. EM 1110-2-2902, "Conduits, Culverts and Pipes," including changes 1 and 2, dated April 1971.

g. EM 1110-2-2701, "Vertical Lift Crest Gates," dated December 1962.

h. ETL 1110-2-184, "Engineering and Design, Gravity Dam Design Stability," dated 25 February 1974.

3. Reinforced concrete. Concrete structures will have a 28-day minimum compressive strength of 3,000 pounds per square inch. In the final design stage, guidance for design of reinforcement for concrete structures will be Technical Report SL-80-4, Strength Design of Reinforced Concrete Hydraulic Structures by Tony C. Liu, prepared for OCE.

4. Concrete reinforcement. Concrete reinforcement will be deformed billet-steel bars conforming to the requirements of ASTM 1615, Grade 40, with a minimum yield stress of 40,000 psi. Concrete cover and minimum reinforcement will comply with EM 1110-2-2103. The spacing, splicing, and bending requirements of the bars will be in accordance with the current ACI-318 and the Manual of Standard Practice for Detailing Reinforced Concrete Structures.

5. Steel sheet piling. Steel for sheet piling will conform to the requirements of ASTM A328-70. Piling is used to prevent undermining of structures and as the foundation of I-walls.

6. Structural steel. Miscellaneous structural steel will conform to the requirements of ASTM A-36. Working stresses for the design of structural steel will be in accordance with EM 1110-1-2101 and will be either A-36 or A-588. A-588 will be used primarily for its enhanced corrosion resistance for prevention of future maintenance and to eliminate painting.

7. Structural aluminum. Construction of aluminum structures will be in accordance with Specifications for Aluminum Structures, Nov 1971, by the Aluminum Association (AA). Aluminum used will be type 6061-T6.

III - DESIGN OF STRUCTURES

8. Gatewells. The rectangular cast-in-place gatewells will be designed using computer program number 713-G1-F474A, "Concrete Frame Analysis." The program abstract is available from RID. The program assumes the loading is a hydrostatic pressure of uniform intensity on all sides of the gatewell and varying linearly in intensity with the height. Then the gatewell is analyzed using moment distribution and incorporating a moment reduction in the end moments due to the high depth to length ratio. Moment, shear, and bond are checked and final steel requirements per foot of height computed. The base slab is designed using one-way slab assumptions. This program has been verified and has been previously accepted by NCD and OCE. Since the analysis and design of gatewells are common to most flood control projects, computations are not inclosed in this report.

9. Reinforced Concrete Box Culverts. The rectangular cast-in-place concrete box culverts are designed using computer program number 713-G1-F411B, "Concrete Box Culvert - Frame Analysis and Design." The

program abstract is available from RID. The design uses loading Condition III, Case I and II, in accordance with EM 1110-2-2902. Normal allowable working stresses will be applied to both Case I and II. As the analysis and design of box culverts are common to almost all flood control projects, no computations will be inclosed in this report.

10. Floodwalls. The main text indicates the types and locations of the floodwalls. All are designed in accordance with EM 1110-2-2501 and EM 1110-2-2502. Same types of walls are used in several locations. One example of analysis is provided for each type.

a. I-walls. I-wall design was made using the computer program in Report K-81-2, Design and Analysis of Sheet Pile Walls by Classical Methods, by W. P. Dawkins for OCE. This program will analyze a cantilever sheet pile wall which depends solely on its embedment for stability. The program includes the effect of seepage in modifying the pressures applied to the wall. Computations begin on Plate 1.

b. Cap wall. A reinforced concrete wall stem extension will be placed atop the structure known as the government sewer. The government sewer was originally designed as a dual purpose structure. The construction of Dam 15 formed a pool which is higher than nearby land on the Iowa side of the Mississippi River. This necessitated a wall along the riverfront from the dam to higher ground upstream. The structure also acts as an interceptor sewer to a point below the dam. Analyses has determined the government sewer is structurally adequate to withstand floods to Elev. 574.4, with installation of a concrete cap extending to design levee grade. Analysis was done with the program STRUPUT. Program documentation is available from WES. STRUPUT is an enhanced graphical version of an earlier plane frame program. Analysis is performed utilizing stiffness techniques. Program input is the structure geometry, boundary conditions, structure loading, and the material properties. Output is moment, shears, and deflections. Computations begin on Plate 2.

c. T-Walls. T-walls were designed using Computer Program #713-G1-F442A for Design and Analysis of Inverted-T Floodwalls. Computations begin on Plate 3. Analysis of stability of foundation soil will be made in final design after testing to determine soil parameters.

d. Folding wall. The folding walls in the downtown Davenport reach are designed as a modified I-wall. The foundation and loading design is similar to that presented in paragraph 10a above and will not be presented again. The folding wall consists of two folding sections hinged to a fixed base with steel struts extending from a footing pedestal to the junctions of the folding sections. The top folding section is constructed of aluminum panels transferring the water load to the top and bottom horizontal channels of the top section which, in turn, transfer the forces laterally to the struts. The lower folding section is constructed of concrete with top and bottom horizontal channels which act similar to the top section. Forces are transferred from the top sections through the hinges and the struts to the lower fixed portion of the wall and to the footing. Computations begin on Plate 4.

11. Closure structures. Numerous closure structures are required for the Davenport Local Flood Protection project. The locations and sizes of each are discussed in the main text. Closures are normally analyzed as a modified wall section and, therefore, stability investigation is similar to one of the previously discussed wall types. Two types of panel closures are discussed in the following paragraphs. Also discussed is a bulkhead closure which forms a part of the structures proposed at the downstream end of Credit Island Slough. Part of the steel sheet pile cells to be used for dewatering will be incorporated into the permanent flood protection system. The permanent cells will separate the levee embankment from the pump station and U-frame bulkhead closure, which are proposed at the downstream end of the slough. To enhance the stability of the pump station and closure structure, the cells will be attached to the structures to form a single intergral unit. The pump station is discussed further in Appendix D, Pumping Stations.

a. Panel and Strut Closure. This type of closure, used for heights up to 16 feet, can be incorporated into wide openings. The foundation of the panel and strut closure is similar to that for a T-wall. Hydraulic forces are transferred to the foundation where the bottom of the panels bear on the concrete sill and to struts installed on the landside of the panels. As the hydraulic forces are not transferred laterally past the struts, this type of closure can be any length. Computations, excluding computer printout, are shown beginning on Plate 5.

b. Panel and Post Closure. This type of closure is used for exposed heights to about 8-1/2 feet and can be used any length. The foundation is similar to that of an I-wall. Removable posts are installed at uniform intervals in openings in the foundation. The posts form a cantilever support for panels which are installed between the posts to complete the wall system above the ground line. Computations begin on Plate 6.

c. Slough Bulkhead Closure. The bulkhead closure proposed at the downstream end of the Credit Island slough and ponding area will have an opening 40 feet wide and 33 feet high from the sill to design levee grade. The structure will be used as a navigation opening into Credit Island Slough and as an outlet for water from Blackhawk Creek. A bulkhead design was chosen over miter gates to eliminate machinery and to allow closure with moderate stream currents. A crane is furnished to place and remove the bulkhead sections. Each bulkhead section will act as its own strongback. Adequate lengths of cable will be fixed to the bulkhead sections and attached to a crane sling. Followers will then be utilized to pull the sling to the bulkhead slot to allow the cable to be secured. To minimize sealing problems, only 4 bulkheads, each 8 foot 3 inches high, will be used. The bulkheads will be constructed from A-588 steel. Computations are shown starting on Plate 7 for stability of the structures and for the bulkheads. Further analysis will be necessary before final design when more test information is obtained on the foundation shale.

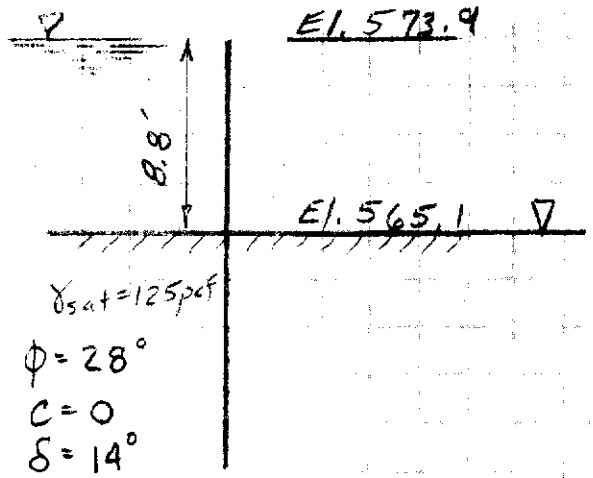
12. Bridges.

a. Slough Bulkhead Closure Bridge. The bulkhead closure bridge will span the opening for the 40-foot bulkhead closure at the end of Credit Island Slough. It will be needed to allow the continuation of the bicycle and pedestrian path and to allow maintenance access. It will be designed for an ASSHTO H-15 load. The bridge will be removable to allow navigation of vessels higher than the clearance afforded from the existing water level to the bottom of the bridge. See Plate 8 for computations.

b. Blackhawk Creek Bicycle Bridge. A bicycle and pedestrian bridge is required to allow the continuation of the bicycle path across Blackhawk Creek. The bicycle bridge is relatively light and will be attached to the outlet wingwalls of a double box culvert crossing under nearby Highway 61 without further foundation construction. Design live loading will be 85 psf as noted in section 1.2.11A, Standard Specifications for Highway Bridges, 12th Ed., ASSHTO. A safety guard rail will be furnished to protect the bicyclist and pedestrians from the highway traffic. Analysis for principal members of this bridge is shown on Plate 9.

Subject	Davenport, Typical I-Wall Analysis	Date	Feb 81
Computed by	Checked by	Sheet	of

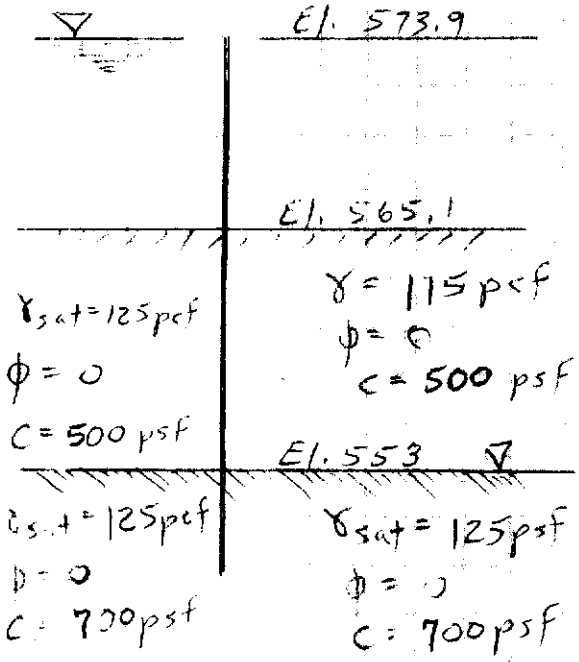
Case I: Long Term Loading



Ref.: soil borings D-52, D-53

For long term load use seepage pressure flow gradient = 0.2
F/S = 1.5

Case II: Short Term Loading



For short term load use no seepage flow gradient

Analysis by CORPS program X0031 (CSHTWAL)

PLATE C-1a

PROGRAM SHTWAL - DESIGN/ANALYSIS OF ANCHORED
OR CANTILEVER SHEET PILE WALLS BY CLASSICAL METHODS
DATE: 08/27/81 TIME: 11:40:15

2. RESULTS

2.A.--HEADING

DAVENPORT WATER PLANT I-WALL
CASE 1

2.B.--SUMMARY OF RESULTS FOR CANTILEVER WALL DESIGN

SOIL PRESSURES DETERMINED BY COULOMB
COEFFICIENTS AND THEORY OF ELASTICITY
EQUATIONS FOR SURCHARGE LOADS

WALL BOTTOM:
PENETRATION = 20.55 (FT)
ELEVATION = 44.55 (FT)

BENDING MOMENT:
MAXIMUM = -36765. (LB-FT)
ELEVATION = 53.9 (FT)

SCALED DEFLECTION:
MAXIMUM = 1.54E+10 (LB-IN3)
ELEVATION = 73.9 (FT)

(NOTE: DIVIDE SCALED DEFLECTION BY MODULUS
OF ELASTICITY IN PSI TIMES PILE MOMENT OF
INERTIA IN IN**4 TO OBTAIN DEFLECTION IN INCHES)

2. RESULTS

2.A.--HEADING

DAVENPORT WATER PLANT I-WALL
CASE 2

2.B.--SUMMARY OF RESULTS FOR CANTILEVER WALL DESIGN

SOIL PRESSURES DETERMINED BY COULOMB
COEFFICIENTS AND THEORY OF ELASTICITY
EQUATIONS FOR SURCHARGE LOADS

WALL BOTTOM:
PENETRATION = 19.99 (FT)
ELEVATION = 45.11 (FT)

BENDING MOMENT:
MAXIMUM = -23315. (LB-FT)
ELEVATION = 55.5 (FT)

SCALED DEFLECTION:
MAXIMUM = 9.40E+09 (LB-IN3)
ELEVATION = 73.9 (FT)

(NOTE: DIVIDE SCALED DEFLECTION BY MODULUS
OF ELASTICITY IN PSI TIMES PILE MOMENT OF
INERTIA IN IN**4 TO OBTAIN DEFLECTION IN INCHES)

PLATE C-16

Subject	Date
Davenport Local Flood Protection, Cap Wall	
Computed by / Checked by	Sheet of

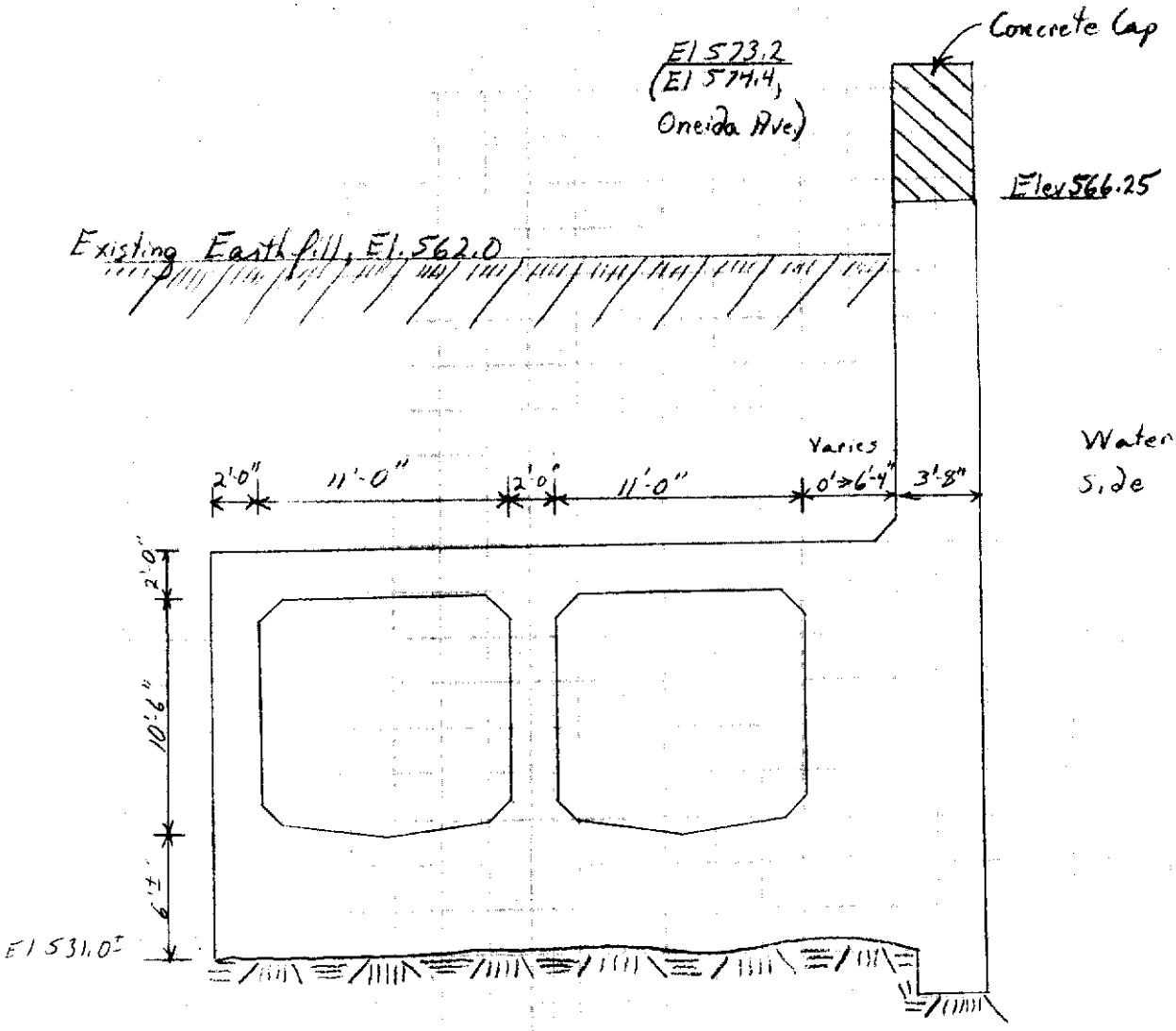


Plate C-2a

NAME OF OFFICE: R10 | COMPUTATION | DATE: Dec 74 | PAGE OF PAG

SUBJECT: Davenport, Iowa - River wall

SOURCE OF DATA

COMPUTED BY: C. Johnson | CHECKED BY | APPROVED BY

LOADING INPUT

MEM NO	DIST LD LEFT (K/FT)	DIST LD RIGHT (K/FT)
2	1.260	1.200
3	1.200	0.840
4	0.840	0.720
5	1.610	1.740
11	1.740	2.300
13	2.300	2.420
14	2.420	2.490
19	2.490	2.490
20	2.490	2.490
23	0.825	0.
6	0.	-2.286
7	-2.415	-2.982
8	-2.982	-3.549
15	-3.549	-6.102
16	-6.102	-6.965

JOINT NUM	APPLIED MOMENT (FT-K)	APPLIED VERT LD (K)	APPLIED HORZ LD (K)
4	0.	-3.00	0.
13	0.	-3.00	0.
20	0.	-5.50	0.
21	0.	-11.94	0.

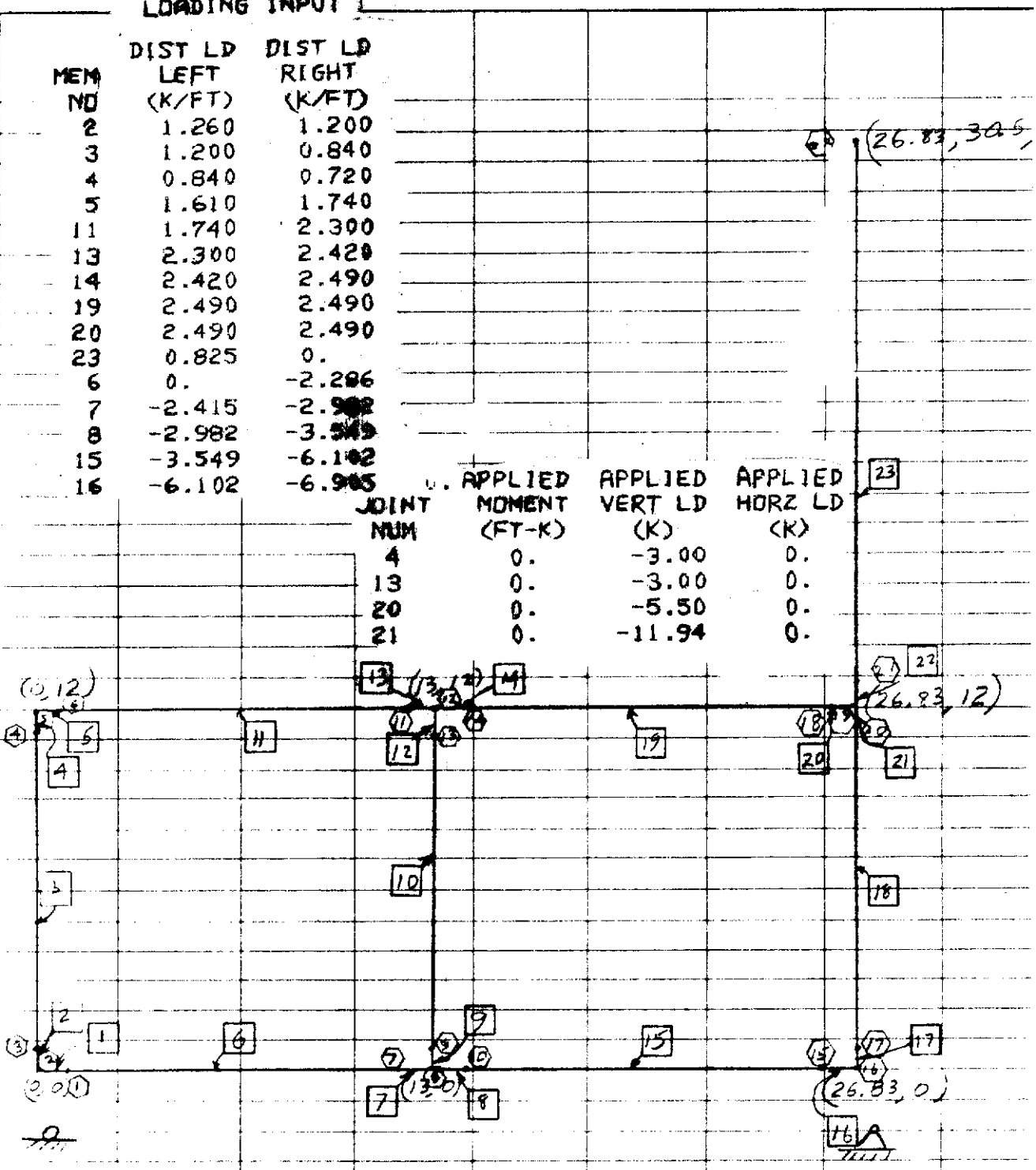


Plate C-26

8.

STRUCTURE INPUT DATA CHECK

22 JOINTS 23 MEMBERS 1 LOAD CASES
MODULUS OF ELASTICITY = 3000. KSI

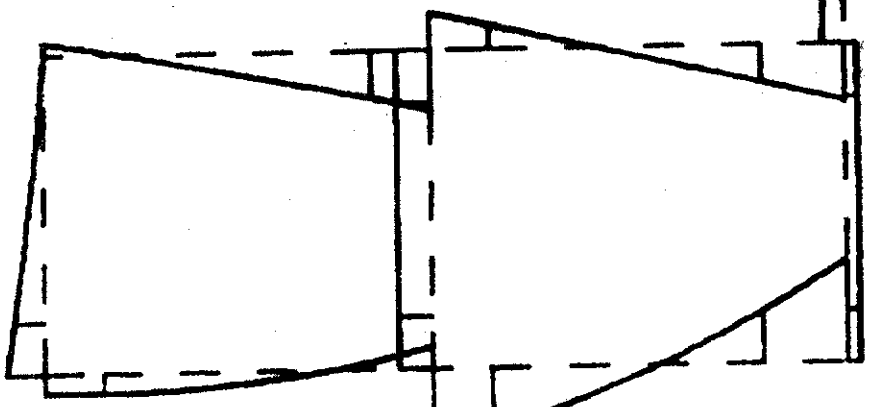
JOINT NUMBER	Y FIXITY	X FIXITY	ROT FIXITY	X (FEET)	Y (FEET)
1	0	0	0	2.00	0.
2	1	0	0	0.	0.
3	0	0	0	0.	2.00
4	0	0	0	0.	10.00
5	0	0	0	0.	12.00
6	0	0	0	2.00	12.00
7	0	0	0	11.00	0.
8	0	0	0	13.00	0.
9	0	0	0	13.00	2.00
10	0	0	0	15.00	0.
11	0	0	0	11.00	12.00
12	0	0	0	13.00	12.00
13	0	0	0	13.00	10.00
14	0	0	0	15.00	12.00
15	0	0	0	24.00	0.
16	1	1	0	26.83	0.
17	0	0	0	26.83	2.00
18	0	0	0	24.00	12.00
19	0	0	0	26.83	12.00

Plate C-2e

21	0	0	0	26.83	14.00
22	0	0	0	26.83	30.50

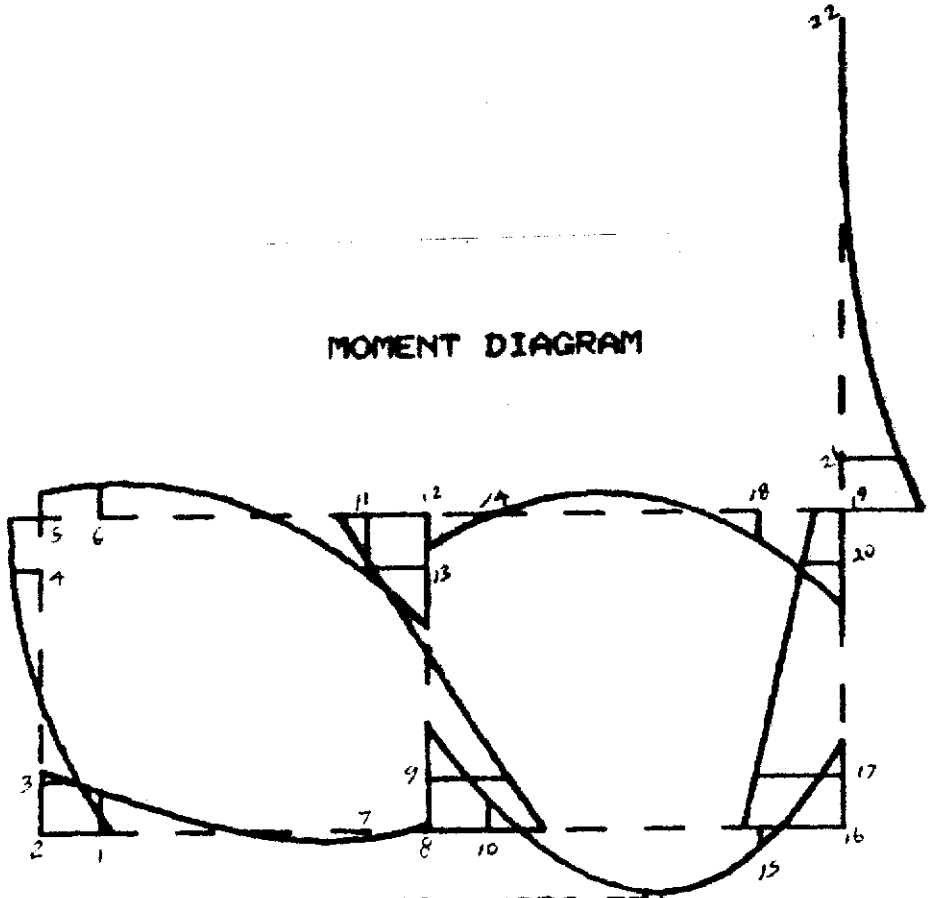
MEMBER NUMBER	JOINT LEFT	JOINT RIGHT	WIDTH OR I IN OR IN^4	HEIGHT OR 0 INCHES	AREA OR 0 INCHES^2
1	2	1	12.00	30.00	0.
2	2	3	12.00	30.00	0.
3	3	4	12.00	24.00	0.
4	4	5	12.00	30.00	0.
5	5	6	12.00	30.00	0.
6	1	7	12.00	24.00	0.
7	7	8	12.00	30.00	0.
8	8	10	12.00	30.00	0.
9	8	9	12.00	30.00	0.
10	9	13	12.00	24.00	0.
11	6	11	12.00	24.00	0.
12	13	12	12.00	30.00	0.
13	11	12	12.00	30.00	0.
14	12	14	12.00	30.00	0.
15	10	15	12.00	24.00	0.
16	15	16	12.00	30.00	0.
17	16	17	12.00	50.00	0.
18	17	20	12.00	44.00	0.
19	14	18	12.00	24.00	0.
20	18	19	12.00	30.00	0.
21	20	19	12.00	50.00	0.
22	19	21	12.00	50.00	0.
23	21	22	12.00	44.00	0.

SHEAR DIAGRAM



SCALE FACTOR=(1 IN= 80. KIPS)

MOMENT DIAGRAM



SCALE FACTOR=(1 IN= 160. KIPS-FT)

Plate C-2e

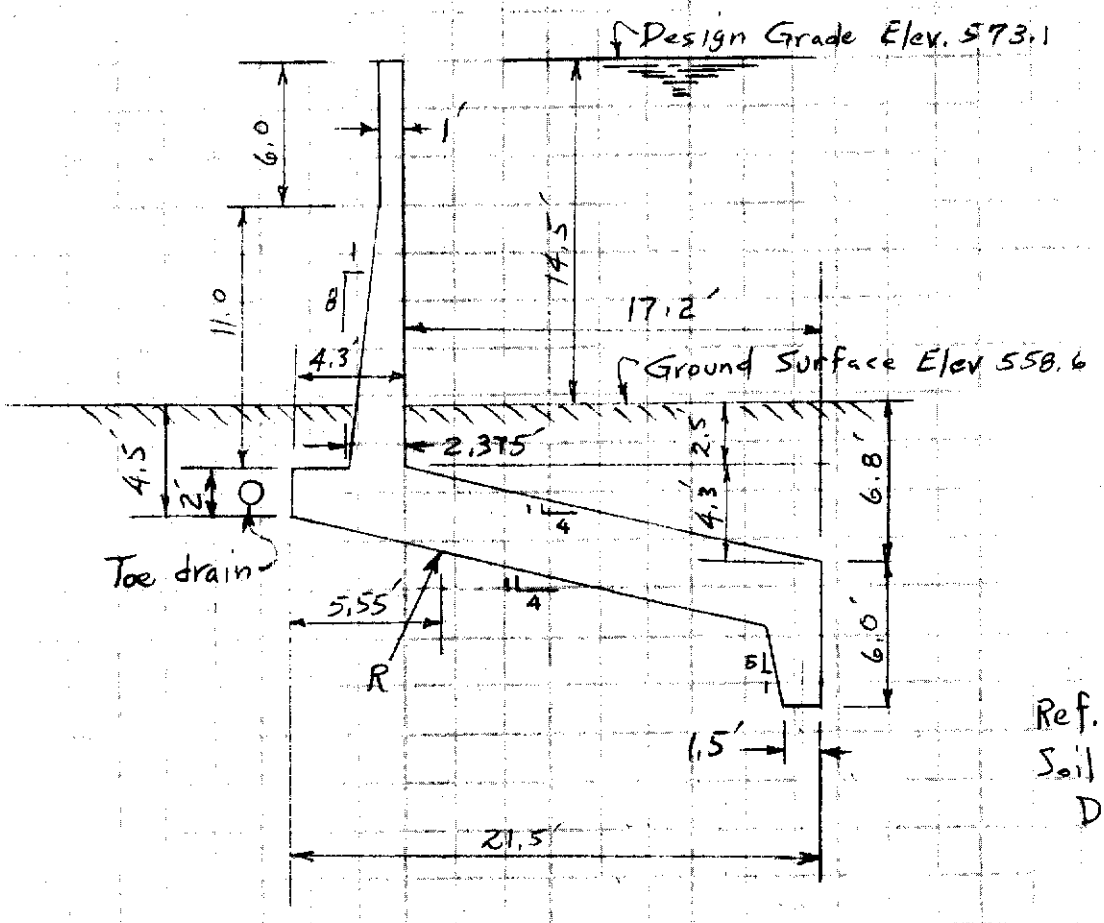
MEMBER END MOMENTS AND FORCES

MEM NO	JOINT LEFT	JOINT RIGHT	LENGTH (FT)	MOMENT LEFT (FT-K)	MOMENT RIGHT (FT-K)	SHEAR LEFT (K)	SHEAR RIGHT (K)	AXIAL LEFT (K)	AXIAL RIGHT (K)
1	2	1	2.00	-46.22	31.08	-7.57	7.57	12.05	-12.05
2	2	3	2.00	46.22	-24.60	12.05	-9.59	7.51	-7.51
3	3	4	3.00	24.60	17.58	9.59	-1.43	7.51	-7.51
4	4	5	2.00	-17.58	18.84	1.43	0.13	4.51	-4.51
5	5	6	2.00	-18.84	24.56	4.51	-1.16	0.13	-0.13
6	1	7	9.00	-31.08	-6.21	-7.57	-2.71	12.05	-12.05
7	7	8	2.00	6.21	4.42	2.71	-8.11	12.05	-12.05
8	8	10	2.00	-77.74	22.88	-30.60	24.07	22.91	-22.91
9	8	9	2.00	73.31	-51.61	10.25	-10.85	38.71	-36.71
10	9	13	8.00	51.61	35.23	10.85	-10.85	38.71	-38.71
11	6	11	9.00	-24.56	-43.02	1.16	17.02	0.13	-0.13
12	13	12	2.00	-35.23	56.94	10.85	-10.85	35.71	-35.71
13	11	12	2.00	43.02	-81.74	-17.02	21.74	0.13	-0.13
14	12	14	2.00	24.80	-1.74	13.97	-9.06	-10.73	10.73
15	10	15	9.00	-22.88	-15.56	-24.07	-19.36	22.91	-22.91
16	15	16	2.83	15.56	64.74	19.36	-37.76	22.91	-22.91
17	16	17	2.00	-64.74	56.90	-3.92	3.92	37.88	-37.83
18	17	20	8.00	-56.90	25.54	-3.92	3.92	37.83	-37.83
19	14	18	9.00	1.74	-21.01	9.06	13.35	-10.73	10.73
20	18	19	2.83	21.01	-68.75	-13.35	20.89	-10.73	10.73
21	20	19	2.00	-25.54	17.70	-3.92	3.92	32.33	-32.33
22	19	21	2.00	51.05	-37.43	6.81	-6.81	11.94	-11.94
23	21	22	16.50	37.43	0.00	6.81	-0.00	-0.00	0.00

STRUCTURE REACTIONS

JNT NO	MOMENT (FT-K)	VERTICAL FORCE (K)	HORIZONTAL FORCE (K)
2	0.	-0.06	0.
16	0.	0.07	-18.99

Subject	Davenport Ballfield Tee Wall	Date	June 81
Computed by	Checked by	Sheet	1 of 3



Ref.
Soil Boring
D-25

Use computer program #713-G1-F442A for tee wall analysis to determine base width (as shown above) to have resultant in middle half. Printout of results on following page.

Location of resultant $R = \frac{5.55}{21.5} = 0.267 > 0.25$ OK

Creep ratio = 1.93 > 1.8 OK

Analysis stability of foundation material in final design stage after testing soil for ϕ and C properties.

PLATE C-3a

```
COME TO THE BCS NETWORK
YOUR ACCESS PORT IS CIX 062

SELECT DESIRED SERVICE: EKS1

 81/06/17. 11.57.01.
EKS1 175G.N0501.31 B 81/06/07.DS-0 03.34.17. 81/06/17.
TERMINAL      346, TTY
RECOVER/USER.ID: DAP

 ◆◆◆  IMPORTANT ENHANCEMENTS SCHEDULED.  SEE MSG,ADVANCE.  ◆◆◆
◆
C>GET,TWALL
C>-TWALL
I>1DAVENPORT BALLFIELD WALL
1                    DAVENPORT BALLFIELD WALL

I>1.0, 8.0, 2.0, 4.5, 14.5, 14.5, 13.0, 4.0
I>4.0, 5.0, 1.5, 6.0BX4.0, 125.0, 5.0, 2.0
I>1.0, 6.0, 4.7, 172.0, 25.0, 1.0
```

STEM	SLO	TT	GL	XHL	XHR	B
1.000	8.000	2.000	4.500	14.500	14.500	13.000

SLB	SLH	SLK	XK	YK	FREE	WES
4.000	4.000	5.000	1.500	6.000	4.000	125.000

RATIO	DELTA	MTL	YBA	C1	C2	WIND
5.000	2.000	1.000	6.000	4.700	172.000	25.000

MINIMUM BASE WIDTH FOR MAX. WATER HT.

(LOADING NO. 1)

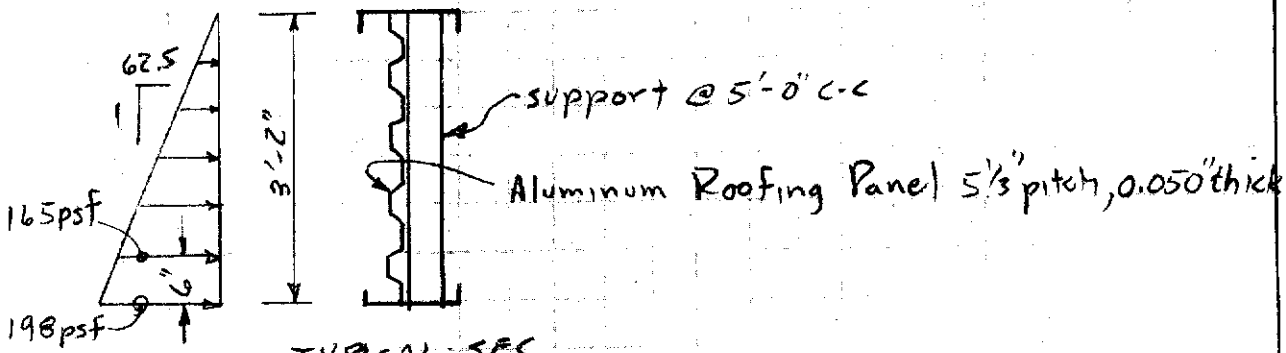
BASE WIDTH	21.50 FT.
VOLUME OF CONCRETE	3.49 CY. PER FT.
LOCATION OF RESULTANT	5.55 FT.
VERTICAL MAGNITUDE OF RESULTANT	19344.03 LBS.
NET HORIZONTAL WATER LOAD	12631.59 LBS.
MAXIMUM COVER OVER KEY	6.80 FT.
LENGTH OF TOE	4.30 FT.
HEEL	17.20 FT.
THICKNESS OF HEEL AT KEY	3.08 FT.
HEEL AT STEM	3.08 FT.
KEY AT BASE	2.19 FT.
TOE AT TOE	2.48 FT.
CREEP RATIO	1.93

PLATE C-36

Subject	Davenport Folding Wall	Date
Computed by DAP	Checked by	Sheet of

Sheeting of Upper Panels:

Ref. "Aluminum Formed Sheet Building Sheathing Design Guide", Aluminum Association, Nov. 69



TYPICAL SEC.

Allowable Stresses for panel using Alclad 3004-H16

Use $F_{ty} = 30$ ksi $F_{cy} = 28$ ksi $E = 10{,}100$ $n_y = 1.95$

$\omega = 2.33$ $b = 1.125$ $t = 0.050$ $\frac{b}{t} = 22.5$ $\frac{\omega}{t} = 46.6$

	$\frac{b}{t} \le 23$	$\frac{\omega}{t} \le 56$	$F_a = \frac{F_b + (\frac{\omega}{3b})F_w}{1 + \omega/3b}$
F_{bt}	15.3		$F_{at} = 17.4$ ksi
F_{bc}	13.7		
F_{wt}		20.5	$F_{ac} = 15.0$ ksi
F_{wc}		17.0	

$$M = \frac{wL^2}{12} = \frac{165}{12}(5)^2 = 348'\#$$

$$f_a \frac{M}{S} = \frac{348 \times 12}{0.354} = 11{,}660 < 15000 \quad \text{OK}$$

Subject	Davenport Folding Wall	Date
Computed by DAP	Checked by	Sheet of

Web Crippling of Panel Sheeting:

Panel supports are 2" wide (comps. follow on panel vertical supports)

At Interior Support: (check exterior support condition in final design)

$$C_p = \frac{2.5E\left(\frac{N}{w}+1\right)}{F_{cy}\left(6+0.4\frac{N}{t}\right)\left[1.1-0.1\left(\frac{\theta}{90}\right)\sqrt{\frac{R_i}{t}}\right]}$$

$$C_p = \frac{2.5(10100)\left(\frac{2}{2.33}+1\right)}{28\left(6+0.04\left(\frac{2}{0.05}\right)\right)\left[1.1-0.1\left(\frac{48.6}{90}\sqrt{\frac{0.125}{0.05}}\right)\right]} = 217$$

$$\frac{w}{t} = \frac{2.33}{0.05} = 47 < 217$$

$$\text{Allowable interior reaction} = 0.6\,\frac{F_{cy}\,d\,t^2}{w}\left(6+0.04\frac{N}{t}\right)\left[1.1-0.1\left(\frac{\theta}{90}\right)\sqrt{\frac{R_i}{t}}\right]$$

$$P_{c\ allow} = 0.6\,\frac{28(1.75)(0.05)^2}{2.332}\left(6+0.04\left(\frac{2}{0.05}\right)\right)\left[1.1-0.1\left(\frac{48.6}{90}\right)\sqrt{\frac{0.125}{0.05}}\right]$$

$P_{c\ allow} = 0.243^{K}$ per web

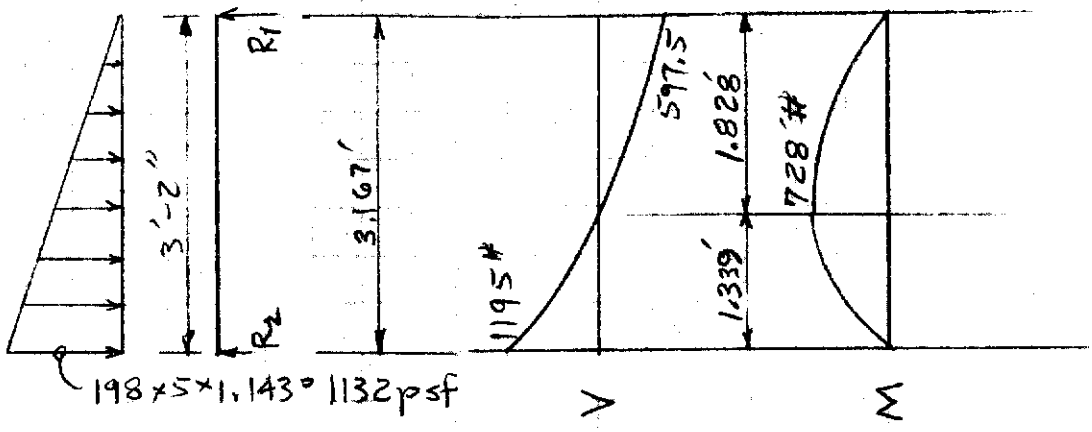
Using safety factor = 1.95, $P_{c\ allow} = \frac{1.65}{1.95}(0.243) = 0.206^{K}$/web

$P_{allow} = 206 \times \frac{12}{5.33} \times 2 = 927^{\#}$ on a per ft. basis

$P_{applied} = 1.143(165)5 = 943 \sim 927$ call OK

Subject	Davenport Folding Wall	Date
Computed by DAP	Checked by	Sheet of

Vertical Supports for Sheeting: Use Alloy 6061-T6



$R_1 = \frac{1}{3}\left(\frac{1}{2}\right)(1132)(3.167) = 597.5\#$

$R_2 = 2R_1 = 1195\#$

$\frac{1}{2}(x)\left(\frac{x}{3.167}\right)(1132) = 597.5$

$x = 1.828'$

$M_{max} = \frac{1}{2}(1.828)^3\left(\frac{1132}{3.167}\right)\left(\frac{1}{3}\right) - 1.828(597.5) = 728.3'\#$

Compression flange secured to sheeting ∴ no buckling

Allowable Stresses:

$F_{tu} = 38\ ksi \div F/S = 2.3$ allow $F_t = 16{,}500\ psi$

$F_{ty} = 35\ ksi \div F/S = 1.95$ allow $F_t = 18{,}000\ psi$

Tension load from strut = 2612# (see following comps. for struts)

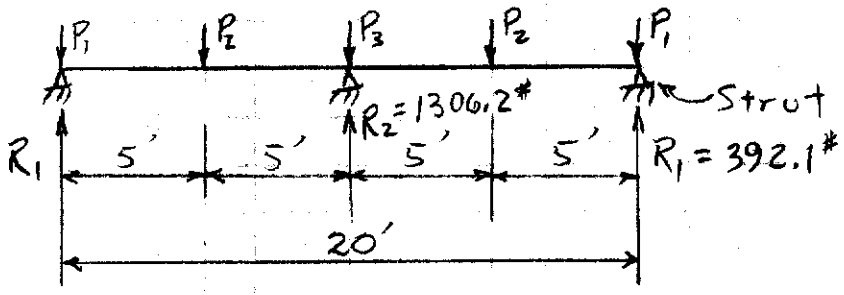
For W 2½ x 2 x 0.928

$$f = \frac{728.3 \times 12}{0.665} + \frac{2612}{0.789}$$

$$= 16{,}453 < 16{,}500 \quad \text{OK}$$

Subject	Davenport Folding Wall	Date
Computed by DAP	Checked by	Sheet of

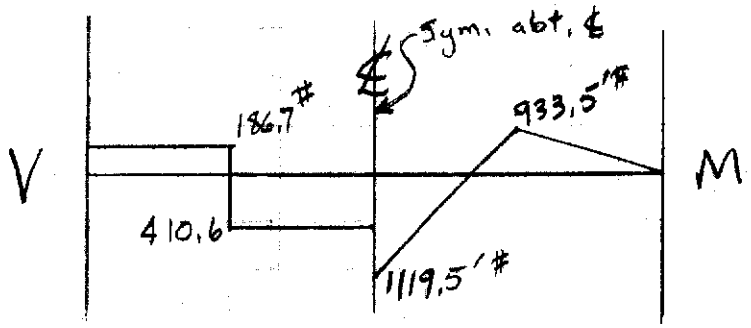
Horizontal Beam Top of Upper Panel:



$P_1 = [\frac{1}{3}(\frac{1}{2})(198)(5)(3.167)(0.393) = 205.4^{\#}$

P_2 ["] $(1.143) = 597.3$

P_3 ["] $(0.928) = 484.9$



With comp. flange welded to vertical supports $L_B = 60''$

$$B_c = F_{cyw}\left[1 + \left(\frac{F_{cyw}}{2250}\right)^{1/2}\right] = 20\left[1 + \left(\frac{20}{2250}\right)^{1/2}\right] = 21.9$$

$$D_c = \frac{B_c}{10}\left(\frac{B_c}{E}\right)^{1/2} = \frac{21.9}{10}\left(\frac{21.9}{10100}\right)^{1/2} = 0.102$$

$$S_1 = \frac{1.2(B_c - F_{cyw})}{D_c} = \frac{1.2\,(21.9 - 20)}{0.102} = 22$$

Subject	Davenport Folding Wall	Date
Computed by DAP	Checked by	Sheet of

Horizontal Beam Top of Upper Panel continued:

$$C_c = 0.41 \frac{B_c}{D_c} = 0.41\left(\frac{21.9}{0.102}\right) = 88$$

$$S_2 = 1.2\,C_c = 1.2(88) = 105$$

For C5 × 1.75 × 1.810 $\quad L/r = \frac{60}{0.571} = 105 > S_1 = S_2$

$$F_{allow.} = \frac{1}{1.95}\left(B_c - \frac{D_c}{1.2}\frac{L}{r_y}\right)$$

$$= \frac{1}{1.95}\left(21.9 - \frac{0.102}{1.2}(105)\right)$$

$$F_{allow.} = 6650$$

$$f_a\,applied = \frac{1120 \times 12}{2.107} = 6379$$

$$\frac{h}{t} = 5.0/.188 = 26.6 > S_1 < S_2$$

$$B_s = F_{syw}\left[1 + \frac{(F_{syw})^{1/3}}{9.3}\right] = 12\left[1 + \frac{(12)^{1/3}}{9.3}\right] = 14.9$$

$$D_s = \frac{B_s}{10}\left(\frac{B_s}{E}\right)^{1/2} = \frac{14.9}{10}\left(\frac{14.9}{10100}\right)^{1/2} = 0.057$$

$$F_s = \frac{1}{1.95}\left[B_s - 1.25\,D_s\left(\frac{h}{t}\right)\right] = \frac{1}{1.95}\left[14.9 - 1.25(0.057)(26.6)\right]$$

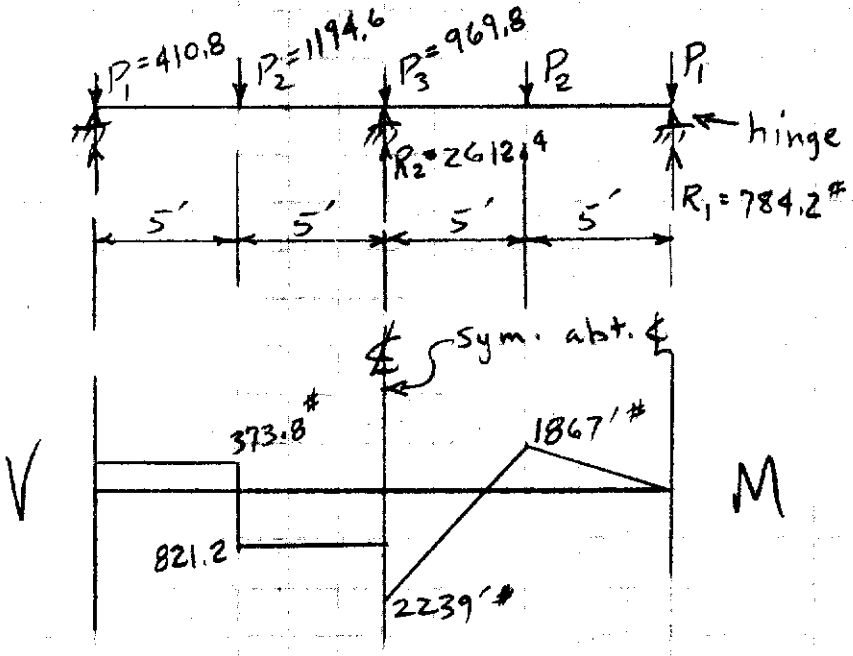
$$F_s\,allow. = 6,670 \quad Use \frac{12000}{1.95} = 6150$$

$$f_s\ applied = \frac{410}{5(0.188)} = 436.$$

$$\frac{f_a}{F_a} + \left(\frac{f_s}{F_s}\right)^2 = \frac{6379}{6650} + \left(\frac{436}{6150}\right)^2 = 0.96 < 1.0 \quad OK$$

Subject	Davenport Folding Wall	Date
Computed by DNP	Checked by	Sheet of

Horizontal Beam Bottom of Upper Panel:



Comp. flange secured to hinges and vertical supports

$L_B = 60''$

For C 5×2.25×2.212 $\quad L/r = \frac{60}{2.05} = 29 > S_1 < S_2$

$$F_{a\,allow.} = \frac{1}{1.95}\left(B_c - \frac{D_c}{1.2}\frac{L}{r_y}\right) = \frac{1}{1.95}\left(21.9 - \frac{0.102}{1.2}\times 29\right) = 9900$$

$$f_a\ applied = \frac{2239\times 12}{3.15} = 8530$$

$$h/t = \frac{5}{0.15} = 33.3$$

$$F_s\ allow = \frac{1}{1.95}\left[B_s - 1.25 D_s\left(\frac{h}{t}\right)\right] = \frac{1}{1.95}\left[14.9 - 1.25(0.057)(33.3)\right]$$

F_s allow = 6424 use $\frac{12000}{1.95} = 6150$

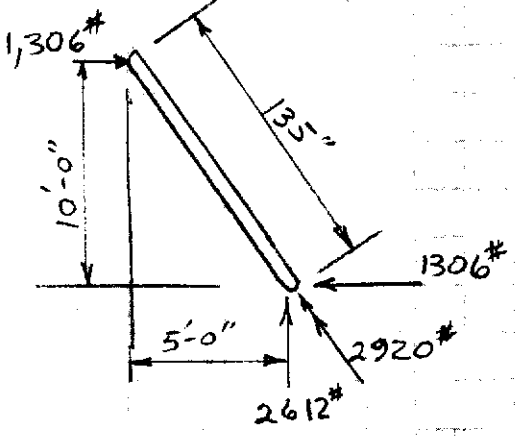
$$f_s\ applied = \frac{821.2}{5\times 0.15} = 1095$$

$$\frac{f_a}{F_a} + \left(\frac{f_s}{F_s}\right)^2 = \frac{8530}{9900} + \left(\frac{1095}{6150}\right)^2 = 0.89 < 1.0 \quad \underline{OK}$$

Subject	Davenport Folding Wall	Date
Computed by DAP	Checked by	Sheet of

Struts for Upper Panels:

Middle Strut



for 2"ϕ steel pipe $\frac{\ell}{r} = \frac{135}{0.787} = 171 > C_c = 126$

Allowable axial stress = $\frac{124{,}000{,}000}{K(\ell/r)^2}$ $K = 1$

$= \frac{124{,}000{,}000}{(171)^2}$

$= 4200$ psi

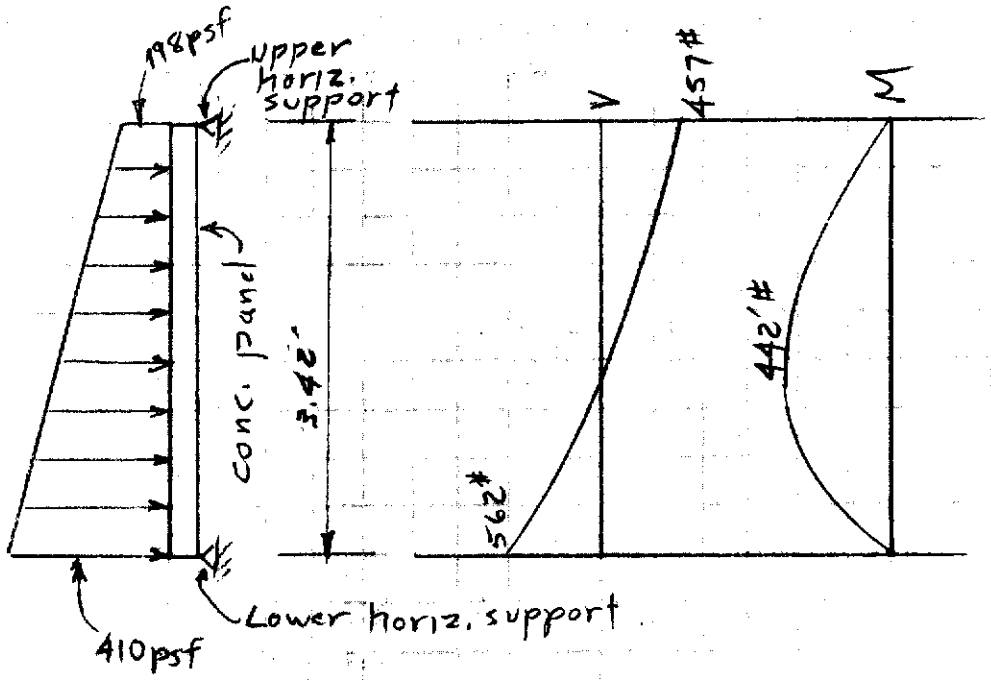
Applied stress = $\frac{2920}{1.07} = 2730$ psi < 4200 OK

Strut wt. = 3.65 plf × 12' = 45# ±

Use same pipe for strut at ends of upper panels, load = 392 × 2 = 785# < 1306 OK

Subject		Date
Computed by	Checked by	Sheet of

Reinforcing for Lower Panel Under Water Loading



$$\frac{M_u}{\phi f'_c b d^2} = \frac{1.4(442\times12)}{0.9(3000)(12)(3)^2} = 0.0255$$

$$\omega = 0.026\ ,\quad \rho = \omega\frac{f'_c}{f_y} = 0.026\left(\tfrac{3}{40}\right) = 0.00195$$

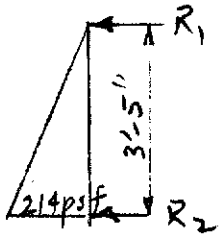
$$A_s = 0.00195(3)(12) = 0.07\ \text{in}^2/\text{ft}$$

Use # 4 @ 12" c-c

$A_s = 0.20\ \text{in}^2$

Subject	Davenport, Folding Wall	Date
Computed by	Checked by	Sheet 2 of

Beam at Top of Lower Panel, Water to Top of Lower Panel:



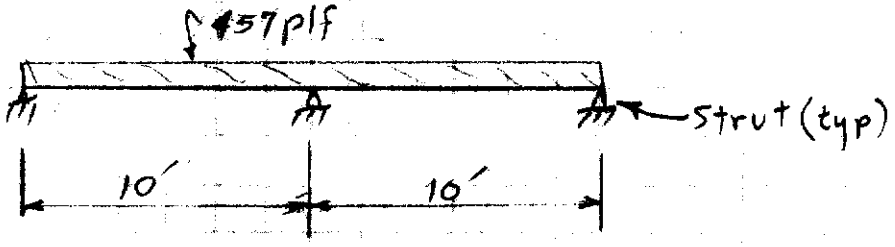
$R_1 = \frac{1}{3}(\frac{1}{2})(214)(3.417) = 121.9 \sim 125$ plf

$M_{max} = \frac{wl^2}{8} = \frac{125}{8}(20)^2 = 6250'\#$

$S_{req'd} = \frac{6250 \times 12}{18000} = 4.17\ in^3$

Use C 6×8.2, $S = 4.38\ in^3$

Beam at Top of Lower Panel, Water to Design Grade:
(Middle Strut in Place)



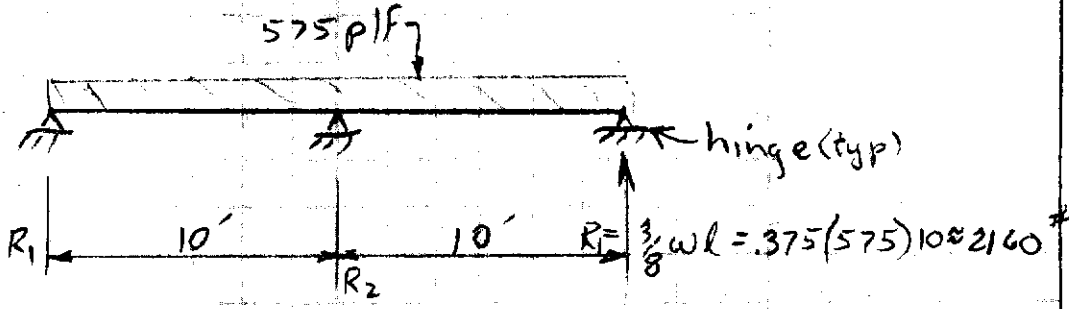
$M_{max} = \frac{1}{8}wl^2 = \frac{1}{8}(460)(10)^2 \approx 5750'\# < 6250$ above

Center reaction $= \frac{10}{8}wl = \frac{10}{8}(460)(10) = 5750\#$

End " $= \frac{3}{8}wl = 1725\#$

Subject	Davenport, Folding Wall	Date
Computed by	Checked by	Sheet 3 of

Beam at Bottom of Lower Panel



At Center Support

$M_{max} = \frac{1}{8}wl^2 = \frac{1}{8}(575)(10.0)^2 = 7188 \sim 7200'^{\#}$

$$\frac{M_u}{\phi f'_c b d^2} = \frac{1.4(7200)(12)}{0.9(3000)(3.5)(12)^2} = 0.0889$$

$\omega = 0.094 \qquad p = 0.094\left(\frac{3}{40}\right) = 0.0071$

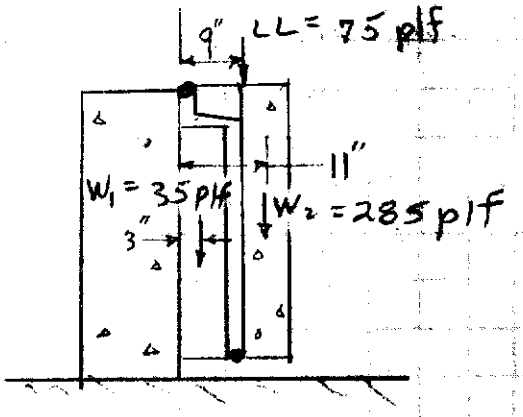
$A_s = 0.0071(3.5)12 = 0.30^{\square''}/ft$

secure edge $\angle$s to conc. w/ studs
edge $\angle$s will provide $> 0.30^{\square''}/ft$

Subject	Davenport, Folding Wall	Date
Computed by	Checked by	Sheet of

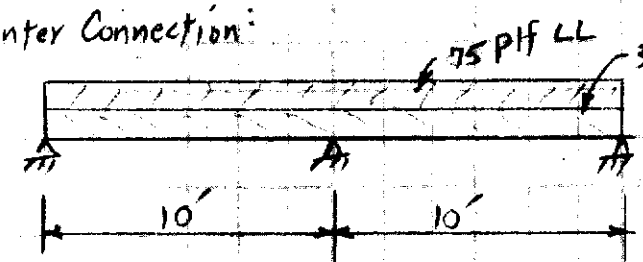
Lower Panel Concrete:

Load Transfer Between Hinge and Panel in Stored Position:



Conc. panel wt
$0.5(3.5)(150) = 262$
Edge Beams ~ 20
~ 285 plf

Load on Center Connection:



Center reaction $= \frac{10}{8} wl = \frac{10}{8}(320)(10) = 4000^{\#}$ DL
$= \frac{10}{8}(75)(10) = 940^{\#}$ LL

$M_{DL} = 4000 \times 11" = 44000"^{\#}$ DL
$M_{LL} = 940 \times 9" = 8{,}460"^{\#}$ LL

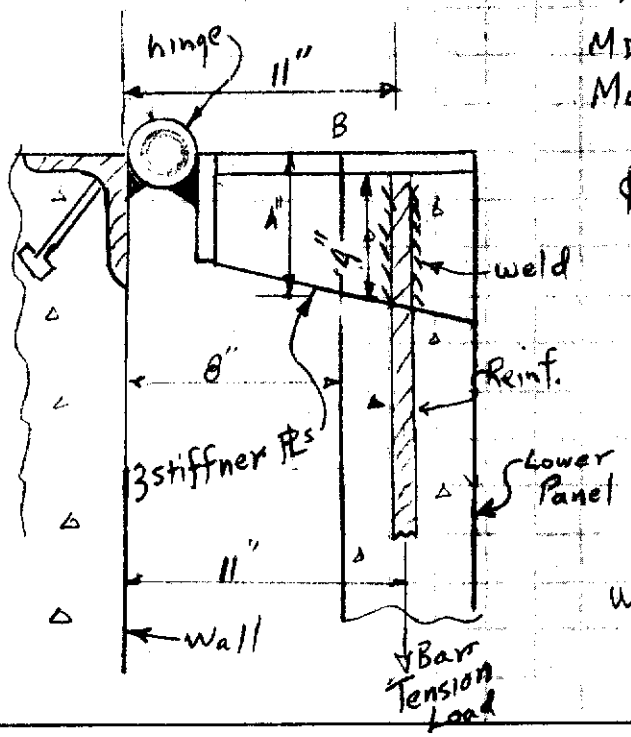
$$\frac{M_u}{\phi f'_c b d^2} = \frac{1.4(44000) + 1.7(8460)}{0.9(3000)(12)(3^2)} = 0.261$$

$\omega = 0.323$

$$\rho = \omega \frac{f'_c}{f_y} = 0.323\left(\frac{3}{40}\right) = 0.0242$$

$A_s = 0.0242(3)(12) = 0.87^{a"}$

Weld a #5 to each stiffner (at least 3, $A_s = 0.91$)



Weld req'd:
$\frac{0.31(20000)}{575(3)} = 3.59"$ of 3/16 weld
Use 4" min. each bar

Plate C-4k

Subject	Davenport Folding Wall	Date
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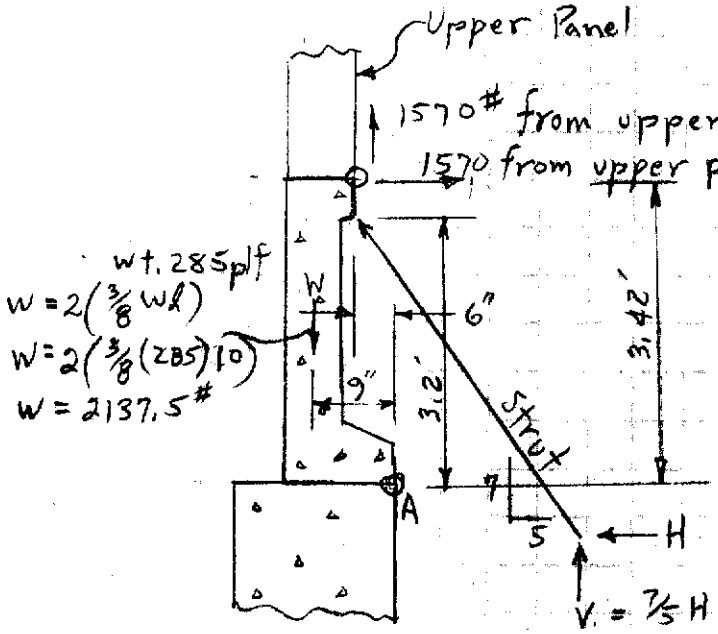
End Strut for Lower Panels:



Note: When upper panel is not raised loading is less critical than loading shown.

Moments about A to find H

$$3.2H - 0.5\left(\tfrac{7}{5}H\right) + 0.75(2137.5) - 1570(0.5) - 5020(3.42) = 0$$

$H = 6540\#$

$V = 9156\#$

Strut load = 11,250#

For 2½" Φ steel pipe $\frac{l}{r} = \frac{105''}{0.947} = 111$ $K = 1$ $C_c = 126$

$$F/S = \frac{5}{3} + \frac{3}{8}\frac{(Kl/r)}{C_c} - \frac{(Kl/r)^3}{8C_c^3} = \frac{5}{3} + \frac{3}{8}\frac{(111)}{126} - \frac{(111)^3}{8(126)^3} = 1.91$$

$$K_1 = \frac{1 - \left[\frac{(Kl/r)^2}{2C_c^2}\right]}{F/S} = \frac{1 - \left[\frac{(111)^2}{2(126)^2}\right]}{1.91} = 0.322$$

Fallowable = 0.322(0.83)36 = 9621 psi

$f_{applied} = 11250/1.70 = 6618 < 9621$ OK

Subject	Davenport Folding Wall	Date
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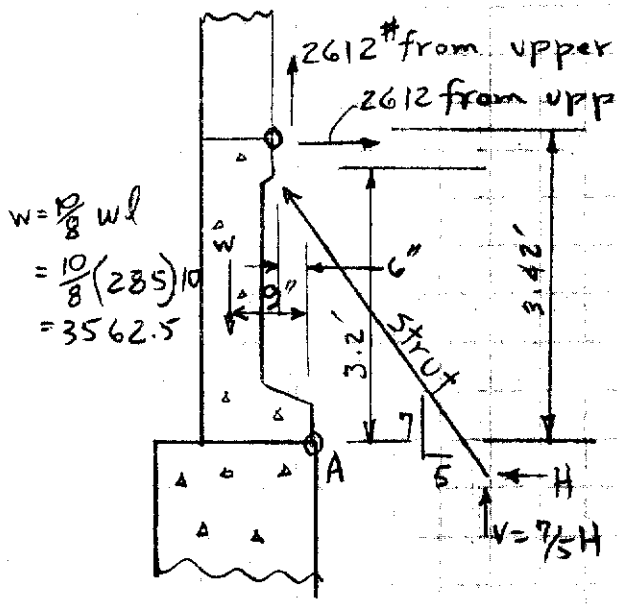
Middle Strut for Lower Panel:



Mom. abt A to find H

$3.2H - 0.5\left(\frac{7}{5}H\right) + 0.75(3562.5) - 2612(0.5) - 8362(3.42) = 0$

$H = 10893$

$V = 15250$

Strut axial load = 18741#

For 3×3×3/16 steel tube $\frac{l}{r} = \frac{105}{1.13} = 93$ $K=1$ $C_c = 126$

$$\frac{F}{S} = \frac{5}{3} + \frac{3}{8}\frac{(K\frac{l}{r})}{C_c} - \frac{(K\frac{l}{r})^3}{8C_c^3} = \frac{5}{3} + \frac{3}{8}\frac{(93)}{126} - \frac{(93)^3}{8(126)^3} = 1.89$$

$$K_1 = \frac{1 - \left[\frac{(K\frac{l}{r})^2}{2C_c^2}\right]}{F/S} = \frac{1 - \left[\frac{(93)^2}{2(126)^2}\right]}{1.89} = 0.385$$

$F_{allow.} = 0.385(0.83)(36) = 11{,}500$ psi

$f_{applied} = \frac{18741}{2.02} = 9278 < 11{,}500$ OK

wt = 6.86 × 9' ± ≈ 65#

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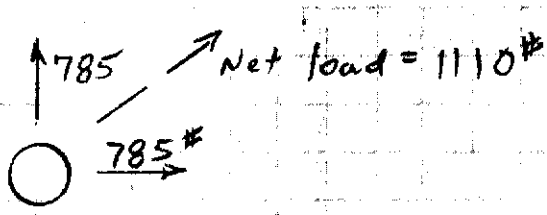
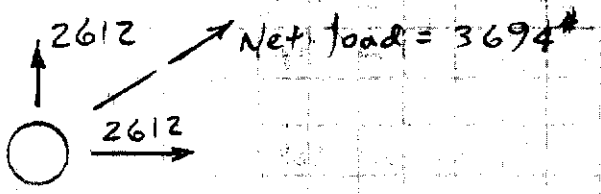
Hinges:

① Upper Panel End Hinge



② Upper Panel Center Hinge:



③ Lower Panel End Hinge:

$V = 785 + \frac{9156}{2} - \left(\frac{3}{8}(285)(10)\right) = 4294$

(lower strut vert. force; Upper Strut thrust; Portion of panel wt.)

$H = 785 + 1725 - \frac{1}{2}(6540) + 2160 = 1400$

Net Load = 4516#

④ Lower Panel Center Hinge:

$2612 + 15250 - 3562 = 14300$

$2612 + 5750 - 10893 + \frac{10}{8}(575)(10) = 4657$

Net Load = 15039

Subject	Davenport Folding Wall	Date
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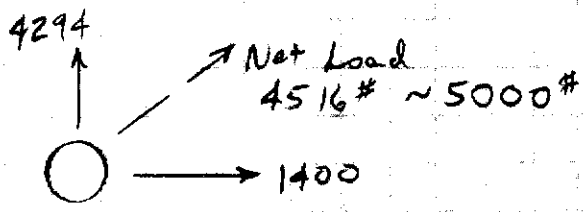
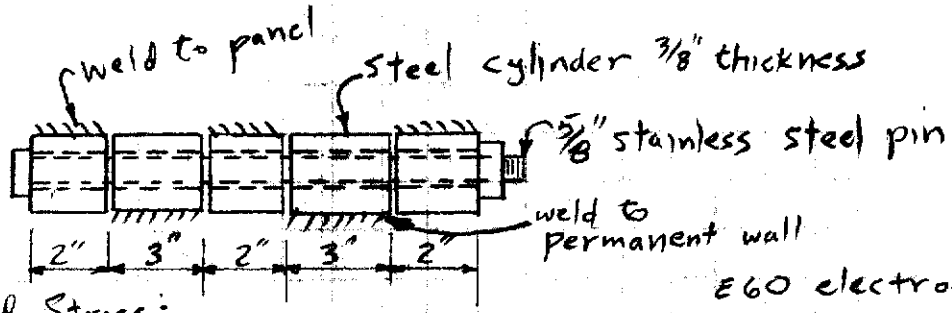
Hinge at End of Lower Panels connecting to Permanent Wall:

I. Water Loading to Design Grade:





Weld Stress:

E60 electrode

allowable load for 3/16" weld 1" long = $0.707(0.0625)(3)(11500)$
$= 2000\#$

applied load = $\frac{5000}{6''} = 830 < 2000$ OK

Pin Shear:

$$\frac{5000}{\pi/4\,(0.625)^2\,4} = 4080 < 10250 \quad \text{OK}$$

Cylinder Shear:

$$\frac{5000}{(0.375)(6)\,2} = 1110 < 10250 \quad \text{OK}$$

Subject	Davenport Folding Wall	Date
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Wind Loading of 30 psf

Check Upper Panel

Upper Panel Sheeting designed for 165 psf > 30 OK

Vertical supports on upper panels:

$$M = \frac{30(5)(1.143)(3.167)^2}{8} = 215'\# < \text{water load}$$

∴ OK

Wind load on upper panel top horiz. beam

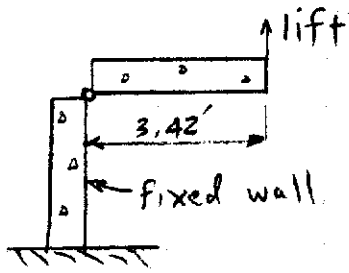
$= 30(5)(1.143)(\tfrac{1}{2}) = 88\# <$ water load OK

Other members of upper panel and struts also OK for wind loading.

Lower Panel

water loading is much larger ∴ OK

Erection Lift Loading



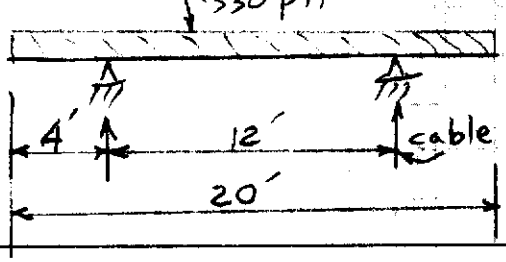
Wt. of Panels and struts = 330 plf

$$M = \frac{wL^2}{8} = \frac{(330/3.42)(3.42)^2}{8}$$

$M = 141'\# <$ water loading

∴ lower panel OK

Check top beam of lower panel using 2 pt. lift



$M_{max} = 3300'\# < 6250'\#$ from water loading ∴ OK

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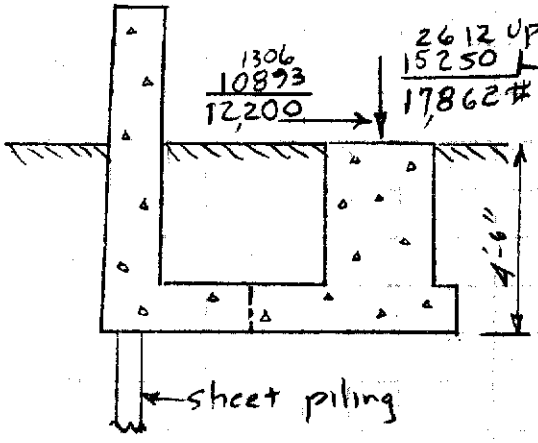
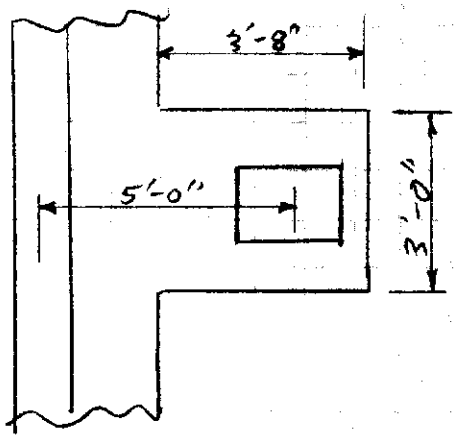
Footing for Struts:



Section

Ref. Borings D-27, D-28

Material is primarily sandy clay. Assume allowable bearing = 1 TSF
Evaluate material more fully before final design



PLAN

$\frac{18000\#}{3 \times 3} = 2000 \text{ psf}$
$= 1 \text{ TSF}$ OK

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Davenport Main St. Closure / Panel & Strut	
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Note: Computer Output is Not Included

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Loading on Lower Panels	1, 2
Computer Analysis Lower Panels	3
Beam and Decking Selection Lower Panels	4, 5
Upper Panel Loading & Member Selection	6
Computer Analysis Upper Panels	7
Design of Vertical Supports	8 - 15
Strut Design	16, 17
Stability	18, 19
Piling	20, 21

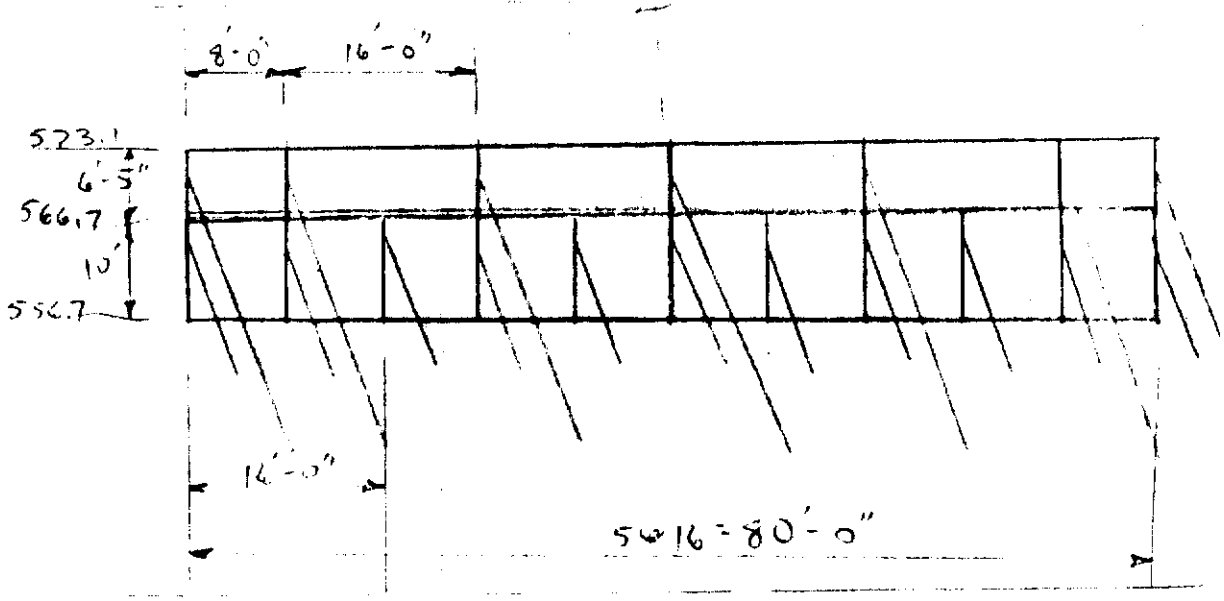

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Plate C-5a

Subject	Davenport, Main St. Closure	Date
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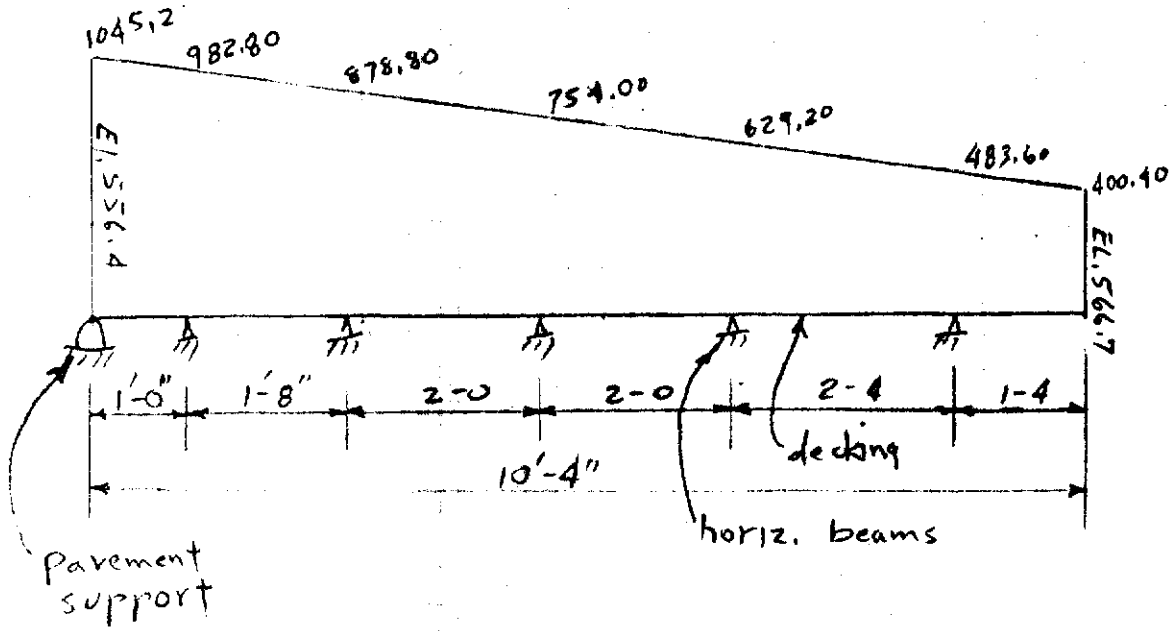
Lower Section of Closure

Loading: Water to Design Grade (Load #1)



Ref. Computer Strudl solution DAY6
for reaction loads on horiz beams and decking
shears & moments.

Wind Load (Load #2) 25 psf uniform load

Subject	Davenport Main St. Closure	Date
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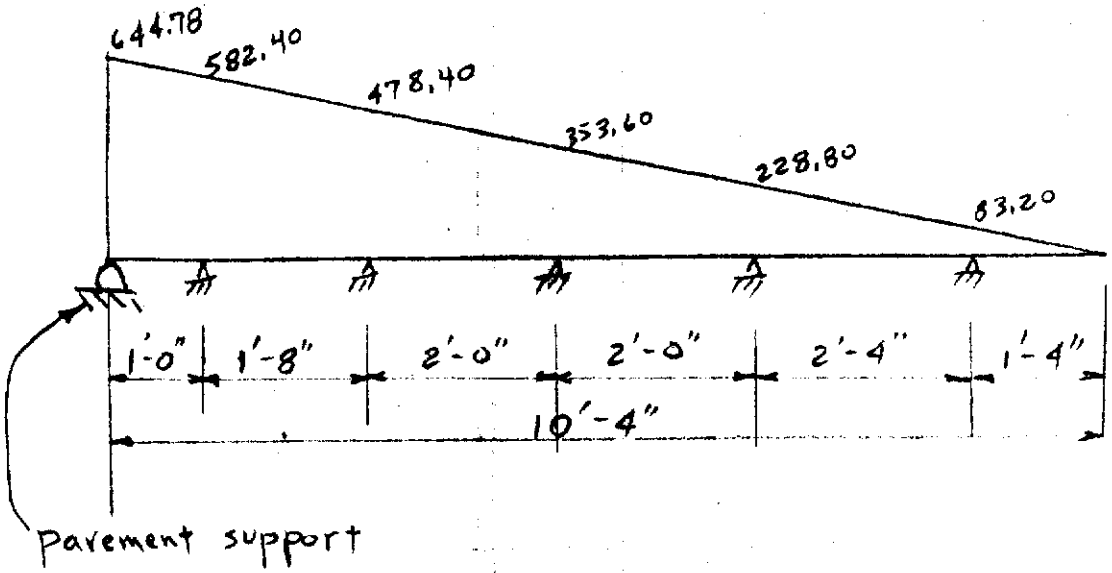
Lower Panel
Water to Top (10'-4") of Panels (Load #3)



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Horizontal Beams, Lower Panels:

Load condition #1, Water to Design Grade,

$$M_{max} = \frac{wl^2}{8} = \frac{w_1(8)^2}{8} = 8w_1$$

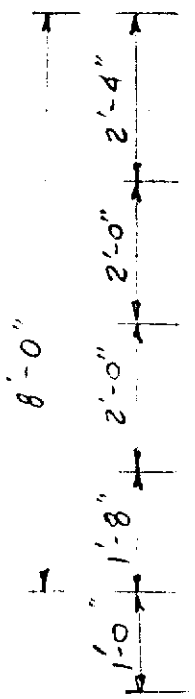
Comp. Flange is supported by decking and vertical beams

Load condition #2 Wind, not significant in magnitude but may be opposite water load, therefore use positive connections

Load condition #3 Water to 10'-4"

$$M_{max} = \frac{wL^2}{8} = \frac{w_3(16)^2}{8} = 32w_3$$

Use Steel Beams Next Page



Beam	Load #1 w_1	$M_{max} = 8w_1$	Load 3 w_3	$M_{max} = 32w_3$	S req'd	Alum. Beam	S provided
5	1288 plf	10304 '#	112	3,584	7.50 in³	C8×3 ×4.147	9.35 in³
4	1242	9920	514	16416	11.94	C8×3.75 ×5.789	13.17
3	1539	12,312	703	22,500	16.36	W8×5 ×7.023	16.94
2	1653	13,224	904	28,930	21.03	W8×6½ ×8.32	21.04
1	1406	11,248	832	26,625	19.36	W8×6½ ×8.32	21.04

Max. allowable reaction on decking with 6½" wide support is 1366 # < 1653 ∴ Use steel decking and steel horiz. beams (galv.).

Subject	Davenport Main St. Closure	Date
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Decking: $M_{max} = 380'^{\#}$

for Steel Deck Institute Type W, 18 gage, wt 2.68 psf galv.

$f_s = \frac{380 \times 12}{0.28} = 16,286 \text{ psi} < 18000$ OK

Horiz. Beams:

	M max	S req'd	Use Steel Beam	S Furnished
2'-4"	10,304'#	6.87 in³	W8×4×10	7.81
2'-0"	16,416	10.94	W8×4×15	11.8
2'-0"	22,500	15.00	W8×5¼×18	15.2
1'-8"	28,930	19.29	W8×6.5×24	20.9
	26,625	17.75	W8×6.5×24	20.9

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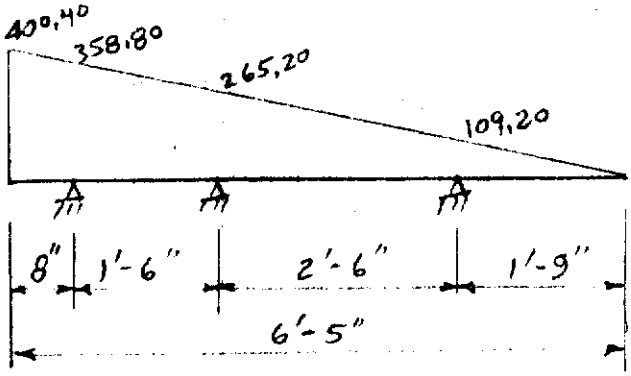
Upper Section of Closure

Loading: Water to Design Grade



Decking:

Steel Deck Institute Type W, 20 gage, wt 2.08 psf galv.

$$f_s = \frac{93 \times 12}{0.20} = 5580 < 18000 \quad OK$$

Horiz. Beams

$$M = \frac{wL^2}{8} = 509(16)^2/8 = 16288'\#$$

$$S_{req'd} = \frac{16288 \times 12}{18000} = 10.86 \text{ in}^3$$

Use W8×4×15 for 2 Lower Beams
$S = 11.8 \text{ in}^3$ OK

$$M = 283(16)^2/8 = 9056'\#$$

$$S_{req'd} = \frac{9056 \times 12}{18000} = 6.04 \text{ in}^3$$

Use W8×4×10 $S = 7.81 \text{ in}^3$ OK

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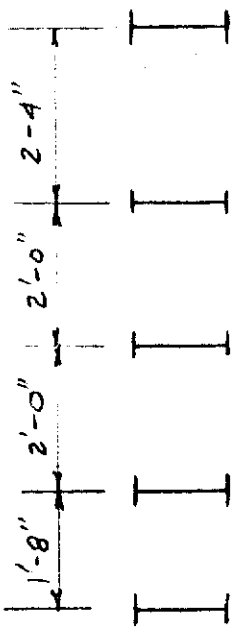
Distribution of Load from Horiz. Beams To Vertical Supports with water to Design Grade:

The shorter vertical beams carry less load than the tall vertical beams since the short beams are located at the ends of the lower panels and the tall ones are at the midpoint.

Load Distribution: $\ell = 8'$



Applied Load plf	Load on Midspan Tall Vert. Beams $\frac{10}{8} w\ell$	Load on End short Vert. Beams $\frac{7}{8} w\ell$
1288	12880	9,020
1242	12420	8,700
1539	15390	10,780
1653	16530	11,570
1406	14060	9,849

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Vertical Support Beams for Upper & Lower Panels:

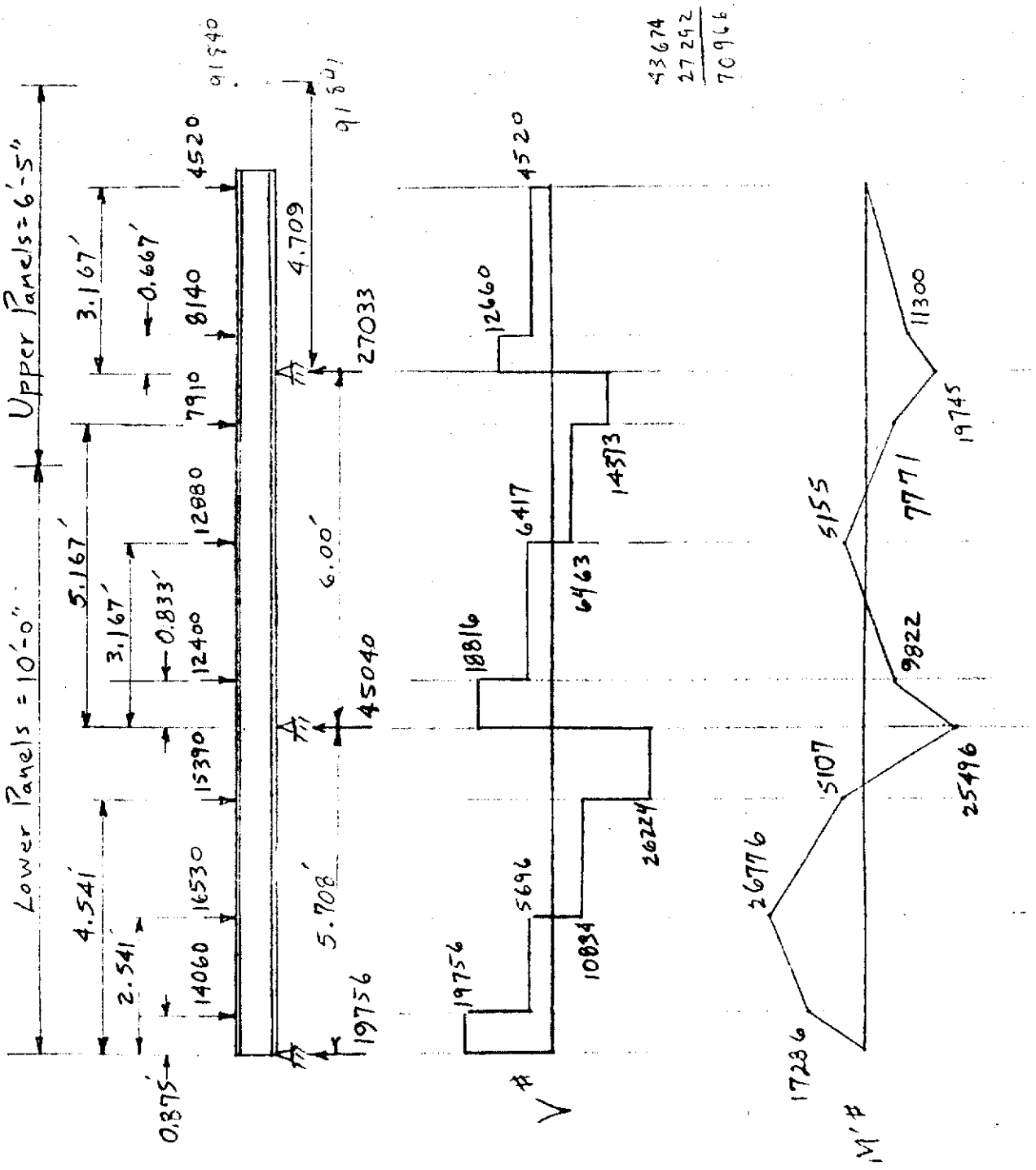

Plate C-5h

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Vertical Support Beams for Upper & Lower Panels: (Aluminum)

Lower 4½' of beam has comp. flange on riverside where horizontal beams will be bolted to flange and will thus provide lateral support against buckling.

Similar support is provided by horizontal beam at 8¾' from bottom. Between 4.5' and 8.7' the vertical beam is not supported against lateral buckling of comp. flange, thus $L = 50''$, $M = 25{,}500'^{\#}$.

From 8.7' height to 14.9' (top) compression flange is not supported, thus $L = 74''$, $M = 19{,}750'^{\#}$.

The two struts place the lower portion of the beam in tension which will aid the lateral stability.

$B_c = 39.4$, $D_c = 0.246$, $C_c = 66$

For W12 × 7 × 11,672 $\quad \frac{L}{r} = \frac{74}{1.65} = 45$

$$S_1 = \frac{1.2(B_c - F_{cy})}{D_c} = \frac{1.2(39.4 - 35)}{0.246} = 21 < 45 < 66$$

Upper Part

$$F_{allow} = \frac{1}{n_y}\left(B_c - \frac{D_c}{1.2}\frac{L}{r}\right) = \frac{1}{1.95}\left(39.4 - \frac{0.246}{1.2}(45)\right) = 15{,}620$$

$$f_{applied} = \frac{19800 \times 12}{42.60} = 5580 < 15{,}620 \quad \text{ok}$$

Middle Part

$$F_{allow} = \frac{1}{1.95}\left(39.4 - \frac{0.246}{1.2}\left(\frac{50}{1.65}\right)\right) = 19{,}000 \text{ use } 16{,}500$$

$$f_{applied} = \frac{25500 \times 12}{42.60} = 7200 < 16500 \quad \text{ok}$$

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(12)

Vertical Support Beams for Upper & Lower Panels, cont.:

Lowest Part

$$f_b \text{ applied} = \frac{26800 \times 12}{42.60} = 7550$$

$$f_t \text{ applied} = \frac{45040 + 27033}{9.925} = 7260$$

$$\frac{f_t}{F_{at}} + \frac{f_L}{F_{ab}} = \frac{7260}{16500} + \frac{7550}{16500} = 0.44 + 0.46 = 0.90 < 1.0 \text{ OK}$$

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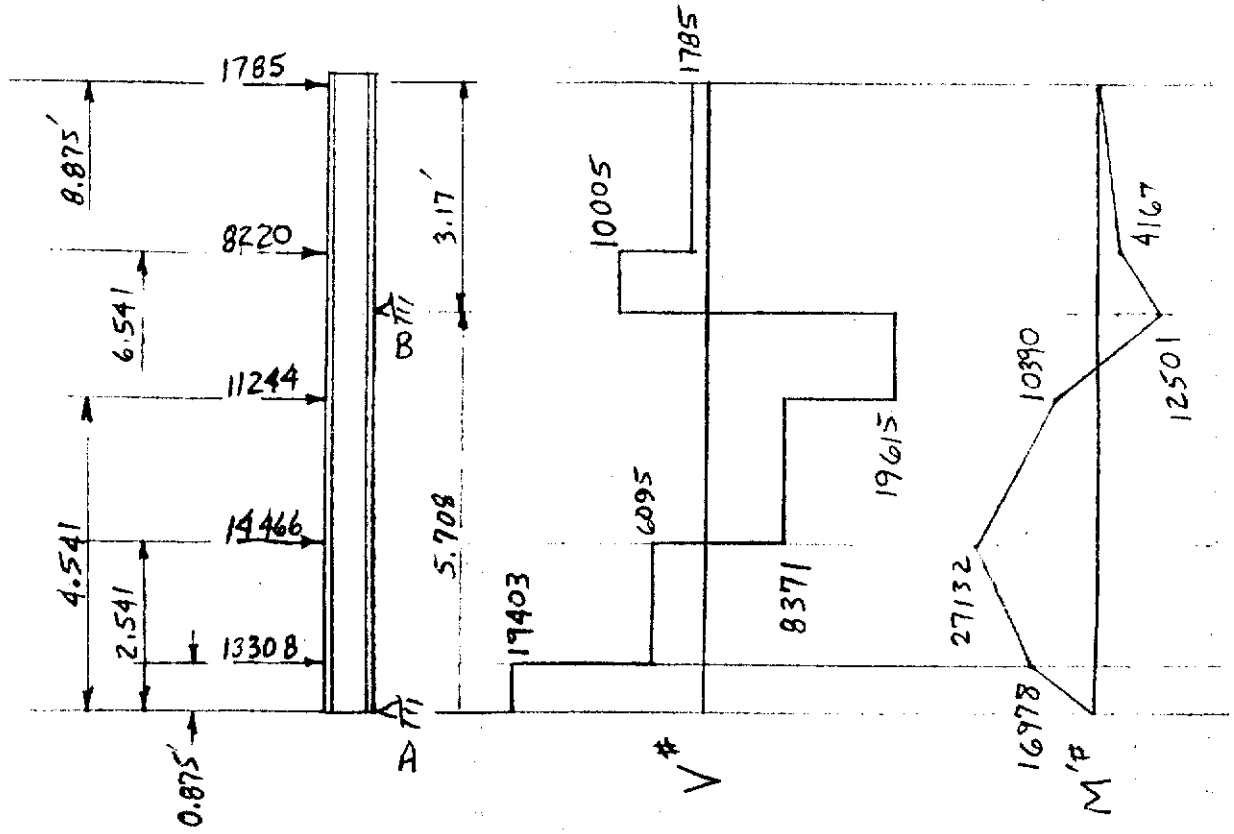
Vertical Support Beams for Lower Panels with 10 ft. Water Loading:



$R_A = \frac{1}{5.708}[(13308 \times 4.833) + (14466 \times 3.167) + (11244 \times 1.167) - (8220 \times 0.833) - (1785 \times 3.167)]$

$= 19,403$

$R_B = \frac{1}{5.708}[(13308 \times 0.875) + (14466 \times 2.541) + (11244 \times 4.541) + (8220 \times 6.541) + (1785 \times 8.875)]$

$R_B = 29,620$

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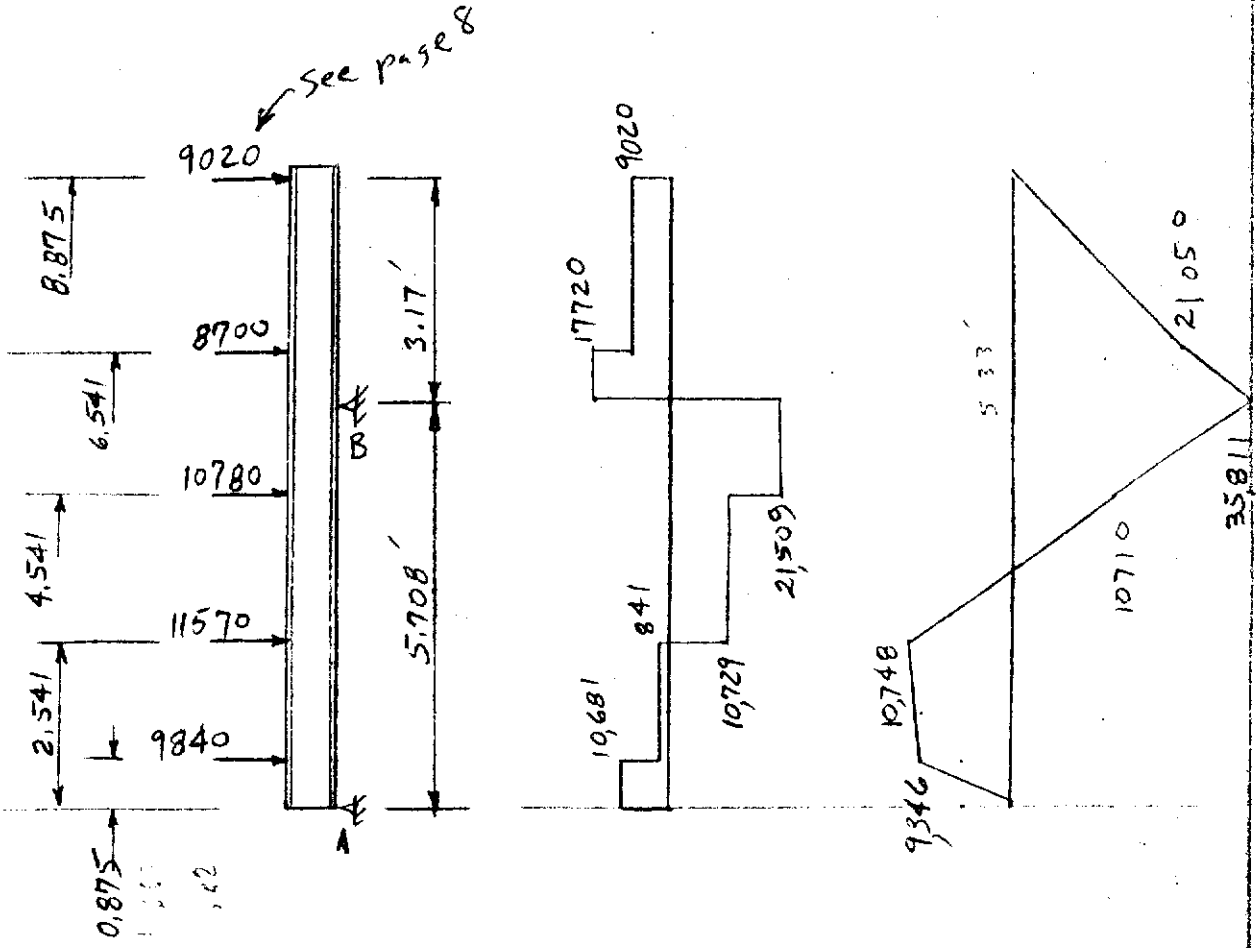
Vertical Beams for Only Lower Panels with Water to Design Grade:



$$R_A = \frac{1}{5.708}\left[(9840\times 4.833)+(11570\times 3.167)+(10780\times 1.167)-(8700\times 0.833)-(9020\times 3.167)\right]$$

$$= 19681$$

$$R_B = \frac{1}{5.708}\left[(9840\times 0.875)+(11570\times 2.541)+(10780\times 4.541)+(8700\times 6.541)+(9020\times 8.875)\right]$$

$$= 39,229$$

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Vertical Beams for Only Lower Panels with Water to Design Grade:

Upper 5½' of beam has compression flange w/o support

for W12×7×11.67Z $\quad l/r = \frac{5.5\times12}{1.65} = 40 > 21 < 66$

$$F_{allow} = \frac{1}{F_y}\left(B_c - \frac{D_c}{1.2}(l/r)\right) = \frac{1}{1.95}\left(39.4 - \frac{0.246}{1.2}(40)\right) = 16{,}000$$

$$f_{applied} = \frac{35811\times12}{42.60} = 10{,}090 < 16{,}000 \text{ OK}$$

$$\frac{f_t}{F_{at}} + \frac{f_b}{F_{ab}} = \frac{39229/9.925}{16500} + \frac{10090}{16500} = 0.24 + 0.61 = 0.85 < 1.0 \text{ OK}$$

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Plate C-5m

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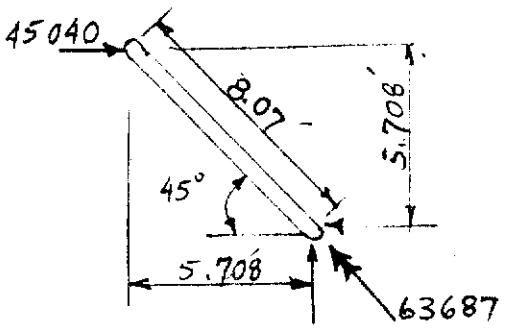
Lower Strut for Panels under 16'-5" Water Load:
(Aluminum)



$B_c = 39.4$
$D_c = 0.246$
$C_c = 66$

for 5" O D Tube, 4" I.D. (Alum.) wt = 8.31 plf

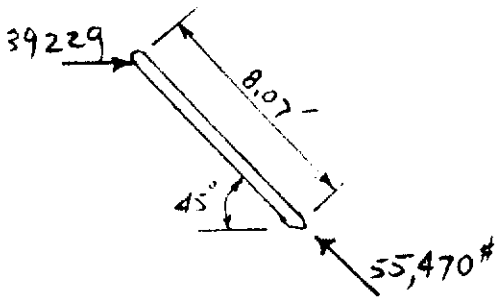
$$l/r = \frac{8.07 \times 12}{1.6008} = 60.5 < 66$$

$$F_{allow} = \frac{1}{n_u}\left[B_c - D_c\, l/r\right] = \frac{1}{2.30}\left[39.4 - 0.246(60.5)\right] = 10{,}660 \text{ psi}$$

$$f_{applied} = \frac{63687}{7.0686} = 9009 < 10660 \text{ OK}$$

Check local buckling in final design

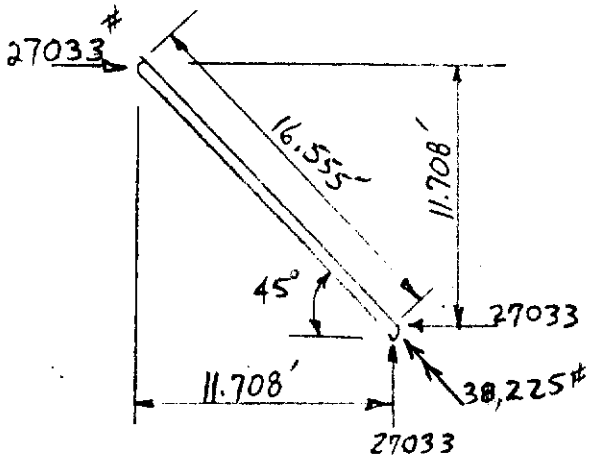
Use same strut for Lower Panel Short Vertical Beams where Horiz. load is 39,229#



$$f = \frac{55470}{7.0686} = 7{,}850 < 10660 \text{ OK}$$

Subject	Davenport Main St. Closure	Date
Computed by	Checked by	Sheet of

Upper Strut for Panels under 16'-5" Water Load



$B_c = 39.4$, $D_c = 0.246$, $C_c = 66$

for 6" Schedule 80 Pipe (6.625 OD) wt = 9.884 plf

$$L/r = \frac{16.56 \times 12}{2.195} = 90.5 > 66$$

$$F_{allow} = \frac{\pi^2 E}{n_u (L/r)^2} = \frac{\pi^2 (10{,}100)}{2.30 (90.5)^2} = 5291 \text{ psi}$$

$$f_{applied} = \frac{38225}{8.405} = 4{,}550 < 5{,}291 \quad \underline{OK}$$

Local Buckling:

$B_t = 43.2$ $D_t = 1.558$ $C_t = 141$

$R = 3.216"$ $R/t = 3.216/0.432 = 7.44$

$$S_1 = \left(\frac{B_t - \frac{n_u F_{cy}}{K_c n_y}}{D_t}\right)^2 = \left(\frac{43.2 - \frac{2.3(35)}{1.12(1.95)}}{1.558}\right) = 4.07 < 7.44 < 141$$

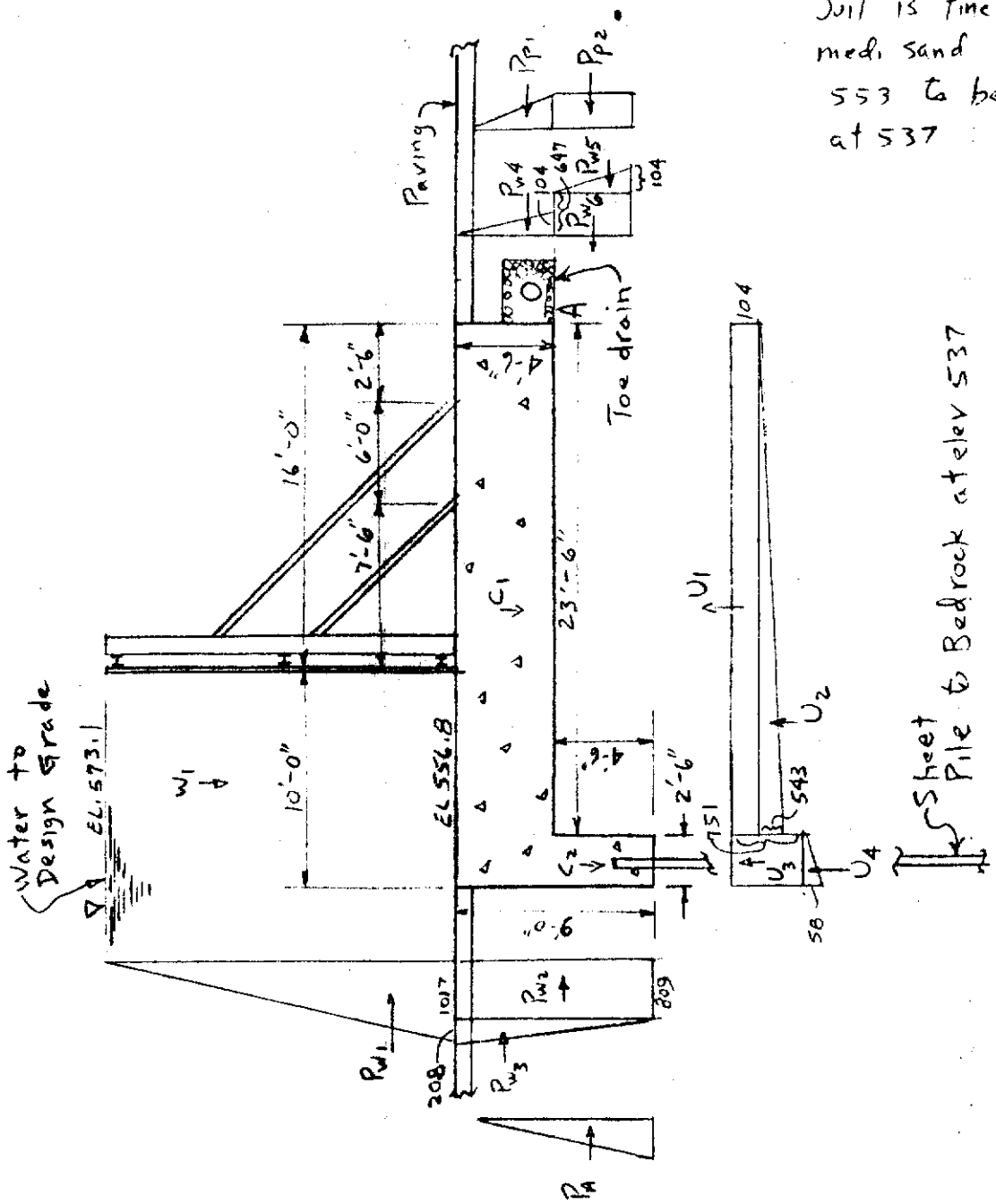
$$F_{allow} = \frac{1}{n_y}\left(B_t - D_t\sqrt{\frac{R}{t}}\right) = \frac{1}{2.3}\left(43.2 - 1.558\sqrt{7.44}\right) = 16{,}935 > 4550$$

OK

Subject	Davenport Main St. Closure	Date
Computed by DAP	Checked by	Sheet of

Stability

Ref. Boring D-27
Soil is fine to medi sand from 553 to bedrock at 537



Subject	Davenport Main St. Closure	Date
Computed by DAP	Checked by	Sheet of

Stability

Item	Factors			Arm	Mom. abt. A	
P_{w1}	$\frac{1}{2}(10)(17)(16.3)$	8,288	→	9.88	81,890	↷
P_{w2}	$(809)(9)$	7,281	→	0.0	0	
P_{w3}	$\frac{1}{2}(208)(9)$	936	→	1.5	1,404	↷
P_{w4}	$\frac{1}{2}(104)(4.5)$	234	←	1.5	351	↶
P_{w5}	$\frac{1}{2}(104)(4.5)$	234	←	3.0	702	↷
P_{w6}	$(647)(4.5)$	2,911	←	2.25	6,550	↷
W_1	$(10)(62.4)(16.3)$	10,171	↓	18.5	188,167	↶
C_1	$(4.5)(26)(150)$	17,550	↓	13.0	228,150	↶
C_2	$(2.5)(4.5)(150)$	1,687	↓	24.75	41,765	↶
U_1	$(104)(23.5)$	2,444	↑	11.75	28,717	↷
U_2	$\frac{1}{2}(543)(23.5)$	6,380	↑	15.75	100,488	↷
U_3	$(751)(2.5)$	1,877	↑	24.75	46,468	↷
U_4	$\frac{1}{2}(58)(2.5)$	72	↑	25.17	1,824	↷
P_A	$\frac{1}{2}(62.5)(9)^2(0.33)$	835	→	1.5	1,252	↶
P_{p1}	$\frac{1}{2}(4.5)^2(62.5)(5)$	3,164	←	3.0	9,492	↶
P_{p2}	$(6.75)(62.5)(5)(4.5)$	9,492	←	2.25	21,357	↷

$\Sigma H = \overleftarrow{16035} - \overrightarrow{17340} = 1305 \rightarrow$ $\quad \Sigma M_1 = 469177$ ↶

$\Sigma V = 18635 \downarrow$ $\quad \Sigma M_2 = 289,400$ ↷

Overturning $F/S = \frac{469177}{289400} = 1.62 > 1.5$ OK

$\frac{\Sigma M}{\Sigma V} = \frac{469177 - 289400}{18635} = 9.65$ in middle ½ OK

Sliding $F/S = \frac{18635 \times 0.4}{1305} = 5.7 > 1.5$ OK

Soil Pressures

$Max = 18635/26 + \frac{18635(13 - 9.65)(13)}{1(26)^3/12} = 1270$ psf

$Min = 717 - 554 = 163$ psf

Subject	Davenport Main St. Closure	Date
Computed by DAP	Checked by	Sheet of

Before final design another boring should be made at the closure, near ℄, with blow count taken below elev 552.
The sand shown in D-27 appears loose, $\phi \sim 29-30°$
N about 7-8

Bearing capacity:

$$q_{ult} = \gamma D N_q + \tfrac{1}{2}\gamma B N_\gamma$$
$$= 60(4.5)(19) + \tfrac{1}{2}(60)(26)(18)$$
$$= 5130 + 14040 = 9.5 \text{ TSF}$$

w/ FS = 3, $q_{allow} = 3.1 > 1270$ OK

To limit settlement to 1":
With high water level, using emperical charts, allowable toe press. above surrounding surcharge; consider footing effective width $B = \frac{b}{3} = 26/3 = 9'$

∴ allow $p = 750$ psf

applied $p = 1270 - 4.5(120 - 62.5) = 1010$ psf

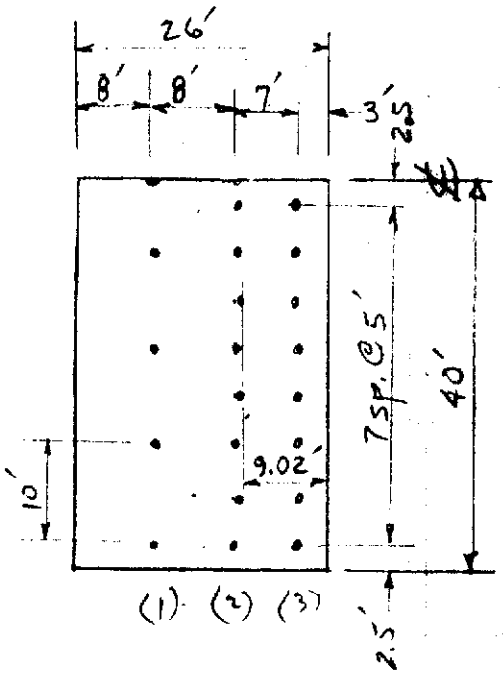
settlement could be > 1"; therefore use piles to bedrock.

Evaluate need for piles further in final design

Plate C-5r

Subject	Davenport, Main St. Closure	Date
Computed by DAP	Checked by	Sheet of

Piling Layout & Loads



CG Piles:

$\left.\begin{matrix} 8 \times 3 \\ 8 \times 10 \\ 4.5 \times 18 \end{matrix}\right\} \div 20.5 = 9.02'$

I of Piles:

$I = 8 \times (6.02)^2 + 8(0.98)^2 + 4.5(8.98)^2$

$= 660$

$$\text{Pile Loads} = \frac{P}{A} \pm \frac{Pey}{I}$$

$$P_{(1)} = \frac{18635 \times 40}{20.5} + \frac{18635(40)(9.65-9.02)(8.98)}{660}$$

$$= 36360 + 6390 = 42750\# \sim 22^T$$

$$P_2 = 36360 + \frac{18635(40)(9.65-9.02)(0.98)}{660}$$

$$= 36360 + 700 = 37060 \sim 19^T$$

$$P_3 = 36360 - \frac{18635(40)(9.65-9.02)(6.02)}{660}$$

$$= 36360 - 4280 = 32080 \sim 16^T$$

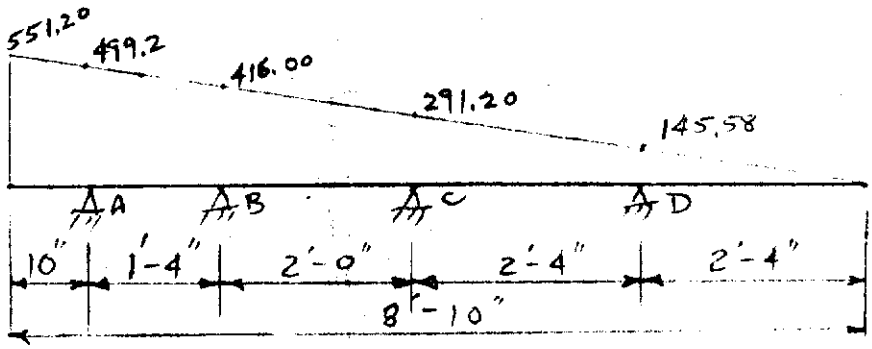
Use batter of 1.5 on 12 for last 2 rows

$8(37000 + 32000)\left(\frac{1.5}{12}\right) = 69{,}000\# > 1305 \times 40 = 53000\#$

Subject	Davenport Water Treatment Plant	Date
Computed by	Checked by	Sheet of

Upstream RR Closure
Aluminum

Loading: Water to Design Grade



Ref. Computer Strudl program ran DAV4

$R_A = 832.3$
$R_B = 574.7$
$R_C = 621.3$
$R_D = 406.0$

Loading #2, wind, not significant except that it can be opposite in direction to water, thus positive connections needed

Subject	Davenport Water Plant	Date
Computed by DAP	Checked by	Sheet of

Upstream RR Closure
Aluminum

Horiz. Beams

$M_{max} = \frac{832.3}{100}(64.58) = 537.5'\#$

$S_{reqd} = \frac{537.5 \times 12}{16500} = 0.391\ in^3$

Use C 3 × 1.75 × 1.597 plf, $S = 1.31\ in^3 > 0.39$ ok

Secure comp flange to decking and to vertical beams @ 5' c-c

$f = \frac{537.5 \times 12}{1.31} = 4923\ psi$

$L/r = \frac{60}{0.55} = 109 > 1.2 C_c = 1.2(66) = 79$

$F_{b\ allow} = \frac{\pi^2 E}{n_y (L_B/1.2 r_y)^2} = \frac{\pi^2 10100}{1.95(109/1.2)^2} = 6185 > 4923$ ok

Subject	Davenport Water Treatment Plant	Date
Computed by DAP	Checked by	Sheet of

Upstream RR Closure
Aluminum

Decking:

$M_{max} = 185.2'\#$

$S_{req'd} = \frac{185.2 \times 12}{15000} = 0.148\ in^3$

Use V-Beam Roof Decking
0.050" thickness, $S = 0.354\ in^3$ wt = 0.91 psf
(45" wide pieces)

Check web crippling

$$C_p = \frac{2.5E\left(\frac{N}{w}+1\right)}{F_{cy}\left(6+0.04\frac{N}{t}\right)\left[1.1-0.1\left(\frac{\theta}{90}\right)\sqrt{\frac{r}{t}}\right]}$$

$$= \frac{2.5(10100)\left(\frac{1.75}{2.33}+1\right)}{28\left(6+0.04\times\frac{1.75}{0.050}\right)\left[1.1-0.1\left(\frac{48.6}{90}\right)\sqrt{\frac{0.125}{0.050}}\right]} \cdot \frac{44214}{210} = 210$$

$\frac{w}{t} = \frac{2.33}{0.05} = 47 < 210$

$$\therefore P_{c\,allow} = 0.6\,\frac{F_{cy}\,d\,t^2}{w}\left(6+0.04\frac{N}{t}\right)\left[1.1-0.1\left(\frac{\theta}{90}\right)\sqrt{\frac{r_i}{t}}\right]$$

$$= 0.6\,\frac{28(1.75)(0.05)^2}{2.33}\left(6+0.04\frac{1.75}{0.05}\right)\left[1.1-0.1\left(\frac{48.6}{90}\right)\sqrt{\frac{0.125}{0.05}}\right]$$

$$= .236$$

with F/S = 1.95 and on a per ft basis:

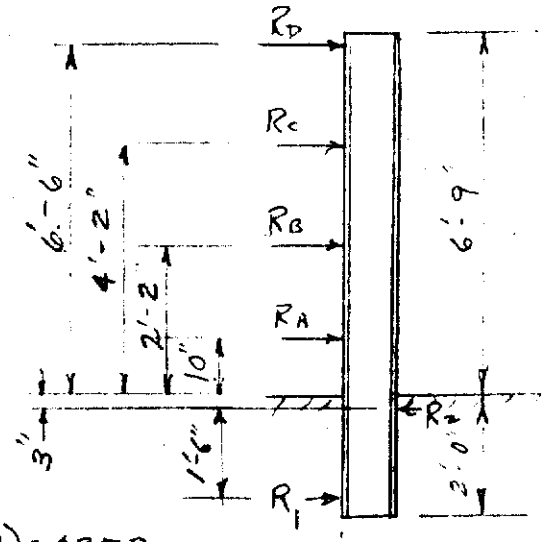
$P_{c\,allow} = 236 \times \frac{1.65}{1.95} \times \frac{12\times 2}{5.33} = 902\#/' > 832.3$ ok

Plate C-6c

Subject	Davenport Water Treatment Plant	Date Jun 78
Computed by DAP	Checked by	Sheet of

Upstream RR Closure
in Aluminum

Vertical Beams:



$R_A = \frac{833}{100}(282.14) = 2350$
$R_B = 575/100$ " = 1622
$R_C = 622/100$ " 1755
$R_D = 406/100$ " 1145

$V_{max} = 2350 + 1622 + 1755 + 1145 = 6872\#$
$M_{max} = 2350(1.083) + 1622(2.41) + 1755(4.41) + 1145(6.83) = 22020'\#$

$B_c = 39.4$, $D_c = 0.246$, $C_c = 66$
For W 8 × 6.5 × 8.32

$L/r = \frac{6.75 \times 12}{1.61} = 50$

$S_2 = 79 > 50$

$F_{b\,allow} = \frac{1}{1.95}\left(B_c - \frac{D_c L_B}{1.2 r_y}\right) = \frac{1}{1.95}\left(39.4 - \frac{0.246}{1.2}(50)\right) = 14{,}900$ psi

$f = \frac{22020 \times 12}{21.04} = 12{,}560 < 14{,}900$ OK

$R_1 = \frac{22020}{1.5} = 14{,}680\#$

$R_2 = 6872 + 14680 = 21{,}552\#$

Check stiffners, Local buckling in final design

Plate C-6d

GIT-ICES: V2M3 *** GEORGIA INSTITUTE OF TECHNOLOGY - I RATED CIVIL ENGINEERING SYSTEM *** 07/07/78 PAGE 0002 *

```
*** PARM= MOD=2,NPCI
INDENT 5
**** SYSERR 1.06 - 'INDENT  ' NOT A VALID COMMAND.
STRUDL 'DAV4' 'WATER PLANT UPSTREAM RR CLOSURE'
*** SYSMSG 1.00 - POOL INCR =  16384


   ***************************************************************
   *                                                             *
   *    ******                    GTICES STRUDL-II               *
   *   ********                THE STRUCTURAL DESIGN LANGUAGE     *
   *   **    **                                                  *
   *   **                  CIVIL ENGINEERING SYSTEMS LABORATORY  *
   *   **  **********          SCHOOL OF  CIVIL ENGINEERING       *
   *   **  **********        GEORGIA INSTITUTE  OF TECHNOLOGY     *
   *   **    ****                ATLANTA, GEORGIA 30332           *
   *   **********                                                *
   *    ****** ** ICES              MARCH, 1978 - V1M3            *
   *           **                                                *
   *           ** S T R U D L                                    *
   *           **                                                *
   *           **            COPYRIGHT (C) 1978, GEORGIA TECH    *
   *           **                  ALL RIGHTS RESERVED           *
   *                                                             *
   ***************************************************************


**** INPUT MODE ASSUMES 72 COLUMN SCANS WITH SEQUENCE NUMBERS
**** IN COLUMNS 73-80; USE "COL80" COMMAND FOR 80 COLUMN SCANS

**** INPUT "*NEWS 4" FOR LATEST FIXES/ENHANCEMENTS ON
**** GT/STRUDL; INPUT "*NEWS ALL" FOR COMPLETE NEWS ITEMS

**** ACTIVE UNITS -  LENGTH   WEIGHT   ANGLE    TEMPERATURE   TIME
**** ASSUMED TO BE     INCH    POUND   RADIAN    FAHRENHEIT   SECOND

 $ DAV4  WATER PLANT UPSTREAM RR CLOSURE
TYPE PLANE FRAME
UNITS FEET LBS
JOINT COORD
1 0.0 0.0
2 0.833 0.0 S
3 2.167 0.0 S
4 4.167 0.0 S
5 6.5 0.0 S
6 8.833 0.0
MEMBER INCIDENCES
1 1 2
2  2 3
3 3 4
4 4 5
5 5 6
JOINT RELEASES MOMENT Z
```

Plate C.6e

THIS FORM PREPARED BY BOEING COMPUTER SERVICES COMPANY

```
2 THROUGH 5
MEMBER PROPERTIES PRISMATIC
1 TO 5  AX 1  IZ 1
LOADING 1 'WAT-ER'
MEMBER LOADS
1 F Y LIN WA 551.20 WB 499.2
2 F Y LIN WA 499.20 WB 416.0
3 F Y LIN WA 416.00 WB 291.2
4 F Y LIN WA 291.20 WB 145.5
5 F Y LIN WA 145.5  WB 0.0
LOADING 2 'WIND'
MEMBER LOADS
1 THROUGH 5 F Y UNI 25
STIFFNESS ANALYSIS
OUTPUT DECIMAL 3
LIST REACTIONS
```

THIS FORM PREPARED BY BOEING COMPUTER SERVICES COMPANY

57.

Plate C-6f

****
RESULTS OF LATEST ANALYSES
****

PROBLEM - DAV4 TITLE - WATER PLANT UPSTREAM RR CLOSURE

ACTIVE UNITS FEET LB RAD DEGF SEC

ACTIVE STRUCTURE TYPE PLANE FRAME

ACTIVE COORDINATE AXES X Y

LOADING - 1 WATER

RESULTANT JOINT LOADS - SUPPORTS

JOINT		FORCE			MOMENT		
		X FORCE	Y FORCE	Z FORCE	X MOMENT	Y MOMENT	Z MOMENT
2	GLOBAL	0.0	-832.274				0.000
3	GLOBAL	0.0	-574.666				-0.000
4	GLOBAL	0.0	-621.305				-0.000
5	GLOBAL	0.0	-406.019				0.000

LOADING - 2 WIND

RESULTANT JOINT LOADS - SUPPORTS

JOINT		FORCE			MOMENT		
		X FORCE	Y FORCE	Z FORCE	X MOMENT	Y MOMENT	Z MOMENT
2	GLOBAL	0.0	-36.750				0.000
3	GLOBAL	0.0	-50.073				-0.000
4	GLOBAL	0.0	-14.943				-0.000
5	GLOBAL	0.0	-119.059				0.000

THIS FORM PREPARED BY BOEING COMPUTER SERVICES COMPANY

Plate C-69

Davenport 11-30-76

Credit Island

SLOUGH CLOSURE PUMP STATION

SRW

STABILITY

Water level is at Design Levee Grade EL 571.0

Uplift

$U_1 = (43 \times 44)(1280) = 2{,}422{,}000^{\#}$ (1892) ↑

Arm = 43/2 = 21.50'

$U_2 = 1/2(43 \times 44)(1187) = 1{,}120{,}000^{\#}$ (446) ↑

Arm = 43 × 2/3 = 28.67'

Σ Forces & Σ Moments

Description	Kips ΣH	Kips ΣV	Ft. Arm	Kip Ft. Moments
W Pump Station		7788 ~~5310~~ ↓	25.0 ~~20.39~~	194 840 ~~108 080~~ ↻
Water W_1		234 ↓	41.75	9 770 ↻
Water W_2		103 ↓	1.25	129 ↻
Uplift U_1		2422 ↑	21.50	52 000 ↺
Uplift U_2		1120 ↑	28.67'	32 150 ↺
Wind (Bldg.)	12 ←		54.63	656 ↺
Wind (Concrete Structure)	6.27 ←		41.88	263 ↺
Water Pressure Force – P_1	2145 ←		13.17	28 250 ↺
" " " P_2	577 →		6.83	3 940 ↻
Total	2163.27 ←	8125 ~~5647~~ ↓		208 679 ~~121 919~~ (Resisting Moment) ↻
	1586.27 ←	3542 ↑		113 319 (Overturning Moment) ↺
		4583 ~~2105~~ ↓		~~8 600~~ ↻ 95 360

Arm = ΣM ÷ ΣV = ~~8600~~ 95360 ÷ ~~2105~~ 4583 = ~~4.08'~~ 20.4'

Eccentricity = e = 43/2 − ~~4.08~~ 20.4 = ~~17.42'~~ 1.10'

Arm − 1/4 Base = 20.4 − 10.75 = 9.65'

1/4 Base − Arm = ~~10.75~~ − 4.08 = 6.67'

One fourth of base = 43/4 = 10.75'

Vertical resultant is located outside the middle half of base. within

Sliding

Coefficient of friction = μ = 0.

Factor of Safety = μ(ΣV)/ΣH = .4 × ~~2105~~ 4583 / 1586 = 1.16 Low; Secure Pump Sta. to Cells and U-frame Outlet for increased F/S; See comps. following.

Overturning

Factor of Safety = Resisting Moment / Overturning Moment

= ΣMR / ΣMo = ~~121919~~ 208679 / 113319 = ~~1.07~~ 1.84 < 1.50

Soil Pressure Underneath the Base Slab

$q = P/A \pm Pec / I_{xx}$

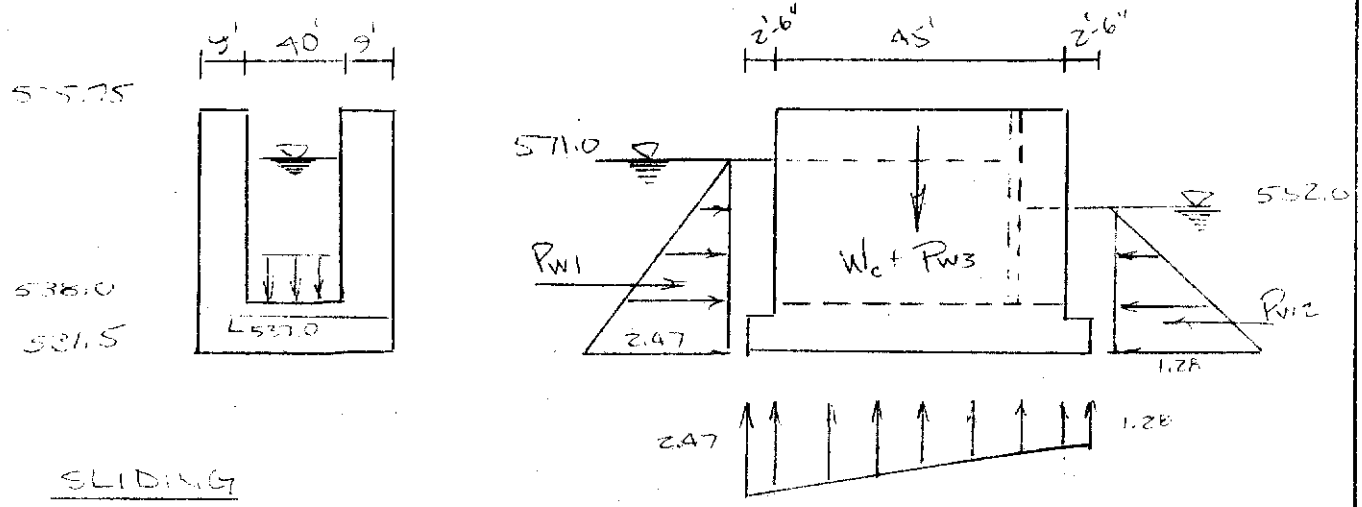
$q_{max} = 4583/1892 + 4583 \times 1.10 \times 21.5 / 291500$ (108200)

= 2.42 + 0.37 = 2.79 KSF

$q_{min} = 2.42 - .37 = 2.05$ KSF

Plate C-7a

Subject	SLOUGH CLOSURE	Date
CREDIT ISLAND (OUTLET STRUCTURE) - STABILITY		8/79
Computed by DJW	Checked by	Sheet of



SLIDING

$W_c = 150[(45)(37.75)(9)(2) + (45)(1)(58) + (50)(5.5)(58)]$ ↓

W_c = 7370.163 k / 58' = 127.08 klf ↓

P_{W1} = 48.76 klf →

P_{W2} = 13.13 klf ←

P_{W3} = (62.5pcf)(45)(40)(33) = 3712.5k/58' = 64.0 klf ↓

U = 93.75 klf ↑

$F.S. = \sum F_R / \sum F_D = f(W_c + P_{W3} - U) + P_{W2} / P_{W1}$

$= 0.40(127.08 + 64.0 - 93.75) + 13.13 / 48.76$

F/S = 1.07 } with full uplift on base.
F/S = 1.17 } with no uplift.

Secure U-frame Outlet to Pump Sta. and Cells for greater F/S; see comps. following

OVERTURNING

M_O = 3233.67

$M_R = (127.08 + 64.0)(\frac{58}{2}) + (13.13)(\frac{20.5}{3})$

M_R = 4866.72

$F.S. = \frac{M_R}{M_O} = \frac{4866.72}{3233.67}$ — **F.S. = 1.51**

Subject	SLOUGH CLOSURE	Date
COFFERDAM – DAVENPORT GEN. DESIGN MEMOR.		7/79
Computed by DJW	Checked by	Sheet 1 of

DESIGN CONDITIONS :

1. LEVEE GRADE ELEV. 571.0'
   FLOOD ELEV. 568.0'
   BEDROCK ELEV. 531.0'

2. CELL FILL FREE DRAINING SAND & GRAVEL
   γ_{moist} = 110 pcf
   γ_{sub} = 65 pcf
   ϕ = 30° tan ϕ = 0.577

3. BEDROCK
   SHALE SOFT TO MEDIUM ;
   COMPARISON TO BEDROCK AT OLD
   LOCK NO. 14 SHOWS SIMILAR
   CONDITIONS ; USE ϕ = 22°

Evaluate FOUNDATION COMPLETELY BEFORE FINAL DESIGN

4. FRICTION COEFFICIENTS
   STEEL ON STEEL f = 0.3
   FOUNDATION ROCK f = tan ϕ = tan 22 = 0.40
   SAND ON STEEL f = tan δ = tan 22 = 0.40

5. FACTORS OF SAFETY
   1.50 FOR PERMANENT STRUCTURE
   1.25 FOR TEMPORARY STRUCTURE

6. SHEET PILING PS28
   GUARANTEED INTERLOCK TENSION 16000 pli
   ALLOWABLE INTERLOCK TENSION 8000 pli

NCR Form 381a
1 Dec 77

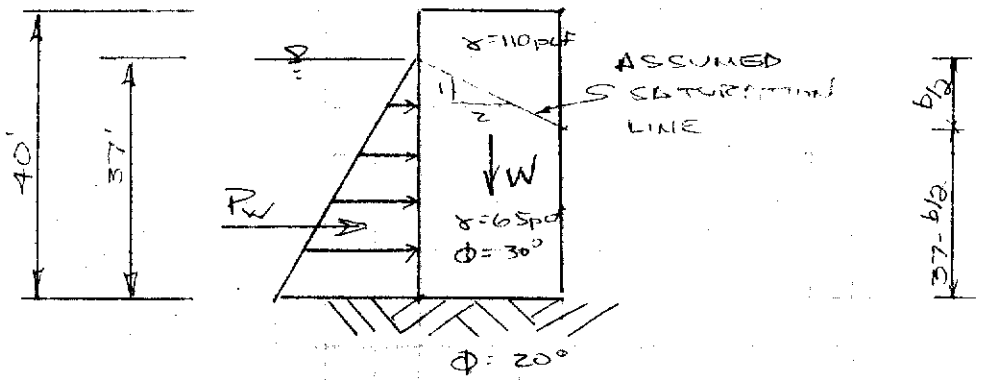
Plate C-7C

61

Subject	SLOUGH CLOSURE	Date
COFFERDAM - DAVENPORT		7/79
Computed by	Checked by	Sheet 2 of
DJW		

Temporary Cells

SLIDING STABILITY (Sliding of Sheeting on Cell fill, Shear failure on ℄, interlock tension - all computed as less critical)



RESISTING FORCE = WT OF CELL SOIL

$W = (3)(b)(110) + (1/2)(b)(b/2)(110) + (1/2)(37+37-b/2)(b)(65)$

$W = 330b + 27.5b^2 + 2405b - 16.25b^2$

$W = 11.25b^2 + 2735b$ #

COEFFICIENT OF FRICTION = $f = \tan\Phi_{RMK.}$

$f = \tan 20° = 0.364$

DRIVING FORCE - DUE TO WATER PRESSURE

$P_W = (1/2)(37 \times 62.5)(37) = 42781$ #

FACTOR OF SAFETY = 1.50 FOR PERMANENT STRUCTURES
1.25 FOR TEMPORARY STRUC.

$$F.S. = \frac{W f}{P_W}$$

$$1.25 = \frac{(11.25b^2 + 2735b)(0.40)}{42781}$$

$11.25b^2 + 2735b - 146913 = 0$

$b = -2735 \pm \sqrt{2735^2 - 4(11.25)(-146913)}/2\times11.25$

b = 46	FS = 1.25
b = 53'	FS = 1.50

Subject		Date
Davenport, SLOUGH CLOSURE		
Computed by	Checked by	Sheet of

Sliding Stability of Cells, Pump Station and Gate U-frame Combined:

Driving force of water to Design grade

$\frac{1}{2}(40)^2(62.5)152 = 7,600^K \leftarrow$

Resisting force of water to Elev. 552.0 on slough side

$\frac{1}{2}(21)^2(62.5)152 = 2,094^K \rightarrow$

Resistance from friction of levee on side of cells:

$\frac{1}{2}(55)(40)^2(0.3)(0.4)(66)2 = 697^K \rightarrow$

Wt. of Cells	21,450^K	↓
Wt. of U-frame Outlet	7370	↓
Wt. water in U-frame	3,712	↓
Wt. Pump Sta. & Bay Water	8007	↓
Uplift on Pump Sta	3542	↑
Uplift on U-frame	5450	↑
	31547^K	↓

$$F/S = \frac{(31547 \times 0.4) + 2094 + 697}{7600} = 2.03 \quad \text{OK}$$

Subject		Date
Davenport Local Flood Protection, Phase II GDM -		9 Jan 79
Computed by	Checked by	Sheet of
Ted A. Stephens		

CREDIT ISLAND SLOUGH CLOSURE BULKHEAD DESIGN

- Maximum net head is 18.5'
- Assume (40'+1') = 41' simple span.

$$M_{max} = (18.5')(.0625 k/ft^3)(41')^2/8$$

$$= 243' K/Ft$$

$$S_{req'd} = (243' K)(12 in/Ft)/18 ksi$$
$$= 162 in^3/Ft.$$

Determine maximum spacing between skinplate supporting members.

- Neglect support of flanges of supporting beams.

$M = wl^2/12$, assume fixed supports

$M = fs$

$$S = bh^2/12 \,/\, h/2 = \frac{bh^2}{6} = \frac{h^2}{6},\ b = 1"$$

$$f = 18 ksi$$

$$\omega = (18.5')(.0625 k/ft^3) = 1.15 ksf = .008 ksi$$

$$\therefore (18 ksi)\left(\frac{h^2}{6}\right) = (.008 ksi)(l^2)/12$$

$$4500\ h^2 = l^2$$

for $h = .375"$, $l = 25"$

for $h = .5"$, $l = 33.5"$

Max. cantilever distance

$$M = \frac{wl^2}{2} = (18 ksi)\left(\frac{h^2}{6}\right),\ l^2 = 750 h^2$$

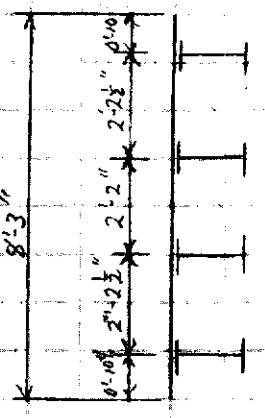
for $h = .375"$, $l = 10.25"$

for $h = .5"$, $l = 13.7"$

Plate C-7f

Subject	Date
Davenport Local Flood Protection - Phase II Design Memo	9 Jan 79
Computed by Ned A. Shepherd	**Checked by**
	Sheet of

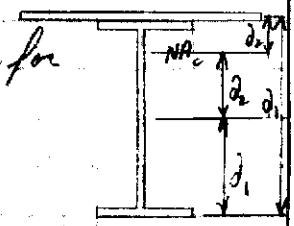
- Determine beam sizing.
- Use the following spacing.



- $S_{max}/beam = (162\ in^3/ft)(2'-2\frac{1}{2}'') = 357.25\ in^3/beam$

Try a W30X99, $S = 270\ in^3$ and assume composite action with skin plate.

- Check for unsupported width of skin plate for composite action.



Unsupported width = $10'' - \frac{10.458''}{2} = 4.771''$

$76/\sqrt{36} = 12.662'' > 4.771''$ ok, AISC 1.9.1.2, 7th Ed.

	d_1	A	Ad_1	d_2	Ad_2^2	I_o	I
WF 30x99	14.82	29.1	431.26	3.02	274.8	4.000	4275
20" x 3/8 skn plate	29.83	7.5	223.7	11.94	1069	—	1069
		36.6	654.9				5344

$NA = 654.9/36.6 = 17.89''$

$S_{min} = I/17.89'' = 298.7\ in^3 < 357\ in^3$, Not ok.

From S_x tables, AISC, 7th Ed., The next heavier section which would be strong enough is the W30X116 but for 2 plf more, use a W33X118, $S_x = 359 > 357.25$ req'd, o.k.

Subject: Davenport Local Flood Protection, Phase II GDM		Date: 9 Jan 79
Computed by: Neal A. Shepherd	Checked by:	Sheet of

Straight stiffeners wheels.

- Wheel loading.

  - Assume 3- double flanged wheels per side of each section. - spaced for equal loading.

    Load/wheel = (19' head)(.0625 K/ft³)(42")(8.25')/6
    = 68.6 K/wheel

  - Wheels will be at rest when underload.

    Allowable pressure / in of roller
    $= \frac{600d\,(F_y - 15{,}000)}{20{,}000}$, Assume F_y = 40 ksi
    " d = 24"

    = 18 K/in , AREA 15.6.4.5

    Req'd width = 68.6 K / 18 K/in = 3.8", use a 4" roller.

NCR Form 381a
1 Dec 77

Plate C-7h

66.

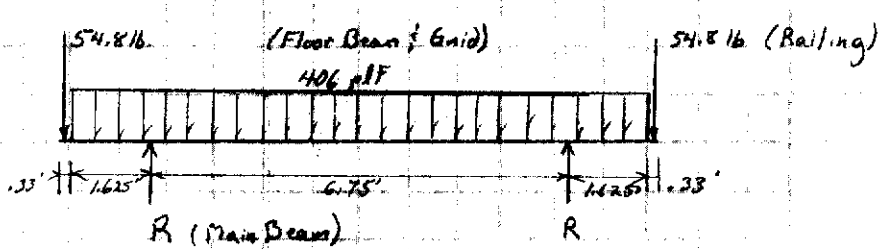
Subject	Date
Davenport Local Flood Protection	
Computed by / Checked by	Sheet of

Service Bridge - Grand Island Slough Outlet

Loadings
1) Flooring, Concrete Filled grid, 70 psf
2) Beams. As noted on dwg.
3) Live load. ASSHTO H-15
4) Impact is .3, ASSHTO 1.2.12c

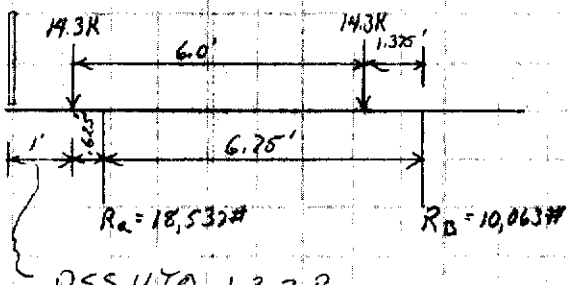
Floor beam loading

Dead



R = 2091.7 #

Live



ASSHTO 1.3.2 B

$V_{DL+LL+I}$ = 1320 - 14,300 = -15,620 #

$M_{Neg\ DL+LL+I}$ = -646.5'# - 8938'# = -9,585'#

$M_{Pos\ DL+LL+I}$ = 1666'# + 13837'# = 15,503'#

Subject	Date
Davenport Local Flood Protection	
Computed by / Checked by	Sheet of

Main Beam Loading

- Dead Load. $\frac{2092\#}{5.5'}$ = 380 plf

Distribution of loads (see previous page)

+ 94 plf main beams

474 plf

$V_{DL} = \frac{(.474)(43')}{2} = 10K$

$M_{DL} = \frac{(.474)(43^2)}{8} = 109.6\ K$

- Live Load $M_{LL} = 281.9\ 'K$, 1955 HTO, Appendix A
$V_{LL} = 29.9\ K$

Fraction of Axle load to each beam = $\frac{18537}{28600} = .648$
Refer to floor beam loading diagram, previous page.

- $V_{LL+I} = (.648)(1.3)(29.9K) = 25.2K$
$M_{LL+I} = (.648)(1.3)(281.9'K) = 237.5'K$

- Total Design Loads $V_{DL+LL+I} = 35.2K$
$M_{DL+LL+I} = 347.1'K$

Davenport

OF PAGES

SUBJECT
Blackhawk Creek Bicycle Bridge

SOURCE OF DATA

COMPUTED BY | CHECKED BY | APPROVED BY

Deck:

Dead load = 10 psf
Live load = 85 psf
95 psf (No impact)

$M = wL^2/8 = 95(4)^2/8 = 190'^{\#}$

for 2" nominal material $S = bh^2/6 = \frac{12}{2}(1.5)^2 = 4.5\ in^3$

$f = 190 \times 12 / 4.50 = 506\ psi$

Allowable bending stress for construction grade fir for < 1 day loading = $1.33 \times 1050 = 1400\ psi$
506 OK

Check other stresses and deflection in final design

Main Beams:

Load from Deck & live load = $\frac{10}{8}wL = \frac{10}{8}(95)(4) = 475$ plf
Beam dead load = 25 plf
500 plf

$M = 0.100\ wL^2 = 0.100(500)(19)^2 = 18{,}050'^{\#}$

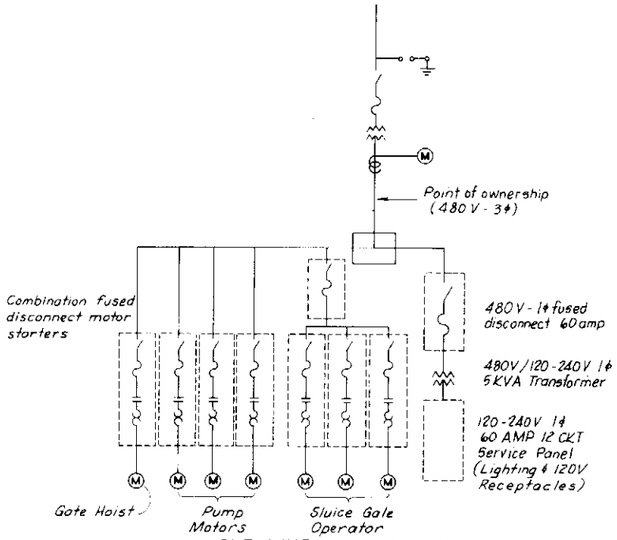
comp. flange braced by deck in pos. mom. region
comp. flange braced at max 6' in neg. mom. region

for W 8×15 $f = \frac{18050 \times 12}{11.8} = 18400 < 22\ ksi$ OK

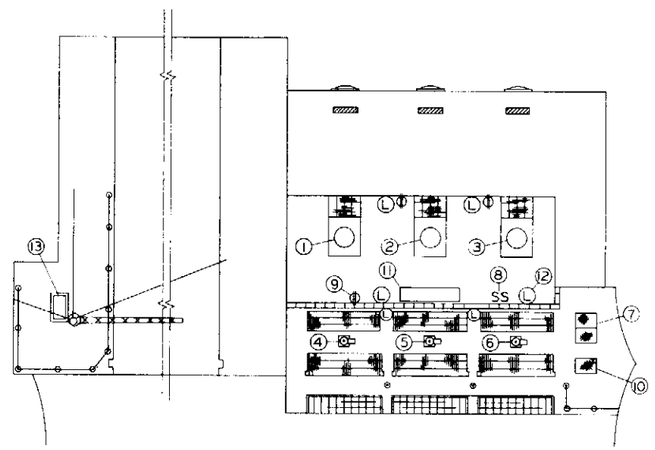
$L_u = 7.2' > 6.0$ OK

Plate C-9a

REPRODUCTION OF FORM 365B WHICH MAY BE USED



ONE LINE DIAGRAM OF PUMPING STATION POWER SERVICE

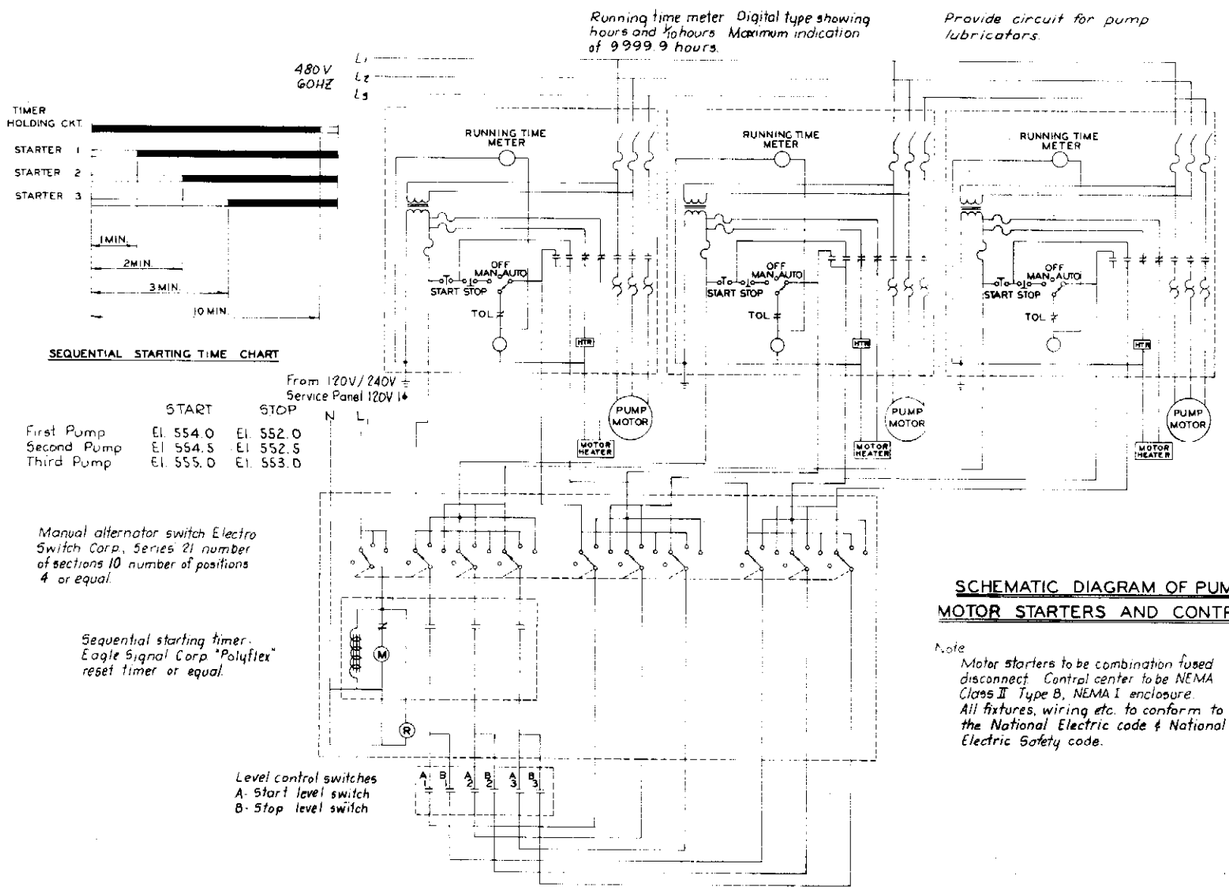


PLAN

ELECTRICAL EQUIPMENT

NO SCALE

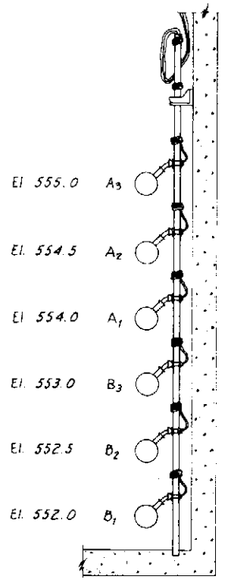
ITEM	DESCRIPTION	MANUFACTURERS DATA
1	Pump Motor No. 1	
2	Pump Motor No. 2	
3	Pump Motor No. 3	
4	Slide Gate Motor No. 1	
5	Slide Gate Motor No. 2	
6	Slide Gate Motor No. 3	
7	Sump Pump Motor	
8	Light Switches	
9	120V 15 AMP Receptacle	
10	Level Control	
11	Control Center	
12	Vapor Tight Luminaire (150 W)	
13	Hoist Motor	



SEQUENTIAL STARTING TIME CHART

SCHEMATIC DIAGRAM OF PUMP MOTOR STARTERS AND CONTROL

Note
Motor starters to be combination fused disconnect. Control center to be NEMA Class II Type B, NEMA I enclosure. All fixtures, wiring etc. to conform to the National Electric code & National Electric Safety code.



Direct acting float switch Consolidated Electric Co. Model 96 or equal

A- Start Level Switch
B- Stop Level Switch

FLOAT SWITCHES

REVISION	DATE	DESCRIPTION	BY

CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
ROCK ISLAND, ILLINOIS

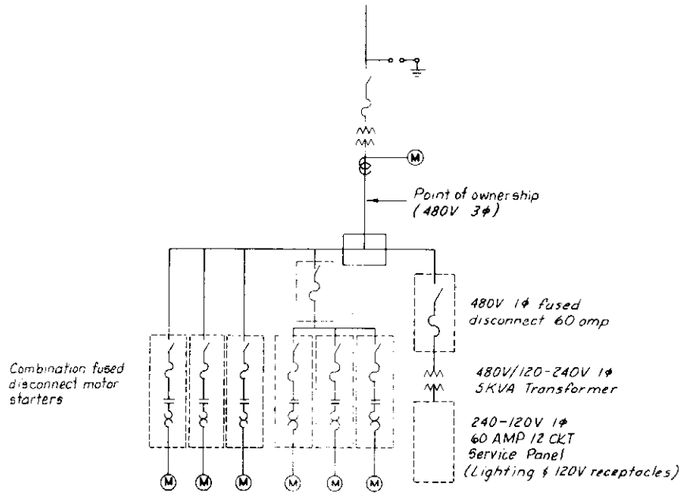
DRAWN BY:
TRACED BY:
CHECKED BY:
SUBMITTED:
CHIEF DESIGN BRANCH
APPROVED:
CHIEF, ENGINEERING DIVISION
DATE:

MISSISSIPPI RIVER
DAVENPORT, IOWA
LOCAL FLOOD PROTECTION

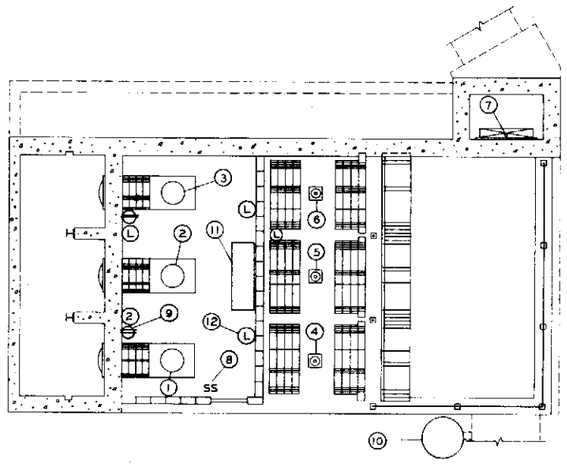
CREDIT ISLAND PUMPING STATION
ELECTRICAL

SHEET DRAWING NO
INVITATION NO DACW25 - - B -

PLATE D-10

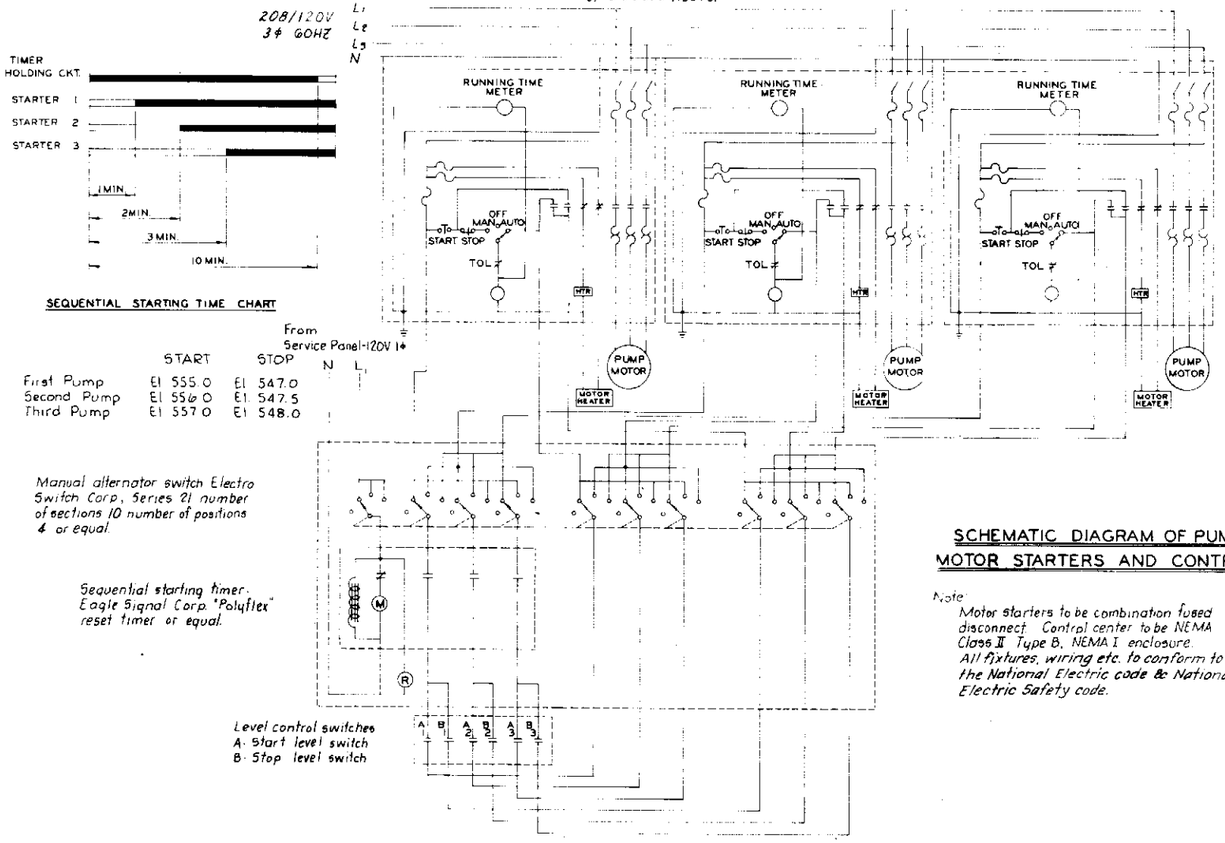


ONE LINE DIAGRAM OF PUMPING STATION POWER SERVICE



PLAN
ELECTRICAL EQUIPMENT
NO SCALE

ITEM	DESCRIPTION	MANUFACTURERS DATA
1.	Pump Motor No. 1	
2.	Pump Motor No. 2	
3.	Pump Motor No. 3	
4.	Slide Gate Motor No. 1	
5.	Slide Gate Motor No. 2	
6.	Slide Gate Motor No. 3	
7.	Slide Gate Motor No. 4	
8.	Light Switches	
9.	120 V 15 AMP Receptacle	
10.	Level Control	
11.	Control Center	
12.	Vapor Tight Luminaire (150 W)	



SEQUENTIAL STARTING TIME CHART

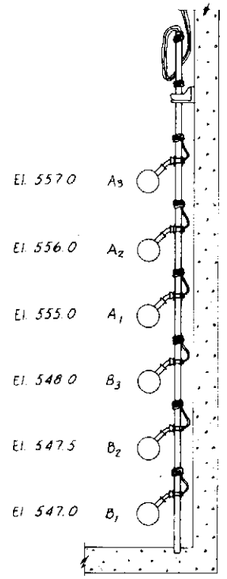
	START	STOP
First Pump	El 555.0	El 547.0
Second Pump	El 556.0	El 547.5
Third Pump	El 557.0	El 548.0

Manual alternator switch Electro Switch Corp., Series 21 number of sections 10 number of positions 4 or equal.

Sequential starting timer- Eagle Signal Corp. "Polyflex" reset timer or equal.

SCHEMATIC DIAGRAM OF PUMP MOTOR STARTERS AND CONTROL

Note
Motor starters to be combination fused disconnect. Control center to be NEMA Class II Type B, NEMA I enclosure. All fixtures, wiring etc. to conform to the National Electric code & National Electric Safety code.



A- Start Level Switch
B- Stop Level Switch

Direct acting float switch Consolidated Electric Co Model 9G or equal

FLOAT SWITCHES

REVISION	DATE	DESCRIPTION	BY

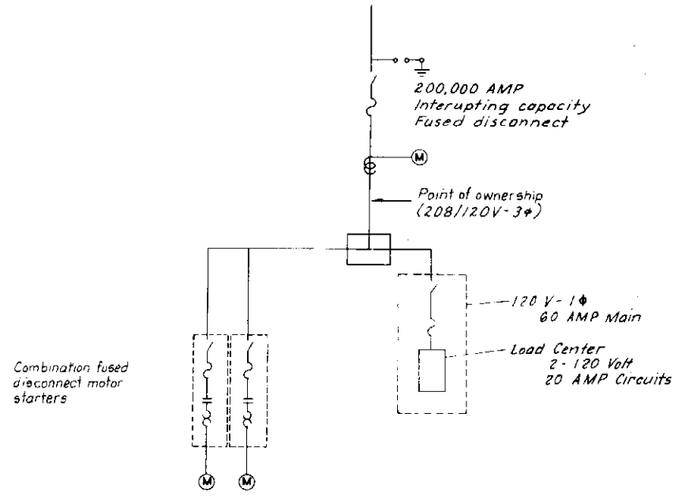
CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
ROCK ISLAND, ILLINOIS

MISSISSIPPI RIVER
DAVENPORT, IOWA
LOCAL FLOOD PROTECTION

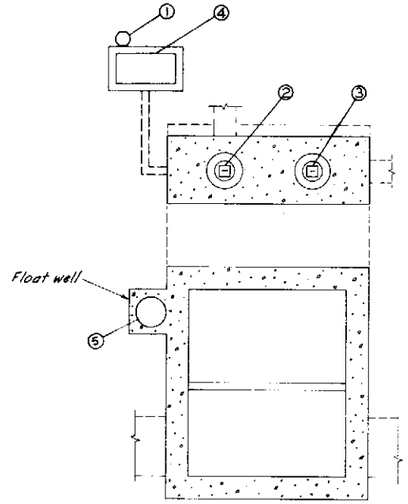
GAINES STREET PUMPING STATION
ELECTRICAL

SHEET DRAWING NO
INVITATION NO DACW25- -8-

PLATE D-11

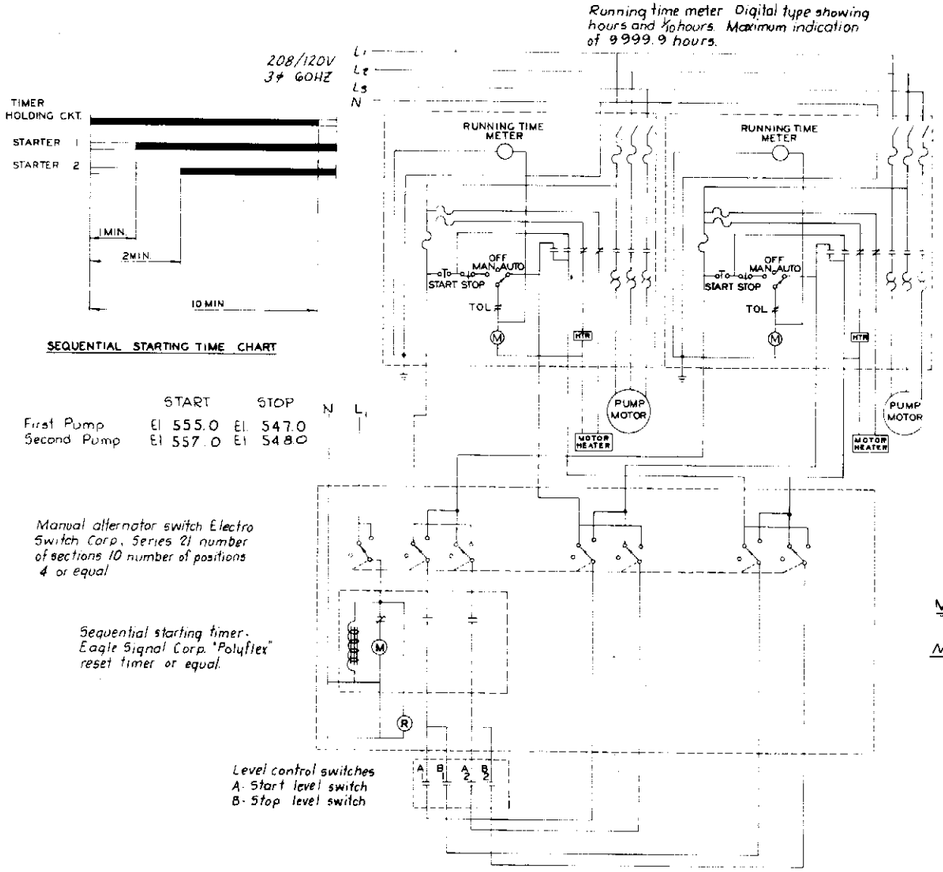


ONE LINE DIAGRAM OF PUMPING STATION POWER SERVICE



PLAN
ELECTRICAL EQUIPMENT
NO SCALE

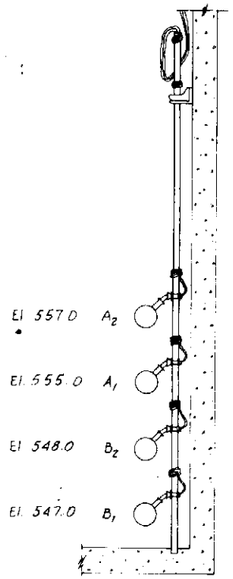
ITEM	DESCRIPTION	MANUFACTURERS DATA
1.	Service Enterance	
2.	Pump No. 1 motor	
3.	Pump No. 2 motor	
4.	Control Center	
5.	Level Control	



SEQUENTIAL STARTING TIME CHART

SCHEMATIC DIAGRAM OF PUMP MOTOR STARTERS AND CONTROL

NOTE:
Motor starters to be combination fused disconnect. Control center weather proof vandal resistant, NEMA IV enclosure. All fixtures, wiring etc. to conform to the National Electric Code & National Electric Safety Code.



Direct acting float switch
Consolidated Electric Co
Model 96 or equal

A- Start Level Switch
B- Stop Level Switch

FLOAT SWITCHES

REVISION	DATE	DESCRIPTION	BY

CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
ROCK ISLAND, ILLINOIS

DESIGNED HDG
DRAWN
TRACED BBC
SUBMITTED
CHIEF OF BRANCH
APPROVED
CHIEF ENGINEERING DIVISION
DATE

MISSISSIPPI RIVER
DAVENPORT, IOWA
LOCAL FLOOD PROTECTION

MAIN STREET PUMPING STATION
ELECTRICAL

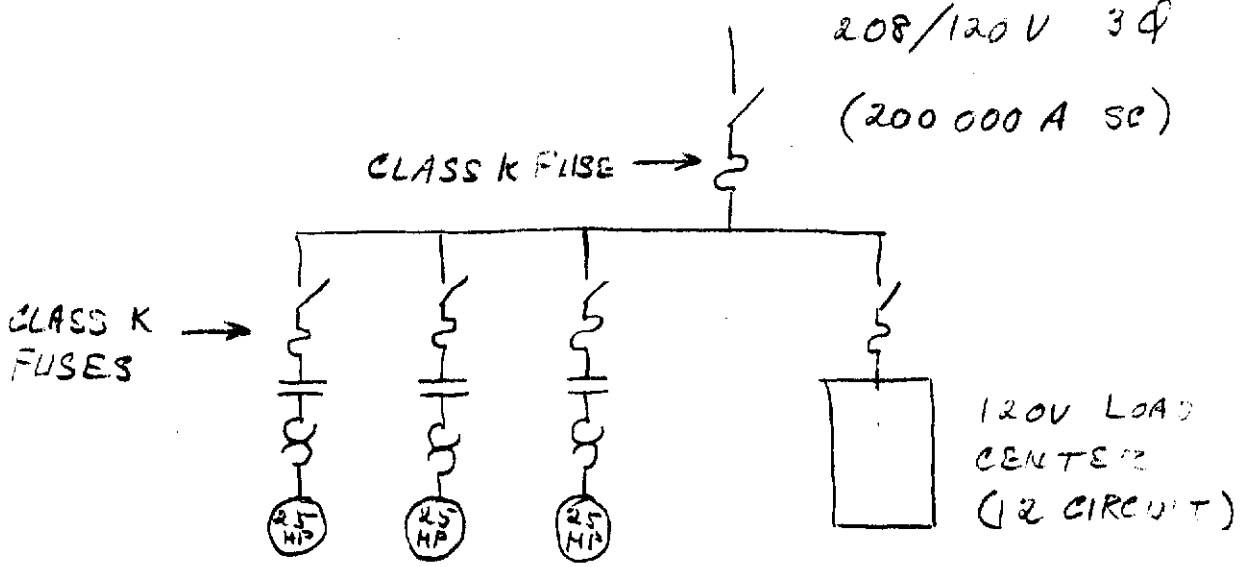
SHEET DRAWING NO
INVITATION NO DACW25 6-

DAVENPORT FLOOD PROTECTION 10-4-75

pc 1-12

MAIN STREET PUMPING STATION

SHORT CIRCUIT PROTECTION AND GENERAL DESIGN.



NOTE. IOWA ILLINIOS GAS AND ELECTRIC CO. HAS A 208/120V 3φ LOOP IN THIS AREA SERVING BUSINESSES IN THIS AREA. THIS CIRCUIT HAS A CALCULATED SHORT CIRCUIT RATING OF 200 000 AMPERES.

A FUSED MAIN DISCONNECT WILL BE USED, WITH CLASS K FUSES (200 000 A INTERRUPTING RATING).

MOTOR DISCONNECT AND 120V LOAD CENTER WILL ALSO BE CLASS K FUSES (200 000 A) WITH SELECTIVE COORDINATION.

ACROSS THE LINE MOTOR STARTERS WILL BE USED CONTROLLED BY FLOAT SWITCHES, WITH SEQUENTIAL STARTING, MANUAL ALTERNATING, AND ELAPSED TIME METERS. MOTOR + CABINET HEATERS WILL CONTROL MOISTURE

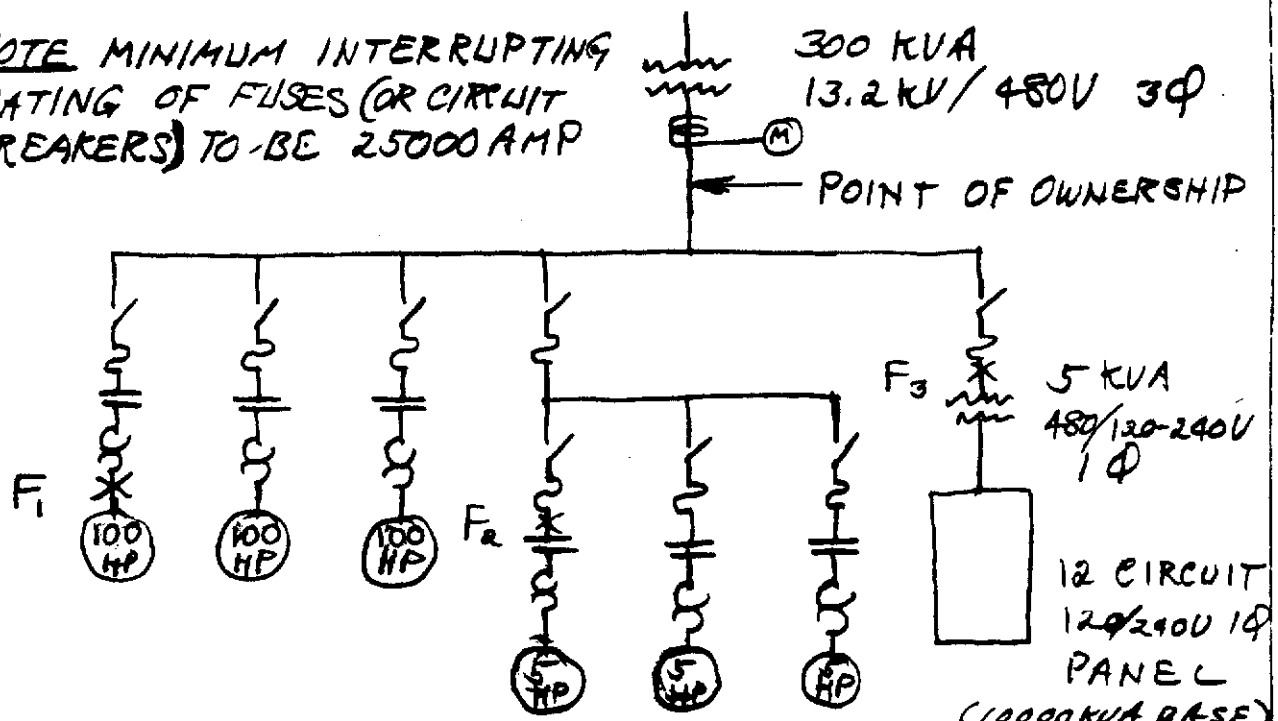
PLATE B-13(a)

Subject		Date
DAVENPORT FLOOD PROTECTION		10-5-78
Computed by	Checked by	Sheet of
		2 - 12

GAINES STREET PUMPING STATION

SHORT CIRCUIT PROTECTION

NOTE MINIMUM INTERRUPTING RATING OF FUSES (OR CIRCUIT BREAKERS) TO BE 25000 AMP

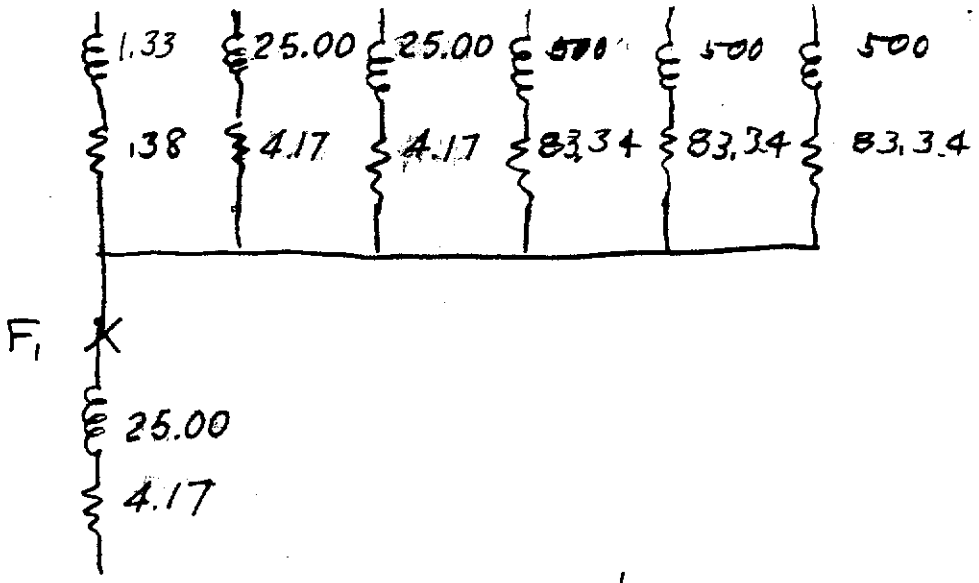


(10000 KVA BASE)

	PU R	PU X
SERVICE TRANSFORMER (300 KVA 4.16%Z) (R = 1.14% X = 4.0%)	.38	1.33
75 HP MOTOR	4.17	25.00
5 HP MOTOR	83.34	500.00
ASSUME INFINATE UTILITY SHORT CIRCUIT KVA	0	0
NEGLECT SERVICE CABLE, SWITCH, FUSE, ETC IMPEADANCE	0	0

Subject	Date
DAVENPORT FLOOD PROTECTION	10-5-78
Computed by	Checked by AC / Sheet 3 of 12

GAINES STREET PUMPING STATION
FAULT #1



$$PU\,Z = \frac{1}{\frac{1}{.38+j1.33} + \frac{1}{4.17+j25.00} + \frac{1}{4.17+j25.00} + \frac{1}{83.34+j500}}$$

$$\frac{}{+\frac{1}{83.34+j500} + \frac{1}{83.34+j500} + \frac{1}{4.17+j25.00}}$$

$$PU\,Z = .31 + j1.14$$

$$= 1.18 \angle 75.00^{\circ}$$

$$I_{SC} = \frac{10000}{\sqrt{3} \times .48 \times 1.18} = 10193.33 \text{ AMP}$$

~~$\frac{X}{R}$~~ = 3.68

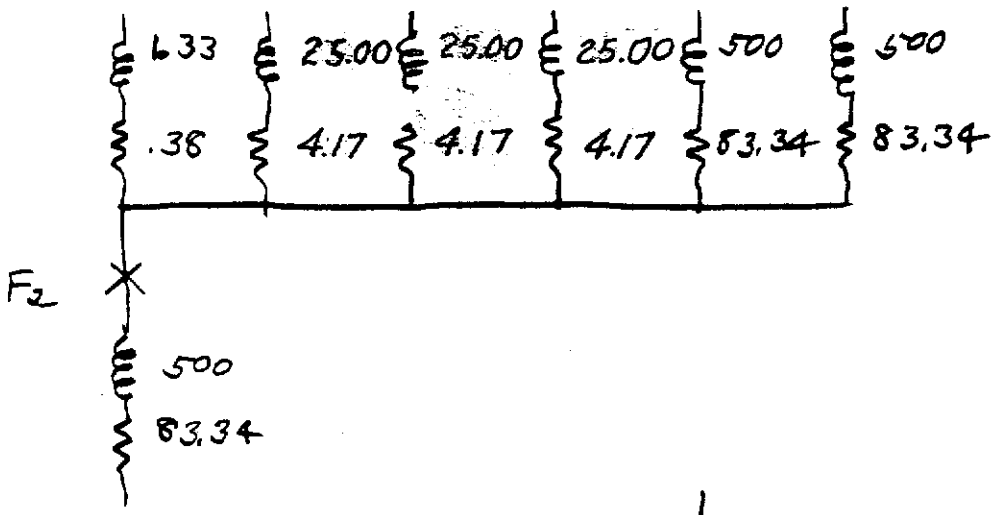
NOTE PUMP MOTOR FUSED DISCONNECT SWITCHES MUST BE RATED AT OR ABOVE 9475A

($\frac{X}{R}$ = 3.63 I_{SC} = 9474.27)

Subject: DAVENPORT FLOOD PROTECTION		Date: 10-5-78
Computed by	Checked by: bc	Sheet 4 of 12

GAINES STREET PUMPING STATION.

FAULT #2



$$PU_z = \frac{1}{\dfrac{1}{.38+j1.33} + \dfrac{1}{4.17+j25.00} + \dfrac{1}{4.17+j25.00} + \dfrac{1}{4.17+j25.00}}$$

$$\frac{1}{83.34+j500} + \frac{1}{83.34+j500} + \frac{1}{83.34+j500}$$

$$PU_z = .31 + j\,1.14$$

$$= 1.18 \angle 75.00°$$

$$I_{sc} = \frac{10000}{\sqrt{3} \times .48 \times 1.18} \qquad 10193.33 \text{ AMP}$$

$$\frac{X}{R} = 3.68$$

NOTE. THE FUSED DISCONNECT SWITCES FOR THE 5 HP MOTORS MUST BE RATED AT OR ABOVE 9805. AMP

$\left(\frac{X}{R} = 3.68 \quad I_{sc} = 9804.77A\right)$

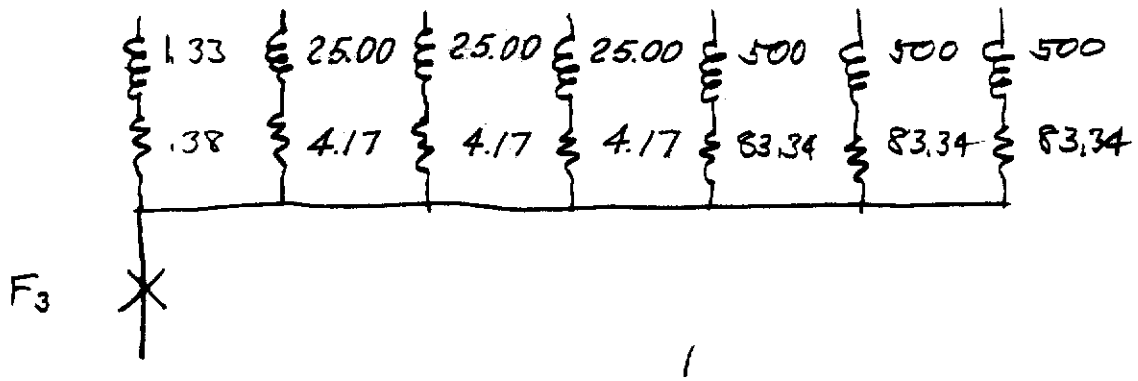
NCR Form 381a
1 Dec 77

PLATE D-13(d)

Subject		Date
DAVENPORT FLOOD PROTECTION		10-5-78
Computed by	Checked by DC	Sheet 5 of 12

GAINES STREET PUMPING STATION

FAULT #3



$$PU_Z = \frac{1}{\frac{1}{.38+j\,1.33} + \frac{1}{4.17+j\,25.00} + \frac{1}{4.17+j\,25.00} + \frac{1}{4.17+j\,25.00} + \frac{1}{83.34+j\,500} + \frac{1}{83.34+500} + \frac{1}{83.34+j\,500}}$$

$$PU_Z = .31 + j\,1.14$$

$$= 1.18 \angle 75.00^\circ$$

$$I_{sc} = \frac{10000}{\sqrt{3} \times .48 \times 1.18} = 10193.33 \text{ AMP}$$

$$\frac{X}{R} = 3.68$$

NOTE THE FUSED DISCONNECT SWITCH FOR THE 5 KVA 480/120-240 VOLT TRANSFORMER MUST HAVE AN INTERRUPTING RATING OF 10000 A OR MORE.

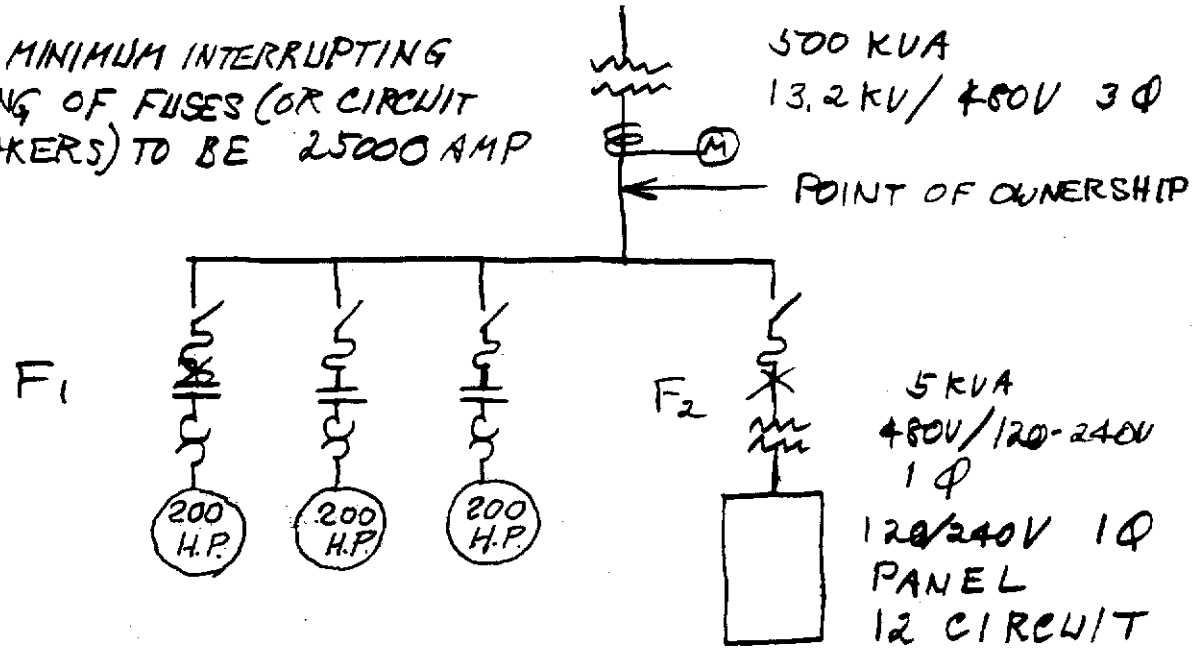
PLATE D-13(e)

Subject: DAVENPORT FLOOD PROTECTION		Date 10-5-78
Computed by	Checked by DC	Sheet 6 of 12

CREDIT ISLAND PUMPING STATION

SHORT CIRCUIT PROTECTION AND GENERAL DESIGN

NOTE MINIMUM INTERRUPTING RATING OF FUSES (OR CIRCUIT BREAKERS) TO BE 25000 AMP



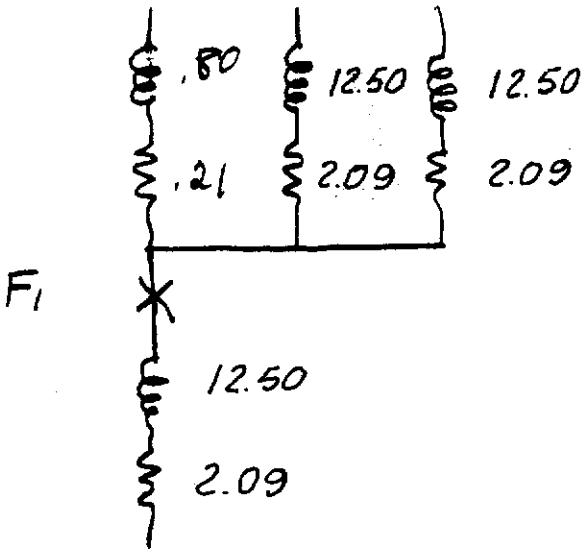
(10000 KVA BASE)

	P_{U_R}	P_{U_Z}
SERVICE TRANSFORMER (500KVA 4.12% Z) (R = 1.04% X = 4.0%)	.21	.80
150 HP MOTOR	2.09	12.50
ASSUME INFINATE UTILITY SHORT CIRCUIT KVA	0	0
NEGLECT SERVICE CABLE, SWITCH, FUSE, ETC IMPEADANCE	0	0

Subject	Date
DAVENPORT FLOOD PROTECTION	10-5-78
Computed by / Checked by	Sheet 7 of 12

CREDIT ISLAND PUMPING STATION

FAULT # 1



$$PU_Z = \frac{1}{\frac{1}{.21+j.80} + \frac{1}{2.09+j12.50} + \frac{1}{2.09+j12.50} + \frac{1}{2.09+j12.50}}$$

$PU_Z = .17 + j.67$

$= .69 \angle 76.15$

$I_{sc} = \frac{10000}{\sqrt{3} \times .48 \times .69} = 17432.07\ A$

$X/R = 3.94$

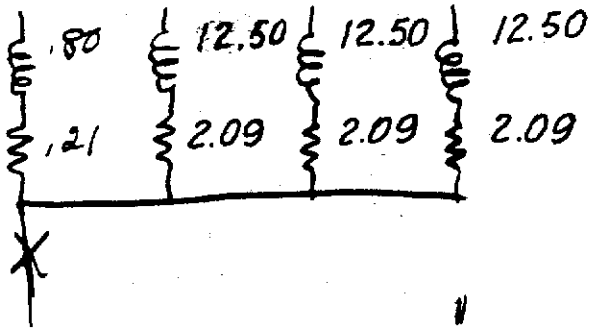
NOTE THE FUSED DISCONNECT SWITCHES FOR THE PUMP MOTORS MUST BE RATED AT OR ABOVE 16000 AMP

($X/R = 3.94$ $I_{sc} = 16654.41$

Subject		Date
DAVENPORT FLOOD PROTECTION		10-5-78
Computed by	Checked by DC	Sheet 8 of 12

CREDIT ISLAND PUMPING STATION

FAULT #2



$$PU_Z = \frac{1}{\frac{1}{.21+j.80} + \frac{1}{2.09+j12.50} + \frac{1}{2.09+j12.50} + \frac{1}{2.09+j12.50}}$$

$$PU_Z = .17 + j.67$$

$$= .69 \angle 76.15$$

$$I_{SC} = \frac{10000}{\sqrt{3} \times .48 \times .69} = 17432.07$$

$$X/R = 3.94$$

NOTE THE FUSED DISCONNECT SWITCH FOR THE 5 KVA 480/120-240 VOLT TRANSFORMER MUST HAVE AN INTERRUPTING RATING OF 17000 A OR MORE

Subject	DAVENPORT FLOOD PROTECTION	Date 10-9-79
Computed by	Checked by	Sheet 9 of 12

GAINES STREET PUMPING STATION

2% Z SERVICE TRANSFORMER (10 000 KVA BASE)

$R = 1.111\%$ $X = 1.615\%$ $PU_R = .370$ $PU_X = .555$

FAULT #1

$$PU_Z = \frac{1}{\frac{1}{.37+j.555} + \frac{1}{4.17+j25.00} + \frac{1}{4.17+j25.00} + \frac{1}{83.34+j500}} + \frac{1}{\frac{1}{83.34+j500} + \frac{1}{83.34+j500} + \frac{1}{4.17+j25.00}}$$

$PU_Z = .33 + j.53$

$= .62 \angle 58.12^\circ$

$I_{SC} = 19400.21$ AMP

I_{SC} – 100 HP MOTOR DISCONNECT RATING

$= 19049.76$ AMP.

$\frac{X}{R} = \frac{.53}{.33} = 1.61$

Subject	DAVENPORT FLOOD PROTECTION	Date 10-9-79
Computed by	Checked by	Sheet 10 of 12

GAINES STREET PUMPING STATION

2 ϕ 2 SERVICE TRANSFORMER

FAULT #2

$$PU_2 = \frac{1}{\frac{1}{.37+j.555} + \frac{1}{4.17+j25.00} + \frac{1}{4.17+j25.00} + \frac{1}{4.17+j25.00} + \frac{1}{83.34+j500} + \frac{1}{83.34+j500} + \frac{1}{83.34+j500}}$$

$$PU_2 = .33 + j.53$$

$$= .62 \angle 58.12^\circ$$

$$I_{sc} = 19400.21 \text{ AMP}$$

I_{sc} – 5 HP MOTOR DISCONNECT FUSE RATING

19369.73 AMP

$$\frac{X}{R} = \frac{.53}{.33} = 1.61$$

PLATE D-13(J)

Subject		Date
DAVENPORT FLOOD PROTECTION		10-9-79
Computed by	Checked by	Sheet 11 of 12

GAINES STREET PUMPING STATION

2¢ SERVICE TRANSFORMER

FAULT #3

$$PU_Z = \frac{1}{\frac{1}{.37+j.555} + \frac{1}{4.17+j25.00} + \frac{1}{4.17+j25.00} + \frac{1}{4.17+j25.00} + \frac{1}{83.34+j500} + \frac{1}{83.34+j500} + \frac{1}{83.34+j500}}$$

$$PU_Z = .33 + j.53$$

$$= .62 \angle 58.12^\circ$$

$$I_{SC} = \frac{10000}{\sqrt{3} \times .48 \times .62}$$

$$= 19400.21 \text{ AMP}$$

$$\frac{X}{R} = \frac{.53}{.33} = 1.61$$

Subject		Date
DAVENPORT FLOOD PROTECTION		10-9-79
Computed by	Checked by	Sheet 12 of 12

CREDIT ISLAND PUMPING STATION

2% Z SERVICE TRANSFORMER (10 000 KVA BASE)

$R = 1.111\%$ $X = 1.665\%$ $PU_R = .370$ $PU_X = .555$

FAULT #1

$$PU_Z = \frac{1}{\frac{1}{.37+j.555} + \frac{1}{2.09+j12.50} + \frac{1}{2.09+j12.50} + \frac{1}{2.09+j12.50}}$$

$PU_Z = .30 + j.50$

$= .58 \angle 59.55°$

$I_{SC} = 20738.16$

I_{SC} - 200 HP MOTOR DISCONNECT RATING

$= 20006.25$ AMP

FAULT #2

$I_{SC} = 2073.81$ AMP

$\frac{X}{R} = \frac{.50}{.30} = 1.67$

IOWA-ILLINOIS GAS AND ELECTRIC COMPANY

ROCK ISLAND, ILLINOIS

February 20, 1979

Mr. Don Logsdon
Corps of Engineers
Clock Tower Building
Rock Island, Illinois 61201

Dear Mr. Logsdon:

The following brief description of our bulk power electric system and interconnections is in response to your letter of February 12, 1979.

We have two fossil fuel electric generating facilities, one in Moline, Illinois, and one in Bettendorf, Iowa. We also have three blocks of combustion turbines of four units each, one at our Moline plant location and one in Coralville, Iowa (adjacent to Iowa City). In addition, we have a 25 percent ownership in the Quad-Cities Nuclear Power Station located up-river from the Quad-Cities near Cordova, Illinois.

We also have a 29 percent ownership in the coal-fired Neal #3 generating unit of Iowa Public Service Company located near Sioux City, Iowa, and 32.4 percent ownership in Iowa Power and Light Company's coal-fired Council Bluffs #3 generating unit just placed in service during December, 1978. We will have an 18.5 percent interest in a new coal-fired unit under construction near Ottumwa, Iowa, scheduled for completion in 1981, and to be operated by Iowa Southern Utilities Company. The Louisa Generating Station, a 650 megawatt coal-fired unit we are building in Louisa County, Iowa, is scheduled for completion in 1983. We will own 46.2 percent of this unit. In addition, we will own 15 percent of a two-unit nuclear generating station planned for completion in 1988 and 1989 in Carroll County, Illinois, to be operated by Commonwealth Edison Company.

We are interconnected with many other utility systems at 345 Kv, 161 Kv and 69 Kv. Our 161 Kv and 69 Kv systems include loops around the Quad-Cities area, with 69 Kv circuits extending to the principal generating stations.

Exhibit D-1

Our Company is a member of MAPP (Mid-Continent Area Power Pool). Members of this group include 12 investor-owned utilities, 8 cooperative organizations, 13 municipalities, 2 Nebraska public power districts and the U. S. Bureau of Reclamation. It is the intent of this power pool to provide collective adequate capacity for all members and to provide reserve capacity for fully adequate and reliable service to the customers of all members.

We are also interconnected with Commonwealth Edison, Chicago (75 percent owner of the Quad-Cities Nuclear Power Station), Union Electric Company, St. Louis, Illinois Power Company, and through others to many other utility systems.

A map showing interconnecting utilities underlined in red is enclosed. Also enclosed is a copy of our 1977 Statistical Supplement showing pertinent information on pages 21 through 23.

If you require additional information, please contact me on 326-7405.

Very truly yours,

Thomas P. House

Thomas P. House
Industrial Engineer

TPH*lg
Enclosures

Exhibit D-1

MISSISSIPPI RIVER
DAVENPORT, IOWA

LOCAL FLOOD PROTECTION
PHASE II
GENERAL DESIGN MEMORANDUM

APPENDIX E

DETAILED ESTIMATE OF COST
(REVISED)

US ARMY ENGINEER DISTRICT, ROCK ISLAND
CORPS OF ENGINEERS
CLOCK TOWER BUILDING
ROCK ISLAND, ILLINOIS 61201

DETAILED ESTIMATE OF COST

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MISSISSIPPI RIVER
DAVENPORT, IOWA

LOCAL FLOOD PROTECTION
PHASE II

GENERAL DESIGN MEMORANDUM
APPENDIX E

DETAILED ESTIMATE OF COST

I - INTRODUCTION

1. General. This appendix contains detailed cost estimates of the Davenport Local Flood Protection Project, including Federal and non-Federal first costs, maintenance and operation costs, and annual charges. The resolution by the city expressing willingness to buy Nahant Marsh is not reflected in costs shown in this appendix. (See Main Report, paragraph 9.)

2. Unit Costs. The unit costs used are based on July 1981 prices. These unit costs are considered to be fair and reasonable costs to a well-equipped and capable contractor, including contractor's overhead and profit. They have been derived from bid abstracts for comparable work, taking into account special factors that might influence the unit costs. Where design has progressed sufficiently to permit reasonably accurate takeoffs of quantities the items are broken down to the estimated quantities tities involved. For some items, at this time, if is necessary to estimate the costs on job or lump-sum basis.

3. Annual Costs. Project costs by year of expenditure are shown in table F-3 in Appendix F, Economic Analysis. Average annual costs are tabulated in table F-4 of appendix F.

II - SUMMARY OF PROJECT COSTS

Cost Acct #	Item	Main Reach Federal	Main Reach Non-Federal	Water Treatment Plant Federal	Water Treatment Plant Non-Federal	Total Federal	Total Non-Federal
01	Lands & Damages		1,284,000[1/]		35,000		1,319,000
02	Relocations		36,000[7/]				36,000
06	Fish and Wildlife Facilities (Nahant Marsh)	[2/][5/]358,000	-263,000[2/][5/]			358,000[5/]	-263,000[5/]
11	Levees & Floodwalls	17,750,000	526,000[6/]	964,000		18,714,000	526,000
13	Pumping Plants	3,960,000				3,960,000	
14	Recreation	[3/]2,438,000	2,543,000[4/]			2,438,000	2,543,000
19	Buildings, Grounds, Utilities	100,000				100,000	
30	Engineering & Design	2,272,000		91,000		2,363,000	
31	Supervision & Admin.	1,698,000		69,000		1,767,000	
TOTAL		28,576,000	4,126,000	1,124,000	35,000	29,700,000	4,161,000
COMBINED FEDERAL AND NON-FEDERAL TOTAL						33,861,000	

NOTES:

1/ Includes 313,000 for Nahant Marsh land.

2/ Includes 261,000 credit to local interests for purchase of Nahant Marsh mitigation land and 20,000 credit for future operation and maintenance of mitigation features.

3/ Includes 326,000 for E&D, S&A on recreation features.

4/ Includes 118,000 for E&D, S&A on recreation features and 1,532,000 recreation land costs and 105,000 non-shared costs for boat rental building and ballfield lighting.

5/ Additional Nahant Marsh preservation costs of 2,500 Federal and 2,500 non-Federal are in Recreation classification.

6/ Includes 65,000 for E&D, S&A (Non-Federal)

7/ Includes 5,000 for E&D, S&A (Non-Federal)

III - DETAIL COSTS BY AREA

ITEM	Qty	Unit	Unit Cost	Amount Federal	Amount Non-Federal
MAIN REACH - LANDS AND DAMAGES					
Levee right-of-way*	1	Job	Sum		820,000*
Ponding Area	1	Job	Sum		150,000
Linwood Stone Quarry Borrow Area	1	Job	Sum		1,000
TOTAL, MAIN REACH - LANDS AND DAMAGES					971,000

* Does not include costs for Nahant Marsh which are listed separately below.

MAIN REACH - RELOCATIONS (Includes allowance for E&D, S&A)					
Buildings					
Picnic Shelter, Sta 26+20	1	Job	Sum		6,500
Snack Shop, Sta 24+00	1	Job	Sum		6,500
Subtotal					13,000
Contingencies					1,500
Total, Buildings					14,500
Electric					
Sta 38+00± Beiderbecke Drive Closure, Relocate one street light pole	1	Job	Sum		1,500
Sta 13+00± vic Dock Restaurant Relocate buried electric cable and manholes to outside toe of T-Wall	250	LF	15.00		4,000
Subtotal					5,500
Contingencies					1,000
Total, Electric					6,500

ITEM	Qty	Unit	Unit Cost	Amount Federal	Amount Non-Federal
Water					
Sta 72+94 Move fire hydrant	1	Job	Sum		1,250
Sta 55+05 Move fire hydrant	1	Job	Sum		1,250
Subtotal, water					2,500
Contingencies					500
Total, water					3,000
Gas					
Sta 247+00 - 237+00± Relocate existing gas line to freeboard of levee	300	LF	7.50		3,000
Sta 139+00 - 128+00± Relocate existing gas line to freeboard of levee	800	LF	7.50		7,000
Subtotal					10,000
Contingencies					2,000
Total, Gas					12,000
Total, Relocations					36,000
MAIN REACH - FISH AND WILDLIFE FACILITIES, NAHANT MARSH					
Lands and Damages					
Land Acquisition**					
Mitigation Lands					315,000
Total, Mitigation, Lands and Damages					315,000
Credit for purchase of mitigation lands*				261,000	-261,000
Mitigation Features					
Levee, impervious fill	8,500	CY	5.00	35,490	7,010
36" RCP	30	LF	80.00	2,000	400
Precast flared-end conc. section	1	EA	1000	835	165

ITEM	Qty	Unit	Unit Cost	Amount Federal	Amount Non-Federal
Mitigation Features (Cont'd)					
Inlet control headwall closure	1	EA	1850	1,545	305
Outlet control closure	1	EA	1500	1,250	250
Landscaping					
Trees	150	EA	125	15,660	3,090
Shrubs	200	EA	60.00	10,020	1,980
Subtotal, Features				66,800	13,200
Contingencies				10,200	2,800
Total, Mitigation Features				77,000	16,000

NOTE: Additional costs associated with Nahant Marsh are listed below under Recreation.

Replacement, Operation & Maintenance of Mitigation Features, Present Worth*				20,000	-20,000
MAIN REACH - LEVEES AND FLOODWALLS					
Levees					
Stripping					
Fed	91,376	CY	1.25	114,220	
Non-Fed	1,500	CY	1.25		1,875
Excavation, common, inspection trench	52,746	CY	1.85	97,580	
Backfill, inspection trench	13,602	CY	3.10	42,165	
Disposal of waste stripping and inspection trench excavation					
Fed	83,963	CY	2.50	209,910	
Non-Fed	1,500	CY	2.50		3,750
Riprap	38,100	CY	21.50	819,150	
Bedding	12,539	CY	21.50	269,590	
Excavation, common, McManus Creek	7,200	CY	1.85	13,320	
Embankment fill, compacted, impervious	154,778	CY	6.25	967,365	

* Federal costs will be in the form of reimbursement to local interests or applied to other cost-shared features.

** Additional Land Acquisition costs associated with Nahant Marsh are listed separately below under Recreation - Lands and Damages.

ITEM	Qty	Unit	Unit Cost	Amount Federal	Amount Non-Federal
Levees (Cont'd)					
Embankment fill, semi-compacted, pervious, hydraulic					
Fed.	968,932	CY	1.90	1,840,970	
Non Fed.	23,518	CY	1.90		44,685
Embankment fill, landside slope, semi-compacted	50,158	CY	1.90	95,300	
Embankment fill, landside slope, semi-compacted, impervious	27,964	CY	6.25	174,775	
Clearing and grubbing	24.5	AC	3750	91,875	
Seeding	53	AC	1250	66,250	
Paving and base					
Fed.	1701	LF	52.00	88,450	
Non Fed.	1950	LF	52.00		101,400
Excavation, common, roadway					
Fed.	8572	CY	1.85	15,860	
Non Fed.	8572	CY	1.85		15,860
Excavation, dredged foundation preparation, Credit Island levee	135,994	CY	4.00	543,980	
Sta 247+00 - 237+00+ Provide existing 8" water line with flexible couplings and granular fill landside of levee	1	Job	Sum	4,000	
Sta 139+000 - 128+00+ Provide existing 8" water line with flexible couplings and granular fill landside of levee	1	Job	Sum	8,500	
Subtotal				5,463,260	167,570
Contingencies				816,740	25,430
Non-Federal E&D, S&A					27,000
Total, Levees				6,280,000	220,000
Drainage Facilities					
Sta 139+03 Install 30" CMP	90	LF	65.00	5,850	
Sta 138+92 Remove existing twin 96" CMP culvert	1	Job	Sum	12,000	

ITEM	Qty	Unit	Unit Cost	Amount Federal	Amount Non-Federal
Drainage Facilities (Cont'd)					
Sta 129+50± Triple 96" CMP culvert					
96" CMP	915	LF	340	311,100	
Excavation	8000	CY	5.60	44,800	
Backfill	6,300	CY	6.75	42,525	
Protection for water and gas lines	1	Job	Sum	13,640	
Site protection & dewatering	1	Job	Sum	50,000	
Temporary Access to Island	1	Job	Sum	5,000	
Sta 122+50 - 98+75 Construct 72" RCP storm sewer					
Section 122+50 - 110+40					
Excavation	6749	CY	4.35	29,360	
Backfill	5040	CY	5.60	28,225	
72" RCP, Class II, on Class C bedding	1199	LF	130	155,870	
Dewatering	1	Job	Sum	12,000	
Manholes on "T" Sections	3	EA	3750	11,250	
Section 110+40 - 104+19					
Excavation	5583	CY	4.35	24,285	
Backfill	4694	CY	5.60	26,285	
72" Class II RCP on Class C bedding	623	LF	130	80,990	
Dewatering	1	Job	Sum	6,000	
Manhole or "T" Section	1	EA	3750	3,750	
Section 104+19 - 102+60					
Backfill, granular	1019	CY	4.35	4,435	
72" RCP, Class II, on Class C bedding	155	LF	130	20,150	
Dewatering	1	Job	Sum	6,000	
Section 102+60 - 98+75					
Excavation	6400	CY	4.35	27,840	
Backfill	5829	CY	5.60	32,640	
72" RCP, Class IV, on Class C bedding	400	LF	165	66,000	
Dewatering	1	Job	Sum	12,000	
Seeding, all sections	1	Job	Sum	1,250	
Outlet	1	EA	7,440	7,440	
Sta 110+00 - 104+50± Remove 27" Sanitary sewer & MH	1	Job	Sum	8,185	

ITEM	Qty	Unit	Unit Cost	Amount Federal	Amount Non-Federal
Drainage Facilities (Cont'd)					
Sta 98+85 - 82+00± Install 30" RCP storm sewer Class IV, Class B bedding	1772	LF	70.00	124,000	
Sta 90+100± Pressurize Division Street Storm Sewer	1	Job	Sum		93,000
Sta 65+00± Pressurize Marquette Street Storm Sewer	1	Job	Sum		78,000
Sta 55+07 - 45+12 Install new storm sewer					
Section 55+07 - 52+63					
Excavation	677	CY	4.35	2,945	
Backfill	665	CY	5.60	3,725	
15" RCP, Class II, on Class C bedding	260	LF	10.00	2,600	
MH Sta 52+47	1	Job	Sum	7,500	
MH Sta 55+07	1	Job	Sum	3,750	
Section 52+47 - 46+07					
Excavation	3821	CY	4.35	16,620	
Backfill	3593	CY	5.60	20,120	
36" RCP, Class III on Class C bedding	623	LF	46.00	28,660	
MH Sta 48+74	1	Job	Sum	3,750	
MH Sta 46+07	1	Job	Sum	7,500	
Section 46+07 - 45+12					
Excavation, braced sides	535	CY	5.60	3,000	
Backfill	512	CY	5.60	2,865	
36" RCP, Class III, on Class B bedding	202	LF	46.00	9,290	
Pavement removal & replacement	124	SY	45.00	5,580	
MH Sta 45+54	1	Job	Sum	7,440	
Seeding, all sections	1.0	AC	1250	1,250	
Sta 45+60 Remove and plug 6'of 18" CMP storm sewer	1	Job	Sum	2,500	
Sta 45+12 to 42+04± New 48" storm sewer interceptor					
48" RCP	220	LF	150	33,000	
Manhole sta 42+04	1	Job	Sum	7,500	
Sta 42+04 to 37+75± New interceptor storm sewer					
30" RCP	3251	LF	70.00	227,570	
Manhole	1	Job	Sum	7,500	

ITEM	Qty	Unit	Unit Cost	Amount Federal	Amount Non-Federal
Drainage Facilities (Cont'd)					
Sta 36+15 Remove 60' of abandoned 20" sewer	1	Job	Sum	4,000	
Sta 32+76 Remove 60' of abandoned 36" sewer	1	Job	Sum	4,000	
Sta 32+00 Pressurize Ripley Street Sewer	1	Job	Sum		62,000
Sta 31+76 - 5+35± Construct new interceptor storm sewer					
Section 31+76 - 24+91					
Excavation	1239	CY	5.60	6,940	
Backfill	1116	CY	5.60	6,250	
Pavement removal & replacement	63	SY	45.00	2,835	
24" RCP, Class II, Class C bedding	676	LF	17.50	11,830	
Manhole Sta 31+76					
Manhole Sta 28+90					
Manhole Sta 27+31	3	EA	7500	22,500	
Section 24+91 - 14+58					
Excavation	1643	CY	5.60	9,200	
Backfill	1384	CY	5.60	7,750	
Pavement removal & replacement	233	SY	45.00	10,485	
30" RCP, Class II Class B bedding	986	LF	32.25	31,800	
Manhole, Sta 23+78					
Manhole, Sta 23+68					
Manhole, Sta 19+67					
Manhole, Sta 14+65					
Manhole, Sta 14+18	5	EA	7500	37,500	
Manhole, Sta 21+97					
Manhole, Sta 15+90	2	EA	4000	8,000	
30" RCP Jacked excavation under RR tracks	30	LF	500	15,000	
Section 14+58 - 5+35					
Excavation, braced	1077	CY	5.60	6,030	
Backfill	930	CY	5.60	5,210	
Manhole, Sta 11+78					
Sta 10+70	2	EA	4000	8,000	
Manhole, Sta 9+98					
Sta 5+35	2	EA	7500	15,000	
Seeding, all sections	1	AC	1250	1,250	
Dewatering, all sections	1	Job	Sum	35,000	

ITEM	Qty	Unit	Unit Cost	Amount Federal	Amount Non-Federal
Drainage Facilities (Cont'd)					
24" RCP, Class II, on Class B bedding	808	LF	17.50	14,140	
Sta 29+25± Remove 60' of abandoned 48" pipe	1	Job	Sum	3,750	
Sta 19+87± Remove 60' of abandoned 18" pipe	1	Job	Sum	3,750	
Sta 14+25 Install new 21" sanitary sewer, class III, class C bedding	120	LF	55.00	6,600	
Manhole	3	EA	3500	10,500	
Sta 14+70± Remove existing 21" RCP san. sewer	1	Job	Sum	3,750	
Sta 14+60± Remove existing MH & 30' of 20" san. sewer	1	Job	Sum	3,750	
Sta 14+50 Install MH and 50' of 36" RCP, Class III, Class B bedding	1	Job	Sum	6,500	
Sta 14+35 Remove existing 36" storm sewer	1	Job	Sum	1,250	
Sta 14+00± Move catch basin and install 45' of 12" RCP	1	Job	Sum	4,500	
Sta 5+10 to 4+85± Construct 8" VCP interceptor					
Manholes					
Sta 5+10	1	EA	3750	3,750	
Sta 4+85	1	EA	3750	3,750	
8" VCP	23	LF	30.00	690	
Subtotal, Drainage Facilities				1,892,795	233,000
Contingencies				287,205	35,000
Non-Federal E&D, S&A					38,000
Total, Drainage Facilities				2,180,000	306,000
Gatewells					
Sta 257+00± at I-280 Twin 10' x 10' Gatewell "A"					
Excavation	530	CY	5.60	2,970	
Backfill	434	CY	6.75	2,930	
Concrete	56	CY	275	15,400	
Grating, handrail, ladder	1	Job	Sum	1,860	
Dewatering, site protection, diversion of existing flows	1	Job	Sum	25,000	
10' x 10' Sluice gate w/ appurtenances	2	EA	37,000	74,000	

ITEM	Qty	Unit	Unit Cost	Amount Federal	Amount Non-Federal
Gatewells (Cont'd)					
Sta 238+50± 96" Gatewell "B"					
Excavation	859	CY	5.60	4,810	
Backfill	494	CY	6.75	3,335	
Concrete	108	CY	275	29,700	
96" Sluice gate w/ appurtenances	1	Job	Sum	31,000	
Diversion of existing sewage flows, dewatering, site protection	1	Job	Sum	35,000	
Grating, ladder, handrail	1	Job	Sum	1,490	
96" RCP, Class IV	190	LF	250	47,500	
Sta 238+40 15" Gatewell "C"					
Excavation, dewatering, and backfill	1	Job	Sum	6,000	
15" RCP, Class III, Class A bedding	165	LF	25.00	4,125	
15" Slide Gate w/lift	1	EA	Sum	8,000	
Concrete, non-reinforced	15	CY	150	2,250	
48" RCP, Class II	18	LF	75.00	1,350	
Grating, handrail, ladder	1	Job	Sum	1,250	
Sta 134+12 Credit Island Slough Inlet, twin 8' x 8' Gatewell "D"					
Slide Gates w/motor lift	2	EA	27,000	54,000	
Grating, handrail, ladders	1	Job	Sum	7,500	
Protection of water and gas lines	1	Job	Sum	10,000	
Site protection & dewatering	1	Job	Sum	50,000	
Excavation, structural	5,500	CY	5.60	30,800	
Backfill	4,500	CY	6.75	30,375	
Concrete, reinforced	970	CY	250	242,500	
Sta 123+10± McManus Creek Outlet twin 10' x 10' Gatewell "E"					
Excavation	1,900	CY	5.60	10,640	
Backfill	600	CY	6.75	4,050	
Concrete, reinforced	1,080	Cy	250	270,000	
Slide gates w/motor lift 10' x 10'	2	EA	37,000	74,000	
Gratings, ladders, handrails on gatewells and headwalls	1	Job	Sum	10,000	
Site protection & dewatering	1	Job	Sum	60,000	

ITEM	Qty	Unit	Unit Cost	Amount Federal	Amount Non-Federal
Gatewells (Cont'd)					
Sta 98+85+ 72" Gatewell "F"					
Excavation, backfill, dewatering, site protection, bypass or pumping of existing flows	1	Job	Sum	60,000	
72" RCP					
Class IV, Class "C" Bedding	48	LF	160.00	7,680	
Class IV, Class "A" Bedding	100	LF	185.00	18,500	
Sluice gate & appurtenances	1	Job	Sum	22,000	
Grating, ladder, handrail	1	Job	Sum	1,250	
Concrete, reinforced	100	CY	280	28,000	
Concrete, non-reinforced	54	CY	150	8,100	
Sta 98+55 60" Gatewell "G"					
60" Sluice gate & appurtenances	1	Job	Sum	20,000	
Concrete, reinforced	58	CY	280	16,240	
Concrete, non-reinforced	41	CY	150	6,150	
60" RCP, Class IV, Class "A" bedding	100	LF	175	17,500	
Excavation, backfill, site protection, dewatering, and removal of existing MH and pipe	1	Job	Sum	30,000	
New MH	1	Job	Sum	7,500	
Grating, handrail, ladder	1	Job	Sum	1,250	
Sta 89+78 Twin 72" Gatewell "H"					
Excavation	2400	CY	5.60	13,440	
Backfill	2250	CY	6.75	15,190	
Dewater, site protection, pump or bypass flows	1	Job	Sum	45,000	
72" Sluice gate & appurt.	2	EA	22,000	44,000	
Concrete	110	CY	280	30,800	
Ladders, grating, handrails	1	Job	Sum	2,000	
Sta 82+10+ 60" Gatewell "I"					
Excavation	1100	CY	5.60	6,160	
Backfill	1000	CY	6.75	6,750	
Concrete	51	CY	280	14,280	
60" Sluice gate w/ appurt.	1	Job	Sum	20,000	

ITEM	Qty	Unit	Unit Cost	Amount Federal	Amount Non-Federal
Gatewells (Cont'd)					
Dewater, Site protection, bypass flows	1	Job	Sum	30,000	
Ladder, grate, handrail	1	Job	Sum	1,250	
Sta 65+85 78" Gatewell "J"					
Excavation	850	CY	5.60	4,760	
Backfill	800	CY	6.75	5,400	
78" Sluice gate & appurt.	1	EA	Sum	25,000	
Grating, handrail, ladder	1	Job	Sum	1,250	
Dewater, Site protection, bypass for storm flows	1	Job	Sum	45,000	
78" RCP, Class IV, Class "A" bedding, incl. excavation, backfill, & existing RCP removal	100	LF	280	28,000	
Concrete, reinforced	60	CY	280	16,800	
Concrete, non-reinforced	60	CY	150	9,000	
Sta 52+58 60" Gatewell "K"					
60" Sluice gate & appurt.	1	Job	Sum	20,000	
Concrete, reinforced	58	CY	280	16,240	
Concrete, non-reinforced	41	CY	150	6,150	
60" RCP, Class IV, Class "A" bedding	100	LF	175	17,500	
Excavation, backfill, site protection, dewatering, and removal of existing MH and pipe	1	Job	Sum	20,000	
Grating, handrail, ladder	1	Job	Sum	1,250	
NOTE: Sta 45+20 Gatewell "L" forms a part of Gaines Street Pump Station					
Sta 42+04 48" Gatewell "M"					
Excavation	108	CY	5.60	605	
Backfill	39	CY	6.75	265	
Concrete, reinforced	12	CY	280	3,360	
Slide gate, 48" w/lift rising stem, guides	1	EA	15,000	15,000	
Handrail, grating, ladder	1	Job	Sum	1,250	
Dewatering	1	Job	Sum	12,000	
Sta 37+88 30" Gatewell "N"					
Excavation	160	CY	5.60	895	
Backfill	145	CY	6.75	980	
Concrete, reinforced	21	CY	280	5,880	

ITEM	Qty	Unit	Unit Cost	Amount Federal	Amount Non-Federal
Gatewells (Cont'd)					
Slide gate, 30" self-contained, non-rising stem, guides, floor box, and packing gland	1	EA	Sum	12,000	
Dewatering	1	Job	Sum	12,000	
Pressurized access cover w/frame 4.5' x 2.0'	1	EA	Sum	600	
Sta 32+06 66" Gatewell "O"					
Excavation	395	CY	5.60	2,210	
Backfill	321	CY	6.75	2,165	
Concrete, reinforced	32	CY	280	8,960	
Slide gate, 66" self-contained, w/nonrising-stem, floor box, and packing gland	1	Job	Sum	18,600	
Dewatering	1	Job	Sum	12,000	
Pressurized access cover w/frame	1	EA	Sum	600	
Sta 31+83 30" New Gatewell "P"					
Excavation	367	CY	5.60	2,055	
Backfill	336	CY	6.75	2,270	
Concrete, reinforced	17	CY	280	4,760	
Slide gate, 30" self-contained, w/non-rising stem, packing gland and floor box	1	Job	Sum	12,000	
Dewatering	1	Job	Sum	12,000	
Pressurized access cover w/frame	1	EA	600	600	
Sta 29+13 48" New Gatewell "Q"					
Excavation	964	CY	5.60	5,400	
Backfill	910	CY	6.75	6,145	
Concrete, reinforced	30	CY	280	8,400	
Slide gate, 48" self-contained, w/non-rising stem, packing gland and floor box	1	Job	Sum	15,000	
Dewatering	1	Job	Sum	12,000	
Pressurized access cover w/frame	1	EA	600	600	
Sta 27+33 48" New Gatewell "R"					
Excavation	382	CY	5.60	2,140	
Backfill	334	CY	6.75	2,255	

ITEM	Qty	Unit	Unit Cost	Amount Federal	Amount Non-Federal
Gatewells (Cont'd)					
Concrete, reinforced	23	CY	280	6,440	
Slide gate, 48" self-contained, w/non-rising stem, packing gland and floor box	1	EA	Sum	15,000	
Dewatering	1	Job	Sum	12,000	
Pressurized access cover w/frame	1	EA	600	600	
Sta 25+33 60" Gatewell "S"					
Excavation	560	CY	5.60	3,135	
Backfill	500	CY	6.75	3,375	
Concrete, reinforced	39	CY	280	10,920	
Slide gate, 60" self-contained, w/non-rising stem, packing gland and floor box	1	EA	Sum	18,000	
Dewatering	1	Job	Sum	12,000	
Pressurized access cover w/frame	1	EA	Sum	600	
RCP, 60", Class III on Class "B" bedding	118	LF	175	20,650	
Outlet headwall	1	Job	Sum	11,000	
Sta 23+75 36" Gatewell"T"					
Excavation	546	CY	5.60	3,060	
Backfill	353	CY	6.75	2,385	
Concrete, reinforced	22	CY	280	6,160	
Slide gate, 36" self-contained, w/non-rising stem, floor box, and packing gland	1	Job	Sum	12,400	
Pressurized access cover	1	EA	600	600	
RCP, 36", Class IV on Class "B" bedding	44	LF	125	5,500	
Sta 19+64 36" Gatewell "U"					
Excavation	486	CY	5.60	2,720	
Backfill	460	CY	6.75	3,105	
Concrete, reinforced	22	CY	280	6,160	
Slide gate, 36" non-rising stem, floor box, and packing gland	1	Job	Sum	12,000	
Dewatering	1	Job	Sum	12,000	
Pressurized access cover	1	EA	600	600	
Sta 15+21 54" Gatewell "V"					
Excavation	720	CY	5.60	4,030	

ITEM	Qty	Unit	Unit Cost	Amount Federal	Amount Non-Federal
Gatewells (Cont'd)					
Backfill	617	CY	6.75	4,165	
Concrete, reinforced	47	CY	280	13,160	
Concrete, non-reinforced	15	CY	150	2,250	
Slide gate, 54" self-contained, w/non-rising stem, floor box, and packing gland	1	Job	Sum	15,000	
Pressurized access cover	1	EA	600	600	
48" Elliptical conc. pipe	25	LF	150	3,750	
Sta 14+88 36" Gatewell "W"					
Excavation	256	CY	5.60	1,435	
Backfill	63	CY	6.75	425	
Concrete, reinforced	22	CY	280	6,160	
Concrete, non-reinforced	23	CY	150	3,450	
Slide gate, 36" self-contained, w/non-rising stem, floor box, and packing gland	1	Job	Sum	12,000	
Pressurized access cover	1	EA	600	600	
48" RCP, Class IV, Class "B" bedding	125	LF	150	18,750	
Sta 10+12 36" Gatewell "X"					
Excavation	108	CY	5.60	605	
Backfill	39	CY	6.75	265	
Reinforced concrete	12	CY	250	3,000	
Slide gate, w/rising stem, lift operator	1	Job	Sum	12,000	
Handrail, grating, ladder	1	Job	Sum	1,250	
Dewatering	1	Job	Sum	12,000	
Sta 5+38 36" Gatewell "Y"					
Excavation	139	CY	5.60	780	
Backfill	47	CY	6.75	320	
Concrete, reinforced	22	CY	280	6,160	
Slide gate w/lift, 36"	1	Job	Sum	12,000	
Handrail, grating, ladder	1	Job	Sum	1,250	
Dewatering	1	Job	Sum	6,000	
Sta 5+05 66" Gatewell "Z"					
Excavation	440	CY	5.60	2,465	
Backfill	295	CY	6.75	1,990	
Concrete, reinforced	47	CY	280	13,160	
Slide gate 66" w/lift	1	Job	Sum	20,000	
Handrail, grating, ladder	1	Job	Sum		
Dewatering	1	Job	Sum	6,000	

ITEM	Qty	Unit	Unit Cost	Amount Federal	Amount Non-Federal
Gatewells (Cont'd)					
NOTES:					
(1) Sta 3+77, 78" Gatewell "AA" forms a part of the River Drive Closure					
(2) Sta 174+24, 24" Gatewell "RI" is for recreation enhancement and is listed with proposed Credit Island recreation features					
Subtotal, Gatewells				2,401,650	
Contingencies				368,350	
Total, Gatewells				2,770,000	
Floodwalls					
I-Wall					
Stripping	569	CY	1.85	1,055	
Excavation	165	CY	3.10	510	
Backfill/embankment fill	2653	CY	6.75	17,910	
Concrete	452	CY	220	99,440	
Sheet Piling PZ-27	8384	SF	14.25	119,470	
Toe drain, 12" perforated pipe	466	LF	22.50	10,485	
Catch Basins	4	EA	1250	5,000	
6" Riprap bedding	130	CY	25.00	3,250	
18" Riprap	545	Ton	18.50	10,085	
Seeding	2	AC	1250	2,500	
Sta 22+25 Protect existing buried telephone cable and provide granular fill	1	Job	Sum	1,000	
"T" floodwall					
Excavation, common	15,540	CY	3.10	48,180	
Backfill,	9725	CY	3.75	36,470	
Concrete, reinforced	5900	CY	220	1,298,000	
Dewatering, site protection	1	Job	Sum	65,000	
Remove & replace existing concrete pavement	1736	SY	44.50	77,250	
Sheet piling vic 14+18	4,000	SF	14.25	57,000	
Sheet piling, dewatering vic 5+86 - 3+86	1	Job	Sum	100,000	
Toe Drain	1224	LF	22.50	27,540	

ITEM	Qty	Unit	Unit Cost	Amount Federal	Amount Non-Federal
Floodwalls (Cont'd)					
Sta 10+60 Protect cable and provide annular granular fill at crossing of wall	1	Job	Sum	1,000	
New Manhole	1	Job	Sum	5,000	
Remove and replace existing 8" railroad tile drains	300	LF	18.50	5,550	
Folding floodwall					
Excavation	894	CY	3.10	2,770	
Backfill	406	CY	6.75	2,740	
Levee inpervious fill	10553	CY	6.25	65,960	
Concrete, reinforced	2226	CY	250	556,500	
Toe Drain 12" perforated CMP, w/granular filter material	2257	LF	22.50	50,780	
Aluminum Panel					
Aluminum	27,588	LB	4.35	120,010	
Seal	3664	LF	2.50	9,160	
Steel	34,400	LB	2.50	86,000	
Concrete Panel					
Concrete	167	CY	300	50,100	
Steel	117,384	LB	2.50	293,460	
Seal	792	LF	2.50	1,980	
Sheet piling (PZ-27)	40,628	SF	14.25	578,950	
Pavement Removal	12,465	SY	10.00	124,650	
New Pavement, concrete	2,671	SY	35.00	93,480	
Site protection, dewatering	1	Job	Sum	66,000	
Seeding	3	AC	1250	3,750	
Concrete grid turf	880	SF	6.00	5,280	
Maintenance road base and paving	2970	SY	13.50	40,100	
	Subtotal			4,143,365	
	Contingencies			626,635	
	Total, Floodwalls			4,770,000	
Closures					
I-280 Railroad Closure					
Dewatering	1	Job	Sum	6,000	
Temporary railroad bridge	1	EA	31,000	31,000	
Excavation, structural	749	CY	5.60	4,195	
Sheet Piling, PZ 27	2088	SF	14.25	29,755	

ITEM	Qty	Unit	Unit Cost	Amount Federal	Amount Non-Federal
Closures (Cont'd)					
Backfill	700	CY	6.75	4,725	
Concrete, I Wall	65	CY	250	16,250	
Concrete, closure foundation	33	CY	220	7,260	
Structural Aluminum	1584	LB	4.35	6,890	
Structural Steel	5542	LB	2.50	13,855	
Toe Drain	60	LF	22.50	1,350	
Timbers	1728	BM	0.75	1,295	
Seal	162	LF	2.50	405	
Flapgate, 12"	2	EA	1250	2,500	
Railroad Closure near Pershing Street Sta 9+40 ±					
Dewatering	1	Job	Sum	8,000	
Temporary bridge	1	Job	Sum	16,000	
Excavation	240	CY	5.60	1,345	
Sheet Piling, PMA 22	220	SF	14.25	3,135	
Backfill	150	CY	6.75	1,010	
Concrete	155	CY	220	34,100	
Steel, structural	5,900	LB	2.50	14,750	
Toe Drain	18	LF	22.50	405	
Sta 45+55 ± Gaines Street Closure Structure					
Remove existing pavement	219	SY	10.00	2,190	
Replace pavement	107	SY	35.00	3,745	
Remove & plug existing 30" RCP storm sewer	1	Job	620	620	
Remove existing catch basin drain pipe	1	Job	620	620	
Excavation	288	CY	5.60	1,615	
Backfill	104	CY	6.75	700	
Concrete	184	CY	220	40,480	
Toe Drain	48	LF	22.50	1,080	
Steel, structural	18,757	LB	2.50	46,895	
Aluminum, structural	1497	LB	4.35	6,510	
Seals	553	LF	2.50	1,385	
Main St. Ramp Closure Structure					
Concrete					
Concrete, reinforced	260	CY	185	48,100	
Piling, Steel Sheet, PMA 22	780	SF	14.25	11,115	

ITEM	Qty	Unit	Unit Cost	Amount Federal	Amount Non-Federal
Closures (Cont'd)					
Piling, Bearing - HP8-36	685	LF	21.00	14,385	
Access Covers Incl. Frames & Mounting Hardware	252	SF	25.00	6,300	
Excavate 800 cy ±, Backfill 400 cy ±. Barricades, Remove and Replace Paving as Required and Dewater	1	Job	Sum	18,000	
Structural Steel	12,500	LBS	2.50	31,250	
Structural Aluminum	3170	LBS	4.35	13,790	
Concrete Turf Blocks	1080	SF	5.00	5,400	
Steel Decking	3180	LBS	2.10	6,680	
Perry Street Closure					
Concrete, reinforced	195	CY	220	42,735	
Site Protection, Excavation Backfill, Dewatering, Removal of Pavement & Traffic Control	1	Job	Sum	10,000	
Access Covers, Frames & Hardware	180	SF	25.00	4,500	
Structural Steel	6,500	LB	2.50	16,250	
Structural Aluminum	1500	LB	4.35	6,525	
Steel Decking	1820	LB	2.10	3,825	
Sta. 3+77 River Drive Closure and Gatewell "AA"					
Dewatering	1	Job	Sum	15,000	
Diversion of existing 78" sanitary sewer flow	1	Job	Sum	25,000	
Excavation, structural	1066	CY	5.60	5,970	
Excavation, rock	42	CY	75.00	3,150	
Sheet piling	450	SF	14.25	6,415	
Backfill	620	CY	6.75	4,185	
Concrete	438	CY	220	96,360	
Toe Drain	80	LF	18.00	1,800	
Removal of existing pavement	325	SY	10.00	3,250	
Replacement of pavement	159	SY	35.00	5,565	
Steel, structural	31,732	LB	2.50	79,330	
Aluminum, structural	2578	LB	4.35	11,215	

ITEM	Qty	Unit	Unit Cost	Amount Federal	Amount Non-Federal
Closures (Cont'd)					
Slide gate w/ non-rising stem, packing box	1	Job	Sum	25,000	
Pressurized access cover	1	EA	600	600	
Seals	1063	LF	2.50	2,660	
Remove & Replace existing RR tracks on River Drive	80	LF	62.00	4,960	
Traffic Diversion	1	Job	Sum	4,000	
East 3rd Street Closure					
Remove and Replace Conc. Street, sidewalk, curb	92	SY	44.50	4,095	
Excavation & Backfill	1	Job	Sum	3,750	
PZ-27 Sheet Pile	820	SF	14.25	11,685	
Closure Concrete & Reinforcement	75	CY	220	16,500	
Imbedded, Metal, Trench Cover, etc.	1	Job	Sum	21,700	
Toe Drain	82	LF	22.50	1,845	
Aluminum Posts and panels, hardware	1	Job	Sum	19,220	
Manhole	1	Job	Sum	3,100	
Traffic control, protection & dewatering	1	Job	Sum	12,000	
granular fill around 6" pipe landward of closure	1	Job	Sum	500	
Provide granular fill around 12" gas line landside of closure	1	Job	Sum	500	
East 2nd Street Closure					
Remove and Replace Conc. Street, sidewalk, curb	78	SY	44.50	3,470	
Excavation & Backfill	1	Job	Sum	2,500	
PZ-27 Sheet Pile	635	SF	14.25	9,050	
Concrete, reinforced	64	CY	220	14,080	
Imbedded metal	1	Job	Sum	18,230	
Toe Drain	70	LF	22.50	1,575	
Aluminum posts and panels	1	Job	Sum	16,120	
Manhole	1	Job	Sum	3,100	
Traffic control and dewatering	1	Job	Sum	12,000	
granular fill around 8" pipe landward of closure	1	Job	Sum	500	

ITEM	Qty	Unit	Unit Cost	Amount Federal	Amount Non-Federal
Closures (Cont'd)					
Beiderbecke Ramp Walkthru Closure					
Remove existing pavement	477	SY	10.00	4,770	
Replace pavement	60	SY	32.25	1,935	
Excavation	18	CY	5.60	100	
Sheet Piling	563	SF	14.25	8,025	
Backfill	27	CY	6.75	185	
Concrete	37	CY	250	9,250	
Toe Drain	40	LF	22.50	900	
Embankment fill, compacted	104	CY	6.75	700	
Steel, structural	6,561	LB	2.50	16,405	
Aluminum, structural	2,214	LB	4.35	9,630	
Closure Structure near Band Shell					
Excavation	633	CY	3.10	1,960	
Backfill, compacted Impervious	595	CY	6.25	3,720	
Concrete, reinforced	39	CY	250	9,750	
Structural Aluminum	2675	LB	4.35	11,400	
Toe Drain, 12" perforated pipe	55	LF	22.50	1,250	
Steps, conc.	1	Job	Sum	1,875	
Access Covers, Cast Iron	825	LB	2.50	2,075	
Sheet Piling, PZ 27	800	SF	14.25	11,650	
Closure Structure at Ripley St. Extended					
Excavation	640	CY	3.10	1,985	
Backfill, compacted Impervious	600	CY	6.25	3,750	
Concrete, reinforced	40	CY	250	10,000	
Structural Aluminum	2670	LB	4.35	11,615	
Toe Drain, 12" perforated pipe	54	LF	22.50	1,215	
Access Covers, Cast Iron	850	LB	2.50	2,125	
Sheet Piling, PZ 27	800	SF	14.25	11,400	
Steps, conc.	1	Job	Sum	2,500	
Closure Structure at Harrison St. Extended					
Excavation	640	CY	3.10	1,985	
Backfill, compacted Impervious	600	CY	6.25	3,750	
Concrete, reinforced	40	CY	250	10,000	
Structural Aluminum	2670	LB	4.35	11,615	
Toe Drain, 12" perforated pipe	54	LF	22.50	1,215	

ITEM	Qty	Unit	Unit Cost	Amount Federal	Amount Non-Federal
Closures (Cont'd)					
Access Covers, Cast Iron	850	LB	2.50	2,125	
Sheet Piling, PZ 27	800	SF	14.25	11,400	
Steps, conc.	1	Job	Sum	2,500	
Brady Street Walkthru Closure					
Remove existing pavement, parking island	595	SY	10.00	5,950	
Replace pavement	75	SY	32.25	2,420	
Excavation	22	CY	5.60	125	
Sheet Piling	700	SF	14.25	9,975	
Backfill	34	CY	6.75	230	
Concrete, reinforced	46	CY	250	11,500	
Toe Drain	50	LF	22.50	1,125	
Embankment fill, compacted	130	CY	6.75	880	
Steel, structural	8160	LB	2.50	20,400	
Aluminum, structural	2755	LB	4.35	12,000	
Steps, conc.	1	Job	Sum	2,500	
Subtotal				1,268,265	
Contingencies				191,735	
Total, Closures				1,460,000	
Beautification					
Downtown					
Planters for Trees	35	EA	500	17,500	
Planters for Shrubs	67	EA	620	41,540	
Trees	150	EA	125	18,750	
Shrubs	300	EA	43.50	13,050	
Ground Cover	1000	EA	1.25	1,250	
Crescent Park Landscaping					
Trees	300	EA	125.00	37,500	
Shrubs	600	EA	43.50	26,100	
Credit Island Landscaping					
Trees	200	EA	125.00	25,000	
Shrubs	400	EA	43.50	17,400	
Subtotal				198,090	
Contingencies				33,910	
Total, Beautification				232,000	

ITEM	Qty	Unit	Unit Cost	Amount Federal	Amount Non-Federal
Mitigation					
Downtown					
Conversation Areas					
Concrete	25	SY	22.50	565	
Furniture	36	EA	250.00	9,000	
Trash receptacles	12	EA	125.00	1,500	
Lighting	1	Job	Sum	6,200	
Concrete walk	666	SY	15.00	9,990	
Credit Island					
Fishing Access Path	5200	SY	3.75	19,500	
Subtotal				46,755	
Contingencies				11,245	
Total, Mitigation				58,000	
TOTAL, MAIN REACH - LEVEES AND FLOODWALLS				17,750,000	461,000
MAIN REACH - PUMPING PLANTS					
Credit Island Pumping Plant and Closure					
Pump Station					
Pumps, 30", motors and controls	3	EA	62,000	186,000	
Slide Gates, 8'x4', w/motor lifts	3	EA	24,000	72,000	
Building, 14x36, w/vents, hatches	1	Job	Sum	15,000	
Flapgates, 30"	3	EA	3,750	11,250	
Sump pump w/shear gates and piping	1	Job	Sum	5,000	
Storage shed and stop logs	1	Job	Sum	1,200	
Trash racks, gratings, handrails, etc.	20,500	LB	1.85	37,900	
Guardrail	150	LF	40,000	6,000	
Concrete	1950	CY	220	429,000	
Closure structure					
Concrete	1875	CY	220	412,500	
Bulkheads, A-588	95,000	LB	1.00	95,000	
Bulkhead Rollers and Appurtenances,	5000	LB	5.00	25,000	
Service Bridge	1	Job	Sum	30,000	
Misc. metal work	4000	LB	1.85	7,400	
Crane	1	EA	Sum	35,000	
Guardrail	90	LF	40.00	3,600	
Misc. Items					
Impervious fill	950	CY	8.00	7,600	

ITEM	Qty	Unit	Unit Cost	Amount Federal	Amount Non-Federal
MAIN REACH - PUMPING PLANTS (Cont'd)					
Riprap	125	CY	22.50	2,800	
Granular bedding	45	CY	25.00	1,150	
Excavation, silt in cofferdam, and cells	14,000	CY	5.60	78,400	
Excavation, rock, in cofferdam	300	CY	37.00	11,100	
Dewatering	1	Job	Sum	100,000	
Pervious Fill for cells	28,400	CY	6.25	177,500	
Sheet pile cells, permanent	29,078	SF	17.50	508,900	
Sheet pile cells, temporary	31,214	SF	12.50	390,200	
Lean concrete for shale protection	150	CY	80.00	12,000	
Concrete for retaining walls	150	CY	250.00	37,500	
Power Supply 500 KVA	1	Job	Sum	25,000	
Gaines Street Pump Station					
Pumps, 20", motors, and controls	3	EA	44,000	132,000	
Handrail, trash racks, grating and stoplog frames	1	Job	Sum	16,000	
Discharge pipe, and flapgates	3	EA	25.00	7,500	
Concrete block building w/ventilators, hatches, etc.	1	Job	Sum	20,000	
Excavation	2200	CY	5.60	12,300	
Backfill,	1850	CY	6.75	12,500	
Dewatering	1	Job	Sum	30,000	
Slide gates, 5' x 5' w/motor lifts	4	EA	20,000	80,000	
RCP, 60"	110	LF	175	19,250	
Discharge headwall	1	Job	Sum	6,000	
Fencing	1	Job	Sum	3,000	
Road gravel & seeding	1	Job	Sum	1,200	
Electrical, supply and interior (225 KVA)	1	Job	Sum	12,000	
Concrete, reinforced	625	CY	250	156,250	
Main Street Pump Station					
Excavation	1450	CY	5.60	8,100	
Backfill	1200	CY	6.75	8,100	
Fill to grade	650	CY	6.75	4,400	
Pumps, motors, and controls	2	EA	36,000	72,000	

ITEM	Qty	Unit	Unit Cost	Amount Federal	Amount Non-Federal
MAIN REACH - PUMP PLANTS (Cont'd)					
Concrete, reinforced	150	CY	250	37,500	
Covers, gratings, and trash rack	1	Job	Sum	7,500	
Flap gates, 18"	2	EA	2000	4,000	
Discharge pipe 18" steel	160	LF	65.00	10,400	
Gate valves, 18"	2	EA	15,000	30,000	
Dewatering	1	Job	Sum	15,000	
Manholes for valves	2	EA	4000	8,000	
Electric service	1	Job	Sum	8,000	
Subtotal				3,445,000	
Contingencies				515,000	
TOTAL, MAIN REACH - PUMP PLANTS				3,960,000	
MAIN REACH - BUILDINGS, GROUNDS, UTILITIES					
Storage Buildings, vic Sta 2+00, Sta 45+00, and 237+00 and Water Treatment Plant	3400	SF	25.00	85,000	
Subtotal				85,000	
Contingencies				15,000	
TOTAL, MAIN REACH - BUILDINGS, GROUNDS, UTILITES				100,000	
WATER TREATMENT PLANT - LANDS AND DAMAGES					
Land Acquisition	1	Job	Sum		35,000
TOTAL, WATER TREATMENT PLANT - LANDS AND DAMAGES					35,000
WATER TREATMENT PLANT - LEVEES AND FLOODWALLS					
Levee and Road Ramp Road Gravel	160	CY	16.00	2,560	
Inspection Trench Excavation	8,400	CY	1.25	10,500	
Inspection Trench Backfill using material excavated	4,400	CY	2.50	11,000	
Inspection Trench waste	4,000	CY	4.35	17,400	

ITEM	Qty	Unit	Unit Cost	Amount Federal	Amount Non-Federal
WATER TREATMENT PLANT - LEVEES AND FLOODWALLS (Cont'd)					
Compacted Impervious fill	13,500	CY	8.00	108,000	
Riprap	650	CY	22.50	14,625	
Granular bedding	220	CY	25.00	5,500	
Seeding	1.5	AC	1,200	1,800	
Cap Wall					
Drill & Grout in Reinf., 2,000 holes 1-1/2" diam 24" deep	1	Job	Sum	20,000	
Concrete	1000	CY	220	220,000	
T-Wall					
Excavation	230	CY	5.50	1,265	
Backfill	190	CY	6.75	1,280	
Concrete	85	CY	250	21,250	
Dewater (175 LF±)	1	Job	Sum	10,000	
I-Wall					
Excavation	190	CY	5.50	1,045	
Backfill	150	CY	6.75	1,015	
Concrete	85	CY	275	23,375	
Sheet Pile PZ 27	1150	SF	14.25	16,400	
Intake Area Closure					
Structural aluminum	565	LBS	6.25	3,530	
Structural steel, galv., embedded	400	LBS	2.50	1,000	
Upstream R R Closure					
Structural aluminum	860	LBS	6.25	5,375	
Structural steel, galv. embedded	625	LBS	2.50	1,560	
Cast Iron frames & covers	475	LBS	2.50	1,200	
Concrete	18	CY	250	4,500	
Sheet Piling PZ 27	275	SF	14.25	3,925	
Excavation	70	CY	5.50	385	
Backfill	50	CY	6.75	350	
Temporary track support and replacing track & ballast	1	Job	Sum	34,000	
Downstream R R Closure					
Structural aluminum	425	LBS	6.25	2,650	
Structural steel, galv. embedded	550	LBS	2.50	1,375	
Cast Iron frames & covers	475	LBS	2.50	1,200	
Concrete	15	CY	250	3,750	
Sheet Piling PZ 27	235	SF	14.25	3,350	
Excavation	70	CY	5.50	385	
Backfill	50	CY	6.75	350	

ITEM	Qty	Unit	Unit Cost	Amount Federal	Amount Non-Federal
WATER TREATMENT PLANT - LEVEES AND FLOODWALLS (Cont'd)					
Temporary track support and replacing track & ballast	1	Job	Sum	34,000	
Gatewell W1					
Rock excavation	34	CY	75.00	2,550	
Excavation, common	1150	CY	5.50	6,325	
Backfill	990	CY	6.75	6,685	
Concrete	88	CY	250	22,000	
Slide gate & lift 78" dia.	1	EA	Sum	25,000	
Slide gate & lift 48" dia.	1	EA	Sum	15,000	
Covers & Handrail	1	Job	Sum	1,750	
Gatewells W2 & W3					
Excavation, common	975	CY	5.50	5,365	
Backfill	900	CY	6.75	6,075	
Concrete	5	CY	250	1,250	
Slide gate & lift 24" diam	2	EA	10,000	20,000	
Covers & Handrail	1	Job	Sum	1,750	
Cut 24" hole Gov. Interceptor	1	Job	Sum	1,200	
Precast pipe	1	Job	Sum	7,400	
Gatewell W4					
Excavation	190	CY	5.50	1,045	
Backfill	160	CY	6.75	1,080	
Slide gate & lift 48" diam	1	EA	Sum	15,000	
Covers and Handrail	1	Job	Sum	1,750	
Concrete	35	CY	250	8,750	
Gatewell W5					
Rock excavation	28	CY	75.00	2,100	
Common, excavation	220	CY	5.50	1,200	
Backfill	170	CY	6.75	1,150	
Concrete	55	CY	250	13,750	
Slide gate & lift 78" dia.	1	EA	Sum	25,000	
Covers & Handrail	1	Job	Sum	1,750	
C.I. Pipe 16" dia. sewer	16	LF	105.00	1,680	
C.I. Pipe 24" dia. sewer	50	LF	125	6,250	
RCP 12" dia. sewer	125	LF	65.00	8,125	
MH 5' dia.	2	EA	3,750	7,500	
MH 4' dia.	4	EA	3,500	14,000	

ITEM	Qty	Unit	Unit Cost	Amount Federal	Amount Non-Federal
WATER TREATMENT PLANT - LEVEES AND FLOODWALLS (Cont'd)					
Excavate at Gov. Interceptor and plug existing sewers	5	EA	1,850	9,250	
Plug drain holes in Gov. Interceptor (from inside Int.)	125	EA	12.50	1,560	
Toe drain, 12" perforated pipe & 0.3 CY graded filter/ft.	165	LF	22.25	3,670	
Remove abandoned sewer	130	LF	10.00	1,300	
Subtotal				838,160	
Contingencies				125,840	
TOTAL, WATER TREATMENT PLANT, LEVEES AND FLOODWALLS				964,000	
RECREATION - LANDS AND DAMAGES					
Land Acquisition	1	Job	Sum		1,532,000*
TOTAL, RECREATION - LANDS AND DAMAGES					1,532,000
RECREATION - FEATURES					
Stripping					
Fed	2,200	CY	1.25	2,750	
Non-Fed	2,200	CY	1.25		2,750
Disposal of waste stripping and inspection trench excavation (hauling and placement at Crescent Park spoil areas)					
Fed	10,772	CY	2.50	13,465	
Non-Fed	10,772	CY	2.50		13,465

* Includes 2,000 land acquisition costs for Nahant Marsh recreation lands.

ITEM	Qty	Unit	Unit Cost	Amount Federal	Amount Non-Federal
Embankment fill, semi-compacted pervious (from river dredging)					
Fed	47,503	CY	1.85	87,880	
Non-Fed	47,503	CY	1.85		87,880
Embankment fill, semi-compacted, landside slope (dredged from slough)					
Fed	4279	CY	6.25	26,750	
Non-Fed	4279	CY	6.25		26,750
Clearing and grubbing					
Fed	1.5	AC	3750	5,625	
Non-Fed	1.5	AC	3750		5,625
Scenic Road Paving, Base and Curb					
Fed	3525	LF	54.00	190,350	
Non-Fed	3525	LF	54.00		190,350
Excavation, common (for roadway)					
Fed	8572	CY	1.85	15,860	
Non-Fed	8572	CY	1.85		15,860
Crescent Park hydraulic fill					
Fed	25,000	CY	4.35	108,750	
Non-Fed	25,000	CY	4.35		108,750
Bicycle Trail along Levee					
Paving and Base	8500	SY	9.50	40,375	40,375
Signs	12	EA	65.00	390	390
Rest Stops (2 benches 20 SY asphalt)	10	EA	620.00	3,100	3,100
Crescent Park Bicycle Trail					
Pavement & base	2,320	SY	9.50	11,020	11,020
Signs	1	Job	Sum	300	300
Rest stops	1	Job	Sum	300	300
Embankment fill, ramps	1400	CY	5.00	7,000	7,000
Baseball Diamond (4)					
Backstops	4	EA	1900	3,800	3,800
Fence	4500	LF	15.00	33,750	33,750
Bleachers 8 x 60'	500	LF	12.50	3,125	3,125
Multi-Purpose Building	1	Job	Sum	61,500	61,500
Water Supply, 4" CIP 1100 feet	1	Job	Sum	7,000	7,000
Wastewater removal 12" RCP 1100 feet	1	Job	Sum	11,000	11,000
Parking lot 200 cars					
Asphalt pavement and base	13,500	SY	13.75	92,800	92,800
Wheel stops	250	EA	40.00	5,000	5,000

ITEM	Qty	Unit	Unit Cost	Amount Federal	Amount Non-Federal
Markings	1	Job	Sum	300	300
Lighting	1	Job	Sum	3,250	3,250
Picnic Area					
Shelters	5	EA	12,500	31,250	31,250
Tables	80	EA	275	11,000	11,000
Grills	40	EA	100	2,000	2,000
Fountains	2	EA	625	625	625
Trash containers	30	EA	125	1,875	1,875
Restroom					
Building	484	SF	100.00	24,200	24,200
4" Water Supply	1000	LF	18.50	9,250	9,250
12" RCP Sanitary Sewer	1000	LF	20.00	10,000	10,000
Electric Supply 700' #4AWG	1	Job	Sum	1,500	1,500
Parking Lot					
2" Asphalt paving and base	7600	SY	13.75	52,250	52,250
Wheel stops	1600	LF	5.00	4,000	4,000
Markings	1	Job	Sum	100	100
Playgrounds	2	EA	15,000	15,000	15,000
Credit Island					
Bicycle Trail					
Pavement and base	15,600	SY	9.50	74,100	74,100
Clearing, heavy	1.5	AC	4,000	3,000	3,000
Fill, compacted	3,800	CY	5.00	9,500	9,500
Rest stops	4	EA	625.00	1,250	1,250
Signs	1	Job	Sum	375	375
Bicycle bridge on existing conc. culvert vic. Hwy 61 & Blackhawk Creek					
Excavation	67	CY	6.00	200	200
Backfill	63	CY	7.00	220	220
Guard rails anchored to conc. parapet	2223	LB	3.75	4,150	4,150
Guard railing with steel post & post hole grouting	40	LF	43.50	870	870
Concrete for abutments	10	CY	275.00	1,375	1,375
Steel Railing	3550	LB	2.50	4,450	4,450
Timber flooring w/ plywood wearing surface	480	SF	2.50	600	600
Misc. steel	300	LB	2.50	375	375
Sta 174+24 24" Gatewell "RI"					
Excavation	1067	CY	5.60	3,000	3,000
24" Slide Gate w/lift	1	EA	Sum	4,000	4,000
Backfill	989	CY	5.60	2,775	2,775
24" RCP, Class III, on Class A bedding	424	LF	50.00	10,600	10,600

ITEM	Qty	Unit	Unit Cost	Amount Federal	Amount Non-Federal
Dewatering	1	Job	Sum	3,000	3,000
Parking Lot, 48 car					
Pavement, 2" asphalt	3900	SY	13.75	26,800	26,800
Marking	1	Job	Sum	100	100
Portable Toilets	2	EA	1850	1,850	1,850
Picnic Ground					
Tables	90	EA	275	12,375	12,375
Shelters	1000	SF	12.50	6,250	6,250
Grills	45	EA	100.00	2,250	2,250
Trash Containers	30	EA	125	1,875	1,875
Portable Toilet	2	EA	1850	1,850	1,850
Water Fountains	2	EA	750	750	750
Parking Lot, (30 car)					
2" Asphalt pavement and base	2250	SY	13.75	15,475	15,475
Embankment fill	1575	CY	5.00	3,950	3,950
Marking	1	Job	Sum	100	100
Turnaround					
2" Asphalt pavement	110	SY	14.00	770	770
Embankment fill	600	CY	5.00	1,500	1,500
Boat Launch					
Clearing	1	AC	3500	1,750	1,750
Excavation	50	CY	5.00	125	125
Embankment fill	2000	CY	5.00	5,000	5,000
2" Asphalt paving	1600	SY	14.00	11,200	11,200
Concrete ramp	45	CY	300.00	6,750	6,750
Cofferdam, dewatering	1	Job	Sum	6,000	6,000
Excavation, Lagoon dredging (disposal on golf course)	100,000	CY	2.20	110,000	110,000
Seeding lagoon dredging on golf course	20	AC	1250	12,500	12,500
Tent Camping					
Picnic Tables	42	EA	275.00	5,775	5,775
Fire Rings	21	EA	40.00	420	420
Trash receptacles	14	EA	125.00	875	875
Nahant Marsh, Educational Development					
Gravel parking lot and access road	1	Job	Sum	2,250	2,250
Subtotal				1,260,870	1,260,870
Contingencies				189,130	189,130
Total, Shared Recreation Features				1,450,000	1,450,000

Non-Sharable Costs (Includes allowance for E&D, S&A)					
Ballfield Lights	1	Job	Sum		32,000
Boat Rental Bldg					
Embankment fill					
(dredged river sand)	19,000	CY	2.30		43,700
Electric feeder supply	1	Job	Sum		4,500
Bldg. Proper	360	SF	32.00		11,520
Subtotal					91,720
Contingencies					13,280
Total, Non-Shared Recreation Features					105,000
Total, Recreation Features				1,450,000	1,555,000

MISSISSIPPI RIVER
DAVENPORT, IOWA

LOCAL FLOOD PROTECTION
PHASE II
GENERAL DESIGN MEMORANDUM

APPENDIX F

ECONOMIC ANALYSIS

(REVISED)

US ARMY ENGINEER DISTRICT, ROCK ISLAND
CORPS OF ENGINEERS
CLOCK TOWER BUILDING
ROCK ISLAND, ILLINOIS 61201

MISSISSIPPI RIVER
DAVENPORT, IOWA

LOCAL FLOOD PROTECTION
PHASE II
GENERAL DESIGN MEMORANDUM

APPENDIX F
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PHASE II
GENERAL DESIGN MEMORANDUM

APPENDIX F
ECONOMIC ANALYSIS

SECTION 1

1.1. Purpose. The purpose of the Phase II report is to provide a functional engineering document which is primarily concerned with the technical design of the measures necessary to achieve the objectives selected in Phase I. The Phase I General Design Memorandum (GDM) for Davenport, Iowa, was approved by OCE on 2 May 1978. However, recognizing that conditions will change over time with regard to such things as the nature and use of the flood plain, economic environment and price levels, plan modifications, and evaluation criteria (e.g. discount rate), the purpose of this appendix is to provide a current analysis of the economic efficiency of the approved plan and to reaffirm economic justification.

1.2. Methodology. Prior to its approval, the Phase I GDM underwent extensive review by elements within the Corps of Engineers, including BERH and OCE. The purpose of such review was to insure, among other things, that the project benefits had been evaluated and presented under a reasonable set of assumptions and under appropriate methodology, given the existing regulations. The subsequent approval of the Phase I report by these higher Corps authorities indicated the acceptability of the economic analysis and the methods employed for the evaluation of project benefits.

a. The Engineering Regulation which prescribed the responsibilities, requirements, and procedures for Phase II postauthorization studies is ER 1110-2-1150. This regulation states that for the Phase II GDM the project benefits presented in Phase I should be updated as appropriate. For this reason, and for the reason expressed in the previous paragraph, the current analysis attempts to adhere to the assumptions and procedures outlined in the Phase I GDM as closely as possible. Accordingly, much of the data generated for the Phase I GDM has been updated and incorporated into this study. To reflect the changes which have occurred in the flood plain in recent years, a substantial amount of new data has been generated and correlated with the previously existing information. This was accomplished through an area canvass, additional interviews with occupants of the flood plain, and a review of recently published reports pertaining to the study area. Resulting from the additional field study effort was, for example, the identification of structures and activities which have vacated the flood plain and of those which are new occupancies of the flood plain.

b. In order to fully account for any changes which may have occurred in the number and nature of physical assets, intensity of economic activities, price levels, and evaluation criteria, it became necessary that the Phase II benefits evaluation be something more than a simple price update exercise. Therefore, there are some deviations from and revisions to the Phase I methodology. Such changes are noted and explained at the appropriate points in the discussion which follows.

c. The development of costs and benefits for economic evaluation of the plan follows standard Corps of Engineers' practices. The values of all goods and services used in plan implementation are estimated as project costs. For a detailed design memorandum, such as this, these estimations are quite detailed and specific. The beneficial effects, from the national perspective (NED account), are the increases in the value of the output of goods and services and improvements in national economic efficiency. The underlying theoretic concepts are the increases in consumers' surplus and economic rent resulting from a plan. The general measurement standard for increases in national output is the total value of the increased output, where the total value is defined as the willingness of users to pay for each incremental output of a plan. For urban flood protection, the effect of a plan on national economic development is manifested by increasing the productivity of land or by reducing the costs of using land resources, thereby releasing resources for the production of goods and services elsewhere in the economy. The estimate of actual or prospective damages prevented is used as an approximation for the net increase in the value of outputs and for the improvement in economic efficiency.

d. The values derived for costs and benefits, which typically accrue at differing times, are made comparable by conversion to an equivalent time basis using the appropriate discount rate and period of analysis by the usual financial conventions. For this project, the applicable discount rate is 7-5/8 percent and the economic life is 100 years. The project base year, or the year in which it is expected to become operational, is 1988. Damages, costs, and benefits in this report are presented as 1988 amounts. The effects of inflation are not included ex ante. The price levels for costs and benefits are for July 1981.

1.3. Procedure. The first step in the evaluation of flood damages and damages reduced was to obtain all available information pertaining to past floods. In this way, actual damages could be correlated with known depths of inundation and flood discharges. Surveys were also made for properties which would be affected by floods of various magnitudes to obtain additional depth-damage relationships. These relationships were based on the damages expected to occur for incremental depths of water measured in half-foot increments below or above the first floors of buildings. Because the first floor elevations were known in terms of feet above the national geodetic vertical datum, damages for individual structures could be aggregated to form the basic elevation-damage relationships over the relevant range of flood heights. These relationships were plotted in the form of elevation-damage curves. In the preparation of these curves, where existing levees or seawalls afforded some protection, these structures were credited with

preventing damages for flood stages up to the limit of safe operation. The existing seawall in Davenport ranges in top elevation from 558 feet, NGVD, to 561 feet, NGVD, between the Centennial Bridge and the Government Bridge, thusly affording protection against floods of approximately 8- to 12-year magnitudes. Table F-1.1 shows the expected depth of flooding for various floods at selected intersections in Davenport.

TABLE F-1.1

Depth of Flooding (Feet)

Location	Recurrence Interval (Years) 50	100	200	500	SPF
River Drive and Iowa	5.2	7.1	9.0	11.1	12.6
Western	4.5	6.4	8.3	10.4	11.9
Brown	6.4	8.3	10.2	12.3	13.8
Division	2.7	4.6	6.5	8.6	10.1
Fairmount	-2.0	-0.1	1.8	3.9	5.4
2nd Street and Iowa	3.2	5.1	7.0	9.1	10.6
Western	-2.5	-0.6	1.3	3.4	4.9
Brown	-0.1	1.8	3.7	5.8	7.3

a. From the resulting elevation-damage curves and the hydrologic information in Appendix A, damage-frequency relationships were prepared. This was accomplished by relating flood profiles (elevations) for given discharges to the elevation-damage curves to obtain damage-discharge curves. These were in turn related to discharge-frequency curves to obtain the basic damage-frequency curves from which average annual damages were computed. When flood frequencies are expressed as exceedence probabilities, average annual damages are obtained by computing the area beneath the damage-frequency curve. Plate F-1 is the aggregate damage-frequency curve for Davenport under existing conditions. It represents the summation of several reach-specific curves. The flood plain in Davenport was divided into several damage reaches so that each reach known depths of inundation and flood discharges. Surveys were also made for properties which would be affected by floods of various magnitudes to obtain additional depth-damage relationships. These relationships were based on the damages expected to occur for incremental depths of water measured in half-foot increments below or above the first floors of buildings. Because the first floor elevations were known in terms of feet above the national geodetic vertical datum, damages for individual structures could be aggregated to form the basic elevation-damage relationships over the relevant range of flood heights. These relationships were plotted in the form of elevation-damage curves. In the preparation of these curves, where existing levees or seawalls afforded some protection, these structures were credited with preventing damages for flood stages up to the limit of safe operation. The existing seawall in Davenport ranges in top elevation from 558 feet, NGVD, to 561 feet, NGVD, between the Centennial Bridge and the Government Bridge, thusly affording protection against floods of approximately 8- to 12-year magnitudes. Table F-1 shows the expected depth of flooding for various floods at selected intersections in Davenport.

b. The estimates of potential damages were obtained during the field surveys. Property owners were asked to estimate losses which could occur to building structures, building contents, grounds and landscaping, and garages and other appurtenant facilities. Business owners or managers were asked to provide information regarding damages to lands and buildings, equipment, stock, supplies, and losses in wages and profits. They were also asked to estimate what portion of wages and profits could be recovered at a later time. Judgment was also exercised to help determine whether such losses could be offset elsewhere in the economy. The loss estimates were then adjusted accordingly to reflect these considerations.

1.4. Flood Damages. Average annual damages for existing conditions are $1,507,600. These are damages accruing to the structures, contents, and activities associated with a total of 677 permanent facilities susceptible to recurring flood losses. Included in this total are 296 residences with average structural values ranging from $26,000 for homes without basements (approximately 18 percent) to $38,000 for homes with basements (approximately 82 percent); 86 industrial firms with average valuations (based on tax assessments) of $1.20 million; and 273 commercial establishments with average valuations (based on tax assessments) of $146,000. There are also 22 public facilities; but as public damages include such items as streets and sewers, utilities, parks, and several non-taxed structures, average values cannot be estimated. Therefore, the total valuation of flood plain facilities susceptible to flooding exceeds $157 million. It should be stressed that this figure is a low estimate and is not intended to capture the full value of assets in the flood plain, especially those located above the first floors of multi-story buildings.

a. Average annual damages, for the base year, by activity type categories appear in Table F-1.2. These are the damage estimates specific to the main project protection area. Other flood losses which are not geographically specific also occur in the form of interrupted water supply, detour costs, and public emergency costs. These categories will be addressed later in this appendix.

TABLE F-1.2

Average Annual Damages, 1988

Type	Number	Mississippi River
Residential	~~296~~ 450	$ 403,000
Structures		240,800
Contents		162,200
Industrial	86	559,800
Structures		24,800
Contents		261,800
Financial		273,200
Commercial	273	277,900
Structures		22,500
Contents		122,700
Financial		132,700
Public 1/	22	266,900
Structures		106,800
Contents		160,100
TOTAL		$1,507,600

1/ Includes damages to streets and sewers, utilities, and park facilities.

1.5. Residual Damages. Residual damages are a measure of the remaining flood damage situation under the "with project" condition. These are the damages resulting from the occurrence of floods exceeding the project's level of protection. To the extent that land use in the flood plain is the same with and without the project, the benefits are the difference in flood damages with and without the project. Because the project will not significantly increase flood profiles (see paragraph A-24), the damage-frequency relation for floods with a probability of 0.5 percent (200-year flood) or less is not substantially changed for the with project condition. Average annual residual damages are, therefore, represented by the area beneath the damage-frequency curve from the origin to 0.5 percent recurrence probability. This total is $449,500. Of this total, $116,300 are attributed to the residential sector, $239,700 to the industrial sector, $62,100 to the commercial sector, and $31,400 to the public sector.

a. The standard project flood profile is approximately 2.7 feet above the 200-year flood profile and an average of 0.3-0.4 foot below the project design grade (see Plate A-2). Under existing conditions, damages from the SPF could amount to as much as $45-50 million. Because the

average leadtime for forecasting flood crests at Davenport is six weeks, action can be taken to minimize the residual hazard to health and life, as well as to property.

b. The preceding is not intended to downplay the remaining flood hazard, which is significant. Because additional development is expected in the flood plain, potential damages due to overtopping or project failure will be even higher in the future. The occupants of the flood plain must guard against a false sense of security and be prepared to act on flood warnings.

c. The remaining flood hazard has an important bearing on the plan formulation process in Phase I. A review of formal correspondence between the District Engineer, Rock Island, and the Mayor of Davenport during Phase I (August 1976) reveals the consideration given the remaining risk. The District Engineer offered the following points regarding the proposed 200-year level of protection:

1. that SPF protection was economically justified;

2. that flood fighting efforts would be required to add freeboard to insure that freeboard would be maintained during higher floods;

3. that overtopping or sudden collapse of a levee or wall system could have catastrophic effects on lives and property; and

4. that the city should be aware that floods greater than the SPF could occur under extraordinary conditions.

In response, the city of Davenport noted the city's professional staff and citizens advisory groups had worked with the Corps staff in the development of design features, project alignment, and level of protection. It was noted that the residual hazard had been considered and that the city preferred a 200-year level of protection, with the following points offered in support of that decision:

1. that the professional staff of Davenport favored a level of protection in the 100-year to 200-year range;

2. that during numerous public meetings strong opposition to high levees and walls had been expressed by a significant portion of the public;

3. that strong, organized opposition to a level of protection higher than the 200-year could be expected;

4. that if higher protection were proposed by the Corps, the city would probably deem such a project unacceptable; and

5. that a 200-year level of protection was consistent with other area projects, principally with that of Rock Island.

It was also indicated that given the slow-rising nature of the river and adequate forecasts, temporary evacuation and emergency raising of the levee would be possible.

d. As is the case with the estimation of average annual damages and benefits for damages reduced, annual residual damages are a probabilistic measure. Therefore, it is possible to quantify the remaining risk in terms of probabilities alone. Using the binominal distribution and the 100-year planning horizon, there is a 60.6 percent probability that no floods exceeding the design flood will be observed in the next 100 years.

1.6. Costs. Detailed estimates of project costs are presented elsewhere in this report. However, to facilitate the comparison of costs and benefits which will be developed in this appendix, summary costs tables are provided below. Table F-1.3 lists project expenditures by calendar year and Table F-1.4 summarizes average annual costs.

TABLE F-1.3

Project Costs by Year of Expenditure (in $1,000's)

Year	Main System		Water Plant		Recreation		Total		
	Federal	Non-Federal	Federal	Non-Federal	Federal	Non-Federal	Federal	Non-Federal	Federal And Non-Federal
1983	374	0	1,034	35	0	0	1,408	35	1,443
1984	2,928	1,284	0	0	12	2	2,940	1,286	4,341
1985	4,206	0	0	0	98	0	4,304	0	4,435
1986	5,887	164	0	0	387	0	6,274	164	6,438
1987	7,834	350	0	0	648	1,730	8,482	2,080	10,462
1988	3,184	66	0	0	1,293	530	4,477	596	4,927
Total	24,413 2/	1,864 2/	1,034	35	2,438	2,262 1/	27,885	4,161	32,046

1/ Includes non-shared cost facilities.
2/ Includes Nahant Marsh mitigation and enhancement.

TABLE F-1.4

Average Annual Costs (in $1,000's)

First Costs	Main System		Water Plant		Recreation		Total		
	Federal	Non-Federal	Federal	Non-Federal	Federal	Non-Federal	Federal	Non-Federal	Federal And Non-Federal
Project Expenditure	24,413 1/	1,864 1/	1,034	35	2,438	2,262	27,885	4,161	32,046
Interest During Construction	4,885	592	39	1	240	227	5,164	820	5,984
Total First Costs	29,298	2,456	1,073	36	2,678	2,489	33,049	4,981	38,030
Annual Costs									
Interest and Amortization	2,291	192 1/	84	3	209	195	2,584	390	2,974
Operation and Maintenance	0	93 1/	0	2	0	85	0	180	180
Average Annual Costs	2,291	285	84	5	209	280	2,584	570	3,154

1/ Includes Nahant Marsh.

1.7. Average Annual Costs. Interest during construction accounts for the cost of capital incurred during the construction period up to that time the project begins to produce benefits. Computation is based on the yearly expenditures, the number of interest periods remaining until the base period for each year's expenditure, and the project discount rate. Since the water treatment plant will receive full protection during the first year of construction, interest on the expenditures allocated for that purpose was based on one interest period. Interest for all other project purposes was based on the appropriate number of conversion periods. More detailed information regarding protection of the treatment plant, costs, and incremental justification is provided later in this appendix.

1.8. Flood Damage Reduction Benefits. The project is designed to eliminate damage from a 200-year flood on the Mississippi River. The annual flood reduction benefits for existing conditions are $1,058,100 for the river. This total represents the average annual reduction in damages sustained by those permanent facilities noted earlier. Additional damages will be prevented or reduced by the project, principally through reduced detour costs, protection of the city's water supply, and reduction in emergency costs. Other beneficial impacts, which would not be experienced without the project, result from advanced replacement of facilities and the employment of unemployed labor. Because the project will protect certain under-utilized land, location benefits also accrue.

a. Average annual flood reduction benefits by land use category are summarized in Table F-1.5. It should be noted that these are benefits accruing to facilities which experience direct physical damages as well as economic losses. Economic losses due to flooding also occur due to inaccessibility, especially in the flood-prone portions of the central business district. These losses, however, are much more difficult to quantify; and the resulting benefits are not included. Therefore, the indicated benefits should be considered somewhat conservative.

b. Table F-1.5 also includes the other major inundation reduction benefit categories.

TABLE F-1.5

Inundation Reduction Benefits
Under Existing Conditions

Category		Amount
Residential	$ 286,700	
Public	235,500	
Commercial	215,800	
Industrial	320,100	
Subtotal		$1,058,100
Detour Costs		5,400
Water Supply		108,900
Emergency Costs		288,000
Total Inundation Reduction Benefits		$1,460,400

1.9. Detour Costs. Because the project will protect access to the Centennial Bridge, which connects Davenport and Rock Island, Illinois, benefits have been credited for the elimination of detour costs. The average daily traffic on the bridge for 1978 was 18,800 vehicles. Based on this, a conservative estimate of 19,000 vehicles (16,000 cars and 3,000 trucks) daily over the life of the project was used to derive the benefits. The Centennial Bridge is a toll bridge with graduated charges ranging from $0.25 for passenger cars to $1.30 for larger trucks. Average daily revenues are estimated at $5,400. The detour, using the next accessible bridge, is 8 miles. At per-mile costs of $0.20 for cars and $0.50 for trucks, the additional costs of detour are $37,600 per day. The total average daily costs of bridge closure are estimated at $43,000.

a. Access to the bridge begins to be affected when the water surface elevation reaches 565 feet, NGVD. This corresponds to approximately the 60-year event. Based on the 200-year hydrograph (Plate A-46), the period of inundation is approximately 15 days for the 200-year flood. Allowing time for cleanup and repairs and the establishment and removal of detours on both sides of the river, the average closure for floods between the 60- and 200-year events is estimated at 14 days. When weighed by the probability of occurrence, average annual benefits are determined to be $5,400.

b. These damages and benefits do not include the reductions in time lost due to induced traffic congestion nor the additional cost of traffic control which becomes necessary when a major arterial bridge is closed. Experience in 1976 with bridge closure indicated that these costs are significant but nearly impossible to quantify. The congestion caused delays up to an hour or resulted in some vehicles taking even longer detours to avoid the congestion. In view of this, the benefits credited are probably quite conservative. The discussion to this point has been limited to the effects resulting from closure of the Centennial Bridge only. It should also be noted that River Drive, which parallels the

Mississippi River, is a major east-west artery subject to flooding. City of Davenport planning officials have indicated that River Drive closure adversely affects all east-west traffic in Davenport as far north as Locust Street, which is 15 blocks above River Drive when measured along Gaines Street. The table which follows provides the average daily traffic volumes for River Drive for 1978 measured at selected locations.

TABLE F-1.6

River Drive Average Daily Traffic

Two-way Volume	Measured at:
11,900	Concord St.
12,800	Division St.
13,300	Marquette St.
15,200	Gaines St.
15,700	Harrison St.
15,600	Brady St.
12,300	Government Bridge

Source: Bi-State Metropolitan Planning Commission

c. Care must be exercised when attempting to draw inferences from these figures. As there are several intervening streets between the measurement points, the totals are not additive. The totals do, however, indicate that the magnitude of the problems ensuing from closure of River Drive can be quite severe. Unfortunately, the economic loss which could be incurred is virtually impossible to evaluate. Because the project will significantly reduce the probability of River Drive closure, it must be recognized that there are project benefits which must remain uncomputed but which would serve to enhance project justification.

d. It should be noted that construction of the project may cause traffic problems. However, the project has been designed, and its construction carefully planned, so as to minimize traffic delays. This consideration has led to the inclusion of additional costs which would not be incurred if traffic movements were not accounted for in design. The net result is that any traffic delays which may be experienced are expected to be relatively minor. As before, any disbenefits which may accrue cannot and have not been evaluated.

1.10. Water Supply. Although the project does not extend upstream of the Government Bridge, the water treatment plant will receive protection as part of the overall plan. The benefits for protecting the treatment plant were based primarily on the benefits to the water users of Davenport, who will receive protection from interruption of this important service. The Davenport Water Company is the sole source of water for over 125,000 people within the Iowa Quad Cities.

a. The computation of the benefits was based on the physical damage to the plant, the costs of emergency purchase and transportation of water during service loss, and business losses which would be sustained during a supply interruption. The physical damage to the plant was estimated as described in paragraph 3 and includes the cost of restoring the plant to a pre-flood condition. The costs of providing water on an emergency basis was based on importing an average of 10 gallons per person per day. A total of five cities in the region which are not affected by Mississippi River flooding have the excess capacity capable of providing the necessary volume of water. These cities are located at distances ranging from 10 to 190 miles from Davenport. The cost of the water itself is minimal, with the bulk of the costs for providing an emergency water supply being attributed to transportation. A total of 120 businesses not susceptible to physical flood damages are dependent on a reliable water supply for the conduct of normal business. As noted in paragraph 3b, business losses were based on the likelihood that they would not be offset by increases elsewhere or recovered at a later date.

b. The top elevation of the existing seawall at the plant is 567.5 feet, NGVD, which corresponds approximately to the profile of the 40-year flood in Pool 15. (The plant is located above Locks and Dam 15, Plate A-25 is not applicable.) The expected length of service interruption due to flooding is an average of 45 days per occurrence. This accounts for the period of inundation and the time needed to restore full service.

Benefits are based on securing the water supply against floods between the 40-year and 200-year magnitudes. Table F-1.7 provides the annualized level of benefits for the project base year, 1988.

TABLE F-1.7

Water Supply Benefits

Cost of Transportation	$ 54,000
Cost of Water	300
Business Losses	40,700
Physical Damage	13,900
Total	108,900

c. The level of the benefits is expected to increase over time due to the increase in business activity in Davenport. Although the business users involved are not confined to the flood plain, the same rates of growth developed for the computation of commercial benefits were applied in order to project business losses. The methodology used is completely analogous to that used for computing losses to wages and profits in the commercial sector under the condition of future growth. Because the treatment plant will receive protection during the first phase of construction, benefits will begin to accrue prior to the project base year. Therefore, the stream of annual benefits was accumulated forward to

the equivalent base year future value. The stream of benefits accruing from the base year through 2081 was discounted to the base year equivalent present value and added to the previous total. The sum was multiplied by the project capital recovery factor to yield the average annual equivalent benefits over the life of the project. This amount is $171,200, of which $39,300 is the annualized value of benefits accruing prior to the project base year and $23,000 is due to future business increases.

d. Because protection of the water treatment plant is a separate feature of the overall project, incremental economic justification is necessary. At this point, a deviation from the project base year will be made. Costs and benefits will be evaluated for 1983, the year in which benefits begin to accrue.

e. The total first cost for flood protection is $1,069,000, Federal and non-Federal. Interest during construction (based on a one-year construction period) is $40,000 for a total economic cost of $1,109,000. The resulting average annual cost is $89,000. The benefits, when evaluated in terms of 1983, amount to $128,100 annually. The incremental benefit/cost ratio is 1.4.

f. The amount of water which could be supplied during a service interruption would be sufficient to meet survival needs. Obviously though, the daily patterns of normal water use could not be maintained. The residents of Davenport would endure personal hardships for as long as 45 days or more, as there would be insufficient water for washing dishes or clothing, bathing, and toilet use. If an economic value could readily be placed on this aspect of an uninterrupted water supply, benefits may be significantly increased.

1.11. Emergency Costs. A review of the prior reports and records indicated that these reports did not include an important category of benefits, those which will accrue from the elimination of emergency costs. All previous benefit analyses were thoroughly researched in order to insure that these benefits had not been included in the amounts reported as public benefits in the documents presented previously. The records have proven that this is not the case; and, therefore, the appropriate benefits will be discussed and included in this report.

a. Emergency costs, which are not necessarily related to physical flood damages, are those public sector costs of flood fighting, evacuation and relocation, disaster relief, increased cost of normal operations such as police patrol, and other public costs which would not normally be incurred in the absence of flooding. For the city of Davenport, records exist for six previous floods for which such costs were analyzed. These are the actual emergency costs only and do not include any physical damage which may have been sustained from these floods. The historical costs are summarized in Table F-1.8, which follows.

TABLE F-1.8

Public Emergency Costs

Date of Occurrence	Gage Height @ L&D 15 (Feet)	Discharge C.F.S.	Cost at Time	Cost at 1981 Prices
1951	18.2	221,500	$ 64,000	$ 357,600
1952	18.6	227,500	115,000	598,000
1965	22.5	307,000	2,452,000	7,199,600
1969	19.2	242,000	526,000	1,173,300
1973	18.8	228,000	91,000	142,900
1975	19.2	216,000	600,000	818,500

Note: Flood stage @ L&D 15 is 15.0 Ft.

b. The costs of emergency measures were updated to equivalent 1981 dollar amounts by price indices based on wage rates for construction, common, and skilled labor. These statistics are published monthly in the Survey of Current Business. The use of these indices is in accordance with the most recent NCD guidance concerning the use of price indices for updating benefits, in this case there is no practical alternative to indexing. Updating the costs was necessary in order to evaluate the data on a common basis.

c. Deriving an estimate of average annual costs associated with emergency activities required the use of regression analysis on the historical data. In all, six mathematical models were tested against the data. The model with the highest correlation coefficient of regression was selected for use in the benefit analysis.

The particular model chosen provided a linear mathematical relationship between river stage and corresponding costs of emergency measures (see note).

d. Once emergency costs were related to river stage, the stage-discharge (Plate A-25) and discharge-frequency (Plate A-3) curves were incorporated to derive a frequency-cost relationship. From this it was possible to compute the annual expected (average) value of emergency costs. For floods up to and including the SPF, this amount is $356,600. The costs which will be eliminated by the plan are $288,000 annually. This amount is the average annual benefit credited to the project (200-year protection) for reducing emergency costs.

e. Analyses of this sort are extremely probabilistic in nature and the underlying theory is quite complex. However, such methods are quite valid and are extensively used in the field of quantitative methods. As is evident from the preceding table, actual emergency costs have varied greatly over a relatively narrow range of river surface elevations. The mathematical treatment of the data attempts to eliminate the variance by predicting the average cost of flood fighting for a given stage. Since the analysis is made with regard to the "long run," the use of averages is appropriate.

f. Much of the variance in costs can be accounted for by the fact emergency efforts occur in response to forecasts of impending floods. For the Mississippi River, advanced warning may be provided as much as a month or more before the expected date of the crest. Naturally, forecasts prepared so far in advance may or may not closely agree with subsequent high water experience. Although the forecasts are revised frequently to reflect changing conditions, the mobilization decision must be made early enough to insure that the measures taken are effective. Thus, regardless of the actual crest elevation eventually experienced, variability in advanced forecasts will lead to variability in emergency costs.

Note: The correlation coefficient statistic is a measure of "strength" of the mathematical relationship among the data, or how well the model "fits" or explains the variance in the data. Generally, a statistic close to 1.0 in absolute value indicates a good "fit". The model used was linear and the resulting equation is: cost = 1,711,344 x stage - 31,513,601. The correlation coefficient, r, is .983; the "t" value is 10.71, indicating that a high degree of confidence can be placed in the linearity of the relationship. The "t" value for the slope, b, is 7.08, indicating with a high level of confidence that b is significantly different from zero. The tabular values for the "t" tests are 4.604 at 4 degrees of freedom for r and 5.032 at 5 degrees of freedom for b.

1.12. Redevelopment Benefits. The purpose of this section is to evaluate the beneficial economic impacts which will accrue to the study area as a result of increased employment. Senate Document 97, 87th Congress, 2nd session, states that redevelopment benefits may be computed for areas having qualifying unemployment or underemployment. Scott County, Iowa, is eligible pursuant to the Area Redevelopment Act (PL 87-27). The EDA designation for Davenport is Title IV, Section 8.

a. Redevelopment benefits are based on the disbursements of wages to hired labor which was previously unemployed or underemployed. Total Federal construction expenditures are $25,244,000, of which $23,132,000 are for flood control purposes and $2,112,000 are for recreation purposes. Non-Federal construction expenditures amount to $1,105,000, with $508,000 and $597,000 for flood control and recreation purposes respectively. These amounts are exclusive of costs for lands and damages, engineering and design, and supervision and administration.

b. It is estimated that 50 percent of the construction costs will be allocated to on-site labor. The total wage bill for the project would be (exclusive of recreation costs) $11,820,000. It is also anticipated that 90 percent of the total workforce will be comprised of unemployed or underemployed local labor. Thus the labor costs which provide the basis for benefit computation are $10,638,000, Federal and non-Federal costs for flood control purposes only. Similar computations result in $1,219,000 as the benefit wages accruing from the associated recreation purposes.

c. Because the wages disbursements occur during the years 1983 through 1988, the redevelopment benefits must be computed for each year's expenditures. The benefits are then brought forward to the base year

equivalent value at the project rate of interest, over the appropriate number of periods remaining until the base year. The resulting total is amortized over the life of the project.

d. The tables which follow provide examples of the procedures and computations involved. The example used in Table F-1.9 is based on expenditures for flood control for the first year of construction. Table F-1.10 presents computation of annual redevelopment benefits over the life of the project.

TABLE F-1.9

Redevelopment Benefits

1. Estimated On-Site Labor Costs:

Construction Cost, 1983 1/	$964,000
Percent To Labor 2/	50
	$482,000

2. Allocation of On-Site Labor by Category:

Labor Category	On-Site Labor Cost	Percent Allocation2/	Amount Of Wages
Skilled	$482,000	40	$192,800
Semi-skilled	482,000	50	241,000
Other	482,000	10	48,200

3. Allocation of Wages to Locally Unemployed or Underemployed Labor:

Labor Category	Amount Wages	Percent To Locally Hired Unemployed Labor	Wages to Previously Unemployed Local Labor
Skilled	$192,800	903/	173,500
Semi-skilled	241,000	90	216,900
Other	48,200	90	43,400
Total			$433,800

Redevelopment benefits accruing in 1983: $433,800

1/ Includes total construction costs, less costs for engineering and design, supervision and adminstration, and lands and damages.

2/ & 3/ Estimates based on previous experience with similar local protection projects.

TABLE F-1.10

Redevelopment Benefits By
Year of Accrual

Year	Wages to Previously Unemployed Local Labor	Periods to 1/ The Base Year	Value in 2/ The Base Year
1983	$ 433,800	11	$ 654,700
1984	1,145,300	9	1,603,900
1985	1,663,700	7	2,161,800
1986	2,664,500	5	3,212,700
1987	3,803,000	3	4,254,800
1988	2,147,000	1	2,228,900
Accumulated present value in the base year:			$14,116,800
Average annual redevelopment benefits (rounded):			$ 1,103,900

1/ Number of half-year periods, equivalent half-year interest rate is 3.8125 percent per period.
2/ Computed as: Wages x $(1.038125)^n$, where n is the appropriate number of conversion periods.

e. Referring to Table F-11, the rationale for the use of half-year conversion periods is that project disbursements are assumed to flow evenly throughout any given year. Mathematically, this is equivalent to disbursing the entire amount at mid-year, which necessitates the use of half-year periods. The procedure outlined in Table F-1.10 is precisely the method used for computation of interest during construction when determining total first costs.

f. Annual redevelopment benefits accruing from flood control measures amount to $1,015,400. Benefits resulting from construction of the recreation features of the plan amount to $88,500, for total benefits of $1,103,900.

g. In order to determine the reliability of the redevelopment benefits, a detailed review was made for the employment experience at Clinton, Iowa. The Clinton project is also a local protection project very similar to that proposed for Davenport. Clinton is located approximately 40 miles up river from Davenport and the project is currently nearing completion.

h. The percentage of on-site construction costs attributed to labor was determined by comparing major work items at Davenport to similar features at Clinton. The labor costs at Clinton were applied to the Davenport features and pro-rated according to the percentage of total on-site costs represented by the particular type of work. In this way, using experience with comparable features, it was possible to determine a weighted average for total labor costs of 51.7 percent. In order to compensate for differences and to simplify computations, the factor used for the labor component of on-site costs was 50 percent. The actual labor

components at Clinton ranged from 46 percent to 59 percent, making a 50 percent estimate for Davenport quite reasonable. A similar procedure was carried out to determine labor costs as a percentage of on-site costs for the construction and installation of the recreation features of the project. The basis for determination was the District's experience with the Saylorville Lake Recreation Plan. In this case, the labor component represented 56.8 percent of on-site costs. Because of the differences in localities and between a reservoir setting and a riverine setting, a conservative estimate of 50 percent was used in computing employment benefits for the recreation aspects of the project.

i. The percentage of local hire for that project was determined through interviews with the contractor's project engineer and through examination of payroll records. These indicated that all except supervisors and several foremen were hired from the local Clinton area. A total of five samplings of employment records was made. In four cases, local employment exceeded 90 percent, once being as high as 97 percent. In the remaining case, local employment was 83 percent.

j. In the Davenport area, construction employment is generally gained through union membership. This particular practice has been verified through discussions with representatives of local unions, the AFL, and the Peoria, Illinois, Regional Office. Briefly summarized, the usual procedure is for a contractor who seeks to hire labor to contact the local unions involved. The unions in turn refer unemployed workers on a priority basis. Thus, when a local union is contacted, an unemployed worker is most likely to be hired since currently employed workers would not be on the referral list.

k. For the month of October 1981, the respective Job Services of Iowa and Illinois reported unemployment rates of 9.0 percent for Scott County, Iowa, and 10.5 percent for Rock Island County, Illinois. The number of claims filed by construction workers was in excess of 500. Since construction of the project is expected, on the average, to employ about 100 workers annually, the supply of labor is sufficient to meet project hiring needs.

l. It has been the District's experience that the primary contractors for project construction are often located well outside of local project areas. This is the case at Clinton, as the primary contractors are based in LaCrosse, Wisconsin, and Belvidere, Illinois. Except for the supervisory personnel who are permanently employed by the contractor, most of the project's hiring needs must be fulfilled by local labor. Based on the foregoing information, it becomes reasonable to expect that 90 percent of the labor required for the Davenport project will be supplied locally. This is the basis for computing the benefit wages to unemployed workers as 90 percent of the total on-site labor costs (see footnote 3, Table F-1.9).

m. The benefit estimation procedure described above is not in strict accordance with regulation ER 1105-2-354, Evaluation of Employment Benefits. The essential difference is that the regulation establishes

guidelines for the percentages of the on-site labor costs which accrue to previously unemployed local workers. Two cases are established, Case I, where local hire is expected to comprise 50 percent of total hire, and Case II, where local hire is expected to comprise 80 percent of total hire. Under section B-2 of the regulation, Case II percentage should be reduced by pro-rating when local hires are expected to be between 50 percent and 80 percent of the total workforce. Based on the preceding discussion, total local hires of 75 percent would be a conservative estimate. In this instance, the "percent to locally hired unemployed labor" percentages in part 3 of Table F-1.9 would be changed from 90 percent as follows:

Skilled	38 percent
Semi-skilled	53 percent
Other	35 percent

Under this case, the redevelopment benefits resulting from flood protection would be decreased to $507,700 and those from the recreation features to $44,300. The project B/C would change to .93 for flood protection alone and to .99 overall.

n. The inclusion of redevelopment benefits is contingent upon qualifying EDA designation. Should this designation be lost, the project B/C would be 0.75 for flood protection and 0.82 overall.

1.13. Future Benefits. Normal economic trends must be accounted for in the analysis of damages and benefits over time. Increases in population, income, employment, and business activities can be expected to result in an increase in damages to physical assets and economic activity in the flood plain. Because the project greatly reduces flood losses, benefits accruing to existing development can be expected to increase over time.

a. Davenport is the largest city of the conurbation known as the Quad Cities. It is also the largest city of the SMSA, which is composed of Scott County in Iowa and Rock Island and Henry Counties in Illinois. The Davenport SMSA is a regional trade center for the surrounding urban and rural communities and Davenport itself is a leader in commercial activity.

b. In 1967 retail sales in Davenport accounted for nearly 31 percent of the total retail volume in the SMSA. The Davenport central business district (CBD) alone accounted for 13 percent of the total SMSA retail trade. Approximately 40 percent of the CBD is located in the flood plain. However, in recent years much of the emphasis on new activity has shifted away from the CBD and the flood plain. The opening of a major shopping center outside of the flood plain resulted in a severe impact on retail activity in the CBD. The current vacancy rate for commercial and industrial space in the flood plain is 12.8 percent, opposed to 8.4 percent city-wide.

c. Having recognized the deterioration of conditions in the CBD and within the study area in general, the city of Davenport and local private business organizations are actively pursuing revitalization of the area.

As approximately 34 percent of all business and professional establishments (including manufacturing firms) in the city are in or near the flood plain, there is tremendous potential for improvement.

d. Over $17,850,000 in public and $32,300,000 in private improvements are scheduled in the next few years. Included will be improvements to streets, sidewalks, and parks, as well as building renovation and new construction. A substantial number of new permanent jobs will be added as a result.

e. In addition to local initiatives, the city of Davenport is participating in several Federal programs designed to revitalize older urban areas. In keeping with the aims of the President's Partnership to Conserve America's Communities and the President's Urban Policy, actions are under way which will have direct bearing on the quality of the residential and business environments in the flood plain. The combined local and national goals being addressed in Davenport are to increase employment in the core area, increase the tax base, decrease use of energy, retard urban sprawl, and to reduce decentralization of population.

f. While several specific cases can be cited, one of the most significant examples of combined city, private, and Federal investments is the multi-purpose Ground Transportation Center. To be located at River Drive and Harrison Street, the Ground Transportation Center will be the major transportation hub of the Iowa Quad Cities. Included will be a hotel, retail and office space, and educational facilities. Financing is approximately 29 percent Federal (UMTA), 13 percent city, and 58 percent private. A good example of the private commitment to downtown redevelopment is the combined office tower and retail specialty shop complex to be located directly across from the Ground Transportation Center. This 85,000-square-foot facility will also provide 550 parking spaces. In both cases, flood plain regulations are satisfied by placing parking and transportation facilities at the lowest levels.

g. The Downtown Davenport Development Commission has reported considerable success in retaining jobs in the core area. This is an excellent indication that the recent trend of out migration is being halted. Near- and long-term prospects for the flood plain are favorable. From the preceding information, the "without project" scenario can be summarized as one of stabilization and growth for activities in the flood plain. Existing development is being renovated and new development is and will be taking place. As a result, population will stabilize and increase as new housing is made available. Employment and commercial activity will also experience upward trends, with existing establishments enjoying increased trade. Under the without project condition, the level of damages to existing activities will increase, based on projected trends. The project is, therefore, credited with increasing benefits, but benefits are only taken for existing activities. Due to the uncertainty involved, physical and economic damages are not projected for new development.

h. The Phase I GDM established the basis for most benefit evaluations. Therefore, the year in which growth is assumed to begin is

1976. Only the base year has been changed, from 1980 to 1986. In no cases were physical damages increased over time (except in residential contents). Future increases in benefits are based on increases in population, income, and business activity and accrue to wages and profits. Table F-1.11 lists these benefits as well as values for other future benefits.

TABLE F-1.11

Future Benefits

Affluence	$ 12,100
Economic Growth	210,700
Location Benefits	269,900
Total	$492,700

1.14. Affluence Benefits. In accordance with the guidelines in ER 1105-2-351, the value of residential contents may be assumed to increase over time. For the purpose of benefit evaluation, a limit is placed on this future growth. This limit is that the value of contents may not be assumed to be greater than 75 percent of the value of the residential structure. The current annual damages to residential structures is $195,300. Damages to contents amount to $116,000. Therefore, the limit can be expected to be achieved in a relatively short period of time.

a. The rate of growth for the increase in value of contents is taken to be the average rate of increase in income per capita. The appropriate rate is 2.84 percent per year, based on the OBERS projections for income per capita for the SMSA of $3,653 in 1970 and $6,400 in 1990. The year that this increase was assumed to begin is 1976, the date of the Phase I report. However, the base year has been changed from 1980 to 1988 for this report. The average annual damages to contents considering future increases are $12,100, the increase over the base year value.

1.15. Economic Growth. Recognizing a reasonable expectation for an upward economic trend in the flood plain, benefits are credited for the increasing value of losses prevented for wages and profits. As described in paragraph 3, these benefits are for the reduction in financial losses which are not recoverable after flooding or which are not offset elsewhere. Benefits are credited for existing establishments only and are, therefore, reflective of the without project condition. Under existing conditions, annual benefits for the prevention of losses to wages and profits in the industrial sector average $1,827 per establishment. In the commercial sector, the average annual amount in $375 per establishment.

a. Interviews were held with all industrial firms in the flood plain with an assessed value of $100,000 or more. A total of 24 firms provided information useful to projecting growth trends. This sample represents over 75 percent of the manufacturing assets involved. The firms were asked to estimate changes in output, employment, and potential

flood damages, for both the recent past and expectations for the future. It was determined that in recent years there had been a substantial increase in economic activity and that this trend could be expected to continue. It was also found that while there had been significant modernization expenditures, there had not been major additions to existing facilities. Most managers felt that while modernization expenditures had increased damage potential, "down time" was the most costly aspect of flooding. Output per employee was found to have closely paralleled economic activity in the past and was expected to continue to do so in the future. The data which follow summarize the survey results with regard to changes in employment, output, and output per employee in the flood plain.

TABLE F-1.12

Annual Rates of Change, Percent

Period	Employment	Output	Output per Employee
1969-1974	2.9	5.4	2.4
1974-1979	1.5	4.6	3.1
MEAN, 1969-1979	2.2	5.0	2.75

b. An annual rate of increase of 2.75 as shown in table F-1.12 percent was used to project the increasing damages and, therefore, the increasing benefits to wages and profits in the industrial sector. The basis for using output per employee is established by the classical theory of wages, prices, employment, and production, according to which real increases in wages and profits result from increasing productivity. Since projections are being made for existing firms and since productivity is a "real" rather than a nominal measure, this is probably the best estimator available. Although overall productivity in the national economy has declined recently, manufacturing productivity has continued to increase.

c. A projection period of 50 years beginning in 1981 was used. The base-year level for benefits to wages and profits in the industrial sector is $157,000. The annual equivalent amount with discounted future growth in $242,600. The increase over the base-year level, $87,600 is the amount attributed to future benefits.

d. Benefits accruing to wages and profits in the commercial sector are also expected to increase in the future. The central business district's capture of shopper's goods sales in Davenport was 25 percent of the city-wide volume in 1971. In a detailed analysis, Gladstone Associates projected the capture to decrease to 18 percent in 1980 and 17 percent in 1985. The market share is expected to stabilize thereafter. Since shopper's goods constitute the major portion of retail activity in the CBD, this is a good proxy for use in evaluation of future flood plain activity. Asssuming that Davenport's share of commercial activity in the SMSA remains constant, the flood plain business activity can be related to the OBERS projections for earnings in wholesale and retail trade, finance,

insurance and real estate, and services. The annual rates of growth have been evaluated as 2.23 percent through 1986 and 3.06 percent through 2026. Again, a 50-year period of projection beginning in 1981 is used.

e. Base year benefits for wages and profits in the commercial sector are $103,500. The average annual value over the life of the project is $169,100. Therefore, the benefits due to growth are $65,600.

f. Similar calculations were carried out with regard to benefits in the public sector. Because public facilities and services cover a wide range of activities, benefits were taken to be directly proportional to population. Based on projections taken from the Official Iowa Population Projections, the expected rate of increase in population over the 1976-2026 period is 0.9 percent annually. The resulting future benefits are $34,400.

g. Future benefits accruing to non-flood plain business activities from protection of the water treatment plant are $23,100 (see paragraph 10c), for total benefits due to economic growth of $210,700.

1.16. Location Benefits. Construction of the project is expected to result in more efficient and economic use of certain vacant lands in the flood plain. Approximately 121 acres have been identified through recent aerial photography, field checks, and discussions with local officials as becoming enhanced for higher uses through the provision of flood protection. The total acreage is dispersed throughout the study area and occurs to a lesser extent in the more intensively developed eastern portion of the flood plain and to a greater extent in the western portion.

a. This land is currently zoned for industrial uses and potential users are actively being sought by city and local development organizations. Both city and private sources have expressed the opinion that the existing flood potential has been a primary deterrent to development of the land.

b. It is expected that the removal of the flood hazard and the flood insurance restrictions will substantially enhance the economic desirability of the land. Other advantages are industrial zoning and the availability of public services, including streets, sewers, and utilities. Existing rail service, river access, and proximity to an interstate highway exchange also provide advantages for development potential. These considerations are particularly attractive when other available lands are examined in this regard. A marketing brochure entitled "Land for Industrial Development, Davenport and Scott County, Iowa," provided by the Davenport Planning and Zoning Commission, lists 17 sites totaling 937 acres as available for industrial location. However, 11 sites totaling 639 acres do not have rail service, 6 sites totaling 334 acres have poor highway access, and 4 sites totaling 387 acres do not have full utility service. Only 1 site of 32 acres has rail and highway access and utility service comparable to the flood plain lands in question. None of the 17 sites have direct river access.

c. In addition to the economic advantages, there are important political considerations bearing on the relative attractiveness of the under-utilized flood plain land. As indicated previously, current White House policy favors the strengthening and redevelopment of urban centers. This same policy along with Congressionally mandated programs is designed to limit urban expansion into prime agricultural areas. The preservation of agricultural land is also of primary importance at the state, county, Bill presently under consideration by the legislature could have a major impact on future industrial siting. At the county and city levels, the productivity of agricultural land is an important consideration in zoning policy. The city of Blue Grass, Iowa, located approximately six miles from the downstream end of the project, recently adopted a zoning ordinance based on the corn suitability ratings of the soils. Muscatine County is currently conducting a soil survey in order to adopt a similar zoning procedure. Such actions, taken with intent to preserve agricultural lands, will have the overall effect of focusing development toward the in-filling of existing urban areas. Under such conditions, the lands considered here will become highly desirable.

d. The location benefits presented in the Phase I GDM were not evaluated by one of the methods sanctioned in ER 1105-2-351. Therefore, a departure from the Phase I methodology will be made at this point. Using recent sales data for lands considered to be comparable to the flood plain lands, the average value per acre is estimated to be $10,800 under current conditions. Again using recent sales of comparable lands, the estimated value with flood protection is $45,100 per acre; for a total increase in value of $34,300 per acre. The sales information was provided by the city of Davenport, based on the results of an independent appraisal conducted for the city.

e. Using recently published data (Bi-State, June 1979) regarding the supply of industrial land in the area, the rate at which land is made available, and the utilization rate, it is estimated that the total increase in value for the flood plain lands will be realized over a period of eight years. Because FIA regulations are relaxed to with project conditions when project construction is 50 percent complete, it was assumed that the utilization would begin in 1986 and continue through 1991.

f. Thus benefits will accrue at the rate of $518,100 per year (15.125 acres per year) for eight years. The process for estimating the annual benefits over the life of the project is analogous to that described for the water treatment plant. The resulting annual location benefits are $269,900. Because the land is presently developed with municipal services, no development costs are deducted from the benefits.

g. Some consideration to Executive Order 11988 must be given. Briefly stated, the objective of the order is to avoid, to the extent possible, any adverse impacts associated with the occupancy and modification of the flood plain and to avoid direct or indirect support of development in the flood plain wherever there is a practicable alternative.

ER 1165-2-26 provides requirements and procedures for implementation of the Executive Order. While the several points of the regulation will not be reiterated here, the pertinent aspects will be addressed.

h. The proposed action subject to evaluation under EO 11988 is the provision of local flood protection through construction of a levee. There is no practicable alternative to location in the flood plain. However, the project serves to reduce the hazard and risk associated with floods, and to minimize the impact of floods on human safety, health, and welfare. The order stresses the avoidance of direct or indirect support of development in the flood plain, which is the primary aspect to be addressed in this section.

i. By virtue of their location throughout the study area, amid existing development, certain lands will be enhanced in value and potential for development by the project. However, protection of these lands cannot be avoided if the project is to achieve its purpose. Except for these under-utilized parcels, the flood plain is already well developed. It is the protection of existing development which must be considered the most practicable alternative. This is in keeping with the spirit of the Executive Order. Additional discussion of EO 11988 can be found elsewhere in this report.

1.17. Social Effects. Under Executive Order 11988, the Corps of Engineers is required to provide leadership and take action to reduce the hazard and risk associated with floods and to minimize the impact of floods on human safety, health, and welfare. The local flood protection plan will fulfill these objectives not only through reducing physical damages to property, but by reducing the adverse impacts to those people least able to withstand the hardships imposed by floods.

a. Table F-1.13 displays basic economic and social characteristics for Davenport, based on data collected in 1979 by Polk (see references). Several conclusions immediately follow which need not be reiterated. However, the 200-year flood plain population represents a significant portion of the total city population, approximately 9.6 percent. But more significantly, it is apparent that the flood plain residents earn lower incomes and tend to be more elderly than the Davenport populations as a whole. This is evidenced by the fact that the average household income in the flood plain is 15 percent lower than the city-wide average, and that the percentage of households headed by retirees is 55 percent higher than the overall city rate. Moreover, 51 percent of the flood plain households have incomes which qualify for some form of public assistance. These are all indications of a higher dependence on social and public services and of a lesser ability to recover financially from the effects of flooding.

TABLE F-1.13

Economic and Social Characteristics

	Flood Plain	Non-Flood Plain	City Total	Ratio 1/
Population	~~9,048~~ ~2300	85,310	94,358	.096
Households (HH)	3,770	31,703	35,473	.106
Average HH Size	2.4	2.7	2.7	.889
Average HH Income	$14,800	$17,630	$17,330	.854
% HH w/Children	32.2	38.5	37.8	.852
% HH w/Retired Heads	31.9	19.3	20.6	1.55
% HH w/Low Income 2/	51.1	37.1	38.6	1.32
Total Housing Units	4,083	33,782	37,865	.108
Vacancy Rate, %	7.7	6.1	6.3	1.22

1/ Ratio of flood plain to entire city
2/ Defined as percent of households with heads whose income is at or below threshold levels for any of the various assistance programs.

(Source: R. L. Polk and Company)

b. The benefits credited to the project thus far have been limited to those directly measurable in pecuniary terms. It must be stressed that the project also affords important benefits which are not so readily measured. Such benefits are normally attributed to the social well-being account and do not appear in benefit-cost ratios. Many of the residents of the flood plain are dependent on public transportation and receive services and buy food within a limited distance from their homes. A severe flood would have tremendous impact on these people. During floods, much of the flood plain would be without fire protection and other essential services. And, as mentioned, those people most directly affected by flooding are those who are least able to endure and fully recover from flooding. As can be seen, the plan will substantially reduce the social hardships inflicted by flooding.

1.18. Summary of Flood Control Benefits. Estimated average annual benefits resulting from reduced flood damages are summarized in Table F-1.14. As stated in paragraph 18, the benefits presented are for direct benefits readily measurable in dollar amounts. Recognized beneficial impacts which are difficult to quantify in dollar terms do not appear in Table F-1.14. Nonetheless, unmeasured benefits are an important consideration, which, if included, would substantially enhance project justification beyond the benefit-cost ratio of 1.17 to 1. It should also be noted that some of the benefits included in the ratio may be significantly under valued. In particular, the detour benefits are based on the increased costs to the regular users of the Centennial Bridge incurred during periods of bridge closure. Not included, however, were the costs to users of other arteries affected by traffic problems, either by direct inundation or induced congestion. Also not included were the costs of productive time lost

because of delays. Another example of under valuation occurs with the water supply benefits. It is doubtful that the benefits presented here measure the true willingness to pay for an uninterrupted supply of water. This is especially true when it is considered that fire-fighting efforts would be severely curtailed.

In addition to providing considerable economic and social benefits, the plan makes other worthwhile contributions. The plan promotes energy efficiency, in that the mass transit routes are most concentrated in the core area. It is consistent with and enhances policies which are designed to strengthen urban core areas and retard growth into agricultural areas. And the plan will serve to secure and promote the success of other Federal and local investments in Davenport.

TABLE F-1.14

Davenport, Iowa
Project Benefits

	Amount	B/C
Existing Benefits		
Flood Damage Reduction	$1,058,100	
Detour Costs	5,900	
Water Supply 1/	148,400	
Emergency Costs	288,000	
SUBTOTAL	$1,499,900	.56
Redevelopment	1,015,400	
SUBTOTAL	$2,515,300	.94
Future Benefits		
Affluence	12,100	
Economic Growth	210,700	
Location	269,900	
TOTAL BENEFITS	$3,008,000	1.13
TOTAL COSTS FOR FLOOD CONTROL ONLY	$2,665,000	

1/ Includes $39,500 from Accumulation to the Base Year

1.19. Recreation Benefits. The project will substantially enhance recreational opportunities on Credit Island and will create new opportunities at the Crescent Park. The recreational features of the project are fully discussed in Appendix I.

a. The present number of annual activity days in the proposed project area are estimated at 190,000. Of these 190,000 activity days, 113,610 occur at LeClaire Park, 73,847 on Credit Island, and the reamianing 1,893 in the Crescent Park Sector. These estimates are based on recent observations and discussion with city park officials. However, the city makes no detailed count of these visits nor for the particular activities for which they occur.

b. The estimated number of recreational days, as shown in Table F-1.15, were estimated for three points over the life of the project, years 1, 30, and 40. These estimates were based on projected population figures and trends for the city of Davenport to the year 2025, projections of future trends concerning parks and recreation, and a report from the Cooperative Extension Service of Iowa State University entitled, "Outdoor Recreation in East Central Iowa: 1970 and 1985." The projections were also influenced and adjusted by observations and discussions with Davenport City Park Officials. It was assumed that a straight-line rate of growth would occur between the 1-, 30-, and 40-year points, and that the annual number of recreation days would remain constant over the remaining life of the project. The discounted annual equivalent value of enhanced recreational facilities is $608,200.

c. Due to the increasing population in the city of Davenport, as well as expanding amounts of leisure time, there has been an increasingly high demand for recreational facilities and programs. Some of the existing city facilities have reached their designed use capacity and projected future user demands could damage existing facilities. New facilities are needed to meet the increase future demand and to protect existing areas from over-use. Unfortunately, the high rate of inflation and budget cuts have significantly impacted the city's ability to establish new recreational facilities. A cost-sharing program between the Corps of Engineers and the city of Davenport to establish recreational facilities related to the proposed project would meet the present and future demand for more recreational facilities. In addition, the establishment of new facilities would protect existing facilities from over-use and degradation. These new facilities would also enhance the recreational experience obtained from existing city recreational facilities, due to less crowded conditions.

TABLE F-1.15

Value of Enhanced Recreational Development

Activity	Estimated 1/ Recreation Days in Year 40	Estimated Unit Value Per Recreation Day
Fishing	32,500	$ 2.50
Picnicking	37,400	2.00
Walking	13,000	2.00
Bicycling	6,400	2.00
Sightseeing	1,400	2.00
Boat Tours	2,000	6.70
Boating	7,400	2.00
Softball	18,400	2.00
Sports Spectating	10,000	1.00
Playgrounds	3,000	2.00
Play Fields	2,800	2.00
Pleasure Driving	8,400	2.00
Camping	7,000	2.00
Nature Walks and Photography	6,300	2.00
Outdoor Cultural Events	85,700	6.70
Total, Year 40	241,700	

Annual Value in Year 40	$ 901,900
Annual Equivalent Value of Increase over Current Usage	$ 7,000

1/ Total Estimated Visitations in Project Year 1 - 165,750, Year 30 - 208,300.

Note: Recreation day use values and benefits calculated in accordance with Water Resources Council Water and Related Land Resources, Principles and Standards for Planning, Federal Register/Vol. 44, No: 242/Friday, December 14, 1979/Table K--2, Pages 72, 963, and Revised Table K-3-1 (FY 1981) - conversion of Points to Dollar Values.

Note: Table F-1.16 which displays the projected 40-year number of annual recreational days has been broken into weekly, weekend, and weekday visits per activity (and sector). The season was based on a 16-week time frame of mid-May to mid-September. The size and the number of proposed facilities for each sector were then analyzed to determine if they could support the projected weekday, weekend, and weekly projected number of activity days in year 40. The weekday and weekend projected activity day use in year 40 of the project will not exceed the daily carrying capacity per each activity.

TABLE F-1.16

Seasonal Activity Days [1]

	Weekly [2] Activity Days	Weekend [3] Activity Days	Weekday [4] Activity Days
Downtown Sector			
Fishing	1,312.5	785.5	525
Picnicking	1,625	975.0	650
Walking	2,791.6	1,675	1,117
Bicycling	1,625	975.0	650
Outdoor Cultural Events	9,375	5,625	3,750
Sightseeing	687.5	412.5	275
Boat Tours [5]	333.3	200	133
Cresent Park Sector			
Picknicking	1,562.5	937.5	625
Fishing	750	450	300
Softball	2,875	1,725	1,150
Sports Spectators	1,250	750	500
Bicycling	1,562.5	937.5	625
Playgrounds	750	450	300
Walking	1,041.6	625	417
Playfields	687.5	413	275
Driving for Pleasure	2,750.0	1,650	1,100
Credit Island Sector			
Driving for Pleasure	3,500	2,100	1,400
Fishing	2,000	1,200	800
Picknicking	3,812.5	2,288	1,525
Camping	437.5	263	175
Bicycling	1,562.5	937.5	625
Walking	500	300	200
Nature Walk & Photography	1,041.5	625	417
Power Boating	937.5	563	375
Non-Power Boating	437.5	263	175

1/ Based on a 16-week recreation season
2/ Derived by dividing 16-week season into projected 40-year use figures displayed in Table F-12A
3/ 60% of the weekly seasonal rate (30% per weekend day)
4/ 40% of the weekly seasonal rate (8% per weekday)
5/ Based on 12-week recreational season

d. As previously mentioned, the redevelopment benefits from separable recreation expenditures are $88,500. The total annual benefits from the recreation features of the project are $696,700. Average annual costs attributed to the development and maintenance of the recreation facilities are $394,200. The incremental benefit-to-cost ratio is 1.8 to 1.0.

20. Summary of Project Benefits and Costs. Total project benefits and costs are summarized in Table F-17.

TABLE F-1.17

Davenport, Iowa
Project Benefits and Costs

	Amount	B/C
Total Existing and Future Benefits from Flood Control	$3,008,000	
Recreation Benefits	608,200	1.15
Redevelopment Benefits from Recreation Expenditures	88,500	
Total Project Benefits	$3,704,700	1.17
Total Project Costs	$3,154,000	

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US Department of Commerce, "Business Statistics, 1977," March 1978, Washington, DC.

US Department of Commerce, "1970 Census of Population," May 1972, Washington, DC.

US Water Resources Council, "OBERS Projections, Series E Population," Volume 5, April 1974, Washington, DC.

In addition to the references listed above, the following organizations provided information through personal interviews and unpublished reports.

Bi-State Metropolitan Planning Commission
City of Davenport, Planning and Zoning Commission
Downtown Davenport Development Corporation, Davenport, Iowa
Quad City Development Group, Rock Island, Illinois

INTERIOR DRAINAGE
DAVENPORT, IOWA

LOCAL FLOOD PROTECTION
PHASE II
GENERAL DESIGN MEMORANDUM

APPENDIX F
ECONOMICS ANALYSIS

SECTION 2

INTERIOR DRAINAGE ECONOMICS

INTRODUCTION

2.1. Section 2 of the Economics Appendix documents the economic studies performed for the interior drainage analysis. The mainline protection along the Mississippi River would block interior drainage outlets and would create problems if some method were not provided to discharge runoff flows through the levee-floodwall protection. An economic study is needed to analyze various alternative methods of solving the problem in an effort to help select the most cost effective design.

BACKGROUND INFORMATION

GENERAL

2.2. Blocked interior drainage would cause runoff to pond resulting in damage in the ponding areas. The economic study provides information needed to help in the process of designing facilities to reduce the damage to acceptable levels. Background information is given to introduce considerations that lead into the identification of the study area and corresponding pumping facilities.

THE STUDY AREA

2.3. The basic study area includes all areas associated with inundation and subsequent damage caused by interior drainage runoff. For the purpose of the economic analysis, the term "ponding area" is used to identify all areas that could be inundated under the worst conditions of interior drainage flooding. Three such ponding areas are identified by plate F-2, and designated as (1) the Credit Island (2) the Gaines Street, and (3) the Main Street ponding areas.

2.4. The "base condition" is a "without project" condition of the ponding area, characterized by existing conditions of land development, and "blocked gravity" conditions for the runoff flowing into the ponding area. "Blocked-gravity" ponding is caused by storm sewers being blocked by closed gatewells.

TYPES OF SERVICE

2.5. Runoff accumulating in the ponding area can be serviced in a number of ways: by outletting through a gravity outlet, by increasing ponding capacity of the area, by connecting two ponding areas, by providing pumping facilities, and by combinations of all of these.

PRIMARY PONDING

2.6. "Primary Ponding" is land owned in fee by the city and dedicated to ponding. Such lands are often provided excavated ponds to reduce real estate needs. The only primary ponding used in this study is the Credit Island Slough, a large backwater area of the Mississippi River that will be separated from the river by the project. Excavation by dredging is also planned for the slough to increase ponding capacity.

SECONDARY PONDING

2.7. "Secondary ponding" is ponding in streets, storm sewers, land depressions, etc. This ponding capacity is included in the hydraulic analysis of the effects of pumping.

PONDING ALTERNATIVES

2.8. Ponding alternatives were studied in an effort to serve the largest area possible with the smallest number of pumping stations.

The Corps study made efforts to design interceptor storm sewers to combine ponding areas and therefore reduce the number of pumping stations needed to serve the study area. The most effective combination would have been combining the Gaines Street Ponding Area with that of the Main Street Ponding Area. However, this was found infeasible because of the presence of a large existing city pressure sewer in Ripley Street. This existing sewer precludes construction of a new interceptor sewer needed to combine the two ponding areas.

BASE CONDITIONS

2.9 The category of land and corresponding Average Annual Damage (AAD) under base conditions is given in table F-2.1.

TABLE F-2.1

Average Annual Damage
Base Condition

Ponding Area	Average Annual Damages					
	Residential					
	Structure ($1,000)	Content ($1,000)	Commercial ($1,000)	Industrial ($1,000)	Public ($1,000)	Total
Credit Island	43.47	22.75	0.00	15.22	6.71	88.15
Gaines Street	3.97	2.05	6.78	50.79	18.05	81.64
Main Street	0.00	0.00	32.27	0.00	4.32	36.59
Totals	47.44	24.80	39.05	66.01	29.08	206.38

2.10. The number and value of structures existing in each ponding area is shown in table F-2.2.

TABLE F-2.2

Number and Value
of Structures

Ponding Area	Residential		Commercial		Industrial		Public	
	Number	Value ($1,000)	Number	Value ($1,000)	Number	Value ($1,000)	Number	Value ($1,000)
Credit Island	91	3185.0	0	0.0	21	5424.3	5	643.0
Gaines Street	90	2700.0	70	1673.0	6	6461.8	10	1166.0
Main Street	0	0.0	44	3819.2	0	0.0	6	205.8

AUTOMOBILE DAMAGE

2.11. In addition to the damageable structures and their contents, the Main Street ponding area has another potential source of damage. Due to the suddeness of interior drainage flooding, cars parked in parking lots could be damaged. There are 1,012 parking spaces subject to such flooding. Using 36 percent average occupancy, the AAD to parked cars was estimated at $7,000.

ECONOMIC ANALYSIS

INTRODUCTION

2.12. The objective of the economic analysis is to identify the most cost-effective measure for reducing the levels of ponding. The economic analysis is coordinated with the hydraulic analysis which furnishes pumping capacity versus ponding heights at various flooding event frequencies.

2.13. The economic analysis provides a monetary basis to consider in the selection of pumping capacities to be used for each ponding area. The analysis includes the effects of future growth as well as those of existing conditions of development.

METHODOLOGY

2.14. The intent of the methodology is to determine the benefit-cost relationships of a range of pumping capacities, such determinations to be made in accordance with land-use data for each ponding area. The analysis is used with a consideration of engineering, economic, environmental, and social factors in the selection of the best site location and plan. The consideration of existing and future development is made to derive benefits occurring for each alternative for the ponding areas identified.

2.15. Benefit from reduced ponding heights is credited to each pumping alternative and is the reduction in Average Annual Damage (AAD) in the ponding area. This damage reduction is computed by subtracting the AAD under pumping condition from the AAD under base conditions. Damage versus frequency relationships are used to compute the AAD's. The source of information used to derive relationships used is described in paragraphs that follow.

DATA COLLECTION

2.16. An approved interview form was used to collect data regarding commercial and public facilities. Information such as the value of plant, equipment and inventory, amounts of physical damage, and wage and business loss that would probably occur from various depths of flooding, was collected using interview procedures and correlated with onsite observations and measurements. Data is also collected regarding house location, value, type of residence (ranch, split level, with or without basement, etc.), and ground and floor elevation data.

SUDDEN INUNDATION

2.17. In addition to damage usually experienced from stream flows there is damage resulting from the sudden inundation typical of interior drainage flows. Due to short time duration of interior drainage floods, there is little warning to allow protective measures to be exercised.

2.18. One source of such damage are automobiles parked in parking lots. Data used to derive damage curves for these cars is based upon the following assumptions:

a. Damage starts when the parking space is inundated to a depth of 1.5 feet.

b. Between 1.5 feet and 2 feet of depth the floor upholstery and seats are damaged.

c. Between 2 feet and 3 feet of depth additional upholstery of the interior is damaged.

d. At 3 feet of depth water enters the engine via intake orfices.

2.19. Price data was obtained from local auto enterprises. The parking lots can be assumed to be nearly empty on Sundays and at night, and only partly occupied during daytime hours. Average occupancy was derived using weighted hourly occupancy figures for each day.

2.20. In addition to auto damage there is sudden inundation damage to other properties creating greater damage than that expected from the Mississippi River. This is due to absence of warning and the short time available to locate labor and materials needed for emergency means of protection. The damage curves derived for the main stream for residential, commercial, industrial, and public facilities were modified to reflect losses that would not have occurred had there been adequate warning to allow preventive measures to be used. Preventive measures include the moving of contents to higher level, sandbagging, evacuating damageable property, and flood-proofing of buildings.

DAMAGE CURVES

2.21. Elevation damage curves for each structure are derived using the data collected in the field as modified for sudden inundation, Damage curves for the various commercial, industrial, and public facilities are added together to produce total damage curves for each category for each ponding area. Likewise, residential damage curves for the various housing structures are added together to produce total residential damage curves for each ponding area. These curves represent the best information available from data collection techniques, and from experience and observations of past floodings of a similar nature and for a similar area.

2.22. Damage curve information for various categories is tabulated in table 2.3 for each ponding area studied.

TABLE F-2.3

Damage Curve Information
for
The Ponding Areas

Elevation (NGVD)	Credit Island Structural ($1,000)	Credit Island Content ($1,000)	Gaines Street Structural ($1,000)	Gaines Street Content ($1,000)	Main Street Structural ($1,000)	Main Street Content ($1,000)
Residential Damage						
556.0	0.0	0.0			0.0	0.0
557.0	38.7	20.0	0.0	0.0	0.0	0.0
558.0	92.5	47.9	0.4	0.3	0.0	0.0
559.0	173.1	89.2	1.4	0.7	0.0	0.0
560.0	267.5	137.7	14.0	7.2	0.0	0.0
561.0	404.1	208.1	22.7	11.7	0.0	0.0
562.0	594.0	366.0	36.0	18.5	0.0	0.0
563.0	701.6	361.5	59.1	30.4	0.0	0.0
Commercial Damage						
	Total Curve		Total Curve		Total Curve	
556.0	0.0					
557.0	0.0		0.0			
558.0	0.0		0.5		0.0	
559.0	0.0		3.0		30.2	
560.0	0.0		18.5		850.4	
561.0	0.0		40.4		149.6	
562.0	0.0		70.5		263.9	
563.0	0.0		121.6		411.5	
Industrial Damage						
556.0					0.0	
557.0					0.0	
558.0	0.0				0.0	
559.0	7.4		0.0		0.0	
560.0	60.6		91.9		0.0	
561.0	220.1		283.0		0.0	
562.0	681.0		621.1		0.0	
563.0	962.1		1010.6		0.0	
Public Damage						
556.0						
557.0						
558.0	0.0				0.0	
559.0	14.7		0.0		2.2	
560.0	34.9		44.2		7.0	
561.0	102.9		110.1		14.2	
562.0	198.3		199.1		53.1	
563.0	253.6		307.1		111.1	
Parking Lot Autos						
560.0	0.0		0.0		0.0	
561.0	0.0		0.0		53.8	
562.0	0.0		0.0		104.2	
563.0	0.0		0.0		226.2	

2.23. Damage frequency curves for the base condition are shown by table F-2.4 for each ponding area.

TABLE F-2.4

Damage-Frequency Relationships
for Base Conditions
(For the Ponding Areas)

	Credit Island		Gaines Street		Main Street	
Elevation (NGVD)	Damage ($1,000)	Frequency (%)	Damage ($1,000)	Frequency (%)	Damage ($1,000)	Frequency (%)
556.0	0.0	32.0				
557.0	58.7	21.0	0.0	30.0		
558.0	140.9	14.0	1.2	24.0	0.0	25.0
559.0	284.4	9.0	5.1	17.5	32.4	17.0
560.0	500.8	5.2	175.9	11.5	92.0	10.5
561.0	935.2	2.0	467.9	6.6	231.0	5.2
562.0	1779.5	0.3	945.4	2.5	447.1	1.7
563.0	2278.9	0.1	1528.9	0.1	805.4	0.1

2.24. The "period of record" analysis described by the Hydraulic appendix was used to establish frequency of flooding. Frequency relationships combined with damage curves provided the basis of the economic analysis of the effects of water inundation and associated damage.

PUMPING ALTERNATIVES

2.25. To establish a basis for computing return on investment, various pumping alternatives were used to establish benefit variations corresponding with variations in cost. Therefore, pumping alternatives to the base conditions represented by plant facilities of various sizes were studied, ranging from lower to higher pumping capacities. Such a study was conducted for each ponding area.

DAMAGE COMPUTATIONS

INUNDATION DAMAGE

2.26. Most properties damaged by water inundation must be restored to near their original condition or replaced by new property in order to establish conditions as they were before the flooding. It is this cost of restoration that establishes the monetary basis for damage.

2.27. Inundation damage is the primary basis for establishing damage "with project" as compared to damage "without project." Computing the annual value of inundation damage is an important step in the economic analysis and is performed as described in paragraphs that follow.

2.28. Computation procedures were used to integrate damage curves with interior drainage frequency curves developed for base conditions to produce average annual damage estimates for each ponding area. Hydraulics "Appendix A" describes methodology used to develop frequency curves. These were combined with elevation-damage curves (representing damageable properties of the ponding area) to produce the damage frequency curves used in this economic appendix to compute average annual damage. Average annual damage represents the damage expected to occur each year, on the average, due to damageable property being inundated by various flooding events. A similar analysis is also made for various pumping conditions.

FUTURE GROWTH

2.29. Inundation damage can also occur in future years when (1) existing properties of the ponding area experience an increase in value or damage potential, or (2) new properties occupy the ponding area and thereby increase the number of damageable units. The damage frequency analysis can be repeated for data representing future conditions of development, the resultant converted through discounting procedures to represent the annual equivalent at base year. The analysis used for new units is represented by a consideration of increased values of land, and is also discounted to the base year.

2.30. The increase of damage resulting from projections of future growth is added to the damage to properties under existing development conditions to result in total AAD from existing and future development conditions.

2.31. Future growth for damage to existing properties often results from increased value of existing units and their contents. For residential properties, the increase in the value of contents represents the growth of value due to the increased affluence of residents. For business properties, increase in value of facilities and contents resultsfrom internal modernization and expansion which increases profits and employment. Projections for these occurrences are based upon reliable sources that describe existing trends, and give methods that can be used to project the trends.

2.32. Future growth from new units projected to occupy the ponding area in the future can be analyzed through a consideration of the increase in the value of certain unoccupied lands as a result of the project making them more attractive for future development. Increases in land values are based upon reliable sources that describe how values of similar lands increase as changes from "flood prone" to "flood free" conditions occur. Increases in the values of land can be related to increases in annual income; a change that can be counted as a future benefit.

2.33. For the purpose of computing future growth, the base year (the year of project completion) was assumed to be 1986, and the interest rate used for discounting procedures was 7-5/8 percent.

FUTURE RESIDENTIAL DAMAGE

2.34. Growth in the value of residential contents is considered to occur from the increased affluence of residents. This growth is assumed to

increase in proportion to per capita personal income. A compound growth rate of 2.80 percent was computed using OBERS series E projections for the SMSA that includes Davenport, Iowa. Growth of residential contents is limited to 75 percent of the value of the residential structure. Using this procedure, maximum value was computed to occur after 31 years. The resulting projections by decade are displayed in tables F-2.5, F-2.6, and F-2.7.

FUTURE COMMERCIAL AND INDUSTRIAL DAMAGE

2.35. The basis of economic growth computed for the interior drainage study is the same as that given for the project analysis of Mississippi River floods as described in section 1 of this appendix. An annual rate of increase of 2.23 percent through 1986 and 3.06 percent through 2026 was used for the commercial sector, and 2.75 percent through 2026 was used for the industrial sector. The resulting projections are shown by decade in tables F-2.5, F-2.6, and F-2.7.

FUTURE PUBLIC DAMAGE

2.36. Basis for the computations of future public damage was also the same as that used in the Mississippi River analysis given by section 1 of this appendix. Increase of damage was assumed to be directly proportional to population or 0.9 percent annually through year 2026. The resulting projections are also shown in tables F-2.5, F-2.6, and F-2.7.

TABLE F-2.5

Projected Damages
Credit Island
Ponding Area

Year	Residential Structure ($1,000)	Residential Content ($1,000)	Commercial ($1,000)	Industrial ($1,000)	Public ($1,000)	Totals ($1,000)
1981	43.47	22.75	0.00	15.22	6.71	88.15
1986	43.47	25.77	0.00	17.43	7.00	93.67
1996	43.47	33.97	0.00	22.86	7.66	107.96
2006	43.47	44.75	0.00	29.98	8.37	126.57
2016	43.47	54.19	0.00	36.25	8.93	142.84
2026	43.47	54.19	0.00	51.60	10.03	159.29
2031	43.47	54.19	0.00	59.08	10.50	167.24
2086	43.47	54.19	0.00	59.08	10.50	167.24
			Discounted at 7-5/8%			
1981	43.47	22.75	0.00	15.22	6.71	88.15
1986	43.47	25.77	0.00	17.43	7.00	93.67
2086	43.47	37.62	0.00	27.99	7.88	116.96

Note: (1) Residential projections are at 2.8 percent rate to year 2013 (27 years).

(2) Industrial projections are at 2.75 percent rate and public projections at 0.90 percent rate to year 2031 (45 years).

TABLE F-2.6

Projected Damages
Gaines Street
Ponding Area

Year	Residential Structure ($1,000)	Residential Content ($1,000)	Commercial ($1,000)	Industrial ($1,000)	Public ($1,000)	Totals ($1,000)
1982	3.97	2.05	6.78	50.79	18.05	81.64
1986	3.97	2.36	7.58	58.04	18.88	90.83
1996	3.97	3.10	10.23	76.12	20.65	114.02
2006	3.97	4.10	13.85	99.86	22.58	144.36
2013	3.97	4.97	17.10	120.75	24.05	170.84
2016	3.97	4.97	18.71	131.00	24.70	183.35
2026	3.97	4.97	25.31	171.81	27.02	233.08
2031	3.97	4.97	29.41	196.77	28.25	263.37
2086	3.97	4.97	29.41	196.77	28.25	263.37
			Discounted at 7-5/8%			
1982	3.97	2.05	6.78	50.79	18.05	81.64
1986	3.97	2.36	7.58	58.04	18.88	90.83
2086	3.97	3.40	13.12	93.46	21.26	135.21

Note: (1) Residential projections are at 2.8 percent rate to year 2013 (27 years).

(2) Commercial projections at 3.06 percent, industrial at 2.75 percent, and public at 0.90 percent rate to year 2031 (45 years).

TABLE F-2.7

Projected Damages
Main Street
Ponding Area

Year	Residential Structure ($1,000	Residential Content ($1,000)	Commercial ($1,000)	Industrial ($1,000)	Public ($1,000)	Parking Lot ($1,000)	Total ($1,000)
1982	0.00	0.00	32.27	0.00	4.32	7.00	43.59
1986	0.00	0.00	35.93	0.00	4.52	7.00	47.45
1996	0.00	0.00	48.58	0.00	4.95	7.00	60.53
2006	0.00	0.00	65.65	0.00	5.40	7.00	78.05
2013	0.00	0.00	81.09	0.00	5.74	7.00	93.83
2016	0.00	0.00	88.75	0.00	5.90	7.00	101.65
2026	0.00	0.00	119.98	0.00	6.45	7.00	133.43
2031	0.00	0.00	139.49	0.00	6.76	7.00	153.25
2086	0.00	0.00	139.49	0.00	6.76	7.00	153.25
			Discounted at 7-5/8%				
1982	0.00	0.00	32.27	0.00	4.32	7.00	43.59
1986	0.00	0.00	35.93	0.00	4.52	7.00	47.45
2086	0.00	0.00	62.39	0.00	5.09	7.00	74.48

Note: Commercial projections at 3.06 percent rate and public projections at 0.90 percent rate to year 2031 (45 years).

BENEFITS

EXISTING BENEFITS

2.37. Most benefits are related to the reduction of average annual damages given by water inundation, and are determined by subtracting average annual damage under pumping conditions from average annual damage under base conditions. An example of this process is given by table F-2.8 for the Credit Island Ponding Area.

TABLE F-2.8

Average Annual Benefits
Under
Existing Conditions
for
The Credit Island Ponding Area

Pumping Capacity (GPM)	Residual AAD ($1,000)	AAB ($1,000)
Base Conditions	88.15	
10,000	39.91	48.24
20,000	26.18	61.97
35,000	15.70	72.45
45,000	8.62	79.53
90,000	2.14	86.01
112,000	0.00	88.15
135,000	0.00	88.15

FUTURE GROWTH BENEFITS

2.38. Future growth benefits are likewise related to the reduction of average annual damage resulting from water inundation. This type of damage is greater in the future because of the increased value of damageable properties. Rates of growth have been discussed previously in this appendix. The computation of future Average Annual Benefits (AAB) from discounted future residual damages is shown by table F-2.9.

TABLE F-2.9

Future Growth Benefits

Pumping Capacity (GPM)	Future Conditions Residual Average Annual Damage ($1,000)	Annual Inundation Benefits ($1,000)
Credit Island Ponding Area		
Base Conditions	116.96	
10,000	52.96	64.00
20,000	34.74	82.02
35,000	20.83	96.13
45,000	11.62	105.34
90,000	2.84	114.12
112,000	0.00	116.96
135,000	0.00	116.96

TABLE F-2.9 (Cont'd)

Gaines Street Ponding Area

Base Conditions	135.21	
5,000	95.55	39.66
10,000	74.98	60.23
20,000	44.82	90.39
40,000	10.45	124.76
60,000	1.42	133.79
75,000	0.00	135.21
80,000	0.00	135.21

Main Street Ponding Area

Base Conditions	74.48	
5,000	44.94	29.54
10,000	15.19	59.29
20,000	1.46	73.02
35,000	0.00	74.48

LOCATION BENEFITS

2.39. Open lands of each ponding area were studied from the standpoint of possible increase in land value resulting from the use of pumping facilities. The analysis is based upon the assumption that future changes in the value of land will be a measure of increased future economic return and therefore a benefit to the economy. Therefore, the annual equivalents of these increases represent the future annual increase in benefit resulting from the project. All future development was assumed to take place between the 1 percent pond level for each alternative and the one percent level for base conditions. Reference is made to page F-23 of section 1 of this appendix and the caption "Location Benefits." The rationale for land values used in the section was assumed to be also applicable to the interior drainage analysis, with average land values of $10,800 per acre used for lands under current conditions as compared to average land values of $45,100 per acres used for lands under protected conditions; for an increase in value of $34,300 per acre. Table F-2.10 shows the total increase in land value computed for each ponding area and the resulting location benefit.

TABLE F-2.10

Location Benefits

Pumping Capacity (GPM)	Acres "Pump" to "No Pump" 1% Levels	Increase in Value per Acre ($1,000)	Total Increase in Land Value ($1,000)	Location Benefit ($1,000)
		Credit Island Ponding Area		
10,000	9	34.3	308.7	73.5
20,000	23	34.3	788.9	60.2
35,000	30	34.3	1029.0	78.5
45,000	36	34.3	1234.8	94.2
90,000	53	34.3	1817.9	138.7
100,000	53	34.3	1817.9	138.7
		Gaines Stret Ponding Area		
5,000	5	34.3	171.5	13.1
10,000	9	34.3	308.7	23.6
20,000	18	34.3	617.4	47.1
40,000	38	34.3	1303.4	99.5
60,000	38	34.3	1303.4	99.5
90,000	38	34.3	1303.4	99.5

Footnote: No location benefits are applicable to the Main Street ponding area.

2.40. Guidance furnished by ER 1105-2-300 indicates that changes in market value can be used to measure location benefits. The ER also indicates that the market anticipates the effects of time lapse before particular land parcels are expected to develop and the assumption of instantaneous value increase at base year can be used. Therefore, annual location benefits are computed by using the appropriate capital recovery factor for 7-5/8 percent interest rate.

OTHER BENEFITS

2.41. Benefits usually considered for a levee project such as "Emergency," "Advanced Replacement," and "Flood Insurance" benefits are not considered applicable to an interior drainage analysis.

OPTIMIZATION

INTRODUCTION

2.42. The optimization analysis is based upon total benefits versus offsetting project costs for each alternative. Benefits and costs are based upon July 1981 prices. Project benefits include inundation reduction benefits representing existing land use, future benefits projected for existing units, and location benefits derived from increased land values for existing unoccuppied lands. Project costs include all expenditures associated with each pumping facility. Interest during construction is based upon the assumption that it will take one year to construct a pumping project. Pumping station costs include the initial expenditure for pump plant construction, cost allowances for future pump replacements, annual maintenance and operation, and annual energy costs. Pump replacement costs include replacement of old pumps and motors with new, such costs are assumed to occur every 33 years. Benefits and corresponding costs are used for the project alternatives of each ponding area to furnish a range of net benefits to consider.

NET BENEFITS

2.43. Net benefits are calculated by subtracting annual costs from annual benefits. The facility with the pumping capacity that represents the greatest net benefits is considered to be the optimum design, or the design that will give the greatest monetary return from monetary expenditures over the period of analysis (100 years of project life).

2.44. The net benefit analysis for the Credit Island Ponding Area included computed results for seven different pump sizes to provide plotting points for a curve that describes the variation of net benefits with pump size.

2.45. The analysis also provided results for seven pump sizes for the Gaines Street and four pump sizes for the Main Street Ponding Area. The derivation of these plotting points for each ponding area is shown by tables F-2.11, F-2.12, and F-2.13.

RESULTS

2.46. Plate F-3 represents the net benefit versus pumping capacity curves that were generated through use of the plotting points of tables F-2.11, F-2.12, and F-2.13. The economic optimum capacity for a pumping facility serving the Credit Island Ponding Area is 100,000 GPM. Likewise, the economic optimum capacity is 43,000 GPM for the Gaines Street Ponding Area and 12,000 GPM for the Main Street Ponding Area.

TABLE F-2.11
Pump Optimization - Credit Island Ponding Area
Davenport, Iowa

Pumping Station Capacity G.P.M.	Residual Average Annual Damages ($1,000)	Average Annual Inundation Benefits ($1,000)	Location Benefits ($1,000)	Total Benefits ($1,000)	Pumping Station Cost ($1,000)	Interest During Construction ($1,000)	Total Pumping Station Cost ($1,000)	Annual Pumping Cost ($1,000)	Annual Pump Replacement Cost ($1,000)	Annual Energy Demand ($1,000)	Annual Operation And Maintenance ($1,000)	Total Annual Cost ($1,000)	Net Benefit ($1,000)	Benefit Cost Ratio
Base Conditions	116.96	0	0	0								0		
10,000	52.96	64.00	23.50	87.50	225.00	8.58	233.58	17.82	0.76	0.30	3.30	22.18	65.32	3.9
20,000	34.74	82.22	60.20	142.22	445.00	16.96	461.96	35.25	0.76	0.40	3.30	39.71	102.71	3.6
35,000	20.83	96.13	78.50	174.63	600.00	22.87	622.87	47.52	0.93	0.60	3.30	52.35	122.28	3.3
45,000	11.62	105.34	94.20	199.54	650.00	24.78	674.78	51.49	1.05	0.80	3.30	56.64	142.90	3.5
90,000	2.84	114.12	138.70	252.82	900.00	34.31	934.31	71.29	1.57	1.75	2.80	77.41	175.41	3.3
112,000	0.00	116.96	138.70	255.66	964.00	36.74	1000.75	76.36	1.84	2.00	2.80	82.90	172.74	3.1
135,000	0.00	116.96	138.70	255.66	1150.00	43.84	1193.84	91.09	2.11	2.10	2.70	98.00	157.66	2.6

TABLE F-2.12
Pump Optimization - Gaines Street Ponding Area
Davenport, Iowa

Pumping Station Capacity G.P.M.	Residual Average Annual Damages ($1,000)	Average Annual Inundation Benefits ($1,000)	Location Benefits ($1,000)	Total Benefits ($1,000)	Pumping Station Cost ($1,000)	Interest During Construction ($1,000)	Total Pumping Station Cost ($1,000)	Annual Pumping Cost ($1,000)	Annual Pump Replacement Cost ($1,000)	Annual Energy Demand ($1,000)	Annual Operation And Maintenance ($1,000)	Total Annual Cost ($1,000)	Net Benefit ($1,000)	Benefit Cost Ratio
Base Conditions	135.21	0	0	0								0		
5,000	95.55	39.66	13.10	52.76	110.00	4.19	114.19	8.71	0.57	0.20	0.80	10.28	42.48	5.1
10,000	74.98	60.23	23.60	83.83	225.00	8.58	233.58	17.82	0.76	0.30	0.80	19.68	64.15	4.3
20,000	44.82	90.39	47.10	137.49	445.00	16.96	461.96	35.25	0.76	0.40	0.80	37.21	100.28	3.7
40,000	10.45	124.76	99.50	224.26	620.00	23.63	643.63	49.11	0.99	0.70	1.30	52.10	172.16	4.3
60,000	1.42	133.79	99.50	233.29	730.00	27.83	757.83	57.82	1.22	1.00	1.30	61.34	171.43	3.8
75,000	0.00	135.21	99.50	234.71	815.00	31.07	846.07	64.55	1.40	1.38	1.40	68.73	165.98	3.4
90,000	0.00	135.21	99.50	234.71	900.00	34.31	934.31	71.29	1.57	1.75	1.50	76.11	158.60	3.1

TABLE F-2.13
Pump Optimization - Main Street Ponding Area
Davenport, Iowa

Pumping Station Capacity G.P.M.	Residual Average Annual Damages ($1,000)	Average Annual Inundation Benefits ($1,000)	Location Benefits ($1,000)	Total Benefits ($1,000)	Pumping Station Cost ($1,000)	Interest During Construction ($1,000)	Total Pumping Station Cost ($1,000)	Annual Pumping Cost ($1,000)	Annual Pump Replacement Cost ($1,000)	Annual Energy Demand ($1,000)	Annual Operation And Maintenance ($1,000)	Total Annual Cost ($1,000)	Net Benefit ($1,000)	Benefit Cost Ratio
Base Conditions	74.48	0	0	0								0		
5,000	44.94	29.54	0.00	29.54	110.00	4.19	114.19	8.71	0.57	0.20	0.80	10.28	19.26	2.9
10,000	15.19	59.29	0.00	59.29	225.00	8.58	233.58	17.82	0.76	0.30	0.80	19.68	39.61	3.0
20,000	0.92	73.56	0.00	73.56	445.00	16.96	461.96	35.25	0.76	0.40	0.80	37.21	36.35	2.0
35,000	0.00	74.48	0.00	74.48	600.00	22.87	622.87	45.47	0.76	0.40	0.80	47.60	25.28	1.6

THE SELECTED PROJECT

FEASIBLE ALTERNATIVES

2.47. The optimization tables indicated a number of capacities that have B/C ratios greater than 1. These are shown in table F-2.14. Table F-2.14 also indicates the apparent optimum capacity as shown by plate F-3 and corresponding net benefits and B/C ratios.

TABLE F-2.14

Comparison of Pump Capacities
Using
Optimization Analysis

Ponding Area	Pump With B/C = 1.0 (GPM)	Economical Optimum Size	Net Benefit ($1,000)	B/C Ratio
Credit Island	10,000 through 135,000	100,000	176.0	3.1
Gaines Street	5,000 through 90,000	43,000	176.0	4.3
Main Street	5,000 through 35,000	12,000	39.8	2.8

THE IMPACT TRAPEZOID ANALYSIS

2.48. The Hydraulic Appendix A describes an "Impact Trapezoid" analysis which defines constraints to consider in the selection of pumping capacity for design purposes. Table F-2.15 lists the range of pumping capacities that the analysis has indicated as being acceptable. The table also lists the most acceptable capacity and corresponding net benefit and B/C ratio.

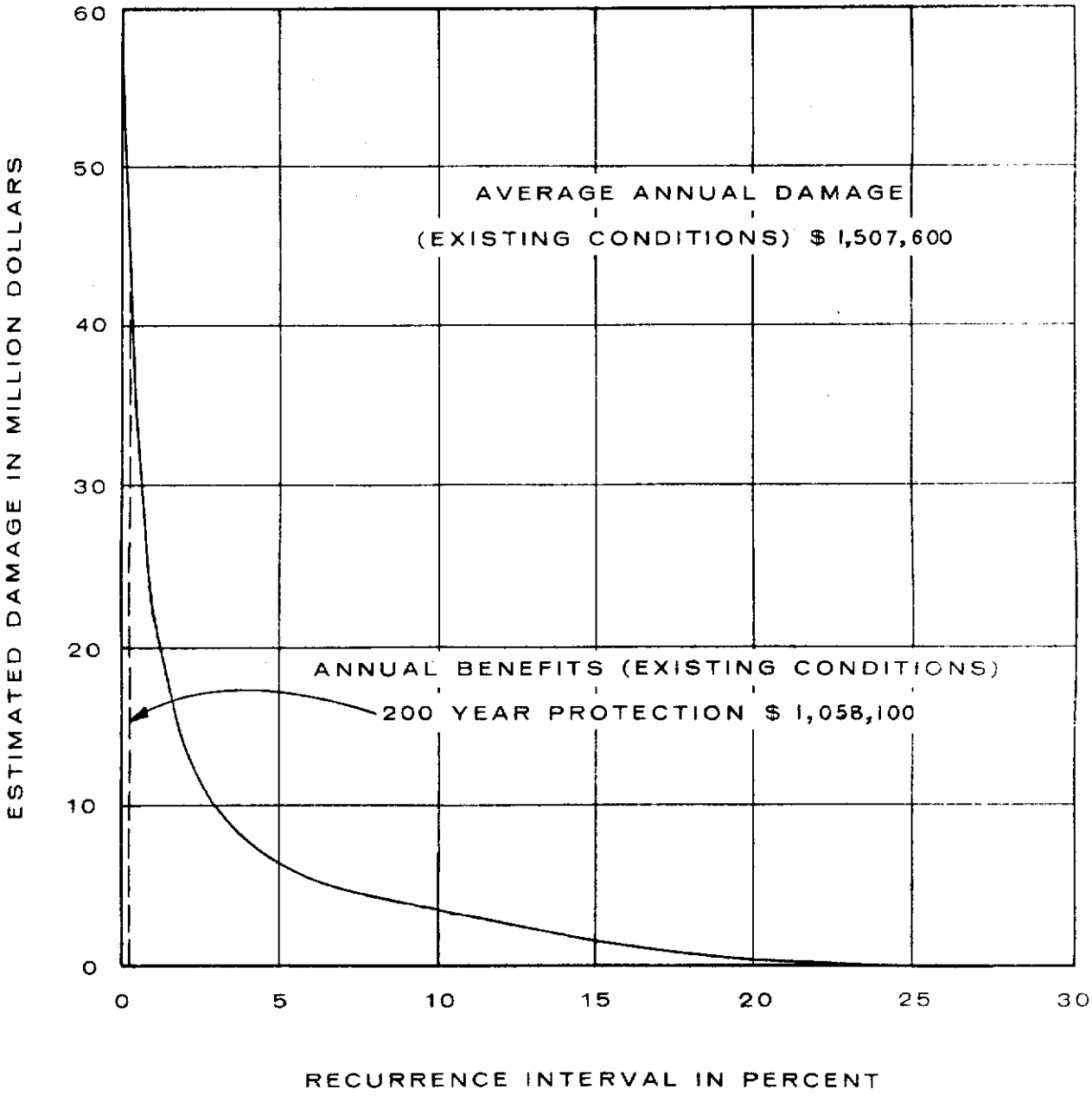
TABLE F-2.15

Pumping Capacities
Indicated by
Impact Trapezoid

Ponding Area	Acceptable Pumping Capacity (GPM)	Most Acceptable Pumping Capacity (GPM)	Net Benefit ($1,000)	B/C Ratio
Credit Island	29,000 to 125,000	72,000	164.0	3.4
Gaines Street	1,700 to 61,000	35,000	155.0	4.2
Main Street	3,600 to 10,000	10,000	39.6	3.0

CONCLUSIONS

2.49. The economic study documented in this appendix presented information that is considered to be of use in the selection of pump station design capacity. The pump station with optimum financial return may be considered important if other constraints do not exist. The impact trapezoid analysis uses limitations such as runoff return interval, drawdown time, and seepage rate which are also considered to be important. Indications of most acceptable capacities shown in table F-2.15 compare favorably with the optimum capacities shown in table F-2.14.



MISSISSIPPI RIVER
DAVENPORT, IOWA
DAMAGE VS. FREQUENCY

PLATE F-1

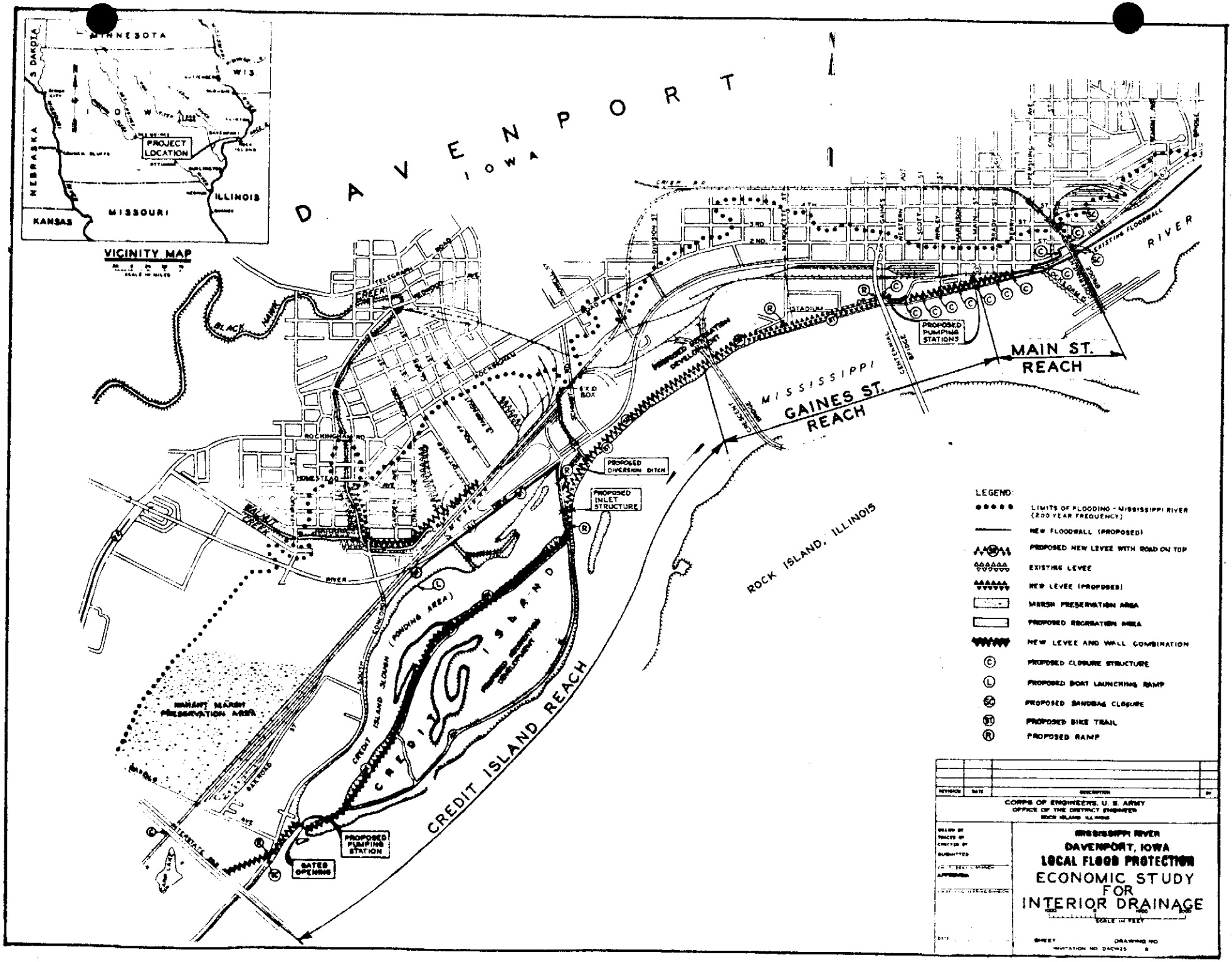

VICINITY MAP
PROJECT LOCATION
MINNESOTA
NEBRASKA
KANSAS
MISSOURI
ILLINOIS
DAVENPORT
IOWA
MISSISSIPPI RIVER
BLACK HAWK
WALNUT CREEK
MAIN ST. REACH
GAINES ST. REACH
CREDIT ISLAND REACH
ROCK ISLAND, ILLINOIS
CREDIT ISLAND (PONDING AREA)
PROPOSED PUMPING STATIONS
PROPOSED DIVERSION DITCH
PROPOSED INLET STRUCTURE
PROPOSED PUMPING STATION
GATED OPENING
MARSH PRESERVATION AREA
LEGEND:
LIMITS OF FLOODING - MISSISSIPPI RIVER (200 YEAR FREQUENCY)
NEW FLOODWALL (PROPOSED)
PROPOSED NEW LEVEE WITH ROAD ON TOP
EXISTING LEVEE
NEW LEVEE (PROPOSED)
MARSH PRESERVATION AREA
PROPOSED RECREATION AREA
NEW LEVEE AND WALL COMBINATION
PROPOSED CLOSURE STRUCTURE
PROPOSED BOAT LAUNCHING RAMP
PROPOSED SANDBAG CLOSURE
PROPOSED BIKE TRAIL
PROPOSED RAMP
CORPS OF ENGINEERS, U.S. ARMY
OFFICE OF THE DISTRICT ENGINEER
ROCK ISLAND, ILLINOIS
MISSISSIPPI RIVER
DAVENPORT, IOWA
LOCAL FLOOD PROTECTION
ECONOMIC STUDY
FOR
INTERIOR DRAINAGE
SCALE IN FEET

PLATE F-2

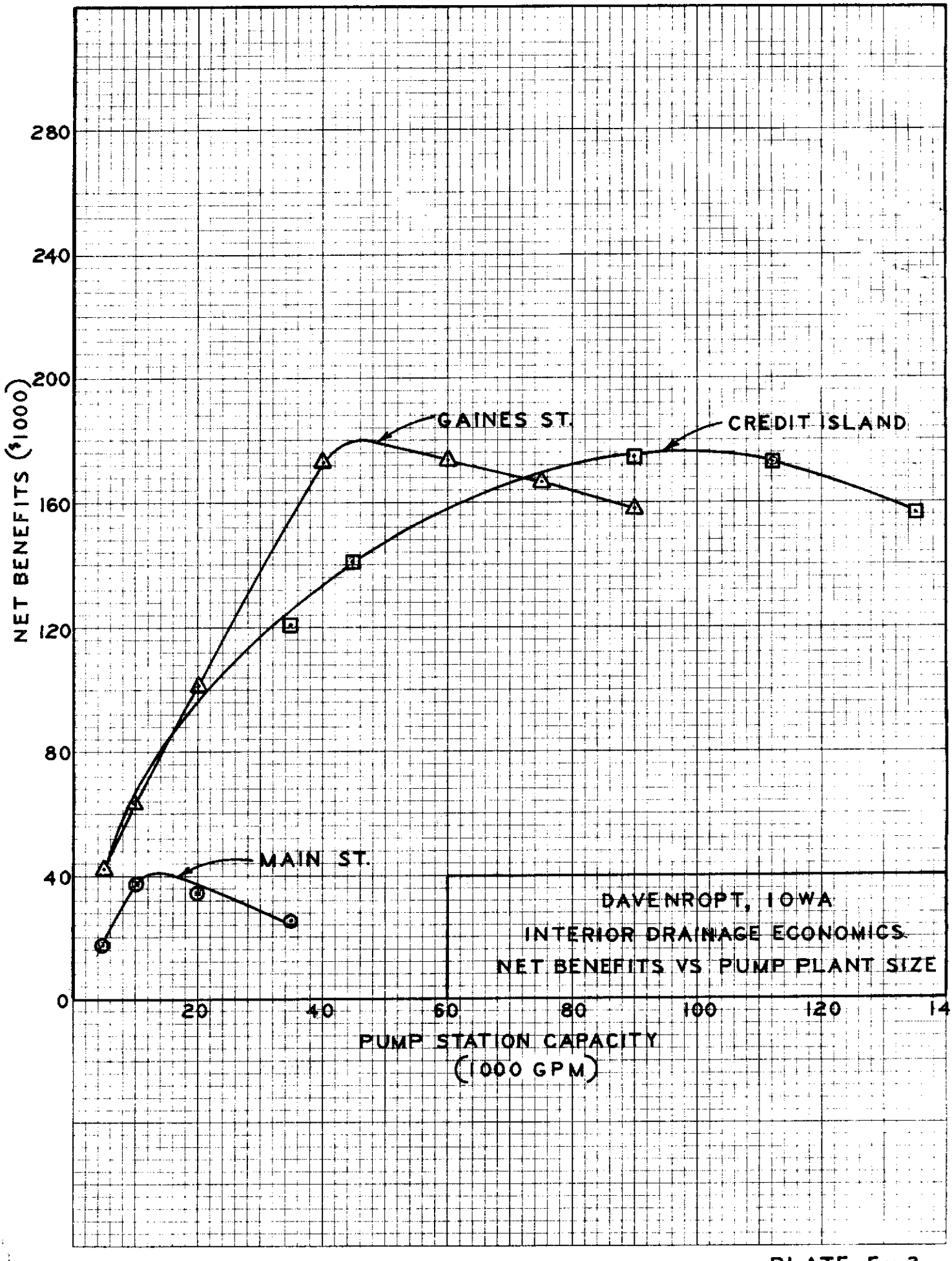

NET BENEFITS ($1000)
280
240
200
160
120
80
40
0
GAINES ST.
CREDIT ISLAND
MAIN ST.
20
40
60
80
100
120
140
PUMP STATION CAPACITY
(1000 GPM)
DAVENROPT, IOWA
INTERIOR DRAINAGE ECONOMICS
NET BENEFITS VS PUMP PLANT SIZE

PLATE F-3

MISSISSIPPI RIVER
DAVENPORT, IOWA

LOCAL FLOOD PROTECTION
PHASE II
GENERAL DESIGN MEMORANDUM

APENDIX G
RELOCATIONS
RAILROADS, HIGHWAYS, AND UTILITIES
(REVISED)

US ARMY ENGINEER DISTRICT, ROCK ISLAND
CORPS OF ENGINEERS
CLOCK TOWER BUILDING
ROCK ISLAND, ILLINOIS 61201

MISSISSIPPI RIVER
DAVENPORT, IOWA

LOCAL FLOOD PROTECTION
PHASE II

GENERAL DESIGN MEMORANDUM
APPENDIX G

RELOCATIONS
RAILROADS, HIGHWAYS, AND UTILITIES

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EXHIBIT A, CORRESPONDENCE

Letter dated 1 Feb 77 to Chicago, Milwaukee, St. Paul, and Pacific Railroad Company
Letter dated 22 Dec 76 to C.M.St.P.&P. RR.
Letter dated 8 Dec 76 to C.M.St.P.&P. RR.
Letter dated 16 Jan 76 to C.M.St.P.&P. RR.

MISSISSIPPI RIVER
DAVENPORT, IOWA

LOCAL FLOOD PROTECTION
PHASE II

GENERAL DESIGN MEMORANDUM
APPENDIX G
RELOCATIONS
RAILROADS, HIGHWAYS, AND UTILITIES

RAILROAD RELOCATIONS

1. General. The proposed flood protection plan for Davenport, Iowa, will affect the facilities of three railroads, namely: (1) the Chicago, Milwaukee, St. Paul and Pacific Railroad Company; (2) the Chicago, Rock Island, and Pacific Railroad; and (3) the Davenport, Rock Island, and North Western Railway Company.

2. Existing Railroad Facilities.

a. The Chicago, Milwaukee, St. Paul and Pacific Railroad operates a main line between Kansas City, Missouri, and Minneapolis, Minnesota, including a double-track section through Davenport, Iowa, which crosses the proposed alignment of the Davenport flood protection at the east limits of the project near the Government bridge.

b. The Chicago, Rock Island, and Pacific Railroad operates a main line double-track through Davenport. This segment of trackage is part of their primary east-west line between Chicago, Illinois, and the western states of Colorado and New Mexico. Their tracks cross the proposed alignment of the Davenport flood protection at the east and west limits of the project.

(1) On the east side of the project, the railroad embankment forms the tie-off as shown on Plate 21.

(2) The railroad crosses under the I-280 Bridge on the west side of the project as shown on Plate 41.

c. The Davenport, Rock Island, and North Western Railway operates a main line track from Davenport to Clinton, Iowa. This track crosses the line of protection at the water treatment plant as shown on Plate 56.

3. Proposed Railroad Alterations.

a. Chicago, Milwaukee, St. Paul, and Pacific Railroad facilities are to be altered or modified as follows:

(1) On the east side of the project near the foot of Pershing Street a double-track closure will be provided where the C.M.St.P.&P. Railroad tracks cross under the Government Bridge and a single-track closure is required for their spur track near the Government Bridge. The spur track closure will actually be part of the large River Drive Closure. These railroad closures are shown in detail on Plates 42 and 47. The opening for the double track closure will be 32 feet wide and 14 feet high, from sill to top of T-wall. The wall in this reach will have 4 feet of freeboard. A concrete toe wall and steel sheet piling will provide an impervious barrier to seepage through the free-draining railroad ballast at the double-track closure. A concrete wall down to bedrock will provide cutoff under the spur track closure. The base-of-rail elevation of the tracks at the double track closure is 559.7. The design flood elevation and design levee grade are 569.2 and 573.2, respectively.

(2) In regard to the closure for the spur track, the C.M.St.P.&P. Railroad reviewed a proposal for the abandonment of the spur track near the Government Bridge and their reply indicated the spur track was needed to serve an industry. Correspondence covering the Railroad's reply is provided in Exhibit A to this Appendix. Closure of the spur track is incorporated into the closure structure for River Drive shown on Plate 47. Should the position of the Railroad change because of abandonment, or for any other reason, closure structure modifications will be accomplished during preparation of final plans and specifications and incorporated into the relocation agreements with the owner.

b. Chicago, Rock Island, and Pacific Railroad facilities are to be modified as follows:

(1) The closure structure at the I-280 overhead bridge provides for a triple-track closure to be constructed across the railroad tracks as shown on Plate 41. The opening will be 51 feet, 6 inches wide and up to 7 feet high, from sill to levee grade. The wall in this reach will have 3 feet of freeboard. A concrete and steel sheet pile wall will be constructed beneath the tracks to provide an impervious barrier to seepage through the free-draining railroad ballast. The base-of-rail elevation of the C.R.I.&P. double tracks is 564.8. The base-of-rail of the C.M.St.P.&P. spur track is at elevation 563.5. The design flood elevation and design levee grade are 568.0 and 571.0, respectively.

(2) There are three tracks at the I-280 overhead bridge; consisting of two main tracks owned by the C.R.I.&P. Railroad and a C.M.St.P.&P. Railroad spur track which terminates approximately 300 feet west of the proposed closure structure. The C.M.St.P.&P. Railroad reviewed a proposal for the abandonment of the spur track west of I-280 and indicated the track could not be abandoned because it is needed to serve a future industrial development area west of the closure. Correspondence from the Railroad is provided in Exhibit A to this Appendix. Should the position of the Railroad change because of abandonments, or for any other reason, closure structure modifications will be accomplished during preparations of final plans and specifications and incorporated into the relocation agreements with the owner.

c. Davenport, Rock Island, and North Western Railway Company facilities are to be modified as follows:

(1) At the Davenport water treatment plant shown on Plate 56 two single-track closures will be necessary where the D.R.I.&.N.W. Railway crosses the line of protection at the upstream and downstream locations. The opening for the two closures will be 17 feet wide. One wall will be approximately 8.8 feet high and the other approximately 5.2 feet high from sill to top of I-wall. The walls in this reach of the project will have 3 feet of freeboard. A steel sheet pile wall will be driven across and beneath the tracks for both of the closures to provide an impervious barrier to seepage through the free-draining railroad ballast. The base-of-rail elevation at the upstream closure is 564.5 and 568.1 at the downstream location. The design flood elevation and design floodwall grade are 570.9 and 573.9, respectively.

(2) A proposed railroad grade crossing is required just north of the Crescent Bridge for the proposed scenic road and bikeway that will cross the two tracks owned by the D.R.I.&N.W. Railway and C.M.St.P.&P. Railroad. Correspondence from the Railroad on the coordination of this item with the City and the Corps is provided in Exhibit A to this Appendix. Details of the cost sharing features of the scenic road and bikeway are contained in Appendix E.

4. Views of Owner. The comments of the three Railroads regarding the replacement and/or modifications of each of their facilities are provided in Exhibit A to this Appendix. Design criteria for trackwork, clearances, and steel sheet piling and sills are based on known acceptable standards of the Railroads and similar designs approved for other recent Federal local flood protection projects.

5. Cost Estimate. The cost estimate for the proposed railroad relocations is contained in Appendix E, "Detailed Estimate of Cost."

6. Design and Construction Responsibilities. All designs and modifications required of railroad facilities will be prepared by the Government. Relocation agreements will be negotiated with each of the three Railroads for modifications of their facilities. Responsibility for construction shall be as follows:

a. Steel Sheet Pile Cutoff Walls. A Government-advertised construction contract for levees and appurtenances will include installation of the steel sheet piling and concrete sills. Removal and replacement of railroad track required to install the walls will be done by the Railroads under negotiated relocation agreements.

b. Other Track Alterations. The railroads will perform all track work on active lines and all signal and communication line alterations.

HIGHWAY RELOCATIONS

7. General. The proposed flood protection plan for Davenport will affect the facilities of the Iowa Department of Transportation, the city of Davenport, and private access roads.

8. Existing Highways. Locations of roads to be relocated and/or modified are shown on Plates 3, 4, 12, 17, 18, 19, 20, 21, and 48.

9. Proposed Highway Alterations.

a. Iowa Department of Transportation.

(1) The levee will tie into the I-280 embankment at the west end of the project as shown on Plate 3.

(2) The flood protection project crosses US Highway 67 (River Drive) at the east end of the project near the Government Bridge as shown on Plate 21. A panel closure structure 80 feet wide and 14 feet high will be necessary at this location.

b. City of Davenport.

(1) Panel type closures are proposed across East Second Street, East Third Street, River Drive, Perry Street, and Gaines Street as shown on Plates 48, 21, and 18.

(2) A panel closure would be installed where the line of protection crosses Main Street. This opening is necessary for access to the Main Street boat ramp as shown on Plate 20.

(3) In the reach from Credit Island Drive to John O'Donnell Stadium, as shown on Plates 12 through 18, the levee will be widened to 30 feet to accommodate a scenic road. A bicycle/hiking path will parallel the scenic road. Details of the cost sharing features of the recreational road and bikeway are contained in Appendix E.

(4) Credit Island Road will be ramped over the levee at levee station 130+00 as shown on Plate 12.

(5) South Concord Street will be ramped over the levee at levee station 245+30 as shown on Plate 4.

10. Views of Owner. The proposed plans for road relocations will be furnished to the owners for their review and comments. Preliminary conferences have been held with the city regarding the street modifications.

11. Cost Estimates. The cost estimates for the proposed highway relocations are contained in Appendix E.

12. Design and Construction Responsibility. Design of highways and ramps will be accomplished by the Government and shall comply with all applicable State, county, and local governmental design standards. Design and construction of highway alterations that are an integral part of the flood protection project will be accomplished at Federal cost. Other construction shall be carried out under the requirements of local interest obligations for non-Federal costs.

UTILITY RELOCATION

13. General. The proposed flood protection plan affects the facilities of the Davenport Water Company, Northwestern Bell Telephone Company, Iowa-Illinois Gas and Electric Company, and the city of Davenport. The Davenport Water Company is a private company that provides the City with water. Northwestern Bell Telephone Company provides telephone service to the City while Iowa-Illinois Gas and Electric Company serves the Davenport residential area with electric power and gas.

14. Existing Utilities Affected

a. Davenport Water Company. There are four waterlines affected by the main river reach of the project ranging in size from six to eight inches. The six-inch waterline will be crossed by the closure structure, as shown on Plate 48. The eight-inch waterline serving Credit Island Park and shown on Plate 12 is owned by the Davenport Park Commission. The eight-inch waterline shown on Plate 4 serving South Concord Street residents is owned by Davenport Water Company. An 8" line in 2nd Street is owned by Davenport Water Company and will be crossed by the closure structure, as shown on Plate 21.

b. Northwestern Bell Telephone Company. There are eight buried transcontinental cables that cross the line of protection and continue across the Mississippi River in the riverbed which will be crossed by the proposed line of protection, as shown on Plate 20 at Station 22+20.

c. Iowa-Illinois Gas and Electric Company. There are three gas lines that cross the main river reach and are shown on Plates 4, 12, and 48. There are two buried electric lines, as shown on Plates 20 and 21, that will be affected by the project. The 2-13 kv powerlines on the Crescent Bridge are not affected by the project.

d. City of Davenport. Two fire hydrants, shown on Plates 17 and 18, will be affected by the project.

15. Proposed Utility Alterations.

a. Davenport Water Company. The eight-inch waterlines at South Concord Street and Credit Island Road shown on Plates 4 and 12, and protection of the waterlines at the closure structures, as shown on Plates 21 and 48, will be items of Federal responsibility.

b. Northwestern Bell Telephone Company. The protection of the telephone cables crossed by the flood protection facilities near Station 22+20 will be a Federal responsibility.

c. Iowa-Illinois Gas and Electric Company. Most of the alterations to the facilities of Iowa-Illinois Gas and Electric Company will be the responsibility of the city of Davenport. All gasline relocations required because of the proposed improvements are items of local responsibility, with the exception of protection for the gasline crossed by the closure structure at Iowa Street and 3rd Street, as shown on Plate 48, which would be a Federal responsibility. Protection of the buried electric line at Station 10+60 where the line is crossed by the line of protection will be a Federal responsibility. Relocation of the buried electric line, vicinity Station 10+65 to 12+95, as shown on Plate 21, will be a local interest responsibility.

16. Views of Owner. Representatives of the city, and the three utility companies affected by the Davenport project will be informed of the proposed plans for utility relocation, and concurrence in these plans will be secured from each company.

17. Cost Estimate. The cost estimate for the proposed utility relocations is contained in Appendix E.

18. Design and Construction Responsibility. The alterations to the water treatment plant facilities will be designed by the Federal Government and constructed under a Federal Government advertised construction contract. All other Federal responsibility alteration of water and gas facilities will be designed and constructed by the respective utility companies. Federal responsibility work to be performed by the utility companies will be covered by negotiated relocation agreements.

RELOCATIONS BRANCH/MERRITT/ser/227

1 FEB 1977

NCRED-RL

Mr. W. F. Bannon
Division Engineer
Chicago, Milwaukee, St. Paul
and Pacific Railroad Company
1116 Rawson Street
Perry, Iowa 50220

Re: Davenport, Iowa, Local Flood
Protection Project

Dear Mr. Bannon:

This is in reply to your letter of 22 December 1976 concerning the proposed road and bicycle path crossing near the Crescent Bridge in Davenport, Iowa.

In conjunction with the City of Davenport, we are presently revising the plans to show a proposed grade crossing over your tracks near the Crescent Bridge for the road and bicycle path.

I will furnish you plans of the grade crossing for your review as soon as they are completed.

Sincerely yours,

DOYLE W. McCULLY
Chief, Engineering Div.

~~MERRITT~~ F.M.
~~FLEISCHMAN~~
~~PETERSON~~
~~HURLBUTT~~
~~McCULLY~~ DWM 4/

Copy Furnished:
Mr. W. E. Fuhr
Assistant Chief Engineer - Staff
Chicago, Milwaukee, St. Paul
And Pacific Railroad Company
898 Union Station
Chicago, Illinois 60606

cc: ~~Dist~~ File (Reloc Br.)
Engineering Div
Design Branch
Gen. Engr Sec (Peterson)
Relocations Branch

Perry, Iowa - December 22, 1976

File: Road & Bicycle Path
Crescent Bridge
Davenport, Iowa

Mr. Doyle W. McCully
Chief, Engineering Div.
Dept. of Army
Rock Island Dist.
Corps of Engineers
Rock Island, Illinois, 61201

Please refer to your letter of April 26, 1976 in which a road and bicycle path was proposed to be placed under the so-called cut-off channel bridges near Crescent Bridge. The proposed path and road would cross under DRI&NW Ry. bridge 36.68 #149 and Milwaukee R. R. bridge Z-2000A. It has now been proposed to fill both of these bridges in the next couple years, which would mean the city would have to provide an underpass or a grade crossing over these two tracks. In talking with the city, this proposed park area will not be developed untill 1980.

You may advise how you wish to proceed with this proposed road and bicycle path.

W. F. Bannon,
Division Engineer

NIL/sk

cc: Charles Heston; Community Developement, City of Davenport
B. A. Webster, DRI&NW
W. E. Fuhr

CHICAGO, MILWAUKEE, ST. PAUL AND PACIFIC RAILROAD COMPANY

Engineering Department

516 West Jackson Boulevard • Chicago, Illinois 60606

W. E. FUHR
Assistant Chief Engineer-Staff

December 8, 1976
M-71236

Mr. Doyle W. McCully
Chief, Engineering Division
Department of the Army
Rock Island District
Clock Tower Building
Rock Island, Illinois 61201

Dear Mr. McCully:

Please refer to your letter of January 16, 1976 concerning your proposed flood control project at Davenport, Iowa.

We have considered both of the track retirements that you suggested but cannot agree to either of them. The track just west of the Government Bridge serves an industry and must be retained so that we can continue to serve it. The track at the Interstate 280 overhead bridge is to serve a future industrial development and must be retained since it is our only access to that area.

Yours truly,

W. E. Fuhr

cc - Mr. W. F. Bannon

RELOCATIONS BRANCH/VanHOOREBEKE-MERRITT/cds/296-227

NCRED-RL

16 January 1976

Mr. W. E. Fuhr
Assistant Chief Engineer - Staff
Chicago, Milwaukee, St. Paul and
Pacific Railroad Company
898 Union Station
Chicago, Illinois 60606

Re: Davenport, Iowa, Local Flood Protection Project

Dear Mr. Fuhr:

Inclosed are preliminary prints for the referenced project, showing the project location, and general plans of proposed Chicago, Milwaukee, St. Paul and Pacific Railroad Company track modifications. The proposed plan of improvement provides for a system of levees, floodwalls, and closure structures on portions of the Railroad's property, as part of the overall flood protection facilities.

The general location of a proposed triple-track railroad closure structure across your two mainline tracks and spur track near the Government bridge is shown on Plate F-16. The spur track is located within an alleyway and is routinely blocked by automobiles parked across the track. Railroad freight cars have only infrequently been observed on this spur. If this track may be removed it would permit reducing the proposed triple-track floodwall closure to a double-track closure. Another benefit of removing this track would be the elimination of the long skewed grade crossing on the heavily traveled East River Drive. Please assess the current and future needs for this track and give me your decision regarding the feasibility for its abandonment.

The closure structure proposed at the Interstate 280 overhead bridge provides for a gated closure to be constructed across the railroad tracks at the general location shown on Plate F-20. There are three tracks at this location; consisting of two main tracks owned by the Chicago, Rock Island and Pacific Railroad Company and your spur track which terminates approximately 300 feet west of the proposed closure structure. On field inspections in October and November 1975, members of my staff noted the spur track has not been used recently, nor has there been any extension

NCRED-RL
Mr. W. E. Fuhr

16 January 1976

or development of the track west of the I-280 overhead bridge since it was constructed. For these reasons, I would like the Railroad to also consider the abandonment of this spur track west of I-280 as shown in red on Plate F-20. The abandonment and removal of this track would reduce the triple-track closure structure at this location to a double-track closure structure, resulting in a better overall closure plan.

When you have completed your review of these considerations, please advise us of your decisions. We will be happy to arrange a meeting with you and members of your staff to discuss the Davenport project if you desire.

Sincerely yours,

3 Incl
1. Plate F-13
2. Plate F-16
3. Plate F-20

DOYLE W. McCULLY
Chief, Engineering Division

Copy furnished w/incl:
Mr. B. A. Webster
General Manager
Davenport, Rock Island and
North Western Railway Company
Union Station
Davenport, Iowa 52801

cc: Dist File (Reloc Br) (w/incl)
Engineering Division (wo/incl)
Design Branch (wo/incl)
General Engrg Section (wo/incl)
Relocations Branch (wo/incl)

~~VanHOOREBEKE~~
~~MERRITT~~
~~FLEISCHMAN~~
~~PETERSON~~
~~HURLBUTT~~
~~McCULLY~~

MISSISSIPPI RIVER
DAVENPORT, IOWA

LOCAL FLOOD PROTECTION
PHASE II

GENERAL DESIGN MEMORANDUM

APPENDIX H

ENVIRONMENT AND CULTURAL RESOURCES
(REVISED

US ARMY ENGINEER DISTRICT, ROCK ISLAND
CORPS OF ENGINEERS
CLOCK TOWER BUILDING
ROCK ISLAND, ILLINOIS 61201

MISSISSIPPI RIVER
DAVENPORT, IOWA

LOCAL FLOOD PROTECTION
PHASE II

GENERAL DESIGN MEMORANDUM

APPENDIX H
ENVIRONMENT AND CULTURAL RESOURCES

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EXHIBIT A PERTINENT CORRESPONDENCE

Letter Dated 28 September 1979 to Iowa Natural Resources Council
Letter Dated 28 September 1979 to Mayor of Davenport
Letter Dated 13 August 1979 to Iowa Department of Environmental Quality
Letter Dated 7 June 1979 to Iowa Department of Environmental Quality
Letter Dated 14 June 1979 to State Historic Preservation Officer
Letter Dated 5 June 1979 to Fish and Wildlife Service
Letter Dated 8 May 1979 from the Park and Recreation Department, City of Davenport

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Letter Dated 13 April 1979 from the Environmental Protection Agency
Letter Dated 12 April 1979 from the Environmental Protection Agency
Letter Dated 6 April 1979 from the Fish and Wildlife Service
Letter Dated 1 December 1978 from the Fish and Wildlife Service
Letter Dated 24 October 1978 from the Iowa Department of Environmental Quality
Letter Dated 2 October 1978 from the Fish and Wildlife Service
Letter Dated 8 August 1978 from the Fish and Wildlife Service
Letter Dated 24 July 1978 from the Fish and Wildlife Service
Letter Dated 14 June 1978 to the Iowa Conservation Commission
Letter Dated 1 June 1978 to the Fish and Wildlife Service
Letter Dated 25 May 1978 to the Fish and Wildlife Service
Letter Dated 24 May 1978 to the Environmental Protection Agency
Letter Dated 28 April 1978 from the Environmental Protection Agency
Letter Dated 28 April 1978 from US Department of the Interior
Letter Dated 27 April 1978 to Mr. Jim Cameron, Midwestern Grain Co.
Letter Dated 27 April 1978 to Mr. F. A. Barton, Chicago Milwaukee, St. Paul and Pacific Railroad Company
Letter Dated 26 April 1978 from the Iowa Conservation Commission
Letter Dated 25 April 1978 from the Fish and Wildlife Service
Letter Dated 21 April 1978 from the State Historic Preservation Officer
Letter Dated 13 April 1978 from Mr. Jim Cameron, Midwestern Grain Co.
Letter Dated 7 April 1978 from Mr. F. A. Barton, Chicago, Milwaukee, St Paul, and Pacific Railroad Company
Letter Dated 4 April 1978 from the Iowa Department of Transportation
Letter Dated 9 July 1976 from the Office of Archaeologicy and Historic Preservation, National Park Service
Letter Dated 15 March 1976 from the State Historical Department of Iowa
Letter Dated 17 March 1976 from the State Historic Preservation Officer
Letter Dated 29 October 1979 from State of Iowa Natural Resources Council
Letter Dated 1 February 1980 from City of Davenport

MISSISSIPPI RIVER
DAVENPORT, IOWA

LOCAL FLOOD PROTECTION
PHASE II

GENERAL DESIGN MEMORANDUM

APPENDIX H
ENVIRONMENT AND CULTURAL RESOURCES

I BACKGROUND

1. The Davenport Local Flood Protection Project was authorized by the Flood Control Act of 1970 in accordance with the provisions of House Document No. 92-161. The purpose of the project is to establish flood protection for residential, commercial, and industrial areas within the one-half percent (200-year frequency) flood plain at Davenport, Iowa.

2. A Draft Environmental Impact Statement (EIS) was prepared by the Rock Island District and filed with the Council on Environmental Quality on 11 November 1974. Comments and recommendations made by Federal, State, and local governmental agencies, organizations, and individuals were incorporated and a final EIS prepared. The Final EIS, dated August 1976, was filed with the Environmental Protection Agency on 3 March 1978. Public distribution of the final EIS was made subsequent to filing with the Environmental Protection Agency.

II PROJECT DESCRIPTION

3. The recommended plan consists of constructing levees and floodwalls along the Mississippi River between Interstate 280 and the Government Bridge, and structural protection for the water treatment plant. The levees and floodwalls will provide protection from the one-half percent (200-year frequency) flood on the Mississippi River. A levee constructed with sand dredged from the Mississippi River would extend from I-280 along the mainland shore, cross Credit Island Slough to Credit Island, cross back to the mainland on the Credit Island causeway and then follow the riverfront to the vicinity of the John O'Donnell Municipal Stadium. From the stadium to the vicinity of Perry Street a folding concrete and aluminum floodwall on a low earthen levee base will provide the necessary flood protection. From Perry Street to the Government Bridge a concrete floodwall will be constructed parallel to River Drive.

4. The total length of levees to be constructed between I-280 and the Centennial Bridge is approximately 4 miles. The levees will be constructed with sand dredged from the Mississippi River. Silt dredged from Credit Island Slough will be used to cap the sand levees and to cover the old city dump site. Portions of the levee exposed to wave action will

be protected with riprap. A total of 905,000 cubic yards of sand and 380,000 cubic yards of silt will be required in construction. From I-280 to the Credit Island Causeway the levee will have a crown width of 12 feet, an average height of 14 feet, and an average base width of 110 feet. Between the Credit Island Causeway and the Centennial Bridge the levee will have a crown width of 30 feet to accommodate a scenic drive. Average height of the levee in this reach is 16 feet and the average base width is 142 feet. A total of 90 acres is required for levee rights-of-way.

5. Between the Centennial Bridge and the Government Bridge the line of protection will consist of 2,028 feet of folding floodwall and 1,785 feet of permanant wall. Removable panel closure structures will be provided where major streets intersect the line of protection. Approximately 2 acres of land will be required for project rights-of-way between Centennial Bridge and the Government Bridge. Although construction will be primarily of concrete and aluminum, 11,000 cubic yards of clay fill will also be used.

6. Interior drainage in the protected area will be removed during flooding by three pumping stations. These are located at the downstream end of Credit Island Slough, at the foot of Gaines Street and at the foot of Main Street. Credit Island Slough will be provided with gated closures at either end and will be used for ponding during major floods. The other pumping stations will remove water from the storm sewer system when oulets are blocked during periods of high flow.

7. The levee design is based on a flood on the Mississippi River having a one-half percent chance of occurring in any year. This is the highest level of protection which is economically, socially, and environmentally acceptable to the public and participating agencies. The flow for this flood would be 385,000 cfs and its stage would be 3.5 feet higher than the maximum flood of record which occurred in 1965. The one-half percent would reach an elevation of about 568.7 feet mean sea level (m.s.l.). This would reach a stage of about 23.7 on the Lock and Dam 15 tailwater gage. The levees will generally have a freeboard of 3 feet from I-280 upstream to the Centennial Bridge and the floodwall will have 4 feet of freeboard above the one-half percent frequency flow when fully extended.

8. A number of recreation facilities will be constructed on a cost-share basis between the Federal Government and Davenport. The recreation features include bike and hike trails, a boat ramp, a scenic drive, athletic fields and and picnicking facilities. Recreation development is discussed in Appendix I.

9. Nahant Marsh will be acquired and preserved by the city of Davenport as a part of the recommended plan. This action will provide Davenport with a unique nature study area.

10. Extensive beautification and mitigation measures will be undertaken to restore areas disturbed by construction. These measures should enhance the beauty of the riverfront area and make it attractive to local residents and visitors alike.

III IMPACTS OF THE PROPOSED PROJECT

11. Geology and Soils

a. The proposed project will have minor impact on the geology and soils of the Davenport area. Sand dredged from the Mississippi River will be used for construction of the levee between I-280 and the Centennial Bridge. Approximately 905,000 cubic yards of sand will be required for this purpose. The levee will be capped with an 18- to 36-inch thickness of silt dredged from Credit Island Slough. Silt will also be used to cover the old city dump site. This covering will provide a suitable base on which to establish vegetative growth and to provide an area suitable for recreation development. Approximately 380,000 cubic yards of silt will be needed for this purpose.

b. The use of sand and silt dredged from the Mississippi River and Credit Island Slough will be a productive use of these resources. Sand, which is a very abundant resource, will be put to the economically beneficial use of flood control. Removal of an average of 4 feet of silt from Credit Island Slough will improve aquatic conditions in the slough. The silt will be placed on the sand levee and on the old city dump in order to provide a base on which vegetation can be established.

c. Approximately 11,000 cubic yards of clay fill will be required in construction of the flood control structures from the Centennial Bridge upstream to the Government Bridge. This material will be used to provide a base for the floodwall in this reach. It is proposed that this material be previously stripped soil obtained from a limestone quarry at Linwood, Iowa. Use of this material will not require additional disturbance of soil resources.

d. Construction activities in the project area may result in some soil compaction of temporary haul roads. The area affected will be small, however, and should cause no significant problems during revegetation of the area.

e. Approximately 37,000 cubic yards of riprap will be required for levee protection. A potential source of this riprap is also the quarry at Linwood, Iowa. Obtaining riprap from this or any other active quarry will result in no major new disturbance of the area's geologic resources.

12. Physiography

a. The most obvious impact on the physiography of the Davenport area will be the construction of a raised ribbon of levee and floodwall. This ribbon of high ground will extend for approximately 5 miles and will be a highly visible feature of the riverfront, due to it passing through level terrain. Also, the massive linear lines of the levee and floodwall will contrast with the urban setting.

b. Another less obvious alteration of the area's physiography will be the further confining of the Mississippi River. Construction of flood control structures will prohibit the river from utilizing a portion of its

flood plain. This construction will result in minor increases in flood heights in this immediate section of the river. The following section on hydrology will discuss these impacts.

13. Hydrology

a. The proposed project will restrict the Mississippi River from a portion of its flood plain. This reduces the land area that would be available for flood water storage and conveyance under natural unconfined river conditions. As a result flood heights will be increased between Locks and Dam 15 and the I-280 bridge. The increase will range from approximately 0.12 feet in the two percent (50-year frequency) flood up to 0.15 feet in the one half percent flood. The flood protection structures for Davenport are designed to accommodate these increases. On the Illinois shore the developed areas adjacent to the river are protected by the Rock Island Local Flood Protection Project. Increases in flood damages that will occur are so small that they cannot be measured.

b. Runoff flowing toward the river during flood periods will be pumped over the line of protection by three pumping stations. A pumping station adjacent to the gated structure at the downstream end of Credit Island Slough will pump out excess water during the periods when the gates at either end of the slough are closed. Water moving toward the river in the upstream portion of the project area will be pumped out by stations at the foot of Gaines Street and Main Street. No ponding areas will be built at these two stations which will pump water collecting in the storm sewer system.

c. The blocking of Credit Island Slough for use as a ponding area during floods will result in a concentration of urban runoff in that water body. Water quality will be degraded during these periods. However, this will occur infrequently and some water movement will occur while the pumping plant is in operation. Therefore, the temporary blockage of normal flow should not result in significant damage to the aquatic life in the slough. The adverse impacts that do occur will be offset by opening the slough to greater flow during normal river levels and by the removal of silt from the slough.

d. The two upstream pumping stations will not alter the hydrological conditions of the area. The plants will pump water that flows directly into the river during periods of normal flow.

14. Water Quality.

a. During construction of the project, water quality in the Mississippi River and Credit Island Slough will be temporarily degraded. The larger part of this will be turbidity resulting from operation of a hydraulic dredge in these areas. There will also be turbidity and silting in areas where construction occurs close to or in the river and slough. These construction impacts are all of a temporary nature and no long term degradation will occur.

b. Placing twin 8 foot by 8 foot culverts at the upstream end of Credit Island Slough will increase water flow into the slough during normal river conditions. There are two 8-foot diameter corrugated metal pipes at the upstream end of the slough, placed there in 1977 when the new regional sewage treatment plant was put into operation. The existing pipes opened the slough to water movement and improved conditions in the slough. The larger culverts will further increase flow and improve aquatic conditions in the slough. Conditions will be greatly improved over pre-1977 conditions when no flow entered the upstream end of the slough under normal river conditions.

c. During major Mississippi River floods it will be necessary to close the gated opening at the upstream and downstream ends of Credit Island Slough. The gates will be closed an average of less than four days per year. Some years the gates will not be closed at all. The maximum time period, on the basis of the 1965 flood of record, that the gates would be closed is 30 days. Urban runoff, primarily from Blackhawk Creek, will be collected in the slough during these periods. The concentration of urban runoff will likely cause a reduction of water quality in the slough. However, some water movement would continue in the slough as the ponded water is discharged by the pumping station. Because of this continued water movement and because the ponding of runoff will occur infrequently, water quality and aquatic life in the slough should not be significantly damaged.

d. A secondary sewage treatment plant is located adjacent to South Concord Street, serving Davenport, Bettendorf, and Riverdale. The outfall structure of this plant is located in the Mississippi River below the gated opening on the downstream end of Credit Island Slough. No sewage effluent is released directly into the slough. The gated opening at each end of the slough will allow water flow and prevent accumulation of this effluent below the downstream gated opening. This water movement will also prevent the effluent from backing up into the slough during rising river conditions. Therefore, the location of the outfall structure will cause no adverse impact on water quality or the environment of the slough.

e. During floods on the river, the gated opening at either end of the slough will be closed to prevent flood damage. This would result in no movement of water out of the slough to dilute the sewage effluent. However, the increased water velocity during floods should adequately disperse the effluent. Also, the pumping station to be located near the downstream gated opening will pump water out of the slough and into the river during floods. This water will further help to disperse the sewage effluent.

f. Silt for use in levee construction will be obtained by dredging in Credit Island Slough. In order to substantially reduce amounts of suspended material flowing in the slough, the water flowing off the silt will be collected in a holding pond or ponds on Credit Island. Water will be retained in the ponds long enough to allow a majority of the suspended material to settle out. With the suspended material many of the pollutants that are trapped in the silt will also settle out. The clarified

water would then be allowed to return to the slough and the materials that settled out would be disposed of. This method will protect the river from high turbidity levels, silting, and introduction of harmful materials.

15. Bottom Sediment Analysis

a. As discussed in the preceeding section, the water flowing off the silt dredged from Credit Island Slough will be retained in holding ponds on Credit Island before being returned to the slough. In view of the proximity of the dredging sites to a number of former wastewater outfalls a possibility existed that these sites could be contaminated by heavy metals and chemical residues. In order to determine the levels of occurrence of contaminants in the proposed borrow sites, a program was undertaken to conduct an analysis of the sediments.

b. The analysis conducted is referred to as elutriate testing. Elutriate sampling consists of shaking a sample of bottom sediment in water and then analyzing the water to determine what contaminants were released during shaking. Six bottom samples were obtained by Rock Island District personnel in November 1978. Samples were taken from proposed borrow sites in the main river and also in Credit Island Slough. These samples were analyzed by the Iowa State Hygienic Laboratory, Des Moines, Iowa, and Iowa City, Iowa. Parameters tested included heavy metals, oil and grease, and chlorinated hydrocarbon pesticide residue.

c. Results of the analysis were provided to the US Environmental Protection Agency (EPA), Fish and Wildlife Service, and the Iowa Department of Environmental Quality. Letters of comment received from EPA and the Fish and Wildlife Service are included in Exhibit A. The Fish and Wildlife Service expressed concern with impacts that could occur even though no parameters were found to be at toxic concentrations. The Fish and Wildlife Service concerns are addressed in a letter (5 June 1979) included in Exhibit A. The EPA commented that "Based on the data provided by your report, discharge to the Mississippi River without pretreatment is acceptable. In the few cases where concentrations are higher than the recommended limits, dilution in the mixing zone will be adequate."

d. Based on the analysis of bottom sediments it is not anticipated that reintroduction of water flowing off the dredged material will pose a threat to the aquatic environment. However, should conditions in the dredging sites change prior to construction then additional actions will be undertaken to protect the river.

16. Federal Water Pollution Control Act

a. The purpose of the Federal Water Pollution Control Act Amendments of 1972 (Clean Water Act) is to restore and maintain the chemical, physical, and biological integrity of the nation's waters. Section 404 of that Act established a permit program to be administered by the Corps of Engineers to regulate the discharge of dredged or fill material into the waters of the United States.

b. To comply with Corps regulations implementing the Act, the Rock Island District issued a Public Notice on 28 March 1978 and a 404 Evaluation Report was prepared. The Public Notice was distributed to concerned Federal, State and local governmental entities, businesses, organizations, and individuals requesting their comments and giving the opportunity to request a public hearing. Seven responses to the Public Notice were received from the following agencies and businesses.

US Environmental Protection Agency
US Fish and Wildlife Service
Iowa Conservation Commission
Director, Iowa Division of Historic Preservation (State Historic Preservation Officer)
Iowa Department of Transportation
Chicago, Milwaukee, St. Paul and Pacific Railroad Company
Midwestern Grain Company

c. The seven responses to the Public Notice did not contain any requests for a public hearing and therefore, none was held. There were no objections expressed in any of the comments to the proposed project provided that certain measures be taken to protect the resources of the area. These measures will be undertaken as specified in the Rock Island District responses. The letters and responses to these letters are included at the end of this appendix (Exhibit A).

d. Certification under Section 401 of the Act was requested from the State of Iowa. That certification was granted in a 24 October 1978 letter from the Iowa Department of Environmental Quality. That letter is found in Exhibit A.

e. The Section 404 Evaluation Report, Public Notice, and related documents were filed on 2 April 1979 with the EPA as supplemental data to the previously filed final EIS. The supplemental data was then provided to all those who had received the final EIS. Two letters of comment were received on the supplemental data. A letter, dated 13 April 1979 (Exhibit A), from the EPA, stated that the proposed project is in compliance with Section 404 requirements. The second letter, dated 7 June 1979 (Exhibit A), from the Iowa DEQ expressed concern regarding disruption of the former city landfill. A reply letter, dated 13 August 1979 (Exhibit A), noted that the proposed use of the landfill site was for a park and the further development of plans for the area will be coordinated with Iowa DEQ.

17. Botanical

a. The only proposed action in the reach from the Government Bridge to the Bettendorf city limits is protection of the Davenport Water Company Treatment Plant. There will be only minor impact on the limited vegetation in proximity of the plant. From the Government Bridge downstream to the Centennial Bridge some of the park vegetation will be replaced by a floodwall. In the landfill area, existing ground cover and immature trees will be covered with dredged silt and the area graded and landscaped for use as

a recreational area. Downstream of the landfill the band of riverfront vegetation widens and includes a limited number of willow and cottonwood trees, many of which will be removed when the levee is constructed.

b. There are approximately 190 acres of medium to high density woods on Credit Island. Another 55 acres of the island are lightly wooded. The proposed plan calls for a levee to be built along the Credit Island Slough shore for the entire length of the island. The required right-of-way consists of a strip 150 feet wide and 8,700 feet long, portions of which are grass and forb covered, with a significant portion being wooded. An existing roadway is also within the required right-of-way so that the vegetation lost will be only a portion of the total acreage required.

c. The upper 3,500 feet of Credit Island through which the levee will be constructed is largely grasses and forbs interspersed with a few mature walnut, oak, and maple trees. Approximately 6 acres of vegetation will be replaced by a grass covered levee in this reach. The middle reach of the levee will extend 4,300 feet through bottomland hardwoods. This area contains a diverse tree stand which provides good wildlife habitat. Approximately 11 acres of habitat will be removed in this reach. The downstream 900 feet of the levee will pass through an area densely covered by large maple trees. This area is flooded frequently but offers good habitat for many mammal and bird species. Approximately three acres of this habitat will be removed.

d. During final design and construction of the project every effort will be made to minimize disruption of the area's botanical resources. Trees will be removed only when there is no way to avoid them. Construction activities will be confined to as small an area as possible in order not to damage vegetation adjacent to the levee alignment. Landscaping with extensive planting of trees and shrubs will be undertaken to mitigate unavoidable losses of vegetation. The levees will be capped with 18 to 36 inches of silt to provide a rich soil for establishment of vegetation. Reclamation of the old landfill by covering it with silt and landscaping will greatly enhance the vegetation diversity and aesthetic quality of that area.

e. As part of the project it is proposed that an area known as Nahant Marsh be acquired by the city of Davenport and preserved as a nature study area. The following section will provide discussion on Nahant Marsh and the proposed plans for it.

f. In summary, although the project will require 94 acres for rights-of-way, much of which has a vegetative cover, these losses will be partially offset by extensive landscaping. The most significant losses of vegetation will occur along the Credit Island shoreline where 20 acres of vegetation wll be disrupted or destroyed. The most significant beneficial gains will be preservation of approximately 160 acres of Nahant Marsh and adjacent wet meadows and the reclamation of the 60-acre landfill.

18. Nahant Marsh

a. Nahant Marsh is located at the downstream end of the project area and east of Interstate 280. The marsh is bordered on the west by Wapello Avenue, on the south by a rail yard and on the north and east by an agricultural area which is undergoing rapid commercial and industrial growth. In a typical year approximately 100 acres are covered by plant species characteristic of wetlands. The area covered by wetland plants fluctuates significantly depending on seasonal and cyclic precipitation and river levels. The center of the marsh contains the deepest water and is classified as inland shallow fresh marsh. The frequently exposed fringes of the marsh are classified as wet meadow.

b. Predominant plant species in the marsh are grasses, sedges, cattails, and smartweeds. A variety of flowering plants including duckweed, swamp milkweed, arrowhead, blue iris, loose strife, sunflowers, and goldenrod have been noted in the marsh area. The most obvious animal inhabitat of the marsh is the muskrat. In favorable years the marsh supports a dense muskrat population and may contain as many as 150 active muskrat houses. A variety of small mammal species including mink, rabbit, and opossum are present in the marsh. The marsh and fringes provide habitat for a variety of song birds, wading birds, shorebirds, waterfowl, and raptors. Representative species include killdeer, sandpipers, mallards, teal, coot, grackles, red-winged blackbirds, various sparrows and hawks. In summary, Nahant Marsh and its fringes contain a variety of habitats within a small area and supports a remarkably diverse flora and fauna.

c. Nahant Marsh is a unique area due to being located within a major urban area. Location in such an area provides easy access for school groups, organizations, and individuals for purposes of nature observation and study. Considerable use of this type occurs at present and the potential for expanded use is great.

d. The proposed flood control project would interfere with the hydrologic processes to which the marsh owes its continued existence. Under present conditions the marsh is flushed and its water supply augumented by flood waters from the Mississippi River. The flood project would cut off the water movement under all moderate and high river conditions. The only water reaching the marsh from the river would be minor backwater entering from the culverts under I-280 and Wapello Avenue. As a result, the natural aging process of the marsh would be speeded up and would accelerate the eventual draining, filling, and development of the marsh for commercial and industrial uses. Davenport would then have lost a unique and irreplaceable natural resource.

e. The flood control project for Davenport, authorized by Congress in the Flood Control Act of 1970, does not include measures to protect Nahant Marsh because the need was not determined until later during the plan formulation process. In order to incorporate protection for the marsh as a part of the overall flood protection project, specific Congressional approval is required. To accomplish this, the Rock Island

District prepared a "Special Report for Fish and Wildlife Mitigation and Enhancement" and forwarded it to higher Corps of Engineers authority in October 1977. The report is currently awaiting Congressional action. The Special Report contains a detailed presentation of the nature of the marsh, proposed features for protection and cost breakdown for the protection plan.

f. The special report contains the following provisions for Nahant Marsh:

(1) 163 acres presently in private ownership be acquired or controlled by the public for fish and wildlife management purposes.

(2) A control structure to regulate the water level in the marsh, be constructed at the outlet to the marsh.

(3) A levee, drainage ditch, and control structure be constructed at the upstream end of the marsh to control the quantity and quality of water entering the marsh.

(4) The estimated cost of the Nahant Marsh plan is $218,000 of which $36,000 will be the local share.

(5) Maintenance and management of the marsh will be the responsibility of the city of Davenport.

g. The concept and specific plans for the presentation of Nahant Marsh have been reviewed and supported by the Fish and Wildlife Service, Iowa Conservation Commission and the city of Davenport. Final development of plans for the marsh will be carried out in coordination with these entities.

h. In summary, the plan for Nahant Marsh will preserve a unique ecological community and provide a significant contribution to the environmental quality of the area. The marsh will provide Davenport with a nature study and green space within the city limits. Few cities have such a unique resource.

19. Zoological

a. With the exception of areas on Credit Island and Nahant Marsh, the project area does not contain any ecosystems which have not previously been severely disturbed by man. Most of the terrestrial ecosystems in the flood plain have been so disturbed that they exhibit extremely low species diversity. Construction of a sewage treatment plant and interceptor sewer line has recently added to disturbance of existing wildlife habitat along the riverfront.

b. Construction of levees and floodwalls always carries the potential for damage to wildlife. Damages can occur in four ways. First, construction will permanently remove existing habitat. Second, construction activitiy can damage or destroy vegetation adjacent to the line of

protection. Third, construction noise can disrupt wildlife beyond the actual area of physical disturbance. Fourth, the presence of the levee or floodwall can restrict wildlife movement patterns.

c. The proposed project will require approximately 94 acres of land for levee and floodwall construction. Approximately 20 acres of this total consists of suitable wildlife habitat almost exclusively on Credit Island. The remaining acreage provides very poor habitiat that supports only small populations of species able to tolerate an urban environment.

d. The removal of trees, which will occur primarily on Credit Island, will reduce habitat for several species of birds. Habitat for mammal species including squirrel, raccoon, opossum, and rabbit will also be lost. However, because a sizable portion of the island provides similar habitat, a portion of the displaced species will be able to move into the remaining habitat. An additional impact of construction will be disturbance resulting from noise. The impact of noise is most adverse during breeding and nesting seasons.

e. Credit Island is a favored roosting area for bald eagles during the winter months. The eagles most frequently use trees along the riverward side of the island. These birds are susceptible to noise and human activity and it will be necessary to avoid construction in this area during the winter use season. The following section on threatened and endangered species provides more information on the bald eagle.

f. The overall environmental quality of Credit Island Slough should be improved as a result of the project. Removal of accumulated silt by dredging and opening the slough to increased water flow will improve aquatic habitat, recreational boating, fishing, and aesthetics. These improvements will be offset to a degree by use of the slough for ponding during flood periods. During the periods when urban runoff concentrates in the slough, water quality will be temporarily degraded. However, because ponding will occur infrequently the overall condition of the slough should be improved.

g. A beneficial impact of the proposed project will be reclamation of the former city landfill. This will provide approximately 60 acres of park vegetation which will provide some urban wildlife habitat.

h. It is planned that materials required for levee construction be obtained by dredging in the Mississippi River. Plate 39 shows the proposed dredging sites. Approximately 905,000 cubic yards of sand for levee construction will be obtained by dredging at selected locations between the Iowa shoreline and the edge of the navigation channel. Silt for capping the levees and covering the old dump site will be obtained by dredging in Credit Island Slough and from the proposed work in Blackhawk Creek.

i. Dredging will result in the disturbance of 115 acres of river bottom in the main river and 88 acres in Credit Island Slough. The immobile aquatic organisms in these areas will be destroyed. Because of

the relatively small extent of the individual dredging sites and presence of large surrounding areas providing a "seed" source, it is anticipated that rapid recolonization by most benthic species will occur. Some of the areas, however, contain mussel populations which will be much slower to return. The dredge sites in the main river have been selected to avoid dense mussel population known from mussel sampling conducted by the US Fish and Wildlife Service. An additional mussel survey is planned in order to more precisely define the limits of mussel beds and to determine if any threatened or endangered mussel species occur in the proposed dredging sites. This survey is discussed in more detail under the section on threatened and endangered species.

j. Preservation of Nahant Marsh will enhance the zoological resources of that area. Management of water levels in the marsh will make the area more attractive to waterfowl and song bird species. Use by ducks will almost certainly increase. Populations of mammals, particularly muskrats, will also benefit from a more stable water level. In past years alternating conditions of either very high water or no water at all have made it difficult for the muskrat population to stabilize. Under the proposed plan for preservation the extremes in water levels will be eliminated. Proposed tree plantings around the margins of the marsh would benefit wildlife by providing a buffer between the marsh and adjacent industries, roads, railroad facilities, and agricultural fields.

20. Noise and Air Quality

a. Moderately high levels of noise are presently a feature of the study area. Diesel trucks and diesel locomotives pass through the area constantly. Furthermore, the immediate project area is an industrial-commercial area and noise is commonly associated with present activities. During construction of the levees, motorized construction equipment and trucks would cause even higher levels of noise. Heavy truck traffic along the route between the borrow areas and the construction sites would increase noise levels intermittently and temporarily. After construction is completed, noise levels are not expected to differ significantly from present levels except when maintenance or operation requires the use of machinery.

b. Air quality will also be temporarily reduced during construction of the project. The operation of a dredge, motorized equipment, and trucks at the construction sites will add exhaust fumes and dust particles into the air. However, since the immediate project area is an industrial-commerical area, exhaust fumes from diesel trucks and locomotives are a constant feature. Heavy truck traffic along the route between the borrow areas and the construction sites would increase exhaust fumes and dust levels intermittently and temporarily in predominantly rural areas. After construction is completed, air quality would not be expected to differ significantly from present levels except when maintenance or operation requires the use of machinery.

21. Social and Economic Conditions

a. Flood protection will provide security from flooding for residential, commercial, industrial, and public property. Approximately 296 residences, 273 commercial establishments, 86 industries, and 22 public facilities are located in the area that would be protected. Security from flooding should heighten levels of productivity, social and economic stability, safety and welfare for the employees and residents of the area. This area is characterized by lower income levels and a greater concentration of elderly and disadvantaged residents than Davenport as a whole. Flood protection would be particularly beneficial to such residents because they are least able to cope economically and emotionally with the effects of flooding. In general, flood protection will benefit the social cohesion of this community and the commitment on the part of its residents to the future.

b. Protection of commercial and industrial facilities will protect the livelihoods of their employees and the economic health of Davenport. Continued flooding and the threat of flooding would be likely to result in a gradual out-migration of businesses from the flood plain and perhaps from the city of Davenport. Such loss of businesses would cause economic dislocations in this part of Davenport and the entire city.

22. Threatened and Endangered Species

a. The bald eagle Haliaeetus leucocephalus is listed as an endangered species by the Department of the Interior* and is a common winter resident of the Davenport area. Eagles gather here from December to March to take advantage of feeding opportunities provided by the ice free water immediately downstream of Lock and Dam 15. Similar concentrations of eagles can be found at other locks and dams and open water areas on the Upper Mississippi River.

b. Within the project area, bald eagles use the large trees on Credit Island as daytime roosts. As many as 10 adult and juvenile eagles have been seen on the island on a winter day. The eagles primarily use the large trees along the riverward shore of the island. Eagle use occurs even though a park roadway passes within a short distance of some of the roosting trees.

c. On 25 May 1978 the Rock Island District requested that the Fish and Wildlife Service initiate formal consultation in accordance with Section 7 of the Endangered Species Act of 1973. One of the species named in the request was the bald eagle. In a letter dated 24 July 1978 the Fish and Wildlife Service responded that the proposed project will not jeopardize the continued existence of the bald eagle and no further consultation is necessary for this species. The Fish and Wildlife Service did request that the southern portion of Credit Island be protected from development as a condition of approval of the local protection project. That portion of the island will be set aside as a nature preserve and the existing roadway will be abandoned. Appendix I provides a discussion of development planned for Credit Island.

d. In addition to the bald eagle, two endangered and one proposed threatened mussel species could be found in this reach of the Mississippi River. These species are the fat pocketbook pearly mussel Proptera (Potamilus) Capax and the Higgins eye pearly mussel Lampsilus higginsi which are classified as endangered and the spectacle case Cumberlandia monodonta which is proposed as threatened. These species were named in the request that Section 7 consultation be initiated. The Higgins eye and the spectacle case have been found recently within this portion of the river although not in the reach where project dredging is proposed. There is no recent record of the fat pocketbook pearly mussel in this area.

e. Approximately 115 acres of river bottom will be disturbed in the main river and 88 acres in Credit Island Slough by dredging.

* Department of the Interior, Fish and Wildlife Service, "Endangered and Threatened Wildlife and Plants," Federal Register 42, No. 135, 14 July 1977, and subsequent updates.

Mussel sampling was conducted by the Fish and Wildlife Service in the project area in 1976,1977, and 1978. See Exhibit A (letter dated 8 August 1978) for a listing of species found. Please note that the reference to Proptera capax is incorrect. The correct scientific name for the species found is Lampsilus ovata ventricosa. Borrow area number seven has been deleted in order to avoid disturbance of its diverse mussel fauna. Borrow area number four has also been deleted because the sand it contains is not needed for construction. Borrow area number 1 will be used only if sufficient material is not found in the other sites.

f. In a letter dated 24 July 1978 the Fish and Wildlife Service requested that additional mussel sampling be conducted in order to conclusively determine whether any threatened or endangered species are present in the proposed dredging sites and the extent and nature of mussel beds in the vicinity of the dredging sites. This sampling will be undertaken as soon as funds are available. The Fish and Wildlife Service will be asked to participate in development of a Scope of Work for the sampling and will be consulted in regard to selection of a Contractor. At a meeting held on 18 September 1978 general requirements for the mussel sampling were discussed. See Exhibit A (letter dated 2 October 1978).

g. Results of the mussel survey will be provided to the Fish and Wildlife Service who will render an opinion concerning project impacts on the threatened and endangered mussel species. If adverse impacts are discovered, measures will be developed to avoid or mitigate these impacts. All coordination will be pursuant to the Endangered Species Act Amendments of 1973 as amended.

23. Archaeological and Historical

a. Prior to the archaeological surveys conducted in 1974 (Weichman and Stack) and 1976 (Overstreet), there were no intensive archaeological investigations conducted in the Davenport area. These two surveys, while intensive, were confined to the impact area of the proposed project and

the results should not be considered as a total representation of the cultural sequence for the area. However, it is possible with the data at hand to present a tentative cultural sequence.

b. While there is no direct evidence in the area to suggest occupation by either the Paleo-Indian or Archaic Cultures, there is a high probability they once inhabited the area. This conclusion is based on certain projectile points reported in local collections (Weichman and Stack, 1974, p. 29.)

c. Numerous mound groups and other cultural features were investigated by members of the Davenport Academy of Science during the late 19th century. Evaluation of the features observed and materials recovered in these reports suggests that the cultural sequence of the study area covers a chronological span from the Middle Woodland period to Historic-Aboriginal cultures, which spans approximately two thousand years.

d. Evidence for the occurrence of Woodland components within the project area is suggested by typical Woodland mortuary manifestations, reported within the project locality. Although conclusive diagnostic materials were not recovered from any site during the investigations these mound groups are generally indicative of a well-established Woodland occupation. Supportive evidence for this hypothesis is provided by 13ST8. The complex construction of the Cook's Farm Mound Group (135+7, 135+8, and 135+9) with burial furnishings which include plain and effigy platform pipes, copper implements, marine shell containers, worked and unworked bear canine teeth, cut mica and other diagnostic artifactual material, suggests that the local Woodland inhabitants either participated in a Hopewellian interaction sphere (Struever and Howart, 1964) or at least adopted certain aspects of that mortuary complex. Evidence for a strong Hopewell influence is provided by accounts and descriptions of several other mortuary manifestations excavated during the same period (Weichman and Stack 1974, pp. 29-30). There is no evidence in the area to suggest occupation by the Mississippian culture; however, subsequent investigations in the area may produce such evidence.

e. The Historic aboriginal components represented within the proposed project area appear to have better documentation than the earlier traditions. Ethno-historic accounts indicate that the Sauk and Fox occupied this portion of the Mississippi River valley during the 18th and 19th centuries. A substantial aboriginal population was apparently located within this area, with the principal Sauk settlement, Saukenauk, being situated on the Rock River near Vandruff's Island. During this same period Credit Island received its name, because the Indians came to the island to get supplies on credit to be paid for with the following season's catch. Cultural materials associated with historic components are undoubtedly located within this portion of the flood plain and in a substantial quantity. However, surface indications of these manifestations have been covered by subsequent Euro-American occupations and alluvial deposition. (Weichman and Stack 1974, p. 33)

f. The site of present-day Davenport was also used for Indian villages, according to some historians. There are many rumors that large villages occupied the north bank of the river prior to Marquette and Joliet although it is impossible to verify these reports. However, the south bank was used extensively by the Sauk and Fox for village sites. They continued to live in the area until 1832 and 1836 when land purchases took their traditional territory. The Winnebagos also were involved in these transactions, which were known as the "Black Hawk Purchase" and the purchase of "Keokuk's Reserve." Both of the treaties for these transactions were signed in Davenport.

g. Davenport became the earliest legal settlement west of the Mississippi River when the homesteading of the Wisconsin Territory (Iowa) opened up in 1833. The area soon developed into a major supply center which attracted extensive commercial river traffic. Brick remnants of an old steamboat levee are still visible at the riverfront of Main Street in LeClaire Park. The park itself dates back to the earliest days of the city and represents a historic setting of the early riverfront development. Over the years encroachment on the riverfront has been experienced including development of expanded parking areas and a stone seawall. More recently the area has been disturbed by construction of a sewer interceptor line. As such LeClaire Park has lost most of the historic integrity it once possessed, and warrants no further consideration at this time.

h. The National Register of Historic Places of 6 February 1979 and all subsequent revisions were consulted and the following properties are listed:

Ambrose Hall
Edward S. Barrow House
Leon Bismark (Bix) Beiderbecke House
Collins House
Frick's Tavern
Gilruth School House
Antoine LeClaire House
Outing Club
Schick Apartments
Siemer House
Trinity Episcopal Church

i. None of these properties will be affected by any project actions. One other property, Credit Island, the site of the Battle of Credit Island on 27 August 1814, was identified as being potentially eligible for inclusion on the National Register. In May 1980, the Rock Island District (RID) requested a determination of eligibility for LeClaire park and Credit Island for inclusion on the National Register of Historic Places. Neither site was found eligible. Copies of all correspondence, including the final determination by the Keeper of the National Register, were furnished to the State Historic Preservation Officer (SHPO) and the Advisory Council on Historic Preservation.

j. The Keeper of the National Register did comment that the LeClaire Park bandshell (Wilma D. Petersen Memorial Music Pavillion) might be individually eligible. Consequently, the SHPO was requested to make determination of the probable effect of the Davenport project on the bandshell. By letter dated 25 August 1980, the SHPO notified RID that there will be no impact and, therefore, no need to seek a determination of eligibility for the structure.

24. Executive Order 11988, Flood Plain Management

a. The objective of Executive Order 11988 is to avoid to the extent possible the long and short term adverse impacts associated with occupancy and modification of flood plains and to avoid direct and indirect support of flood plain development wherever there is a practicable alternative. The Order requires Federal agencies to provide leadership and take action to:

(1) avoid the base flood plain unless it is the only practicable alternative;

(2) reduce the hazard and risk of flood loss;

(3) minimize the impact of floods on human safety, health, and welfare; and,

(4) restore and preserve the natural and beneficial flood plain values. Direct support of flood plain development is an action in the flood plain that encourages, allows, serves, or otherwise facilitates additional flood plain development. An example of direct suport would be provision of flood protection measures to undeveloped or underutilized flood plain lands for the purpose of permitting future development and growth.

b. Several alternatives to the selected plan were scrutinized during the planning process and are discussed in the Phase I GDM and the final EIS. Protection of the small areas of land that are presently undeveloped is preferable to moving the proposed levee to exclude these areas. Loss of natural and beneficial resources through flood protecting areas is very limited. Impacts are limited because these areas are generally highly disturbed, are already serviced by urban utilities and they are surrounded or border developed areas. These areas posses little aesthetic, habitat, or recreational value. Developments of these areas would be a natural infilling of voids left in an otherwise highly urbanized area.

c. The proposed project would have an overall positive impact on the flood plain environment. In addition to benefits that will occur to the residents and employees of the area from a heightened level of security against flooding, actions will be taken to improve environmental conditions. These are preservation of Nahant Marsh, enhancement of Credit Island Slough by removal of accumulated silt and reclamation of an old municipal landfill. These actions offset the limited losses resulting

from protection of 121 acres of undeveloped land in the 2,260 acre protected area. Davenport's flood plain will be more attractive from an aesthetic, habitat and recreational standpoint.

d. The proposed project will provide a high degree of security against flooding. It will not, however, provide complete security. Residual damages, which are annualized damages from floods exceeding the project design, are estimated to be $477,300.

e. It is the opinion of the Rock Island District that the project is in compliance with Executive Order 11988 and with the flood plain protection standards of the State of Iowa and the city of Davenport. Letters have been received from the Iowa Natural Resources Council and from the city of Davenport expressing concurrence that the project satisfies state and local flood plain protection standards. Copies of these letters are incorporated into this report in Exhibit A, Pertinent Correspondence.

DAVIS/lmr/285

NCRED-PB

28 SEP 1979

Mr. James R. Webb, Director
Natural Resources Council
Wallace State Office Building
Des Moines, Iowa 50319

Dear Mr. Webb:

The final environmental impact statement and the Phase I General Design Memorandum for the Davenport, Iowa, Local Flood Protection Project were provided for review and comment in May 1978. In those documents, a determination was made that the project conforms to applicable state and local flood plain protection standards. Comments received did not, however, specifically indicate conformance. To satisfy the requirements of Executive Order 11988, Flood Plain Management, it is requested that a statement be furnished which indicates agreement or disagreement with the determination that the project conforms with flood plain protection standards.

Please let us know if additional information is required for your evaluation. Thank you for your attention to this matter.

Sincerely,

DOYLE W. McCULLY, P.E.
Chief, Engineering Division

CF:
Dist File (ED-PB)
ED
ED-O
ED-FP
ED-PR
ED-PD-ER

DAVIS/lmr/285

MCRED-PB

28 SEP 1979

Honorable Charles Wright
Mayor of Davenport
City Hall
Davenport, Iowa 52801

Dear Mayor Wright:

The final environmental impact statement and the Phase I General Design Memorandum for the Davenport, Iowa, Local Flood Protection Project were provided for review and comment in May 1978. In those documents, a determination was made that the project conforms to applicable state and local flood plain protection standards. Comments received did not, however, specifically indicate conformance. To satisfy the requirements of Executive Order 11988, Flood Plain Management, it is requested that a statement be furnished which indicates agreement or disagreement with the determination that the project conforms with flood plain protection standards.

Please let us know if additional information is required for your evaluation. Thank you for your attention to this matter.

Sincerely,

DOYLE W. McCULLY, P.E.
Chief, Engineering Division

CF:
Dist File (ED-PB)
ED
ED-D
ED-FP
ED-PB
~~ED-PB-LD~~ EA

1c

PETERSON/lmr/305

NCRED-D

13 AUG 1979

Mr. Rodney Vlieger
Iowa Department of Environmental
Quality
Henry A. Wallace Building
Des Moines, Iowa 50319

Dear Mr. Vlieger:

This is in reply to your letter of 7 June 1979, regarding work proposed at a former landfill site in the Davenport Local Flood Protection Project. The city of Davenport desires that the site be developed into a park as a part of the flood protection project. The major usage of the park would be for baseball and picnicking.

Site preparation would include some leveling and grading of the existing material, but the primary activity would be placing silt dredged from Credit Island Slough as an inert cover over the area. It is not anticipated that any material would be removed from the landfill site.

Development of the park is presently scheduled for 1983 to 1985. While our preliminary plans are essentially complete, detail planning must await congressional funding. We will coordinate further development of the plans with your office and will furnish details of all proposed activities regarding the site to you when further planning is undertaken.

Sincerely,

DOYLE W. McCULLY, P.E.
Chief, Engineering Division

CF:
Dist File (ED-D)
ED-PB-EA
ED-D-ST

iowa department of environmental quality

reply to: Rodney Vlieger
phone: 515/281-8914

June 7, 1979

Department of the Army
Rock Island District, Corps of Engineers
Clock Tower Building
Rock Island, IL 61201

ATTENTION: Colonel F. W. Mueller, Jr.

Dear Sir:

The Supplemental Information to the Final Environmental Impact Statement for the Davenport, Iowa, Local Flood Protection Project dated 24 April 1979 indicated that "Silt dredged from Credit Island Slough will be used to cap the sand levees and to cover the old city dump site". No further mention of the old dump site is made, and no assessment of the environmental impact of this action is indicated. Additionally, it is unclear whether there will be any excavation or disruption of deposited waste. A portion of the rules of the Solid Waste Disposal Commission applies to disruption and excavation of deposited waste and reads as follows:

> 400--26.7(455B) Disruption and excavation of sanitary landfills or closed dumps. No person shall excavate, disrupt, or remove any deposited material from any active or discontinued sanitary landfill or closed dump without having first notified the department in writing.
>
> 26.7(1) Notification shall include an operational plan stating the area involved, lines and grades defining limits of excavation, estimated number of cubic yards of material to be excavated, sanitary disposal project where excavated material is to be disposed and estimated time required for excavation procedures.
>
> 26.7(2) An excavation shall be confined to an area consistent with the number of pieces of digging equipment and trucks used for haulage.
>
> 26.7(3) Adequate measures shall be taken during excavation to control dust, odors, fires, rodents, insects, and blowing litter.
>
> 26.7(4) The disposal of all solid waste resulting from excavation shall be in conformity with chapter 455B and these rules.

We would appreciate receiving detailed plans for any disruption or excavation of the old dump site as well as the assessment of the environmental impact of such activity. Please note that Departmental approval of any excavation of a closed dump is required prior to initiation of the planned activity.

If you have any questions regarding Departmental requirements relating to this project, please feel free to contact Rod Vlieger (515/281-8914) or me.

Sincerely,

AIR AND LAND QUALITY DIVISION

B. Z. Karachiwala, P.E.
Chief, Land Quality Branch

BZK:RV:mjk

cc: DEQ Regional Office No. 6 - Washington, Iowa
Al Carpenter, DEQ

NCRPD-PB

McLEAN/gms/308 14 JUN 1978

Dr. Adrian Anderson, Director
State Historic Preservation Officer
Division of Historic Preservation
Iowa State Historical Department
26 East Market Street
Iowa City, Iowa 52240

Dear Dr. Anderson:

Inclosed is a nomination form for determination of eligibility of Credit Island for the National Register of Historic Places. This is being forwarded for your evaluation and recommendation. Also inclosed is a letter from the Director, Davenport Park and Recreation Department, with whom this action has been coordinated.

Credit Island will be impacted by the proposed Davenport, Iowa, Local Flood Protection project. A portion of the proposed levee will be constructed on Credit Island, as described in the FEIS for the Davenport LFP project (1976).

In order to resolve public concern about the cultural significance of Credit Island, all comments and recommendations will be forwarded to the Heritage Conservation and Recreation Service for final evaluation.

Thank you for your cooperation.

Sincerely,

2 Incl
As stated

DOYLE W. McCULLY, P.E.
Chief, Engineering Division

CF:
Dist file (ED-PB) (wo/incl)
ED (wo/incl)
ED-PB (wo/incl)
~~ED-D (wo/incl)~~ Structural
ED-PB (EA) (w/incl)

NCRED-D LOSS/lmr/347

Mr. Thomas M. Groutage
Field Supervisor
Fish and Wildlife Service
Rock Island Field Office (ES)
1830 Second Avenue
Rock Island, Illinois 61201

Dear Mr. Groutage:

This letter is in response to your letters, both dated 6 April 1979 regarding the Davenport, Iowa, and Union Township, Missouri, projects. On 23 April 1979, Ms. Gail Peterson and Mr. Bruce Klein, of your office, met with representatives of this office and discussed the two projects in the light of your letter.

A letter has also been received from Region VII, US Environmental Protection Agency, regarding the Davenport project (Inclosure 1). In that letter EPA concluded that discharge of dredged materials to the Mississippi River is acceptable since, for the few cases where concentrations are higher than recommended limits, dilution in the mixing zone will be adequate. Ms. Peterson of your office said that since the elutriate tests are acceptable to the EPA, she did not feel the FWS would object.

To date, we have not received a similar letter on Union Township. However, EPA's comments on the Davenport project would seem to apply also to the Union Township project. That is, there will be adequate dilution in the mixing zone for those areas at Union Township in which the level of phenols tested did exceed the recommended limits. It is noted that phenols in the ambient water were also above the recommended limit. The highest ammonia nitrogen concentration was measured to be .07 mg/l. This is significantly less than the 9.4 mg/l found at Davenport, which EPA found acceptable. The manganese concentrations measured were all lower than the recommended limit.

With regard to the detection limits used for the pesticide and PCB analyses, we again make reference to EPA's letter on the Davenport project.

NCRED-D
Mr. Thomas H. Croutage

The "Quality Criteria for Water" publication is for specific uses. Factors such as the dilution due to having a mixing zone in which the momentary disturbance of these sediments does not create a chronic level of toxicity, the ambient water quality, and the particular species of aquatic life that would be in contact with this water must be considered. We have followed accepted recommended limits for the elutriate tests and question whether lower detection limits would be justified.

National Pollution Discharge Elimination System Permits (Section 402) are not required for projects which involve the discharge of dredged or fill material and have been processed under Section 404. The discharge on Burgens Creek is less than 5 c.f.s. and work therein is covered under the Nationwide permit program.

The 404 Evaluation Report for the Davenport project was signed by the District Engineer on 4 January 1979 and included as part of the Supplemental Information to the FEIS for the Davenport LFP Project. This Supplemental Information was filed with EPA on 2 April 1979. The Final 404 Evaluation Report for the Union Township project has been prepared and is presently undergoing in-house review. After signature by the District Engineer, it will be placed on file and will be available for review upon request.

Thank you for your review, and if you should have any additional comments please call upon us.

Sincerely,

1 Incl
As stated

F. W. MUELLER, Jr.
Colonel, Corps of Engineers
District Engineer

CF:
Dist File (ED-D)
ED
OD-S
ED-H
ED-PE(EA)
ED-D (Dav. Files) Peterson
ED-D (U.T. Files)

CITY OF DAVENPORT
PARK AND RECREATION DEPARTMENT
2816 Eastern Avenue
Davenport, Iowa 52803
Telephone 326-7812

May 8, 1979

Mr. Doyle W. McCully, P.E.
Chief, Engineering Division
Rock Island District
Corps of Engineers
Rock Island, Illinois 61201

Dear Mr. McCully:

Thank you for your communication regarding the nomination of Credit Island Park to the National Register of Historic Places. I have reviewed the Nomination Form and personally surveyed the Park with regard to its historical significance.

It is my opinion considering the condition of the Park, its current and potential usage, and the ultimate advantages of pursuing this nomination that it would not be prudent for the City of Davenport to grant permission for this action to proceed.

Please contact me if I may be of further assistance and thank you for your coordination in this matter.

Sincerely,

Wayne S. Boyer
Director

WSB/slc
cc: Del D. Borgsdorf
City Administrator

(4

UNITED STATES ENVIRONMENTAL PROTECTION AGENCY

REGION VII
324 EAST ELEVENTH STREET
KANSAS CITY, MISSOURI - 64106

OFFICE OF THE
REGIONAL ADMINISTRATOR

APR 13 1979

Lt. Colonel George F. Boone, USA
Assistant Director of Civil Works,
Environmental Programs
U.S. Army Corps of Engineers
Office of the Chief of Engineers
Washington, D.C. 20314

Dear Colonel Boone:

We have reviewed the supplemental information on the Section 404(b)(1) evaluation for the Davenport, Iowa Local Flood Protection project transmitted by your April 2, 1979, letter.

Our evaluation indicates that the proposed project is in compliance with the 404(b) guidelines and meets the conditions of Section 404(r).

We are available for consultation in the event that the dredged or fill material is contaminated or could cause environmental problems. Please contact Mr. Robert Koke, Chief, 404 Section, for this consultation.

Sincerely yours,

Kathleen Q. Camin, Ph.D.
Regional Administrator

cc: Rock Island District, COE

UNITED STATES ENVIRONMENTAL PROTECTION AGENCY

REGION VII
324 EAST ELEVENTH STREET
KANSAS CITY, MISSOURI - 64106

April 12, 1979

Mr. Doyle W. McCully
Chief, Engineering Division
U.S. Army Engineer District, Rock Island
Clock Tower Building
Rock Island, Illinois 61201

Dear Mr. McCully:

We have reviewed the water quality report on the Local Protection Project at Davenport, Iowa, transmitted by your March 1, 1979, letter. We offer the following comments for your consideration.

1. EPA "Quality Criteria for Water" referenced in your report is for specific uses and the limits established are for the stream. The effluent from the dredge material settling ponds will be allowed a mixing zone, except for the toxic materials that may be present in the discharge. A direct comparison of the numbers is not entirely correct without establishing a mixing zone.

2. The elutriate results for phenols are almost identical to the ambient water. This indicates that the sediment is not adding phenols to the water.

3. The ammonia concentration is reported as ammonia nitrogen. The recommended limit is based on un-ionized ammonia which is much less than ammonia nitrogen. The amount of ammonia nitrogen in the un-ionized portion depends on the temperature and pH. For a temperature of $0^{o}C$ and a pH of 7.6, Table 2, page 10 of EPA Quality Criteria for Water (July 1976) indicates elutriate tests 1, 2, 3 and 4 are below the recommended maximum limits; elutriate tests 5 and 6 are very near the recommended maximum limits.

Based on the data provided by your report, discharge to the Mississippi River without pretreatment is acceptable. In the few cases where concentrations are higher than the recommended limits, dilution in the mixing zone will be adequate.

Sincerely yours,

Robert J. Koke, P.E.
Chief, 404 Section

cc: FWS, Rock Island
DEQ

(6)

United States Department of the Interior

FISH AND WILDLIFE SERVICE
IN REPLY REFER TO:
ROCK ISLAND FIELD OFFICE (ES)
1830 SECOND AVENUE Com: 309-788-3991/3925
ROCK ISLAND, ILLINOIS 61201 FTS: 360-9217/9274

April 6, 1979

Colonel Frederick W. Mueller
District Engineer
U.S. Army Engineer District
Rock Island
Clock Tower Building
Rock Island, Illinois 61201

Dear Colonel Mueller:

This is in reference to Mr. McCully's letter dated March 1, 1979, transmitting a water quality report for the Davenport Local Flood Protection Project.

We note that phenol, ammonia nitrogen, and manganese are over the standard limits for many of the samples evaluated. Although these results are not present in toxic quantities, some physiological and pathological effects to aquatic resources may occur. Phenol concentration greater than 1 ug/l may cause tainting of fish flesh locally and ammonia nitrogen concentrations greater than 0.2 mg/l may exert adverse physiological effects.

We question the detection limits used for the analysis for pesticides and PCBs. For dieldrin, endrin, methoxychlor, toxaphene, and PCBs, the recommended limits identified in the U.S. Environmental Protection Agency's "Quality Criteria for Water" are on the order of 10 to 100 magnitudes less than the analysis results. Since these compounds are highly susceptible to bioaccumulation and may be toxic to aquatic resources, we request that these parameters be re-evaluated.

In addition, we suggest that you contact the Iowa Department of Environmental Quality to determine if a National Pollution Discharge Elimination Permit is necessary for this project.

Thank you for the opportunity to review this report.

Sincerely yours,

Thomas M. Groutage

Thomas M. Groutage
Field Supervisor

United States Department of the Interior

FISH AND WILDLIFE SERVICE
Federal Building, Fort Snelling
Twin Cities, Minnesota 55111

IN REPLY REFER TO:
AFA-SE

DEC 1 1978

Colonel Daniel L. Lycan
District Engineer
U. S. Army Engineer District
Rock Island
Clock Tower Building
Rock Island, IL 61201

Dear Colonel Lycan:

On November 10, President Carter signed into law the Endangered Species Act Amendments of 1978. A provision of the Amendments is that all consultations are to be completed within 90 days after initiation or within a time period mutually agreeable to the Federal agency and the Fish and Wildlife Service.

Your agency and the Service are currently in consultation (Davenport Local Flood Protection Project) and the issuance of a biological opinion has been delayed pending completion of a survey. It is the Service's belief that before we can render an opinion on the project other than "may jeopardize," the survey should be completed. Please advise if it is agreeable to your agency to extend the consultation period so that results from the study are available to the Service for consideration in developing our biological opinion.

If you have further questions on this matter, please contact the Regional Endangered Species Office (telephone 612-725-3596).

Sincerely yours,

D.H. Rasmussen

(8)

iowa department of environmental quality

reply to Wayne C. Farrand, P.E.
phone: 515/281-8983

October 24, 1978

District Engineer
U.S. Army Corps of Engineers
Rock Island District
Clock Tower Building
Rock Island, IL 61201

ATTENTION: Permits & Statistics Branch

RE: Certification No. 78-6-CE10
Corps of Engineers Project; Local Flood
Protection for Davenport, Iowa
Mississippi River and Blackhawk Creek

Dear Sir:

We have reviewed the request for State certification of the above-referenced project as required by Corps regulations and the Clean Water Act Section 404.

This letter certifies, subject to the following conditions, under provision of Section 401 of the Clean Water Act, that this Department as the State Water Quality Agency has determined that there is reasonable assurance the proposed activity will be conducted in a manner which will not violate water quality standards of the State of Iowa.

1. Construction methods undertaken in the project shall minimize water quality impacts and maintain Iowa Water Quality Standards.

2. The applicant must assure that upon completion of earthwork operations, all fills in the watercourse or onshore and other areas onshore disturbed during construction will be seeded, riprapped or given some other type of protection from subsequent soil erosion.

3. Special care must be taken to avoid any spillage of oils, fuels, or other types of hazardous chemicals or pollutants while working within or along the banks of the river. Specific plans should be formulated in advance of construction to contain such spills in the event of any contingency.

Sincerely,

CHEMICALS AND WATER QUALITY DIVISION

Wayne C. Farrand, P.E.
Construction Grants Section

WCF:mla
cc: DEQ Regional Office No. 6, Washington
Doyle W. McCully, P.E., Chief Engineering Division, Rock Island District, Rock Island, Illinois

(9)

United States Department of the Interior

FISH AND WILDLIFE SERVICE
Federal Building, Fort Snelling
Twin Cities, Minnesota 55111

IN REPLY REFER TO:
AFA-SE

OCT 2 1978

Colonel Daniel L. Lycan
District Engineer
U. S. Army Engineer District
Rock Island
Attention: Environmental Resources
Branch
Clock Tower Building
Rock Island, Illinois 61201

Dear Colonel Lycan:

On September 18, representatives from the Corps of Engineers, Rock Island District, and the U. S. Fish and Wildlife Service met at Rock Island to discuss the scope of work necessary to evaluate potential effects upon endangered mussels which might result from the proposed Davenport Flood Protection Project. Six sites were proposed as potential areas to be dredged to provide material for the dike system. In addition, Davenport harbor would be dredged to better accommodate boat traffic.

The following survey needs were discussed by the individuals in attendance:

1. Dredge sites 1, 2 and 3 were to be intensively surveyed for mussels. The extent and composition of beds lying within these sites were to be delineated. Surveys were to extend for approximately 1,000 feet beyond the dredge site.

2. Dredge sites 4, 5, and 6 were to be surveyed but in a more cursory manner.

3. Davemport Harbor should be surveyed. A preliminary survey could be accomplished by the Rock Island Office, U. S. Fish and Wildlife Service. This would determine a need for further survey efforts.

4. Wing dams within sampling parameters defined in item 1 should be surveyed for mussels.

Attached is a copy of the Preliminary Report on Clam Survey on Dubuque Bridge Site as requested by Mr. Bill Davis.

If any additional questions arise please contact the Office of Endangered Species, FTS 725-3596.

Sincerely yours,

Charles A. Hughlett

Charles A. Hughlett
Acting Regional Director

Attachment

United States Department of the Interior

FISH AND WILDLIFE SERVICE
U.S. FISH & WILDLIFE SERVICE
ROCK ISLAND FIELD OFFICE
1830 - 2nd AVENUE
ROCK ISLAND, ILL. 61201

IN REPLY REFER TO:

Com: 309-788-3991/3925
FTS: 360-9217/9274

August 8, 1978

Col. Daniel L. Lycan
District Engineer
U.S. Army Engineer District
Rock Island
Clock Tower Building
Rock Island, Illinois 61201

Dear Colonel Lycan:

Enclosed is the mussel sampling data we have collected for the Davenport Local Flood Protection Project. This information should be helpful in completing the Section 7 consultation process for the proposed project. No further sampling in this area is anticipated by this office.

Sincerely yours,

John Popowski
Field Supervisor

Enclosure

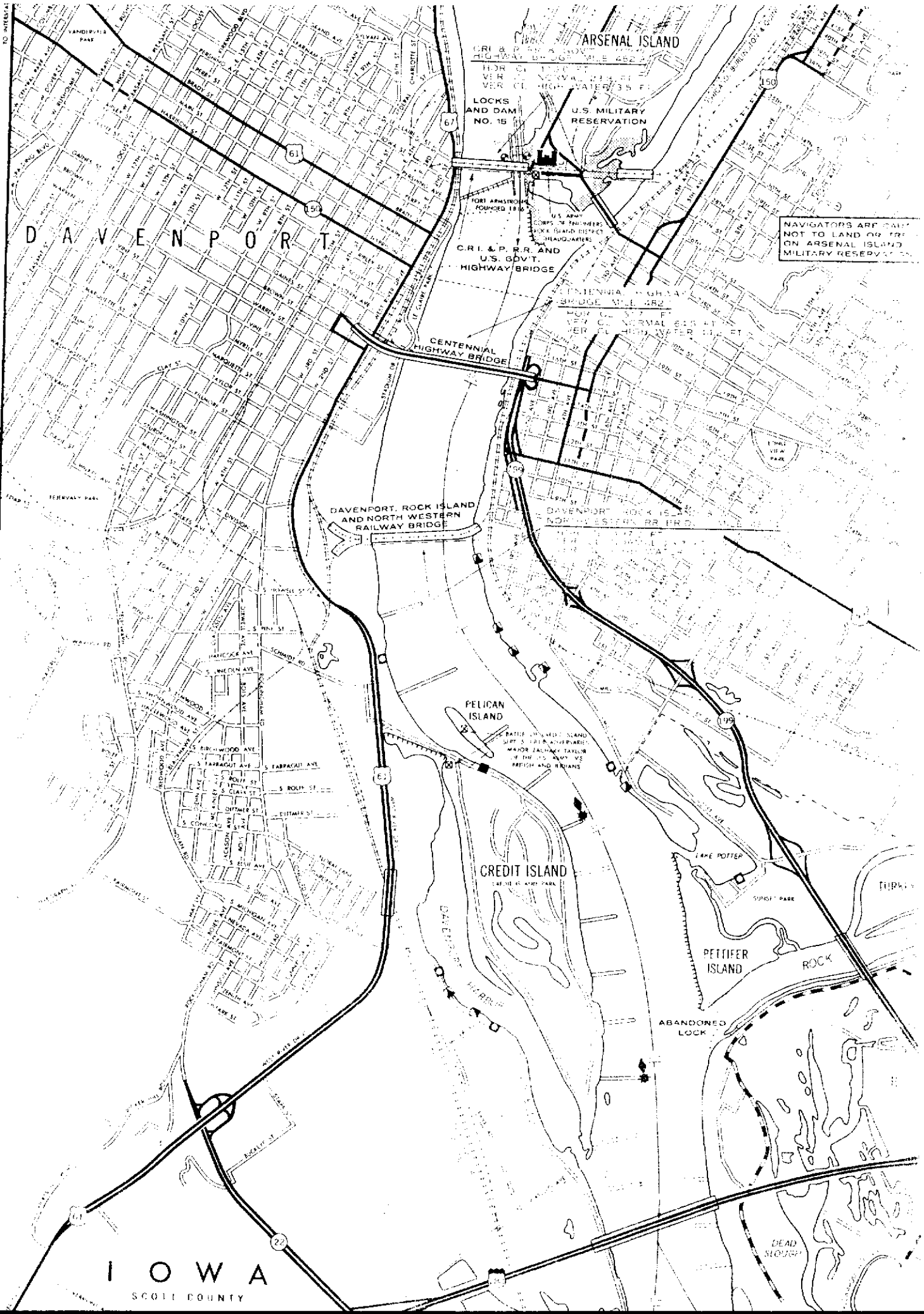
MUSSEL COLLECTION DATA FOR DAVE[illegible]T LOCAL FLOOD PROTECTION PROJECT

Borrow Area	Date	Transect Location	Catch	Substrate	Species Composition
1	9/30/77	a. 75' from RB	a. 1st drag: 19 mussels 2nd drag: 43 mussels	a. gravelly	Three ridge (Amblema plicata) Pimpleback (Quadrula pustulosa) Pocketbook (Proptera capax) Pig-Toe (Fusconaia flava)
		b. 150' from RB	b. 1st drag: 0 mussels 2nd drag: 0 mussels	b. soft muck	
2	No data available				
3	8/23/76	a. 150' from RB	a. 1 mussel	a. sand	Threeridge (Amblema plicata) Pocketbook (Proptera capax)
		b. 400' from RB	b. 10 mussels	b. gravel	
		c. 300' from RB	c. 0 mussels		
4	8/23/76	a. 400' from RB	a. 8 mussels	a. unknown	Unknown
		b. 300' from RB	b. 1st drag: 13 mussels 2nd drag: 11 mussels	b. gravel - black sediments	
		c. 150' from RB	c. 0 mussels	c. unknown	
5	6/27/78	a. 200' from RB	a. 0 mussels	sand[1]	
		b. 300' from RB	b. 0 mussels		
		c. 250' from RB	c. 0 mussels		
6	6/27/78	a. 200' from RB	a. 0 mussels	sand[1]	
		b. 250' from RB	b. 0 mussels		
		c. 300' from RB	c. 0 mussels		
7	6/27/78	a. 200' from RB	a. 0 mussels	sand[1]	Hickory nut (Obovaria olivaria) Pig-Toe (Fusconaia flava) Mucket (Actinonaias carinata) Three ridge (Amblema plicata) Pocketbook (Proptera capax) Maple Leaf (Quadrula quadrula) Fragile Paper Shell (Leptodea fragilis) Warty back (Quadrula nodulata) Pimpleback (Quadrula pustulosa)
		b. 350' from RB	b. 59 mussels		
		c. 500' from RB	c 3 mussels		

MUSSEL COLLECTION DATA FOR DAVENPORT LOCAL FLOOD PROTECTION PROJECT

Borrow Area	Date	Transect Location	Catch	Substrate	Species Composition
8	6/27/78	a. 40' from shore-between 2 islands	a. 1 mussel	soft silt[1]	Threeridge (Amblema plicata)
		b. Between small island and navigation channel	b. 0 mussels		

[1] Surface of borrow area consists of 1 to 12 feet of sand. Actual substrate of sampling sites were not taken.


ARSENAL ISLAND
LOCKS AND DAM NO. 15
U.S. MILITARY RESERVATION
FORT ARMSTRONG FOUNDED 1816
C.R.I. & P. R.R. AND U.S. GOV'T. HIGHWAY BRIDGE
CENTENNIAL HIGHWAY BRIDGE
DAVENPORT
DAVENPORT, ROCK ISLAND AND NORTH WESTERN RAILWAY BRIDGE
PELICAN ISLAND
CREDIT ISLAND
PETTIFER ISLAND
ROCK
ABANDONED LOCK
DEAD SLOUGH
LAKE POTTER
SUNSET PARK
IOWA
SCOTT COUNTY

United States Department of the Interior

FISH AND WILDLIFE SERVICE
Federal Building, Fort Snelling
Twin Cities, Minnesota 55111

IN REPLY REFER TO:
AFA-SE

JUL 24 1978

Colonel Daniel L. Lycan
District Engineer
U. S. Army Engineer District
Rock Island
Clock Tower Building
Rock Island, Illinois 61201

Dear Colonel Lycan:

This refers to your letter dated May 25, 1978, requesting formal consultation to meet the requirements of Section 7 of the Endangered Species Act of 1973 for the proposed Davenport LOCAL Flood Protection Project on the Mississippi River and Blackhawk Creek at Davenport, Iowa. The proposed project may have an effect on three endangered species and one species proposed for listing.

The bald eagle (Haliaeetus leucocephalus) is a common winter resident in the project area. This species utilizes both Credit Island and Pelican Island as feeding roost sites during the winter months and during the fall and spring migrations. Approximately 10 adult eagles and several immatures may be sighted on the island on any winter day.

The proposed flood control project includes a plan for recreational development of Credit Island. This development may intensify winter recreational use of the island and as a result affect the bald eagles' roosting areas. Adverse development may include removal of timber on the island, snowmobiling, and increased human encroachment in the southern portion of the island.

It is our opinion that in perspective, intensive development on Credit Island will not jeopardize the continued existence of the bald eagle. However, in the interest of the species, we encourage you to provide protection to the southern portion of Credit Island as a condition of approval of the local flood protection project. The authority for this is given in Section 2(c) and Section 7 of the Act which call for the conservation of endangered species and threatened species by federal agencies. No further consultation is necessary for this species.

In addition to the bald eagles, two endangered mussel species may be located in the project area. These are the fat pocketbook pearly mussel

(Potamilus capax) and Higgins eye pearly mussel (Lampsilis higginsii). In addition, the spectacle-case mussel (Cumberlandia monodonta), a proposed threatened species, may be in the project impact area. If located in the area, these mussels may be affected by dredging borrow material for the proposed levees and by the ensuing degradation of water quality. Eight potential borrow areas have been identified in the area from Centennial Bridge (RM 482.1) downstream to the tip of Credit Island (RM 478.6). (See figure 1) To date, our Rock Island Field Office has sampled portions of each borrow area except Borrow Area 2. Information obtained from this sampling has indicated that several shellfish beds are established in two or more of the borrow sites. These beds appear to be well diversified and of various age classes.

Neither of the endangered species were identified during the samplings; however, this does not negate the possibility that the species may be located in the vicinity of the Centennial Bridge (Fuller, 1978). In addition, the proposed borrow areas contain suitable substrates for mussel habitat.

Based on the above information, it can not be determined whether endangered mussels will be jeopardized by the flood control project, as proposed. Therefore, at this time, information necessary to render a biological opinion is not available. We request you conduct a survey sufficient in scope to determine if listed species occur within the impact area of borrow sites. Sixty days after receipt of this information, we will issue a biological opinion.

If further information is required, please contact the office of Endangered Species, FTS 725-3596.

Sincerely yours,

Charles A. Hughlett

Charles A. Hughlett
Deputy Regional Director

cc: Iowa Conservation Commission
U. S. Environmental Protection Agency
(Kansas City)

(12)

DAVIS/jav/285

NCRED-PB

1 . JUN 1978

Mr. Fred A. Priewert, Director
Iowa Conservation Commission
Wallace State Office Building
Des Moines, Iowa 50319

Dear Mr. Priewert:

Thank you for your comments, dated 26 April 1978, on the Davenport Local Flood Protection Project which were forwarded by the Office of Planning and Programming. Nearly all the recommendations contained in your letter have been incorporated into project planning as a result of previous coordination with the Conservation Commission and the US Fish and Wildlife Service. Your comments are addressed in following paragaphs in the same format as in your letter.

Black Hawk Creek Channel

Low stone baffles will be placed at approximately 500 foot intervals in the creek channel. These structures will create a series of pools and riffles, and a meandering low flow channel will be cut. Water-tolerant grass species will be established in the channel bottom, and vine species will be planted at the top of the gabions. Landscaping with shrubs and trees will be done on the upper banks of the channel where compatible with the wishes of the property owners. The design of the baffle structures, the low flow channel configuration, and selection of plant species will be coordinated with the city of Davenport, Iowa, the Conservation Commission, and the US Fish and Wildlife Service. The creek bottom will be left in natural soil with the exception of approximately a 300-foot reach in the vicinity of the Clark Street bridge. It is necessary to line this reach with concrete to prevent exposed soft rock from eroding, which could lead to sloughing of the channel sides.

Mississippi River Dredging

The proposed borrow sites in the Mississippi River will be sampled to determine the nature and concentration of any contaminants which may be present in the bottom sediments. Mussel and benthic populations in the proposed dredging sites will also be sampled. The US Fish and Wildlife

Service will conduct the mussel and benthic sampling. If either the bottom sediment or benthic sampling indicates a need, plans will be developed in coordination with the Conservation Commission and the Fish and Wildlife Service to minimize impacts to the resources present.

Credit Island Slough

Dredging plans for Credit Island Slough will be developed in coordination with all interests. Dredging will be accomplished in a fashion that will provide maximum fish and wildlife and recreation benefits. The proposed plan does not include fishing docks or jetties. A crushed stone walk will provide convenient fishing access. A small boat harbor is not included in the plan. Instead, a boat launching ramp will be constructed on the landward or northwest side of the slough.

Dredged material will be temporarily placed on Credit Island before being moved to other project areas. Placement will be done in a fashion to minimize impacts on shrubs and trees, as well as in a manner compatible with the recreational uses proposed by the city.

Rock Island District has requested the Fish and Wildlife Service to initiate the formal consultation process set forth in Section 7 of the Endangered Species Act of 1973. The species named in the request are the bald eagle and the mussel species Potamilus capax, Lampsilis Ligginsii, and Cumberlandia monodonta. If it is determined that the proposed project could adversely affect any of the species, a plan will be developed in coordination with the Conservation Commission to protect these resources.

It is not anticipated that accumulation of toxic pollutants will be a problem during these periods when Credit Island Slough is used for ponding. Ponding will occur infrequently and be of relatively short duration. The large volume of water in the slough and water movement resulting from operation of the pumping station should dilute any pollutants entering the slough from Black Hawk Creek.

Nahant Marsh

Assuming authorization of the addition of preservation of Nahant Marsh as a project purpose, the city of Davenport will be required to purchase the marsh and to operate and maintain it. The operation and maintenance plan will be developed in coordination with the Conservation Commission and other interests. Provision will be made by means of a low levee and control structure to allow drainage to continue to flow into the marsh under normal conditions.

NCRED-PB
Mr. Fred A. Priewert, Director

1 : JUN 1976

Seawall Fishing

Fishing opportunities existing on the Davenport seawall will be maintained. The collapsible floodwall will be set back from the seawall. Landscaping and provision of features such as benches should improve the convenience and quality of fishing opportunities.

Please let us know any further questions or comments concerning the proposed Davenport Local Flood Project. The Rock Island District will continue to coordinate all aspects of the project with the Conservation Commission.

Sincerely yours,

DANIEL L. LYCAN
Colonel, Corps of Engineers
District Engineer

CF:
Dist File
ED
ED-D
✓ED-D (Gen Eng)
ED-PB
ED-PB (KRS)

(13)

PETERSON/cak/236

1 JUN 1978

NCRED-D

Mr. Jack Hemphill
Regional Director
Fish & Wildlife Service
Federal Building, Fort Snelling
Twin Cities, Minnesota 55111

Dear Mr. Hemphill:

This is in reply to your letter LWR, dated 25 April 1978 which contained recommendations to improve fish and wildlife resources in conjunction with the proposed Davenport flood control project.

In the Blackhawk Creek channel, low stone baffles will be constructed at approximately 500-foot intervals. Water-tolerant vegetation will be planted in the channel bottom and vines will be planted near the top of the gabions at appropriate locations. Landscaping with suitable trees and shrubs will be accomplished on the banks of the channel where compatible with the interests of the property owners.

Soil explorations have shown that a section of soft rock will be exposed in the vicinity of the Clark Street bridge when the channel bottom is excavated. To protect the soft rock from erosion, which might lead to sloughing of the channel sides, the soft rock will be covered with a concrete lining. The extent of such lining will be limited to only that necessary to insure stability of the completed channel.

Borrow areas proposed in the Mississippi River and Credit Island Slough will be analyzed to determine the extent of possible contaminants and results will be furnished to your office and to the Iowa Conservation Commission. Benthic populations in the proposed borrow areas will be surveyed in cooperation with your Rock Island Field Office under the FY 78 Transfer Fund Agreement and plans developed as necessary to minimize impacts to the natural resources of the borrow areas.

Dredging of the slough will be coordinated with the ICC and your agency to minimize impacts on shore birds. In general, the proposed dredging will be in the deep water portions of the slough away from the landside shore. The dredged material, which will be placed temporarily on Credit

Island, will be stored so as to minimize impact to the island and in a manner compatible with the recreational use proposed by the city.

The city of Davenport will be required, as part of the basic cooperation for the flood control project, to purchase all necessary rights-of-way and to operate and maintain the system. Assuming authorization of the addition of preservation of Nahant Marsh as a project purpose, as I have recommended to our higher authority, the city will be required to purchase the marsh acreage and to operate and maintain it. The operation and maintenance plan will be formulated in cooperation with your agency, the ICC, and other appropriate groups.

Provision will be made in the low levee and other control features proposed as part of the marsh preservation system to allow drainage to continue to flow into the marsh under normal conditions, although the 700 acres of watershed outside the marsh itself will not necessarily be controlled to enhance the quality and quantity of flow into the marsh. Part of the plan for preservation of the marsh will be to provide a dependable water supply by pumping from the river, if necessary, during critical periods when there is not enough runoff to support marsh habitat.

By separate letter, I have requested that you initiate formal consultation under the Section 7 process (43FR870) to insure that the proposed activities will not jeopardize the existence of endangered species or critical habitat.

Sincerely yours,

DANIEL L. LYCAN
Colonel, Corps of Engineers
District Engineer

Copy furnished:
Mr. Fred A. Priewert, Director
Iowa Conservation Commission
Wallace State Office Building
Des Moines, Iowa 50319

Mr. Robert J. Koke
Chief, Federal Activities Program
United States Environmental Protection Agency
Region VII
1735 Baltimore
Kansas City, Missouri 64108

Dist file
ED
ED-D (Gen Eng Sec)
ED-PB (ERS)
OD-S

DAVIS/jav/285

NCRED-PB

25 MAY 1978

Mr. Jack Hemphill
Regional Director
US Fish and Wildlife Service
Federal Building, Fort Snelling
Twin Cities, MN 55111

Dear Mr. Hemphill:

Pursuant to 50 C.F.R. 402.4(n)(3) promulgated under Section 7 of the Endangered Species Act of 1973, this District has informally consulted with the Rock Island Field Office (ES) of the US Fish and Wildlife Service concerning possible impacts of the proposed Davenport, Iowa, Local Protection Project. Based on these discussions and on comments received on the final environmental impact statement from the Office of the Secretary, Department of the Interior, we request initiation of formal consultation procedures on the impacts of the proposed project.

Three endangered and one threatened species could possibly be affected. The species are the bald eagle, Haliaeetus leucocephalus, and the mussel species Potamilus capax, Lampsilis higginsii, and Cumberlandia monodonta.

Please direct any questions concerning the proposed project and coordination of the consultation process to Mr. Frank Collins, Chief, Environmental Resources Section, telephone 309/788-6361, extension 285.

Sincerely yours,

DANIEL L. LYCAN
Colonel, Corps of Engineers
District Engineer

CF:
Dist File
ED
SD-D
ED-D (Gen Eng)
ED-PB
✓ ED-PE (ERS)

NCRED-D

PETERSON/cak/236
24 MAY 1978

Mr. Robert J. Koke
Chief, Federal Activities Program
US Environmental Protection Agency
Region VII
1735 Baltimore
Kansas City, Missouri 64108

Dear Mr. Koke:

This is to acknowledge receipt of your letter of 28 April 1978, expressing no objection to the proposed Davenport flood control project and an interest that construction be performed in a manner to minimize turbidity increases in the adjacent waters.

Since part of the plan involves removal of material from the Mississippi River and Credit Island Slough, we will sample the bottom sediments to determine the existence or extent of deleterious substances. Results of the sampling and the plans for construction activities will be coordinated with your office as they are developed.

Sincerely yours,
Signed by:
GEORGE J. DOUGHERTY
LTC, Corps of Engineers
Deputy District Engineer

DANIEL L. LYCAN
Colonel, Corps of Engineers
District Engineer

Copy furnished:
Department of Interior
US Fish & Wildlife Service
1504 - 3rd Avenue
Rock Island, Illinois 61201

Mr. Fred A. Priewert, Director
Iowa Conservation Commission
Wallace State Office Building
Des Moines, Iowa 50319

Dist file
ED
ED-PB (ERS)
ED-D (Gen Engr Sec)
OD-S

(15)

UNITED STATES ENVIRONMENTAL PROTECTION AGENCY

REGION VII
1735 BALTIMORE
KANSAS CITY, MISSOURI 64108

April 28, 1978

Colonel Daniel L. Lycan, USA
District Engineer
U.S. Army Engineer District, Rock Island
Clock Tower Building
Rock Island, Illinois 61201

Attention: Regulatory Functions Branch

Dear Colonel Lycan:

We have reviewed your Public Notice No. 0664, dated March 28, 1978, regarding the Corps of Engineers Rock Island District, proposal to construct new levees and floodwalls, gatewells and closure structures, drainage structures and pump stations along the Mississippi River and channel improvements along Blackhawk Creek in conjunction with the Local Flood Protection Project for Davenport, Iowa.

The Environmental Protection Agency has no objections to the proposed project. The construction activities should be performed in a manner to minimize turbidity increases in the waterway which degrade water quality and damage aquatic life.

We appreciate the opportunity to review this public notice.

Sincerely yours,

Robert J. Koke

Robert J. Koke, P.E.
Chief, Federal Activities Program

cc: U.S. Fish and Wildlife Service, Rock Island
Iowa Conservation Commission

United States Department of the Interior

OFFICE OF THE SECRETARY
WASHINGTON, D.C. 20240

PEP ER-74/1418

APR 28 1978

Colonel Daniel L. Lycan
District Engineer
Corps of Engineers
Clock Tower Building
Rock Island, Illinois 61201

Dear Colonel Lycan:

The Department of the Interior has reviewed the final environmental statement for Davenport Local Flood Protection Project, Davenport, Iowa. Since the final statement was prepared several species which may be found in the project area have been placed on the endangered species list.

Based on our review of the document, the endangered species Potamilus capax and Lampsilis higginsii and the threatened species Cumberlandia monodonta could be impacted by dredging for borrow materials in the Mississippi River. Use of the river for borrow is a project design change since our review of the draft statement.

In addition the bald eagle, Haliaeetus leucocephalus, is a common winter resident in the project area. The bald eagle is mentioned in the final statement; however, its endangered status is not. The species may be impacted through increased winter recreation activity on Credit Island and loss of roosting sites from both project construction and development of recreational facilities.

We recommend that the Corps investigate the potential impacts to these four species and begin Section 7 consultation procedures with the Department's Fish and Wildlife Service if appropriate.

Sincerely,

Larry E. Meierotto
Deputy Assistant SECRETARY

PETERSON/jav/236

NCRED-D

27 APR 1978

Mr. Jim Cameron
Midwestern Grain Co.
4800 Main Street, Suite 436
Kansas City, Missouri 64112

Dear Mr. Cameron:

Inclosed is a print of drawing P-1.0 marked in red, as you requested in your letter of 13 April 1978, to show the proposed levee for the Davenport flood control project. In the upper right corner of the sheet is shown a typical section of the proposed levee. The levee plans are in a preliminary stage and are under review by our higher authority.

As mentioned in our letter of 16 April 1976, to Mr. Henry O. Beckmann, who furnished us a print of drawing P-1.0 at that time, if any facility is existing or if construction is imminent when final plans are prepared, the flood protection system would be adapted to current conditions to avoid or minimize interference. Final plans for the flood protection work in the vicinity of your property are scheduled for completion about June 1979. We will coordinate development of the final plans with you.

Sincerely yours,

1 Incl
Print P-1.0

DOYLE W. McCULLY
Chief, Engineering Division

CF:
Dist File (all wo/incl)
ED
Relocations Br.
OD-S
✓Gen Engr Sec (w/incl)

NCRED-RL

27 APR 1978

Mr. F. A. Barton, Division Manager
Illinois-Iowa Division
Chicago, Milwaukee, St. Paul and
Pacific Railroad Company
1900 North Central Avenue
Chicago, Illinois 60639

Re: Davenport, Iowa Local Flood Control Project

Dear Mr. Barton:

This is in reply to your letter of 7 April 1978 regarding the subject project and your request for detailed plans.

We have coordinated the early planning of this project as it affects the Milwaukee Railroad with your Mr. W. E. Fuhr. We will review further developments with the railroad and furnish final plans when they are available.

Your continued cooperation on this project is appreciated.

Sincerely yours,

DOYLE W. McCULLY
Chief, Engineering Division

Copy furnished:
Mr. W. E. Fuhr, Assist. Chief Engineer - Staff
Chicago, Milwaukee. St. Paul & Pacific Railroad Company
898 Union Station
Chicago, Illinois 60606

cc: ED
ED-D (Peterson)
OD-S (Hines)
ED-RL

OMMISSIONERS
OHN C. THOMPSON, Chairman—Forest City
OMAS A. BATES—Bellevue
OHN C. BROPHY—Lansing
OHN G. LINK—Burlington
AROLYN T. LUMBARD—Des Moines
ARIAN PIKE—Whiting
ERBERT T. REED—Winterset

IOWA CONSERVATION COMMISSION

FRED A. PRIEWERT, Director
Wallace State Office Building, Des Moines, Iowa 50319
515/281-5145

An EQUAL OPPORTUNITY Agency

RECEIVED
APR 26 1978
OFFICE FOR PLANNING AND PROGRAMMING

April 26, 1978

A. Thomas Wallace, Jr.
Federal Funds Coordinator
L O C A L

Dear Mr. Wallace:

Field personnel have reviewed the E.I.S. and Phase I G.D.M. for the Davenport Levee project. The following are comments and suggestions relative to the project. It is noted that the Conservation Commission has no sovereign land jurisdiction on this project and the following are suggestions only.

Black Hawk Creek Channel. This channel is to be widened, deepened, and lined with rock-filled wire enclosures called gabions. The creek channel will be approximately 10 feet deep and 67 feet wide. The gabions forming the channel sides will be constructed in the form of steps three tiers high. The following items should be addressed to improve fish and wildlife habitat in the creek area.

1. At periodic intervals, riffle structures should be installed to provide habitat diversity. Fish and Wildlife Service and Iowa Conservation Commission biologists should assist in this planning.

2. A meandering low flow channel be cut within the deepened channel.

3. Suitable soil be placed on the upper faces of the gabions and vegetation planted to replace destroyed habitat. Plantings should be those species that are most advantageous as wildlife habitat.

4. The creek bottom should be left in natural soil. Those portions which require concrete lining (about 300 feet) should be reviewed and evaluated by Iowa Conservation Commission and Fish and Wildlife Service biologists.

5. The above measures should be incorporated into the Phase I G.D.M. planning.

Mississippi River Dredging. Approximately 905,000 cubic yards of sand is to be dredged from the Mississippi River along the project reach. The following measures should be incorporated into the Phase I G.D.M.

1. Iowa Conservation Commission and Fish and Wildlife Service biologists should sample the dredge area. A plan be developed that would minimize impacts to local mussel beds.

Credit Island Slough. Approximately 380,000 cubic yards of silt will be dredged from Credit Island Slough and recreational benefits will be added. It is recommended the following measures be incorporated into the Phase I G.D.M.

1. The Corps, I.C.C., Fish and Wildlife Service, and the City of Davenport develop a recreational plan for this area. Measures to consider include:

   a. The areas and amount of dredging to be accomplished.

   b. The establishment of fishing docks or jetties.

   c. Final determination of what the proposed small boat harbor is to be.

2. All dredged silt allowed to drain on Credit Island be placed in large open areas to lessen impacts on trees and shrubs.

3. Credit Island is a favored roosting area for bald eagles during the winter months. Construction should be avoided in this area during the winter use season.

4. Credit Island Slough is to be used as a ponding area during floods. If the runoff contains toxic substances in harmful concentrations, it is suggested the area be used as a ponding area.

Nahant Marsh. The preservation of Nahant Marsh is an important part of the total plan. It is recommended the Corps ensure that the surface drainage area that feeds the Nahant Marsh continue to drain into the marsh under normal conditions.

Seawall Fishery. Excellent fishing opportunities exist along the seawall areas below the Government Bridge and in the LeClaire Park area. Continued access for anglers must be allowed for in these areas.

I feel that the above-mentioned items should become part of the Phase I G.D.M. Please contact me if you have questions.

Sincerely,

FRED A. PRIEWERT, DIRECTOR
IOWA CONSERVATION COMMISSION

FAP:rt

cc: Marion Conover
Dale Stufflebeam
Daniel L. Lyons

United States Department of the Interior

FISH AND WILDLIFE SERVICE
Federal Building, Fort Snelling
Twin Cities, Minnesota 55111

IN REPLY REFER TO:
LWR

APR 25 1978

Colonel Daniel L. Lycan
District Engineer
U.S. Army Engineer District
Rock Island
Clock Tower Building
Rock Island, Illinois 61201

Dear Colonel Lycan:

The U.S. Fish and Wildlife Service has reviewed the final Phase I General Design Memorandum for a local flood protection project on the Mississippi River and Blackhawk Creek at Davenport, Iowa, as requested in your undated transmittal letter to the Interior Department's Assistant Secretary, Program, Budget and Administration. This letter addresses the document and also provides comments for the Section 404 Public Notice issued for the flood control project.

On January 8, 1975 the Department of the Interior made comments on the draft Environmental Impact Statement and draft Phase I General Design Memorandum (GDM). Our review of the final Phase I GDM finds that our concerns at that time have been adequately addressed in the final document. However, the selected plan has been modified since the draft Phase I GDM. Also, three wildlife species that may be found in the project area have since been added to the list of Endangered and Threatened Wildlife and Plants. The following comments are made with regard to these subjects.

In lieu of the concrete lined rapid flow channel originally proposed for Blackhawk Creek, it is now proposed that the channel be widened and deepened to a cross section approximately 67 feet wide and 10 feet deep. The creek bank will be protected from erosion with rock filled gabions. Although this method of channelization is preferable to the concrete channel, we recommend that the following measures be taken to improve fish and wildlife resources along Blackhawk Creek:

1. Stone baffles should be constructed at 500-foot intervals within the channel to create pools and riffles in the creek.

2. Vegetative species should be established in the channel bottoms adjacent to the low flow channel.

3. Suitable soil should be placed on the upper faces of the gabions and vegetation planted to develop wildlife habitat.

4. Suitable trees and shrubs should be planted on the banks of the completed channel.

5. The creek bottom in the area of the Clark Street bridge should be the only portion of the creek lined with concrete. The remainder of the creek bottom should be left in natural soil.

Specific design of the above measures should be coordinated with the Iowa Conservation Commission and the U.S. Fish and Wildlife Service during the Phase II General Design planning.

Another departure from the draft Phase I GDM plan is the location of the major borrow areas for levee construction. Approximately 905,000 cubic yards of sand will be dredged from the Mississippi River and 380,000 cubic yards of silt dredged from Credit Island Slough. The use of these borrow areas will impact fish and wildlife resources by both the physical taking of the borrow and the resulting leachate. Therefore the following recommendations are made:

1. Analysis should be made of all borrow material to determine extent of contamination and impacts that may be expected on fish and wildlife resources in the area of the borrow sites and downstream of the sites. Results from this analysis should be provided to the Iowa Conservation Commission and the U.S. Fish and Wildlife Service for review and comment.

2. Mississippi River borrow areas should be sampled to determine the amount and utilization of fishery resources and the extent of benthic populations. If significant populations are found, a plan should be made in cooperation with the Iowa Conservation Commission and the U.S. Fish and Wildlife Service to minimize impacts to the benthic organisms and fishery resources. The Rock Island Field Office will conduct the needed surveys under the FY 78 Transfer Fund Agreement.

3. A plan for dredging Credit Island Slough should be made in cooperation with the Iowa Conservation Commission and the U.S. Fish and Wildlife Service to maximize benefits to fishery resources and minimize adverse impacts to migratory shore birds.

4. All silt allowed to drain on Credit Island should be contained in the large open areas on the island. Impacts to trees and large shrubs should be minimized by use of protective structures.

The preservation of Nahant Marsh is an important addition to the final Phase I GDM. We have previously reviewed the plan for the marsh and have found it to be acceptable. Our comments of May 10, 1976 remain, however, we would like to add the following:

1. The City of Davenport should be required to purchase the 163 acres, including Nahant Marsh, prior to the start of construction of the proposed flood control project.

2. The City of Davenport should be required to establish a plan for operation, maintenance, and improvement of the marsh. The plan should be made in cooperation with the Iowa Conservation Commission, U.S. Army Corps of Engineers, U.S. Fish and Wildlife Service, and any other interested parties.

3. The Corps should ensure that the surface drainage area that feeds the Nahant Marsh (approximately 700 acres) continues to drain into the marsh under normal conditions.

4. Provisions should be made to ensure that the marsh will always receive an adequate quality and quantity of water.

The final Phase I GDM (page 9) states that "no rare or endangered species are known to inhabit the project area". Revision of the document is necessary to include the following information. The bald eagle (Haliaeetus leucocephalus) is a common winter resident in the project area. The species was added to the list of Endangered and Threatened Wildlife and Plants on February 14, 1978 (43 FR 6230). The flood control project includes recreational development of Credit Island. This development may intensify winter recreational use on the island and as a result affect the bald eagles' roosting areas.

Also, two endangered mussel species may be in the area of the proposed project. These are the fat pocketbook pearly mussel (Potamilus capax) and Higgins eye pearly mussel (Lampsilis higginsii). Both were listed on June 14, 1976 (41 FR 24064). The species spectacle-case (Cumberlandia monodata) is proposed to be listed as a threatened species and may be found in the project area. If located in the project area, these species could be adversely impacted by removal of borrow from the Mississippi River and from the resulting degradation in water quality.

Under Section 7 of the Endangered Species Act of 1973, all Federal agencies are directed to insure that their activities or programs will not jeopardize the continued existence of any listed species or result in the destruction or modification of designated critical habitat. Therefore, it is suggested that the Corps conduct investigations to determine if any listed species will be affected by the proposed project or by future recreational development plans of the local sponsor. If any species will be affected, the Corps should initiate the Section 7 formal consultation process (43 FR 870) by writing to the Regional Director, Region III, U.S. Fish and Wildlife Service, Federal Building, Fort Snelling, Twin Cities, Minnesota 55111.

You have also requested our views on this project by way of Public Notice NCROD-S-070-0X6-1-06664 dated March 28, 1978 for a Section 404 permit. The U.S. Fish and Wildlife Service will not object to issuance of the permit provided the above measures are stipulated to minimize damages to fish and wildlife resources.

These comments have been prepared in accordance with provisions of the Fish and Wildlife Coordination Act (48 Stat. 401, as amended; 16 U.S.C. 661 et seq.).

Sincerely yours,

Charles A. Hughlett

Acting Regional Director

cc: Iowa Conservation Commission
U.S. Environmental Protection Agency, Kansas City
U.S. Army Corps of Engineers, Rock Island
attn: Mr. Henry G. Pfiester

DIVISION OF HISTORIC PRESERVATION
IOWA STATE HISTORICAL DEPARTMENT

ADRIAN D. ANDERSON, DIRECTOR
STATE HISTORIC PRESERVATION OFFICER

April 21, 1978

Colonel Daniel L. Lycan
Rock Island District, Corps of Engineers
Clock Tower Building
Rock Island, IL 61201

ATTN: Regulatory Functions Branch

RE: NCROD-S-070-0X6-1-0664; Public Notice, construction in conjunction with the Local Flood Protection Project for Davenport, Iowa. OPP No. 750305.

Dear Colonel Lycan:

We have no objection to the above proposed action and will be pleased to assist in implementation of measures pursuant to Section 4.08 of the final EIS for the Davenport Local Flood Protection Project. Please feel free to contact our office when your agency initiates the procedures of 36CFR800.

Sincerely,

Adrian D. Anderson, Director
State Historic Preservation Officer

ADA/lv

cc: Thomas Wallace, OPP

EAST MARKET STREET • IOWA CITY, IOWA 52240
TELEPHONE (319) 353-4186/353-6041

(22)

Midwestern Grain Co.

DIVISION OF GARNAC GRAIN CO., INC.

April 13, 1978

District Engineer
Rock Island District Corps of Engineers
Clock Tower Building
Rock Island, Illinois 61201

Attn: Regulatory Functions Branch

Ref: NCROD-S-070-0X6-1-06664
Section 404 Public Notice
March 28, 1978

Gentlemen,

Being a property owner in the area of the proposed new levee construction; referenced in the above public notice, we are concerned with the effect upon that property, and of future plans we have.

We find the information contained in the public notice not quite clear enough to ascertain the effect upon our property.

Enclosed you will find (2) two copies of Drawing P-1.0. We request that you clarify your proposed construction, and should there be any effect upon our property, please indicate on one of the drawings and return it to us promptly.

Very truly yours,

Jim Cameron
Project Engineer

JC/lmh
Enclosures

CHICAGO, MILWAUKEE, ST. PAUL AND PACIFIC RAILROAD COMPANY

ILLINOIS — IOWA DIVISION
1900 N. Central Avenue · Chicago, Illinois 60639

FRED A. BARTON
Division Manager

Galewood, Illinois
April 7, 1978
File: Davenport, Iowa - 0
(Flood Control)

District Engineer
Rock Island District Corp of Engineers
Clock Tower Building
Rock Island, Illinois 61201

Dear Mr. Lycan:

This office has received your public notice (NCROD-S-070-0X6-1-06664) dated March 28, 1978 concerning proposed flood control in the Davenport, Iowa area.

Please be advised that the Milwaukee Railroad and DRI&NW Rwy. have trackage, drainage systems, sewer lines, and water lines that will be affected by this construction and that provisions be made by the Corp. of Engineers to provide for the continued operation of these facilities.

Please send detailed plans of this project when they become available.

F. A. Barton

F. A. Barton
Division Manager

CC: Mr. W. E. Fuhr - Room 898 - CUS

(24)

IOWA DEPARTMENT OF TRANSPORTATION

Department of Transportation

PLANNING AND RESEARCH DIVISION

800 LINCOLN WAY AMES, IOWA 50010 515-296-1661

April 4, 1978

REF. NO. 700.010

District Engineer
Rock Island District Corps of Engineers
Clock Tower Building
Rock Island, IL 61201

Attention: Regulatory Functions Branch

Dear Sir:

We have reviewed the following application for work permit and find it does not conflict with any programs in which the Iowa Department of Transportation is involved at this time.

NCROD-S-070-0x6-1-06664

We thank you for the opportunity of reviewing this submittal.

Very truly yours,

C. I. MacGillivray
Director
Planning & Research Division

CIM:mkf

cc: James F. Cobb
Office of Program Mgt., IA DOT

COMMISSIONERS

JULES M. BUSKER Sioux City | BARBARA DUNN Des Moines | DONALD K. GARDNER Cedar Rapids | WILLIAM F. McGRATH Melvese | ROBERT R. RIGLER New Hampton | L. STANLEY SCHOELERMAN Spencer | ALLAN T. Dubuq

United States Department of the Interior
NATIONAL PARK SERVICE
INTERAGENCY ARCHEOLOGICAL SERVICES - DENVER
OFFICE OF ARCHEOLOGY AND HISTORIC PRESERVATION
1978 SOUTH GARRISON - ROOM 107
DENVER, COLORADO 80227

IN REPLY REFER TO:

H22-(RMR)PI

JUL 09 1976

Doyle W. McCully
Chief, Engineering Division
Department of the Army
Rock Island District, Corps
of Engineers
Clock Tower Building
Rock Island, Illinois 61201

Dear Mr. McCully:

In reply to your letter of request dated July 2, 1976, we are offering comments here covering a report entitled: "An Intensive Archaeological Survey: Davenport, Iowa Local Flood Protection Project", by David F. Overstreet of the Great Lakes Archaeological Research Center, Inc. The negative finding of the Principal Investigator obtained by coring, shovel testing, and test pitting at all locations previously suspected as sites of prehistoric occupation seem to be amply documented by thorough and systematic probing. Based on this review, Interagency Archeological Services - Denver, concurs favorably with the procedures and findings of the contractor.

Thank you for participating in the Interagency Archeological Salvage program.

Sincerely yours,

Jack R Rudy
Chief, Interagency
Archeological Services - Denver

STATE HISTORICAL DEPARTMENT OF IOWA
DIVISION OF HISTORIC PRESERVATION

ADRIAN D. ANDERSON, DIRECTOR
HISTORIC PRESERVATION OFFICER

March 15,1976

Mr. Larry McLean
Environmental Division
Rock Island District
Corps of Engineers
Clock Tower
Rock Island, Illinois 61201

Dear Mr. McLean,

This note is a re-statement of our recent telephone conversation concerning the Davenport,Iowa levee project. In reviewing the information on the project we learned that a large portion of the Davenport water-front was disturbed by an E.P.A. sponsored sewer and wastewater treatment facility. Any archaeological sites in the path of this E.P.A. project would have been destroyed or at least badly damaged. Therefore, we recommend that shovel testing, as part of the cultural assessment for the levee project, be limited to the Cedit Island portion of the project area which was not disturbed by the sewage and wastewater facilities project. The few remaining undisturbed areas of the Davenport river-front need only be monitored at the time of construction. This should afford sufficient protection for any portions of sites which might remain in the Davenport water-front area.

Sincerely

Robert Alex
Chief, Archaeological Survey

B.13 MAC LEAN HALL • IOWA CITY, IOWA 52242
TELEPHONE 319-353-6949/319-353-4186

STATE HISTORICAL DEPARTMENT OF IOWA
DIVISION OF HISTORIC PRESERVATION

ADRIAN D. ANDERSON, DIRECTOR
HISTORIC PRESERVATION OFFICER

March 17, 1975

Mr. Larry McLean
Corps of Engineers
Rock Island District
Clock Tower Building
Rock Island, Illinois 61201

Re: Davenport - Flood Control Project

Dear Mr. McLean:

I have reviewed the described project, correspondence from residents of Davenport and Mr. Weichman's survey and offer the following comments:

There are serval problem-areas which are either noted in the report or surfaced as points of disagreement in correspondence.

1. LeClaire Park
   This city park dates from the earliest days of the city of Davenport and has long been a part of community life, occupying as it does a central river-front location.
   The proposed levee would certainly be an intrusion and, in my opinion, such a visual and esthetic intrusion would constitute an adverse effect.
   I would suggest that details of the place of the park in Davenport's history be researched and submitted to the Secretary of the Interior for determination of eligibility, pursuant to 36 CFR 800.

2. The Cook Farm Mound Group (13 ST 9)
   This mound group played an important role in the earliest speculations concerning prehistoric man in Iowa. There is no agreement on the relationship of this mound group to the project area. Before the project is implemented there should be a survey to locate the mound area, testing to determine the presence or absence of cultural remains, and if any are present, to request the Secretary of the Interior's determination of site eligibility for the National Register of Historic Places.

3. Credit Island
   The effect of the project upon historic Credit Island must be examined more thoroughly. The fact that it was the site of an early Trading

B.13 MAC LEAN HALL • IOWA CITY, IOWA 52242
TELEPHONE 319.353.6949/319.353.4186

Post, that the Mesquakie Indians today regard it as one of their traditional village sites, and that a military engagement was fought there, all make me feel that the Island meets the criteria of eligibility for the National Register. I recommend that 36 CFR 800 be implemented and that the Secretary of the Interior's determination of eligibility be made.

If you will advise me when the Advisory Council review process is begun I will be at your service in preparing the memorandum of agreement.

Sincerely,

Adrian D. Anderson

Adrian D. Anderson, Director
Historic Preservation Officer

ADA:pah

Iowa
a place to grow

Natural Resources Council

Wallace State Office Building
Des Moines, Iowa 50319
(515) 281-5913

JAMES R. WEBB, Director
LOUIS F. GIESEKE, Water Commissioner

October 29, 1979

Mr. Doyle McCully
Chief, Engineering Division
Rock Island District
U.S. Army Corps of Engineers
Clock Tower Building
Rock Island, Illinois 61201

Dear Mr. McCully:

Please refer to your letter of 28 September, 1979 regarding the Phase I General Design Memorandum for the Davenport, Iowa, Local Flood Protection project.

The INRC supports the structural flood protection works proposed for the City of Davenport and finds that the project, as proposed, conforms with flood plain protection standards for the State of Iowa. It is also recommended that the Corps in cooperation with the City, assure that those areas not protected from flooding such as Credit Island and unique ecological areas such as Nahant Marsh, be preserved through zoning or public acquisition.

If you have any questions concerning the above, please feel free to contact me.

Very truly yours,

Wayne Gieselman

Wayne Gieselman, P.E.
Chief Engineer

WG/pk

COUNCIL MEMBERS:

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Knoxville

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CITY OF DAVENPORT

OFFICE OF THE MAYOR

DAVENPORT, IOWA 52801

CHARLES J. WRIGHT
MAYOR

February 1, 1980

Doyle W. McCully, P.E.
Chief, Engineering Division
Rock Island District, Corps of Engineers
Clock Tower Building
Rock Island, Illinois 61201

Re: Davenport Flood Control Project

Dear Mr. McCully:

Pursuant to your letter of 09/28/79, City Staff has researched the issue of whether the Davenport flood control project satisfies the requirements of E. O. 11988 regarding flood plain management for the Mississippi River and Blackhawk Creek.

The City of Davenport (Community #190242) is in the regular flood insurance program effective 03/01/78. This status was preceded by a flood insurance study conducted by DeWild, Grant, Reckert & Associates Co., Consulting Engineers from Rock Rapids, Iowa, under contract to the Federal Insurance Administration, U. S. Department of Housing and Urban Development. The City Council passed an ordinance concerning flood damage prevention on 02/15/78 which is Chapter 14.001 of the City Code. This ordinance has been approved for use by the Iowa Natural Resources Council in correspondence of 04/07/78, Council Order No. 78-3R.

The City is managing this ordinance for the Mississippi River and the five streams which are within the corporate limits. The regulatory flood elevation in all cases is one foot above the 100 year flood based on the FIS. The design level flood as we understand the Phase II design will protect the community from flood frequencies of 200 years and 100 years on the Mississippi River and Blackhawk Creek respectively. The project, therefore, meets or exceeds the regulatory requirements of the flood insurance program.

Sincerely,

Charles J. Wright
Mayor

MISSISSIPPI RIVER
DAVENPORT, IOWA

LOCAL FLOOD PROTECTION
PHASE II

GENERAL DESIGN MEMORANDUM

APPENDIX I

RECREATION AND AESTHETICS
(REVISED)

US ARMY ENGINEER DISTRICT, ROCK ISLAND
CORPS OF ENGINEERS
CLOCK TOWER BUILDING
ROCK ISLAND, ILLINOIS 61201

MISSISSIPPI RIVER
DAVENPORT, IOWA

LOCAL FLOOD PROTECTION
PHASE II

GENERAL DESIGN MEMORANDUM

APPENDIX I
RECREATION AND AESTHETICS

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MISSISSIPPI RIVER
DAVENPORT, IOWA

LOCAL FLOOD PROTECTION
PHASE II

GENERAL DESIGN MEMORANDUM

APPENDIX I
RECREATION AND AESTHETICS

I INTRODUCTION

1. The Davenport Local Protection project is located adjacent to the Mississippi River in the central and southwestern portion of the city of Davenport, Iowa. The project area extends from Mississippi River mile 478.6 to river mile 482.1. The project was authorized by the Flood Control Act of 1970, generally in accordance with the provisions of House Document No. 92-161.

2. This appendix has been prepared in accordance with the provisions of ER 1165-2-2, "Consideration of Aesthetic Values in Water Resource Development" and EM 1110-2-38, "Environmental Quality in Design of Civil Works Projects" for the Davenport Phase II General Design Memorandum. The purpose of this appendix is to give full consideration to aesthetic elements, both existing and potential, within the project area and to propose ways and means to preserve or enhance certain water and related resources. This will be accomplished by both the engineering design process of the proposed structure and the recreation and beautification features integrated into this plan. It will be essential to provide pedestrian circulation and activity along the river's edge and present aesthetically pleasing views by encouraging full use of the riverfront. Some of these proposed measures can be considered as strictly mitigation while others will qualify for Federal cost-sharing under Public Law 89-72. Other features that are unrelated to the construction of the local protection structures will be the financial responsibility of the city.

II EXISTING RECREATION AND AESTHETICS

Downtown Sector

3. This sector extends from the Government Bridge to the Centennial Bridge. Existing land uses are scattered city buildings and parks along the river edge and downtown commercial areas north of River Drive. In this sector, River Drive is bordered towards the river side by open spaces consisting of parking lots, LeClaire Park, and railroad tracks. To the north, it is screened by the unattractive backs of commercial buildings that front on Second Street. The Government Bridge and Locks and Dam 15 are important landmarks along the riverfront, providing strongly visible objects across the river surface and views of boat traffic. A concrete

sea wall lines the edge of the river from the dam downstream to Marquette Street. The wall is topped with a steel railing and provides a convenient facility for elderly and young fishermen. The sea wall urbanizes the riverfront and permits land and building development to the waters' edge. A popular restaurant is located on the riverfront at the toe of Perry Street.

4. Antoine LeClaire Park in this sector is the "front yard" of the downtown area. This 8.85 acre city park begins at the foot of Main Street and extends west to the Municipal Stadium. The municipal boat-launching ramp is heavily used by people from a wide area during summer months. It is estimatd that approximately 100 people participate in Mississippi River boat tours operating from this area on a peak Sunday. The park offers dramatic and scenic views of the Mississippi River, the activities in the park, and the view back to the skyline of the downtown area with wooded hills as a background. The park is characterized by open lawn areas, sparsely planted trees and shrubs, and narrowly-paved promenades. The trees include hard maple, ash, locust, oak and sycamore. In addition to the swimming pool, the recreation facilities include a picnic shelter, picnic tables, horseshoe pits, a band shell, and a lighted baseball stadium. The band shell is used for evening concerts, outdoor festivities, and is the home of the annual Bix Beiderbeck jazz music festival. This event generates approximately 60,000 recreation days per year. The municipal stadium is the home of the Quad-Cities Cubs, a professional baseball team with scheduled daytime and nighttime games. It is also used for football and other sports events. The park receives heavy use by fishermen who find the facilities easy to use and spend many hours fishing from the sea wall.

Crescent Park Sector

5. This area is bounded on the west by Credit Island and on the east by the Centennial Bridge. The shoreline in this section maintains some natural characteristics, but these can only be rated as fair in aesthetic quality. Present land uses do not present aesthetically pleasing sights, odors, or smells. The area includes municipal garages, railroad yards, steel scrap yards, and the city's former sewage treatment plant and sanitary landfill. At the east boundary, the riverfront has a narrow park space at the water's edge, but this is bordered by the city's maintenance facilities. Although nominally a park, this waterfront area does not attract many users because access to the area is difficult. Further downstream the former city landfill lies upon what once was a large, marshy area along both sides of the Crescent Bridge. The rail approaches to this railroad bridge bisect the area and destroy the continuity of access along the riverfront. Adjoining the landfill to the north and west is the former city sewage treatment plant. The discharge from this plant until a few years ago contributed organic materials into the river in this vicinity. The remainder of the distance to Schmidt Road is narrow and unimproved. It possesses the only existing woody vegetation in the sector.

6. Dispersed recreation users including drivers, hikers, and bicyclists desiring to follow the riverfront in this sector are confronted by many obstacles. River Drive swings away from the rivershore for about ten blocks before crossing the railroad tracks to again meet the riverfront. Through this area, views of the river are screened by industrial facilities. Marquette Street penetrates south of River Drive, but users must compete with large trucks and trains for access to the river. Dense residential neighborhoods exist north and adjacent to this sector, but are cut off from safe and convenient access by industries, railroad tracks, and River Drive. Stadium Drive extends into this sector from the Centennial Bridge, but ends at Marquette Street. The Royal American Shows set up in this area annually for a week-long engagement in the Quad-Cities. It is also used as a carnival lot and as a parking lot for the Municipal Stadium. Of the three major sectors involved in this project, this is presently the least attractive or useful, but it does have potential value for future development. This is chiefly because of its central location between the adjacent upstream and downstream sectors, and its proximity to the river.

Credit Island Sector

7. Credit Island has been a popular recreation facility since the 1890's. The name "Credit" came from the fact that it was an early trading post for the Indians and trappers in the region. In the early days, it was accessible only by boat or trolley car, but in 1921, a causeway was built connecting it to the mainland near Schmidt Road. It is the largest park in Davenport, approximately 420 acres, and the only existing park west of the downtown area. The west end of the island, comprising about 100 acres, is preserved as a natural area. The remaining acreage is developed with playgrounds, tennis courts, baseball diamonds, horseshoe pits and an 18-hole golf course. Picnic areas are scattered on the island, and along the periphery of the island is a scenic drive. The body of water between the north shore of the island and the river edge is called Credit Island Slough and serves as an area for boating and fishing. The causeway which connects the island to the mainland restricted the natural flow of the river through the slough. This blocked flow and sedimentation from Blackhawk Creek nearly filled the upper one-third of the slough. Except during high water, boat travel within the upper portion of the slough is greatly restricted. At extremely low water, the presence of mud flats and debris reduce the aesthetic value of the area.

8. Most of Credit Island's natural vegetation has been modified for developed recreation facilities. Much of the west end of the island is wooded, and the dominant species includes soft maple, sycamore, pin oak, willows, and cottonwood. This wooded area, in close proximity to the city, makes it valuable as a natural area. The flora and fauna of the west end of the island has preserved the natural characteristics of the river environs. The east end of the park provides dramatic views of the Davenport skyline, the river, and the gentle landscape and distant timber-covered bluffs in Illinois. The scenic drives along the timbered periphery of the island yields gratifying vistas to visitors driving for pleasure or to bicyclists who use this roadway. Because of its unique setting, the

golf course is considered especially attractive amidst the natural surroundings of water and wooded areas. Periodic flooding, however, has interrupted use of the golf course.

III EFFECT OF THE PROPOSED PROJECT

Downtown Sector

9. The greatest impact of the proposed structure will occur in this sector which is presently used for passive recreation. The structure, an earthen levee, a permanent wall, and a folding wall, will become a physical and visual barrier in the riverfront area. Gated openings, however, will be provided to allow continued pedestrian access to the shore for dispersed recreation activities. The levee height, including the permanent wall, will be about seven feet above the existing parking lots in the riverfront area and will screen cars from the view of the users in the LeClaire Park. To some extent, it will also act as a screen to the people using the protected side of the levee who wish to view the river. However, people can view the river from the low levee which will have a maintenance road on top which will be available as a walkway and bicycle path when the river is not in a flood condition. The project also strengthens the linear design of the riverfront between Main and Ripley Streets. The structure will protect the band shell and seating area from flooding and an opening will be provided in the wall to allow boaters who anchor in this vicinity to continue the use of riverfront for this purpose. Traffic congestion may be possibly increased before and after scheduled sporting events at the stadium, for outdoor concerts, and during periods of peak weekend use at the excursion boat dock because of levee ramps.

10. Improvements to the park should result in greater accessibility by foot, bicycle and car, and increased opportunities for passive recreation. River use will be encouraged and appreciation of this outdoor resource should be expanded and intensified. The river edge is the one element of this urban landscape that unifies the city and should offer new opportunities for cohesive development. It can become a useful corridor of scenic drives and pathways offering a variety of both natural and urban views to active and passive recreationists.

Crescent Park Sector

11. The impact of the proposed structure on present recreation activities is minimal in this sector because this area has very limited usable open space or park along the riverfront. Existing uses such as the railroad yards and municipal garages would not be adversely affected by the location or size of the proposed structure. The sparse existing vegetation adjacent to the riverbank will be lost by construction of the levee near the shore. The structure will also partially destroy the existing narrow, open, park located downstream and adjacent to the Centennial Bridge. The levee will make the sector accessible by developing a roadway into this area, and landscape grading and tree planting will improve its aesthetic quality.

Credit Island Sector

12. Land required for the project structures would have an adverse effect on terrestrial ecosystems on the island. The levee would be constructed on the landward side of the island parallel to the Credit Island Slough. The removal of trees for the project from the island would reduce habitat for many species of birds. There are many other areas, however, with existing tree stands on the island, and no significant losses can be expected. The construction of the levee on the landside of the island would affect the aesthetic qualities of the island's recreation developments. The view of the slough from the island would be restricted, as well as the view of the island from the slough and the mainland. The project will, however, provide a permanent flow through the slough increasing the water quality of the slough by reducing siltation. In addition, the dredging of the slough will enhance the potential of water oriented recreational activities. Although Credit Island is developed for recreation (golf, fishing, and natural area), the levee would not protect these developments. Continuing subjection of the island to flooding will reduce the quality of developed recreation facilities and increase the costs of maintaining these developments.

IV PROPOSED RECREATION AND LANDSCAPE PLAN

13. Construction of recreation facilities and associated landscaping and beautification will be accomplished on either a mitigation or cost-shared basis with one additional feature constructed entirely with city of Davenport funds. Table I-1 lists mitigation and cost-shared items and the following paragraphs describe these features. Mitigation items are 100% Federally funded and include replacement or rehabilitation of recreation and associated features that are unavoidably disrupted or destroyed by construction activities. New recreation development will be carried out on a 50-50 cost shared basis between the Federal government and the city of Davenport. Total cost of recreation and associated features is $4,074,000. Federal cost is $2,438,000 and non-Federal cost is $2,543,000. A detailed breakdown of recreational costs is presented in Appendix E. Appendix F describes recreation benefits.

Downtown Sector

14. This sector is being constructed and landscaped in response to the city's desire to continue the use of the riverfront for recreation. The low levee profile will permit scenic views of the river from existing vistas and openings in the wall structure will allow pedestrain access to the riverfront via walks leading from existing parking lots and the downtown business district. The proposed flood wall structure, constructed atop the levee, will be textured or patterned to give architectural relief and beauty to the structure. The folding panels will have some type of textured surface and the free board will be constructed of aluminum that will possess a natural finish. The back slope will be graded and sodded to blend with the lawns in LeClaire Park and aid in beautifying the structure. The river side of the levee slope will be built in a terraced

pattern to protect the structure from river scour and provide an area for plantings to improve the appearance of the levee berm. The low walls forming the terraces will be patterned to match the floodwall. Steps will be used at openings to allow pedestrians to walk down the slope to the river edge.

15. The unprotected portion of the riverfront, and the land area between the existing seawall and the levee structure, is to be developed and utilized as an esplanade; a grassy parkway to be utilized by pedestrians for passive outdoor recreation pursuits. Low walled landscape planters will aid in screening the levee and also provide area for shrub and flower plantings. Park benches and conversation pits will be arranged to invite the public to continue the use of the riverfront for passive recreational enjoyment. Trees will also be included in this plan to add quality to the site, create a parklike setting, and provide shade for the users. Maximum efforts have been made to enlarge the useful area of LeClaire Park and mitigate the loss of land area to the levee structure. The riverfront will remain accessible for fishermen, and the boat ramp and excursion boat landing at the foot of Main Street will remain operable.

16. A paved bycycle/hike trail will be located atop the levee beginning at a ramp at Perry Street. The trail will proceed downstream along the riverfront, the Crescent Park sector, and on to Credit Island. This facility will link the city's major riverfront park developments with a recreation facility which can be used for pleasure walking, jogging, or bicycling. This route, most importantly, will be isolated from vehicular traffic routes.

17. Planting of shade trees and flowering trees and shrubs will enhance the riverfront and provide pleasing space relationships and scenic variety to the landscape. All plants used in this project will be tolerant of local climatic and seasonal conditions.

Crescent Park Sector

18. The construction of this sector will provide a public corridor and link between two important existing recreation areas on Davenport's riverfront: LeClaire Park on the east end and Credit Island on the west end. A 24-foot wide concrete paved roadway will be located atop the levee as a scenic drive. An asphalt paved bikeway will also be built on the river side of the levee with some portions joining the roadway to give variety to the experience. This walkway will open and provide unlimited access of river shore to fishermen who presently are not able to use this area of the river because of difficult access.

19. The 60-acre landfill area north of the levee will be developed as a park. The scenic drive will provide much needed vehicular access to the area as well as provide flood protection to developments within the park. Recreation facilities to be cost shared with the city in this sector include the bicycle/walking path, softball diamonds, a 200-car parking lot, picnic areas, bleachers, a multipurpose restroom/storage building, picnic shelters, restroom building, a 50-car parking area, and the scenic

TABLE I-1
COST-SHARED AND MITIGATION FEATURES

Downtown Sector	
Textured floodwall structure	Mitigation
Grading and sodding	Mitigation
Plantings	Mitigation
Steps on levee	Mitigation
Landscaping planters	Mitigation
Park benches	Mitigation
Trees	Mitigation
Bicycle/hiking trail	Cost-Shared
Crescent Park Sector	
24' concrete roadway	Cost-Shared
Asphalt paved bikeway	Cost-Shared
Bicycle/hiking path	Cost-Shared
Light fixtures for baseball diamonds	City of Davenport
4 bleachers	Cost-Shared
200-car parking lot	Cost-Shared
Restroom/storage building	Cost-Shared
Picnic shelters	Cost-Shared
50-car parking lot	Cost-Shared
Park benches	Cost-Shared
Grading and seeding	Mitigation
Trees and shrubs	Mitigation
Credit Island Sector	
Bicycle/hiking trail	Cost-Shared
Bicycle and pedestrian bridge	Cost-Shared
Boat launching site	Cost-Shared
Lagoon dredging and golf course fill, grade, and seed	Cost-Shared
Boat concession building	City of Davenport
Picnic area	Cost-Shared
Walking path at foot of levee	Mitigation
Landscaping	Mitigation
Nahant Marsh Sector	
Acquisition of lands (Davenport bears 16-1/2% of mitigation costs and 50% of costs of recreation/education features)	City of Davenport*
Landscaping	Mitigation

* Any excess in the cost of mitigation lands over and above the local share for mitigation will be applied to other cost-sharing features.

roadway. Mitigation and beautification measures include grading and seeding and planting of trees and shrubs throughout this park of approximately 50 acres.

Credit Island Sector

20. On Credit Island the levee will follow the entire length of the northern shore of the island. The average width of the levee will be 110 feet at the base while the average height of the reach will be 14 feet. Due to the surrounding aesthetics of the area, grass will be utilized as the cover for most of this segment of the project. A 3 to 1 slope from the crown to the toe of the levee will be established on the riverside. The addition of a levee into this park area will introduce new horizons for recreation as well as possibly enhance present forms of recreation.

21. A paved combination bikeway and hiking path will be constructed on top of this portion of the levee. This trail system has been designed with safety and the scenic view of the area as prime concerns. The trail will be eight feet wide and will run the entire length of the levee. At the west end of the island the trail will continue west and cross the harbor closure at the west end of Credit Island Slough. This trail system will then connect into a planned trail on the land side of the slough. The trail will run south of Concord Street starting at Wapello Avenue and will extend eastward past the boat launch site, across Blackhawk Creek to Credit Island Causeway Road. A combination bicycle and pedestrian bridge will be constructed across Blackhawk Creek for this trail. The trail will then swing back onto the Credit Island access road and connect into the trail on top of the levee, thus creating a loop trail.

22. The 18-hole golf course on the south side of the island will be converted by the city into an improved 9-hole course. Low spots in the course will be filled and leveled in conjunction with the proposed lagoon dredging. This will be done to reduce damage to greens and fairways from inundation by the Mississippi River. The greens and fairways will be designed on higher elevations of the present course. Excess land that will result due to the redesign of the course will be utilized for additional picnic area as well as a play area near the lagoon.

23. The northeast finger of the lagoon will be expanded in width and depth. This portion of the lagoon presently occupies about 5-1/2 acres, but will be expanded to about 9 acres. The bottom of the lagoon will be dredged to elevation 542. This section of the lagoon will be utilized as a paddle boat and canoe area. A small dock system will be provided by the city for loading and unloading, as well as rental tieup.

24. A boat concession building will be constructed by the city near the previously mentioned segment of the lagoon. It will serve as a fee collection and control point for boat rentals in the lagoon area as well as the slough area. This rental area will be equipped with pedestrian ramps to the lagoon and slough area, a vehicle ramp leading into a 48-car parking lot, and portable toilets. Off of Concord Street on the land side of the slough, a public boat launch and 40-car parking lot will be built.

25. In addition to the new boating and bicycle facilities, a campground, trails, fishing access points, and an additional picnic area will also be provided. A tent camping area will be established west of the lagoon area. This will offer Quad City residents the opportunity of tent camping without traveling a long distance to a state, county, or private campground. The area will be kept in a natural state without the grass and natural vegetation of the area being disturbed.

26. A family picnic area is planned in the area west of the lagoon and south of the camping area. A shelter for picnickers will be provided. The camping and picnic area will be equipped with a thirty-car parking lot as well as a portable toilet system. Cooking grills will also be stationed at accessible points in the picnic areas.

27. A walking path will be provided on the levee slope, near the waters edge on the slough side. This trail will make it easier for fishermen to utilize the area and could also be used for hiking, pleasure walking, and educational purposes dealing with the slough. The trail will be a six foot wide berm with a gravel surface. The path will be constructed at Federal expense as mitigation for disruption of access.

28. The areas adjacent to the levee will be screened and enhanced with landscape plantings. An estimated 200 shade trees and over 400 flowering shrubs will be planted to beautify this construction. These beautification plantings will be a Federal cost to the project.

Nahant Marsh

29. The Nahant Marsh area is a unique biological area located north of the Credit Island. It is unique to the immediate Quad-Cities area because it is easily accessible for high school and colleges. The marsh has been utilized for such purposes by Fairmont School, Bettendorf School System, Augustana College, Marycrest College, Scott County Sportsman's Association, Muscatine School System, West High School, and the Youth Conservation Corps.

30. It has been recommended that the Davenport Local Protection Project be modified to include the enhancement and preservation of Nahant Marsh. If the recommendation is authorized by Congress, the City of Davenport would be required to provide all lands, easements, and right-of-ways necessary for the Nahant Marsh Preservation Plan. In addition, the city would be required to bear 16-1/2 percent of mitigation costs and 50 percent of costs for recreation/education features.

31. The Scott County Sportsman Association owns a plot of land comprising the southern portion of the marsh. This area is utilized for target and skeet shooting. Through mitigation, various trees and shrubs will be planted to screen this area from the remainder of the marsh for aesthetic and safety purposes.

V RECREATION USE OF THE PROJECT AREA

Introduction

32. The number of yearly activity days in the proposed project area is estimated to be 190,000. Of these, 113,610 occur at the LeClaire Park, 74,847 on Credit Island, and the remaining 1,893 in the Crescent Park sector. The city makes no detailed count of these visits or the particular activities for which these visitations occur. These numbers are, however, estimates based on recent observation and discussion with city park officials. The estimated annual number of activity days at the project by activities is listed on Table I-2.

Conclusion

33. Development of this riverfront will disperse and distribute recreation use along the shore. A variety of recreation facilities will be introduced into the natural and urban environment producing increased use of the shoreline. Expansion of recreation facilities, increased acreage accessible for public use and the development of the landfill area will be the chief reasons for enhanced recreational conditions.

TABLE I-2

Estimated Number of Annual Activity Days*

	Project Year		
	Year One	Year Thirty	Year Forty
LeClaire Park			
Fishing	17,000	20,000	21,000
Picnicking	15,000	20,000	26,000
Walking	45,000	55,000	67,000
Bicycling	20,000	22,000	26,000
Outdoor cultural events	90,000	126,000	150,000
Sightseeing	7,000	9,000	11,000
Boat tours	2,000	3,000	4,000
	196,000	255,000	305,000
Crescent Park Sector			
Picknicking	15,000	19,000	25,000
Fishing	8,000	11,000	12,000
Softball	40,000	44,000	46,000
Sports spectators	15,000	18,000	20,000
Bicycling	20,000	22,000	25,000
Playgrounds	10,000	11,000	12,000
Walking	15,000	19,000	25,000
Playfields	7,000	10,000	11,000
Driving for pleasure	40,000	44,000	44,000
	170,000	198,000	220,000
Credit Island Sector			
Driving for pleasure	50,000	56,000	56,000
Fishing	25,000	29,000	32,000
Picnicking	45,000	55,000	61,000
Camping (tent)	5,000	6,000	7,000
Bicycling	20,000	22,000	25,000
Walking	6,000	9,000	12,000
Nature walk & photo	15,000	21,000	25,000
Boating (power)	10,000	14,000	15,000
Boating (non-power)	5,000	6,000	7,000
	181,000	218,000	240,000

* See Appendix F for a discussion of recreation benefits based on these visitation figures.

TABLE I-3

Estimated Number of Annual Recreation Days*

	Project Year		
	Year One	Year Thirty	Year Forty
LeClaire Park			
Fishing	8,500	10,000	10,500
Picnicking	5,000	6,700	8,700
Walking	5,000	6,900	8,400
Bicycling	1,700	1,800	2,200
Outdoor cultural events	51,400	72,000	85,700
Sightseeing	1,000	1,100	1,400
Boat tours	1,000	1,500	2,000
	73,600	100,000	118,900
Crescent Park Sector			
Picknicking	5,000	6,400	8,300
Fishing	4,000	5,500	6,000
Softball	16,000	17,600	18,400
Sports spectators	6,000	7,200	8,000
Bicycling	1,700	1,800	2,100
Playgrounds	2,500	2,800	3,000
Walking	1,900	2,400	3,100
Playfields	1,800	2,500	2,800
Driving for pleasure	3,300	3,700	3,700
	42,200	49,900	55,400
Credit Island Sector			
Driving for pleasure	4,200	4,700	4,700
Fishing	12,500	14,500	16,000
Picnicking	15,000	18,300	20,300
Camping (tent)	5,000	6,000	7,000
Bicycling	1,700	1,800	2,100
Walking	750	1,100	1,500
Nature walk & photo	3,800	5,300	6,300
Boating (power)	3,300	4,700	5,000
Boating (non-power)	1,700	2,000	2,400
	47,950	58,400	65,300

* See Appendix F for a discussion of recreation benefits based on these visitation figures.

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