WATER RESOURCES DEVELOPMENT PROJECT

PARK RIVER LOCAL PROTECTION

CONNECTICUT RIVER BASIN

HARTFORD, CONNECTICUT

DESIGN MEMORANDUM NO.1

HYDROLOGY

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASS.

JANUARY 1973

26

Engr

DAEN-CWE-B (NEDED-E, 25 Oct 74) 3rd Ind
SUBJECT: Park River Local Protection, Connecticut River Basin, Hartford, Connecticut, Design Memorandum No. 7 - Detailed Design of Structures: Part 1 - Box Conduit

DA, Office of the Chief of Engineers, Washington, D.C. 20314 17 January 1975

TO: Division Engineer, New England, ATTN: NEDED-E

The information furnished and the actions indicated in the 2nd Indorsement are satisfactory.

FOR THE CHIEF OF ENGINEERS:

1 Incl
wd

C.E. Slaton
For HOMER B. WILLIS
Chief, Engineering Division
Director of Civil Works

NEDED-T (25 Oct 74) 2nd Ind
SUBJECT: Park River Local Protection, Connecticut River Basin, Hartford, Connecticut, Design Memorandum No. 7 - Detailed Design of Structures: Part 1 - Box Conduit

DA, New England Division, CE, Waltham, Mass. 02154 12 December 1974

TO: HQDA (DAEN-CWE-B), WASH DC 20314

The following actions and clarifications are presented in response to referenced comments:

a. Paragraph 2. Earthquake Forces (Paragraph 12e, DM-7) shall be revised as follows:

Earthquake forces for the conduits and junction box will be considered in final design using 0.05g, loaded laterally, of all concrete elements and dead loads above the top of base slab in addition to applying laterally the difference of at-rest and active pressure opposite to the direction of motion. Allowable stresses will be increased one-third.

b. Paragraph 3a. The possible variation of soil conditions for the conduits founded on till is in a spectrum above that required to produce uniform soil bearing pressure. The modulus of subgrade reaction required to produce uniformity was found to be less than 20 kips per cubic foot (see inclosure) which is the modulus for loose sand or soft clay, similar to conditions existing at Sections 7 and 9 Conduits which are founded on piles. This variation is remote in the reaches proposed for Sections 2, 4 and 5 and, as presently designed, the conduits are adequate for a range of subgrade reaction above 60 kips per cubic foot.

c. Paragraph 3b. Reference the article by Karl Terzaghi entitled "Evaluation of Coefficients of Subgrade Reaction" found in Geotechique March 1955 Volume 5, Number 1, Page 314 Table 1, Page 315 Table 2 and correction factor, equation 8. The dense and hard range was selected to represent till characteristics and reduced by the referenced correction factor, resulting in ks of 170 kips per cubic foot. Note the units expressed for Tables 1 and 2 are Tons per cubic foot.

d. Paragraph 4. The sheet pile cutoff will be relocated and placed vertically just inside the first upstream batter pile.

FOR THE DIVISION ENGINEER:

George T. Sarandis
for JOHN Wm. LESLIE
Chief, Engineering Division

Incl (2 cys)
As stated

DAEN-CWE-B (NEDED-E, 25 Oct 74) 1st Ind
SUBJECT: Park River Local Protection, Connecticut River Basin, Hartford, Connecticut, Design Memorandum No. 7 - Detailed Design of Structures: Part 1 - Box Conduit

DA, Office of the Chief of Engineers, Washington, DC 20314 4 Dec 74

TO: Division Engineer, New England
ATTN: NEDED-E

1. Approved, subject to the comments furnished in the following paragraphs.

2. Paragraph 12e. This paragraph states that seismic forces have not been considered but GDM Phase II, Part I - Box Conduit, paragraph 18 states that an earthquake acceleration of .05g will be considered. This discrepancy should be corrected.

3. Paragraph 13.

a. The possibility of non-uniform soils conditions resulting in uniform bearing pressures should be investigated. If it is possible that uniform bearing can occur at some time during the life of the box conduit, the design should be checked for such a condition. This loading condition may result in overstressing the thin sections of the boxes.

b. The selected value of modulus of subgrade reaction for foundation till is unreasonably high and should be justified. In addition, the stated units for this constant are incorrect.

4. Appendix A, Plate 7A-3, Typical "Tee" Wall Section. Consideration should be given to a pile arrangement that uses batter pile and vertical pile. This would eliminate the need for driving the MA-22 sheet piling cut off on a batter. Consideration should also be given to a scheme using vertical pile and a key and sloping base to develop passive resistance.

FOR THE CHIEF OF ENGINEERS:

1 Incl
wd

HOMER B. WILLIS
Chief, Engineering Division
Directorate of Civil Works

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
424 TRAPELO ROAD
WALTHAM, MASSACHUSETTS 02154

REPLY TO
ATTENTION OF:

NEDED-E

25 October 1974

SUBJECT: Park River Local Protection, Connecticut River Basin, Hartford, Connecticut, Design Memorandum No. 7 - Detailed Design of Structures: Part I - Box Conduit

HQDA (DAEN-CWE-B)
WASH DC 20314

In accordance with ER 1110-2-1150, there is submitted herewith for review and approval Design Memorandum No. 7, Detailed Design of Structures: Part I - Box Conduit, for the Park River Local Protection, Connecticut River Basin, Hartford, Connecticut.

FOR THE DIVISION ENGINEER:

John Wm. Leslie

Incl (10 cys)
as

JOHN Wm. LESLIE
Chief, Engineering Division

WATER RESOURCES DEVELOPMENT PROJECT

PARK RIVER LOCAL PROTECTION
CONNECTICUT RIVER BASIN
HARTFORD, CONNECTICUT

DESIGN MEMORANDA INDEX

Number	Title	Anticipated Submission Date	Date Submitted	Date Approved
1	Hydrology		16 Feb 73	12 Apr 73
2	GDM - Phase I - Plan Formulation		30 Mar 73	16 Jul 73
2	GDM - Phase II - Project Design, Site Geology & Interior Drainage			
	Part I - Box Conduit		30 Aug 74	18 Oct 74
	Part II - Auxiliary Conduit	Nov 74		
3	Hydraulic Analysis	Oct 74		
4	Concrete Materials	Jan 75		
5	Embankment & Foundations			
	Part I - Box Conduit	Jan 75		
	Part II - Auxiliary Conduit	Aug 75		
6	Pumping Stations	Nov 74		
7	Detailed Design of Structures			
	Part I - Box Conduit		25 Oct 74	
	Part II - Auxiliary Conduit	Aug 75		
8	Site Geology			
	Part II - Auxiliary Conduit	Mar 75		

WATER RESOURCES DEVELOPMENT PROJECT

PARK RIVER LOCAL PROTECTION

CONNECTICUT RIVER BASIN

HARTFORD, CONNECTICUT

DESIGN MEMORANDUM NO. 7

DETAILED DESIGN OF STRUCTURES

PART I - BOX CONDUIT

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASSACHUSETTS

OCTOBER 1974

WATER RESOURCES DEVELOPMENT PROJECT

PARK RIVER LOCAL PROTECTION
CONNECTICUT RIVER BASIN
HARTFORD, CONNECTICUT

DESIGN MEMORANDUM NO. 7
DESIGN OF STRUCTURES

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WATER RESOURCES DEVELOPMENT PROJECT

PARK RIVER LOCAL PROTECTION
CONNECTICUT RIVER BASIN
HARTFORD, CONNECTICUT

A. PERTINENT DATA

PURPOSE	Flood Control
LOCATION	
State	Connecticut
County	Hartford
City	Hartford
River	Park River & North and South Branch Park River
River Basin	Connecticut
PARK RIVER DRAINAGE AREAS	
Park River at the Mouth	78.7 Square Miles
North Branch Park River	27.7 Square Miles
South Branch Park River	47.0 Square Miles

RECORD OF MAJOR FLOODS

Year	Month	Peak Discharge, c.f.s. *
1936	March	5,400
1938	January	5,650
1938	September	5,320
1955	August	14,000
1955	October	6,420

*Gage at Riverside St. on Park River about 600 feet below junction of North and South Branches.

AREAS	
Subject to flooding, Acres	3,000
Inundated 1955 flood of record, Acres	2,300
Properties protected	Industrial, commercial, residential and public

TWIN-RECTANGULAR BOX CONDUIT

Material: Reinforced Concrete

Conduit Sections	Length in Feet	Half Section Width	Half Section Height
Existing (12,743 ft):			
Original (1944)	5,600	30'-0"	19'-4"
Section 1	1,213	34'-0"	26'-6"
Section 3	1,710	34'-0"	26'-6"
Section 6	1,460	36'-0"	27'-6"
Section 8	2,760	22'-0"	25'-0"
Proposed (4,036 ft):			
Section 2	1,232	34'-0"	26'-6"
Section 4	1,337	34'-0"	26'-6"
Section 5	103	36'-0"	27'-6"
Section 7	1,044	22'-0"	25'-0"
Section 9	320	22'-0"	25'-0"

SYSTEM DISCHARGE FOR DESIGN FLOOD CONDITION

(Conn. River Stage of 30 ft. MSL)

Total System Design Discharge		23,800 CFS
Park River Conduit	18,400	
Auxiliary Conduit	5,400	
Total	23,800 CFS	
Coincident Discharges in		
North Branch Conduit	5,400	
South Branch Conduit	18,400	
Total	23,800 CFS	

B. INTRODUCTION

1. PURPOSE. The purpose of this memorandum is to present for review and approval the basic structural design criteria for the structures to be included in the Park River Local Protection Project, Hartford, Connecticut. The basic criteria, typical design computations, definitive plans and other pertinent data are presented.

2. SCOPE. This memorandum covers the structural design of the twin box conduits, junction structure, the inlet structure and headwalls at the North Branch. The auxiliary conduit and pumping stations will be the subject of future Design Memoranda.

3. PREVIOUS REPORTS. A detailed report on the design of structural features for this project has not been previously submitted. The latest previous description of the project and proposed structures is contained in Design Memorandum No. 2, Phase II, "Project Design, Site Geology and Interior Drainage, Part I - Box Conduit," submitted 30 August 1974.

4. LOCATION. The structures for the project are all located within the boundaries of the City of Hartford. Structures are located on the North and South Branches, at their confluence and on the Park River downstream of the confluence. A General Plan of the project area is shown on Plate 7A-1.

5. DESCRIPTION OF PROJECT. The recommended protection requires the construction of five sections of twin rectangular box conduit totaling 4030 feet in length, 8,900 feet of 22'-0" diameter reinforced concrete conduit, 195 feet of tee type floodwalls, 380 feet of I-type floodwalls, an inlet structure, a reinforced concrete junction structure, one complete pumping station and one pumping station superstructure. Plans and elevations of the project structures covered by this memorandum are shown on Plates 7A-2 to 7A-12.

C. HYDROLOGY AND HYDRAULICS

6. GENERAL. Approved Design Memorandum No. 1 contains the hydrology for the project. This data is summarized in Design Memorandum DM-2 Phase II. Detailed hydraulic analysis and model study results on the junction structure will be presented in Feature Design Memorandum No. 3, "Hydraulic Analysis."

D. FOUNDATIONS

7. FOUNDATION CONDITIONS.

a. General. The Park River flows into the project area from the north over local alluvial features underlain by thick deposits of varved

silts and clays. The typical geologic profile displays a variably thick deposit of compact glacial till intermediate to the clay and bedrock surface. Recent fills of variable composition are in evidence along the stream channel throughout the project area. The thickness of the overburden deposits thin rapidly to the south and east becoming conformable with the bedrock surface at the easterly limits of the project. The depth to the water table throughout the limits of the project is generally less than 10 feet below the land surface during part of the year.

b. At Proposed Conduit Locations.

(1) Section 2, Station 8+68 to Station 21+00. Bedrock at the section varies between 0 and 20 feet M.S.L. and is overlain by a compact glacial till from 1 to 15 feet in thickness. The bedrock consists of a moderately hard gently dipping red shale and sandstone. The bedrock surface has been variably eroded by the Park River and by natural excavations but, in general, presents a gently sloping bedrock surface of adequate bearing for support of the proposed structures. A projected fault defined by the alinement of the basalt ridge at Zion Hill and previous borings made for a city tunnel intersects the project at the lower end of the structures in the vicinity of Station 11+00. The fault projection trends northeasterly with the upthrown side of the fault to the southeast. There is no surficial evidence of the fault at the location of the project structures. High natural fills and buildings border the present stream channel on both sides. This section of conduit will be founded on rock or till with limited areas requiring removal of unsuitable material and replacement with concrete fill.

(2) Section 4, Station 38+10 to Station 51+33. The bedrock surface dips to the north normal to the direction of the conduit section and presents an irregular surface along the alinement of the section between +5 and -5 M.S.L. On the easterly side the bedrock is overlain by a compact glacial till varying in thickness from 2 to 20 feet and is blanketed by a layer of soft varved silt and clay which continues up the slope above the cut bank of Pope Park. These upper slopes surficially display typical slide failures with numerous springs which have developed pronounced gullies normal to the stream. The west side of the alinement consists of a compact glacial till overlying the bedrock surface and is mantled by recent fills. Seismic profiles supplemented by borings have delineated the rock surface along the alinement profile and normal to the stream along the critical areas of slope stability. Bedrock throughout the section consists of a moderately hard red shale and sandstone generally unweathered except for localized weathering along open high angle joints. Thin calcareous coatings along the joints and calcite seams occur throughout the rock cores. Section 4 Conduit will be founded on the compact till or rock. Limited areas of soft clay will be removed and replaced with fill concrete.

(3) Section 5, 7 and Junction Structure, Station 1+55 to 12+20. Geologic conditions applicable to the upstream limits of Section 4 are generally conformable with Section 5 except for the addition of a continuous thickness of soft silt and clay on the northeast side of the structure. A continuation of the compact till and soft silt and clay layer are relatively uniform in the downstream limits of the section. Beyond the mid-point the bedrock drops rapidly with an increase in the thickness of the varved clay deposits. The till mantle maintains a relatively uniform thickness averaging approximately 20 feet over the entire alinement with ground water surface throughout this reach approximating the stream gradient. The bedrock surfaces are moderately irregular with a gradual deepening of the rock from elevation 0 at the downstream limits to in excess of elevation -60 at the junction with existing Section 8. Fills of highly variable composition encroach on both sides of the present streams with thicknesses averaging from 15 to 20 feet. Section 5 and the junction structure will be founded on the compact till with fill concrete replacing soft clay deposits overlying the till. At Sections 7 where the layer of soft clay below the base exceeds 10 feet in thickness, the conduit will be founded on end bearing piles.

(4) Section 9 and Intake Structure, Station 39+80 to 43+00. Northerly of Section 8 the bedrock surfaces as depicted by borings and seismic survey rises from elevation -90 at Farmington Avenue to elevation -60 approximately 800 feet north. The moderately hard red shale is overlain by thick deposits of compact glacial till as great as 45 feet and soft varved silt and clay to depths of 90 feet. The ground water surface approaches the natural stream gradient. Fill deposits of variable thickness blanket the soft silt and clay deposits. Bedrock in this section consists of a moderately hard red shale and sandstone with little surface weathering. The conduit and concrete floodwall in this section will be founded on end bearing piles.

c. Foundation Design. Detailed foundation design studies will be presented in Design Memorandum No. 5, "Embankments and Foundations," Part I. Maximum bearing pressures on the till and rock foundation is 6,000 pounds per square foot.

E. CONCRETE

8. GENERAL. Concrete for this project will be supplied from nearby commercial sources which have been previously found acceptable for use in Civil Works Projects. Concrete will be covered in detail in Design Memorandum No. 4, "Concrete Materials."

F. ARCHITECTURAL DESIGN

9. GENERAL. The major architectural treatment for the project will be in the form of landscaping and use of the areas above the buried

conduits. The proposed landscape architecture is discussed in Design Memorandum No. 2, Phase II, Part I. The only location where the structures covered by this memorandum are visible is at the inlet structure at the Section 9 Portal. The proposed alignment of the headwalls, trash bars and conduit entrance will present a pleasing appearance from the apartment facility located to the north. Exposed concrete at the headwalls and portal will be architecturally treated by use of feature strips to reduce the visual impact.

G. STRUCTURAL DESIGN

10. PURPOSE AND SCOPE. This section presents the design criteria, basic data, assumptions and procedures used in the structural design of twin rectangular reinforced concrete conduits, concrete junction structure, inlet structure, concrete headwalls and steel sheet pile headwalls. Typical computations are included in the Appendix showing the stability, maximum loading conditions and typical designs for critical sections. A graphical presentation of loading conditions is shown on Plate No. 7A-15. The structural design of the auxiliary conduit and pumping stations are not included in this memorandum. They will be the subject of future design memoranda.

11. DESIGN CRITERIA.

a. General. All working stresses conform to those specified in Engineering Manual EM 1110-1-2101, Working Stresses for Structural Design, dated 1 November 1963. General loading conditions, design assumptions and other design criteria are based on the following applicable parts of the Engineering Manual for Civil Works: Standard Practice for Concrete (EM 1110-2-2000, 1 Nov. 1971), Details of Reinforcement-Hydraulic Structures (EM 1110-2-2103, dated 21 May 1971), Conduits, Culverts, and Pipes (EM 1110-2-2902, dated 3 March 1969), Retaining Walls (EM 1110-2-2502, dated 29 May 1961), Wall Design - Flood Walls (EM 1110-2-2501, Change 3, dated 18 June 1962), and Design of Pile Structures and Foundations (EM 1110-2-2906, Appendix A to Draft, dated 16 November 1970).

b. Concrete. Concrete working stresses are in general accordance with ACI Standard Building Code Requirements for Reinforced Concrete (ACI 318-63), as modified by EM 1110-1-2101, using concrete with a minimum ultimate compressive stress of 3,000 pounds per square inch for walls and 4,000 pounds per square inch for the twin conduits, junction and inlet structures.

Flexure	(Extreme fiber stresses in compression)	- P.S.I.
	Twin conduits and junction structure	- 1800
	Headwalls	- 1050
	Inlet arch and trash bars	- 1400

c. Reinforcement.

(1) General. Details of reinforcement and placement will be in accordance with EM 1110-2-2103, dated 21 May 1971, Details of Reinforcement - Hydraulic Structures.

(2) Grade and Working Stress. All reinforcement in the structures is designed for a working stress of 20,000 p.s.i. in flexural tension. The reinforcement will be deformed bars made of new billet steel, intermediate grade (ASTM A-615, Grade 40), conforming to the requirements of Federal Specification QQ-S-632C, Type II, Class B40.

(3) Spacing. The clear distance between parallel bars will not be less than 1-1/2 times the nominal diameter of the bars except that in no case will the clear distance between parallel bars be less than 1-inch, or 1-1/2 times the maximum size of the coarse aggregate. No. 18 bars will not be spaced less than 8" center to center. Parallel reinforcement in two or more layers will have the bars directly over each other with not less than 6" clear distance between layers.

(4) Minimum Cover for Reinforcement.

Item	Min. Cover (inches)
Bottom of base slabs	6"
All other surfaces	4"

(5) Splices. All lap splices are in accordance with the ACI Building Code. Tension splices will be staggered longitudinally so that no more than half the bars are spliced at any section within the required lap length. Bars larger than No. 11 will be butt spliced in accordance with requirements of EM 1110-2-2103, dated 21 May 1971, Details of Reinforcement - Hydraulic Structures.

(6) Temperature and Shrinkage Reinforcement. Temperature and shrinkage reinforcement is provided where main reinforcement extends in only one direction. Such reinforcement will provide for a ratio of steel area to concrete area of .001 at each face with a minimum of No. 4 bars at 12 inches spacing in thin sections and a maximum of No. 6 bars at 12 inches in thick sections.

(7) Minimum Transverse Steel. A minimum ratio of steel area to concrete area of .002 will be provided at each face in the transverse direction of the twin conduits and junction structure including areas carrying no computed stresses.

d. Steel Sheet Piling. The design of the steel sheet piling utilized in the I-type head walls is in accordance with EM 1110-2-2906, Appendix A to Draft. The piling will be ASTM Designation A-328 with an allowable working stress of 20,000 p.s.i. Investigation has shown that no protection of the piling against corrosion is required.

e. Steel Bearing Piles. Steel bearing piles supporting the concrete headwalls and portions of the twin conduits will be H-piles driven into the till overlying bedrock. They will conform to ASTM designation A-36 with an allowable compressive stress of 10,000 p.s.i. Pile sections have been selected for a design capacity of 50 tons at the concrete headwalls and 100 tons where used to support the twin conduits and inlet structure. Test piles will be loaded to verify design assumptions. No protection against corrosion is required.

f. Increase in Normal Working Stresses. Allowable working stresses have not been increased for any loading conditions.

g. Waterstops. Rubber or polyvinylchloride waterstops will be used in all concrete contraction and expansion joints.

12. BASIC DATA AND ASSUMPTIONS.

a. Loads.

(1) Dead Loads

Concrete	150	p.c.f.
Soil, saturated	120	p.c.f.
Soil, submerged	60	p.c.f.

(2) Live Loads

Water	62.5	p.c.f.
Wind	50	p.s.f.
Highway	A.A.S.H.O.	HS-20 Loading

Twin Conduits: The following minimum live loading was applied to the top slab of the conduits to provide for future land use over them.

Depth of Fill Over Top Slab	Min. Live Load Lbs./Sq. Ft.
0	500
2	400
4	300
6	200
8	100
9	0

Trash bar loading at inlet, 5 ft. head differential

Junction Structure:

Top slab	-	450 lbs. per sq. ft.
Top slab beams	-	150 lbs. per sq. ft. over entire slab

b. External Water Pressures. Hydraulic pressure is assumed to act over the entire area in question under the full head available. Various elevations of ground water were assumed for critical loading conditions.

For design of the twin conduits and junction structure, the following assumptions were made:

(1) When conduit is empty (for maximum bouyancy) ground water is assumed mid-height between the invert of lateral drain and top of roof slab.

(2) When conduit is full under pressure (for maximum foundation reaction) no ground water is assumed.

(3) For design of exterior walls only, ground water is assumed at top of conduit with no water inside.

For design of headwalls; hydrostatic pressure, assuming water to the top of the wall, is taken as full pressure on riverside and varying from full head at bottom of riverside base to zero at ground surface landside by the creep method.

c. Internal Water Pressure. An internal bursting pressure equal to the difference in head between maximum hydraulic grade line and the soffit elevation of the roof slab was assumed. The differential for the various sections of conduits is as follows:

Section No.	Bursting Head
2	6'
4	7'
5	6.5'
7	varies 7' to 10'
9	6'

For design of the center parting wall a construction condition was assumed with the water inside one barrel to a depth of six feet and no water in the adjacent barrel. For this condition it was assumed the roof slab is not in place.

d. Earth Pressure. Earth pressures were determined in general accordance with EM 1110-2-2502, Retaining Walls, dated 29 May 1961. Lateral pressure coefficients were assumed for either at rest or active pressure conditions to attain the critical loading for the various components of the twin conduits. An at rest coefficient of 0.5 was assumed.

e. Earthquake Forces. Earthquake forces have not been considered in the design of the structures covered in this memorandum.

f. Frost Protection. The base of all structures are buried well below the 4-foot depth required for frost protection in the area.

g. Uplift Factor of Safety. A minimum factor of safety against uplift of 1.1 is provided for the conduits, assuming ground water mid-height between the invert of lateral drain and top of roof slab and no water in the conduit. For this condition no live load is assumed acting; friction between the soil and exterior walls is considered developed by at rest pressure on the walls, and the minimum depth of fill is assumed in place on the roof slab. A friction coefficient of 0.5 was assumed for soil against the walls.

h. Location of Resultants. In the investigations of stability of the headwalls, the resultant of the horizontal and vertical forces has been held within the middle half of the base when flood is at the top of the wall and within the middle third of the base when flood is within three feet of the top of the wall.

i. Sliding. Sliding forces on the headwalls will be resisted by the steel foundation piles.

13. PREPARATION OF DESIGN COMPUTATIONS. Extensive use has been made of the computer in analyzing the twin conduits and junctions structure. The program "EFFRAM, Analysis of Plane Frames on Elastic Foundation" (Program No. 713 D0F7110) was used. This program utilizing the direct stiffness method of matrix analysis considers the foundation to be elastic based on Winkler's concept of analogous springs. A corrected modulus of subsoil reaction of 170,000 pounds per cubic foot was used for conduits founded on till and a corrected modulus of 500,000 pounds per cubic foot for conduits founded on rock. Representative calculations of typical design sections have been included in the Appendix.

14. DESIGN OF STRUCTURES.

a. General. The structures required as part of the project and covered in this memorandum include five sections of twin box conduits totaling 4,036 feet, a junction structure joining conduit Sections 4,

5, and 7, concrete headwalls and inlet structure with trash structure at the north branch. The twin conduits will enclose reaches of open channel between existing conduits. Construction of the conduits will require construction sheeting, soldier beams and lagging, river diversion and roadway detours in certain areas. Construction procedures and diversion plans are discussed in detail in Design Memorandum No. 2, Phase II, Part I, submitted 30 October 1974. A general plan of structure locations is shown on Plate 7A-1.

b. Twin Conduits.

(1) General. The five sections of twin conduit are of various lengths and cross sectional area, and will generally be constructed in 30-foot long monoliths. Section 2 is approximately 1,232 feet long with each barrel measuring 34' wide by 26.5 high inside. It will extend from the east end of existing Section 3 downstream to the west end of existing Section 1. The proposed conduit will carry the Flower and Broad Street crossings in their existing locations, eliminating the need for the existing highway bridges and a pedestrian overpass. This section of the conduit will be founded on rock or till and will be constructed in two halfs to accomodate channel diversion. Section 4 will extend from the proposed junction structure downstream to the west end of existing Section 3, a distance of approximately 1,323 feet. Eash barrel will measure 34 feet wide by 26.5 feet high inside. The conduit will be founded on rock or till and the barrels must be built in two stages to provide for channel diversion. Section 5 will enclose the short reach between the end of existing Section 6, carrying the south branch, and the junction structure, a distance of 125 feet. Each barrel will measure 36 feet wide by 27.5 feet high. The section will be founded on compact till and the cross section must be constructed in two halfs to allow for water control. Section 7 will extend from existing Section 8, carrying the north branch, to the proposed junction structure, a distance of 1,065 feet. The inside dimensions of each barrel will be 22 feet wide by 25 feet high. The structure will be supported on steel piles located beneath the exterior and parting walls. The entire cross section of each monolith, except for three monoliths adjacent to the junction structure, can be constructed in one stage as the river will be carried in a temporary flume located outside the limits of conduit construction. Soldier beams, lagging and sheeting existing on the south side of the proposed conduit will be utilized as construction sheeting. The conduit will carry Laurel Avenue in its present location eliminating the existing bridge and the Laurel Avenue detour crossing during construction. Section 9 is an extension of the north end of Section 8, the existing intake conduit for the north branch of the river. The proposed conduit will carry the Farmington Avenue crossing in its present location and provide for a detour crossing during construction

at Framington Avenue. The north end of Section 9 will be the new inlet and will be flared to join new concrete headwalls which will follow a serpentine alignment. Concrete trash bars and a drop inlet will be integral features of inlet end of the conduit. The conduit will be supported by steel piles located under the walls driven into till. The culvert cross section must be constructed in two stages to accomodate the channel diversion scheme.

(2) Loading Conditions. Seven general loading conditions were considered in design of the conduits. Four of these cases produced critical design parameters for the various points on the structure. In addition, special construction and service loading conditions applicable to specific monoliths were applied where appropriate. The four critical loadings are graphically presented on Plate 7A-15.

(a) Loading Case 2. Assumes earth and live load acting on roof of the conduit with active earth pressure and surcharge equivalent to live load acting on exterior walls. No water is assumed in the conduit for this loading condition. This loading produces the critical moments for the design of reinforcing at bottom of roof slab between walls, top of roof slab over parting wall for all sections. It also produces critical moments for the design of reinforcing at top of base slab between walls, and bottom of base slab at parting wall for those sections founded on till or rock.

(b) Loading Case 3. Assumes conduit full with applicable bursting head, active earth pressure on the exterior walls, earth in place on roof slab and no ground water outside. This loading produced maximum foundation loading for all sections and a check on reversal of stress at inside faces at corners. A similar loading considering live load on the roof with normal ground water outside was less critical for foundation design. This case also produced maximum moments for design of reinforcement at top of base slab at walls and bottom of base slab between walls for sections founded on piles.

(c) Loading Case 6. Assumes ground water to top of conduit with sufficient water inside to balance vertical forces, at rest earth pressure equal to 0.50 of its submerged weight and lateral water pressure acting on the exterior walls and earth in place on roof slab. This loading was intended to produce maximum moment for design of reinforcing on inside face of exterior walls, but was not critical for any of the conduit design sections.

(d) Loading Case 7. Same as Case 6, except live load acting on roof slab and equivalent live load surcharge acting on exterior walls. This loading produced maximum moments for reinforcing on exterior face of roof slab at the exterior walls and for both faces of the exterior walls.

The remaining general loading cases with varying conditions of ground water and conduit flow were considered and found not to be critical for design.

(3) Typical Design Sections. There is one typical design cross section for each of the conduit sections. In addition there are two design cross sections covering the highway crossings at Broad Street and Farmington Avenue in Sections 2 and 9 respectively. The typical design cross section for the section in which they are located will suffice for the Flower Street and Laurel Avenue crossings. The design sections for highway crossings will be applicable for monoliths at the existing crossing and for those monoliths affected by detours of the roadways during construction. Structural and geometric details of the design cross sections are shown on Plate Nos. 7A-13 and 7A-14. Typical stability computations for the conduits are included in the Appendix.

c. Junction Structure.

(1) General. The junction structure to be located at the confluence of the north and south branches will combine the flows from the conduit system enclosing each branch. It will divide the outlet flows between the existing outlet at Section 1 and the proposed auxiliary conduit which will outlet downstream of Section 1. The structure measures approximately 138 feet in the direction of flow with the maximum width equaling the combined width of Sections 5 and 7 entering it. The reinforced concrete structure will consist of a roof slab supported by beams which will run continuous over internal concrete piers resting on a base slab of uniform thickness. The exterior walls will be constructed integral with the base and roof slabs. The base slab will be founded on till or on fill concrete which will replace a layer of varved clay overlying the till.

(2) Loading Conditions. Due to its size and configuration, the structure will be designed in parts with the loading selected to produce critical design conditions in each of the parts. The live loading has been selected to allow for future use of any part of the roof for vehicle parking. The general loading conditions for stability and design of the base slab and exterior walls will be the same as those listed for the twin conduits. The bearing pressures on the base slab will be computed considering the foundation soil to be elastic. The design of the roof slab and walls will consider stresses induced by deflection of the roof beams, under various conditions of roof slab loading.

(3) Typical Design Sections. Because of its geometry, there is, as such, no typical design section for the junction structure.

Design computations have been made for the roof slab at a specific location and for a representative roof beam. The base slab will be analyzed necessarily in its entirety. The structural details for these designs and the general layout of the structure have been shown on Plate Nos. 7A-7 and 7A-8. Calculations covering the structure stability and representative designs have been included in the Appendix.

d. Concrete Headwalls.

(1) General. Tee-type concrete floodwalls will extend from the flared ends of the intake structure (Section 9) to form the headwalls for the North Branch conduit. This wall will have a serpentine configuration in plan and the elevation of the base slabs will be stepped generally paralleling the slope grading. The east flood wall will have approximately 135 feet of tee-type wall with a maximum height of 32.5 feet. Approximately 60 feet of tee-type wall with the same maximum height will be included in the flood protection on the west side of the conduit. The tee-type walls will be founded on steel H-piles driven into the till overlying the bedrock.

(2) Loading Conditions.

(a) Loading I - Water surface to top of wall riverside. Uplift is assumed varying from full head at bottom of base slab riverside to zero at the ground surface landside by method of creep. Resultant to fall within middle-half of base.

(b) Loading II - Same as Loading I, except water surface 3 feet below the top of the wall and the resultant of forces to fall within the middle-third of the base.

(c) Loading III - No flow in river and 50 pounds per square foot wind load applied from landside on the height of wall above landside grade. A 3'-0" earth surcharge in addition to the wind load is applied landside to allow for vehicular parking along the wall. Resultant of forces to fall within middle-third of base under this loading condition.

(d) Loading IV - Because of special ponding conditions behind the east side floodwalls, they will be checked for an additional loading case (Case IV) which assumes water at elevation 27.0 riverside and at elevation 40.0 landside with a 50 pounds per square foot wind load applied to exposed height of wall from landside.

(3) Typical Design Sections. Several design sections will be required for the varying conditions of wall height and finished grade elevation. The first three sections of wall on the east side will be

analyzed as total monoliths rather than as one foot long sections because of the trapezoidal shape of their base slabs. The Appendix contains representative design calculations for wall sections and stability calculations for one of the curved monoliths. Plate No. 7A-3 shows the general layout of the headwalls and typical structural details. Loading conditions are shown on Plate 7A-15.

e. I-Type Sheet Pile Headwalls.

(1) General. Where the required flood protection at the headwall is 10 feet or less, I-type walls will be used. They will extend from the ends of the tee walls on each side to a point where the grade meets flood elevation. This will require 265 feet of I-type floodwall on the west side and 115 feet on the east side. The walls will consist of Z-32 section steel sheet piling driven the required penetration into existing soil. The portion of wall extending above grade and the top 5'-6" portion below grade will be constructed of reinforced concrete. The steel piling will be embedded 4 feet into the concrete section.

(2) Loading. The critical loading used in design of the I-walls assumes water at the top of the wall riverside and at the ground surface landside. Ground surface is assumed at the same level of both sides of the wall.

(3) Typical Design Section. The design computations for the typical I-wall section are included in the Appendix. A general layout of the walls and structural details are shown on Plate No. 7A-3.

f. Inlet Structure.

(1) General. The inlet is a pile supported concrete monolith consisting of an entrance weir forming a drop inlet and a section of twin conduit transitioning the inlet section to that of the conduit at Section 9. Also included are five 18-inch diameter concrete trash bars extending from the top of the weir to a horizontal concrete arch spanning the mouth of the inlet structure. The thrust of the arch is carried to the foundation by concrete buttresses built integral with the inlet walls. The major features of the inlet structure are shown on Plate 7A-4.

(2) Loading Conditions. The transition portion of the inlet structure is subject to the same loadings applied to the twin conduits. The trash bars, arch and buttresses are designed for two conditions of static loading. Condition 1 considers the inlet completely clogged with a five foot differential head acting on the projected area of the inlet at the trash bars. This condition produces maximum thrust in

the arch and governs the design of the buttresses. Condition 2 considers one-half of the trash ring loaded, thereby producing maximum moment in the arch. The dynamic condition of water flowing freely through the trash bars is not critical for design. The static loading governs the design of all members.

(3) Design. The stability of the inlet monolith will be checked as a unit. The steel piles supporting the base slab will be designed for 100 ton capacity. The base slab will be designed for two way action. Because the arch will carry no load under normal conditions, it will be designed as three hinged to minimize bending induced by shrinkage and temperature changes. Representative design calculations for the various parts of the inlet structure are included in the Appendix.

15. CONSTRUCTION PROCEDURE. Certain procedures will have to be followed to accomodate stream flow and maintain traffic during construction. The necessary temporary construction and phasing of operations are discussed in detail in Design Memorandum No. 2, Phase II, Part I. Plate 2A-13 of that memorandum is a plan of the phasing of construction operations for conduit sections 4, 5, 7 and the junction structure. Basically, under the proposed diversion plan, section 7 and the junction structure may be constructed in complete monoliths, as the flow will be carried in a temporary flume outside the limits of construction. At sections 2, 4, 5 and 9, the monoliths must be constructed in two-half sections (transverse). The longitudinal construction of monoliths must also be completed in required sequence to provide detour crossings for traffic and utilities in sections 2, 7 and 9. Temporary construction will include a semi-circular flume, cantilevered and braced sheeting, tied back soldier piles with lagging and sand bag dikes. Cross sections showing typical sheeting installation and diversion locations are shown on Plate Nos. 2A-4, 5, 7, 8, 9, 10 and 11 of Design Memorandum No. 2, Phase II, "Project Design, Site Geology and Interior Drainage," Part I - Box Conduit.

H. COST ESTIMATES

16. ESTIMATE OF COSTS. No feasible alternates to the proposed construction were found as the project is basically the completion of a partially constructed works. Comparative estimates are, therefore, not applicable. The latest estimate of construction costs is contained in Design Memorandum No. 2, Phase II, Part I. That estimate is currently being revised for refined quantities and unit costs, and will be presented in Design Memorandum No. 2, Phase II, "Project Design, Site Geology and Interior Drainage," Part II - Auxiliary Conduit, which is scheduled for submittal in November 1974.

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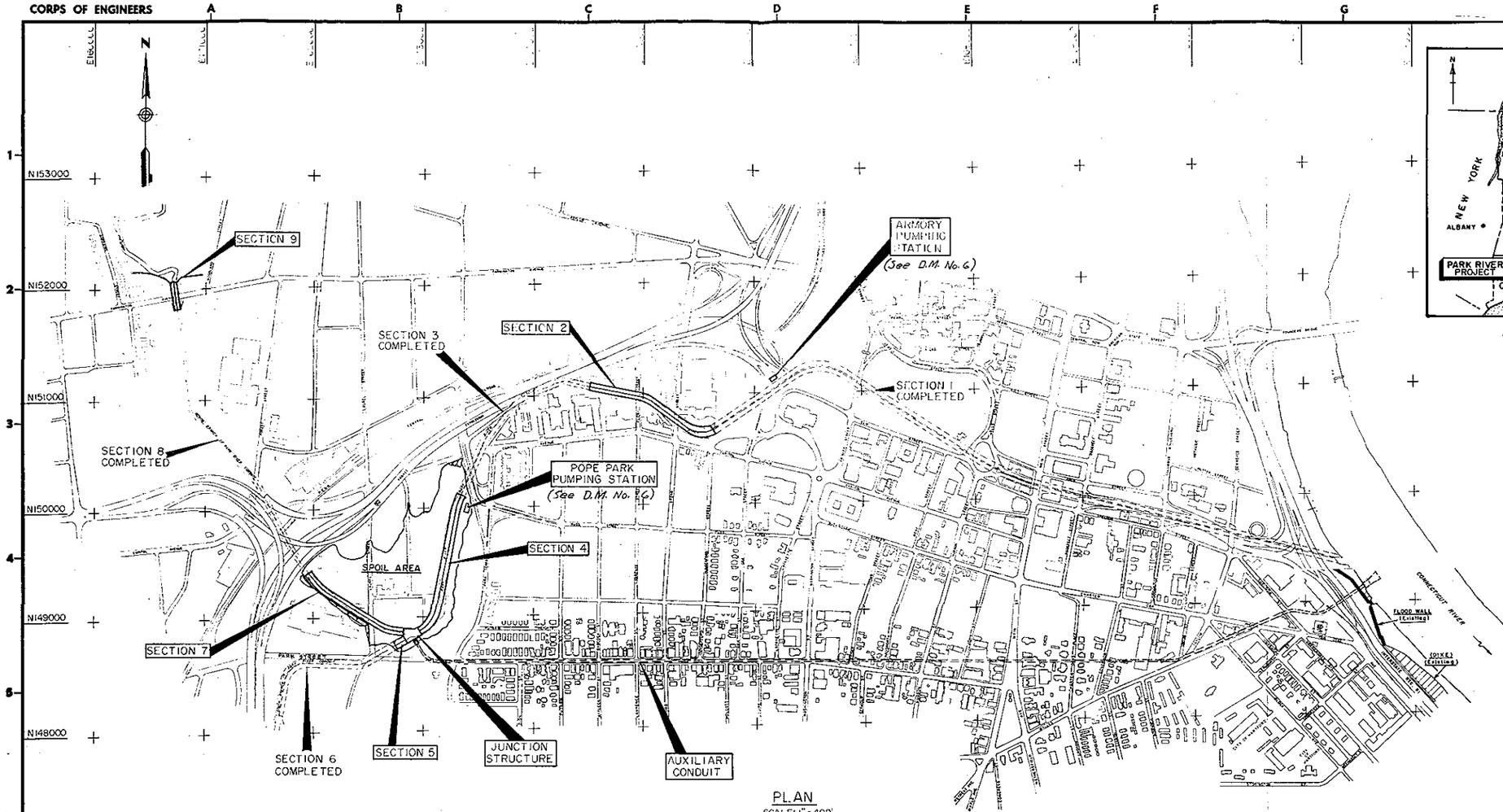
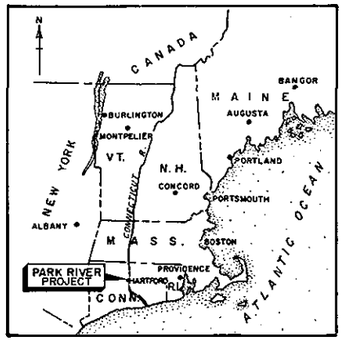


PLAN
SCALE: 1" = 400'



LOCATION MAP
SCALE IN MILES
40 0 40 80

LEGEND

- ● 5 Greater Hartford Flood Commission Boring (1960-1963)
- ● FC3 Flood Commission Boring (1960)
- ● BH6 Connecticut Highway Department (1959)
- ● FD5 U.S. Army Corps of Engineers Boring
- —— A5 Seismic Line
- W Water
- SD Strom Drain
- SS Sanitary Sewer
- G Gas
- E Electric
- T Telephone
- Bedrock Outcrop

GENERAL NOTES:

1. Elevations refer to Mean Sea Level Datum.
2. All elevations are plus except as noted otherwise.
3. Plane coordinates refer to Connecticut Lambert Grid System.
4. For Geologic Profile and Logs of Typical Explorations see Design Memo. No. 2, Phase II, Part I "Project-Design, Site Geology and Interior Drainage"

GRAPHIC SCALES
1" = 400' — 200' 0 400' 800'

SAFETY PAYS

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WALTHAM, MASS.

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CHIEF, SECTION
APPROVAL RECOMMENDED:
CHIEF, TECH. ENG. BRANCH
REVIEWED:
PROJECT ENGINEER
APPROVAL RECOMMENDED:
CHIEF BRANCH

WATER RESOURCES DEVELOPMENT PROJECT
PARK RIVER LOCAL PROTECTION
GENERAL PLAN

HARTFORD CONNECTICUT

APPROVED DATE
CHIEF, ENGINEERING DIVISION

SCALE | SPEC. NO.
DRAWING NUMBER
SHEET

PLATE 7A-1

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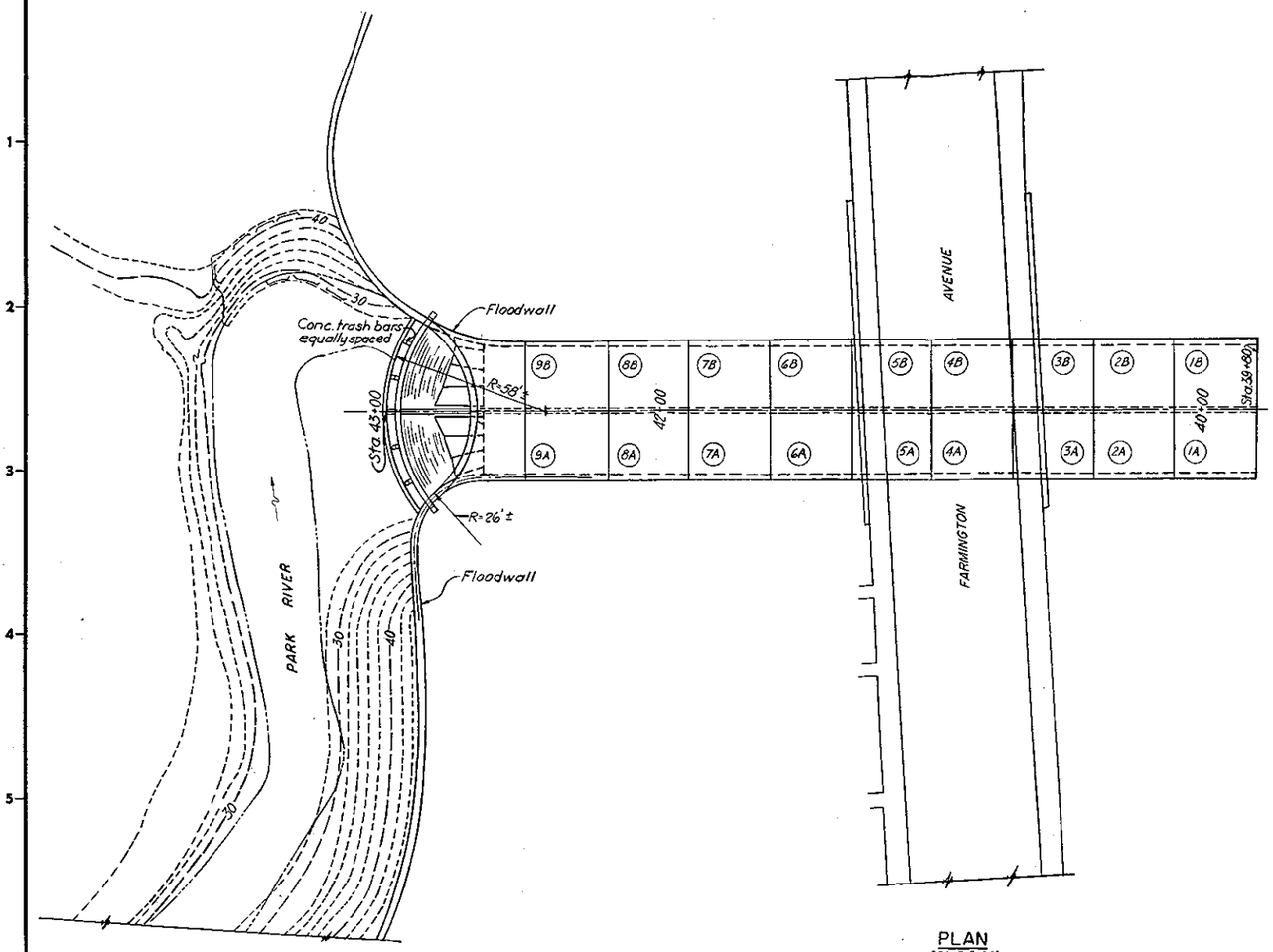
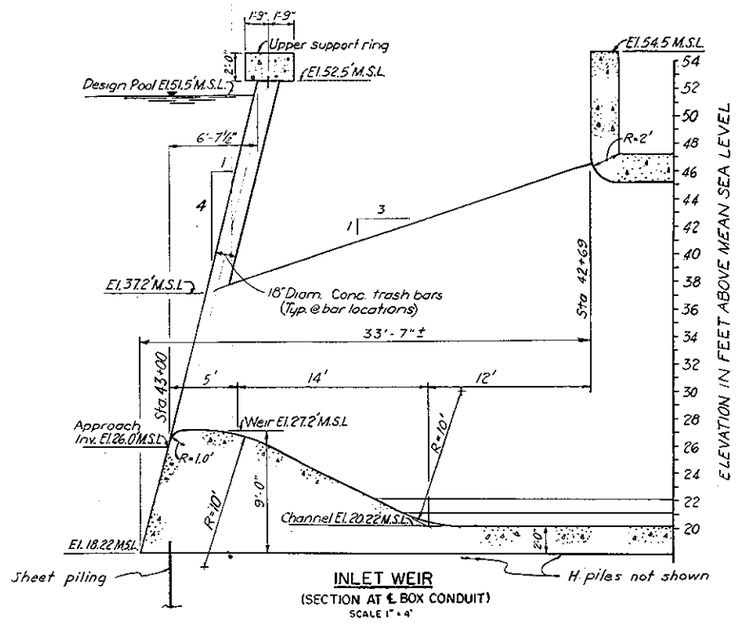
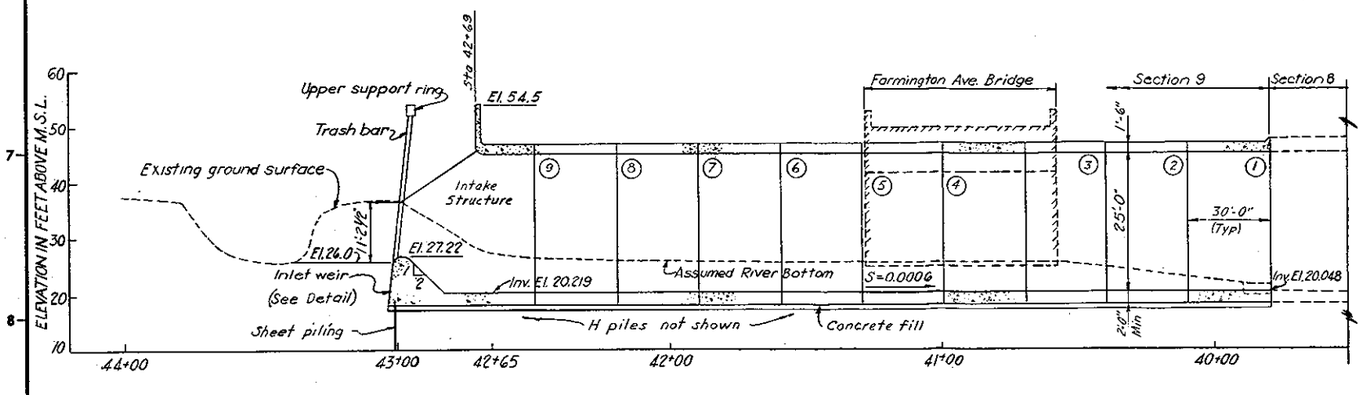


PLAN
SCALE 1"=20'



INLET WEIR
(SECTION AT ℄ BOX CONDUIT)
SCALE 1"=4'



SECTIONAL PROFILE - SECTION 9
SCALE HORZ. 1"=20'
VERT. 1"=10'

GRAPHIC SCALES

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NEW ENGLAND DIVISION
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DES. BY W.R. | DR. BY R.D.P. | CK. BY

WATER RESOURCES DEVELOPMENT PROJECT
PARK RIVER LOCAL PROTECTION
NORTH BRANCH CONDUIT - SECTION 9
PLAN, PROFILE AND SECTION

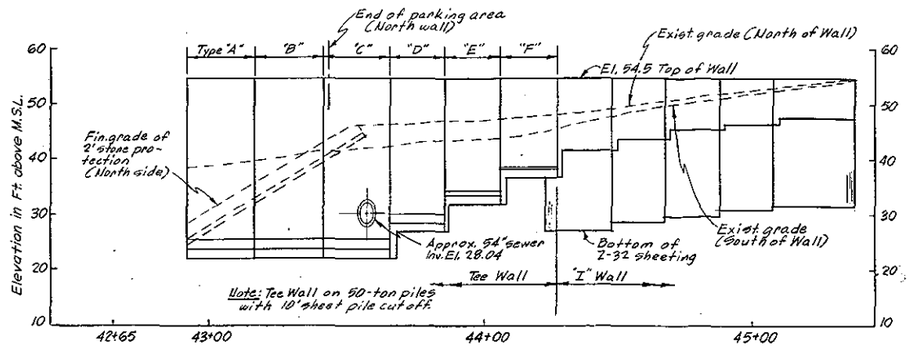
HARTFORD CONNECTICUT

SUBMITTED: CHIEF, SECTION
APPROVAL RECOMMENDED: CHIEF, TECH. ENG. BRANCH
REVIEWED: PROJECT ENGINEER
APPROVAL RECOMMENDED: CHIEF BRANCH
APPROVED: CHIEF, ENGINEERING DIVISION
DATE

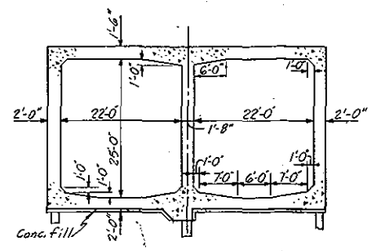
SCALE AS SHOWN | SPEC. NO.
DRAWING NUMBER
SHEET

PLATE 7A-2

CORPS OF ENGINEERS — U. S. ARMY



PROFILE EAST FLOOD WALL
SCALE HORZ. 1" = 20'
VERT. 1" = 10'



TYPICAL CONDUIT SECTION
SECTION 9
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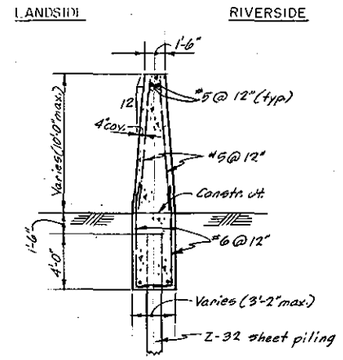

TYPICAL "I" WALL SECTION
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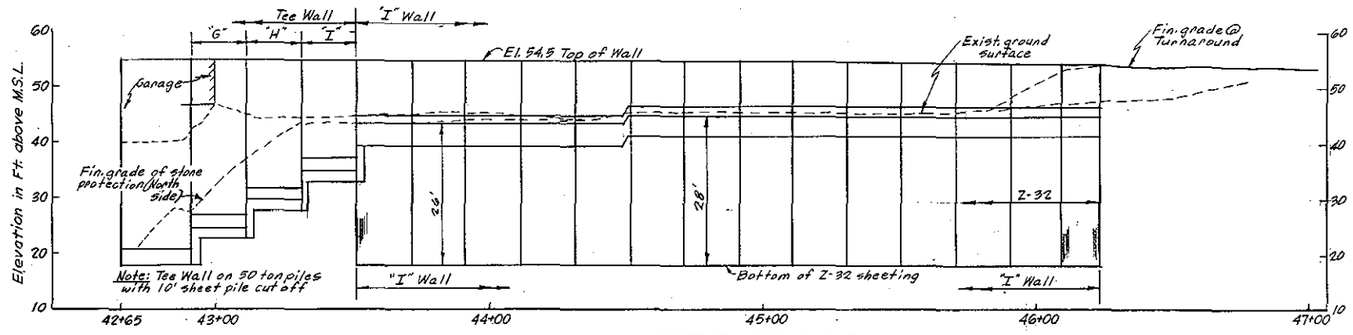

PROFILE WEST FLOOD WALL
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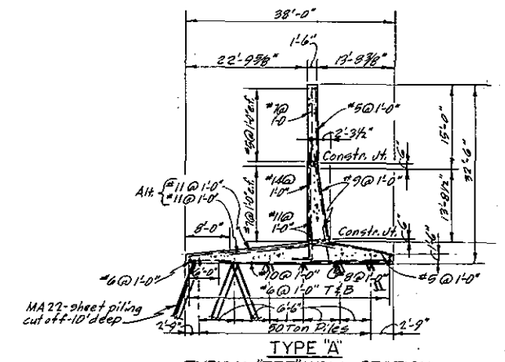
VERT. 1" = 10'



TYPE "A"
TYPICAL "TEE" WALL SECTION
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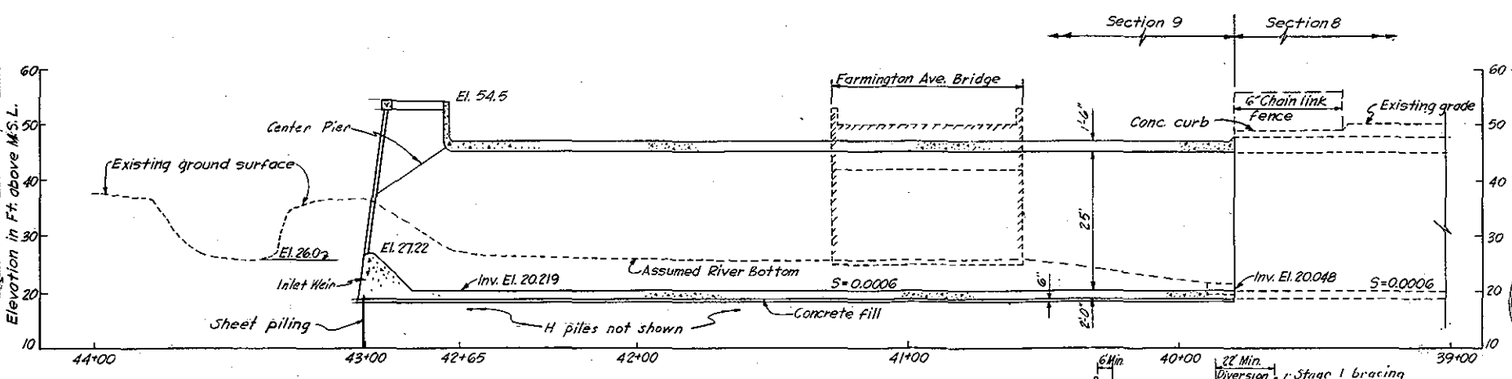
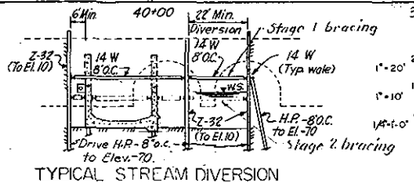


SECTIONAL PROFILE - SECTION 9
SCALE HORZ. 1" = 20'
VERT. 1" = 10'



TYPICAL STREAM DIVERSION
(SECTION 9 at Sta. 40+80)
SCALE 1" = 20'

GRAPHIC SCALES

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DES. BY — DR. BY J.A.P. — CK. BY

SUBMITTED: CHIEF, SECTION
APPROVAL RECOMMENDED: CHIEF, TECH. ENG. BRANCH
REVIEWED: PROJECT ENGINEER
APPROVAL RECOMMENDED: CHIEF, BRANCH

WATER RESOURCES DEVELOPMENT PROJECT
PARK RIVER LOCAL PROTECTION
BOX CONDUIT AT SECTION 9
PROFILES AND SECTIONS

HARTFORD — CONNECTICUT

APPROVED — DATE
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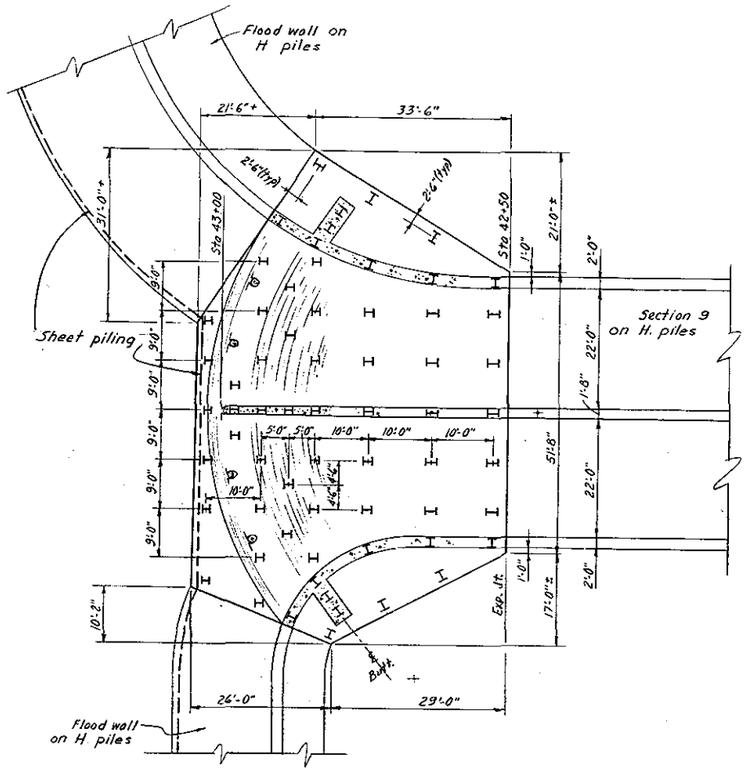
SCALE — SPEC. NO. — DRAWING NUMBER — SHEET

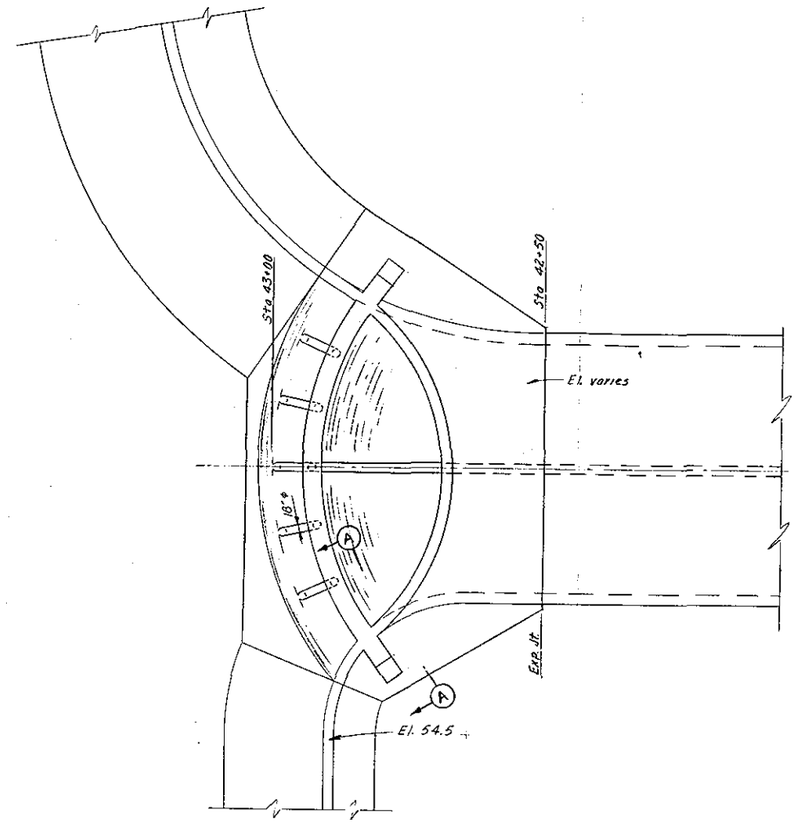
PLATE 7A-3

CORPS OF ENGINEERS U. S. ARMY



FOUNDATION & PILE PLAN

SCALE: 1"=10'



ROOF PLAN

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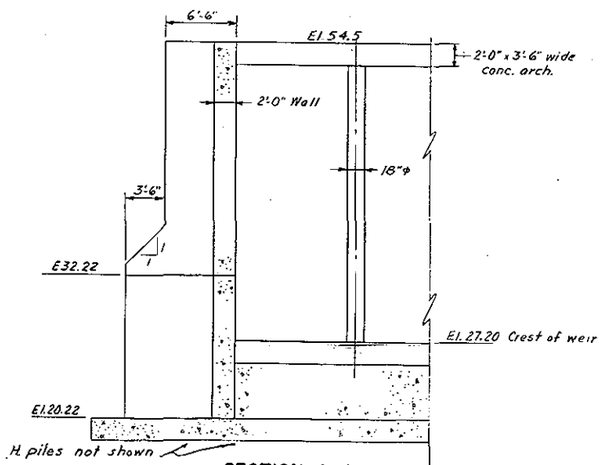

SECTION A-A

SCALE: 1"=5'

GRAPHIC SCALES

1"=5' 5' 0 5' 10'

1"=10' 10' 0 10' 20'

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DES. BY W. R. | DWL. BY R.D.P. | CK. BY

SUBMITTED:

CHIEF, SECTION

APPROVAL RECOMMENDED:

CHIEF, TECH. ENG. BRANCH

REVIEWED:

PROJECT ENGINEER

APPROVAL RECOMMENDED:

CHIEF BRANCH

WATER RESOURCES DEVELOPMENT PROJECT

PARK RIVER LOCAL PROTECTION

INLET STRUCTURE-SECTION 9

PLANS AND SECTION

HARTFORD CONNECTICUT

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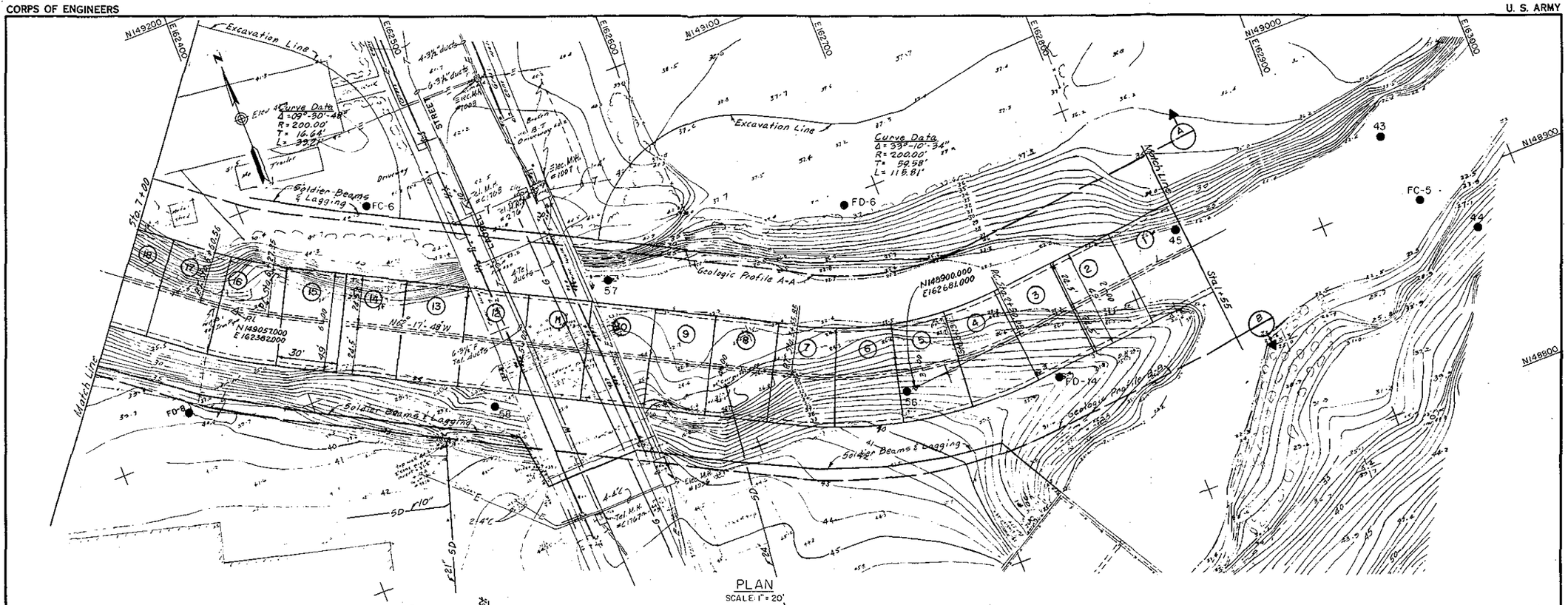
PLATE 7A-4



PLAN
SCALE: 1" = 20'

SECTIONAL PROFILE - SECTION 7
SCALE: HORIZ. 1" = 20'
VERT. 1" = 10'

GRAPHIC SCALES
1" = 20' 20' 0 20' 40'
1" = 10' 10' 0 10' 20'

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CHIEF, BRANCH
REVIEWED
APPROVAL RECOMMENDED

WATER RESOURCES DEVELOPMENT PROJECT
PARK RIVER LOCAL PROTECTION
BOX CONDUIT AT SECTION 7
PLAN AND PROFILE NO. 1

HARTFORD CONNECTICUT
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SHEET

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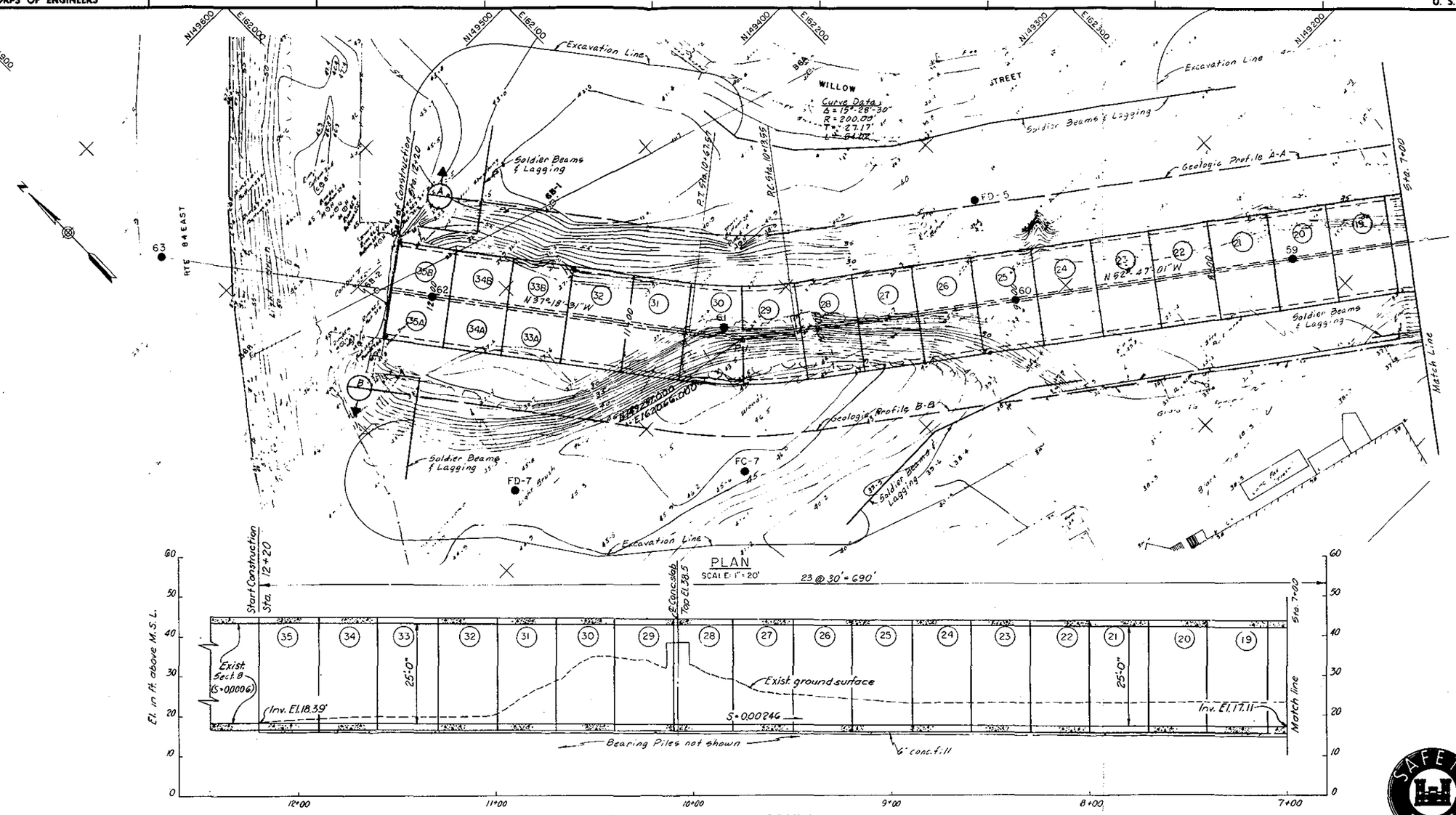

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APPROVAL RECOMMENDED:
CHIEF, TECH. ENG. BRANCH
REVIEWED:
APPROVAL RECOMMENDED:

WATER RESOURCES DEVELOPMENT PROJECT
PARK RIVER LOCAL PROTECTION
BOX CONDUIT AT SECTION 7
PLAN AND PROFILE NO. 2

HARTFORD CONNECTICUT
APPROVED DATE
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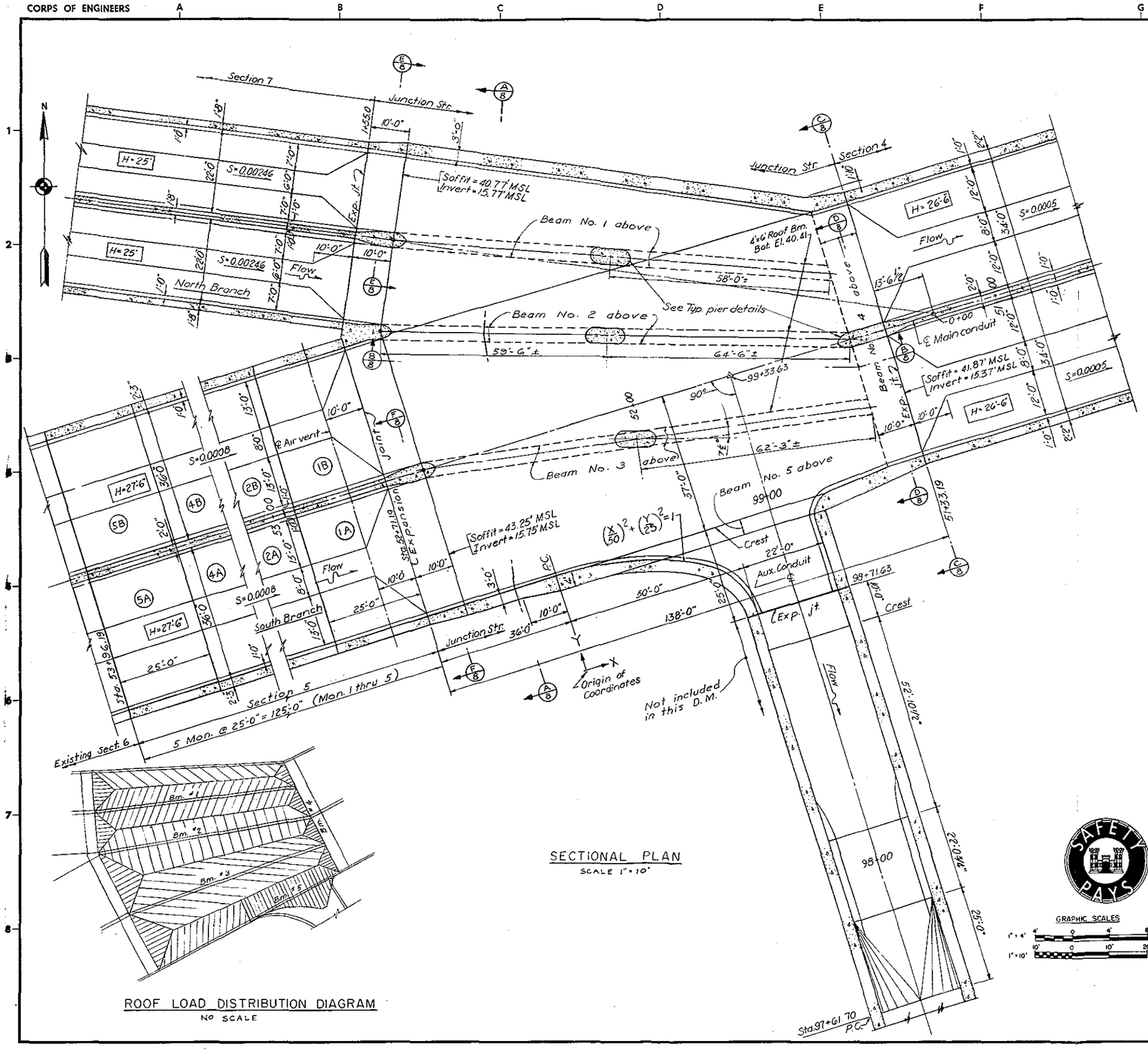
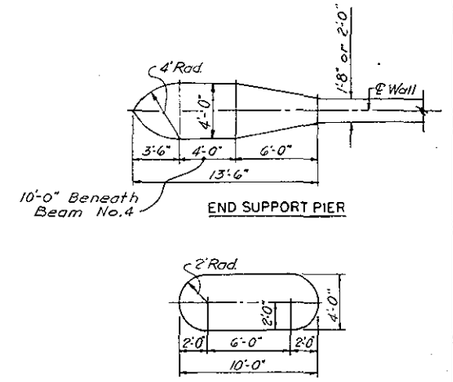
PLATE 7A-6

CORPS OF ENGINEERS U. S. ARMY



SECTIONAL PLAN

SCALE 1"=10'



TYPICAL PIER DETAILS

SCALE 1"=4'-0"

ROOF LOAD DISTRIBUTION DIAGRAM

NO SCALE

GRAPHIC SCALES

REVISION	DATE	DESCRIPTION	BY

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

DES. BY | DR. BY | CK. BY

SUBMITTED:
CHIEF, SECTION
APPROVAL RECOMMENDED:
CHIEF, TECH. ENG. BRANCH
REVIEWED:
PROJECT ENGINEER
APPROVAL RECOMMENDED:
CHIEF BRANCH

WATER RESOURCES DEVELOPMENT PROJECT
PARK RIVER LOCAL PROTECTION
PLAN OF JUNCTION STRUCTURE

HARTFORD CONNECTICUT

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SCALE | SPEC. NO.
DRAWING NUMBER
SHEET

PLATE 7A-7

CORPS OF ENGINEERS — U. S. ARMY

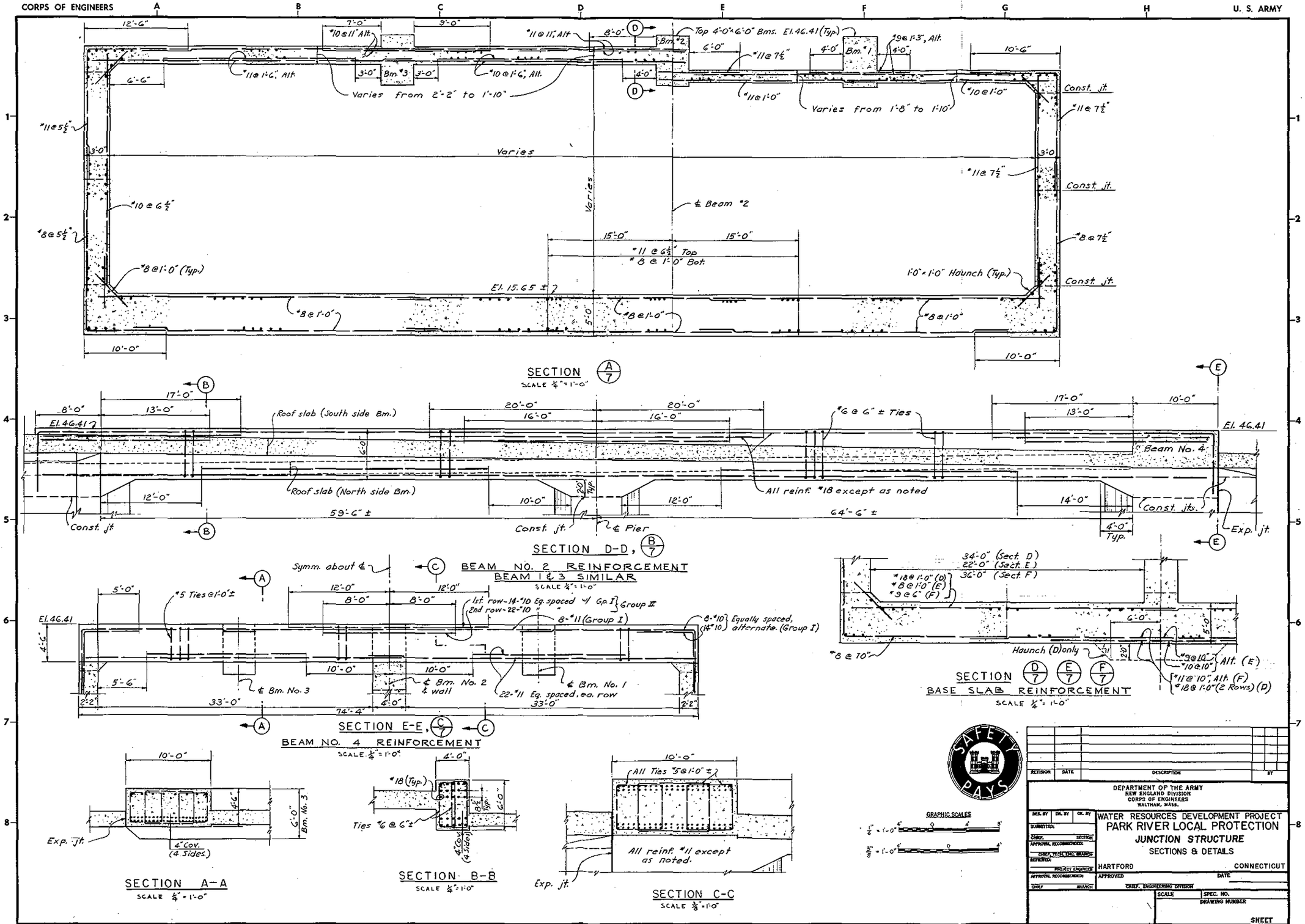
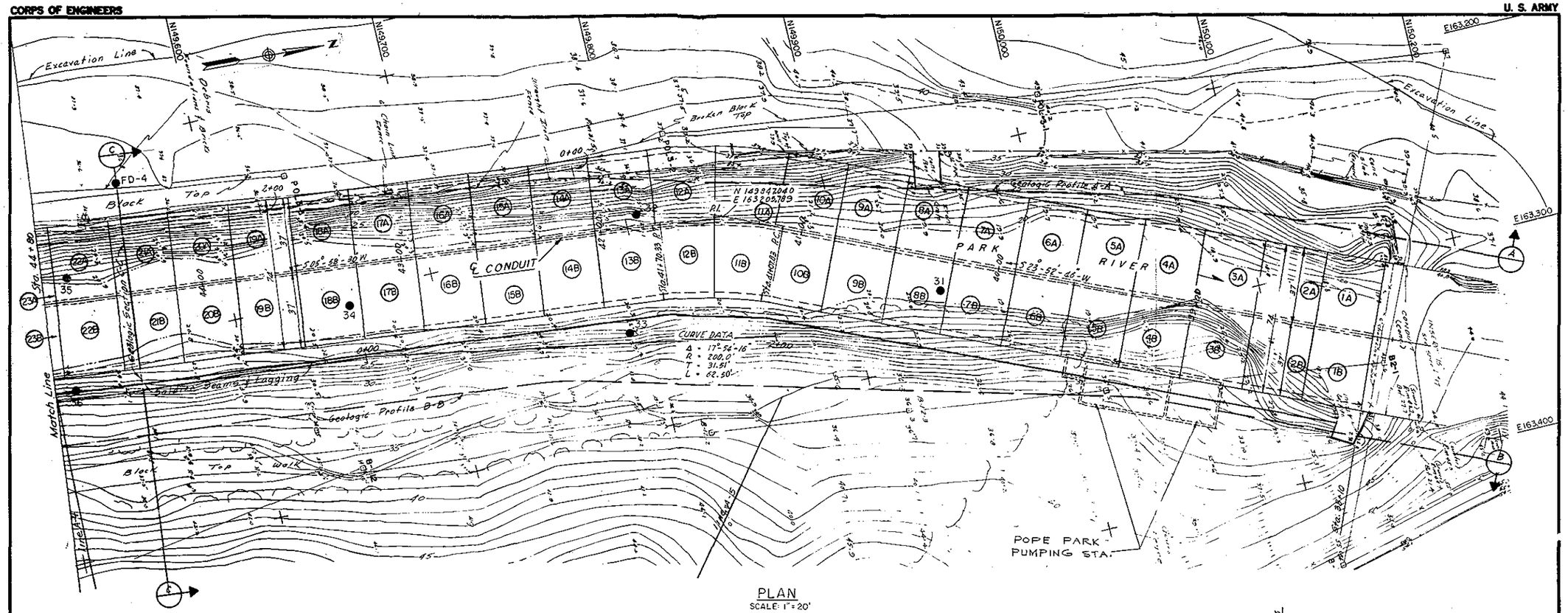


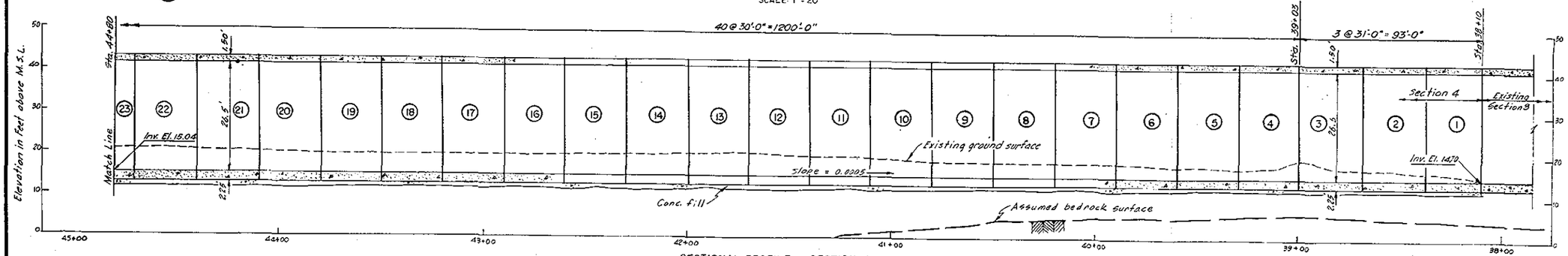
PLATE 7A-8

CORPS OF ENGINEERS

U. S. ARMY



PLAN
SCALE: 1" = 20'



SECTIONAL PROFILE — SECTION 4
SCALE: HOR. 1" = 20'
VERT. 1" = 10'

GRAPHIC SCALES
1" = 20' 20' 0 20' 40'
1" = 10' 10' 0 10' 20'

SAFETY PAYS

REVISION	DATE	DESCRIPTION	BY

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

DES. BY | DR. BY | CK. BY

SUBMITTED:
CHIEF, SECTION
APPROVAL RECOMMENDED:
CHIEF, BRANCH
REVIEWED:
PROJECT ENGINEER
APPROVAL RECOMMENDED:
CHIEF, PROJECT BRANCH

WATER RESOURCES DEVELOPMENT PROJECT
PARK RIVER LOCAL PROTECTION
BOX CONDUIT AT SECTION 4
PLAN AND PROFILE NO. 1

HARTFORD CONNECTICUT

APPROVED DATE
CHIEF, ENGINEERING DIVISION

SCALE | SPEC. NO.
DRAWING NUMBER
SHEET

PLATE 7A-9

CORPS OF ENGINEERS

U. S. ARMY

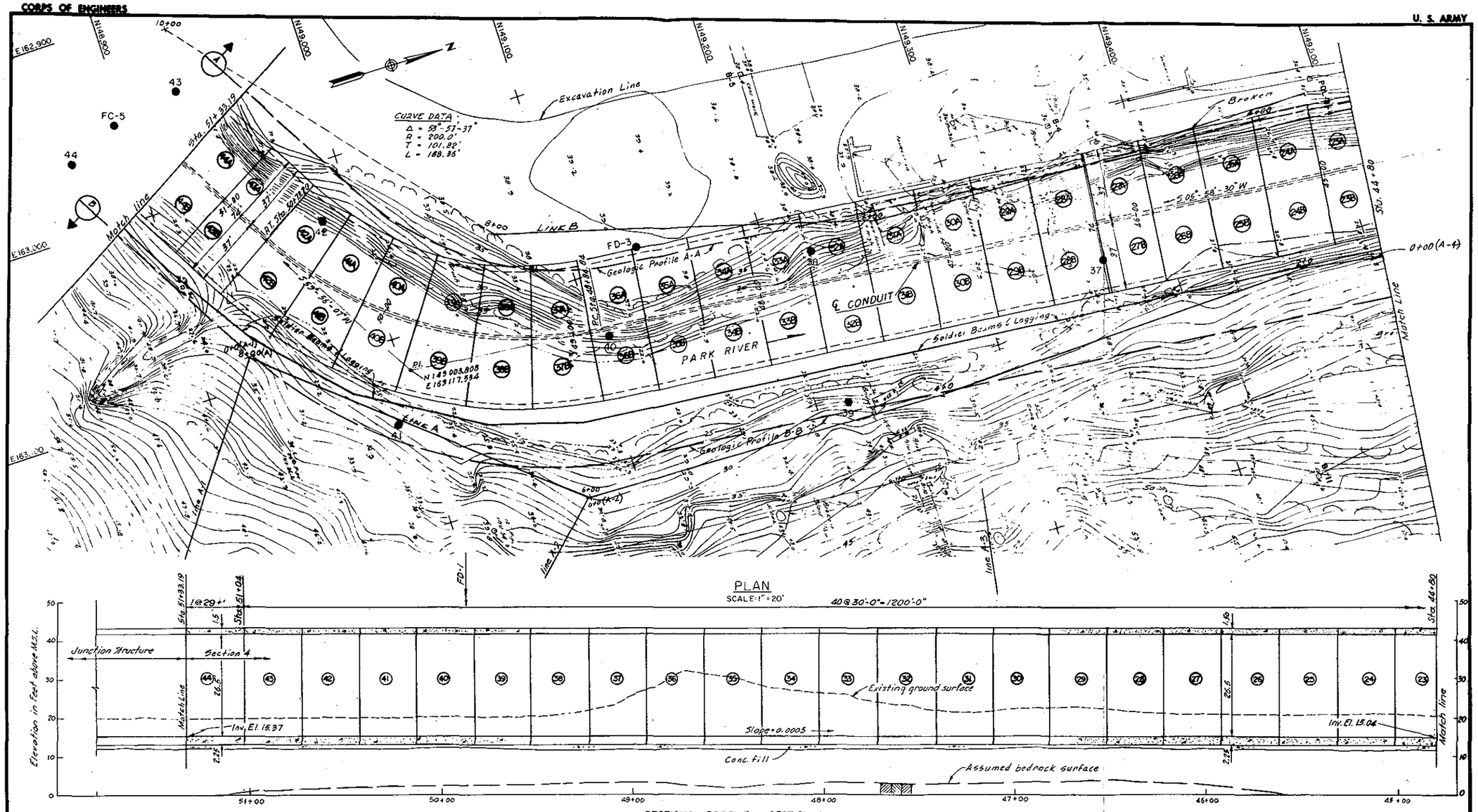
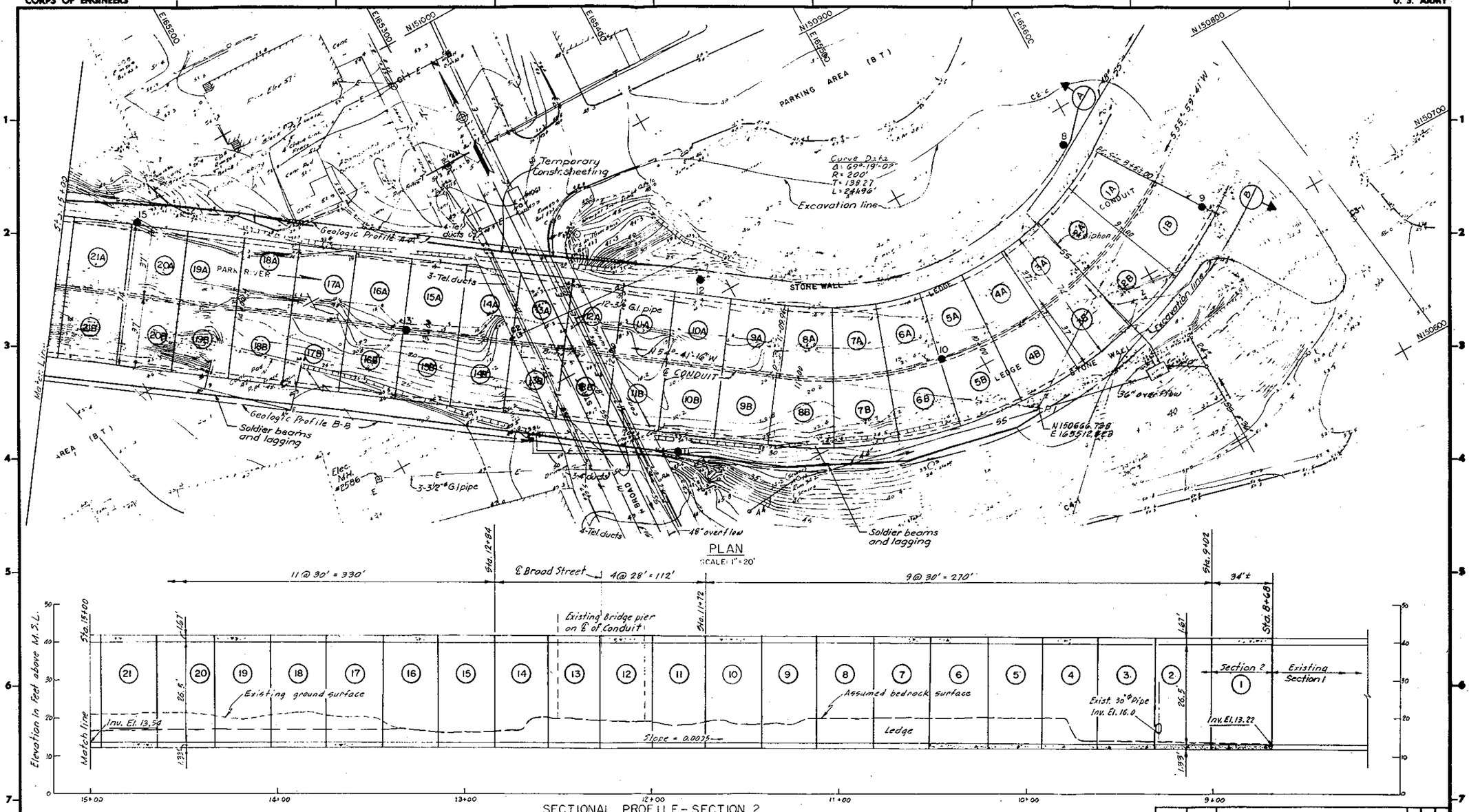


PLATE 7A-10

CORPS OF ENGINEERS U. S. ARMY



DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

WATER RESOURCES DEVELOPMENT PROJECT

PARK RIVER LOCAL PROTECTION

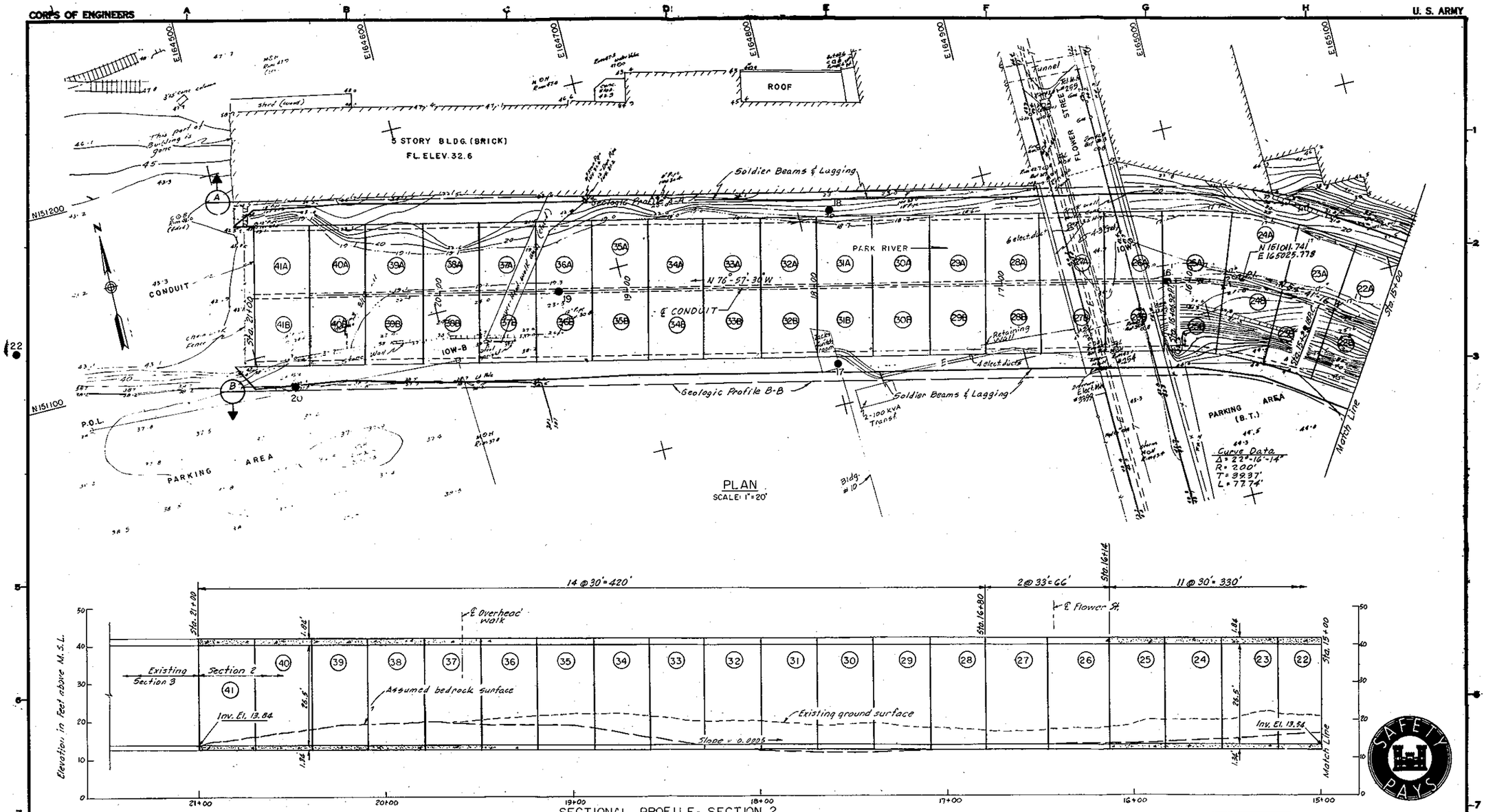
BOX CONDUIT AT SECTION 2

PLAN AND PROFILE NO. 1

HARTFORD CONNECTICUT

PLATE 7A-11

CORPS OF ENGINEERS — U. S. ARMY



REVISION	DATE	DESCRIPTION	BY

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

DES. BY | DR. BY | CK. BY

SUBMITTED: CHIEF, SECTION
APPROVAL RECOMMENDED: CHIEF, TECH. ENG. BRANCH
REVIEWED: PROJECT ENGINEER
APPROVAL RECOMMENDED:

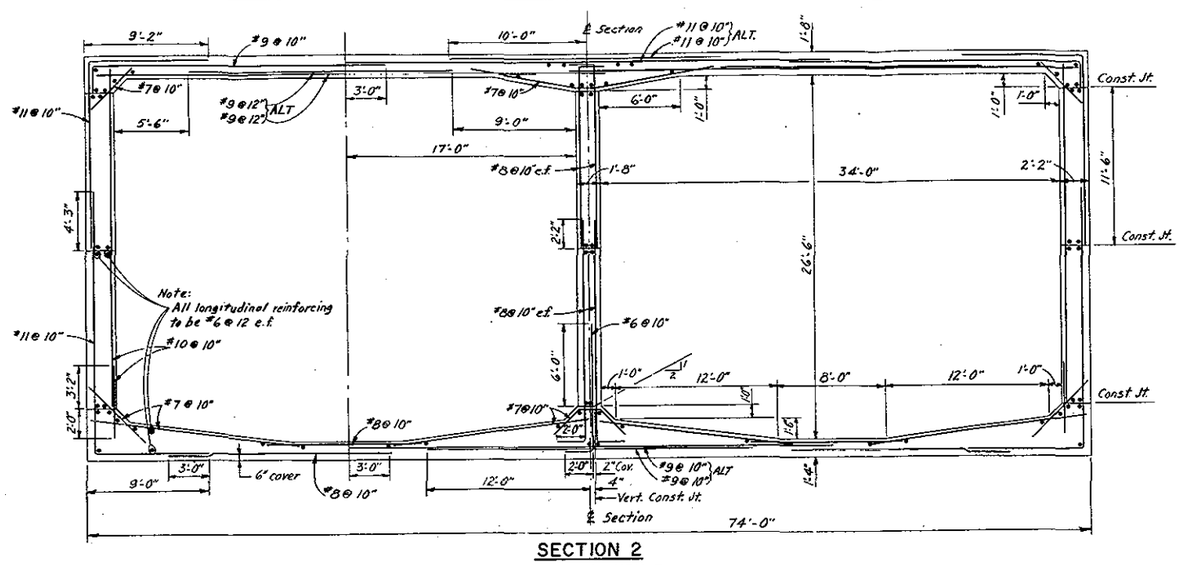
WATER RESOURCES DEVELOPMENT PROJECT
PARK RIVER LOCAL PROTECTION
BOX CONDUIT AT SECTION 2
PLAN AND PROFILE NO. 2

HARTFORD — CONNECTICUT
APPROVED — DATE
CHIEF, ENGINEERING DIVISION

SCALE | SPEC. NO. | DRAWING NUMBER | SHEET

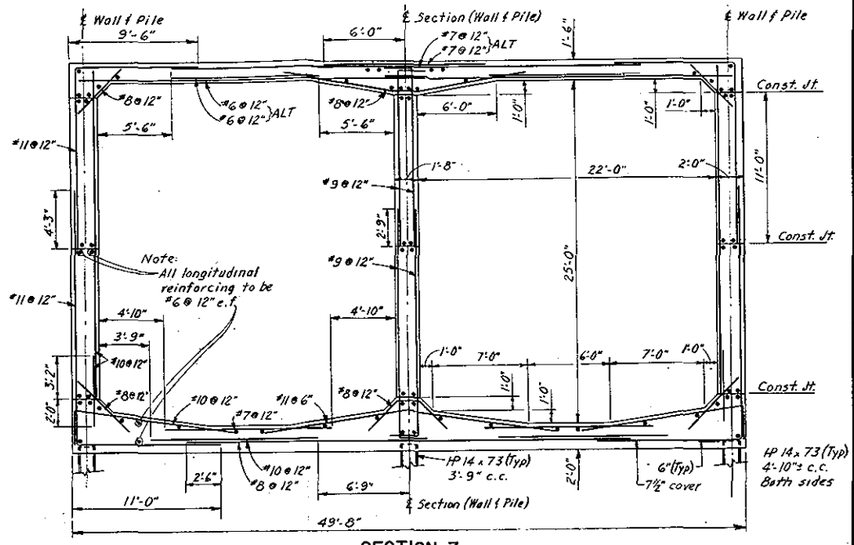
PLATE 7A-12

CORPS OF ENGINEERS — U. S. ARMY



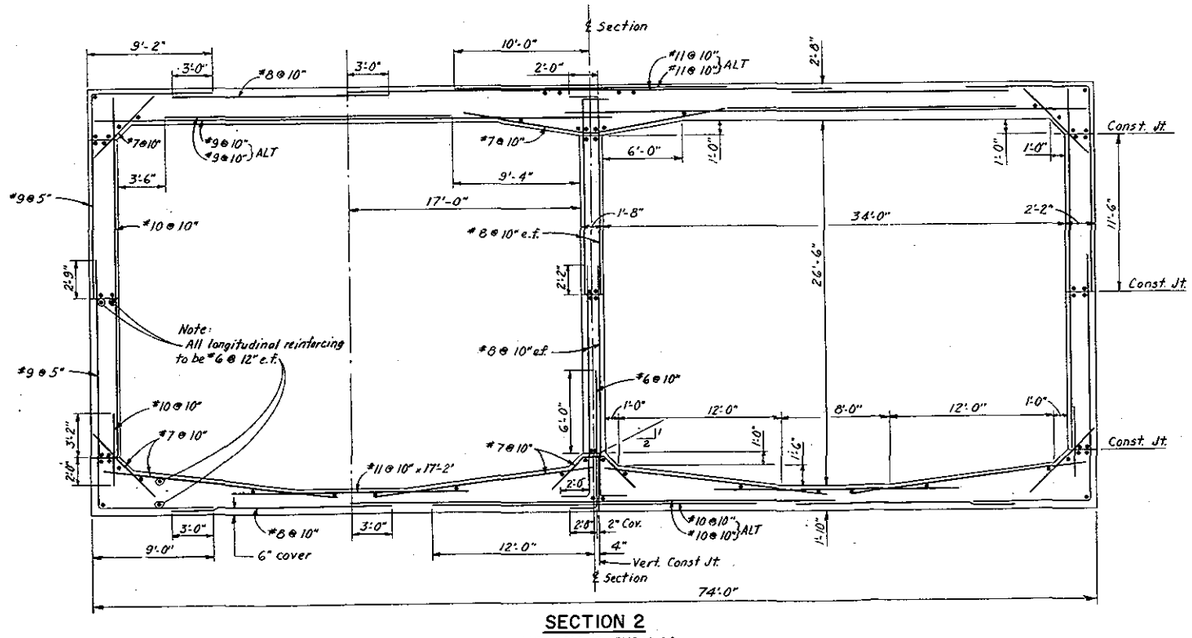
SECTION 2

MON. NO. 1 THRU 5
MON. NO. 15 THRU 41



SECTION 7

MON. NO. 1 THRU 32
MON. NO. 33 THRU 35
(SIMILAR TO SECTION 9)



SECTION 2

MON. NO. 6 THRU 14

GENERAL NOTES:

1. Box conduit sections shown looking downstream except as noted.
2. Minimum cover to steel reinforcement shall be 4" except for bottom of base slabs which shall be 6" on soil and 7½" on piles unless noted otherwise.
3. New box conduit sections are to be poured with new expansion joints on existing sections with existing waterstops in all walls and slabs except center wall.
4. Provide expansion joints at every other monolith and contraction joints in balance.
5. Provide Type "Y" waterstops at all expansion and contraction joints in all walls and slabs except center wall.

GRAPHIC SCALES

1/8" = 1'-0" 4' 0 4' 8'

REVISION	DATE	DESCRIPTION	BY

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

DES. BY W.R. | DR. BY R.D.P. | CK. BY

SUBMITTED: CHIEF, SECTION

APPROVAL RECOMMENDED: CHIEF, TECH. ENG. BRANCH

REVIEWED: PROJECT ENGINEER

APPROVAL RECOMMENDED: CHIEF BRANCH

WATER RESOURCES DEVELOPMENT PROJECT

PARK RIVER LOCAL PROTECTION

BOX CONDUITS AT SECTIONS 2 & 7 SECTIONS

HARTFORD CONNECTICUT

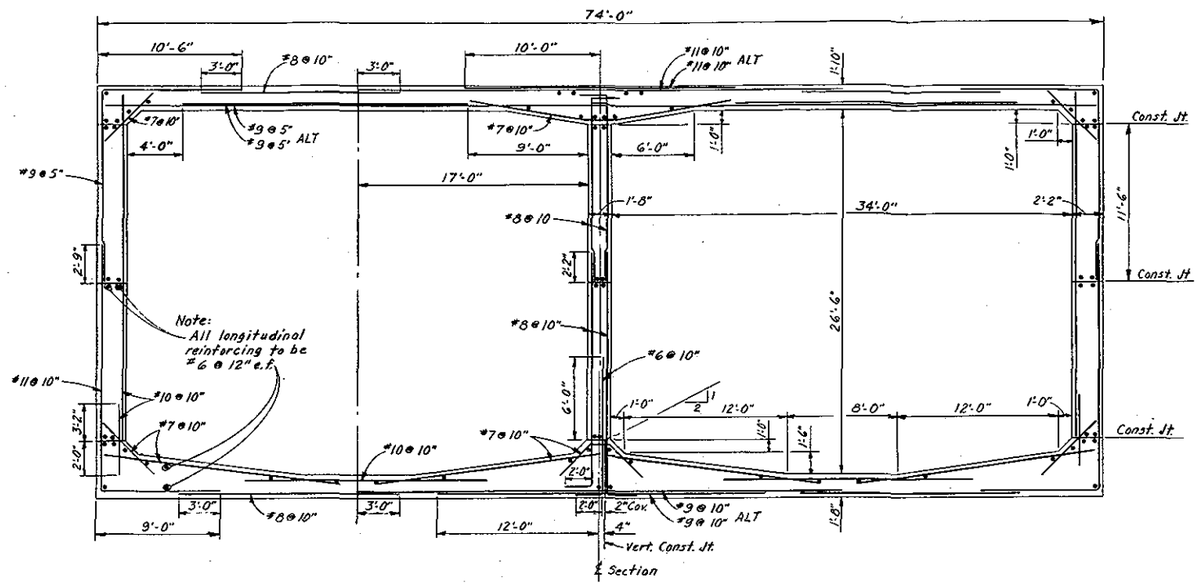
APPROVED DATE

CHIEF, ENGINEERING DIVISION

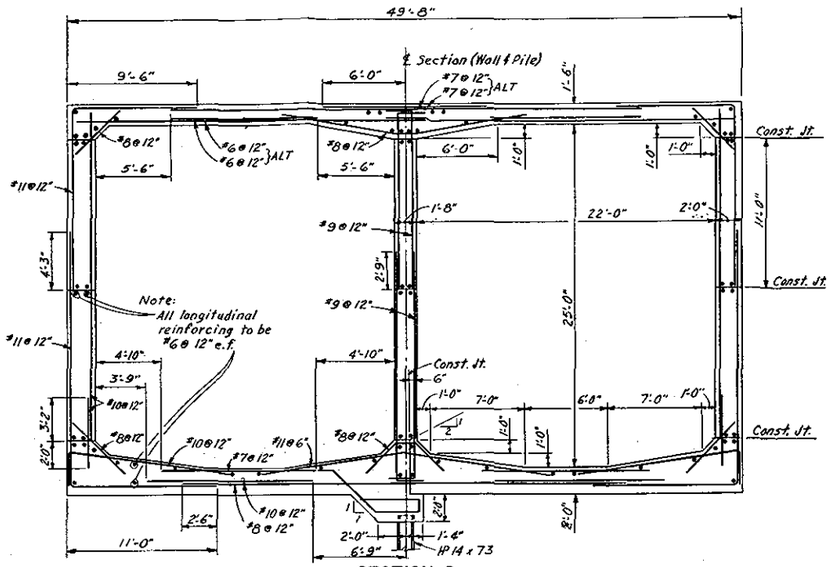
SCALE | SPEC. NO. | DRAWING NUMBER | SHEET

PLATE 7A-13

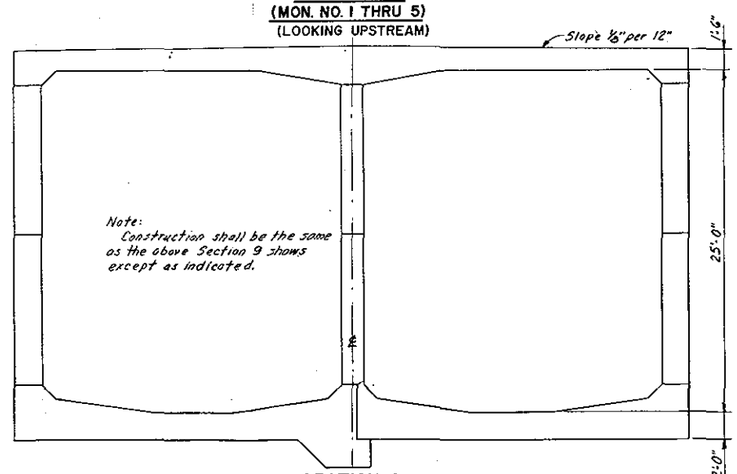
CORPS OF ENGINEERS — U. S. ARMY



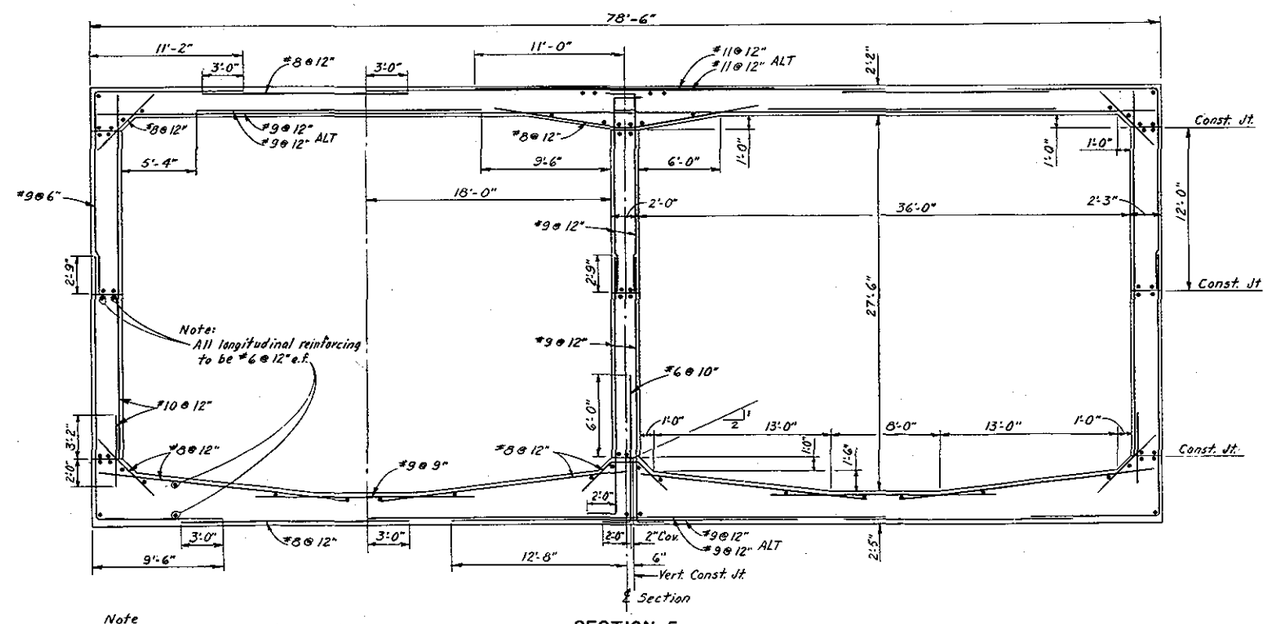
SECTION 4
(MON. NO. 1 THRU 44)



SECTION 9
(MON. NO. 1 THRU 5)
(LOOKING UPSTREAM)



SECTION 9
(MON. NO. 6 THRU 9)
(LOOKING UPSTREAM)



SECTION 5
(MON. NO. 1 THRU 5)

Note
All dimensions & reinforcement sizes on all sections are symmetrical about ℄'s, except at bottom center of all sections construction shall be one way as indicated.

GRAPHIC SCALES
1/4" = 1'-0" 4' 0 4' 8'

REVISION	DATE	DESCRIPTION	BY

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

DES. BY W.R. | DR. BY R.D.P. | CK. BY

SUBMITTED:
CHIEF, SECTION
APPROVAL RECOMMENDED:
CHIEF, TECH. ENG. BRANCH
REVIEWED:
PROJECT ENGINEER
APPROVAL RECOMMENDED:
CHIEF BRANCH

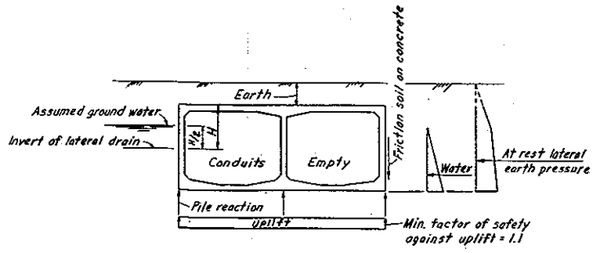
WATER RESOURCES DEVELOPMENT PROJECT
PARK RIVER LOCAL PROTECTION
BOX CONDUITS AT SECTIONS 4,5 & 9
SECTIONS
HARTFORD CONNECTICUT

APPROVED DATE
CHIEF, ENGINEERING DIVISION

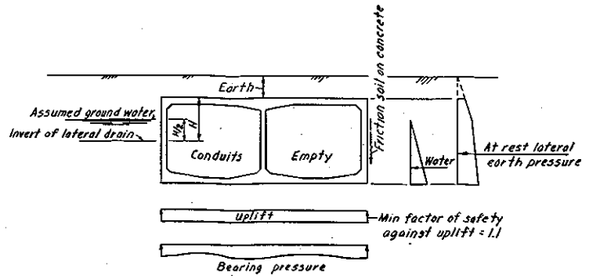
SCALE 1/4" = 1'-0" | SPEC. NO.
DRAWING NUMBER
SHEET

PLATE 7A-14

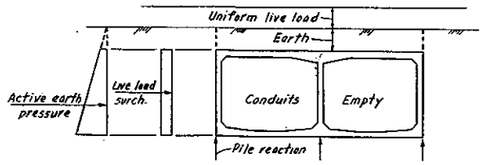
CORPS OF ENGINEERS — U. S. ARMY



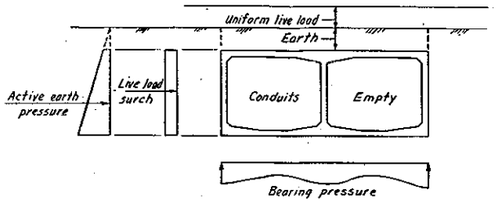
GENERAL LOADING CASE-STABILITY



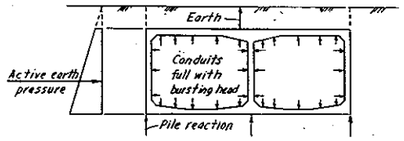
GENERAL LOADING CASE-STABILITY



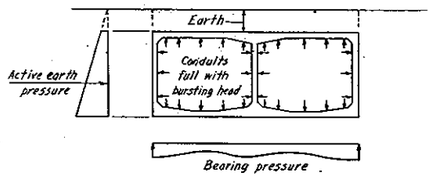
GENERAL LOADING CASE 2
(CONDUITS FOUNDED ON H PILES)



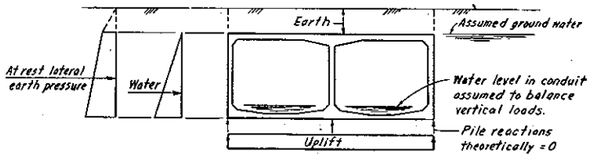
GENERAL LOADING CASE 2
(CONDUITS FOUNDED ON TILL OR ROCK)



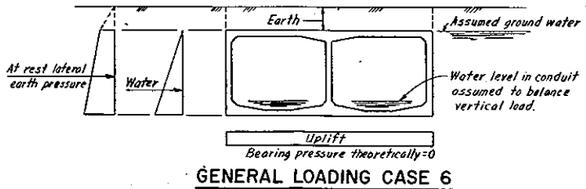
GENERAL LOADING CASE 3
(CONDUITS FOUNDED ON H PILES)



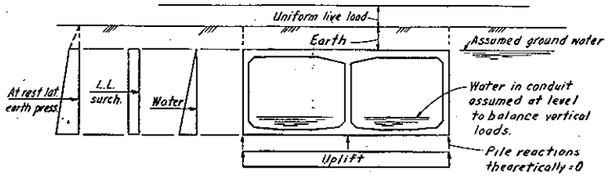
GENERAL LOADING CASE 3
(CONDUITS FOUNDED ON TILL OR ROCK)



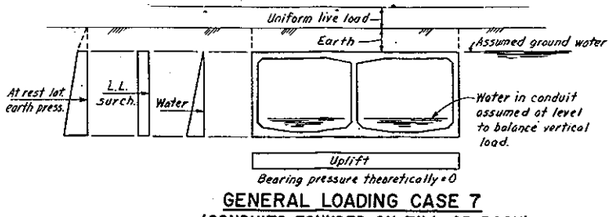
GENERAL LOADING CASE 6
(CONDUITS FOUNDED ON H PILES)



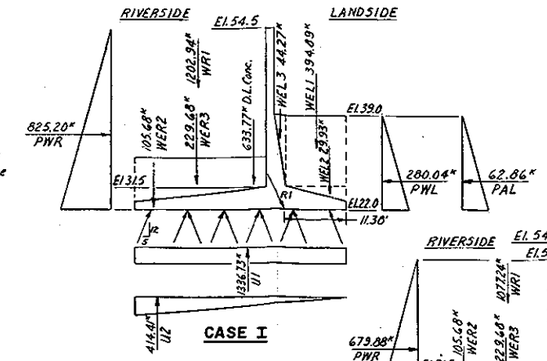
GENERAL LOADING CASE 6
(CONDUITS FOUNDED ON TILL OR ROCK)



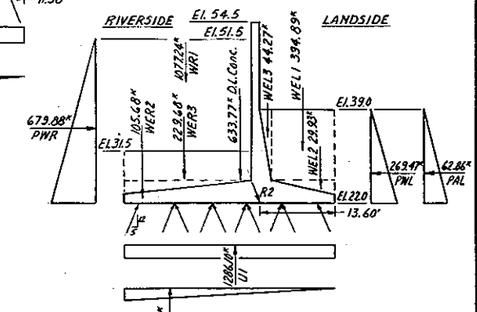
GENERAL LOADING CASE 7
(CONDUITS FOUNDED ON H PILES)



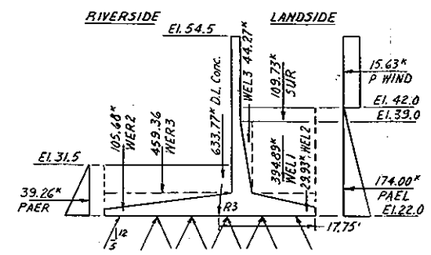
GENERAL LOADING CASE 7
(CONDUITS FOUNDED ON TILL OR ROCK)



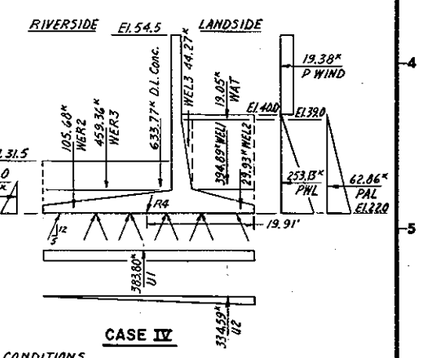
CASE I



CASE II

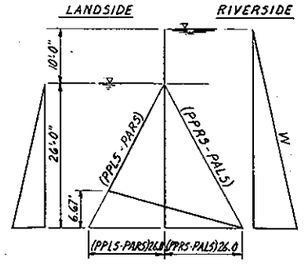


CASE III



CASE IV

Note:
Cases I, II, III & IV are for Type "A" Tee Wall - Section 9.
Loading Diagram is for Typical "I" Wall - Section 9



LOADING DIAGRAM

GRAPHIC SCALES

LOADING CONDITIONS

CASE I
Water to top of wall, uplift varying from full head at riverside heel to 0 at landside ground surface by creep method. Resultant to fall in middle half. Normal design stresses.

CASE II
Same as Case I except water down 3'-0" and resultant to fall in the middle third.

CASE III
Water down 50%/10' wind load applied to landside and 3'-0" surcharge applied on landside.

CASE IV
Water at elevation 27.0 on riverside, at elevation 40.0 on landside, 50 psf wind on landside.

REVISION	DATE	DESCRIPTION	BY

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

DES. BY R.H. | DR. BY R.D.P. | CK. BY

SUBMITTED:
CHIEF, SECTION
APPROVAL RECOMMENDED:
CHIEF, TECH. ENG. BRANCH
REVIEWED:
PROJECT ENGINEER
APPROVAL RECOMMENDED:
CHIEF BRANCH

WATER RESOURCES DEVELOPMENT PROJECT

PARK RIVER LOCAL PROTECTION

LOADING DIAGRAMS

HARTFORD — CONNECTICUT

APPROVED — DATE
CHIEF, ENGINEERING DIVISION

SCALE NONE | SPEC. NO.
DRAWING NUMBER
SHEET

PLATE 7A-15

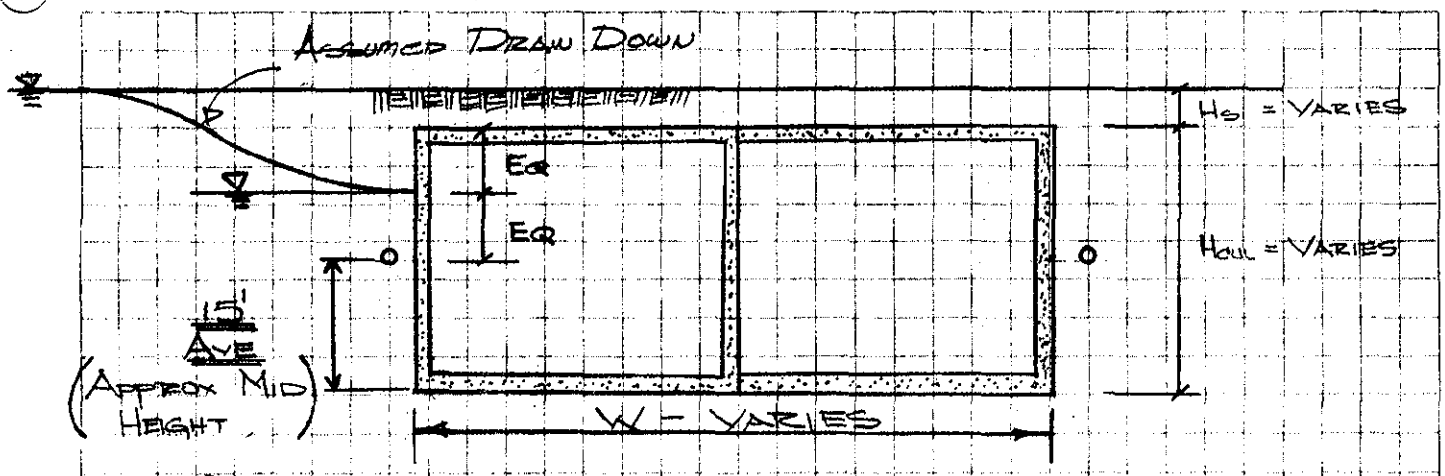
APPENDIX

TYPICAL DESIGN COMPUTATIONS

INDEX TO SHEETS

TITLE		PAGE NO.
TWIN CONDUITS	- STABILITY	DM7-1 TO DM7-7
	- DESIGN SECTION 5	DM7-8 TO DM7-28
	- DESIGN SECTION 7	DM7-29 TO DM7-43
INLET STRUCTURE		DM7-44 TO DM7-59
JUNCTION STRUCTURE	- STABILITY	DM7-60
	- ROOF BEAM NO. 2	DM7-61 TO DM7-73
	- FRAME @ ENT. TO SEC. 4	DM7-74 TO DM7-89
	- FRAME @ SEC. (A/8)	DM7-90 TO DM7-112
	- FRAME @ EXIT FROM SEC. 5&7	DM7-113 TO DM7-120
HEADWALLS	- CONCRETE "T" TYPE, STABILITY	DM7-121 TO DM7-132
	- CONCRETE "T" TYPE, PILES	DM7-133 TO DM7-138
	- "I" TYPE SHEET PILE	DM7-139 TO DM7-142

SUBJECT: Park River Local Protection
COMPUTATION: Uplift - Factor of Safety, Geo Requirement
COMPUTED BY: W. Rodger CHECKED BY: DATE: Aug 1974



Assumptions for Uplift:

1. Max Water Level Half Way Between Drain and Top of Culvert
2. Factor of Safety (Required Minimums)
   A. With Zero Flow In Culvert F.S. = 1.10
3. Normal Weights of Material

   Concrete 150 pcf
   Sat. Soil 120 pcf
4. Resisting Forces To Include DL Box, DL Soil & Side Friction

   $F = N \cdot \mu$ (Where μ = .50 b/w Earth & Concrete)

   N = Normal Force - Based on At Rest Soil Pressures of Sat. Earth or Buoyant Earth Below Water Table
5. Check Sections 2, 4, 5, 7, & 9

SUBJECT: PARK RIVER LOCAL PROTECTION
COMPUTATION: SECTION #2-TYPICAL SECTION
COMPUTED BY: W. Rodger CHECKED BY: DATE: 19 Aug 74

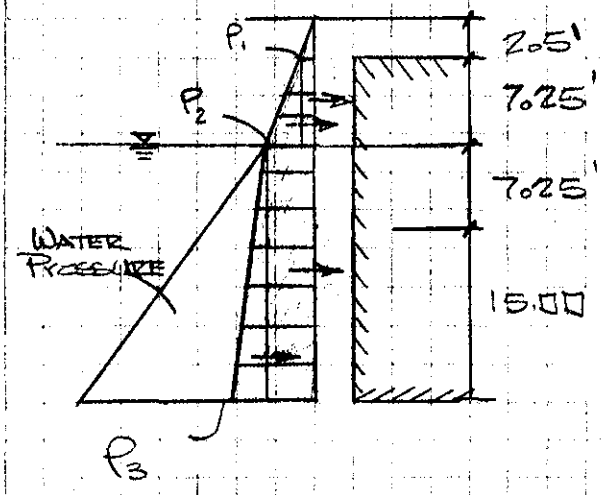
CHECK UPLIFT SECTION #2 (REFER P 2-2)

(1-A) $H_{CUL} = 29.50'$ $W = 74.0$

$H_{GW} = 29.50 - 15.0 = 14.50/2 = 7.25'$

$= 15.00 + 7.25' = 22.25'$

$U_P = .0625^{kcf} \times 22.25' \times 74' = 102.91^k \uparrow$



$K_o = .50$ $\gamma_{SAT} K_o = .06000$
$\gamma_{BUOY} K_o = .02875$

$P_1 = 2.5(.06) = .150$
$P_2 = 9.75(.06) = .585$
$P_3 = .585 + 22.25(.02875)$
$.585 + .640$
$= 1.225$

$P_U = 7.25'(.150) = 1.088$
$= 1/2 \cdot (.435)(7.25) = 1.577$
$= 22.25(.585) = 13.016$
$= 1/2 \cdot (.640)(22.25 = 7.120$
22.801

$F_T = 2 \times 22.801 \times .5 = 22.801^k$

$W_{DL\ CONC} = 64.8^k$ (Pg) $= 64.8^k$
$W_{DL\ SOIL} = .120(2.5)(74) = 22.2^k$

$\% F.S. = \dfrac{64.8 + 22.2 + 22.801}{102.91} = \dfrac{109.801}{102.91} = 1.067 \approx 1.11$

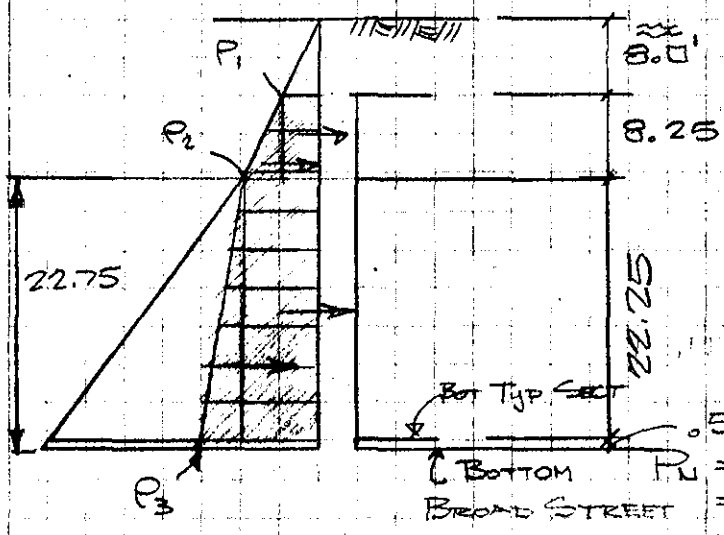
SUBJECT: Park River Local Protection
COMPUTATION: Section #2 - Broad Street
COMPUTED BY: W. Rodger CHECKED BY: DATE: 19 Aug 74

2-A

$H_{OUL} = 31.0'$ $W = 74'$

$H_{GW} = 22.25' + .50 = 22.75$

$\therefore W = 22.75(.0625) = \underline{105.22^k} \uparrow$



$P_1 = 8(.06) = .480$

$P_2 = 1625(.06) = .975$

$P_3 = .975 + 22.75(.02975)$
$= .975 + .654$
$= 1.629^k$

P_1	$= 8.25(.480)$	$= 3.960$
	$= \frac{1}{2}(.495)(8.25)$	$= 2.042$
	$= 22.75(.975)$	$= 22.181$
	$= \frac{1}{2}(.654)(22.75)$	$= 7.440$
		35.623

$F_T = 2 \times 35.623 \times .50 = 35.623$

W_{DL} Conc $= 81.45^k$ (Pg) $= 81.45$
W_{DL} Soil $= .12 \times 8 \times 74$ 71.04

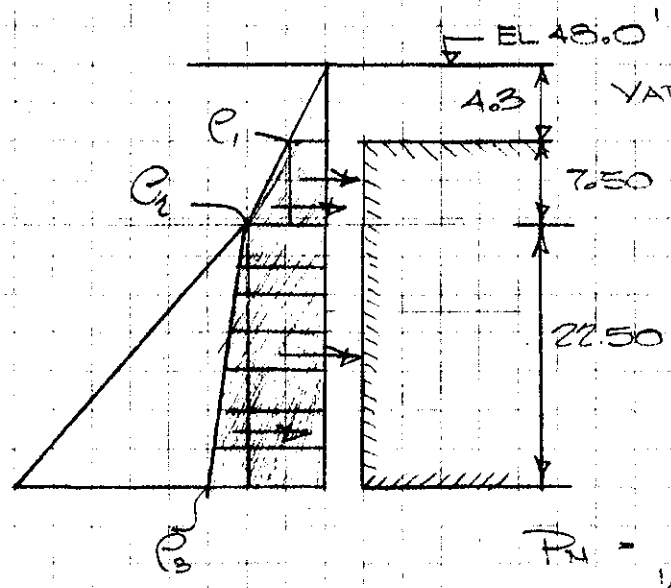
$$FS = \frac{81.45 + 71.04 + 35.623}{105.22} = \frac{188.113}{105.22} = \underline{1.788}$$

SUBJECT: Park River Local Protection
COMPUTATION: Section #4 – Typical Section
COMPUTED BY: W. Rodger CHECKED BY: DATE: 20 Aug 74

(3-A) $H_{cut} = 30'$ $W = 74$

$H_{eff} = 15 + 7.50 = 22.50'$

$\therefore W = 22.50 \times .0625 \times 74.00 = 104.06^k$



VARIES from 4.3' @ Junc Struct
To 5.0' @ Int W Sect #3

$P_1 = 4.3(.06) = .258$
$P_2 = 11.8(.06) = .709$
$P_3 = .709 + 22.50(.02875)$
$.709 + .647$
$= 1.355$

P_H = $7.50(.258) = 1.935$
$\frac{1}{2} \cdot 7.50(.450) = 1.688$
$22.50(.709) = 15.930$
$\frac{1}{2} \times 22.50(.647) = 7.279$
26.832

$F_{TOT} = 2 \times 26.832 \times .50 = 26.832^k$

W_{DL} Conc = 70.35^k (Pg) = 70.35
W_{DL} Soil = $.12 \times 4.3 \times 74$ = 38.18

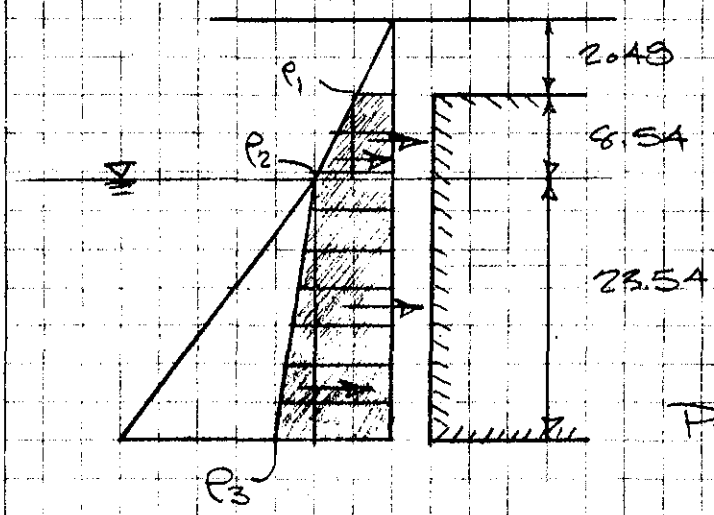
$$F.S. = \frac{\overset{135.362}{70.35 + 38.18 + 26.832}}{104.06} = \underline{1.300}$$

SUBJECT: Park River Local Protection
COMPUTATION: Section #5 - Typical
COMPUTED BY: W. Rodger CHECKED BY: DATE: 20 Aug 74

(A-A) $H_{GW} = 32.08'$ $W = 78.5'$

$H_{GW} = 23.54'$

$\therefore W = 23.54 \times .0625 \times 78.5 = 115.50^k$



$P_1 = 2.48(.06) = .149$
$P_2 = 11.02(.06) = .661$

$P_3 = .661 + (.02875)(23.54)$
$= .661 + .667$
$= 1.328$

$P_H = 8.54(.149) = 1.272$
$\frac{1}{2} \times 8.54(.512) = 2.186$
$23.54(.661) = 15.560$
$\frac{1}{2} \times 23.54(.667) = 7.851$
$F_{TOT} = 26.869$ ✓

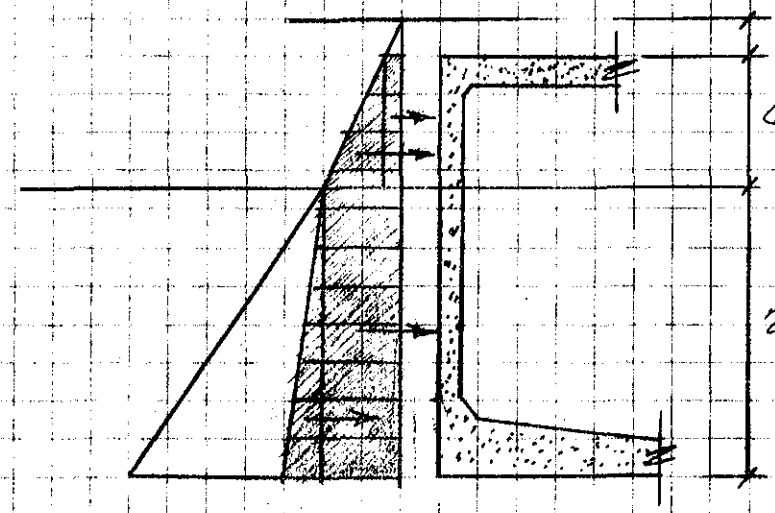
Wgt Conc = 88.88^k (Pg) = 88.88
Wgt Soil = $2.48 \times .12 \times 78.5$ = 23.36

$$FS = \frac{\overset{139.109}{88.88 + 23.36 + 26.869}}{115.50} = 1.204$$

SUBJECT: PARK RIVER LOCAL
COMPUTATION: SECTIONS #7 (SECTION "A")
COMPUTED BY: W. Rodger CHECKED BY: DATE: 20 AUG 74
REVISED 6 SEPT 74 (WHR)

(5-A) $H_{GW} = 28.42$ $W_T = 49.67'$
GROSS = 49.67 × 28.42 = 1411.62 $W_B = 49.67'$
− 2 (536.5) = 338.62
$W_{GW} = 21.92'$ ✓
ACTUALLY 49.83

∴ U = 21.92 × (49.67) × .0625 = 68.05* ↑ ok



(WILL BE 4 to 5' @ Street Crossings)

$P_1 = 2.5(.06) = .150$
$P_2 = 9.41(.06) = .565$
$P_3 = .565 + .02875(21.92)$
$.565 + .630$

$P_N = 6.91(.150) = 1.04$
$= \frac{1}{2} \times 6.91 \times (.415) = 1.43$
$= 21.92(.565) = 12.38$
$= \frac{1}{2} \times 21.92 \times .630 = 6.90$
(21.75)

* NEGLECT DL of PILES

P_{DL} Conc = 50.79^K (P8) = 50.79
P_{DL} Soil top = 3.79 × .120 × 49.67 = 22.59

$$F_o S_o = \frac{50.79 + 22.59 + 21.75}{68.05} = 1.400$$

(5-B)

SUBJECT: Park River Local Protection
COMPUTATION: Section #9, North Section (Park)
COMPUTED BY: W. Radzyt CHECKED BY: DATE: 18 Sept 74

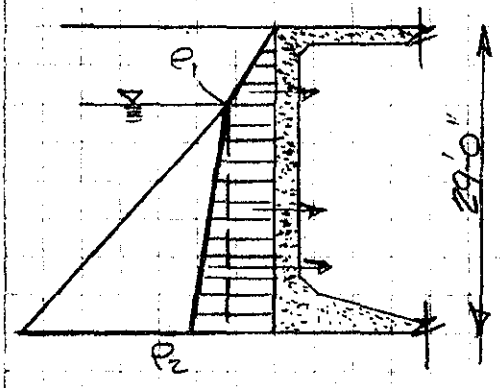
$H_{cutwall} = 29'\text{-}0''$ $W = 49.83'$

Net Weight Conc $= (29 \times 49.83) - 2(536.5) = 372.07\ ft^3$

$W_{conc} = 372.07 \times .150 = 55.81^k$

$H_{GW} = 22'$ (drains 50% Effective)

$U = .0625(22)(49.83) = 68.52^k \uparrow$



$e_1 = 7'(.06) = .42$

$e_2 = .42 + 22'(.02875)$
$= .42 + .633 = 1.053$

$P_b = .42 \times 7' \times 1/2 = 1.47^k$
$.42 \times 22' = 9.24$
$.633 \times 22' \times 1/2 = 6.93$
17.64^k

$F.S. = \dfrac{55.81 + 17.64}{68.52} = \underline{\underline{1.072}} \approx 1.01$ ok

South Section Has 2.5 Fill

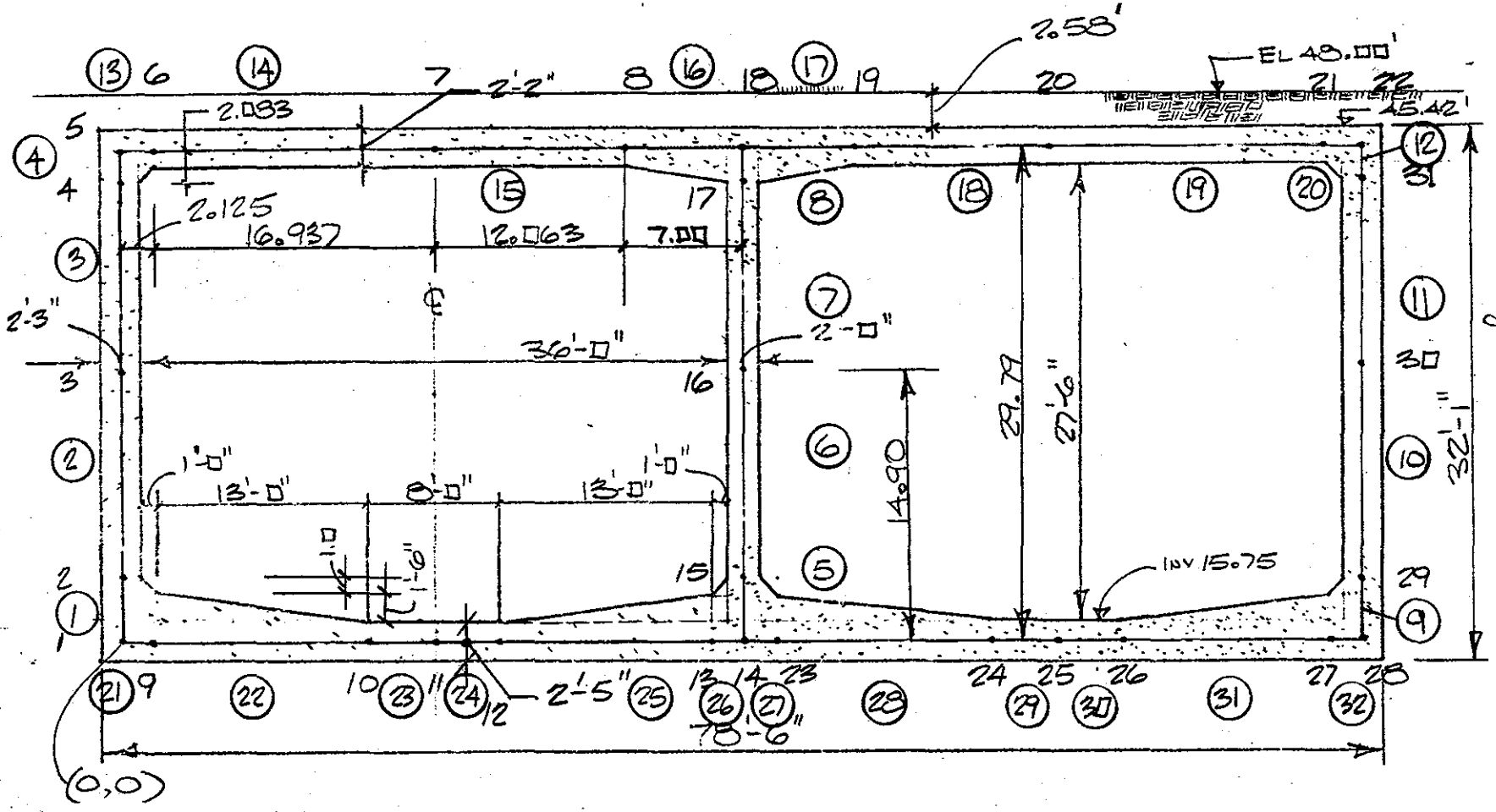
$W_{fill} = .120 \times 49.83 \times 2.5 = 14.95^k$

$F.S. = \dfrac{55.81 + 18^{\circledast} + 14.95}{68.52} = \underline{1.295}$

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U.S. ARMY

SUBJECT Park River Local Protection
COMPUTATION Section #5
COMPUTED BY W. Rodger CHECKED BY DATE 7 Aug 74
PAGE DM2-8 5-1



$$\text{Void Vol} = 27.5 \times 36 - \tfrac{1}{2}(1)(1) - \tfrac{1}{2}(6)(1) - \left[\tfrac{1}{2}(13 \times 1.5) + \tfrac{1}{2}(1 \times 1) + 1(1.5)\right] 2$$

$$990 - 0.50 - 3.0 - 19.5 - 1.0 - 3.0 = 963 \text{ Ft}^3$$

$$\text{Vol Conc} = 2518.52 - 1926 = 592.52 = 21.95 \text{ yd}^3/\text{Ft}$$

SUBJECT: Park River

COMPUTATION: Member Properties Section #5

COMPUTED BY: W. Rodger CHECKED BY: DATE: 9 Aug 74

	Members	Area (A_x)	Mom of Inertia (I_{zz})
Ext Walls	1, 9	$A = 1 \times 6.5 = 6.5\ Ft^2$	$I = 22.90\ Ft^4$
	2, 3, 10, 11	$A = 2.25\ Ft^2$	$I = .949\ Ft^4$
	4, 12	$A = 2.25$	$\bar{d} = 2.75$, $I = 1.733\ Ft^4$
Int. Walls	5	$A = 13 \times 1 = 13$	$I = 183\ Ft^4$
	6, 7	$A = 2.0 \times 1 = 2$	$I = .667\ Ft^4$
	8	$A = 7.0$	$I = 28.00$
Top Slab	13, 16, 17, 20	$A = 2.667$	$I = 1.580$
	14, 15, 18, 19	$A = 2.167$	$I = .848$
	21, 26, 27, 32	$A = 3.917$	$I = 4.417$
	22, 25, 28, 31	$A = 3.167$	$I = 2.647$
	23, 24, 29, 30	$A = 2.417$	$I = 1.177$

SUBJECT: Park River Local Protection 5-3

COMPUTATION: Nodal Point Arrangement

COMPUTED BY: W. Rodger CHECKED BY: DATE: 9 Aug 74

Consider 1'-0" Slice Thru Culvert

Nodal Point Arrangement		
Node	X-Coor	Y-Coor
1	0.	0.
2	0.	3.709
3	0.	14.900
4	0.	27.707
5	0.	29.790
6	2.125	29.790
7	19.062	29.790
8	31.125	29.790
9	2.125	0.
10	15.125	0.
11	19.125	0.
12	23.125	0.
13	36.125	0.
14	38.125	0.
15	38.125	3.709
16	38.125	14.900
17	38.125	27.707
18	38.125	29.790
19	45.125	29.790
20	57.188	29.790
21	74.125	29.790
22	76.250	29.790
23	40.125	0.
24	53.125	0.
25	57.125	0.
26	61.125	0.
27	74.125	0.
28	76.250	0.
29	76.250	3.709
30	76.250	14.900
31	76.250	27.707

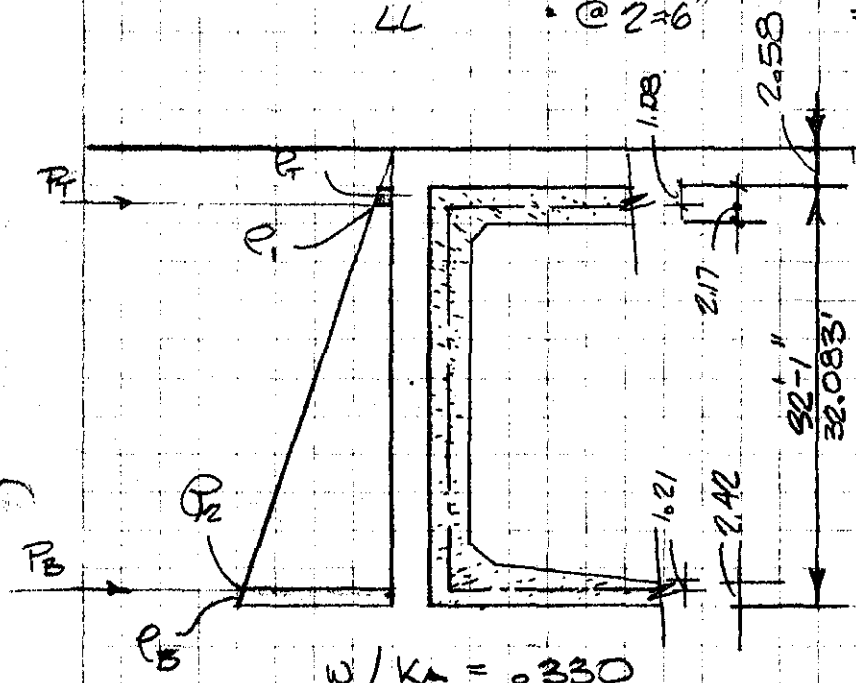
Element Properties				
Elt	I	J	A_{AVE}	I_{AVE}
1	1	2	6.5	22.90
2	2	3	2.25	.949
3	3	4	2.25	.949
4	4	5	2.25	1.733
5	14	15	13.0	183.
6	15	16	2.00	.667
7	16	17	2.00	.667
8	17	18	7.00	28.00
9	28	29	6.5	22.90
10	29	30	2.25	.949
11	30	31	2.25	.949
12	31	22	2.25	1.733
13	5	6	2.667	1.58
14	6	7	2.167	.848
15	7	8	2.167	.848
16	8	18	2.667	1.58
17	18	19	2.667	1.58
18	19	20	2.167	.848
19	20	21	2.167	.848
20	21	22	2.667	1.58
21	1	9	3.917	4.417
22	9	10	3.167	2.647
23	10	11	2.417	1.177
24	11	12	2.417	1.177
25	12	13	3.167	2.647
26	13	14	3.917	4.417
27	14	23	3.917	4.417
28	23	24	3.167	2.647
29	24	25	2.417	1.177
30	25	26	2.417	1.177
31	26	27	3.167	2.647
32	27	28	3.917	4.417

Overburden

$DL_{soil} = 2.583' \times .120 = .310$

$DL_{slab} = 2.167 \times .150 = .325$

LL @ 2'-6" $= \frac{.375}{1.011\ ksf} \longrightarrow 1\ ksf$



$\gamma = 120\ \#/ft^3$

$K_o = .50$
$K_A = .33$

No Ground Water

$w/K_A = .330$

$\rho_t = .04(3.12) = .125 \quad P_T = .135^k$

$\rho_1 = .04(3.66) = .146^{ksf}$

$\rho_2 = .04(33.45) = 1.338^{ksf}$

$\rho_b = .04(34.06) = 1.362 \quad P_B = 1.648^k$

$w/K_o = .50$

$P_T = .203^k$

$\rho_1 = .219$

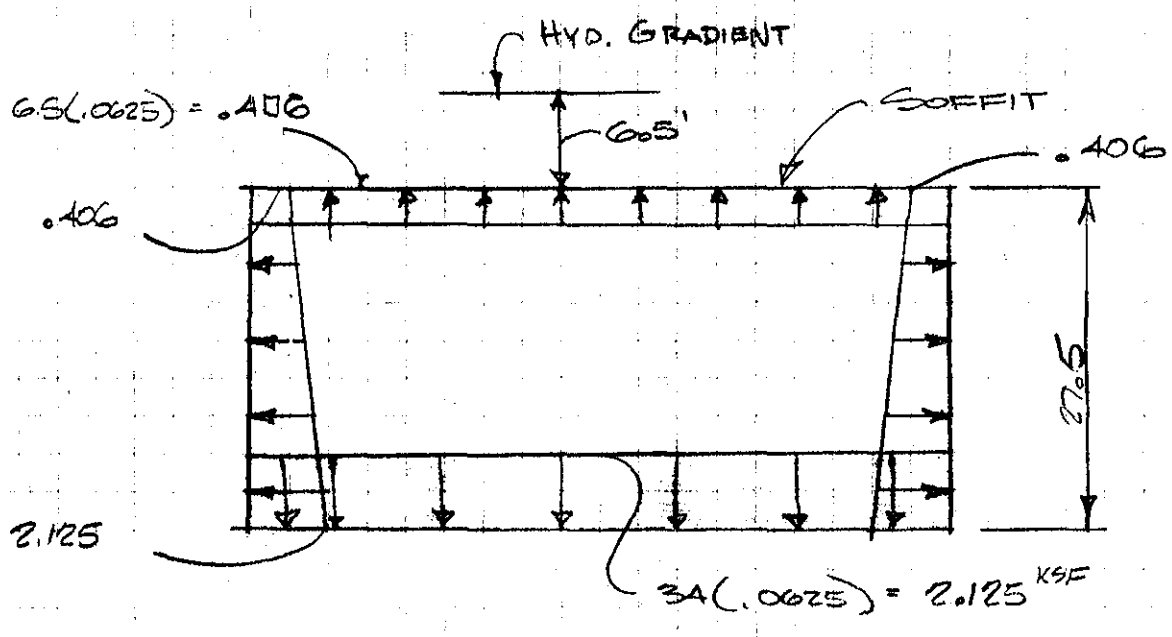
$\rho_2 = 2.00$

$P_B = 2.472^k$

SUBJECT Park River Local Protection 5-6

COMPUTATION Section 5 - Internal Pressure

COMPUTED BY W. Rodger CHECKED BY DATE 8 AUG 74

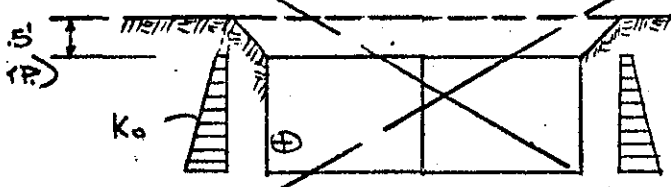


SUBJECT: PARK RIVER LOCAL PROTECTION

COMPUTATION: LOADING CASES SECTION #5

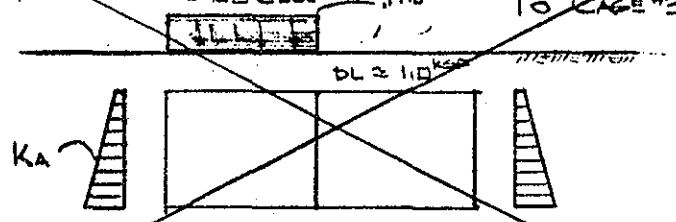
COMPUTED BY: W. Rodger CHECKED BY: DATE: 8 AUG 74

CASE #1



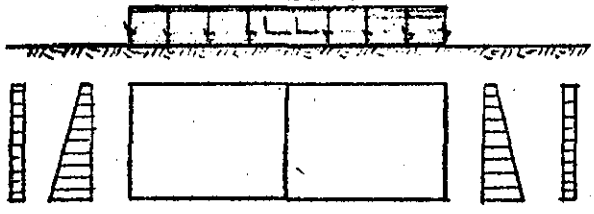
* PERFORMED BUT NOT CRITICAL

CASE #VOID (RESULTS WILL BE SIMILAR TO CASE #3)

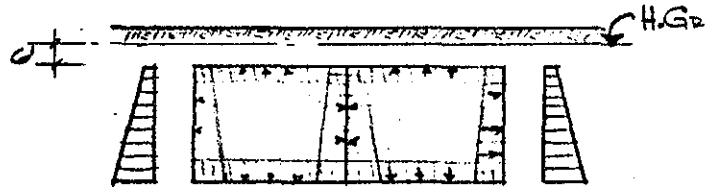


CASE #3 WILL BE SUFFICIENT TO DETERM. DESIGN MOM & SHEARS

CASE #2

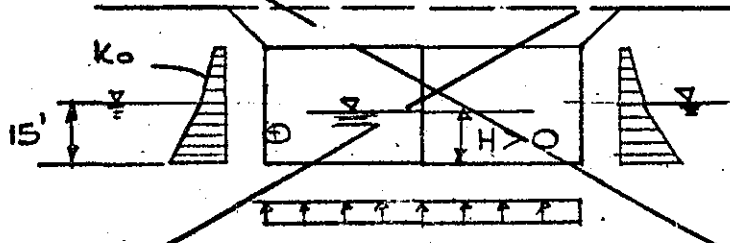


CASE #3 - BURST 6' OF HEAD

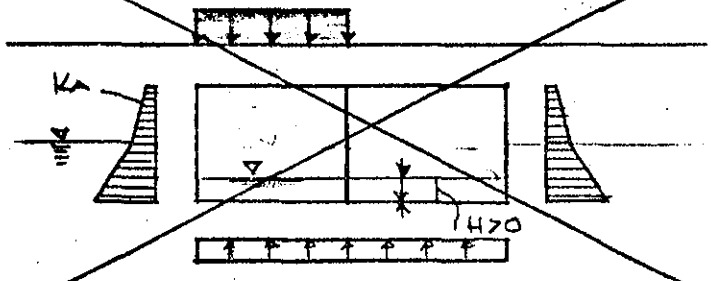


NOT CRITICAL FOR BOX - BUT DOES HAVE HIGHEST BASE PRESSURES

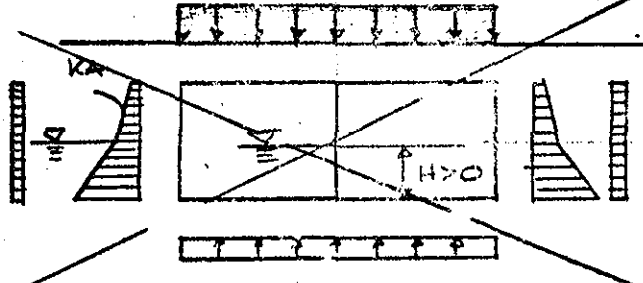
CASE #4 * PERFORMED BUT NOT CRITICAL



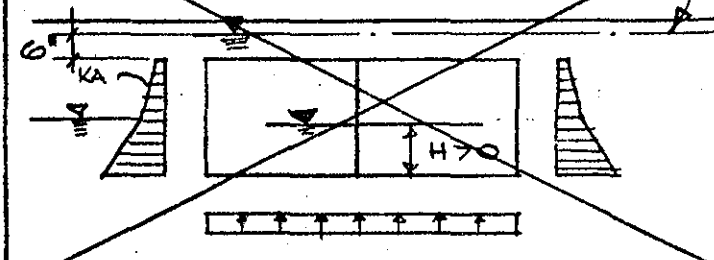
CASE #VOID



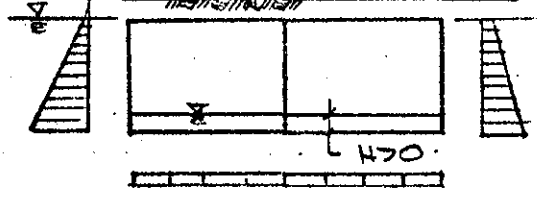
CASE #5 * PERFORMED BUT NOT CRITICAL



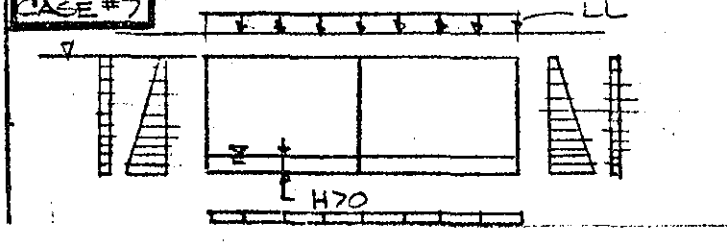
CASE #VOID



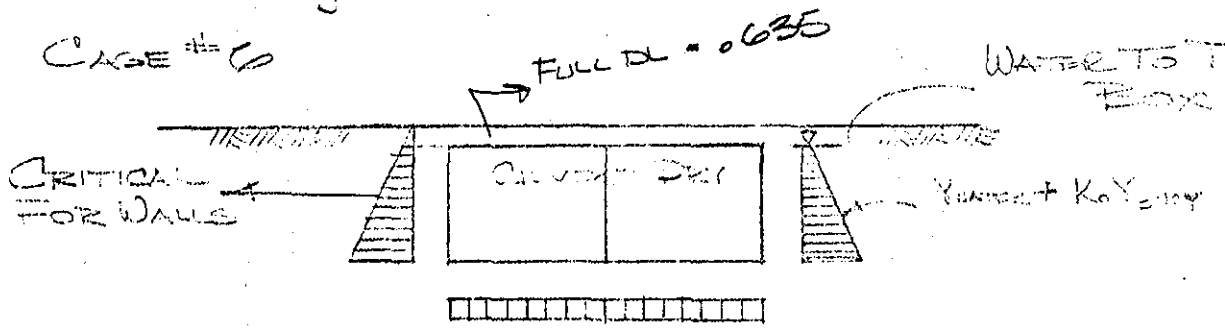
CASE #6



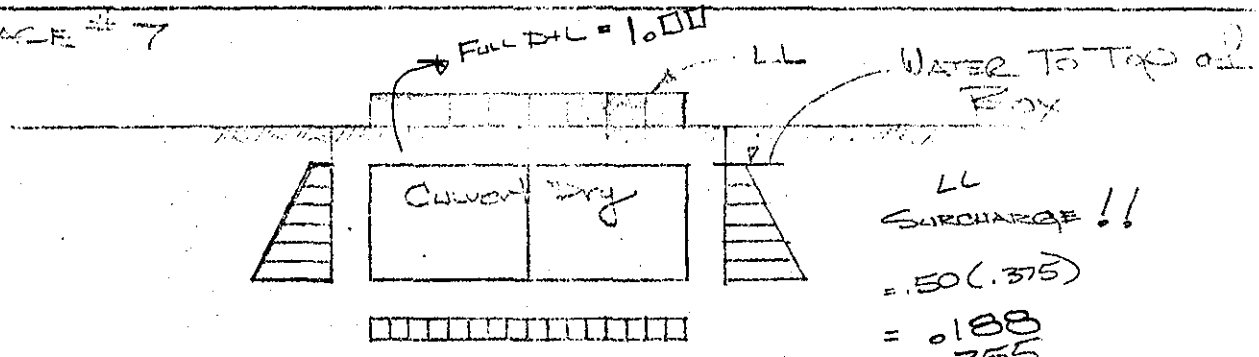
CASE #7



CASE #6



CASE #7



LL SURCHARGE !!

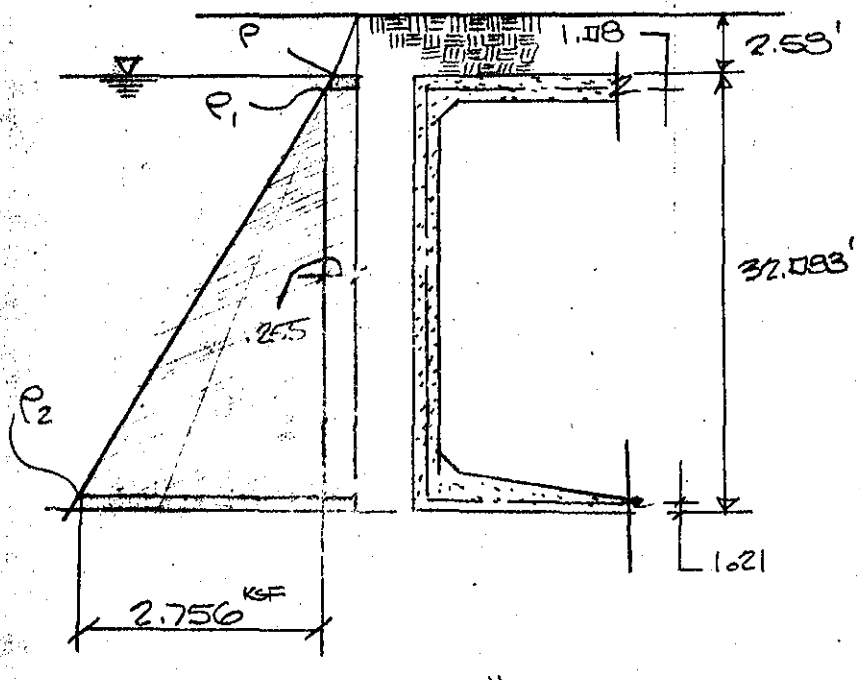
= .50(.375)

= .188

.255

.443 KSF

REFER Pg 5-4



$P = 2.58(.06) = .155$

$P_1 = .155 + 1.08(.0925)$ (.100)

$= .255$

$P = \bar{P}(1.08) = .221^{K}$

$P_2 = .155 + .0925(30.083)$ (2.856)

$= 3.011$

$P_B = 3.067$

$P_B = 3.011^{K}$

Use FILE "PARKS5"

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

PAGE DM 7-15
5-7A

SUBJECT: Park River Local Protection
COMPUTATION: Typ. Computer Output, Section #5, LC 2-3-6-7
COMPUTED BY: W. Rodger CHECKED BY: DATE: Aug 74

ELEMENTS ON AN ELASTIC FOUNDATION (LC-2)

ELEMENT	SPRING PRESSURE I	J
21	.233177E+01	.254200E+01
22	.254200E+01	.136960E+01
23	.136960E+01	.123190E+01
24	.123190E+01	.134361E+01
25	.134361E+01	.214425E+01
26	.214425E+01	.215692E+01
27	.215692E+01	.214434E+01
28	.214434E+01	.134923E+01
29	.134923E+01	.123257E+01
30	.123257E+01	.137046E+01
31	.137046E+01	.254232E+01
32	.254232E+01	.233063E+01

END ACTIONS WITH ELEMENT LOADS LC-2

	ELEMENT	AXIAL I / AXIAL J	SHEAR I / SHEAR J	MOMENT I / MOMENT J
Left Wall	1	24.931	17.340	113.564
		-24.931	-12.133	-63.960
	2	21.057	12.133	63.960
		-21.057	.017	-.536
	3	15.779	-.017	.536
		-15.779	7.333	-57.353
	4	15.503	-7.333	57.354
		-15.503	3.433	-74.362
Center Wall	5	51.009	.002	.015
		-51.009	-.002	-.009
	6	47.439	.002	.009
		-47.439	-.002	.009
	7	43.439	.002	-.009
		-43.439	-.002	.029
	8	43.240	.002	-.029
		-43.240	-.002	.032
Right Wall	9	24.924	-17.340	-113.550
		-24.924	12.139	63.943
	10	21.051	-12.139	-63.943
		-21.051	-.015	.502
	11	15.772	.015	-.502
		-15.772	-7.336	57.799
	12	15.501	7.336	-57.799
		-15.501	-3.433	74.304
	13	3.623	15.503	74.362
		-3.623	-14.273	-41.263
	14	3.623	14.273	41.263

SUBJECT: PARK RIVER LOCAL PROTECTION 5-7B

COMPUTATION: TYP. Computer OUTPUT, Section #5, LC-2,3,6,7

COMPUTED BY: W. Rodger CHECKED BY: DATE: Aug 74

LC-2

Roof Slab	16	-3.620	21.000	-172.259
		3.621	21.613	172.227
	17	-3.621	-14.613	-45.499
		3.621	14.613	45.499
	18	-3.621	-2.553	53.076
		3.621	2.553	-53.076
	19	-3.621	14.032	-42.069
		3.621	-14.031	42.069
	20	-3.621	16.551	-74.134
		13.933	-25.663	-113.584
	21	-13.933	21.069	63.910
		13.933	-21.676	-63.910
	22	-13.933	1.572	-62.926
		13.933	-1.572	62.927
	23	-13.933	-2.098	-61.713
		13.933	2.098	61.713
	24	-13.933	-5.721	-46.253
		13.933	5.721	46.253
Base Slab	25	-13.933	-22.716	129.333
		13.933	22.716	-129.332
	26	-13.933	-25.314	173.336
		13.990	-25.315	-173.351
	27	-13.990	22.713	129.313
		13.990	-22.719	-129.310
	28	-13.990	5.719	-46.253
		13.990	-5.719	46.253
	29	-13.990	2.035	-61.704
		13.990	-2.035	61.704
	30	-13.990	-1.530	-62.397
		13.990	1.530	62.397
	31	-13.990	-21.036	69.100
		13.990	21.036	-69.100
	32	-13.990	-25.601	113.552

ELEMENTS ON AN ELASTIC FOUNDATION

LC-3

ELEMENT	SPRING PRESSURE I	J
21	.432635E+01	.405235E+01
22	.405235E+01	.235257E+01
23	.235257E+01	.274493E+01
24	.274493E+01	.230524E+01
25	.230524E+01	.328324E+01
26	.328324E+01	.323425E+01
27	.323425E+01	.322326E+01
28	.322326E+01	.230546E+01
29	.230546E+01	.274526E+01
30	.274526E+01	.235307E+01
31	.235307E+01	.405336E+01
32	.405336E+01	.432657E+01

SUBJECT: Park River Local Protection 5-7C

COMPUTATION: TYP. Computer Output, Section 5, LC-2,3,6,7

COMPUTED BY: W. Rodger CHECKED BY: DATE: AUG 74

END ACTIONS WITH ELEMENT LOADS LC-3

	ELEMENT	AXIAL I AXIAL J	SHEAR I SHEAR J	MOMENT I MOMENT J
LEFT WALL	1	10.371	-3.576	-6.461
		-10.371	5.773	-20.000
	2	6.493	-5.773	20.391
		-6.493	-1.137	-43.714
	3	2.219	1.137	43.714
		-2.219	-6.440	7.234
	4	1.949	6.440	-3.234
		-1.949	-7.012	22.245
Center WALL	5	21.333	[illegible]	[illegible]
		-21.333	-.000	-.000
	6	17.764	.000	.000
		-17.764	-.000	.000
	7	13.314	.000	-.000
		-13.314	-.000	.000
	8	13.564	.000	-.000
		-13.564	-.000	.000
RIGHT WALL	9	10.370	3.576	6.461
		-10.370	-5.773	20.000
	10	6.493	5.773	-20.000
		-6.493	1.137	43.709
	11	2.217	-1.137	-43.709
		-2.217	6.440	-3.244
	12	1.946	-6.440	3.244
		-1.946	7.012	-22.255
Roof SLAB	13	-6.334	1.949	-22.244
		6.334	-1.462	25.376
	14	-6.334	1.461	-25.376
		6.334	2.416	17.797
	15	-6.334	-2.416	-17.797
		6.334	5.130	-23.042
	16	-6.334	-5.130	23.042
		6.334	6.733	-69.910
	17	-6.334	6.732	69.903
		6.334	-5.179	-23.037
	18	-6.334	5.179	23.037
		6.334	-2.413	17.771
	19	-6.334	2.413	-17.771
		6.334	1.462	25.367
	20	-6.334	-1.461	-25.367
		6.334	1.947	22.255
BASE SLAB	21	-6.923	-11.346	6.461
		6.923	7.949	-26.601
	22	-6.923	-7.949	26.601
		6.923	-2.061	-51.364
	23	-6.923	2.061	51.364
		6.923	-3.239	-40.321
	24	-6.923	3.239	40.322
		6.923	-4.241	-25.243
	25	-6.923	4.241	25.243

SUBJECT Park River Local Protection 5-7D

COMPUTATION Typ. Computer Output, Section #5, LC-2,3,6,7

COMPUTED BY W. Rodger CHECKED BY DATE Aug 74

	6.923	9.936	64.309
23	-6.923	-9.933	-64.307
	6.923	4.341	-25.257
29	-6.923	-4.341	25.257
	6.923	3.233	-48.335
30	-6.923	-3.233	48.335
	6.923	2.053	-51.071
31	-6.923	-2.353	51.071
	6.923	-7.951	-26.522
32	-6.923	7.953	26.522
	6.923	-11.047	-5.462

END ACTIONS WITH ELEMENT LOADS

LC-6

ELEMENT		AXIAL I AXIAL J	SHEAR I SHEAR J	MOMENT I MOMENT J
1	9	20.574	34.171	174.936
		-20.574	-23.637	-68.096
2	10	16.701	23.637	68.096
		-16.701	.424	50.979
3	11	12.422	-.424	-50.979
		-12.422	13.746	-55.981
4	12	12.151	-13.746	55.981
		-12.151	14.476	-85.401
5		31.889	.001	.005
		-31.889	-.001	-.002
6		28.319	.001	.002
		-28.319	-.001	.007
7		24.369	.001	-.007
		-24.369	-.001	.017
8	13	24.119	.001	-.017
		-24.119	-.001	.019
	14	14.697	12.151	85.401
		-14.697	-10.799	-60.959
	15	14.697	10.799	60.959
		-14.697	-.048	30.862
	16	14.697	.048	-30.862
		-14.697	7.616	-14.810
	17	14.696	-7.616	14.810
		-14.696	12.061	-83.680
	18	14.696	12.058	83.661
		-14.696	-7.613	-14.809
	19	14.696	7.613	14.809
		-14.696	.045	30.831
	20	14.695	-.045	-30.831
		-14.695	10.802	-61.035
			-10.801	61.035
			12.143	-85.361

Left Wall & Right Wall

Center Wall

Roof Slab

SUBJECT Park River Local Protection 5-7E

COMPUTATION Typ. Computer Output, Section #5, LC-2,3,6,7

COMPUTED BY W. Rodger CHECKED BY DATE Aug 74

	21	37.882	-21.251	-174.936
		-37.882	19.007	132.021
	22	37.882	-19.008	-132.021
		-37.882	4.860	-20.181
	23	37.882	-4.860	20.181
		-37.882	1.351	-32.453
	24	37.882	-1.351	32.453
		-37.882	-1.969	-31.244
	25	37.882	1.969	31.244
Base Slab		-37.882	-14.246	71.388
	26	37.882	14.245	-71.389
		-37.882	-16.256	101.898
	27	37.883	-16.257	-101.904
		-37.883	14.247	71.392
	28	37.883	-14.247	-71.392
		-37.883	1.966	-31.233
	29	37.883	-1.966	31.233
		-37.883	-1.356	-32.426
	30	37.883	1.356	32.426
		-37.883	-4.868	-20.127
	31	37.883	4.868	20.127
		-37.883	-19.016	132.201
	32	37.883	19.016	-132.201
		-37.883	-21.247	174.919

END ACTIONS WITH ELEMENT LOADS

LC-7

	ELEMENT	AXIAL I AXIAL J	SHEAR I SHEAR J	MOMENT I MOMENT J
	1	26.632	36.754	195.867
		-26.632	-25.523	-80.737
Left Wall	2	22.759	25.523	80.737
		-22.759	.641	47.672
	3	18.480	-.641	-47.672
		-18.480	16.372	-77.502
	4	18.209	-16.372	77.502
		-18.209	17.493	-112.791
	5	47.607	.001	.012
		-47.607	-.001	-.007
Center Wall	6	44.037	.001	.007
		-44.037	-.001	.010
	7	40.087	.001	-.010
		-40.087	-.001	.028
	8	39.837	.001	-.028
		-39.837	-.001	.031
	9	26.626	-36.756	-195.849
		-26.626	25.524	80.713
Right Wall	10	22.753	-25.524	-80.713
		-22.753	-.640	-47.712
	11	18.474	.640	47.712
		-18.474	-16.370	77.445
	12	18.203	16.370	-77.445
		-18.203	-17.492	112.731

SUBJECT Park River Local Protection 5-7F

COMPUTATION Typ. Computer Output, Section #5, LC-2,3,6,7

COMPUTED BY W. Rodger CHECKED BY DATE Aug 74

	13	20	17.714	18.209	112.791
			-17.714	-16.079	-76.273
	14	19	17.714	16.079	76.274
			-17.714	.851	52.633
Roof	15	18	17.714	-.851	-52.633
			-17.714	12.921	-30.471
Slab	16	17	17.714	-12.921	30.471
			-17.714	19.921	-145.415
	21		40.465	-27.309	-195.867
			-40.465	23.848	141.368
	22		40.465	-23.849	-141.368
			-40.465	4.921	-38.108
	23		40.465	-4.921	38.108
			-40.465	.575	-48.926
	24		40.465	-.575	48.926
			-40.465	-3.649	-42.872
Base	25		40.465	3.649	42.872
			-40.465	-21.077	111.280
Slab	26		40.465	21.077	-111.281
			-40.465	-24.115	156.478
	27		40.467	-24.116	-156.490
			-40.467	21.078	111.290
	28		40.467	-21.079	-111.290
			-40.467	3.645	-42.860
	29		40.467	-3.645	42.860
			-40.467	-.581	-48.895
	30		40.467	.581	48.895
			-40.467	-4.931	-38.045
	31		40.467	4.931	38.045
			-40.467	-23.861	141.597
	32		40.467	23.861	-141.597
			-40.467	-27.303	195.849

SUBJECT: Park River Local Protection 5-8

COMPUTATION: Design of Reinf. Steel - Section #5

COMPUTED BY: W. Rodger CHECKED BY: DATE: 27 Aug 74

Roof Slab Max M⊖ from LC-#2 & LC-#5

$M = 172.36^{'k}$ $V = 21.62^{k}$ $N = 8.621^{k}$ Lin

$M_{DESIGN} = 172.36 - \frac{21.62(2)}{3} = 157.95^{'k}$

$e = \frac{157.95(12)}{8.621} + 14.3 = 234.16$ $E = 19.51$ $d = 33.3''$ $d'' = 14.3''$

$e/d = 7$ $\ell = 1.14$

$A_{s\ominus} = \frac{19.51(8.621)}{1.44 \times 1.14 \times 33.3} = 3.09$ Use → 3.12 #11@6" (3.05 #10@5)

$v = \frac{21.62}{12 \times 33.3} = 54.1^{psi}$ ok

$d = 21.3$ $d'' = 8.3$

$e_{\ominus} = \frac{61.395(12)}{8.621} + 8.3 = 93.76$

$E = 7.81$ $e/d = 4.4$ $\ell = 1.25$

$A_{s\ominus} = \frac{7.81(8.621)}{1.44 \times 1.25 \times 21.3}$

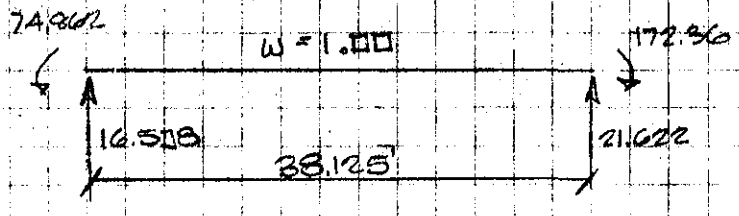
$= 1.76 \text{ in}^2$

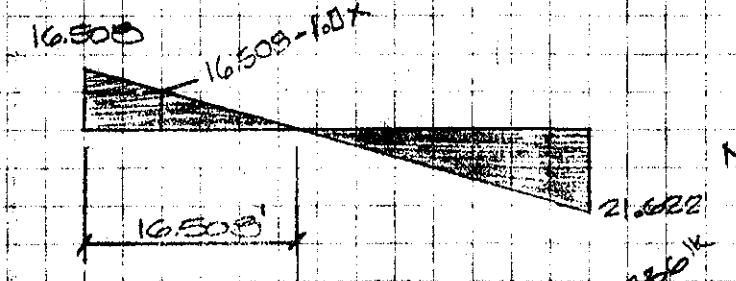
Provide #9@6 2.00

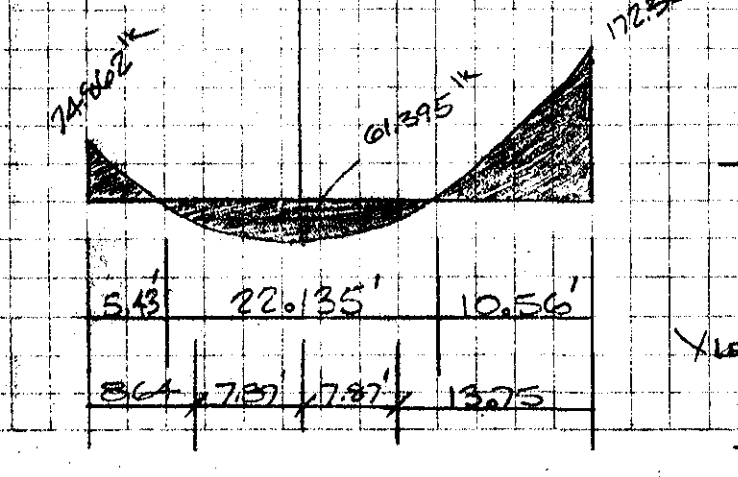
Cut off Half of Steel

$M_{\ominus} = 61.395^{'k}$ ✓

LC #2







$M = -.50x^2 + 16.50x - 74.862$

@ $x = 38.125$ $M = 172.56$ ok

$(x - 16.50)^2 = 272.25 - 149.72$

$x = \pm 11.07 + 16.50$

$x_1 = 5.43$ $x_2 = 27.57$ (10.56)

$x^2 = \left(\frac{31}{.5}\right)^{1/2} = 7.87'$

$X_{LGT} = 8.64 - 2.125 - 1.25$

$= 5.39'$ Use 5'-4"

SUBJECT: Park River Local Protection 5-9

COMPUTATION: Design Reinf Steel - Section #5

COMPUTED BY: W. Rodger CHECKED BY: DATE: 27 Aug 74

Member 13 & 20 — Corner of Roof Slab Governed by LC-#7

M = 112.79 V = 18.209 N = 17.714

$M_{DES} = 112.79 - \frac{18.209(2.25)}{3} = 99.13$ d = 33.3, d'' = 14.3

e = 81.45 E = 6.79 e/d = 2.5 ℓ = 1.52

$A_{s\Theta} = \frac{6.79(17.714)}{1.44 \times 1.52 \times 33.3} = 1.65 \text{ in}$

$v = 46.6^{psi}$ ok

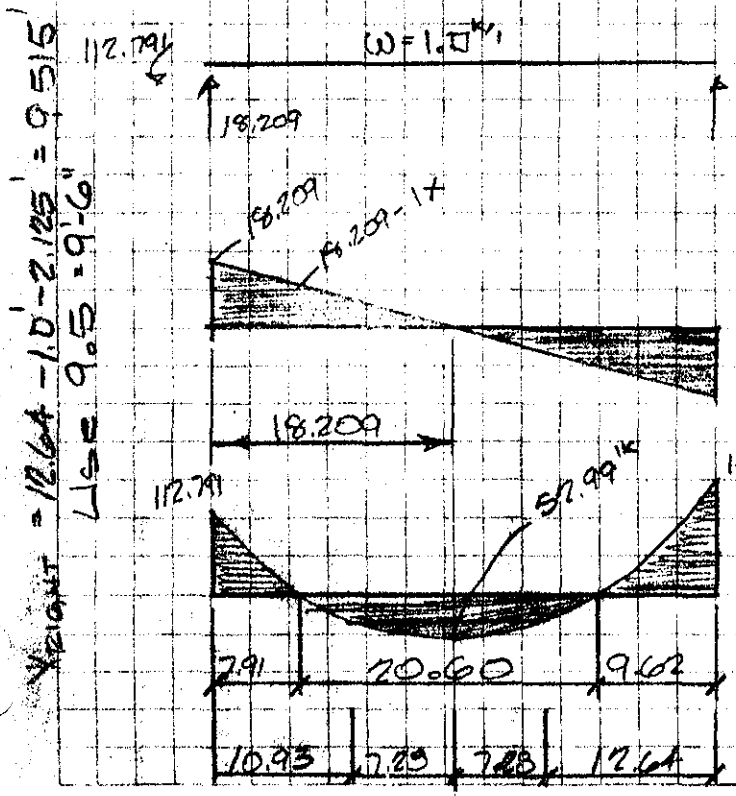
Member 14 & 19 M = 76.273 V = 16.079 N = 17.714

d = 21.3, d'' = 8.3 e = 60 E = 5 e/d = 2.8 ℓ = 1.44, NE = 86.57

$A_s = \frac{86.57}{1.44 \times 1.44 \times 21.3} = 2.00 \text{ in}^2$ #9@6"

LC-#7

$v = 16.079/12 \times 21.3 = 63^{psi}$



$M_\Theta = -112 + .5 \times 18.209 \times 18.209$

$= 52.99^{1k}$ $A_{s\Theta}$ 1.26 Less than LC-#2

$M = -.50x^2 + 18.209x - 112.791 = 0$

@ x = 8.125 M = 145.33^1k ok

$(X - 18.209)^2 = -225.53 + 331.57$

$X = \pm 10.3 + 18.209$

$X_1 = 7.91$ $X_2 = 28.51$ (9.62)

$X_{1/2\Theta} = \left(\frac{26.5}{.5}\right)^{1/2} = 7.28$

SUBJECT Park River Local Protection 5-10

COMPUTATION Steel Details

COMPUTED BY W. Rodger CHECKED BY DATE 27 Aug 74

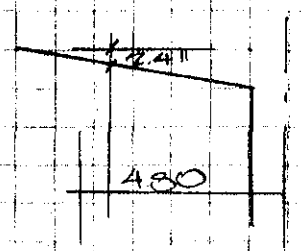
Cutoff for Negative Steel - Base On LC-2

$$M_{cut} = \frac{1.56}{3.12}(172.36)\frac{25.10}{33.3} = 64^{'k}$$

$$(X-16.50)^2 = (-21.72+272.25)^{1/2} = 15.83$$

$$X = \mp 15.83 + 16.50 \qquad X_1 = .67 \qquad X_2 = 32.33\ (5.80')$$

$V = 8.621 \quad M = 64^{'k}$



$d'' = 8.3$

$c = 97.4 \quad E = 8.12 \quad c/d = 4.1 \quad i = 1.26$

$d = 26 - 2.4 - 4.7 = 23.7$ in

$$A = \frac{8.12(8.621)}{1.44 \times 1.26 \times 23.7} = 1.063\ in^2 \qquad \#11@12'' = 1.56\ in^2$$

So $5.90 + ld\ \#11$ (Top Bar) $4.583 = 10.383'$

Say Use 11'-0"

Wall Design D = 2'-3" = 27" d = 22.3 d" = 8.8"

LC #4 M⊕ = 39.5'^k V = 0 N = 11^k

e = 51.89 E = 432 e/d = 2.3 i 1.060

$A_s\oplus = \frac{432(11)}{1.44 \times 1.60 \times 22.3} = .93 \text{ in}^2$

$A_{se} = \frac{.01db}{2} = 1.34 \text{ in}^2$ #10@12" ±

LC-#7 M = 47.67 V = .641 N = 20^k Avg

Smy M⊕ = 50'^k ± $e = \frac{50(12)}{30} + 8.8 = 38.81$ E = 323

e/d = 1.7 i = 2.03 #9@12"

$A_s\oplus = \frac{20(3.23)}{1.44 \times 2.03 \times 22.3} = .99 \text{ in}^2$ Use #10@12"

Steel Outside Face Bottom LC-#7 Governs

2 & 10 M = 80.74'^k V = 25.523^k N = 25^k

e = 47.56 E = 4.00 ± e/d = 2.1 i = 1.69

*v = 95.4^psi

Check by Univ of Ill formula $A_s\oplus = \frac{4.0(25)}{1.44 \times 1.69 \times 22.3} = 1.04 \text{ in}^2$ Use #11@10" (1.87) → #9@6" (2.00)

Steel Outside Face Top LC-#7 Governs

J end of 3 & 11 M = 77.5'^k V = 16.37 N = 18^k v = 61.2^psi ok

e = 60.5 E = 5.04 e = 2.7 i = 1.47

$A_s = \frac{5.04(18)}{1.44 \times 1.47 \times 22.3} = 1.02 \text{ in}^2$ #9@6" = 2.00

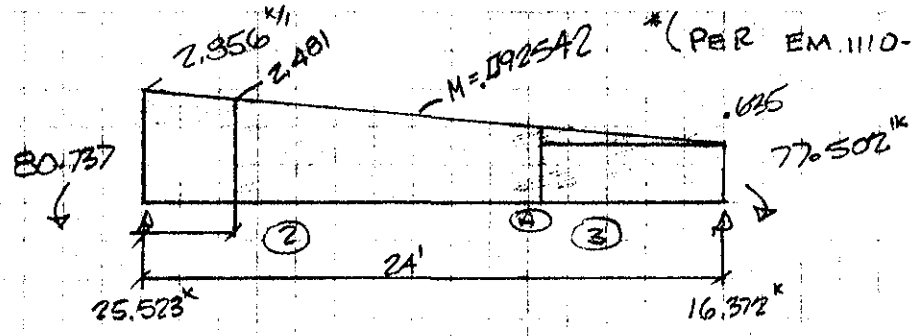
* Matches Top Steel Roof Slab

SUBJECT: Park River Local Protection 5-12

COMPUTATION: Check Shear in Ext. Walls (Wiu of Ill. Method)*

COMPUTED BY: W. Rodger CHECKED BY: DATE: 26 Aug 74

*(PER EM 1110-2-2902)



$\Sigma F_y = .003$ ok

EQUATION FOR MOMENT @ ANY POINT A.

$$77.502 - 16.372x + \frac{.635x^2}{2} + \frac{.092542x^2}{2} \cdot \frac{x}{3}$$

$$M_{@} = 77.502 - 16.372x + .318x^2 + .015423x^3$$

$$@\ x = 24 \quad M = 80.95 \approx 80.74 \text{ ok}$$

Find Roots

X	M
5.0	5.52
6.0	-5.95
18.00	-24.22
19.00	-12.98
20.00	.646

Root = $\frac{5.52}{11.47} = .48 = 5.48'$

Root = 19.95'

$l' = 19.95 - 5.48 = 14.47'$

$A_s = \#7@6'' = 2.0\ in^2/ft$

$\rho = \frac{2.0}{12 \times 22.3'} = .0075$

$N = 25^{\pm}$

$V_2 = 25.523^k - 10.81 = 14.713^k$

$V_2 = 14.713^k$

$\frac{N}{V} = 25/14.713 = 1.699$

$\frac{l'}{d} = \frac{14.47 \times 12}{22.3'} = 7.79$

$$v_c = \frac{(.046 + .0075)(12 + 1.699)}{7.79(19 + 7.79)} 281 = .811 \qquad v_{act} = \frac{2.30^k}{12} = .192$$

$$F.S. = \frac{.811}{.192} = 4.22 > 2.5 \text{ ok}$$

SUBJECT: Park River Local Protection

COMPUTATION: Base Slab Design

COMPUTED BY: W. Rodger CHECKED BY: DATE: 27 Aug 74

Members

21 & 32 LC-7 M = 195.867 N = 40.465 V = 27.309

$$M_{max} = 195.867 - \frac{27.309(2.25)}{3} = 175.39^{'k}$$

$f_v = 42^{psi}$

d" = 17.8" d_F = 52.3 d_v = 54.3

$$e = \frac{175.39(12)}{40.465} + 17.8 = 69.81 \quad E = 5.82 \quad e/d = 1.3 \quad C = 2.96$$

$$A_{s\ominus} = \frac{5.82(40.465)}{1.44 \times 2.96 \times 52.3} = 1.06 \text{ in}^2$$

22 & 31 $f_v = 47^{psi}$

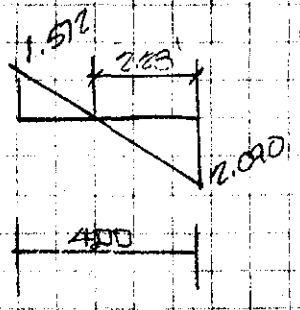
M = 141.37'k N = 40.465 V = 23.849 d" = 7.8 d = 40.3

e = 59.72 E = 5.05 e/d = 1.5 C = 2.34

$$A_s = \frac{5.05(40.465)}{1.44 \times 2.34 \times 40.3} = 1.050 \text{ in}^2$$

Bend #9@6 in wall into base slab 2.00

$M_{\oplus}$ =



LC-#2

d" =

$M_{\oplus}$ = 62.937 + ½(2.09)(2.09) = 65.32'k

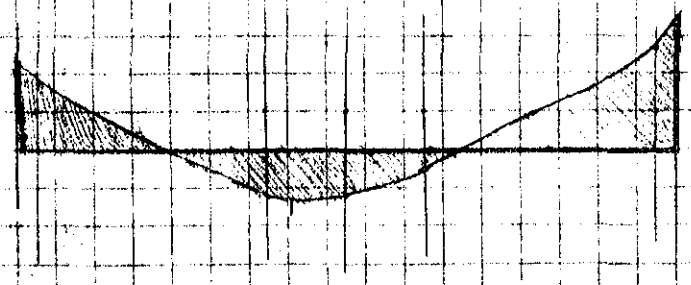
N = 19'k

d = 24.3" d" = 8.8

e = 50 E = 4.07 e/d = 2.1 C = 1.64

$$A_{s\oplus} = \frac{4.07(19)}{1.44 \times 1.64 \times 24.3} = 1.034 \text{ in}^2$$

#9@9" (1.33 in²)



SUBJECT: Park River Local Protection 5-14

COMPUTATION: Base Slab - Lock #2

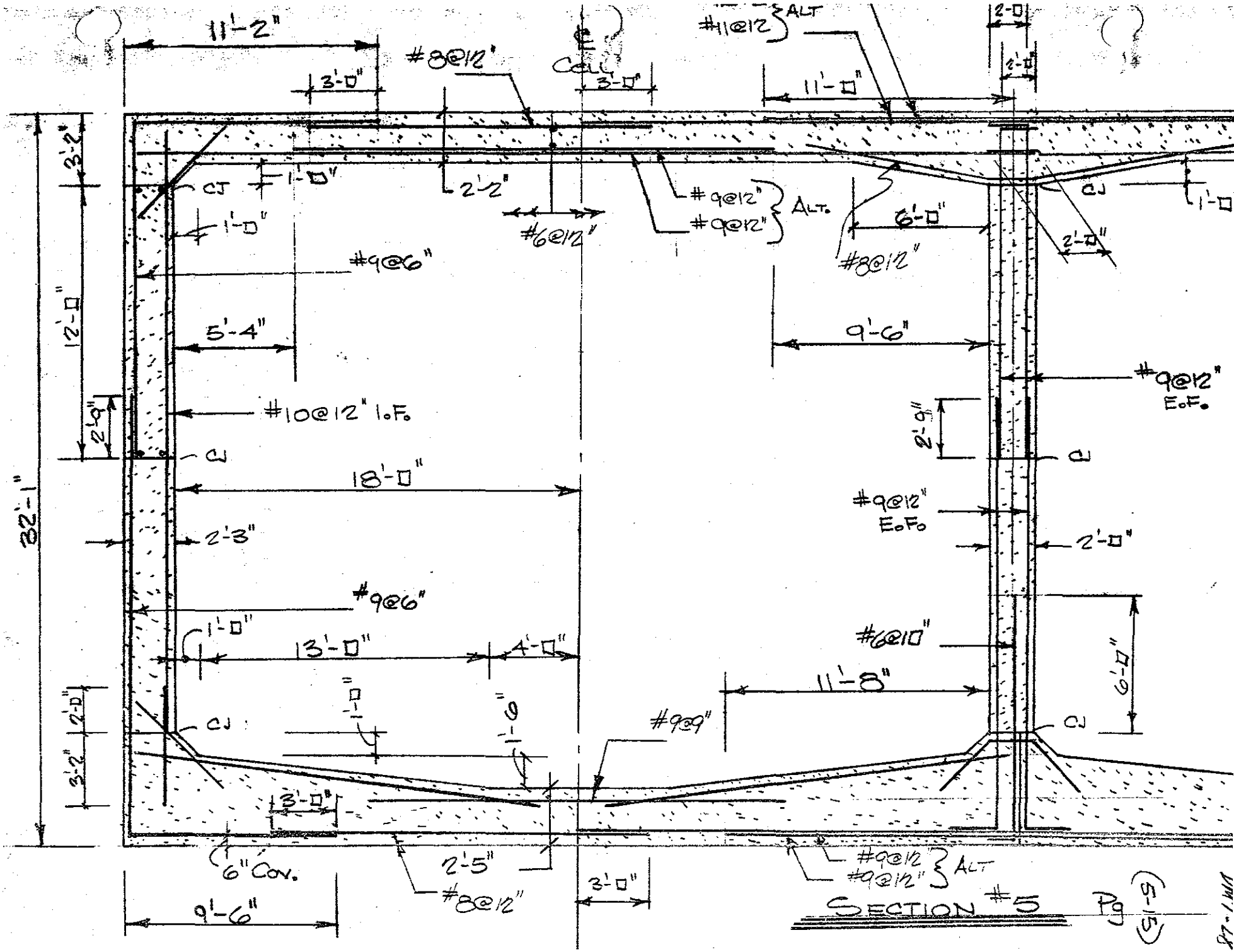
COMPUTED BY: W. Rodger CHECKED BY: DATE: 23 Aug 74

Member 25 @ 29 M = 129.8 N = 22.716 V = 19K

d = 40.3 d'' = 17.8

C = 99.78 E = 8.32 S/d = 2.5 ℓ = 1.52

$$A_{so} = \frac{8.32(19)}{1.44 \times 1.52 \times 40.3} = \underline{1.79 \text{ in}^2} \quad \#9 @ 6''$$



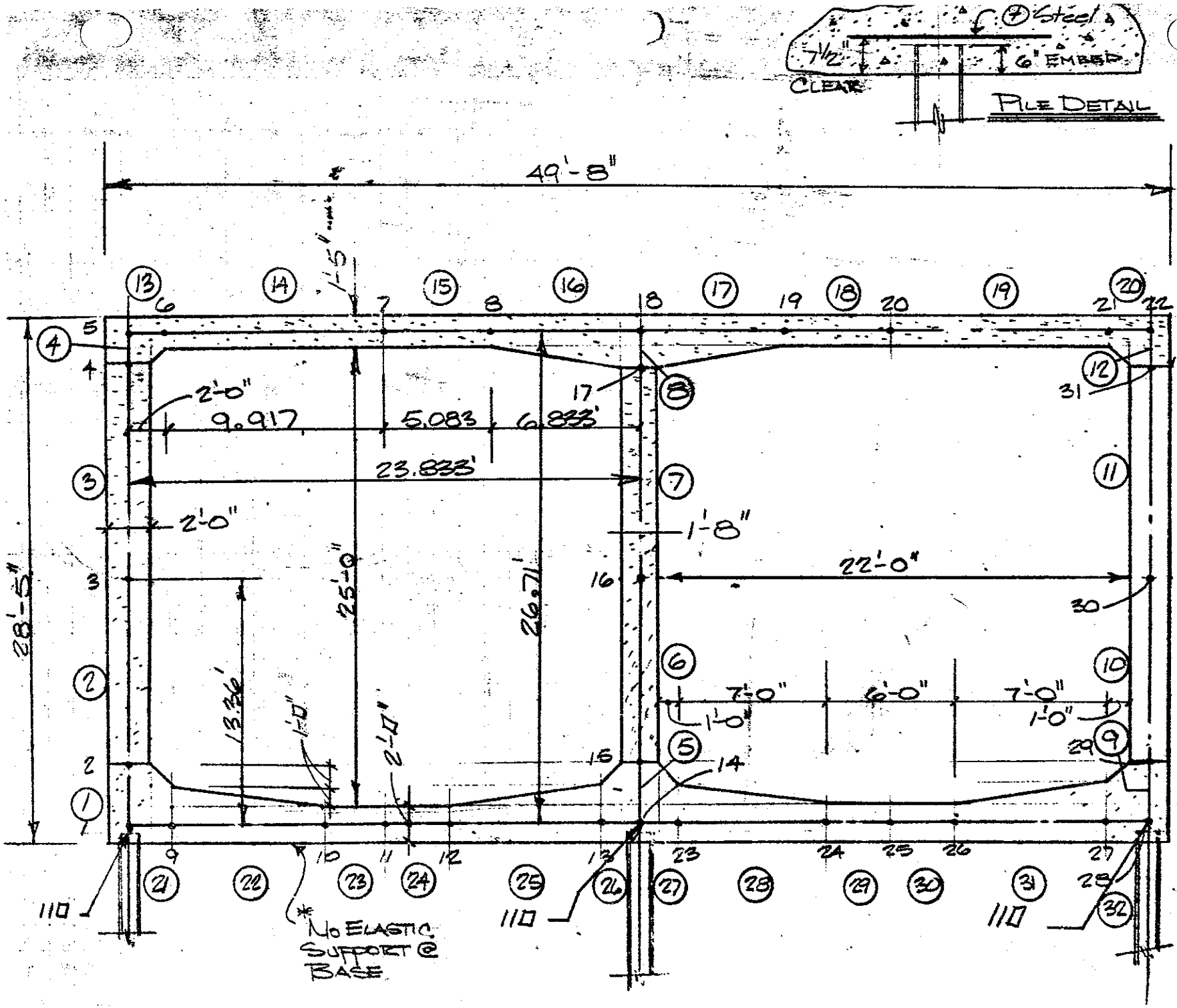
SECTION #5 Pg (5-15)

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT: SECTION #7
COMPUTATION: PARK RIVER
COMPUTED BY: WILLIAM KENNY
CHECKED BY:
DATE: 8 Sept 74

PAGE DM7-29
E-1



NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT 27 Sept 49

COMPUTATION Section #7 PARK RIVER

COMPUTED BY W. Rodger CHECKED BY DATE 13 Sept 74

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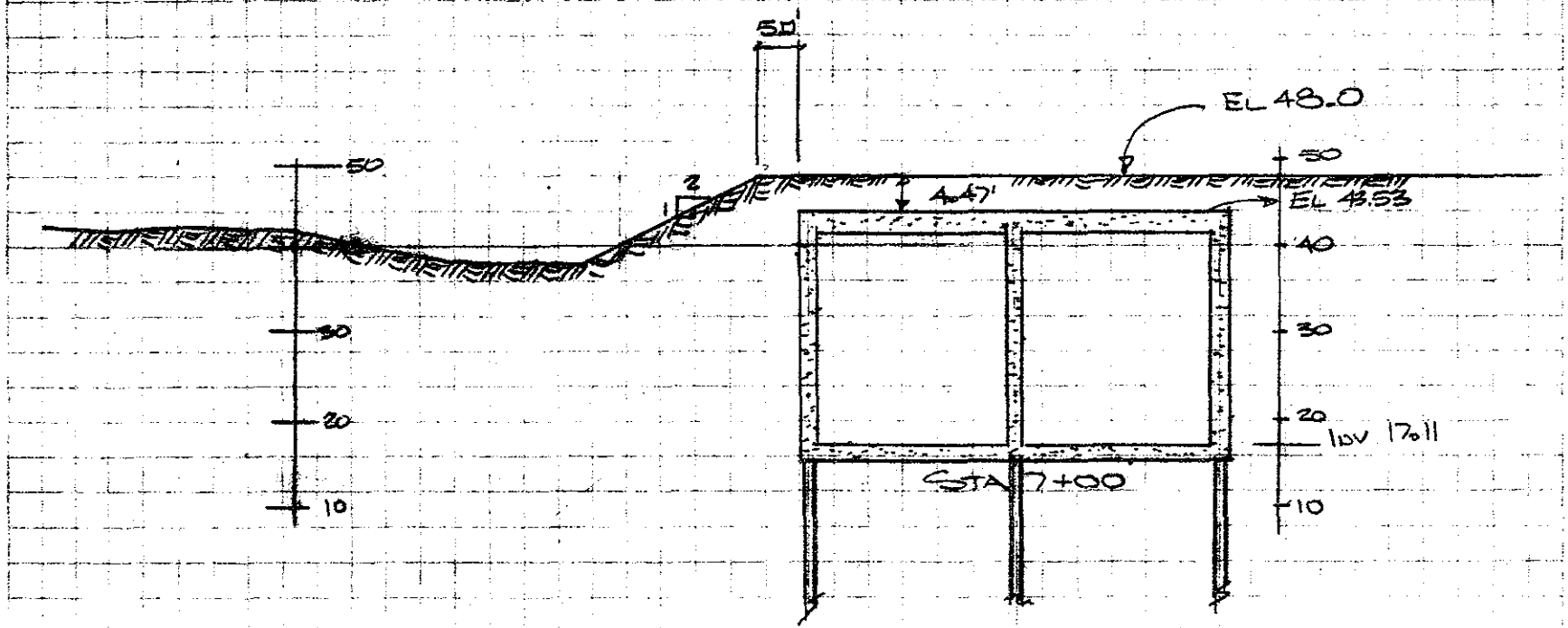


CROSS SECTION STA 7+00

SECTION #7

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT: Park River
COMPUTATION: Member Properties - Section #
COMPUTED BY: W. Rodger CHECKED BY: DATE: 4 Sept 74

$I = \frac{bd^3}{12}$

	Members	Area (A_x)	Mom. Inertia (I_z)
Ext. Walls	1, 9,	$A = 1 \times 3.5 = 3.5$	$I = 3.573\ Ft^4$
	2,3,10,11	$A = 2.0\ Ft^2$	$I = .667\ Ft^4$
	4, 12	$A = 2.0\ Ft^2$	$d = 2.50$ $I = 1.302\ Ft^4$
Int. Walls	5	$A = 7.0$	$I = 28.00\ Ft^4$
	6, 7	$A = 1.667$	$I = .386\ Ft^4$
	8	$A = 7.0$	$I = 28.0\ Ft^4$
Top Slab	13,16 17,20	$A = 1 \times 1.917 = 1.917$	$I = .587\ Ft^4$
	14,15 18,19	$A = 1 \times 1.417 = 1.417$	$I = .237\ Ft^4$
	21,26,27,32	$A = 3.0$	$d = 3.5$ $I = 3.573\ Ft^4$
	22,25,28,31	$A = 2.5$	$I = 1.302\ Ft^4$
	23,24,29,30	$A = 2.0$	$I = .667\ Ft^4$

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION

CORPS OF ENGINEERS, U. S. ARMY

PAGE DM 7-32
6-3

SUBJECT: PARK RIVER

COMPUTATION: NODAL POINT ARRANGEMENT

COMPUTED BY: W. Rodger CHECKED BY: DATE: 4 Sept 74

	NODAL POINT ARRANGEMENT		
	NODE	X-COOR	Y-COOR
110	1	0.00	0.00
	2	0.	3.00
	3	0.	13.36
	4	0.	25.00
	5	0.	26.71
	6	2.00	26.71
	7	11.92	26.71
	8	17.00	26.71
	9	2.00	0.
	10	9.00	0.
	11	12.00	0.
	12	15.00	0.
	13	22.00	0.
110	14	23.83	0.
	15	23.83	3.00
	16	23.83	13.36
	17	23.83	25.00
	18	23.83	26.71
	19	30.66	26.71
	20	35.74	26.71
	21	45.66	26.71
	22	47.66	26.71
	23	25.66	0.
	24	32.66	0.
	25	35.66	0.
	26	38.66	0.
	27	45.66	0.
110	28	47.66	0.
	29	47.66	3.0
	30	47.66	13.36
	31	47.66	25.00

ELEMENT PROPERTIES				
ELT	I	J	A_{AVE}	I_{AVE}
1	1	2	3.5	3.573
2	2	3	2.0	.667
3	3	4	2.0	.667
4	4	5	2.0	1.302
5	14	15	7.0	28.00
6	15	16	1.667	.386
7	16	17	1.667	.386
8	17	18	7.0	28.00
9	28	29	3.5	3.573
10	29	30	2.0	.667
11	30	31	2.0	.667
12	31	22	2.0	1.302
13	5	6	1.917	.587
14	6	7	1.417	.237
15	7	8	1.417	.237
16	8	18	1.917	.587
17	18	19	1.917	.587
18	19	20	1.417	.237
19	20	21	1.417	.237
20	21	22	1.917	.587
21	1	9	3.0	3.573
22	9	10	2.5	1.302
23	10	11	2.0	.667
24	11	12	2.0	.667
25	12	13	2.5	1.302
26	13	14	3.0	3.573
27	14	23	3.0	3.573
28	23	24	2.5	1.302
29	24	25	2.0	.667
30	25	26	2.0	.667
31	26	27	2.5	1.302
32	27	28	3.0	3.573

Section #7

NED FORM 225
27 Sept 49

NEW ENGLAND DIVISION

CORPS OF ENGINEERS, U. S. ARMY

SUBJECT: Park River Local Protection

COMPUTATION: Fill To El 48 - Revise Depth of Overburden

COMPUTED BY: W. Rodger CHECKED BY: DATE: Sept 74

Cover on Section #7 + 25'-0" + Roof Slab

	Station	Inv. El	Roof El	Fill El	Fill Dp	Hyd. Gd
	1+76	15.82	42.24	47.50	5.26'	10
Sect B	2+□□		42.30	47.62	5.32	10
	3+□□		42.55	48.00	5.45	9.07
Laurel St	4+□□		42.79	48.00	5.21	9.4
← →	5+□□	16.62	43.04	48.00	4.96'	9.1
	6+□□		43.28	48.00	4.72	8.80
	7+□□	17.11	43.53	48.00	4.47	8.50
	8+□□		43.78	48.00	4.22	8.20
	9+□□	17.60	44.02	48.00	3.98'	7.90
	10+□□		44.27	48.00	3.73	7.60
*Sect A	11+□□		44.50	48.00	3.50	7.30
	12+□□		44.76	48.00	3.24	7.00
Designed	12+20	18.39	44.81	48.00	3.19'	7.00

Section B

$w = 5.5(0.120) = .600$
LL $= .225$
DL slab $= .213$
1.038

$.478$
$.300$
$.213$
$.991$

Consider Also $1.12 \times \frac{1.038}{.968} = \underline{1.20}$

$v = 40 \times \frac{1.038}{.968} = \underline{43 \text{ Psi}}$

Section A 968 TOTAL Load Used

ok

27 Sept 49

SUBJECT: Park Ridge Profile Section #7

COMPUTATION

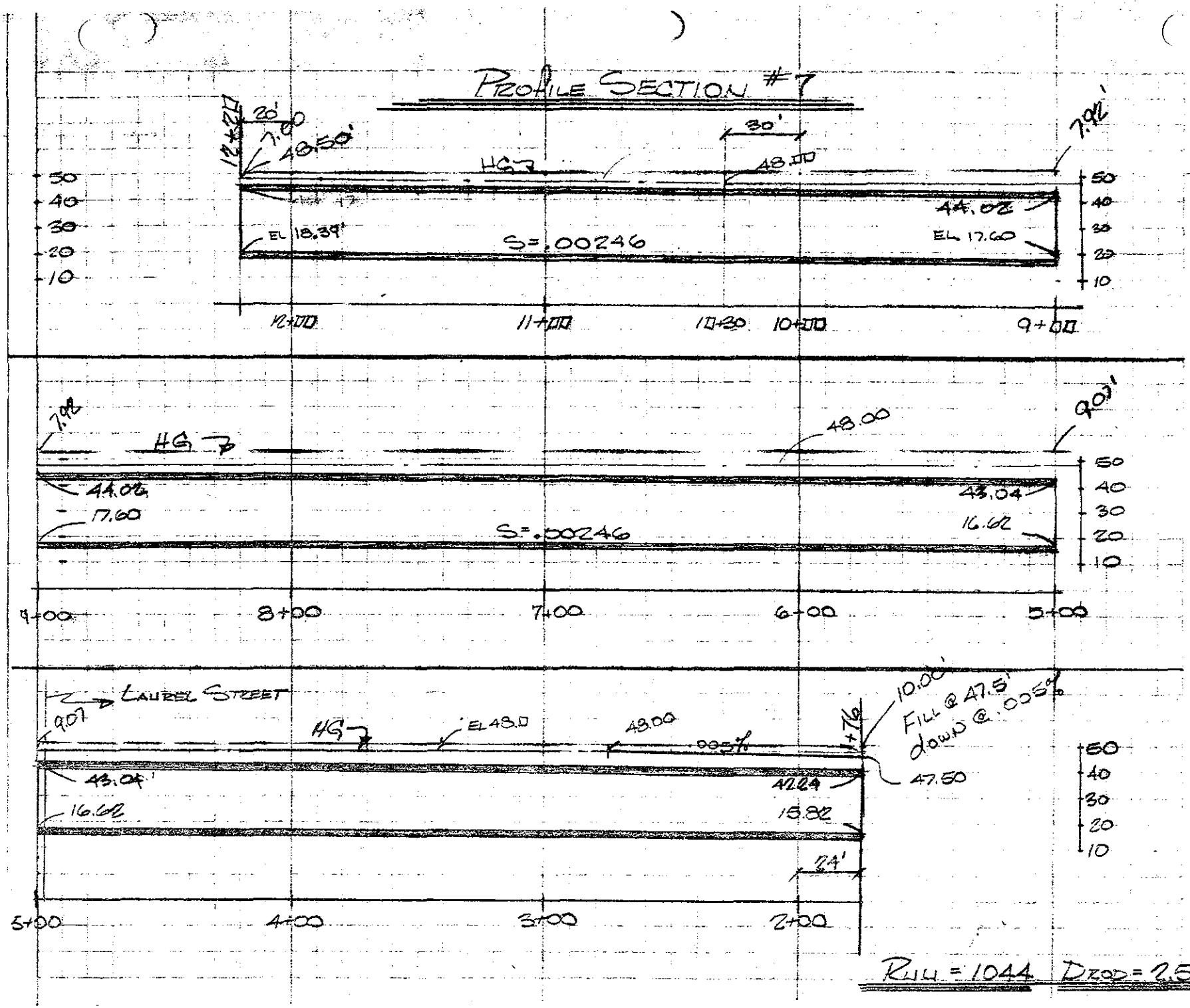
COMPUTED BY: W. Rodger

CHECKED BY

DATE: 3 Sept 74

CORPS OF ENGINEERS, U.S. ARMY

PAGE DM2-34 C-5



SUBJECT: Park River Local Protection 6-6

COMPUTATION: Preliminary Check - Walls & Slabs

COMPUTED BY: W. Rodgers CHECKED BY: DATE: 3 Sept 74

Consider Section @ Sta 1+76

Conc. Slab =	.213
5.26' Fill	.631
LL	.200
	1.044

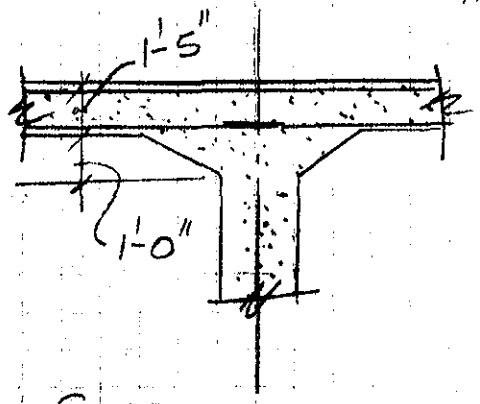
Say 1 k/'

(A) for Conduit Under 10' Head ↑ U = .625 ksf < DL ok

(B) for DL (No Uplift)

$$-M = \frac{wL^2}{9} = \frac{1(24)^2}{9} = 64^{'k} \qquad \frac{wL^2}{8} = 72^{'k}$$

For Roof Slab $A_s(-) = \frac{72}{1.44(24.3)} = 2.06 \text{ in}^2 \approx$ #9@6"



$M(+) = \frac{wL^2}{12} = 48^{'k}$ $d(+) = 12.3$

$A_s(+) = 2.71 \text{ in}^2 \approx$ #10@6 Will Work

For Base Slab

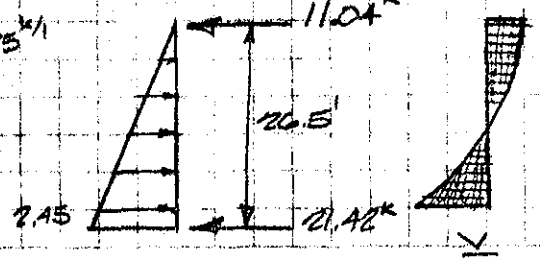
25' Water (+10') = 35'(.0625) = 2.188
Slab = .300
2.488 k/'

$$M(-) = \frac{2.488(24)^2}{9} = 159.2^{'k} \checkmark$$

Assume - 1'-8" Wall (4" Cov)

Ext. Walls

$\gamma = .0625^{k/'}$



$v = \frac{21.42^k}{12 \times 15.3} = 117^{psi}$

* Increase Wall Section To Two feet

$v \approx 96^{psi}$ ok Should Work

SUBJECT Park River 6-7

COMPUTATION Section #7

COMPUTED BY W. Rodger CHECKED BY DATE 4 Aug 74

Section A Station 1+76 to 8+00

Section B Station 8+00 to 12+20

1. (A) Section A Max d of Fill = 3.79' = .455 KSF
→ + .300 KSF LL
.755
+ .213 Slab
.969 KSF

(B) ↑ = 10' head = .625 ↑ ↓ = .669 Sta 1+76

2. (A) Section B Max d of F = 2.50' = .300 KSF
→ .375 KSF LL
.675
.213 Slab
.888

@ Station 12+20

↑ = 7' head = .438 ↓ = .453

Seems oh - there Are A Few Points Where there is a Net ↑ Force, From the Case of Conduit Under Pressure

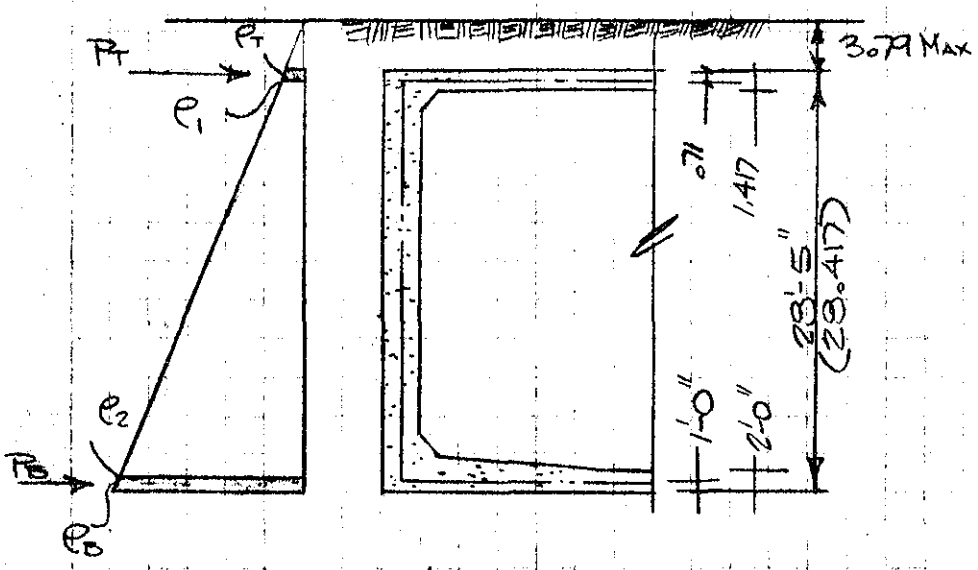
SUBJECT: Park River — 6-8

COMPUTATION: Section #7 - Earth Pressure

COMPUTED BY: W. Rodger CHECKED BY: DATE: 4 Sept 74

Section A

DL slab = 213 psf
TDL = 668 psf
D+L = 968 psf



$\gamma_{BACKFILL} = 120\ \#/FT^3$

$K_o = .50$
$K_a = .33$

NO GROUNDWATER

W / Ka = .330

$\rho_t = .04(4.15) = .166$ $\quad P_T = .118^k$

$\rho_1 = .04(4.50) = .180$

$\rho_2 = .04(31.21) = 1.249$

$\rho_B = .04(31.71) = 1.269$ $\quad P_B = 1.268^{k/1}$

W / Ko = .50

$P_T = .177^k$ ✓

$\rho_1 = .270^{KSF}$ ✓

$\rho_2 = 1.872^{KSF}$ ✓

$P_B = 1.902^k$ ✓

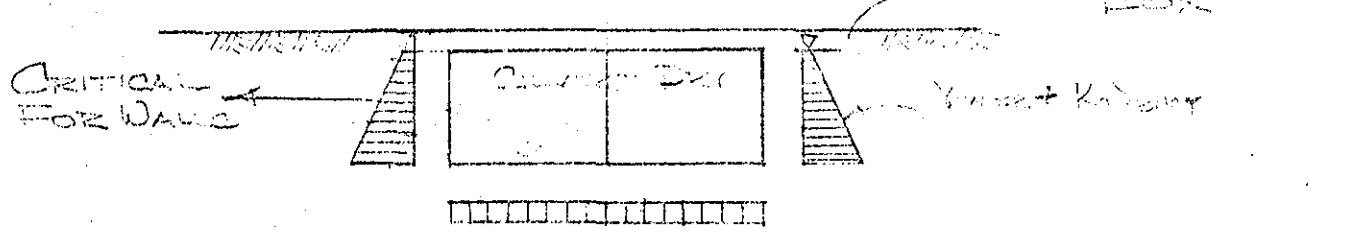
SUBJECT: [illegible]

COMPUTATION: [illegible]

COMPUTED BY _____ CHECKED BY _____ DATE _____

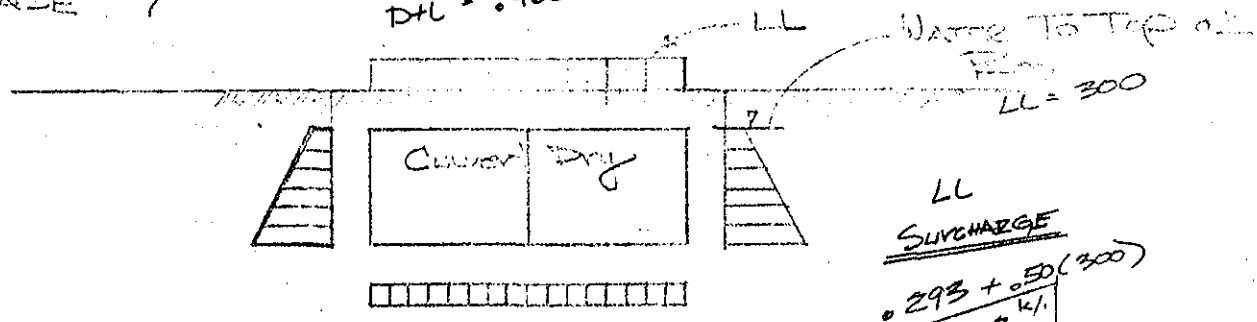
CASE #6

FULL D = .668 K/L



CASE #7

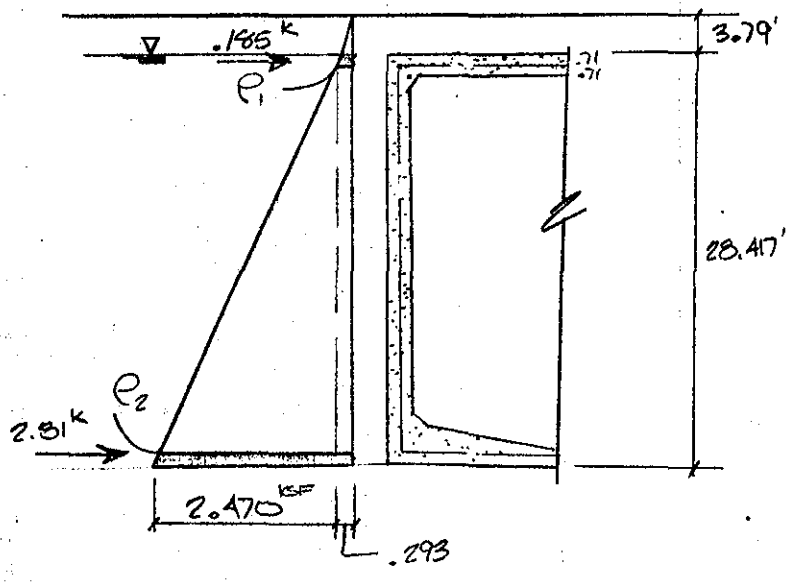
D+L = .968 K/L



LL SURCHARGE

$.293 + .50(300)$

$= .443^{K/L}$



$P = 3.79(.06) = .227$

$P_I = .227 + .71(.0925) = .293$

$P_T = .185^K$

$P_2 = .227 + 27.417(.0925)$

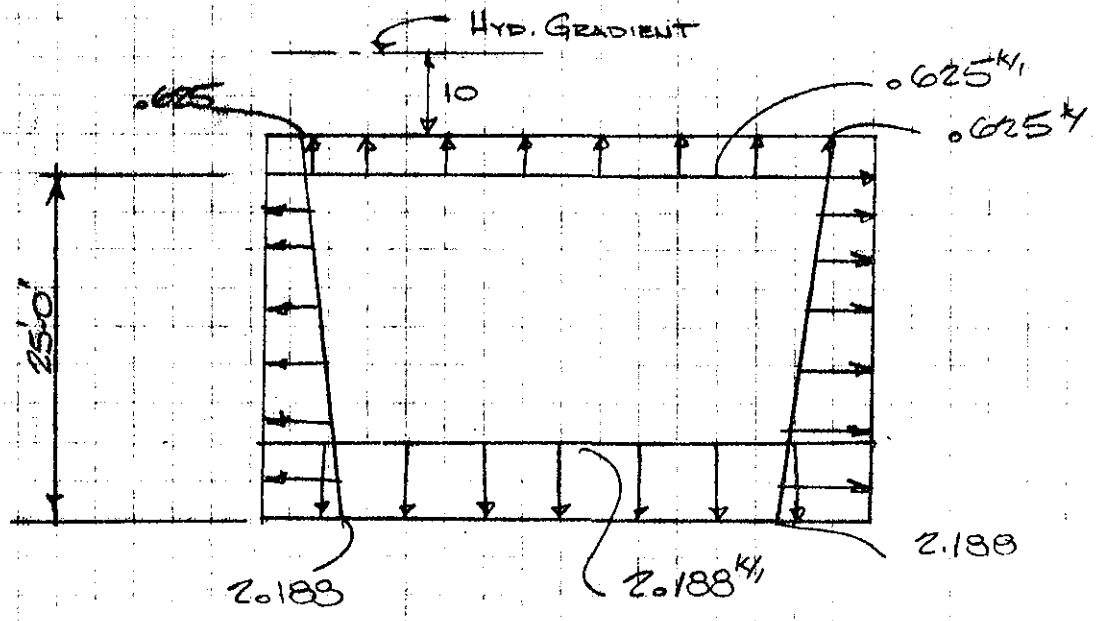
$= .227 + 2.536 = 2.763$

$P_B = 2.809$

$P_B = 2.809^K$

SECTION 7

INTERNAL PRESSURE



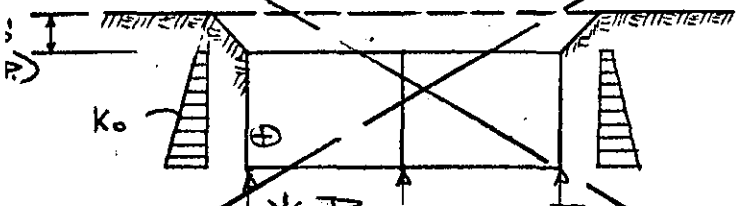
NET FORCE = WEIGHT of WATER ↓ = 25'(.0625) ok ✓

SUBJECT: PARK RIVER LOCAL PROTECTION 6-11A

COMPUTATION: SECTION #7 (PILE SUPPORTED) GEN. LOAD CASES

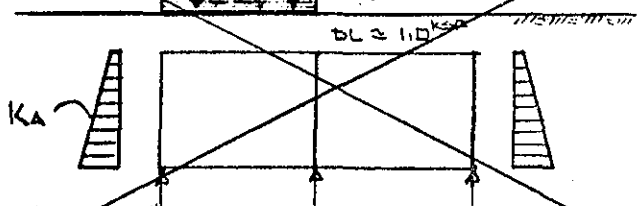
COMPUTED BY: W. Rodger CHECKED BY: DATE: Sept 74

CASE #1



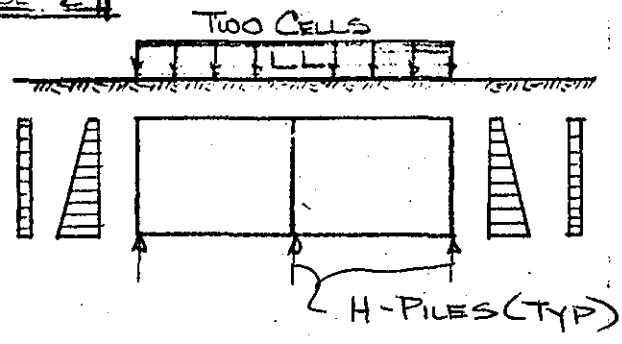
* PERFORMED BUT NOT CRITICAL

CASE #VOID (RESULTS WILL BE SIMILAR TO CASE #3)



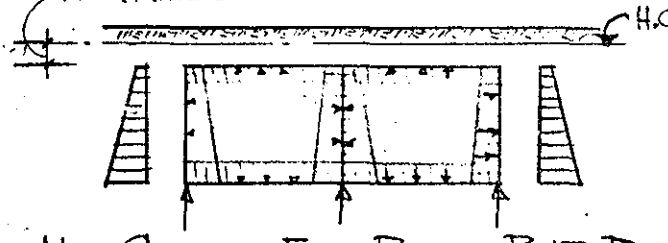
CASE #3 WILL BE SUFFICIENT TO DETERM. DESIGN MOM & SHEARS

CASE #2



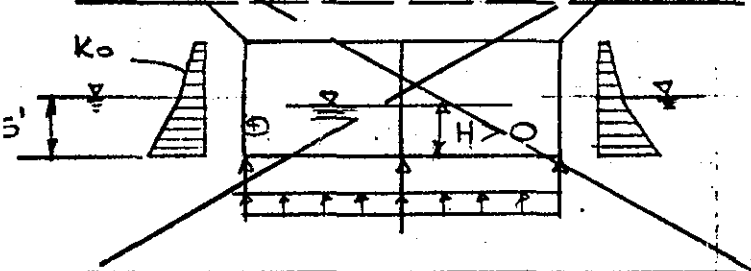
CASE #3 - BURST 6' OF HEAD

H VARIES 7'-10'

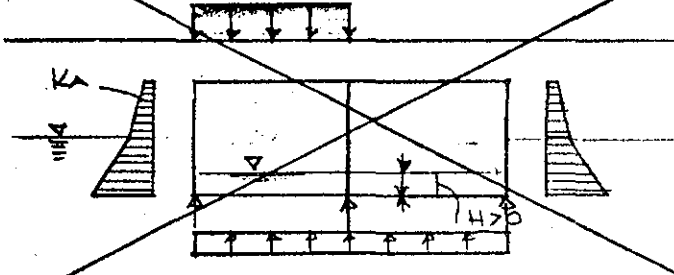


NOT CRITICAL FOR BOX - BUT DOES HAVE HIGHEST BASE PRESSURES

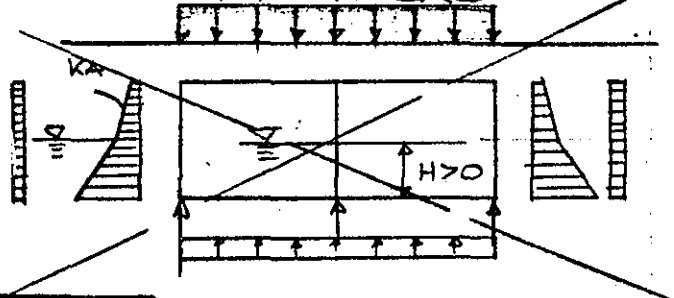
CASE #4 * PERFORMED BUT NOT CRITICAL



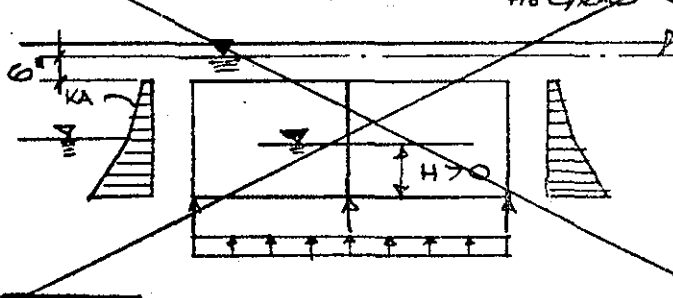
CASE #VOID



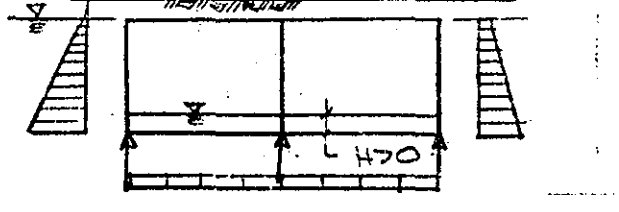
CASE #5 * PERFORMED BUT NOT CRITICAL



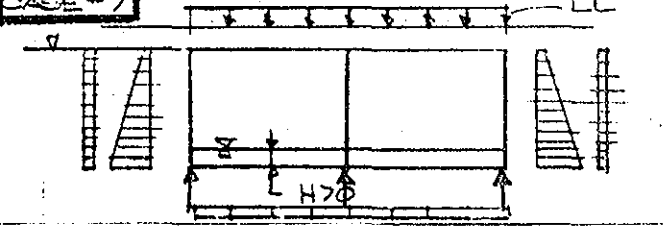
CASE #VOID



CASE #6



CASE #7



NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

PAGE DM7-41
6-11D

SUBJECT Park River Local Protection
COMPUTATION Typ. Computer Output, Section #7, LC-2,3,6,7
COMPUTED BY W. Rodger CHECKED BY DATE Sept 74

NODAL POINT DISPLACEMENTS LC-3

NODE	X-DISPL	Y-DISPL	ROT-DISPL
1	0.	0.	-.32382E-03
2	.78647E-03	-.10694E-04	-.20983E-03
3	-.93554E-03	-.46052E-04	.21770E-03
4	-.52938E-03	-.48826E-04	-.23359E-03
5	-.12973E-03	-.48834E-04	-.23141E-03
6	-.12082E-03	-.48194E-03	-.20178E-03
7	-.61019E-04	-.93145E-03	.74140E-04
8	-.30396E-04	-.46945E-03	.37403E-04
9	0.	-.72533E-03	-.39310E-03
10	0.	-.35904E-02	-.27751E-03
11	0.	-.38217E-02	.12475E-03
12	0.	-.29243E-02	.44409E-03
13	0.	-.12720E-03	.13012E-03
14	0.	0.	.20993E-09
15	-.59754E-09	-.62097E-05	.18917E-09
16	.14297E-07	-.66979E-04	-.24086E-03
17	.36798E-07	-.98300E-04	-.63032E-09
18	.37863E-07	-.99206E-04	-.62096E-09
19	.30472E-04	-.46946E-03	-.37403E-04
20	.61095E-04	-.93145E-03	-.74159E-04
21	.12090E-03	-.48194E-03	.20177E-03
22	.12981E-03	-.48834E-04	.23141E-03
23	0.	-.12720E-03	-.13012E-03
24	0.	-.29243E-02	-.44409E-03
25	0.	-.38217E-02	-.12475E-03
26	0.	-.35904E-02	.27751E-03
27	0.	-.72532E-03	.39310E-03
28	0.	0.	.32381E-03
29	-.78646E-03	-.10694E-04	.20988E-03
30	.93658E-03	-.46052E-04	-.21771E-03
31	.52995E-03	-.48326E-04	.23358E-03

END ACTIONS WITH ELEMENT LOADS

	ELEMENT		AXIAL I AXIAL J	SHEAR I SHEAR J	MOMENT I MOMENT J
LEFT WALL	1	LC-3	6.362	-13.681	-93.764
			-6.362	10.944	56.369
	2		3.754	-10.944	-56.369
			-3.754	2.776	-12.484
	3		.262	-2.776	12.484
			-.262	-4.023	-2.762
	4		.005	4.023	2.762
			-.005	-4.815	4.803
Center WALL	5		7.969	.000	.000
			-7.969	-.000	-.000
	6		5.378	.000	.000
			-5.378	-.000	-.000
	7		2.467	.000	.000
			-2.467	-.000	.000
	8		2.039	.000	-.000
			-2.039	-.000	.000
RIGHT WALL	9		6.362	13.681	93.764
			-6.362	-10.944	-56.369
	10		3.754	10.944	56.369
			-3.754	-2.776	12.484
	11		.262	2.776	-12.484
			-.262	4.023	2.762
	12		.005	-4.023	-2.762
			-.005	4.815	-4.807
Roof SLAB	13		-4.698	.005	-4.803
			4.698	.031	4.732
	14		-4.698	-.031	-4.732
			4.698	.507	1.814
	15		-4.698	-.507	-1.814
			4.698	.726	-1.318
	16		-4.698	-.726	1.318
			4.698	1.020	-7.279
	17		-4.698	1.020	7.279
			4.698	-.726	-1.318
	18		-4.698	.726	1.318
			4.698	-.507	1.814
	19		-4.698	.507	-1.814
			4.698	-.031	4.732
	20		-4.698	.031	-4.732
			4.698	.005	4.807
BASE SLAB	21, 32		0.	27.439	93.764
			0.	-22.113	-44.179
	22, 31		0.	22.113	44.179
			0.	-4.034	46.795
	23, 30		0.	4.034	-46.795
			0.	3.330	47.852
	24, 29		0.	-3.330	-47.852
			0.	10.344	26.517
	25, 28		0.	-10.344	-26.517
			0.	23.334	-111.803
	26, 27		0.	-23.334	111.803
			0.	32.786	~~169.151~~

SUBJECT: Park River Local Protection

COMPUTATION: Pile Loads / Foot

COMPUTED BY: W. Rodger CHECKED BY: DATE: 12 Sept 74

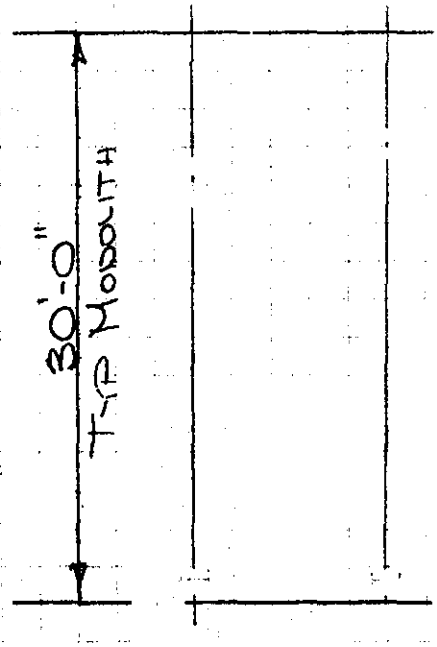
Load Case	Ext Walls	Int. Walls
#2 Sat Earth D+L	$18.207^k + .478^k$ 18.69^k	$29.365^k + 8.677$ 38.04^k
#3 Conduit Full	$6.862^k + 27.489^k$ 34.35^k	$7.969 + 33.806$ 41.78^k
#5 Uplift	5.09^k	19.22

Size of Pile $= 100^T = 200^k$

Prelim. Spacing Ext Walls $= 200/34.35 = 5.82'$ $5'-9"$

Int Walls $200/41.78 = 4.79$ $4'-8"$

Also 41.78^k DO/LL $+ .625 \times 49.67 \times .300 = 51.09$



Spacing $= \frac{200}{51.09} = 3.915'$ $3'-11"$

Say $3'-10"$

$\frac{1044}{5.75} = 181.565$ 181 equal

$\frac{1044}{3.833} = 272.372$

HP 14 × 73 OR
HP 12 × 74

27 Sept 49

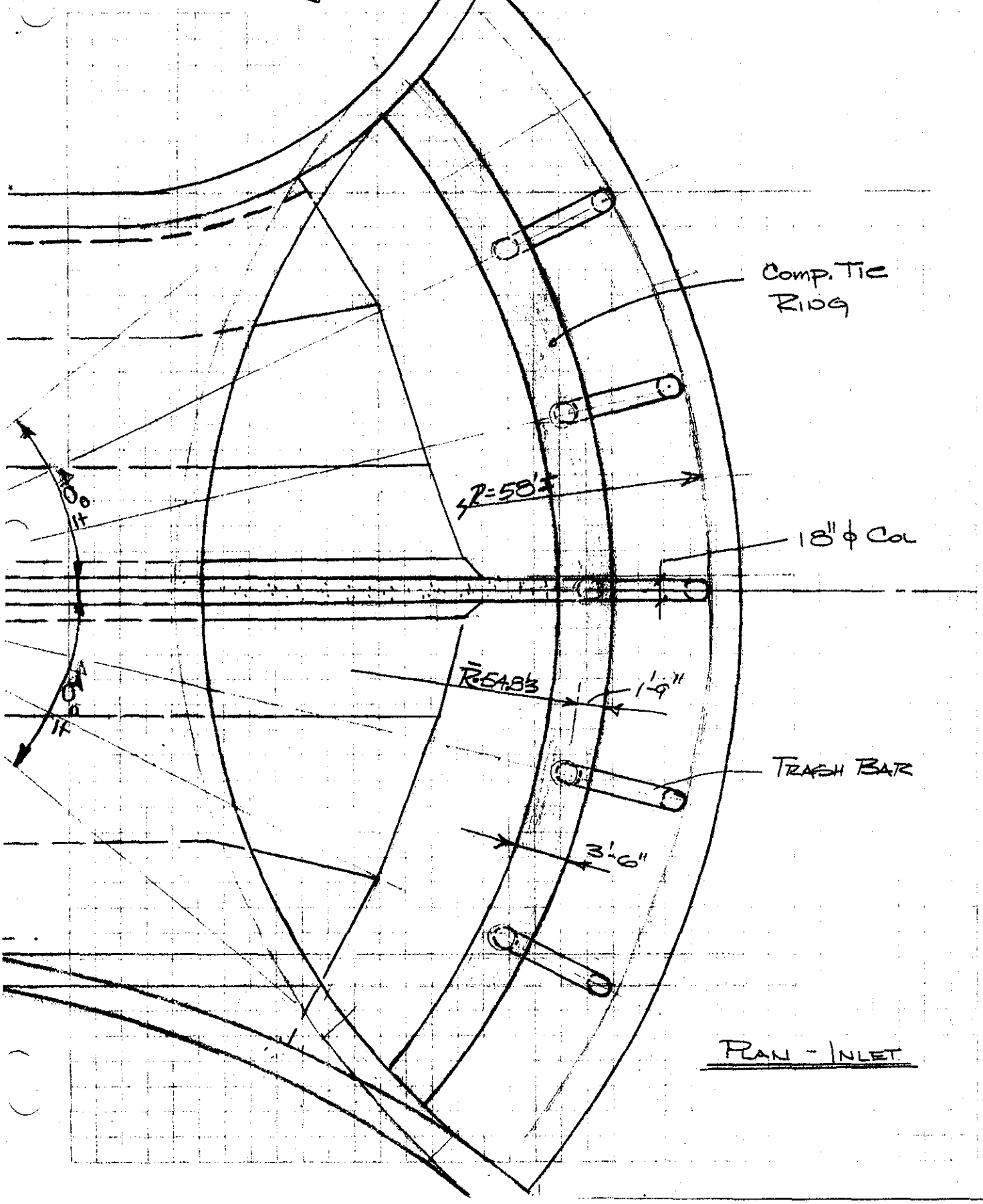
CORPS OF ENGINEERS, U. S. ARMY

PAGE UM1-74

SUBJECT PARK RIVER

COMPUTATION INLET STRUCTURE

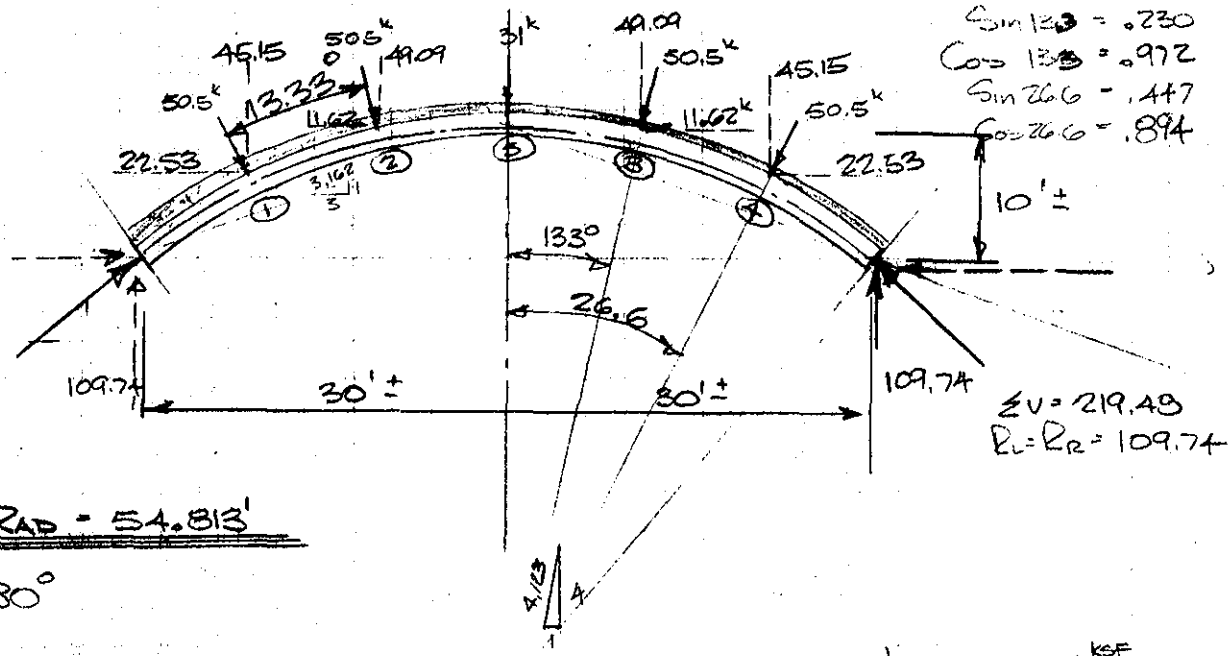
COMPUTED BY W. Rodger CHECKED BY DATE 16 Sept 74



SUBJECT: PARK RIVER LOCAL PROTECTION — I-2

COMPUTATION: ENTRANCE STRUCTURE

COMPUTED BY: W. Rodger — CHECKED BY: — DATE: 13 Sept 74



AVE RAD = 54.813'

≈ 80°

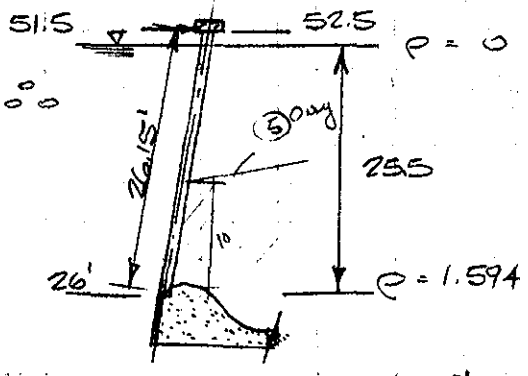
$\bar{C} = 2\pi R \cdot \frac{80}{360}$

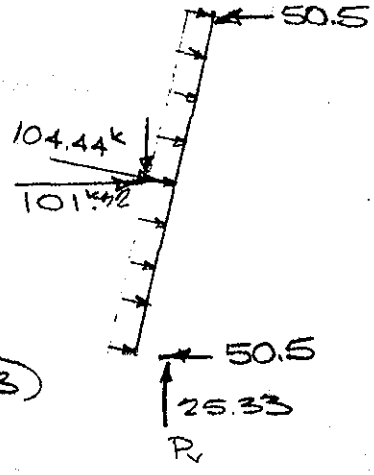
$= 76.53^{FT}$

$\bar{C} = 2\pi R \cdot \frac{13.33}{360}$

$= 12.76^{FT}$

5' Head = .313 KSF





$P_{1,2,3,4} = \frac{1}{2}(25.50')(12.76')(.313)$

$= 50.5$

$P_V = \frac{1}{4.123}(104.44) = 25.33$

$P_S = \frac{1}{2}(15.5)(12.76)(.313)$

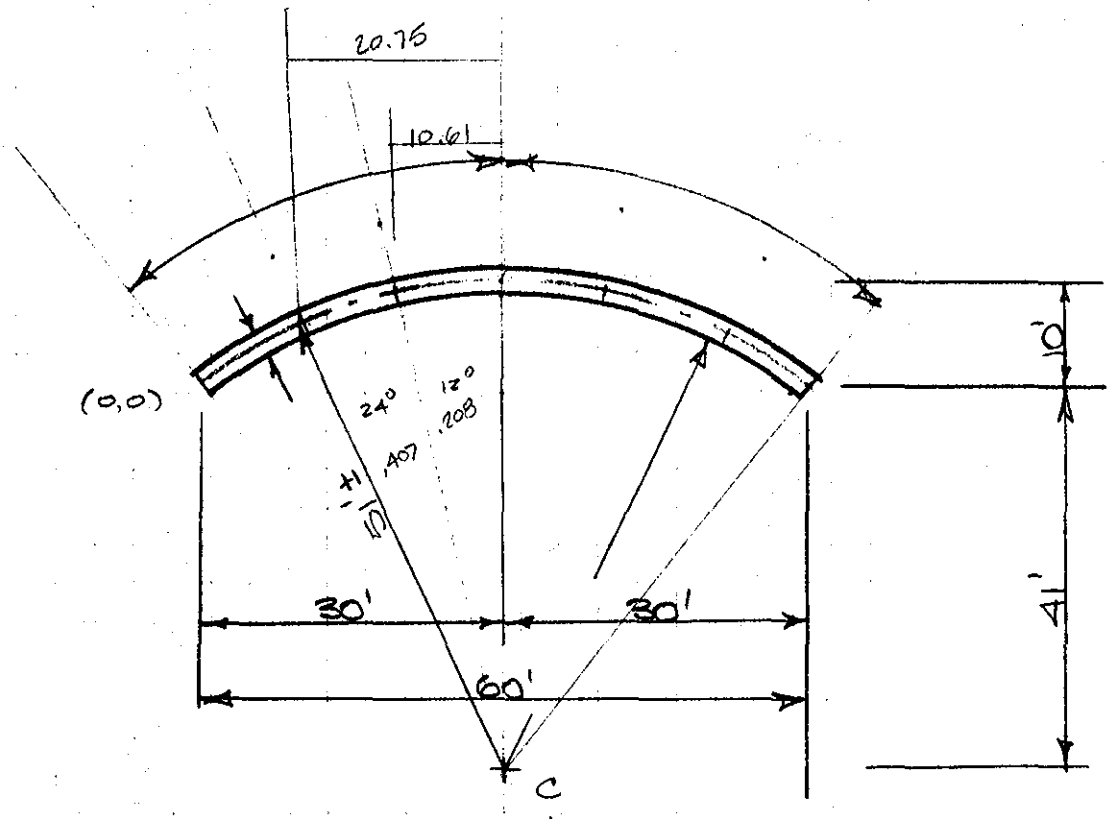
$= 31^k \pm$

Comp = 2 × Σ Sin 50.5 + 31

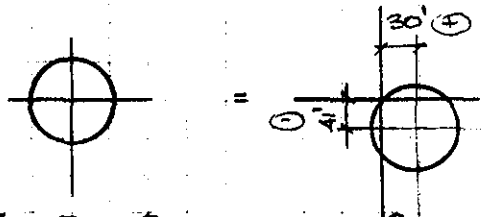
SUBJECT: PARK RIVER — I-3

COMPUTATION: GEOMETRY of Circle

COMPUTED BY: W. Rodger CHECKED BY: DATE: 16 Sept 74



TRANSFERRED AXIS



$x^2 + y^2 = R^2$ $\quad (x-30)^2 + (y+41)^2 = R^2$

Where $R = 51'$ $\quad$ @ $x = 0$ $\quad (-30)^2 + (y+41)^2 = 2601$

$y + 41 = (1701)^{1/2} = 41.24'$

$y = .24'$

x	y
0	.24' ≈ 0
5	3.45
10	5.91
15	7.74
20	9.00
25	9.75
30	10.00

27 Sept 49

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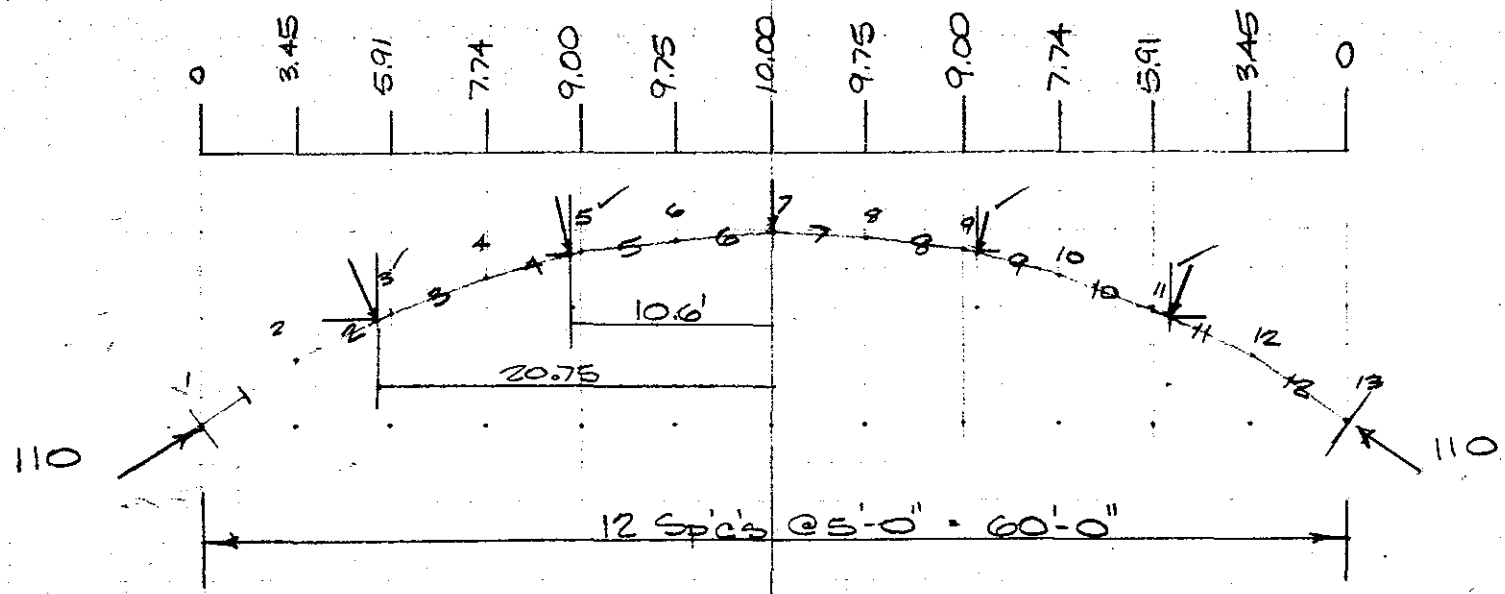
SUBJECT Park River

COMPUTATION Compressed Ribs

COMPUTED BY W. Rodger CHECKED BY DATE 16 Sept 74

PAGE DM7-4-1 I-4

$\sin 12^\circ = .208$
$\cos 12^\circ = .978$
$\sin 24^\circ = .407$
$\cos 24^\circ = .914$



for Convenience Apply Loads @ Nodal Points 3, 5, 9 & 11

Points, 5 & 9 $P_T = 50.5^k$ $P_y^{12^\circ} = 49.4^k$ $P_x = 10.5^k$

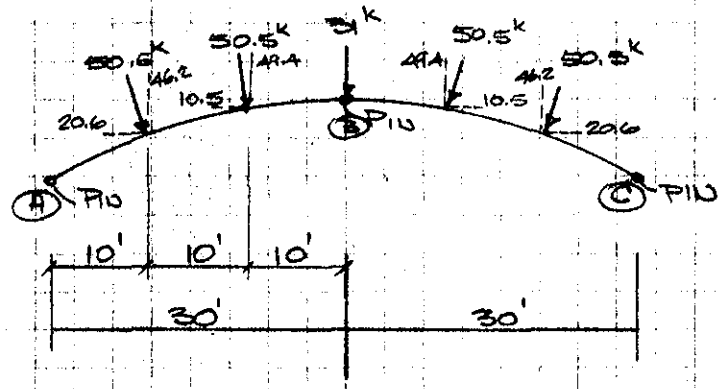
Points 3 & 11 $P_T = 50.5^k$ $P_y^{24^\circ} = 46.2^k$ $P_x = 20.6^k$

SUBJECT: PARK RIVER LOCAL PROT.

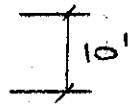
COMPUTATION: 3-HINGED ARCH

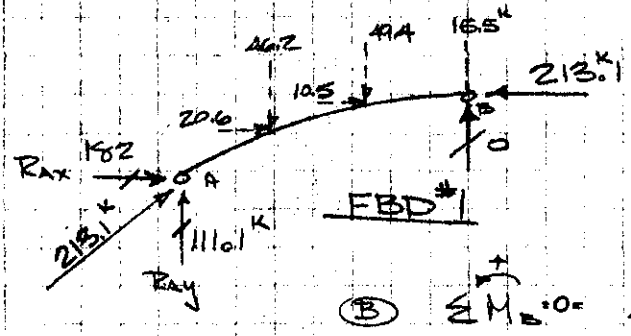
COMPUTED BY: W. Rodger CHECKED BY: DATE: 17 Sept 74

CONSIDER THREE HINGED ARCH



LC-1 TOTAL LOAD





FBD #1

(A) Due To Symmetry $R_{Ay} = R_{Cy}$

$\Sigma F_y = 222.2$

$\therefore R_{Ay} = R_{Cy} = 111.01^k$

(B) $\Sigma M_B = 0 =$

15.5×0	=	0
$49.4^k \times 10'$	=	494
$46.2^k \times 20$	=	924
$10.5^k \times 1'$	=	10.5
$20.6^k \times 4.09'$	=	84.3
-111.01×30	=	-3333
		-1820.20

$\text{THRUST} = (111.01^2 + 182^2)^{1/2}$
$= 213.10^k$

$\therefore R_{Ax} = R_{Cx} = \dfrac{1820.20^{ik}}{10'} = 182^k$

$F_{By} = 0$, F_{bx} from $\Sigma H = 182^k + 20.6 + 10.5 = 213.1^k$

- Comp INSIDE F.
+ Comp OUTSIDE F

$M_A = M_B = M_C = 0$

$M @ Pt_2 = 111.01(5) - 182(3.45) = -72.40^{ik}$

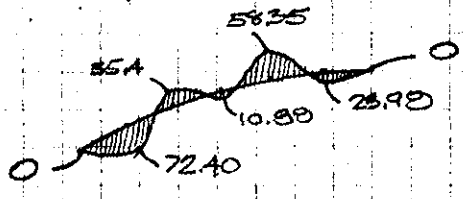
* $M @ Pt_3 = +111.01(10') - 182(5.91) = 35.38^{ik}$

$M @ Pt_4 = +111.01(15') - 182(7.74) - 46.2(5) - 20.6(1.83) - 10.88$

$M @ Pt_5 = +111.01(20) - 182(9.00) - 46.2(10) - 20.6(3.09) = 58.35^{ik}$

$M @ Pt_6 = 111.01(25) - 182(9.75) - 46.2(15) - 20.6(3.84) - 49.4(5) - 10.5(.75) = -23.98$

$M @ Pt_7 = 111.01(30) - 182(10) - 46.2(20) - 20.6(4.09) - 49.4(10) - 10.5(1) = 0^{ik}$

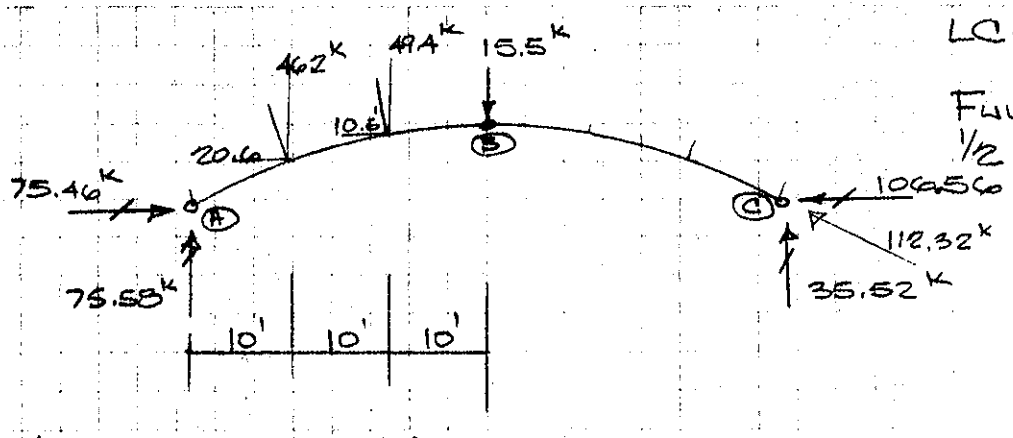


MOM. DIAG

SUBJECT: PARK RIVER LOCAL PROTECTION
COMPUTATION: 3 HINGED ARCH
COMPUTED BY: W. Rodger CHECKED BY: DATE: 17 Sept 74

LC-2

FULL LOAD ON 1/2 of ARCH



$\Sigma M_A = 0$

46.2(10)	=	462
49.4(20)	=	988
15.5(30)	=	465
20.6(59.1)	=	121.75
10.5(9.00)	=	94.50
Rcy (60)	=	-60 Rcy

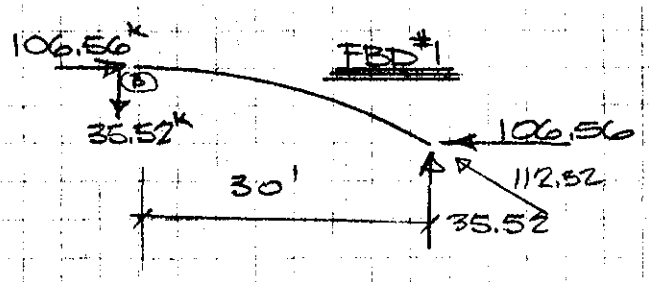
$R_{cy} = \frac{2131.25}{60} = \underline{35.52^k}$

$\Sigma F_y \quad R_{ay} = \underline{75.58^k}$

CHECK

15.5(30)	=	465
49.4(40)	=	1976
46.2(50	=	2310
-10.5(9)	=	-94.50
-20.6(59.1)	=	-121.75
- Ray (60)		

$R_{ay} = \frac{4534.75}{60} = 75.58^k$ ok



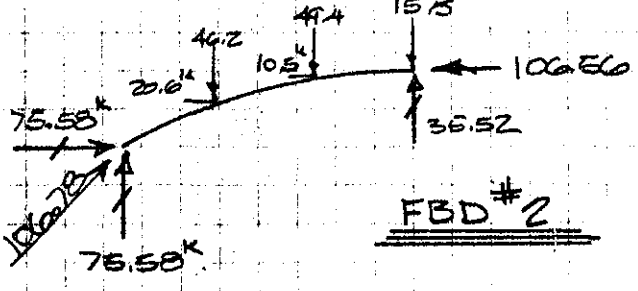
FBD #1

$\Sigma M_B = 0$

$35.52(30) - 10(R_{cx}) = 0$

$R_{cx} = 106.56$

From Horizontal $R_{ax} = 106.56 - 20.6 - 10.5 = 75.46^k$



FBD #2

SUBJECT: PARK RIVER I-9

COMPUTATION: CONSIDER MOMENT UNBALANCED CASE

COMPUTED BY: W. Rodger CHECKED BY: DATE: 17 Sept 74

DETERMINE MOMENTS

M @ Pt 2 = 75.58(5) − 75.46(3.45) = 117.6 ^k

" " Pt 3 = 75.58(10) − 75.46(5.91) = 309.8 ^k

" " Pt 4 = 75.58(15) − 75.46(7.74) − 46.2(5) − 20.6(1.83) = 280.94 ^k

" " Pt 5 = 75.58(20) − 75.46(9.00) − 46.2(10) − 20.6(3.09) = 306.81 ^k

" " Pt 6 = 75.58(25) − 75.46(9.75) − 46.2(15) − 20.6(3.84) − 49.4(5) − 10.5(.75) = 126.79

" " Pt 7 = 75.58(30) − 75.46(10) − 46.2(20) − 20.6(4.09) − 49.4(10) − 10.5(1) = .046 ≈ 0 ^k ok

M @ Pt 13 = 0

Pt 12 = 35.52(5) − 106.56(3.45) = −190.032

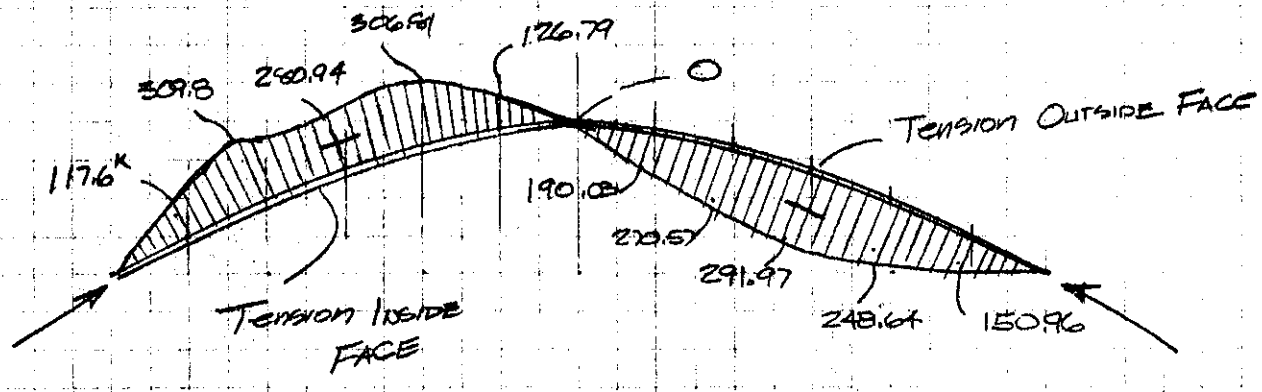
Pt 11 = 35.52(10) − 106.56(5.91) = −270.569

Pt 10 = 35.52(15) − 106.56(7.74) = −291.974

Pt 9 = 35.52(20) − 106.56(9.00) = −248.640

Pt 8 = 35.52(25) − 106.56(9.75) = −150.96

Pt 7 = 35.52(30) − 106.56(10.0) = 0.00



SUBJECT: Park River

COMPUTATION: Relief For Ring

COMPUTED BY: W. Rodger CHECKED BY: DATE: 19 Sept 74

I-10

Case 1 M = 72.4'k N = 213.1^k

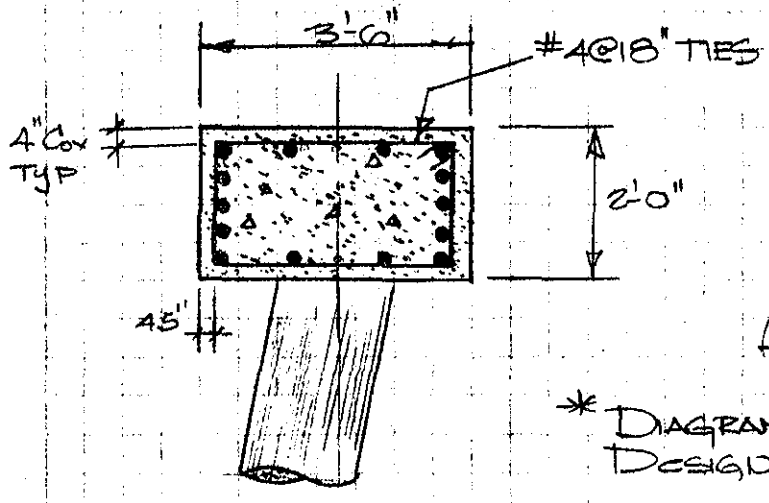
Case 2 M = 307'k N = 106.78^k

for Arch

$f'_c = 4000$

$f_c = .35 f'_c = 1400$ PSI

$f_s = 20{,}000$



b = 24" 4" Cover

D = 42"

gt = 42" - 9" = 33"

$g = \frac{33}{42} = .79$ Say .80

$A_g = 24(42) = 1008 \text{ in}^2$

* Diagram 35b, Pg 144, "Working Stress Design Handbook"

Case #1 $\frac{N}{f'_c A_g} = \frac{213.1^k}{4(1008)} = .053$ Compression Controls

$f_y = 40$, $f'_c = 4$

$g = .80$

$\frac{12M}{f'_c t A_g} = .005$

* $\rho_g = .001^{\pm}$ $\rho_{min} = .01$

$A_{st} = .01(1008) = 10 \text{ in}^2$

$S_{p'c'g} = 24" - 8" = 16"$ b # o o d

#10 = 1.27" ϕ CLEAR Spc = $\frac{16" - 5(1.27)}{4}$ = 2.41" Clear

Case II $\frac{N}{f'_c A_g} = .0265$ * $\rho = .01$ 10 in^2

$\frac{12M}{f'_c t A_g} = .022$

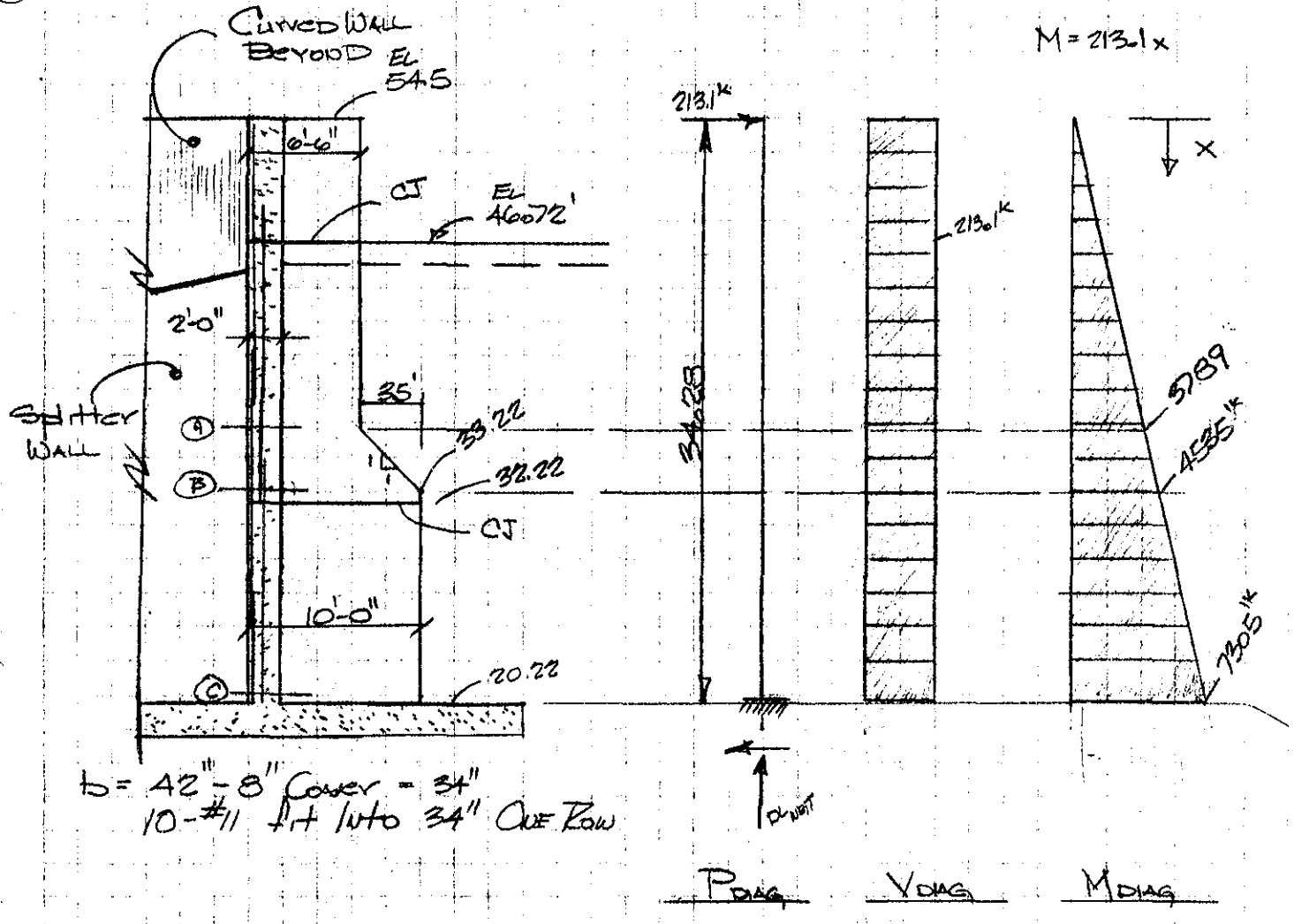
Ties = 48 × .5 = 24"
16 × 1.26 = 20" Use 18"
least Dim = 24"

Use 5-#9 Each Face Long Dir
Add 2-Add'l #9 in Web

SUBJECT: Park River

COMPUTATION: Buttress Design

COMPUTED BY: W. Rodger CHECKED BY: DATE: 19 Sept 74



b = 42" − 8" Cover = 34"
10-#11 fit into 34" One Row

1-Row Steel

(A) Check Shear 6'-6" Section $\bar{d} = 73"$ $D = 78"$

$$v = \frac{213.1^K}{bd} = \frac{213.1}{(3.5\times12)(.78')} = 69.5^{PSI} < 70 \text{ ok}$$

$M_{MAX} = 3789$ $A_{s(A)} = \frac{3789}{1.44\times73"} = 36 \text{ in}^2$ 23 Say 24"

$A_{s(B)} = \frac{4535}{1.44\times108} = 29.2 \text{ in}^2$

2 row $d = 120" - 12 + 108$ $A_{s(C)} = \frac{7305}{1.44\times108} = 46.97 \text{ in}^2 \approx 30-\#11$ = three layers

27 Sept 49

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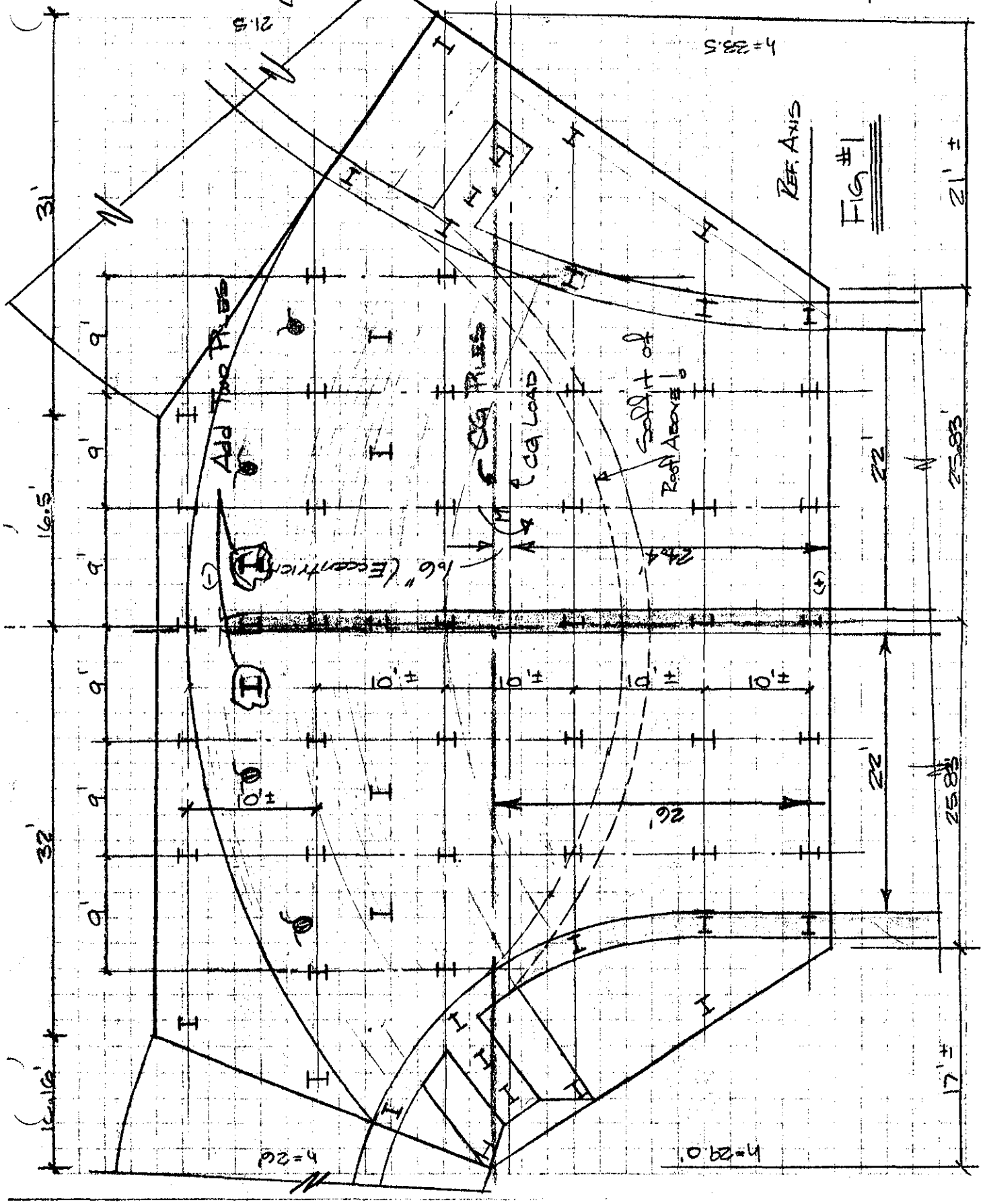
PAGE DM7-53

SUBJECT: PARKS RIVER — F-1

COMPUTATION: FOUNDATION PLAN - INLET - Preliminary

COMPUTED BY: W. Rodger CHECKED BY: DATE: 23 Sept 74

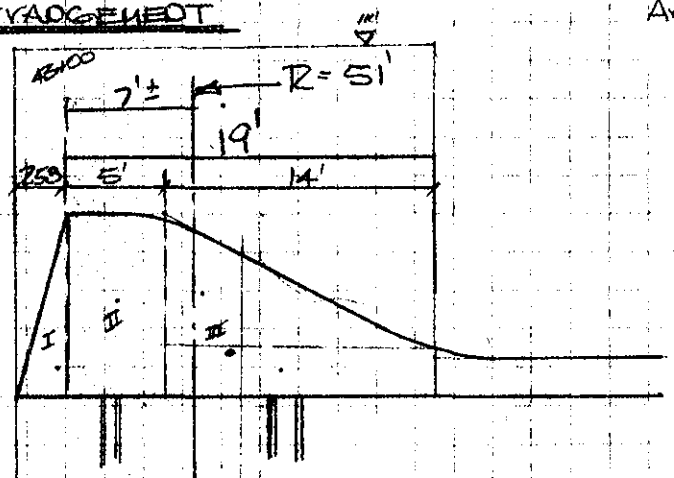


SUBJECT: Park River F-2

COMPUTATION: Piles @ Weir - Inlet

COMPUTED BY: W. Rodger CHECKED BY: DATE: 18 Sept 74

Pile Arrangement



$$\text{Area} = \frac{9.33' \times 2.58}{2} + \frac{(9.33 + 2.5)}{2}(14) + 5(9.33)$$

$$12.04 + 82.81 + 46.65 = 141.50^{\square}$$

A	x	$\bar{y}$	Ay
12.04	×	1.72'	20.71
46.65	×	5.08	236.98
82.81		13.41 ±	1110.48
141.50			1368.17

$$\bar{y} = \frac{1368.17}{141.50} = 9.67'$$

$$L_{ARC} = 2\pi R \times \frac{72^\circ}{360} = 2 \times 3.1416 \times \frac{72}{360} \times 51' = 64.1'$$

$$\text{Weight of Weir} = 64.1 \times 141.50 \times .150 = 1360.53^K - 1360^K$$

$$\text{Height Design Pool} = \text{El } 51.5' + 20.2 = 31.3'$$

$$\text{Area}_{\square} = 31.3 \times 21.58 - 141.50 = 534^{\square'}$$

$$L_{ARC} = 2 \times 3.1416 \times 49.79' \times \frac{72}{360} = 62.57'$$

$$W_{HCO} = 62.57' \times 534^{\square} \times .0625^{\#/\square'} = 2088^K$$

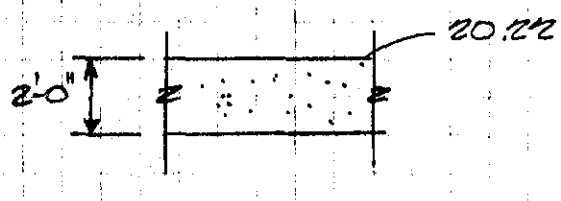
$$\begin{array}{r} 2088 \\ \hline 3448^K \\ 1724^T \end{array}$$

$$1724^T / 100^{T/PILE} = 17.24 \text{ Piles Along Arc of Weir}$$

Say 18 Piles

$$\text{℄ Total Load} \downarrow = \frac{51 + 49.79}{2} = 50.395' \quad 50'\text{-}3''$$

Area Mat = 89.66' × 55' − ½(10.16)(26) − ½(29)(17) − ½(31)(21.5) − ½(21)(33.5)
= 3867.72 ft²



Walls

44' − 2'-0" Wall × 27' ± (Inc Portion Slab) = 356.4^k
45' − 2'-0" Wall × 27' ± " " = 364.5^k
52' − 1'-8" Wall × 25' ± = 325.7^k
1046.4^k Walls

Slab + Weir

$A_{weir} = \pi(R_o^2 - R_i^2) \times \frac{72°}{360}$

$= \pi(60.58^2 - 39^2) \times \frac{72°}{360} = 430 \pm ft^2$

Net Area$_{slab}$ = 3868 ± − 430 = 3438

Weight$_s$ = 3438 ft × 2' × .150 = 1031.40^k
" w = 1360.00^k
2391.40^k

Water To El 51.5'

Over Weir = 62.57' × 534# × .0625 = 2088^k

± = 32 × 44 = 1408 + 400 outer Portion = 1800# ±

1800# × 31.3 ft × .0625 = 3521.^k
5609^k Water

Buttresses (Not Inc Wall Inc)

2 × [8' × 3.5' × 12' × 0.150 + 6' × 4.5' × 3.5 × .150 − 4.5 × 3.5 × 17.78 × .150]
= 213.16^k

Soil On Sides 32 × 8 + ½(37)(13) = 497# × 26.48 × .120
= 1579.^k

Roof Slab = 20 × 50 × 1.5 × .150 = 225^k
Parapet 100' ± × 2 × 7 × .150 210^k
435^k

Walls	-	1047
Para. & Slab	-	435
Base + Weir	-	2392
Buttresses	-	214
Water	-	5609
Soil	-	1579
		11276^k

$\frac{11276^k}{200^k/\text{pile}} = 56.38^k$

Preliminary Plan Has 63 Piles Assumed ok

(A) Consider Pile in Transition Portion of Inlet (in Slab)

Area ≈ 10×9 = 90$^{\square}$

Concrete = 90×2×.150 = 27^k
Water = 90×25×.0625 = 140.6^k
167.6^k < 200^k ok

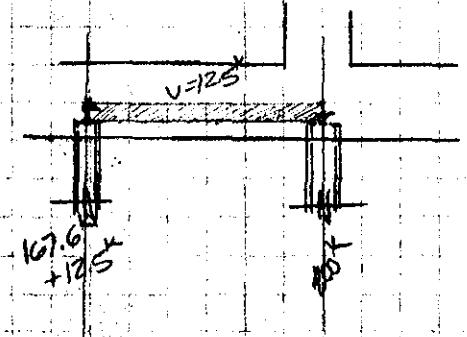
(B) Under Wall - Center 9×10 = 90 - 16.7 = 73.3

Concrete$_{BS}$ = 90×2×.150 = 27^k
Concrete$_{BS}$ = 90×1.5×.150 = 20.3
Concrete$_W$ = 1.67×25×10×.150 = 62.6
Water = 73.3×25×.0625 = 114.5^k
224.40^k (Not inc Live Load)

!! But Live Load is Not Considered To Act At the Same Time As Max Flood !!

We Have An Excess Load In Piles Under Center Wall But Pile Under Chandel Have < 200^k. Consider Transferring 25^k/2 sides = 12.5^k Constant Shear /10' Strip

M = 12.5×10' = 125$^{'k}$



Base Slab Mapol
125$^{'k}$
d = 24 - 8.2 = 15.8

$A_s = \frac{125^{'k} \cdot \frac{1}{10}}{1.44 \times 15.8} = .55^{in^2}$ Add'l

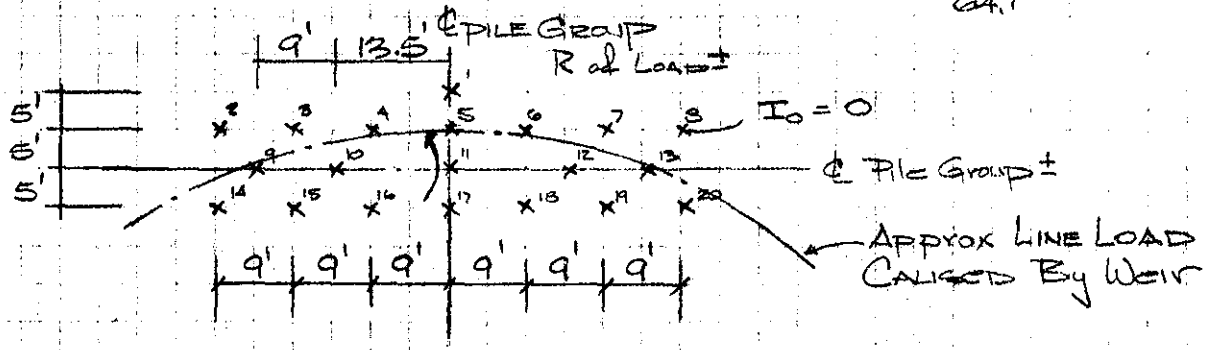
$v = \frac{12.5^k}{10 \times 12 \times 15.8} = .007^{psi}$

No Problem Transferring Shear

Piles Under Weir

$$P = \frac{P_T}{N} \pm \frac{M_y x}{\Sigma x^2} \pm \frac{M_x y}{\Sigma y^2}$$

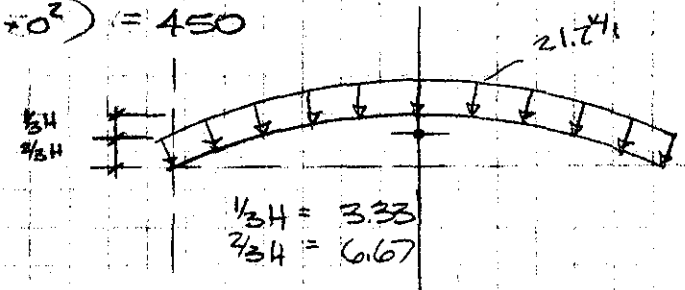
$$\frac{1360^k}{64.1} = 21.2^k/(FT)$$



P_{TOTAL} = 1360^k Conc
2088^k Water
3448^k

$M_x = P_{ey} = 3448 \times 1.67 = 5758$
$M_y = P_{ex} = 0$!

$\Sigma Y^2 = (14 \times 5^2) + (1 \times 10^2) + (5 \times 0^2) = 450$



$E_y = 5.0' - 3.33' = 1.67'$

Pile	P/N
1	172.4^k
2	
3	
4	
5	
6	
7	
8	
9	
10	
11	
12	
13	
14	
15	
16	
17	
18	
19	
20	172.4^k

Load < 200^k ok

* Add'l Loads (±) Due To Eccentricity Between CG of Load & CG of Pile Group Will Be Checked For Entire Mat

Preliminary Check - Seems ok

SUBJECT: Park River F-6

COMPUTATION: Pile Foundation - Prel.

COMPUTED BY: W. Rodger CHECKED BY: DATE: 23 Sept 74

CG of Piles

Y-Axis ≈ Under ℄ of Interior Wall

Piles		$\bar{y}$	$P \cdot \bar{y}$	x
7	x	0	0	26
9	x	10	90	16
9	x	20	180	6
2	x	25	50	1
3	x	28	84	2
1	x	29	29	3
9	x	30	270	4
7	x	35	245	9
8	x	40	320	14
1	x	45	45	19
6	x	50	300	24
62 Piles			1613	

$Y_B = \frac{1613}{62} = 26' \pm$

$Y_T = 24'$

See Fig #1 Pg F-1

		W	$\bar{y}$	$W \cdot \bar{y}$
Walls	A Left	364.5	16	5832
	B Center	325.7	20	6514
	C Right	356.4	18	6415
Base	A Front	120	48	5760
	B Weir	1360.	38	51680
	C Trans	911	15	13665
Buttresses	–	214	25	5350
Water	–	5609	24±	134616
Soil	A	1579	26'	4105
Roof	–	435	10	4350
		11275±		275236

$\bar{Y}_{wt} = \frac{275236}{11275} = 24.4' \pm$

Sum of ΣY^2

7	x	26^2	=	4732
9	x	16^2	=	2304
9	x	6^2	=	324
2	x	1^2	=	2
3	x	2^2	=	12
1	x	3^2	=	9
9	x	4^2	=	144
7	x	9^2	=	567
8	x	14^2	=	1568
1	x	19^2	=	361
6	x	24^2	=	3456

$\Sigma Y^2 = 13{,}479^k$

$E_x = 1.6$

$M_x = 1.6(11275^k) = 18{,}040^{ik}$

SUBJECT: Park River Local Prot. F-7

COMPUTATION: Prel. Check Pile Loads

COMPUTED BY: W. Rodger CHECKED BY: DATE: 23 Sept

$$P_Q = Q/N = \frac{11275}{62} = 182^{K}/\text{Pile} \pm$$

$M_x = 18{,}040$ (+) Comp (−) Tension

$$P = Q/N \pm \frac{M_x(y)}{\Sigma y^2}$$

	(No of Piles)		P_Q	$\frac{M_x y}{\Sigma y^2}$	P_{TOTAL}
1ST Row	(7)	y = 26	182^K	+ 19.43^K	201.43
2ND "	(9)	y = 16	182^K	+ 11.95^K	193.95
3RD "	(9)	y = 6	182^K	+ 4.50^K	186.50
4TH "	(2)	y = 1	182	+ .75^K	182.75
5TH "	(3)	y = 2	182	− 1.50^K	180.50
6TH "	(1)	y = 3	182	− 2.24^K	179.76
7TH "	(9)	y = 4	182	− 3.00	179.00
8TH "	(7)	y = 9	182	− 6.72	175.28
9TH "	(8)	y = 14	182	− 10.46	171.54
10TH "	(1)	y = 19	182	− 14.20	167.80
11TH "	(6)	y = 24	182	− 17.93	164.07
	62				

ok

But Add two More Piles

∴ 64 - 100TON Pile Are Adequate To Resist The Applied Loads On The Inlet Structure

SUBJECT PARK RIVER - JUNCTION STRUCTURE
COMPUTATION S.F. AGAINST UPLIFT
COMPUTED BY WTH. CHECKED BY DATE

SAFETY FACTOR AGAINST UPLIFT
FOR JUNCTION STRUCTURE

EL. 43.66 - AVE. TOP OF ROOF SLAB

EL. 41.93 - " BOTTOM " " "

EL. 27.5 - DRAIN INVERT

EL. 15.56 - AVE. TOP OF BASE SLAB

EL. 10.56 - " BOTTOM " " "

ASSUME WATER LEVEL 1/2 WAY BETWEEN
TOP OF BOX AND DRAIN INVERT
(43.66 - 27.5) x .5 + 27.5
ELEV. 35.58

35.58 - 10.56 = 25.03
25.03 x .0625 = 1.56 K/SQ.FT. TOTAL
UPLIFT ON BASE SLAB
W/ NO WATER INSIDE
STRUCTURE

D.L. + L.L. = 16659K + 14348 x .75
SEC. P. 59

TOTAL PLAN OF STRUCTURE

5' BASE SLAB

L.L.

- .15 x 14348 = 25268K

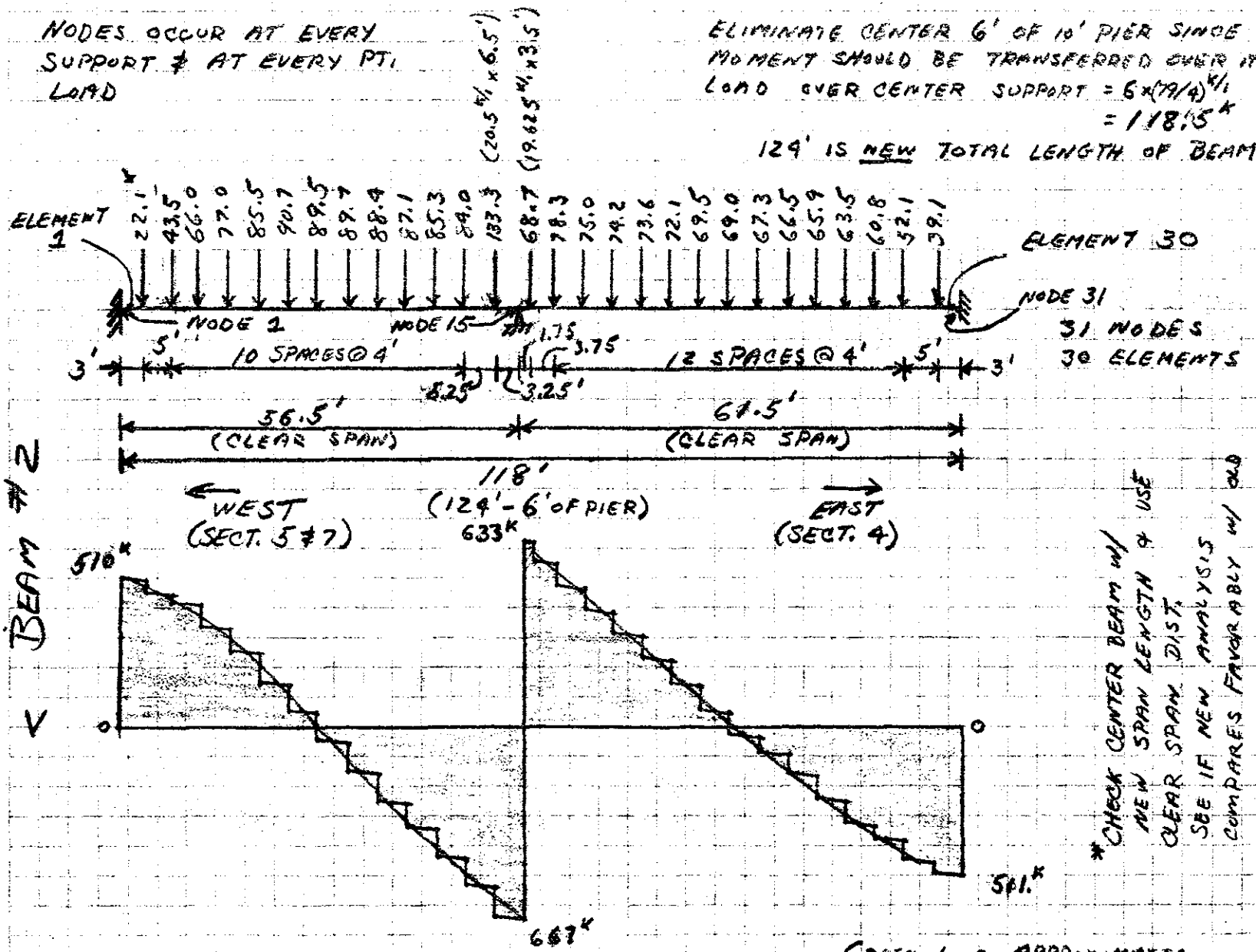
1.56 x 14348 = 22383K

S.F. = 25268/22383 = 1.13 O.K.

NODES OCCUR AT EVERY SUPPORT & AT EVERY PT. LOAD

ELIMINATE CENTER 6' OF 10' PIER SINCE MOMENT SHOULD BE TRANSFERRED OVER IT ANYWAY.
LOAD OVER CENTER SUPPORT $= 6 \times (79/4)^{K/'}$
$= 118.5^{K}$

124' IS NEW TOTAL LENGTH OF BEAM

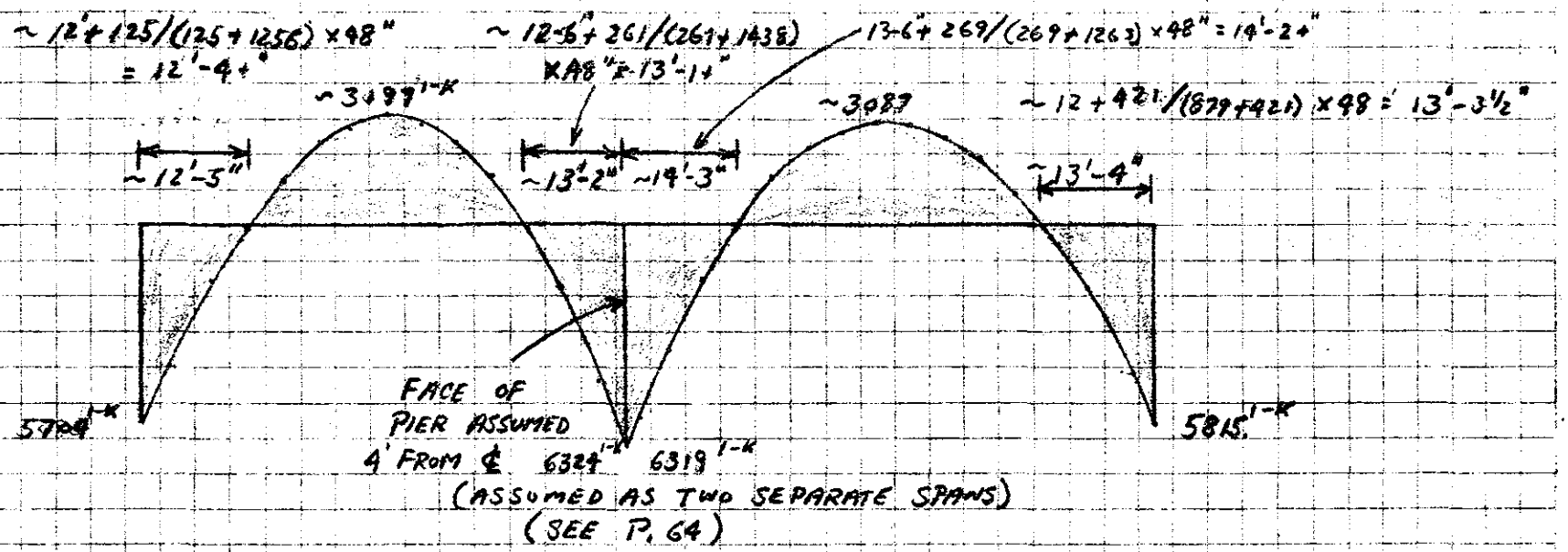


*CHECK CENTER BEAM W/ NEW SPAN LENGTH & USE CLEAR SPAN DIST. SEE IF NEW ANALYSIS COMPARES FAVORABLY W/ OLD

TOTAL LOAD ON CENTER PIER
$\approx 118.5 + 667 + 633 = 1418.5^{K}$

GREEN LINE APPROXIMATES ACTUAL SHEAR SINCE THE LOAD IS REALLY MORE UNIFORM

SUBJECT PARK RIVER - JUNCTION STRUCTURE
COMPUTATION ANALYSIS OF JUNCTION STRUCTURE CENTER BEAM (124') 2-SPAN
COMPUTED BY WJN CHECKED BY DATE 5 SEPT. 74



(ASSUMED AS TWO SEPARATE SPANS)
(SEE P. 64)

SEE P. 65 FOR STEEL CALC.

***

FROM COMPUTER OUTPUT
DEFLECTIONS & MOMENTS COMPARE FAVORABLY W/ OLD ANALYSIS SO USE OLD DESIGN FOR OTHER 2 BEAM IN DETERMINING REINF. ETC FOR THEM.

PER ACI 8.4.2 FIXED END MOMENT MAY BE USED WHERE THE RATIO OF COLUMN STIFFNESS TO BEAM STIFFNESS EXCEEDS 8 AT EACH END OF THE SPAN

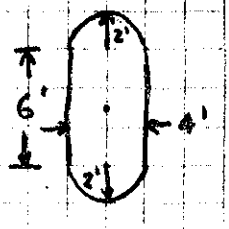
I OF PIER $I = I_o + AD^2$

$2(1.75 + 6.28(3.85)^2) + 1/12(4)(6)^3$

$= 261.67 \text{ FT}^4$

$L = (40.41 - 15.56) + 3.0' = 27.85'$

$I/L = 9.4$



I OF BEAM

$I = 1/12(4)(6)^3 = 72 \text{ FT}^4$

L = CLEAR SPAN + DIST. TO ℄ OF SUPPORTS

$= 55.5 + 4'(\text{PIER}) + \sim 2' = 61.5$

$I/L = 1.17$

$9.4/1.17 = 8.03 > 8$ O.K.

∴ DON'T HAVE TO INVESTIGATE ALTERNATE LIVE LOADINGS TO DETERMINE MAX. POSITIVE MOMENTS SINCE BEAM ACTS LIKE 2 SEPARATE BEAMS RATHER THAN ONE <u>CONTINUOUS</u> BEAM.

* MOMENTS CAN BE COMPUTED USING CLEAR SPAN DISTANCES RATHER THAN USING C-C DISTANCES AND TAKING A REDUCTION BACK TO THE FACE AS DONE FOR BOX CULVERTS.

M_{max} (NEG.) = ~ 6100 '-K

72-4.5

$A_s = M/f_s jd = 6324 \times 12 / (20 \times 7/8 \times 67.5)$

= 64.8" NEED 16+ #18 BARS
2 ROWS

ADJUST "d"

#18 BAR DIAM. = 2.257"
AREA = 4.0 □"

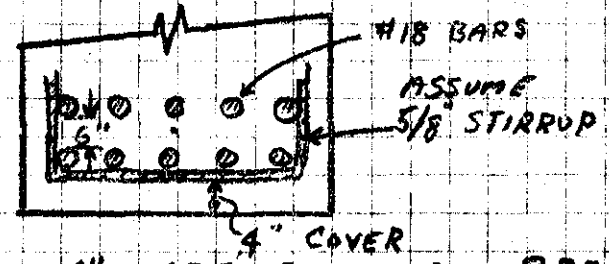
FOR 8-#18's YOU NEED A WIDTH OF
7×8" + 2.257" + 8" + 1.25" = 67.5" > 48
COVER

EM 1110-2-2103 2-#5 STIRRUPS
8" C-C SPACING

TRY 5-#18's
4×8 + 2.257 + 8 + 1.25 = 43.51" < 48"

RATHER THAN INCREASE THE WIDTH DETERMINE WHAT DEPTH OF CONCRETE IS NEEDED TO GET 40 □" OF STEEL.

CALCULATE DIST. FROM EXTREME FIBER TO CENTROID OF TENSION STEEL.



4" + .625 + 2.257 + 3 = 9.88" SAY 10"

FOR BALANCE DESIGN

$d = \sqrt{6324/(.324 \times 4)} = 69.9"$
+ 10" = 79.9"

$d = 6324 \times 12 / (20 \times .86 \times 40) = 140.3"$

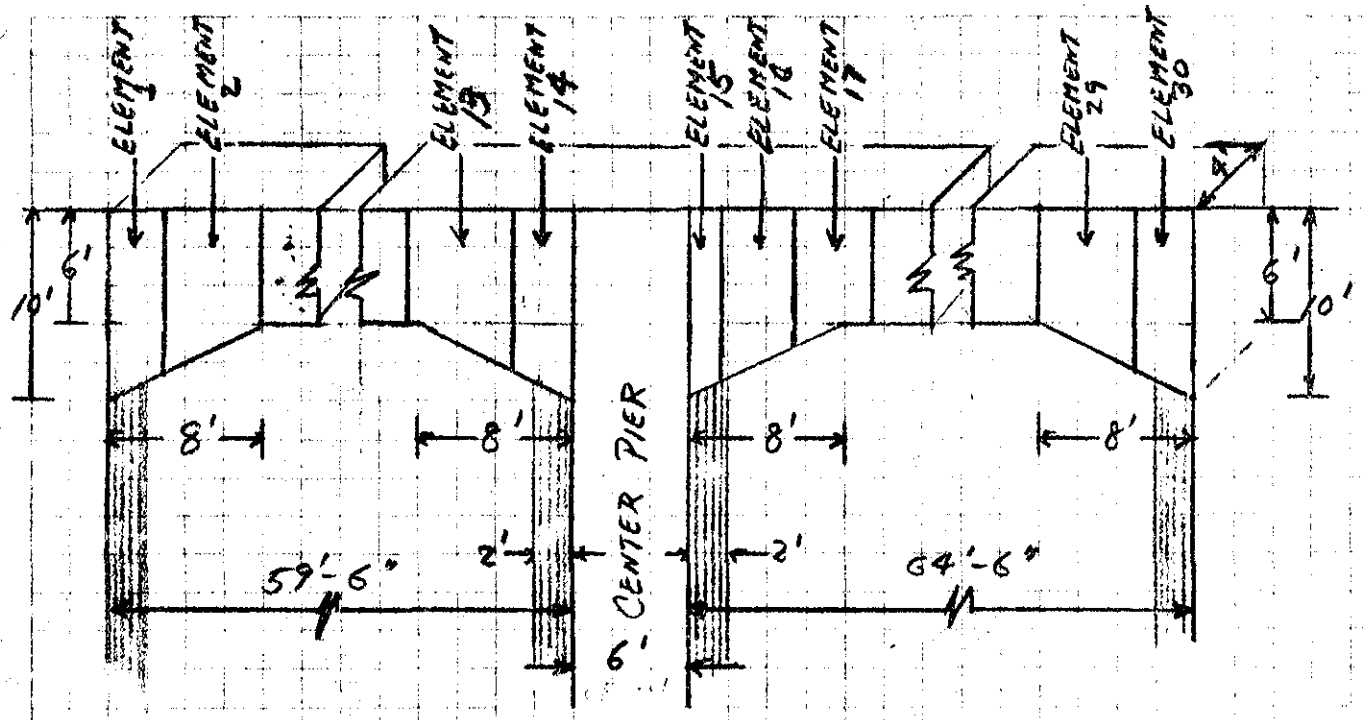
140.3 + 10 = 120.3 120.3/12 = 10.03'
SAY 10'

* TRY RUNS W/ HAUNCH STIFFNESS INPUT

SUBJECT PARK RIVER – JUNCTION STRUCTURE

COMPUTATION BEAM HAUNCH STIFFNESSES FOR 124' CENTER BEAM

COMPUTED BY WJH CHECKED BY DATE 19 SEPT 49



ELEMENT	AVE. DEPTH	AREA	I
1	9.4'	37.6 ft	276.9 ft⁴
2	7.25	29.0	127.
13	7.25	29.0	127.
14	9.25	37.0	263.8
15	9.5	38.0	285.8
16	8.0	32.0	170.6
17	6.4	25.6	87.4
29	7.25	29.0	127.
30	9.4	37.6	276.9

*EXTRA LOAD PER FT. ON BEAM DUE TO ADDITION OF HAUNCHES [EXTRA OVER 2.4 K/' (6×4×.15)]

ELEM.	LOAD* K/'
1	2.04
2	.75
13	.75
14	1.95
15	2.1
16	1.2
17	.24
29	.75
30	2.04

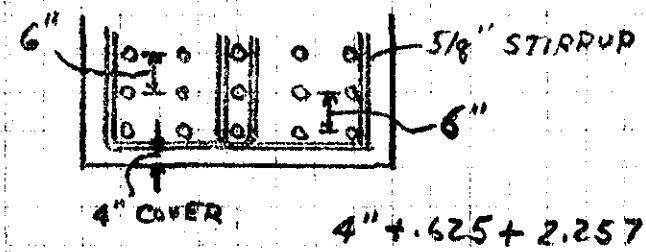
SUBJECT PARK RIVER - JUNCTION STRUCTURE

COMPUTATION ANALYSIS OF 124' CENTER BEAM IN JUNCTION STRUCTURE WITH

COMPUTED BY _____ CHECKED BY _____ DATE 20 SEPT. 74

MAX. NEGATIVE MOMENT = 7227'-K FOR BEAM IN CENTER OF JUNCTION STRUCTURE W/ HAUNCH STIFFNESS ADDED. SINCE 6' DEEP BEAM + 4' HAUNCH (10') W/ 2 ROW OF #18 BARS (40 SQ. IN) HAS A CAPACITY OF ~6400'-K TRY ADDING MORE STEEL RATHER THAN INCREASING THE SIZE OF THE HAUNCH.

TRY 3 ROWS OF #18 BARS $A_s = 60 \square''$



$4'' + .625 + 2.257 + 6'' + 2.257/2 \approx 14''$

$d = 7227 \times 12 / (20 \times .86 \times 60) = 84.03''$

$84.03 + 14 = 98.03$ ← USE

$d = \sqrt{7227/(.324 \times 4)} = 74.68''$

$74.68 + 14 = 88.68''$

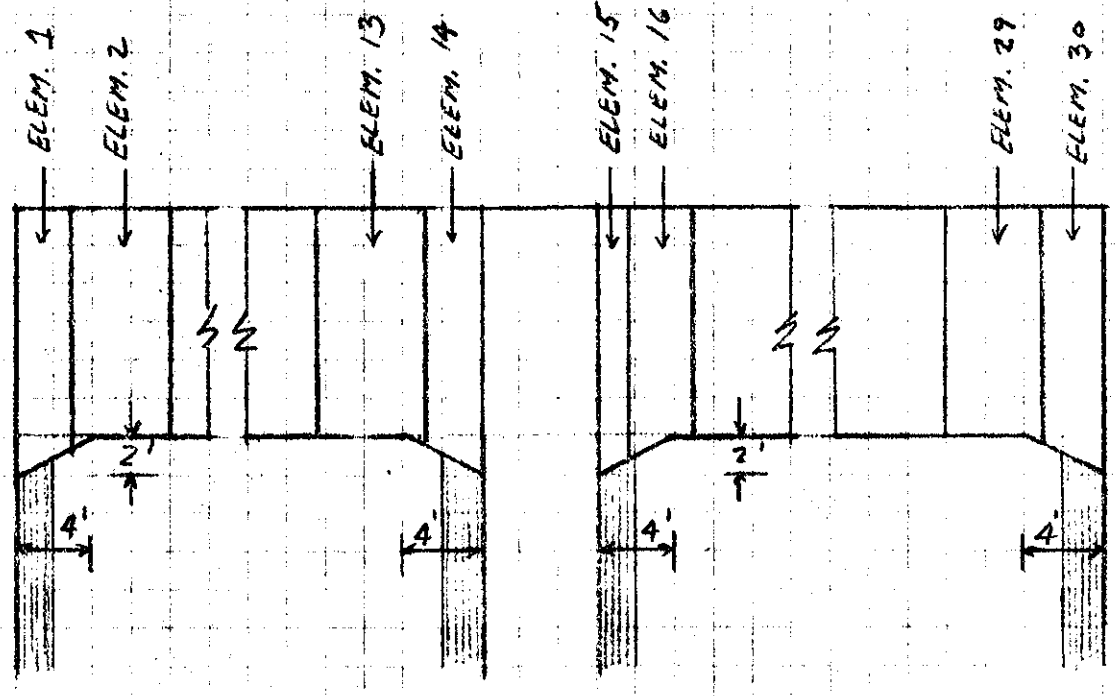
$98.03/12 = 8.17'$

SAY 8' - RERUN BEAM W/ APPROPRIATE STIFFNESS FOR 6' BEAM W/ 2' HAUNCH AT EACH PIER.

SUBJECT PARK RIVER - JUNCTION STRUCTURE

COMPUTATION BEAM HAUNCH STIFFNESSES FOR 124' CENTER BEAM

COMPUTED BY WJH CHECKED BY DATE 19 SEPT 74

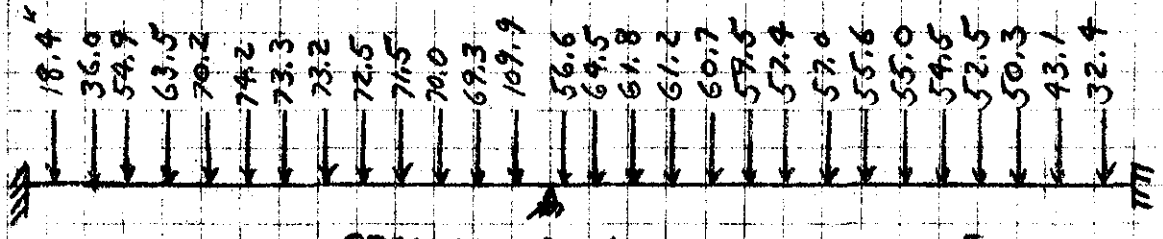


ELEM.	AVE. DEPTH	AREA	I	LOAD*
1	7.25'	29.4	127.	.75K
2	6.05'	24.2	73.8	—
13	6.07'	24.3	74.5	—
14	7.3'	29.2	129.7	.78
15	7.56'	30.2	144.	.94
16	6.34	25.4	84.9	.2
29	6.05'	24.2	73.8	—
30	7.25'	29.	127.	.75K

*SEE P. 66A

TO GET COMPLETE MOMENT ENVELOPE BEAMS MUST BE RUN W/ D.L. ONLY (INFLECTION PT. LOCATIONS WILL CHANGE)

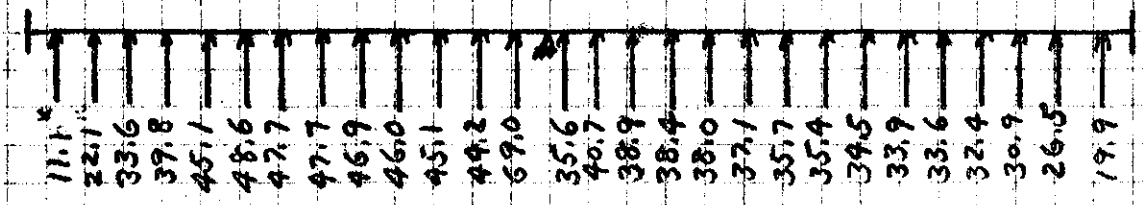
D.L. ONLY ON BOTH SPANS
(CENTER SUPPORT FIXED)



SPACING OF LOADS IS THE SAME AS ON P. 62

* TRY ALTERNATE LIVE LOADING ALSO W/ CENTER SUPPORT PINNED EVEN THOUGH CALCULATION ON P. 64 SEEMS TO REINFORCE THEORY OF FIXED CENTER SUPPORT. THIS ALTERNATE LIVE LOADING WILL GIVE MAX. POSITIVE MOMENT ON EACH SPAN.
(SEE P. 62 & THIS PAGE FOR INDIVIDUAL LOADS)

* TRY ALSO A RUN W/ BURSTING PRESSURES ASSUMING A HYDRAULIC GRADIENT OF ELEV. 49 AND AN AVE. ROOF SOFFIT ELEV. OF 41.93
(49 − 41.93) × .0625 = .442 K/SQ. FT. ↑



COMBINE BURSTING PRESSURES W/ PREVIOUS RUNS

D.L. ONLY ON BOTH SPANS	CENTER SUPPORT FIXED & PINNED
D.L. + ALTERNATING L.L.	CENTER SUPPORT PINNED
D.L. + L.L. ON BOTH SPANS	CENTER SUPPORT FIXED

SUBJECT PARK RIVER - JUNCTION STRUCTURE

COMPUTATION 124' CENTER BEAM DESIGN - OUTPUT - CASE 1

COMPUTED BY _____ CHECKED BY _____ DATE _____

NODAL POINT DISPLACEMENTS

NODE	X-DISPL	Y-DISPL	ROT-DISPL
1	0.	0.	0.
2	0.	-.34806E-03	-.20119E-03
3	0.	-.25533E-02	-.61641E-03
4	0.	-.53262E-02	-.73690E-03
5	0.	-.82536E-02	-.70371E-03
6	0.	-.10795E-01	-.54963E-03
7	0.	-.12533E-01	-.31134E-03
8	0.	-.13226E-01	-.23338E-04
9	0.	-.12759E-01	.25911E-03
10	0.	-.11200E-01	.51092E-03
11	0.	-.37724E-02	.63723E-03
12	0.	-.53553E-02	.74871E-03
13	0.	-.29875E-02	.65665E-03
14	0.	-.44095E-03	.25433E-03
15	0.	0.	0.
16	0.	-.12031E-03	-.13341E-03
17	0.	-.13414E-02	-.48939E-03
18	0.	-.38607E-02	-.73326E-03
19	0.	-.70179E-02	-.81447E-03
20	0.	-.10192E-01	-.75255E-03
21	0.	-.12399E-01	-.53624E-03
22	0.	-.14737E-01	-.34835E-03
23	0.	-.15633E-01	-.72356E-04
24	0.	-.15361E-01	.20937E-03
25	0.	-.13992E-01	.46793E-03
26	0.	-.11692E-01	.67040E-03
27	0.	-.87440E-02	.78667E-03
28	0.	-.55536E-02	.78671E-03
29	0.	-.26440E-02	.64153E-03
30	0.	-.35623E-03	.22659E-03
31	0.	0.	0.

END ACTIONS WITH ELEMENT LOADS

ELEMENT	AXIAL I AXIAL J	SHEAR I SHEAR J	MOMENT I MOMENT J
1	0.	509.935	5910.261
	0.	-509.535	-4394.430
2	0.	473.435	4394.431
	0.	-466.435	-2032.394
3	0.	422.935	2032.394
	0.	-413.335	-359.763
4	0.	347.336	359.763
	0.	-337.736	1010.331

SUBJECT PARK RIVER - JUNCTION STRUCTURE

COMPUTATION 124' CENTER BEAM DESIGN - OUTPUT - CASE 1

COMPUTED BY MTH CHECKED BY DATE 24 SEPT 74

352	5	0.	260.736	-1010.383
353		0.	-251.136	2034.126
354	6	0.	165.639	-2034.123
355		0.	-155.039	2677.435
356	7	0.	65.340	-2677.433
357		0.	-55.740	2919.644
358	8	0.	-33.752	-2919.641
359		0.	43.353	2765.409
360	9	0.	-133.055	-2765.404
361		0.	142.655	2213.933
362	10	0.	-231.055	-2213.929
363		0.	240.655	1270.553
364	11	0.	-327.755	-1270.556
365		0.	337.355	-59.665
366	12	0.	-422.654	59.666
367		0.	432.254	-1769.484
368	13	0.	-516.255	1769.484
369		0.	523.855	-4512.896
370	14	0.	-662.155	4512.896
371		0.	672.490	-6631.693
372	15	0.	635.768	6592.534
373		0.	-629.923	-5435.054
374	16	0.	561.223	5435.054
375		0.	-551.473	-3398.750
376	17	0.	473.173	3398.751
377		0.	-463.573	-1525.253
378	18	0.	388.574	1525.253
379		0.	-378.974	9.839
380	19	0.	304.774	-9.837
381		0.	-295.174	1209.732
382	20	0.	221.577	-1209.730
383		0.	-211.977	2076.840
384	21	0.	139.377	-2076.839
385		0.	-130.277	2617.145
386	22	0.	60.779	-2617.143
387		0.	-51.179	2341.063
388	23	0.	-17.816	-2341.059
389		0.	27.416	2750.592
390	24	0.	-94.716	-2750.533
391		0.	104.316	2352.525
392	25	0.	-170.315	-2352.522
393		0.	180.415	1650.061
394	26	0.	-246.314	-1650.062
395		0.	255.914	645.608
396	27	0.	-319.412	-645.603
397		0.	329.012	-651.239
398	28	0.	-389.811	651.239
399		0.	399.411	-2229.631
400	29	0.	-451.511	2229.631
401		0.	463.511	-4517.234
402	30	0.	-502.610	4517.234
403		0.	512.860	-6039.248
404				

SUBJECT PARK RIVER – JUNCTION STRUCTURE

COMPUTATION TABULATION OF MOMENTS FOR 124' CENTER BEAM W/2' HAUNCHES

COMPUTED BY WJH. CHECKED BY DATE 24 SEPT 49

CENTER SUPPORT FIXED (1) CENTER SUPPORT PINNED (2)

(1) CASE 1 – D.L. + 150# L.L. ON BOTH SPANS
(1) CASE 2 – D.L. ONLY ON BOTH SPANS
(2) CASE 3 – D.L. + 150# L.L. ON LEFT SPAN, D.L. ONLY ON RIGHT SPAN
(2) CASE 4 – D.L. + 150# L.L. ON RIGHT SPAN, D.L. ONLY ON LEFT SPAN
(1) CASE 5 – D.L. + BURSTING LOAD + 150# L.L. ON BOTH SPANS
(1) CASE 6 – D.L. + BURSTING LOAD ON BOTH SPANS
(2) CASE 7 – D.L. + BURSTING LOAD ON BOTH SPANS, 150# L.L. ON LEFT SPAN
(2) CASE 8 – D.L. + BURSTING LOAD ON BOTH SPANS, 150# L.L. ON RIGHT SPAN
(2) CASE 9 – D.L. + BURSTING LOAD ON BOTH SPANS

NODE	1	2	3	4	5	6	7	8	9
1	-5910.	-4976.	-6219.	-4699.	-3154.	-2219.	-3411.	-1889.	-2173.
2	-4394.	-3694.	-4656.	-3459.	-2328.	-1627.	-2596.	-1347.	-1588.
3	-2032.	-1702.	-2215.	-1538.	-1060.	-730.	-1212.	-534.	-703.
4	-360.	-296.	-480.	-188.	-125.	-111.	-275.	+17.	-93.
5	+1010.	+852.	+953.	+903.	+542.	+384.	+495.	+445.	+393.
6	+2034.	+1708.	+2040.	+1703.	+1073.	+746.	+1077.	+740.	+795.
7	+2677.	+2249.	+2746.	+2183.	+1403.	+969.	+1460.	+896.	+959.
8	+2920.	+2445.	+3051.	+2328.	+1526.	+1052.	+1636.	+911.	+1032.
9	+2765.	+2315.	+2960.	+2141.	+1444.	+993.	+1606.	+785.	+964.
10	+2214.	+1854.	+2472.	+1623.	+1155.	+794.	+1369.	+519.	+756.
11	+1271.	+1064.	+1591.	+777.	+662.	+455.	+929.	+113.	+407.
12	+60.	-50.	+324.	-394.	-33.	-29.	+285.	-434.	-81.
13	-1769.	-1483.	-1323.	-1883.	-928.	-642.	-558.	-1119.	-708.
14	-4513.	-3785.	-3984.	-4259.	-2370.	-1641.	-1931.	-2206.	-1720.
15(L)	-6682.	-5605.	-6101.	-6125.	-3509.	-2430.	-3027.	-3049.	-2516.
15(R)	-6593.	-5580.	-6101.	-6125.	-3607.	-2594.	-3027.	-3049.	-2516.
16	-5485.	-4694.	-5143.	-5038.	-3005.	-2163.	-2578.	-2472.	-2089.
17	-3399.	-2879.	-3330.	-2995.	-1868.	-1348.	-1722.	-1387.	-1281.
18	-1525.	-1294.	-1693.	-1168.	-844.	-613.	-944.	-418.	-554.
19	+10.	+5.	-341.	+320.	-3.	-8.	-285.	+367.	+44.
20	+1210.	+1022.	+727.	+1474.	+656.	+468.	+224.	+971.	+512.
21	+2077.	+1757.	+1515.	+2294.	+1135.	+815.	+613.	+1394.	+851.
22	+2617.	+2216.	+2025.	+2788.	+1435.	+1034.	+825.	+1638.	+1062.
23	+2841.	+2407.	+2268.	+2965.	+1561.	+1127.	+1012.	+1709.	+1148.
24	+2751.	+2331.	+2295.	+2828.	+1515.	+1096.	+1024.	+1607.	+1108.
25	+2353.	+1995.	+1966.	+2384.	+1299.	+941.	+913.	+1336.	+947.
26	+1650.	+1400.	+1418.	+1635.	+914.	+664.	+679.	+896.	+662.
27	+646.	+549.	+618.	+584.	+362.	+265.	+323.	+288.	+255.
28	-651.	-550.	-429.	-760.	-353.	-252.	-152.	-483.	-270.
29	-2230.	-1889.	-1716.	-2385.	-1226.	-886.	-742.	-1411.	-912.
30	-4517.	-3833.	-3595.	-4730.	-2500.	-1815.	-1618.	-2754.	-1851.
31	-6039.	-5128.	-4851.	-6287.	-3354.	-2442.	-2212.	-3649.	-2489.

SUBJECT PARK RIVER – JUNCTION STRUCTURE

COMPUTATION REINF.

COMPUTED BY WJH. CHECKED BY DATE 24 SEPT 74

MAX. NEG. MOMENTS

AT LEFT SUPPORT = 6219 '-K

" CENTER " = 6682 '-K

" RIGHT " = 6287 '-K

USE 3 ROWS #18 BARS 5 BARS PER ROW

AT SUPPORTS BEAM IS 8' DEEP (6'+2' HAUNCH)

d (FOR 3 ROWS STEEL) = 96"-14" = 82"

$d = \sqrt{6682/(.329 \times 4)} = 71.8" < 82$ O.K.

$d = 6682 \times 12/(20 \times .86 \times 60) = 77.7 < 82$ O.K.

$A_s = 6219 \times 12/(20 \times .86 \times 82) = 52.9" < 60.0"$ O.K.

AT END OF HAUNCH 4' FROM SUPPORT FACE DEPTH IS 6'

TRY 2 ROWS OF #18 BARS

MOMENT CAPACITY = $40 \times 20 \times .86 \times 62/12$ (62" = 72"-10")

= 3555.1 '-K < MAX. FOR ALL SUPPORTS

ALLOWABLE BOND

$u = 2.1\sqrt{f'_c} = 2.1(63.25) = 132.8$ PSI

$L = f_s D/4u = 20000(2.257)/(4 \times 132.8)$

= 85" SAY 7'

MOMENT CAPACITY FOR 1 ROW OF STEEL

$= 20 \times 20 \times \quad \times 66.25/12$ (66.25 → 72"-5.75")

= 1899 '-K

LEFT SUPPORT

FIND WHERE 3617 '-K & 1932 '-K OCCUR

CASE 3 – NODE 2 (3' FROM SUPPORT)

(4656 - 2215)/5 = 488.2 '-K (~ DROP IN MOMENT PER FT.)

(4656 - 3555)/488.2 = ~2.3' FROM NODE 2

3 + 2.3 = 5.3 SAY 6' FROM SUPPORT

6 + 7 = 13' FROM SUPPORT DROP 1 ROW OF BARS

CASE 3 – NODE 3 (8' FROM SUPPORT)

(2215 - 480)/4 = 433.75 '-K (~ DROP IN MOMENT PER FT.)

(2215 - 1899)/433.75 = .73'

8 + .73 = 8.73' SAY 10' FROM SUPPORT

10 + 7 = 17' FROM SUPPORT DROP 2ND ROW OF BARS

SUBJECT PARK RIVER - JUNCTION STRUCTURE
COMPUTATION REINF.
COMPUTED BY WJH CHECKED BY DATE 24 SEPT. 74

RIGHT SUPPORT
MOMENT IS SIMILAR
DUE THE SAME AS FOR LEFT SUPPORT

MAX. POSITIVE MOMENTS
LEFT SPAN ~ 3051 '-K SAY 3100 '-K CASE 3 NODE 8
RIGHT " ~ 2965 '-K SAY 3000 '-K CASE 4 NODE 23

TRY 2 ROW OF #18 BARS 5 BARS PER ROW
$A_s = 40 \square''$ $d = 72-10 = 62''$

$d = \sqrt{3100/(.324 \times 4)} = 48.9'' < 62''$ O.K.

$d = 3100 \times 12/(20 \times .86 \times 40) = 54.1'' < 62''$ O.K.

$A_s = 3100 \times 12/(20 \times .86 \times 62) = 34.9 \square''$
$< 40 \square''$ O.K.

MOMENT AT WHICH ONLY 1 ROW OF BARS IS NEEDED (SEE P.)
= 1899 '-K

LEFT SPAN
NODE 6 CASE 1 & 3 M = 2034 & 2040 '-K
~ = 1899 ∴ 1' TO LEFT OF NODE 6
SAY WE ONLY NEED 1 ROW OF BARS
NODE 6 IS 20' FROM LEFT SUPPORT FACE
∴ 19 - 7 = 12' EXTEND TWO ROWS
OF BOTTOM STEEL TO W/IN 12' OF LEFT
SUPPORT BEFORE DROPPING A ROW.

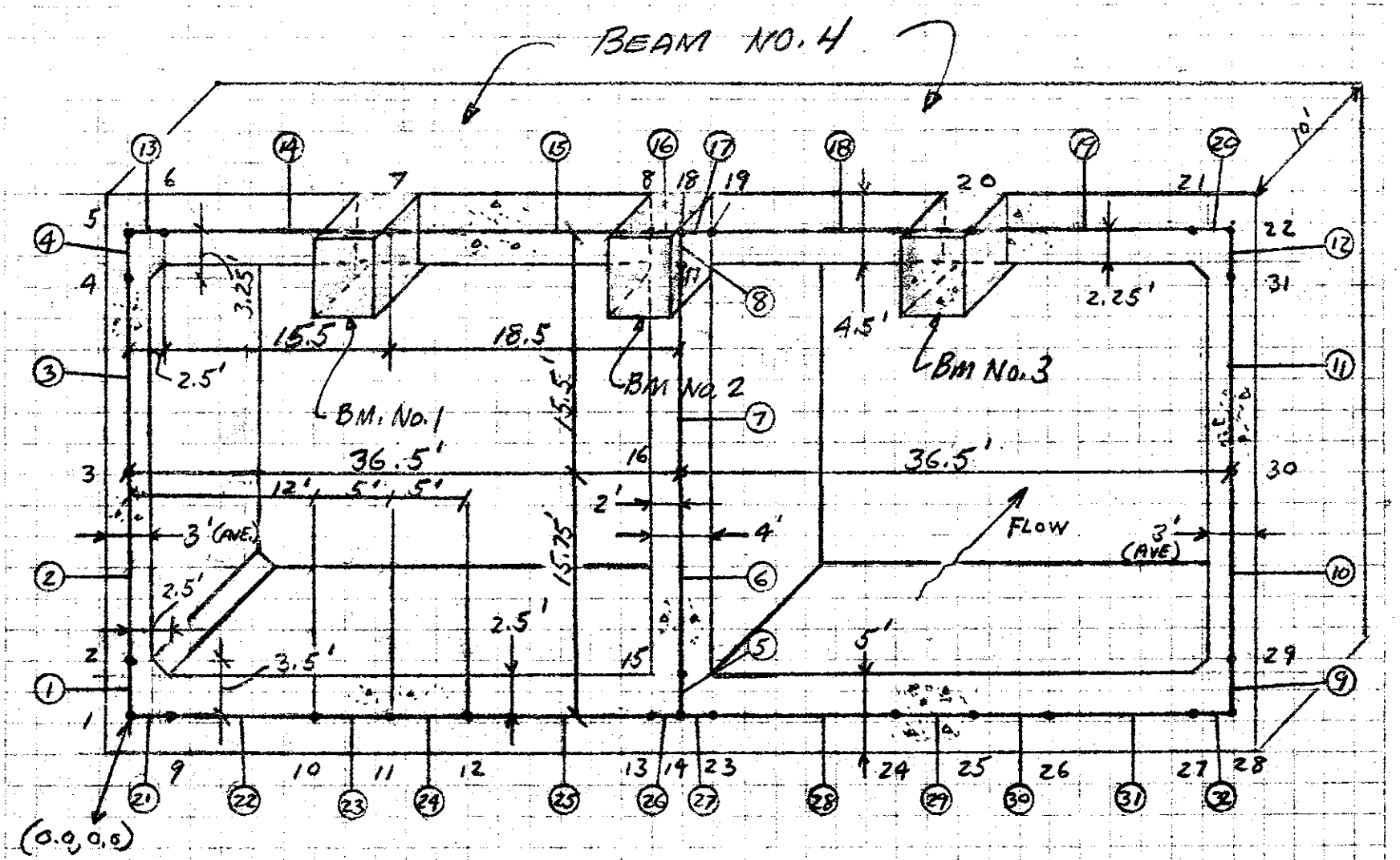
CASE 3 NODE 10 20.5' FROM CENTER PIER FACE
= 2472 '-K
NODE 11 16.5' " " " "
= 1591 '-K
(2472-1591)/4 = 220.25 '-K DROP PER FT. BETW.
NODES 10 & 11
(2472 - 1899)/220.25 = 2.6' TO RIGHT OF NODE 10
20.5-2.6 = 17.9' SAY 17' FROM FACE OF
CENTER PIER ONLY 1 ROW OF STEEL IS NEEDED
17-7 (DEVELOPMENT LENGTH OF #18) = 10'
∴ 10' FROM FULL FACE OF CENTER PIER DROP OF 1 ROW OF $+A_s$

CORPS OF ENGINEERS, U. S. ARMY

SUBJECT PARK RIVER - JUNCTION STRUCTURE

COMPUTATION DESIGN OF SECTION 4 ROOF BEAM AT END OF JUNCTION BOX

COMPUTED BY WDH CHECKED BY _____ DATE 23 SEPT 74



BEAM ON ELASTIC FOUNDATION

MODULUS OF SUBGRADE REACTION = 10 × 170 = 1700 #/CU. FT.

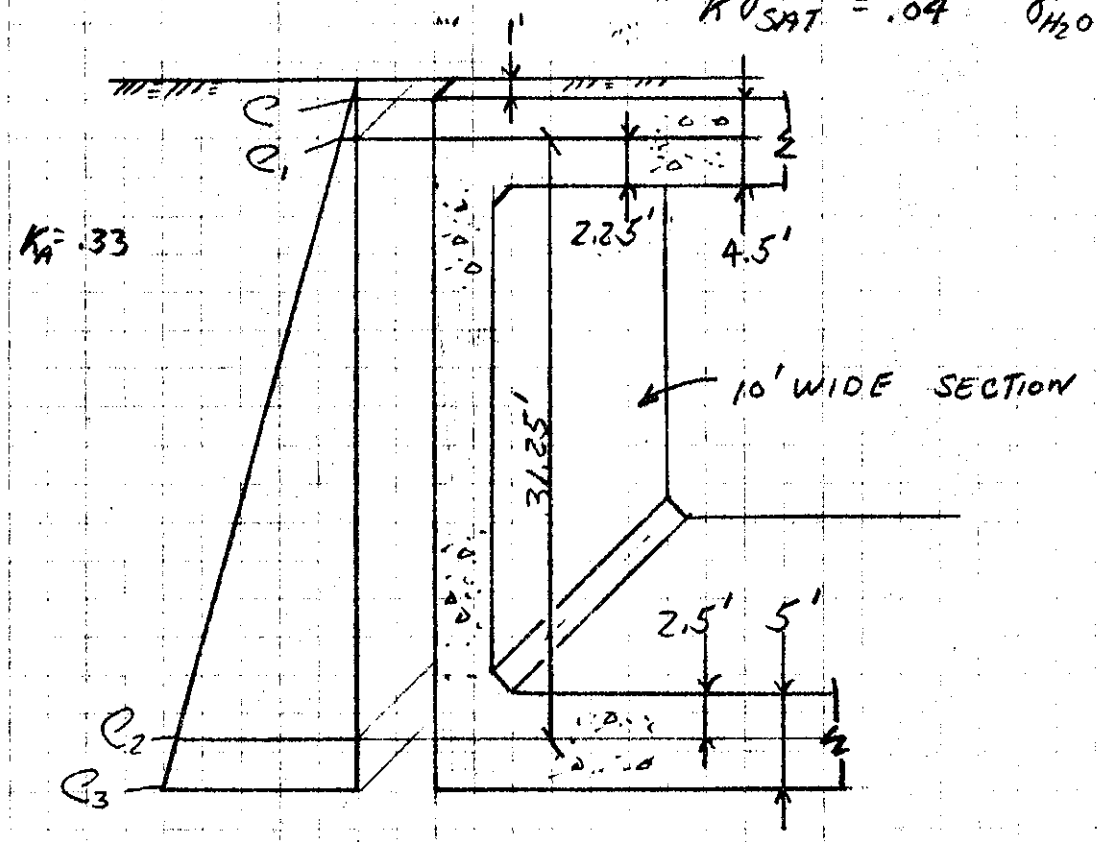
SUBJECT PARK RIVER – JUNCTION BOX

COMPUTATION BEAM AT ENTRANCE TO SECTION 4 (4.5 X 10')

COMPUTED BY WJH CHECKED BY DATE 24 SEPT 74

NO WATER OUTSIDE OR IN
NO SURCHARGE

$K\gamma_{SAT} = .04 \quad \gamma_{H_2O} + K\gamma_{BUOY} = 82.3$



$C = .04 \times 10 = .4^k$

$C_1 = .04\,(3.25)(10) = 1.3^k$

$P_T = (.4 + 1.3)/2 \times 2.25 = 1.91^k$

$C_2 = 34.5\,(.04)(10) = 13.8^k$

$C_3 = 37.\,(.04)(10) = 14.8^k$

$P_B = 14.3\,(2.5) = 35.75^k$

SUBJECT PARK RIVER

COMPUTATION BEAM AT ENTRANCE (BOX SECTION) TO SECT. 4 WITH

COMPUTED BY _____ CHECKED BY _____ DATE 24 SEPT 74

WATER TO TOP OF BOX OUTSIDE
NO WATER INSIDE CULVERT L.L. = 150#
$K_o = .5$ $K\gamma_{SAT} = 60$ $\gamma_{H_2O} + K\gamma_{BUOY} = 92.5$

$e = 10 \times .06 = .6$
$e_1 = .6 + .0925 \times 2.25 \times 10 = 2.68$
$P_T = (.6 + 2.68)/2 \times 2.25 = 3.69^K$

$e_2 = .6 + 33.5 \times .0925 \times 10 = 31.59$
$e_3 = .6 + 36. \times .0925 \times 10 = 33.9$
$P_B = (31.59 + 33.9)/2 \times 2.5 = 81.86^K$

SURCHARGE $= .5 \times .15 = .75^{K/'}$
$\therefore$ $3.69 + .75 \times 2.25 = 5.38^K$
$81.86 + .75 \times 2.5 = 83.74^K$

UPLIFT $= 10 \times 36 \times .0625 = 22.5^{K/'}$

CASE LOADINGS

CASE 1 D.L. + 150# L.L. ON BOTH SPANS; NO WATER INSIDE OR OUT; $K_A = .33$; NO SURCHARGE

CASE 2 D.L. + 150# L.L. ON LEFT SPAN; ONLY D.L. ON RIGHT SPAN; NO WATER INSIDE OR OUT; $K_A = .33$; NO SURCHARGE.

CASE 3 D.L. + 150# L.L. ON RIGHT SPAN; ONLY D.L. ON LEFT SPAN; NO WATER INSIDE OR OUT; $K_A = .33$; NO SURCHARGE.

CASES 4-6 SAME AS CASES 1-3 RESPECTIVELY BUT W/ $K_o = .5$; WATER TO TOP OF BOX OUTSIDE; W/ UPLIFT; W/ SURCHARGE.

SUBJECT PARK RIVER - JUNCTION BOX

COMPUTATION BEAM AT ENTRANCE (BOX SECTION) TO SECT. 4

COMPUTED BY WJH CHECKED BY DATE 24 SEPT 74

UNIFORM LOAD OF 4.5 X 10' BEAM + 1' EARTH

$4.5 \times 10 \times .15 = 6.75^{K/'}$

$1 \times 10 \times .12 = 1.2$

$7.95^{K/'}$

UNIFORM LOAD OF 5' BASE SLAB

$= 5 \times 10 \times .15 = 7.5^{K/'}$

WALL LOADS

AT NODES 15 & 16

$13.25 \times 4 \times 10 \times .15 = 79.5^{K}$

AT NODES 2, 3, 29 & 30

$13.25 \times 3 \times 10 \times .15 = 59.625^{K}$

UNIFORM LIVE LOAD (150#/SQ.FT)

$10 \times .15 = 1.5^{K/'}$

TRIANGULAR DISTRIBUTED LOAD (SEE P. 15-17 & 69), APPLIED AS CONCENTRATED LOAD AT C.G. OF EACH TRIANGULAR LOAD.

	ELEMENT	CONCENTRATED LOAD	DIST. FROM NODE I
D.L. + L.L. (150#)	14	-18.865 K	1.167'
	14	-42.875	7.17
	15	-20.326	3.833
	15	-45.071	10.0
	18	-40.793	11.5
	18	-31.08	5.33
	19	-32.051	5.5
	19	-37.879	11.5
D.L. ONLY	14	-15.461	1.167
	14	-35.138	7.17
	15	-16.661	3.833
	15	-36.943	10.0
	18	-33.311	11.5
	18	-25.38	5.33
	19	-26.173	5.5
	19	-30.932	11.5

SUBJECT PARK RIVER – JUNCTION STRUCTURE 71A/
COMPUTATION DESIGN OF 4.5 X 6.0' BEAM ACROSS SECT. 4 ENTRANCE
COMPUTED BY WTH CHECKED BY DATE 24 SEPT. 74

AT THIS STAGE OF ANALYSIS ONLY REACTIONS FOR THE CENTER JUNCTION STRUCTURE BEAM ARE POSITIVELY KNOWN, BUT FROM THESE ANALYSIS IT IS POSSIBLE TO DETERMINE WHAT TYPES OF LOADING WILL PRODUCE MAX. & MIN. REACTIONS FROM THE BEAMS BETWEEN SECTIONS 5 & 4 AND 7 & 4 TO BE APPLIED ON THE BEAM AT THE ENTRANCE TO SECT. 4.

FULL D.L. + L.L ON THE LEFT SPAN + D.L. ONLY ON THE RT. SPAN OF JUNCTION STRUCTURE BEAMS WILL PRODUCE MIN. REACTION ON SECT. 4 BEAM. WITH L.L. SHIFTED TO RT. SPAN LEAVING THE LEFT W/ D.L. ONLY WE GET MAX. REACTION ON SECTION 4 BEAM.

IT HAS BEEN SEEN FROM ANALYSING THE DESIGN DATA FOR THE CENTER BEAM IN THE JUNCTION STRUCTURE THAT THE REACTIONS OBTAINED FROM A FULL D.L. + L.L. LOADING CONDITION WILL INCREASE ~3% WHEN L.L. IS REMOVED FROM THE OPPOSITE SPAN AND DECREASE ~20% IF REMOVED FROM THE SPAN ON WHICH THE REACTION INVESTIGATED IS LOCATED.

∴ BEAM BETWEEN SECT. 7 & 4
REACTION ON SECT. 4 BEAM FROM FULL D.L. + L.L. LOADING ≈ ~427 K
TO BALANCE MOMENTS BETTER, THE CENTER PIER, SUPPORTING BEAM BETW. SECT. 5 & 4, MAY BE MOVED BACK TOWARD SECTION 7 RESULTING IN A HIGHER BEAM REACTION AT SECT. 4 ∴ ASSUME 440 K

$440 - 440 \times .2 = 352^K$
$440 + 440 \times .03 = 453^K$

BEAM BETWEEN SECT. 5 & 4

$513 - 513 \times .2 = 410^K$
$513 + 513 \times .03 = 528^K$

SUBJECT PARK RIVER - JUNCTION STRUCTURE

COMPUTATION BOX CONDUIT AT ENTRANCE TO SECT. 4 - OUTPUT CASE 3

COMPUTED BY WJH CHECKED BY DATE 9 OCT 74

ELEMENTS ON AN ELASTIC FOUNDATION

CASE 3

ELEMENT	SPRING I	PRESSURE J
21	.385751E+02	.379169E+02
22	.379169E+02	.370662E+02
23	.370662E+02	.389730E+02
24	.389730E+02	.424088E+02
25	.424088E+02	.510769E+02
26	.510769E+02	.515443E+02
27	.515443E+02	.513570E+02
28	.513570E+02	.448708E+02
29	.448708E+02	.430204E+02
30	.430204E+02	.432547E+02
31	.432547E+02	.503865E+02
32	.503865E+02	.527679E+02

NODAL POINT DISPLACEMENTS

NODE	X-DISPL	Y-DISPL	ROT-DISPL
1	-.22072E-02	-.22691E-01	.14817E-03
2	-.25365E-02	-.22769E-01	.51672E-04
3	-.23623E-02	-.23044E-01	-.33366E-04
4	-.29245E-03	-.23275E-01	-.44350E-03
5	.14166E-02	-.23327E-01	-.61692E-03
6	.14073E-02	-.24921E-01	-.65308E-03
7	.13435E-02	-.32761E-01	-.17225E-03
8	.12755E-02	-.32181E-01	-.11371E-03
9	-.22214E-02	-.22304E-01	.15628E-03
10	-.22869E-02	-.21804E-01	-.12400E-03
11	-.23181E-02	-.22925E-01	-.32229E-03
12	-.23494E-02	-.24946E-01	-.47228E-03
13	-.24212E-02	-.30045E-01	-.22499E-03
14	-.24336E-02	-.30320E-01	-.40948E-04
15	-.23294E-02	-.30512E-01	-.42925E-04
16	-.14650E-02	-.31481E-01	-.10085E-03
17	.69565E-03	-.32402E-01	-.23860E-03
18	.12673E-02	-.32558E-01	-.26992E-03
19	.12580E-02	-.33277E-01	-.44003E-03
20	.11812E-02	-.41326E-01	-.57555E-04
21	.11090E-02	-.33731E-01	.74262E-03
22	.10985E-02	-.31923E-01	.69557E-03
23	-.24452E-02	-.30210E-01	.14156E-03
24	-.25114E-02	-.26395E-01	.31494E-03
25	-.25402E-02	-.25306E-01	.10450E-03
26	-.25690E-02	-.25444E-01	-.16262E-03
27	-.26294E-02	-.29639E-01	-.55839E-03
28	-.26425E-02	-.31040E-01	-.55532E-03
29	-.89317E-03	-.31143E-01	-.45460E-03
30	.32535E-02	-.31518E-01	-.24222E-03
31	.29550E-02	-.31848E-01	.45681E-03

SUBJECT PARK RIVER – JUNCTION STRUCTURE

COMPUTATION BOX CONDUIT AT ENTRANCE TO SECT 4 – OUTPUT CASE 3

COMPUTED BY WJH CHECKED BY DATE 9 OCT 74

END ACTIONS WITH ELEMENT LOADS

	ELEMENT	AXIAL I AXIAL J	SHEAR I SHEAR J	MOMENT I MOMENT J
	1	429.688	135.926	752.204
		-429.688	-90.076	-358.129
	2	370.063	90.076	358.130
		-370.063	31.811	-62.534
	3	310.440	-31.811	62.534
		-310.440	93.674	-892.407
	4	310.443	-93.674	892.407
		-310.443	100.012	-1208.290
	5	1687.938	-13.330	6.516
		-1687.938	13.330	-39.841
	6	1608.441	-13.330	39.841
		-1608.441	13.330	-216.460
	7	1528.944	-13.330	216.461
		-1528.944	13.330	-393.080
	8	1528.961	-13.330	393.080
		-1528.961	13.330	-423.072
	9	564.607	-122.604	-752.622
		-564.607	76.754	405.175
	10	504.982	-76.754	-405.175
		-504.982	-45.134	272.777
	11	445.359	45.134	-272.777
		-445.359	-106.996	1265.848
	12	445.365	106.996	-1265.848
		-445.365	-113.333	1625.028
	13	101.921	310.465	1208.291
		-101.921	-290.590	-456.965
	14	101.921	290.590	456.964
		-101.921	-116.766	2577.882
	15	101.921	-235.234	-2577.882
		-101.921	420.013	-2836.848
ROOF SLAB	16	101.921	-420.024	2836.813
		-101.921	435.924	-3692.745
	17	115.244	569.367	4115.594
		-115.244	-550.467	-2995.761
	18	115.244	550.442	2995.765
		-115.244	-322.644	4249.024
	19	115.244	-205.355	-4249.023
		-115.244	421.760	-541.191
	20	115.244	-421.698	541.220
		-115.244	445.323	-1624.981
	21	171.677	-429.676	-752.196
		-171.677	352.803	-225.558
	22	171.677	-352.833	225.552
		-171.677	42.300	-2291.154
	23	171.677	-42.299	2291.149
		-171.677	-109.594	-2126.888

1022	24	171.678	109.599	2126.890
1023		-171.678	-275.018	-1172.528
1024	25	171.678	275.017	1172.531
1025		-171.678	-730.943	4512.125
1026	26	171.678	731.023	-4512.039
1027		-171.678	-818.656	6061.597
1028	27	158.351	-869.213	-6067.979
1029		-158.351	781.289	4417.556
1030	28	158.353	-781.300	-4417.526
1031		-158.353	310.992	-1787.205
1032	29	158.353	-310.988	1787.211
1033		-158.353	129.509	-2884.566
1034	30	158.353	-129.507	2884.573
1035		-158.353	-47.734	-3089.485
1036	31	158.353	47.731	3089.487
1037		-158.353	-454.416	-520.018
1038	32	158.353	454.457	519.988
1039		-158.353	-564.632	752.594

CASE 1

ELEMENTS ON AN ELASTIC FOUNDATION

ELEMENT	SPRING I	PRESSURE J
21	.461960E+02	.447916E+02
22	.447916E+02	.413525E+02
23	.413525E+02	.425292E+02
24	.425292E+02	.455592E+02
25	.455592E+02	.539368E+02
26	.539368E+02	.543279E+02
27	.543279E+02	.540235E+02
28	.540235E+02	.463496E+02
29	.463496E+02	.438431E+02
30	.438431E+02	.433775E+02
31	.433775E+02	.489138E+02
32	.489138E+02	.508961E+02

SUBJECT PARK RIVER - JUNCTION STRUCTURE

COMPUTATION BOX CONDUIT AT ENTRANCE TO SECT. 4 (OUTPUT CASE 2)

COMPUTED BY WTH CHECKED BY DATE 9 OCT 79

NODAL POINT DISPLACEMENTS

NODE	X-DISPL	Y-DISPL	ROT-DISPL
1	-.44761E-03	-.27174E-01	.32446E-03
2	-.13863E-02	-.27267E-01	.22297E-03
3	-.29283E-02	-.27600E-01	.63399E-04
4	-.12159E-02	-.27890E-01	-.49403E-03
5	.70900E-03	-.27956E-01	-.69976E-03
6	.69919E-03	-.29765E-01	-.74059E-03
7	.63160E-03	-.38012E-01	-.83244E-04
8	.55965E-03	-.34351E-01	.85962E-04
9	-.46131E-03	-.26348E-01	.33015E-03
10	-.52456E-03	-.24325E-01	-.17417E-04
11	-.55469E-03	-.25017E-01	-.25660E-03
12	-.58481E-03	-.26800E-01	-.44073E-03
13	-.65409E-03	-.31728E-01	-.20768E-03
14	-.66614E-03	-.31958E-01	-.12601E-04
15	-.63483E-03	-.32162E-01	-.12636E-04
16	-.37545E-03	-.33197E-01	-.31854E-04
17	.35076E-03	-.34184E-01	-.83100E-04
18	.55093E-03	-.34352E-01	-.94984E-04
19	.54178E-03	-.34738E-01	-.28145E-03
20	.46627E-03	-.41048E-01	.66923E-05
21	.39533E-03	-.32715E-01	.78419E-03
22	.38503E-03	-.30802E-01	.73884E-03
23	-.67780E-03	-.31779E-01	.18169E-03
24	-.74485E-03	-.27264E-01	.38902E-03
25	-.77400E-03	-.25790E-01	.18445E-03
26	-.80315E-03	-.25516E-01	-.77943E-04
27	-.86437E-03	-.28773E-01	-.46487E-03
28	-.87762E-03	-.29939E-01	-.46112E-03
29	.53991E-03	-.30040E-01	-.35940E-03
30	.35564E-02	-.30406E-01	-.15691E-03
31	.23958E-02	-.30729E-01	.50827E-03

END ACTIONS WITH ELEMENT LOADS

ELEMENT	AXIAL I AXIAL J	SHEAR I SHEAR J	MOMENT I MOMENT J
1	509.135	129.926	769.747
	-509.135	-84.076	-396.674
2	449.511	84.076	396.675
	-449.511	37.812	-174.584
3	389.888	-37.812	174.584
	-389.888	99.674	-1077.961
4	389.893	-99.674	1077.961
	-389.893	106.012	-1413.346
5	1798.254	-5.347	-6.278
	-1798.254	5.347	-7.090

SUBJECT PARK RIVER - JUNCTION STRUCTURE

COMPUTATION BOX CONDUIT AT ENTRANCE TO SECT. 4 (OUTPUT CASE 1)

COMPUTED BY WJH CHECKED BY DATE 2 OCT. 74

392		6	1718.760	-5.348	7.091
393			-1718.760	5.348	-77.950
394		7	1639.262	-5.348	77.950
395			-1639.262	5.348	-148.809
396		8	1639.273	-5.348	148.809
397			-1639.273	5.348	-160.842
398		9	553.504	-124.581	-761.667
399			-553.504	78.731	407.301
400		10	493.877	-78.731	-407.301
401			-493.877	-43.157	250.685
402		11	434.253	43.157	-250.684
403			-434.253	-105.019	1219.537
404		12	434.258	105.019	-1219.538
405			-434.258	-111.356	1572.292
406		13	107.922	389.933	1413.342
407			-107.922	-366.308	-468.082
408		14	107.922	366.293	468.073
409			-107.922	-158.078	3446.748
410	ROOF	15	107.922	-294.922	-3446.747
411			-107.922	516.244	-3256.270
412	SLAB	16	107.922	-516.220	3256.255
412			-107.922	535.120	-4307.596
413		17	113.267	580.513	4468.265
414			-113.267	-561.613	-3326.140
415		18	113.267	561.548	3326.186
416			-113.267	-333.750	4101.855
417		19	113.267	-194.249	-4101.855
418			-113.267	410.654	-516.219
419		20	113.267	-410.571	516.280
420			-113.267	434.196	-1572.222
421		21	165.676	-509.098	-769.736
422			-165.676	414.150	-383.591
423		22	165.676	-414.167	383.587
424			-165.676	46.089	-2767.142
425		23	165.676	-46.088	2767.133
426			-165.676	-125.267	-2571.645
427		24	165.676	125.271	2571.645
428			-165.676	-307.338	-1496.463
429		25	165.676	307.335	1496.470
430			-165.676	-797.552	4759.811
431		26	165.676	797.662	-4759.736
432			-165.676	-890.936	6448.179
433		27	160.330	907.272	-6441.778
434			-160.330	813.857	4720.722
435		28	160.330	-813.903	-4720.684
436			-160.330	319.123	-1705.023
437		29	160.331	-319.119	1705.028
438			-160.331	131.867	-2827.242
439		30	160.331	-131.863	2827.245
440			-160.331	-47.756	-3036.534
441		31	160.331	47.755	3036.535
442			-160.331	-447.490	-488.638
443		32	160.331	447.553	488.648
444			-160.331	-553.521	761.693
445					

SUBJECT PARK RIVER - JUNCTION STRUCTURE

COMPUTATION REINF. FOR 4'1/2' x 10' BEAM AT ENTRANCE TO SECT 4

COMPUTED BY WJH CHECKED BY DATE 9 OCT. 74

POSITIVE STEEL

CASE 3

$M_{MAX\ (RIGHT\ SPAN)} = 4249^{1-K}$ $b = 10'$

$N = 115^K$ $S_s = 20$ ksi $S_y = 40$ ksi $S'_c = 4000$

$S_c = .45\ S'_c = 1800$ $k = 324$ $a = 1.44$ $j = .86$

ASSUME 2 ROWS OF #11'S W/#5 STIRRUP

$d = 54'' - 4 - .625 - 1.41 - 1.41/2 = 47.26''$ $d'' = 27 - 6.74 = 20.26''$

CHECK CONCRETE CAPACITY

$d = \sqrt{4249/(.324 \times 10)} = 36.2'' > 47.26''$ O.K.

$e = \frac{12M}{N} + d'' = \frac{12(4249)}{115} + 20.26 = 463.6$

$E = 38.6$

$e/d = 463.6/47.26 = 9.81$

$i = \frac{1}{1 - j d/e} = \frac{1}{1 - .86(47.26)/463.6} = 1.1$

$F = bd^2/12000 = 120(47.26)^2/12000 = 22.3$

$KF = 7225$ $NE = 4439$ $NE < KF$

$\therefore$ NO COMP. STEEL

$A_s = \frac{NE}{adi} = \frac{4439}{1.44 \times 47.26 \times 1.1} = 59.3\ \square''$

$M_{MAX\ (LEFT\ SPAN)} = 3613^{1-K}$ $b = 10'$

$N = 110^K$

$e = \frac{12(3613)}{110} + 20.26 = 414.4$ $E = 34.5$

$i = 1.1$ $e/d = 8.77$

$KF = 7225$ $NE = 3795$ $KF > NE$

$\therefore$ NO COMP. STEEL

$A_s = \frac{NE}{adi} = \frac{3795}{1.44 \times 47.26 \times 1.1} = 50.7\ \square''$

SUBJECT PARK RIVER - JUNCTION STRUCTURE

COMPUTATION REINF. - 4½' x 10' BEAM AT ENTRANCE TO SECT. 4

COMPUTED BY WJH CHECKED BY DATE 9 OCT 79

NEGATIVE STEEL

REINF. (OVER CENTER PIER) La = 4'

CASE 1

ASSUME 2 ROWS OF #11's

$M_{(LEFT)} = 4308^{'-K}$

$N = 107.9 \quad V = 535^{K}$

$d = 47.26 \quad d'' = 20.26$

$M = 4308 - 1/3\,(535)(4)$

$= 3595$

$e = \frac{12(3595)}{107.9} + 20.26$

$= 420.1$

$E = 35.01$

$j = 1/(1 - .86(47.26)/420.1)$

$= 1.107$

$F = 22.3$

$KF = 7225 \quad KF > NE$

$NE = 3777.6 \quad \therefore$ NO COMP. STEEL

$A_s = \frac{3777.6}{1.44 \times 47.26 \times 1.107}$

$= 50.14^{\#''}$

$M_{(RIGHT)} = 4468^{'-K}$

$N = 113.3 \quad V = 581^{K}$

$d = 47.26 \quad d'' = 20.26$

$M = 4468 - 1/3\,(581)(4)$

$= 3693$

$e = \frac{12(3693)}{113.3} + 20.26$

$= 411.4$

$E = 34.28$

$j = 1/(1 - .86(47.26)/411.4)$

$= 1.11$

$F = 22.3$

$KF = 7225 \quad KF > NE$

$NE = 3883.9 \quad \therefore$ NO COMP STEEL

$A_s = \frac{3883.9}{1.44 \times 47.26 \times 1.11}$

$= 51.4^{\#''} \leftarrow$ USE

CHECK NEGATIVE STEEL IN WALLS AT TOP
(IT MAY CONTROL OVER SLAB STEEL)
WALLS ARE 3' WIDE

REINF. (NORTH WALL)
ASSUME CRITICAL SECTION IS AT BOTTOM OF HAUNCH

CASE 6 $\quad M = 1606^{'-K} \quad N = 475^{K} \quad V = 227^{K}$

$d = 36 - 4 - .7 = 31.3'' \quad d'' = 13.3''$

$M = 1606 + 1/6\,(227)(4.5) = 1776^{'-K}$

$e = \frac{12(1776)}{475} + 13.3 = 58.17 \quad E = 4.85$

$j = 1/(1 - .86(31.3)/58.17) = 1.86$

$F = bd^2/12000 = 120\,(31.3)^2/12000 = 9.797$

SUBJECT PARK RIVER – JUNCTION STRUCTURE

COMPUTATION REINF. – 4½' x 10' BEAM AT ENTRANCE TO SECT. 4

COMPUTED BY WJH CHECKED BY DATE 9 OCT 79

$KF = 324 \times 9.797 = 3174$ $NE < KF$

$NE = 475 \times 4.85 = 2303.8$ ∴ NO COMP. STEEL

$$A_s = \frac{2303.8}{1.44 \times 31.3 \times 1.86} = \underline{\underline{27.48}}\ \square'' \leftarrow$$

CASES 4 & 5 AREN'T AS CRITICAL

<u>REINF. (SOUTH WALL)</u>

CASE 5

$M = 1469'^{-K}$ $N = 432$ $V = 221.8$

$d = 31.3$ $d'' = 13.3$

$M = 1469 + 1/6\,(4.5)(221.8) = 1635'^{-K}$

$$e = \frac{12(1635)}{432} + 13.3 = 58.72$$

$E = 58.72/12 = 4.89$

$j = 1/(1 - .86(31.3)/58.72)$

$= 1.846$

$F = 9.797$

$KF = 3174$ $KF > NE$

$NE = 432 \times 4.89 = 2112.5$ ∴ NO COMP. STEEL

$$A_s = \frac{2112.5}{1.44 \times 31.3 \times 1.846} = \underline{\underline{25.39}}\ \square'' \leftarrow$$

CASES 4 & 6 AREN'T AS CRITICAL

** WALL STEEL CONTROLS OVER SLAB STEEL AT CORNERS

SINCE STEEL AREAS FOR BOTH SIDES ARE CLOSE I'LL USE THE LARGEST AND MAKE STEEL SYMMETRICAL.

SUBJECT PARK RIVER - JUNCTION STRUCTURE

COMPUTATION REINF. - 4½' x 10' BEAM AT ENTRANCE TO SECT. 4

COMPUTED BY WJH CHECKED BY DATE 9 OCT. 74

RESULTS

POSITIVE STEEL

59.3/1.56 = 38.2-#11'S
USE 2 ROW OF #11 BARS
22 BARS PER ROW

NEGATIVE STEEL
(OVER CENTER PIER)

51.4/1.27 = 40.5-#10'S
USE 2 ROWS OF #10 BARS
22 BARS PER ROW

(OVER END WALLS)

27.48/1.27 = 21.6-#10'S
USE 1 ROW OF 22 #10 BARS
BENT DOWN INTO WALLS

CUTOFF POINTS

POSITIVE STEEL

MOMENT WHERE ONLY 1-ROW OF #11'S NEEDED
$d = 54 - 4 - .625 - 1.41/2$
$= 48.67$
$22 \times 1.56 \times 1.44 \times 48.67 = 2405'^{-K}$

MOMENT WHERE ONLY 8-#11'S ARE NEEDED
$8 \times 1.56 \times 1.44 \times 48.67 = 875'^{-K}$

NEGATIVE STEEL

MOMENT WHERE ONLY 1-ROW OF #10'S NEEDED
$d = 54 - 4 - .625 - 1.27/2$
$= 48.74$
$22 \times 1.27 \times 1.44 \times 48.74 = 1961'^{-K}$

MOMENT WHERE 8-#10'S ARE NEEDED
$1961/22 \times 8 = 713'^{-K}$

SHEAR

$v = V_{max}/bd = 525/(120 \times 47.26) = 92.5$ PSI $> 1.1\sqrt{f'_c} = 70$ PSI
BUT O.K. W/ STIRRUPS $92.5 < 5\sqrt{f'_c} = 316$ PSI

SUBJECT PARK RIVER - JUNCTION STRUCTURE

COMPUTATION REINF. — BASE SLAB

COMPUTED BY WOTH CHECKED BY DATE 9 OCT 74

BASE SLAB (5' DEEP)

UNDER CENTER PIER

CASE 1 $M_{max} = 6441$ $N = 160.3$ $V = 907$

$D = 60''$ ASSUME

$d = 60 - 6 - 2.257 - 3 = 48.74$

$d' = 30 - 6 - 2.257 - 3 = 18.74$

$M = 6441 - \frac{1}{3}(4)(907) = 5232'^{-\#}$

$e = \frac{12(5232)}{160.3} + 18.74 = 410.4$ $E = 34.2$

$i = 1/(1 - .86(48.74)/410.4) = 1.114$

$F = bd^2/12000 = 120(48.74)^2/12000 = 23.76$

$KF = 324 \times 23.76 = 7698$ $KF > NE$

$NE = 160.3 \times 34.2 = 5482.3$ ∴ NO COMP. STEEL

$A_s = \frac{5482.3}{1.44 \times 48.74 \times 1.114} = 70.12^{\square''}$

$70.12/4 = 17.53$ - #18'S

USE 2 ROWS OF #18'S

9 PER ROW EQUALLY SPACED

OR ~ #18 @ 13 1/2"

BUT USE #18 @ 12"

NEGATIVE STEEL AT END WALLS

CASE 4 $M = 1624$ $N = 421$ $V = 539$

$d = 72 - 6 - .7 = 65.3$ $d'' = 29.3''$

$M = 1624 - 539 = 1085$

$e = \frac{12(1085)}{421} + 29.3 = 60.2$ $E = 5.02$

$i = 1/(1 - .86(65.3)/60.2) = 14.9$

$F = 120(65.3)^2/12000 = 42.64$

$KF = 13815$ $KF >> NE$ ∴ NO COMP. STEEL

$NE = 2113.4$

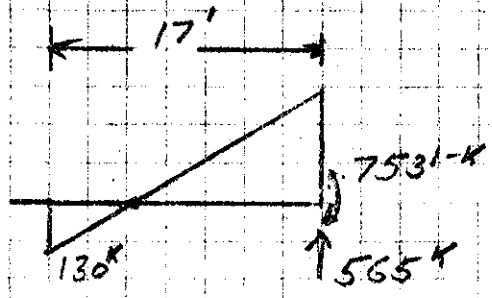
$A_s = \frac{2113.4}{1.44 \times 65.3 \times 14.9} = 1.56^{\square''} <<<$ MIN.

USE #8'S @ 10"

SUBJECT PARK RIVER - JUNCTION STRUCTURE
COMPUTATION REINF. - BASE SLAB
COMPUTED BY WJH CHECKED BY DATE 9 OCT. 74

POSITIVE STEEL

CASE 3



$130 + 565 = 695$

$695/17 = 40.88^{K/'}$

$565/40.88 = 13.82$

$M_{MAX.} = 753 + 40.88(13.82)^2/2 - 565(13.82)$

$= 3151\ ^{'-K}$

ASSUME #18 BARS

$M = 3151 \quad N = 158.4^K \quad d = 60 - 6 - 2.257/2 = 52.87$

$d'' = 22.87$

$e = \frac{12(3151)}{158.4} + 22.87$

$= 261.6 \quad E = 21.8$

$i^\circ = 1/(1 - .86(52.87)/261.6)$

$= 1.21$

$F = bd^2/12000 = 120(52.87)^2/12000 = 27.95$

$KF = 324 \times 27.95 = 9056. \quad KF > NE$

$NE = 158.4 \times 21.8 = 3453.1 \quad$ NO COMP. STEEL

$A_s = \frac{3453.1}{1.44 \times 52.87 \times 1.21} = 37.48^{\square''}$

$37.48/4 = 9.37$ - #18'S

USE 10-#18 EQUALLY SPACED
OR ~ #18 @ 12

SHEAR

$v = V_{MAX}/bd \quad v = 165.7^K$

CASE 1 $V_{AT\ FACE\ OF\ CENTER\ PIER} = 798^K$

$V_{12.5'\ FROM\ FACE} = 307^K$

$\therefore (798 - 307)/12.5 = 39.28$

ASSUME $d = 48.74 \therefore 798 - 39.28 \times 48.74/12 = 638^K$

$638/(120 \times 48.74) = 109.1$ PSI > 70 PSI N.G.

NEED $638/(120 \times 70) = \sim 76'' + \sim 8''$ (COVER) ← ASSUME #18 WILL BE BENT

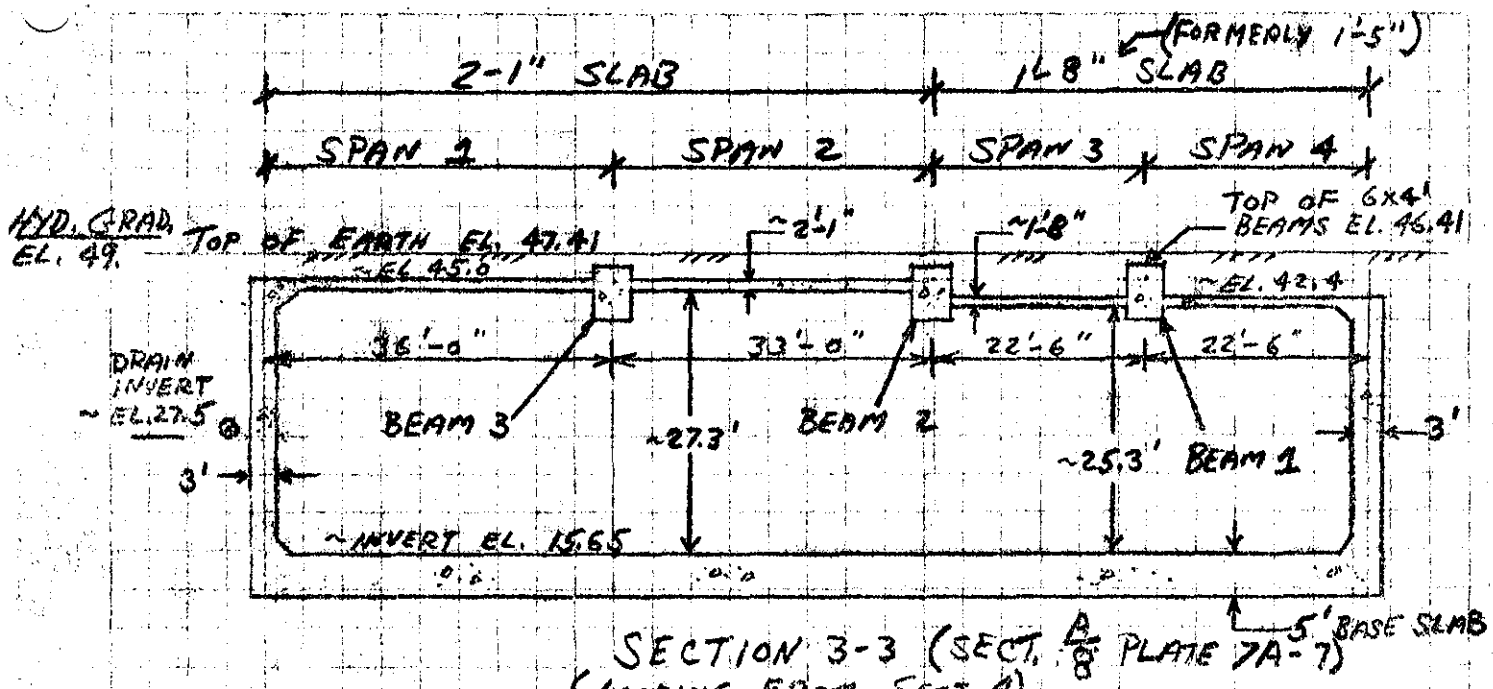
$= 84''/12 = 7'-0''$

ADD 2'-0" HAUNCH 6' EITHER SIDE OF ℄ OF CENTER PIER

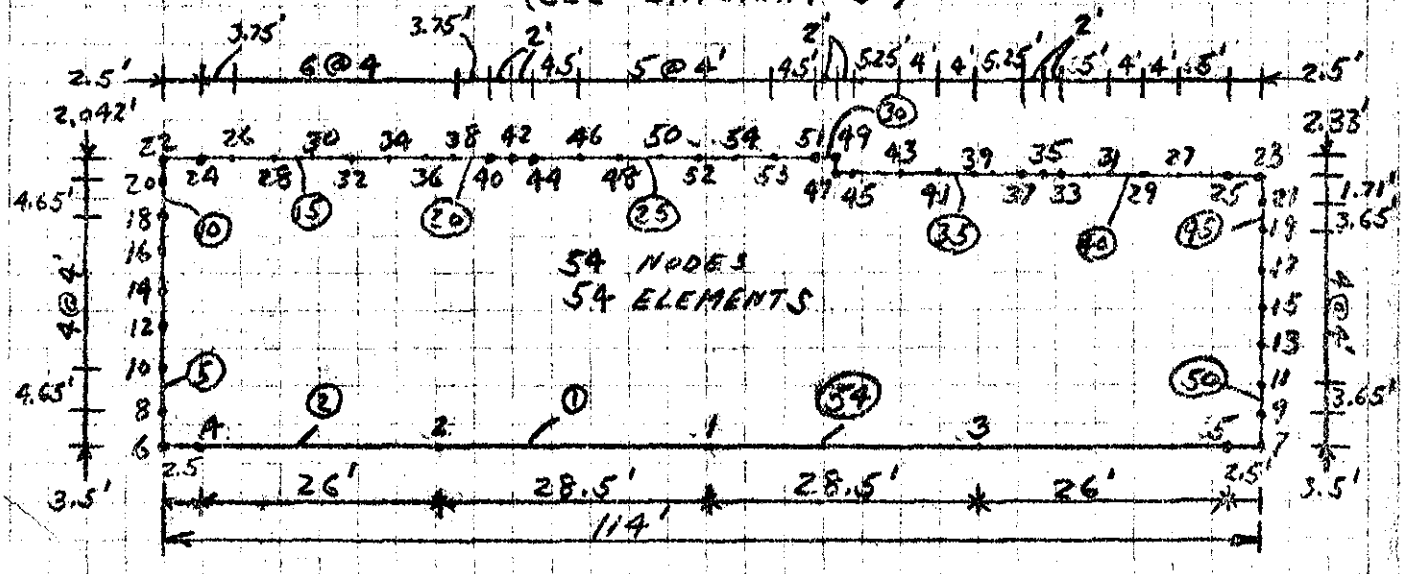
SUBJECT PARK RIVER – JUNCTION STRUCTURE

COMPUTATION SLAB DESIGN USING A FULL CROSS-SECTION (A/8)

COMPUTED BY WTH CHECKED BY DATE 13 SEPT 74

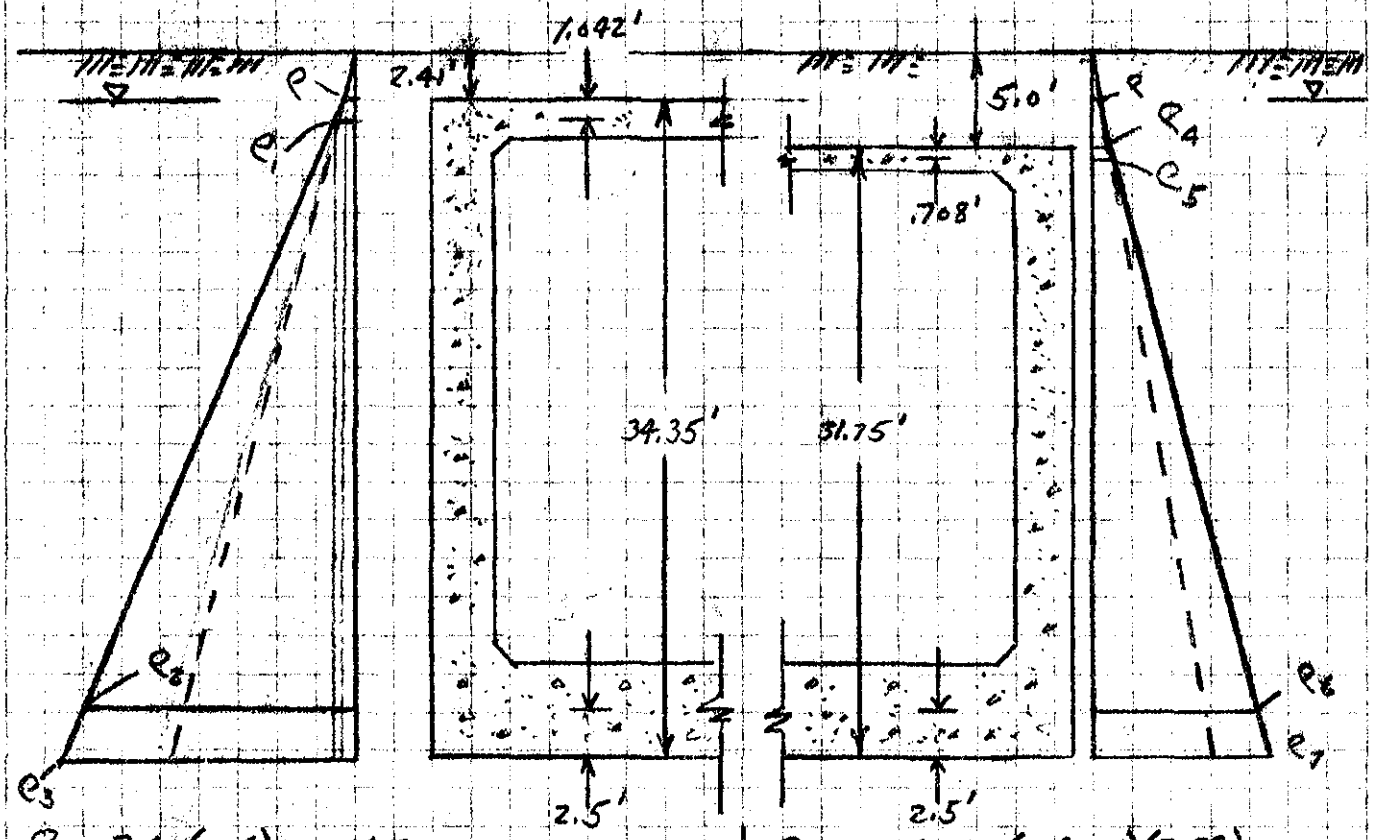


SECTION 3-3 (SECT. A/8 PLATE 7A-7)
(LOOKING FROM SECT. 4)
(SEE DIAGRAM "D")



SUBJECT PARK RIVER LOCAL PROTECTION – JUNCTION STRUCTURE 74/
COMPUTATION WALL & ROOF SLAB DESIGN WITH
COMPUTED BY _____ CHECKED BY _____ DATE 17 SEPT 74

$K_{AT\ REST} = .5 \quad K\gamma_{SAT} = 60 \quad \gamma_{H_2O} + K\gamma_{BUOY} = 92.5$



$P = 2.41(.06) = .145$

$P_1 = .145 + .0925(1.042) = \underline{.241}$

$P_T = (.145 + .241)/2 \times 1.042$
$= .201^K$ (CONCENTRATED LOAD)

$P_2 = .145 + .0925(31.85) = \underline{3.091}$

$P_3 = .145 + .0925(34.35) = 3.322$

$P_B = (3.091 + 3.322)/2 \times 2.5$
$= 8.016^K$ (CONCENTRATED LOAD)

SURCHARGE $.5 \times \overset{L.L.}{.450} = .225^{K/'}$

$\therefore .201 + (.225 \times 1.042) = .435^K$
PT. LOAD AT NODE 22

$8.016 + (.225 \times 2.5) = 8.579^K$
PT. LOAD AT NODE 6

$P_4 = .145 + (.0925)(2.59)$
$= .385$

$(.145 + .385)/2 \times 2.59$
$= .686$ (PT. LOAD AT NODE 49)

$P_5 = .145 + (.0925)(2.59 + .708)$
$= \underline{.450}$

$P_7 = (.385 + .45)/2 \times .708$
$= .296^K$

$P_6 = P_2 = \underline{3.091}$

$P_B = 8.016^K$

SURCHARGE $= .225^{K/'}$

$\therefore .686 + .225(2.59) = 1.27^K =$
PT. LOAD AT NODE 49

$.296 + .225(.708) = .455^K =$
PT. LOAD AT NODE 23

$8.016 + .225(2.5) = 8.579^K =$
PT. LOAD AT NODE 7

NO WATER INSIDE OR OUT ; FULL D.L. + L.L. ; $K_A = .33$
SURCHARGE (.3 × L.L.) $K\gamma_{SAT} = 39.6$
CASE III $\gamma_{H_2O} + K\gamma_{BUOY} = 82.3$

$\rho = 2.41 \times .04 = .096$
$\rho_1 = (2.41 + 1.042)\ .04 = .138$
$P_T = (.096 + .138)/2 \times 1.042 = .122^K$

$\rho_2 = (2.41 + 31.85)\ .04 = 1.37$
$\rho_3 = (2.41 + 34.35)\ .04 = 1.47$
$P_B = 1.42 \times 2.5 = 3.55^K$

SURCHARGE $= .33 \times .450 = .149^{K/'}$
$\therefore\ .122 + (.149 \times 1.042) = .277^K$ (PT. LOAD AT NODE 22)

$3.55 + (.149 \times 2.5) = 3.923^K$ (PT. LOAD AT NODE 6)

$\rho_4 = 5 \times .04 = .2$
$(.096 + .2)/2 \times 2.59 = .383^K$

$\rho_5 = (5.0 + .708)\ .04 = .228$
$P_T = (.2 + .228)/2 \times .708 = .152^K$

$\rho_6 = \rho_2 = 1.37$
$\rho_7 = \rho_3 = 1.47$
$\therefore\ P_B = 3.55^K$
SURCHARGE $= .149^{K/'}$
$\therefore\ .383 + .149(2.59) = .769^K$ (PT. LOAD AT NODE 49)

$.152 + .149(.708) = .257^K$ (PT. LOAD AT NODE 23)

$3.55 + .149(2.5) = 3.923^K$ (PT. LOAD AT NODE 7)

BURSTING PRESSURES - CASE IV
ASSUME H.G. AT ELEV. 49
SOFFIT ELEV. ON 2'-1" SLAB = 42.92 $(49-42.92).0625 = .38^{K/'}$
" " " 1'-5" " = 40.98 $(49-40.98).0625 = .50^{K/'}$

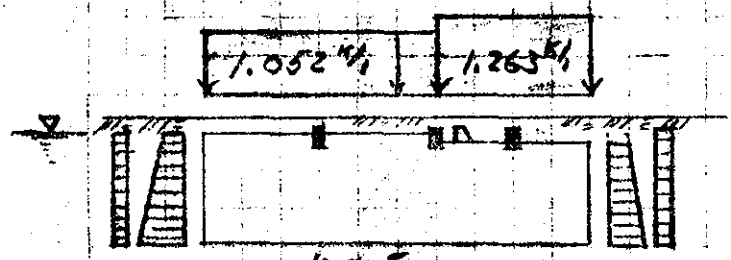
AVE. TOP OF BASE SLAB = ELEV. 15.65

$49-15.65).0625 = 2.084^{K/'}$

DEFLECTIONS OF BEAMS
USE DEFLECTIONS FROM OLD SLAB DESIGN P. 25
(FULL D.L. + L.L. ON BEAM DESIGN)
REDUCE DEFL. 15% FOR ONLY DEAD LOAD
" BASE ON 150# L.L.

SUBJECT PARK RIVER - JUNCTION STRUCTURE
COMPUTATION SLAB & WALL DESIGN - CASE LOADINGS
COMPUTED BY WTH CHECKED BY DATE 17 SEPT 74

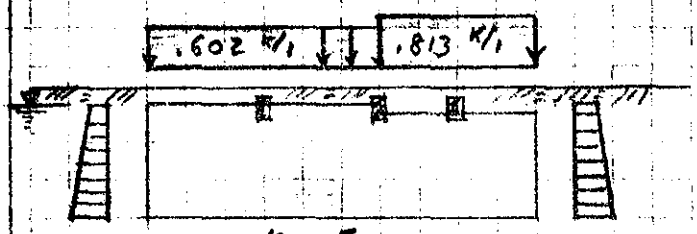
CASE I



$K_o = .5$

WATER TO TOP OF BOX; FULL D.L.+L.L. SURCHARGE (.5 x L.L.); NO WATER INSIDE

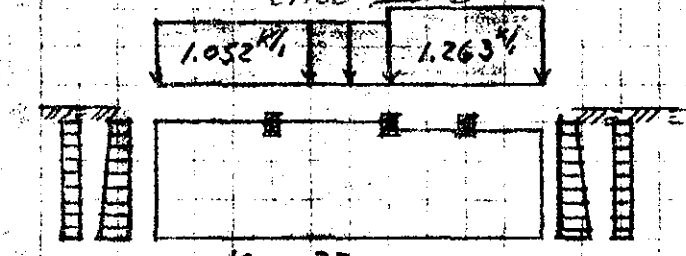
CASE II



$K_o = .5$

WATER TO TOP OF BOX; FULL D.L.; NO L.L.; NO WATER INSIDE

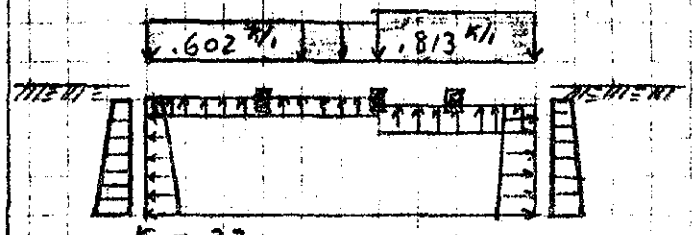
CASE III A W/ SURCH.
CASE III B W/O "



$K_A = .33$

NO WATER INSIDE OR OUT; FULL D.L.+ L.L.; SURCHARGE (.3 x L.L.)

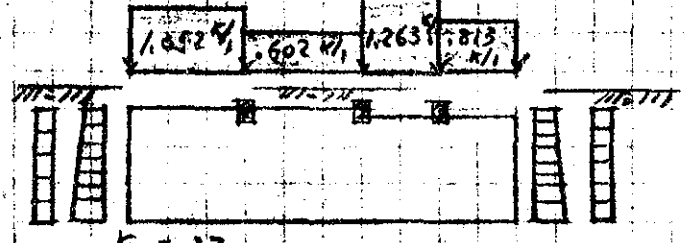
CASE IV



$K_A = .33$

BURSTING PRESSURES; D.L. ONLY

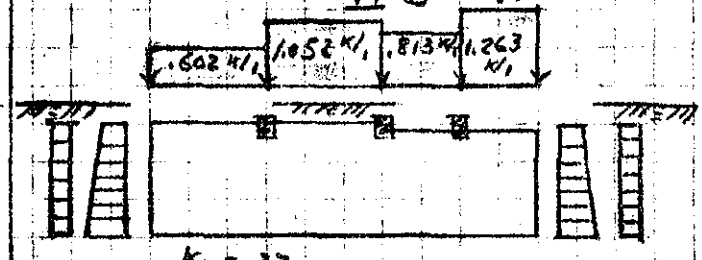
CASE V A W/ SURCH.
V B W/O "



$K_A = .33$

NO WATER INSIDE OR OUT; FULL D.L.+ L.L. ON ALTERNATE SPANS

CASE VI A W/ SURCH.
VI B W/O "



$K_A = .33$

NO WATER INSIDE OR OUT; FULL D.L.+ L.L. ON ALTERNATE SPANS

SUBJECT PARK RIVER - JUNCTION STRUCTURE

COMPUTATION ~~SLAB~~ WALL & ROOF DESIGN SECT 3-3 - OUTPUT CASE 1

COMPUTED BY WJH CHECKED BY DATE 20 SEPT 74

	ELEMENT	AXIAL I AXIAL J	SHEAR I SHEAR J	MOMENT I MOMENT J
452				
453				
454				
455				
456	1	0.	-.011	-77.331
457		0.	.011	77.076
458	2	0.	-.011	-77.650
459		0.	.011	77.331
460	3	0.	-.011	-77.637
461		0.	.011	77.550
462	4	20.309	24.[illegible]	77.687
463		-20.309	-23.057	22.003
464	5	20.309	23.057	-22.003
465		-20.309	-10.141	93.414
466	6	20.309	10.141	-93.414
467		-20.309	-.629	119.461
468	7	20.309	.629	-119.461
469		-20.309	7.405	105.417
470	8	20.309	-7.404	-105.417
471		-20.309	13.959	62.196
472	9	20.309	-13.959	-62.196
473		-20.309	19.036	-4.236
474	10	20.309	-19.036	4.236
475		-20.309	23.073	-102.975
476	11	20.309	-23.073	102.975
477		-20.309	24.221	-151.235
478	12	24.656	24.309	151.235
479		-24.656	-17.679	-103.301
480	13	24.656	17.679	103.301
481		-24.656	-13.734	-44.902
482	14	24.656	13.734	44.902
483		-24.656	-9.526	1.613
484	15	24.656	9.526	-1.613
485		-24.656	-5.313	31.305
486	16	24.656	5.313	-31.306
487		-24.656	-1.110	44.161
488	17	24.656	1.110	-44.161
489		-24.656	3.093	40.185
490	18	24.656	-3.093	-40.185
491		-24.656	7.306	19.377
492	19	24.656	-7.306	-19.377
493		-24.656	11.514	-13.262
494	20	24.656	-11.514	13.262
495		-24.656	15.459	-63.336
496	21	24.656	-15.459	63.334
497		-24.656	17.563	-101.354
498	22	24.656	17.274	101.355
499		-24.656	-15.170	-62.412
500	23	24.656	15.163	62.409
501		-24.656	-10.434	-11.306
502	24	24.656	10.434	11.306
503		-24.656	-5.226	21.513
504	25	24.656	5.226	-21.513
505		-24.656	-2.013	33.000

SOUTH WALL

ROOF SLAB

NED FORM 223 — NEW ENGLAND DIVISION — CORPS OF ENGINEERS, U.S. ARMY — PAGE DM7-95

27 Sept 49

SUBJECT: PARK RIVER — JUNCTION STRUCTURE

COMPUTATION: ~~[illegible]~~ WALL & ROOF DESIGN SECT 3-3 — OUTPUT WITH CASE 1

COMPUTED BY: _____ CHECKED BY: _____ DATE: 20 SEPT 74

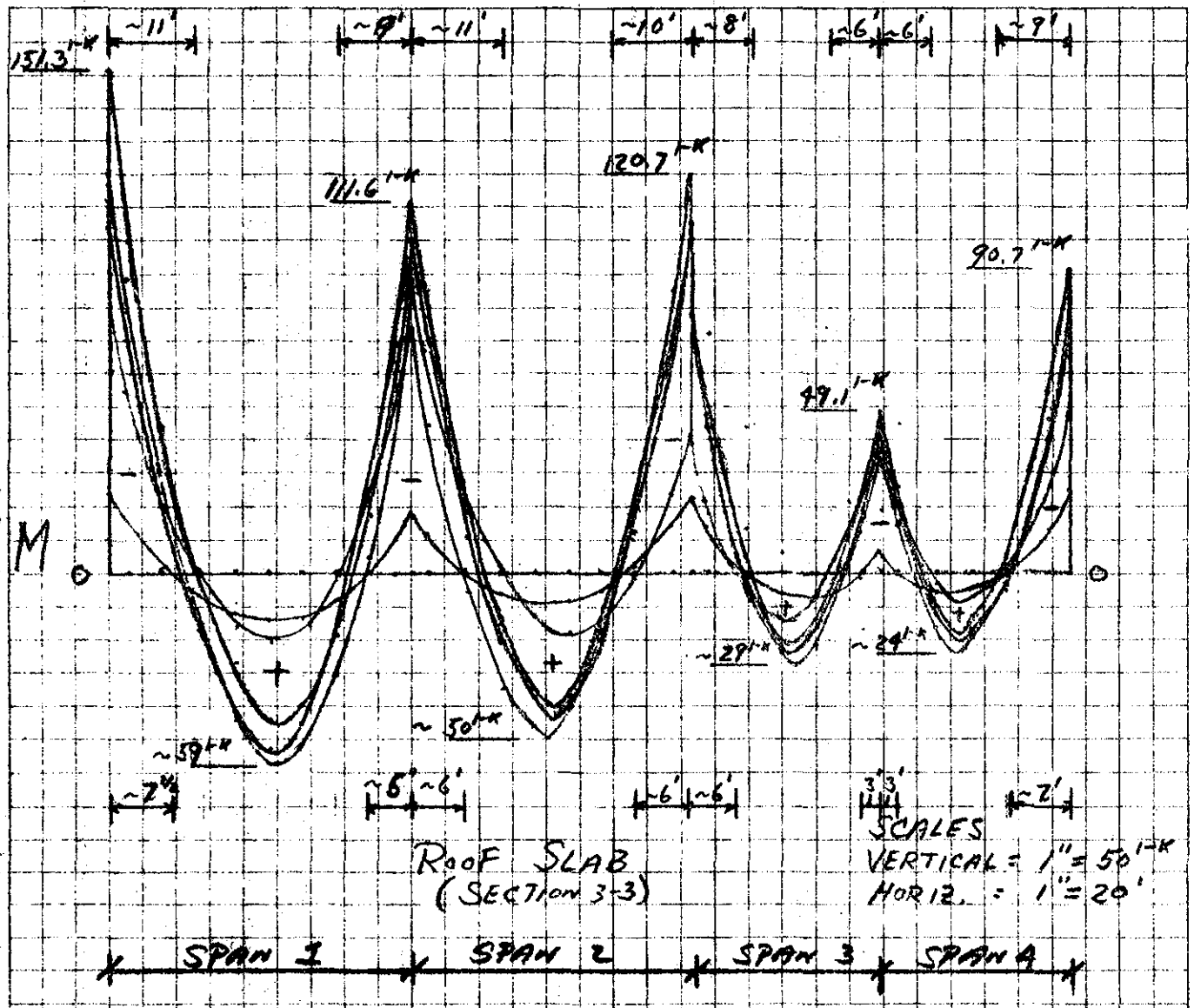
506	ROOF SLAB	26	24.656	2.013	-33.000
507			-24.656	2.190	37.656
508		27	24.656	-2.190	-37.656
509			-24.656	5.393	20.479
510		28	24.656	-5.393	-20.479
511			-24.656	10.606	-13.529
512		29	24.656	-10.606	13.529
513			-24.656	15.340	-71.907
514		30	24.656	-15.343	71.906
515			-24.656	17.447	-104.693
516		31	17.444	13.963	104.694
517			-17.444	-13.963	-72.161
518		32	23.122	15.497	72.163
519			-23.122	-12.971	-43.695
520		33	23.122	12.975	43.696
521			-23.122	-5.344	7.016
522		34	23.122	5.344	-7.016
523			-23.122	-1.292	22.237
524		35	23.122	1.292	-22.237
525			-23.122	3.760	17.351
526		36	23.122	-3.760	-17.351
527			-23.122	10.391	-19.795
528		37	23.122	-10.394	19.792
529			-23.122	12.920	-43.106
530		38	23.122	12.039	43.111
531			-23.122	-9.563	-21.457
532		39	23.122	9.569	21.459
533			-23.122	-3.254	10.601
534		40	23.122	3.254	-10.601
[illegible]			[illegible]	[illegible]	[illegible]
535			-23.122	1.793	13.515
536		41	23.122	-1.798	-13.515
537			-23.122	5.350	-3.730
538		42	23.122	-5.350	3.730
539			-23.122	13.165	-53.315
540		43	23.122	-13.165	53.315
541			-23.122	15.322	-90.673
542		44	15.322	22.569	90.673
543			-16.322	-21.330	-52.973
544		45	16.322	21.330	52.973
545			-15.322	-17.721	18.761
546		46	15.322	17.721	-18.761
547			-15.322	-12.294	79.234
548		47	16.322	12.294	-79.234
549	NORTH WALL		-15.322	-5.334	115.134
550		48	15.322	5.335	-115.134
551			-16.322	3.007	120.333
552		49	16.322	-3.007	-120.333
553			-15.322	12.331	39.102
554		50	15.322	-12.331	-39.102
555			-16.322	23.133	23.551
556		51	16.322	-23.133	-23.551
557			-16.322	34.222	-76.468

NED FORM 225
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U.S. ARMY

PAGE DM7-96

SUBJECT PARK RIVER - JUNCTION STRUCTURE
COMPUTATION MOMENT ENVELOPE FOR ROOF SLAB OF SECTION 3-3
COMPUTED BY WTH CHECKED BY DATE 20 SEPT 74



ROOF SLAB

2'-1" SLAB

POSITIVE STEEL

SPAN 1 $M_{max} \sim 60'^{k}$ CASE 5B

$N = 12.59^{k}$

$d = 25 - 4 - .7 = 20.3''$ $d'' = 7.8''$

$e = 12M/N + d'' = 60(12)/12.59 + 7.8 = 64.99$

$e/d = 3.2$ $E = 5.42$

$k = 324$ $F = .412$

$NE = 12.59 \times 5.42 = 68.24$

$KF = 324 \times .412 = 133.5$

$KF > NE$ ∴ NO COMPRESSION STEEL REQ.

$A_s = 68.24/(1.44 \times 20.3 \times i)$ $i = 1.37$

$= 1.70^{\square''}$

USE #10's @ 10" $A_s = 1.87$

CASES 5A, 3A & 3B WERE ALSO INVESTIGATED AND FOUND TO BE LESS CRITICAL THAN CASE 5B

SPAN 2 $M_{max} \sim 50'^{k}$ CASE 6B

$N = \sim 10^{k}$

$d = 20.3''$ $d'' = 7.8''$

$e = 50(12)/10 + 7.8 = 67.8$

$e/d = 3.34$ $E = 5.65$

$k = 324$ $F = .412$

$NE = 56.5 < 133.5$ ∴ NO COMP. STEEL REQ.

$i = 1.35$

$A_s = 56.5/(1.44 \times 20.3 \times 1.35)$

$= 1.43^{\square''}$

USE #10's @ 10" $A_s = 1.52^{\square''}$

CASES 6A, 3A & 3B WERE ALSO INVESTIGATED

SHEAR $V_{max} \sim 18^{k}$

$18000/12 \times 20.3 = 73.9$ PSI > 70

BUT CLOSE ENOUGH SINCE 450# L.L. IS BEING USED.

SUBJECT PARK RIVER – JUNCTION STRUCTURE
COMPUTATION STEEL FOR 2'-1" ROOF SLAB IN SECTION 3-3
COMPUTED BY WJH CHECKED BY DATE 19 SEPT 79

2'-1" SLAB
NEGATIVE STEEL (AT WALL)
M AT END OF HAUNCH PRESUMED TO BE CRITICAL

CASE 1 $M = 103.8 + 1/6\,(20.369)(3) = 113.95'^{-K}$
$N = 24.656^K$
$d = 20.3''$ $d'' = 7.8''$
$e = 12(113.95)/24.656 + 7.8 = 63.26$
$e/d = 3.12$ $E = 5.27$
$K = 324$ $F = .412$
$NE = 24.656 \times 5.27 = 129.94$
$KF = 324 \times .412 = 133.5$
$KF > NE$ ∴ NO COMP. STEEL REQ.
$j = 1.37$
$A_s = 129.94/(1.44 \times 20.3 \times 1.37)$
$= 3.24^{\square''}$ ← USE

CASE 5A $M = 89.86 + 1/6\,(20.25)(3) = 100'^{-K}$
$N = 14.9^K$ $d = 20.3$ $d'' = 7.8''$
$e = 100(12)/14.9 + 7.8 = 88.34$
$e/d = 4.35$ $E = 7.36$
$NE = 14.9 \times 7.36 = 109.66$
$KF = 133.5$
$KF > NE$ O.K.
$j = 1.25$
$A_s = 109.66/(1.44 \times 20.3 \times 1.25)$
$= 3.0^{\square''}$

USE #11 @ 5½" $A_s = 3.4^{\square''} > 3.24^{\square''}$

MAKE NEG. STEEL AT TOP OF SOUTH WALL THE SAME.

SUBJECT PARK RIVER – JUNCTION STRUCTURE
COMPUTATION STEEL FOR 1'-5" ROOF SLAB OF SECT. 3-3
COMPUTED BY WJH CHECKED BY DATE 20 SEPT 79

1'-5" SLAB

POSITIVE STEEL — SPAN 3

CASE 5B $M = -29'^{-k}$ $N = 9.59$ $d = 17-4-.7 = 12.3$

$d'' = 3.8''$

$e = 29(12)/9.59 + 3.8$
$= 40.09$

$e/d = 3.26$ $E = 3.34$

$K = 324$ $F = .152$

$NE = 9.59 \times 3.34 = 32.03$

$KF = 324 \times .152 = 49.25$

$KF > NE \therefore$ No COMP. STEEL REQ.

$i = 1.36$

$A_s = 32.03/(1.44 \times 12.3 \times 1.36)$
$= 1.33 \square''$ ←

CASE 1 $M = -29'^{-k}$ $N = 23.122$

$e = 29(12)/23.122 + 3.8$
$= 18.85$

$e/d = 1.53$ $E = 1.57$

$NE = 18.85 \times 1.57 = 29.6 < 49.25$ O.K.

$i = 2.28$

$A_s = 29.6/(1.44 \times 12.3 \times 2.28) = .73 \square''$

USE #11's @ 12" $A_s = 1.56 \square''$

SPAN 4

CASE 6B $M = -24'^{-k}$ $N = 11.016^k$

$e = 24(12)/11.016 + 3.8$
$= 29.94$ $e/d = 2.43$ $E = 2.5$

$NE = 11.016 \times 2.5 = 27.54 < 49.25$ O.K.

$i = 1.54$

$A_s = 27.54/(1.44 \times 12.3 \times 1.54)$
$= 1.0 \square''$

USE #10's @ 12" $A_s = 1.27 \square''$

SUBJECT PARK RIVER – JUNCTION STRUCTURE

COMPUTATION STEEL FOR 1'-5" ROOF SLAB OF SECT. 3-3

COMPUTED BY WTH CHECKED BY DATE 20 SEPT 74

NEGATIVE STEEL
(AT WALL)

$d' = 4.7''$

CASE 1 $M = 53.8 + 1/6\,(16.322)\,3$
$= 61.96^{1-k}$ $N = 23.122$
$e = 61.96\,(12)/23.122 + 3.8 = 35.96$
$e/d = 2.92$ $E = 3.0$
$NE = 23.122 \times 3.0 = 69.37$
$KF = 324 \times .152 = 49.25$
$KF < NE$ ∴ COMP. STEEL IS REQ.

RATHER THAN ADD COMPRESSION STEEL TRY INCREASING SLAB DEPTH BY 3" TO 1'-8"
$d = 20 - 4.7 = 15.3''$
$d'' = 10 - 4.7 = 5.3''$
ASSUME THE SAME MOMENT EVEN THOUGH IT MAY INCREASE A SMALL AMOUNT ON THE NEGATIVE SIDE SINCE THE STIFFNESS WOULD INCREASE.

$e = 61.96\,(12)/23.122 + 5.3 = 37.46$
$e/d = 2.45$ $E = 3.12$
$NE = 23.122 \times 3.12 = 72.14$
$KF = 324 \times .234 = 75.8$
$KF > NE$ O.K.
$i = 1.54$
$A_s = 72.14/(1.44 \times 15.3 \times 1.54)$
$= 2.13\ \square''$

USE 11'S @ 8½" $A_s = 2.2\ \square''$
MAKE NEGATIVE STEEL AT TOP OF NORTH WALL THE SAME
(SEE NEXT PAGE)

NEGATIVE STEEL OVER BEAM #3

CASE 3B $M = 25.5 + 1/6\,(13.)\,4 = 34.17^{1-k}$ $N = 10.63^{k}$
$e = 34.17\,(12)/10.63 + 5.3 = 43.87$
$e/d = 2.87$ $E = 3.66$
$NE = 10.63 \times 3.66 = 38.9$ $KF > NE$ O.K.
$KF = 324 \times .234 = 75.8$
$i = 1.43$
$A_s = 38.9/(1.44 \times 15.3 \times 1.43) = 1.23\ \square''$
USE #9'S @ 7½" $A_s = 1.6\ \square''$

27 Sept 49

CORPS OF ENGINEERS, U. S. ARMY

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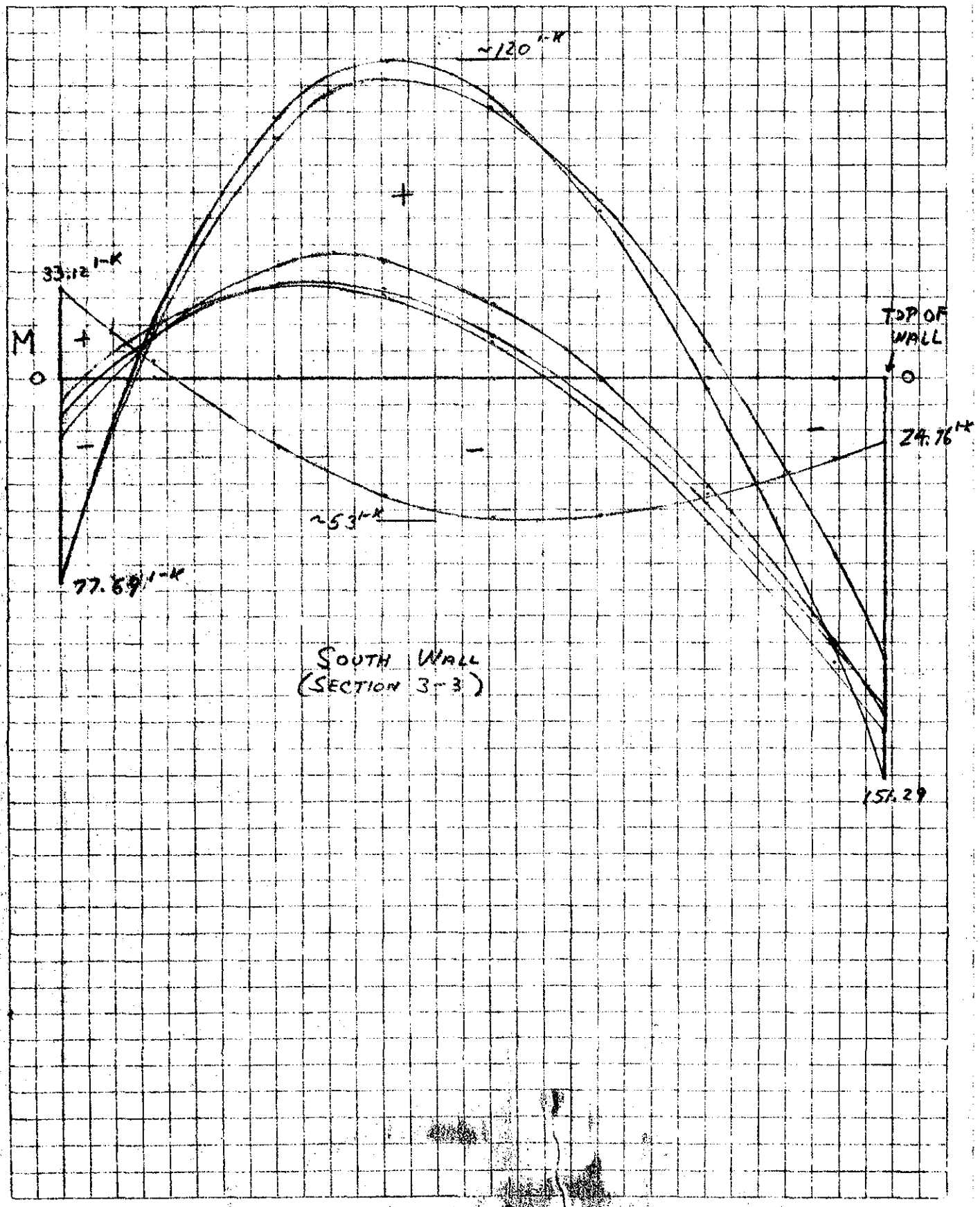
SUBJECT PARK RIVER - JUNCTION STRUCTURE

COMPUTATION MOMENT ENVELOPE FOR SOUTH WALL SECTION 3-3

COMPUTED BY WJH CHECKED BY DATE 20 SEPT 79



SUBJECT PARK RIVER – JUNCTION STRUCTURE
COMPUTATION STEEL FOR WALLS OF SECTION 3-3
COMPUTED BY WJH CHECKED BY DATE 18 SEPT 74

WALLS (SOUTH) (36" WIDE)

GIVEN: $S_s = 20000$ PSI, $n = 8$, $S_c = 1800$ PSI, $S'_c = 4$ KSI

$d = 31.3$ " (ASSUMING #11 BARS)

$d' = 4.7$, $d'' = 13.3$", $b = 12$" $J = .86$

(CASE 1) $M = 120^{1-K}$

$N = 20.61$

$e = \frac{12M}{N} + d'' = \frac{12 \times 120}{20.31} + 13.3 = 84.2$"

$e/d = 84.2/31.3 = 2.69$

$E = e/12 = 7.02'$

$K = .324$ $F = .98$

$NE = 20.31 \times 7.02 = 142.6$

$KF = 324 \times .98 = 317.5$

$NE - KF$ $142.6 - 317.5 = -174.9$

∴ NO COMPRESSION REINF. REQ.

$d = \sqrt{120/.324} = 19.25 < 31.3$ O.K.

$A_s = NE/qdi$ $i = 1.47$

$= 142.6/(1.44 \times 31.3 \times 1.47)$

$= 2.15^{\square''}$

USE #10 @ 6½" POSITIVE STEEL (INSIDE FACE)

$A_s = 2.35^{\square''}$

CASE 2 CHECKED ALSO $A_s = 2.15^{\square''}$

NEGATIVE STEEL

design M $= 151.29 - VLa/3$ (CASE 1)

$= 151.29 - 24.2(2.083)/3$ $N = 20.31^K$

MOMENT $= 126.1$ M AT END OF HAUNCH $= 107.98^{1-K}$

AT FACE OR OF SUPPORT $= 137.2 - 14.6(2.083)/3$ (CASE 5A) ←

$= 127.$ ← $N = 20.25^K$

AT FACE OF SUPPORT WALL IS 4' DEEP < 20.31

CHECK MOMENT AT END OF HAUNCH

$M = 108^{1-K} + 1/6\ VLa = 113.1$

$d = \sqrt{127/.324} = 19.8 < 43.3$" $d = \sqrt{113.1/.324} = 18.68 < 31.3$

SUBJECT PARK RIVER - JUNCTION STRUCTURE
COMPUTATION STEEL FOR WALLS OF SECT 3-3
COMPUTED BY WJH CHECKED BY DATE 18 SEPT 74

AT FACE OF SUPPORT (TOP OF WALL) CASE 5A

$d = 43.3$ $d' = 4.7$ $d'' = 19.3$ $b = 12''$ $J = .86$

$N = 20.25$

$e = 12(127)/20.25 + 19.3 = 94.35$

$e/d = 2.18$ $E = 7.88$

$K = 324$ $F = 1.79$

$NE = 20.25 \times 7.88 = 159.6$

$KF = 324 \times 1.79 = 580$

No COMPRESSION STEEL REQ.

$A_s = NE/(ndi)$ $i = 1.65$

$= 159.6/(1.44 \times 43.3 \times 1.65)$

$= 1.55 \square''$

AT END OF HAUNCH

$d = 31.3$

$e = 12(113.1)/20.25 + 13.3 = 80.32$

$e/d = 2.57$ $E = 6.69$

$K = 324$ $F = .98$

$NE = 20.25 \times 6.69 = 135.47$

$KF = 324 \times .98 = 317.5$

∴ No COMPRESSION STEEL REQ.

$A_s = NE/(ndi)$ $i = 1.50$

$= 135.47/(1.44 \times 31.3 \times 1.50)$

$= 2.07 \square''$

CHECK CASE 5B AT END OF HAUNCH

$M = 108.364 + 6(12.47)(2.083) = 112.69'^k$ $N = 20.108^k$

$d = 31.3''$ $d'' = 13.3''$

$e = 12(112.69)/20.108 + 13.3 = 80.55$

$e/d = 2.57$ $E = 6.71$

$NE = 20.108 \times 6.71 = 134.92 < 317.5$ O.K.

$A_s = NE/(ndi)$ $i = 1.50$

$= 134.92/(1.44 \times 31.3 \times 1.50)$

$= 2.0 \square''$ ← USE

USE #10 @ 7" (BUT SEE WHAT NEG. STEEL FOR TOP SLAB IS)

$A_s = 2.18 \square''$

CASES 1, 3A & 3B ALSO INVESTIGATED & FOUND NOT AS CRITICAL AS CASE 5B

SHEAR $V_{max.}$ (d" dist. FROM SUPPORT FACE) = ~ 21^k

$21000/12 \times 31.3 = 55.9$ PSI < 70 O.K.

SUBJECT: PARK RIVER

COMPUTATION: BASE SLAB SECT 3-3 CASE 3A-W/BOEF

COMPUTED BY: W. Rodger CHECKED BY: DATE: 9 OCT 74

ELEMENT	SPRING PRESSURE I	SPRING PRESSURE J
1	.335251E+00	.321436E+00
2	.208139E+01	.335251E+00
3	.221362E+01	.221362E+01
52	.175343E+01	.134257E+01
53	.370330E+00	.175343E+01
54	.321436E+00	.370330E+00

@ Left Side 2.21 ksf
Right 1.76 ksf

NUMBER OF ITERATIONS = 0

ELEMENT	ZONE	LIMITS OF INTEGRATION (MAX=11)	
1	ODD	0. ,	23.500,
2	ODD	0. ,	26.000,
3	ODD	0. ,	2.500,
52	ODD	0. ,	2.500,
53	ODD	0. ,	26.000,
54	ODD	0. ,	23.500,

** DATE 10/09/74
** TIME 11:33:34
** SECTION 1
** LOADING 1

NODAL POINT DISPLACEMENTS

NODE	X-DISPL	Y-DISPL	ROT-DISPL
1	-.34924E-03	-.37142E-02	-.44115E-05
2	-.17220E-03	-.59074E-02	.14721E-03
3	-.52455E-03	-.51246E-02	-.10365E-03
4	-.13271E-04	-.12046E-01	.21915E-03
5	-.63442E-03	-.13344E-01	-.21136E-03
6	0.	-.12016E-01	.89414E-03
7	-.39347E-03	-.13345E-01	-.13566E-03
8	-.77583E-03	-.13651E-01	.16328E-03
9	-.23456E-03	-.13373E-01	-.68780E-04
10	-.12233E-02	-.13185E-01	.53647E-04
11	-.22017E-03	-.10907E-01	.12431E-04
12	-.13943E-02	-.13151E-01	.33679E-04
13	-.23634E-03	-.13044E-01	.10784E-04
14	-.15437E-02	-.13198E-01	.35261E-04
15	-.26253E-03	-.13931E-01	-.33694E-04
16	-.16327E-02	-.13044E-01	-.33328E-05
17	-.12259E-03	-.11019E-01	-.33396E-04
18	-.14135E-02	-.13291E-01	-.12173E-03
19	-.38132E-04	-.11055E-01	.46235E-05

SUBJECT PARK RIVER

COMPUTATION BASE SLAB SECTION 3-3 CASE 3A W/ BOEF

COMPUTED BY _____ CHECKED BY _____ DATE 9 OCT 79

24	.54035E-03	-.15063E-01	-.32159E-03
25	-.50442E-03	-.12121E-01	.63473E-03
26	.60199E-03	-.10181E-01	-.12491E-03
27	-.43790E-03	-.17573E-01	.12864E-03
28	.56106E-03	-.24083E-01	-.19248E-02
29	-.33469E-03	-.21303E-01	.38375E-03
30	.52013E-03	-.23533E-01	-.39242E-03
31	-.33143E-03	-.24003E-01	.31127E-03
32	.47923E-03	-.31125E-01	-.33763E-03
33	-.26496E-03	-.25363E-01	.46505E-03
34	.43323E-03	-.31639E-01	.11703E-03
35	-.25363E-03	-.26300E-01	.47233E-03
36	.39735E-03	-.33436E-01	.45362E-03
37	-.25239E-03	-.27953E-01	.47964E-03
38	.35542E-03	-.23495E-01	.47372E-03
39	-.13255E-03	-.33345E-01	.54355E-03
40	.31305E-03	-.27275E-01	.46277E-04
41	-.12934E-03	-.31676E-01	-.67053E-04
42	.31094E-03	-.27209E-01	.27515E-04
43	-.76126E-04	-.30251E-01	-.57579E-03
44	.30334E-03	-.27165E-01	.35505E-05
45	-.52835E-05	-.27774E-01	-.42214E-04
46	.25779E-03	-.23502E-01	-.49472E-03
47	0.	-.27700E-01	-.31347E-04
48	.21635E-03	-.30531E-01	-.43350E-03
49	0.	-.27713E-01	.35562E-04
50	.17594E-03	-.32367E-01	-.22540E-03
51	.71055E-05	-.27307E-01	.52681E-04
52	.13501E-03	-.32259E-01	.13134E-03
53	.53159E-04	-.29222E-01	.47294E-03
54	.94079E-04	-.31109E-01	.41353E-03

END ACTIONS WITH ELEMENT LOADS

	ELEMENT	AXIAL I AXIAL J	SHEAR I SHEAR J	MOMENT I MOMENT J
BASE SLAB	1	16.917 -16.917	1.730 -.215	56.865 -13.240
	2	16.917 -16.917	-15.703 -1.730	-55.520 -56.365
	3	16.917 -16.917	-13.197 15.734	-33.079 57.520
	4	19.193 -19.193	16.401 -11.339	33.079 -50.693
	5	19.193 -19.193	11.339 -5.342	50.693 -13.251
	6	19.193 [illegible]	5.343 [illegible]	13.251 [illegible]

SUBJECT PARK RIVER

COMPUTATION BASE SLAB SECT 3-3 CASE 3A - W/ BOEF

COMPUTED BY _____ CHECKED BY _____ DATE 9 OCT 74

9	19.193	-5.397	22.493
	-19.193	8.633	-52.166
10	19.193	-8.633	52.166
	-19.193	10.779	-97.633
11	19.199	-10.779	97.633
	-19.199	11.447	-120.332
12	11.724	19.199	120.332
	-11.724	-16.569	-75.622
13	11.724	16.569	75.622
	-11.724	-12.624	-20.336
14	11.724	12.624	20.336
	-11.724	-3.416	21.194
15	11.724	3.416	-21.194
	-11.724	-4.203	46.441
16	11.724	4.203	-46.441
	-11.724	-.000	54.358
17	11.724	.000	-54.358
	-11.724	4.203	46.442
18	11.724	-4.203	-46.442
	-11.724	3.416	21.195
19	11.724	-3.416	-21.195
	-11.724	12.624	-20.334
20	11.724	-12.624	20.334
	-11.724	16.569	-75.620
21	11.724	-16.561	75.610
	-11.724	13.665	-110.340
22	11.724	17.663	110.337
	-11.724	-15.564	-77.609
23	11.724	15.564	77.602
	-11.724	-10.330	-13.214
24	11.724	10.330	13.214
	-11.724	-5.622	16.692
25	11.724	5.623	-16.692
	-11.724	-2.415	34.766
26	11.724	2.415	-34.766
	-11.724	1.793	36.003
27	11.724	-1.793	-36.008
	-11.724	5.001	20.419
23	11.724	-6.001	-20.419
	-11.724	10.209	-12.002
29	11.724	-10.209	12.002
	-11.724	14.943	-63.595
30	11.724	-14.946	63.600
	-11.724	17.050	-100.591
31	17.043	13.979	100.593
	-17.043	-13.979	-63.023
32	10.363	15.075	63.023
	-10.363	-12.549	-40.396
33	10.363	12.553	40.399
	-10.363	-5.923	3.101
34	10.363	5.923	-3.101
	-10.363	-.371	21.633
35	10.363	.371	-21.633
	-10.363	4.131	15.067

SUBJECT PARK RIVER

COMPUTATION BASE SLAB SECT 3-3 CASE 3M W/ BOEF

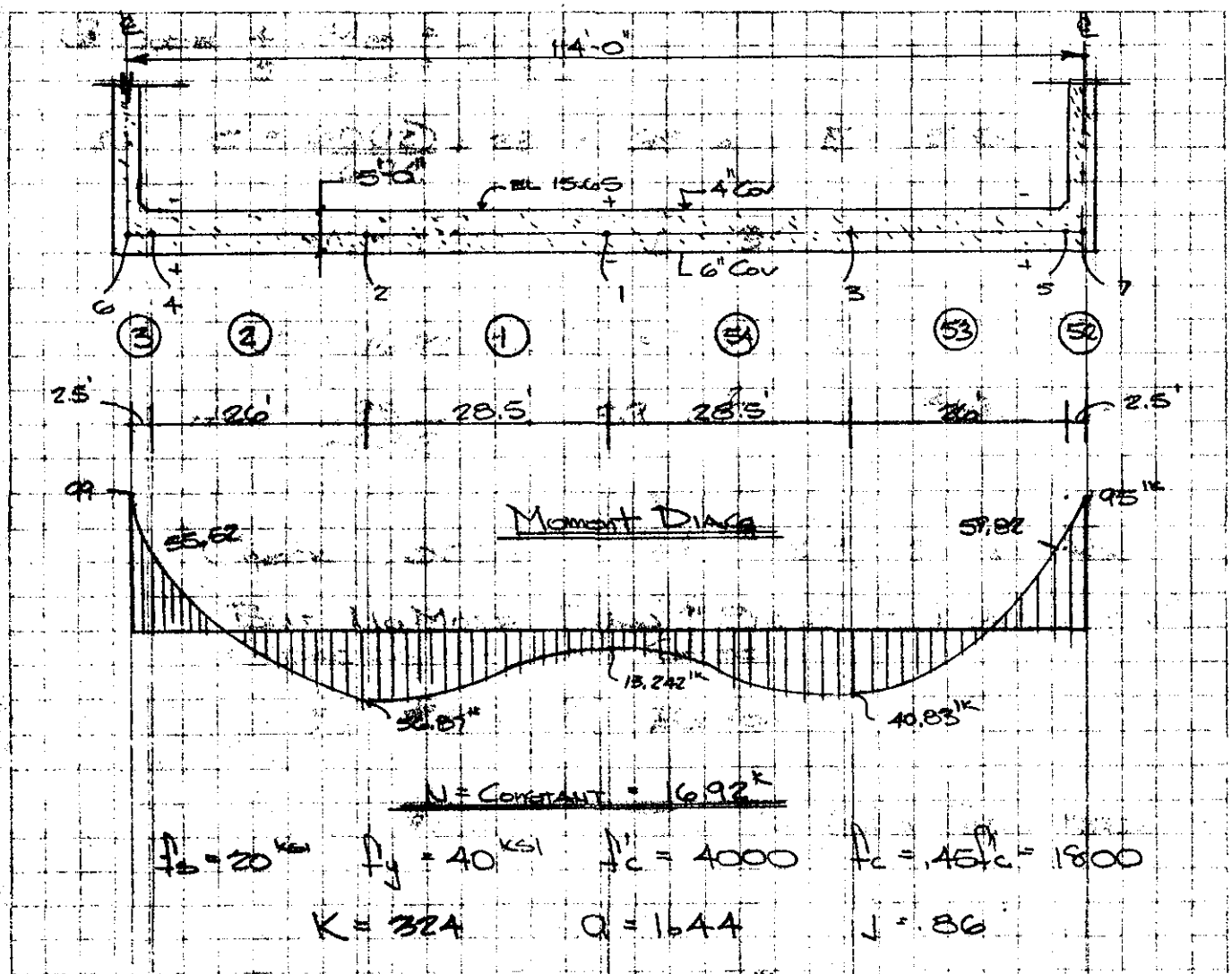
COMPUTED BY _____ CHECKED BY _____ DATE 9 OCT 74

36	10.363	-4.131	-15.067
	-10.363	10.812	-24.991
37	10.363	-10.314	24.233
	-10.363	13.340	-43.426
38	10.363	12.999	43.439
	-10.363	-10.472	-24.967
39	10.363	10.439	24.363
	-10.363	-4.174	11.633
40	10.363	4.174	-11.633
	-10.363	.373	13.230
41	10.363	-.373	-13.230
	-10.363	5.930	4.663
42	10.363	-5.930	-4.663
	-10.363	12.245	-40.775
43	10.363	-12.245	40.775
	-10.363	15.403	-75.335
44	15.403	10.111	75.335
	-15.403	-9.403	-53.630
45	15.403	9.403	53.630
	-15.402	-7.515	-27.534
46	15.402	7.515	27.534
	-15.402	-4.733	-2.635
47	15.402	4.733	2.635
	-15.402	-1.499	10.132
48	15.402	1.499	-10.132
	-15.402	2.478	3.455
49	15.402	-2.478	-3.455
	-15.402	7.030	-10.429
50	15.402	-7.030	10.429
	-15.402	11.345	-44.335
51	15.402	-11.345	44.335
	-15.402	15.917	-94.995
BASE SLAB 52	15.917	12.773	-59.322
	-15.917	-15.401	94.994
53	15.917	-1.062	40.323
	-15.917	-12.773	59.321
54	15.917	.215	13.242
	-15.917	1.062	-40.323

SUBJECT: Park River Local Protection

COMPUTATION: Base Slab - Reinf

COMPUTED BY: W. Rodger CHECKED BY: DATE: 9 Oct 74



N = Constant = 1692^k

$f_s = 20^{ksi}$ $f_y = 40^{ksi}$ $f'_c = 4000$ $f_c = .45 f'_c = 1800$

$K = 324$ $Q = 1.44$ $J = .86$

Consider Member 3 $M_\ominus = 99^{1k}$

$$M_{FWALL} = M - \frac{Vo}{3} = 99^{1k} - \frac{(19.197)(3)}{3} = 80^k$$

$D = 5' \times 12'' = 60''$ $d_\ominus = 60'' - 6'' - 1.4 - .7 = 52''$ Node 4, 64" @ Face of Wall

$$e = \frac{12M}{N} + d'' = \frac{12(80)}{1692^k} + 23'' \pm = 79.74 \quad E = 6.645$$

$e/d = 79.74/64 = 1.25$ $i = 3.20$

$f = \frac{12(52)^2}{12000} = 2.7$ KF = 875 NE = 112.4 (NE - KF) = −
No Comp Steel

$$A_s = \frac{NE}{adc} = \frac{112.43}{1.44 \times 3.20 \times 64} = .38^{in}$$

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION

CORPS OF ENGINEERS, U. S. ARMY

SUBJECT: Park River Local Protection

COMPUTATION: Base Slab - Reinf

COMPUTED BY: W. Rodger CHECKED BY: DATE: 9 Oct 74

@ Node 4 $M_\Theta = 55.52$
5 $M_\Theta = 59.82$

$\therefore \quad e = \frac{60(12)}{16.92} + 23 = 65.6 \qquad E = 5.50$

$e/d = 1.26 \qquad i = 3.2$

$NE = 16.92(5.50) = 93$

$A_s = \frac{93}{1.44 \times 3.2 \times 52} = .39 \text{ in}^2 \qquad \#6@12" = .44$

Check $.002bt = 1.44 \text{ in}^2/2 = .72 \text{ in}^2/\text{ea face}$

But No More Than #9@12 - Match 11½" Spacing of Wall Steel

$\therefore$ #9@11½" Bottom,

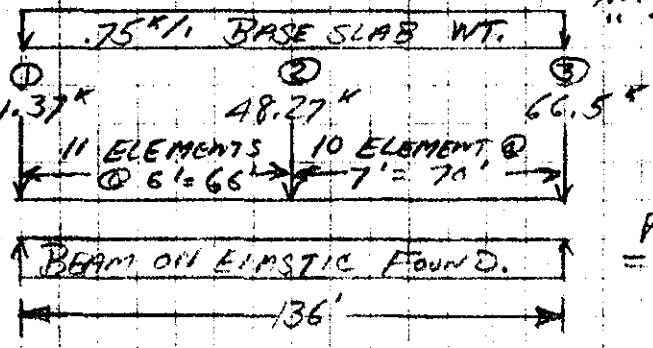
#9@11½ Top is Also Required

SUBJECT PARK RIVER – JUNCTION STRUCTURE
COMPUTATION REINF. – BASE SLAB (SECT. A/B PLATE 7A-7)
COMPUTED BY WJH CHECKED BY DATE 9 OCT. 74

USE 30' WIDE SECTION IN CENTER OF BOX
~136' LONG FROM PANTS SECTION OF SECTION
5 & 7 TO PANTS SECTION OF SECTION 4

INPUT FILE "PARK 15"

*ALL PIER LOADS WERE DIVIDED BY 30 TO OBTAIN LOADING FOR A 1' STRIP



FOUND. MODULUS = 170 K/CU.FT.

FOR PIER LOADS FROM BEAM SEE OUTPUT FOR 6'x4' BEAM IN CENTER OF JUNCTION BOX & 4.5'x10' OVER SECTION 4 OUTPUT

① 310K BEAM REACTION + 13 x 10 (5 x .15 + 1.67 x .12) ROOF & EARTH LOAD ON SECTION 7 REACTING ON CENTER PIER + 10 x 20 (2.167 x .15 + 2 x .12) ROOF & EARTH LOAD ON SECTION 5 + 26 x 10 x 5 x .15 ~PIER WT.
= 941K 941/30 = 31.37K

② 672.5K + 635.8K (BEAM REACTIONS ON CENTER PIER FOR BEAM # 2 + 140K (PIER WT.)
= 1448.3K 1448.3/30 = 48.27K

③ 512K (6'x4') BEAM REACTION + 1343K 4.5x10' BEAM OVER SECTION 4 REACTION + 140K PIER WT. = 1995K
= 1995K/30 = 66.5K

SECTION A/B ON PLATE 7A-7 FALLS APPROXIMATELY 18' FROM LEFT PIER ∴ WE'RE INTERESTED IN THE MOMENT AT NODE 4 OR ELEMENT 4 NODE I

M = 208.2 I-K a = 1.44 d = 60" - 4 - .5 - .7
TOP STEEL = 54.8

A_s = 208.2 / (1.44 x 54.8) = 2.64□"

USE #11's @ 8½" A_s = 2.88□"

SUBJECT PARK RIVER - JUNCTION STRUCTURE
COMPUTATION REINF. - BASE SLAB (SECT A/B PLATE 7A-7)
COMPUTED BY WJH CHECKED BY DATE 9 OCT 74

ELEMENTS ON AN ELASTIC FOUNDATION

ELEMENT	SPRING I	PRESSURE J
1	.387173E+01	.291262E+01
2	.291262E+01	.209411E+01
3	.209411E+01	.148257E+01
4	.148257E+01	.109131E+01
5	.109131E+01	.903967E+00
6	.903967E+00	.889754E+00
7	.889754E+00	.101082E+01
8	.101082E+01	.122308E+01
9	.122308E+01	.147180E+01
10	.147180E+01	.168379E+01
11	.168379E+01	.175830E+01
12	.175830E+01	.157014E+01
13	.157014E+01	.122106E+01
14	.122106E+01	.867122E+00
15	.867122E+00	.630801E+00
16	.630801E+00	.624828E+00
17	.624828E+00	.967698E+00
18	.967698E+00	.178269E+01
19	.178259E+01	.317194E+01
20	.317194E+01	.515702E+01
21	.515702E+01	.757960E+01

NODAL POINT DISPLACEMENTS

NODE	X-DISPL	Y-DISPL	ROT-DISPL
1	0.	-.22775E-01	.96860E-03
2	0.	-.17133E-01	.88799E-03
3	0.	-.12318E-01	.70662E-03
4	0.	-.87210E-02	.49069E-03
5	0.	-.64194E-02	.27942E-03
6	0.	-.53175E-02	.93067E-04
7	0.	-.52338E-02	-.59006E-04
8	0.	-.59460E-02	-.17126E-03
9	0.	-.71946E-02	-.23613E-03
10	0.	-.86577E-02	-.23975E-03
11	0.	-.99847E-02	-.15958E-03
12	0.	-.10343E-01	.35639E-04
13	0.	-.92351E-02	.25100E-03
14	0.	-.71827E-02	.31418E-03
15	0.	-.51007E-02	.26388E-03
16	0.	-.37106E-02	.11774E-03
17	0.	-.36755E-02	-.12430E-03
18	0.	-.56923E-02	-.46957E-03
19	0.	-.10486E-01	-.91529E-03
20	0.	-.18658E-01	-.14240E-02
21	0.	-.30335E-01	-.18921E-02
22	0.	-.44586E-01	-.21149E-02

END ACTIONS WITH ELEMENT LOADS

ELEMENT	AXIAL I AXIAL J	SHEAR I SHEAR J	MOMENT I MOMENT J
1	0.	-31.370	-.000
	0.	15.558	-137.900
2	0.	-15.558	137.900
	0.	5.131	-197.506
3	0.	-5.130	197.506
	0.	-.989	-208.094
4	0.	.990	208.094
	0.	-4.103	-191.643
5	0.	4.103	191.643
	0.	-5.494	-162.290
6	0.	5.494	162.290
	0.	-6.298	-126.874
7	0.	6.298	126.874
	0.	-7.442	-86.020
8	0.	7.442	86.019
	0.	-9.611	-35.500
9	0.	9.611	35.500
	0.	-13.194	32.164
10	0.	13.194	-32.164
	0.	-18.201	125.708
11	0.	18.201	-125.708
	0.	-24.127	252.463
12	0.	-24.143	-252.463
	0.	17.594	107.168
13	0.	-17.594	-107.168
	0.	13.031	1.417
14	0.	-13.031	-1.417
	0.	11.007	-81.261
15	0.	-11.007	81.262
	0.	11.116	-157.719
16	0.	-11.116	157.719
	0.	12.139	-239.078
17	0.	-12.139	239.078
	0.	12.055	-325.149
18	0.	-12.055	325.149
	0.	7.988	-398.619
19	0.	-7.988	398.620
	0.	-3.750	-419.122
20	0.	3.750	419.122
	0.	-27.326	-318.469
21	0.	27.327	318.469
	0.	-66.500	-.000

27 Sept 49

SUBJECT Park Place Local Protection

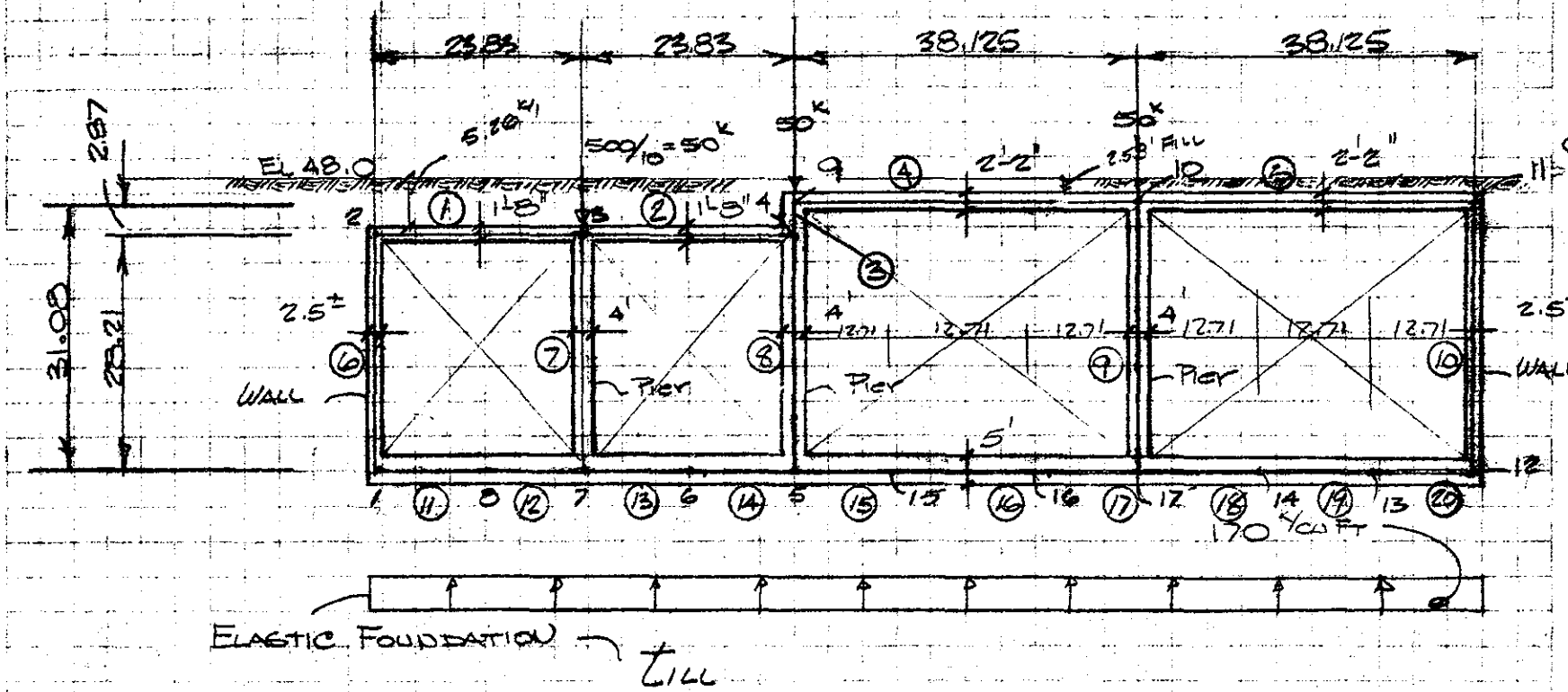
COMPUTATION 1-Strip @ Junc. Struct @ Col 7 & 5

COMPUTED BY W. Rodney CHECKED BY DATE 10 Oct 74

CORPS OF ENGINEERS, U.S. ARMY

PAGE P-1

DMT-113



SUBJECT Pack River

COMPUTATION 1-Step Jorct. Struct @ Col 7 # 5

COMPUTED BY W. Podger CHECKED BY DATE 10 Oct 74

Members	Area	Moment of Inertia
Roof Slabs		
1, 2	A = 1×1.67 = 1.67	I = .383
4, 5	A = 1×2.17 = 2.17	I = .852
Walls		
6	A = 2.5	I = 1.302
7, 8, 3, 9	A = 4.0	I = 5.333
10	A = 2.5	I = 1.302
Base Slab		
11 - 20	A = 5.0	I = 10.417

SUBJECT: Park River

COMPUTATION: Junction Structure

COMPUTED BY: W Rodger CHECKED BY: DATE: 10 OCT 74

NODAL POINT ARRANGEMENT		
NODE	X-COOR	Y-COOR
1	0.	0.
2	0.	28.21
3	23.83	28.21
4	47.66	28.21
5	47.66	0.
6	35.75	0.
7	23.83	0.
8	11.92	0.
9	47.66	31.08
10	85.79	31.08
11	123.92	31.08
12	123.92	0.
13	111.21	0.
14	98.50	0.
15	60.37	0.
16	73.08	0.
17	85.79 ✓	0.

ELEMENT PROPERTIES				
ELT	I	J	AAUC	IAUC
1	2	3	1.67	.388
2	3	4	1.67	.388
3	4	9	4.00	5.333
4	9	10	2.17	.852
5	10	11	2.17	.852
6	1	2	2.5	1.302
7	7	3	4.0	5.333
8	5	4	4.0	5.333
9	17	10	4.0	5.333
10	12	11	2.5	1.302
11	1	8	5.0	10.42
12	8	7	5.0	10.42
13	7	6	5.0	10.42
14	6	5	5.0	10.42
15	5	15	5.0	10.42
16	15	16	5.0	10.42
17	16	17	5.0	10.42
18	17	14	5.0	10.42
19	14	13	5.0	10.42
20	13	12	5.0	10.42

SUBJECT: Park River Local Protection P-4

COMPUTATION: Typ. Loads - Lat Press & Gravity

COMPUTED BY: W. Rodger CHECKED BY: DATE: 11 Oct 74

Roof Slab

DL	1.67 × .150	=	250	DL	2.17 × .150	=	325
Soil	5.26 × .120	=	631	Soil	2.58 × .120	=	310
LL			250	LL			400
			1131 PSF				1035 PSF

Concentrated Loads @ Nodes 3, 9, 10 from Beams = 50^k

Including Weights of Walls

2 = 2.5(.150) × 28' = 10.5^k

3 = 50^k + 4(.150) × 28' = 66.8^k

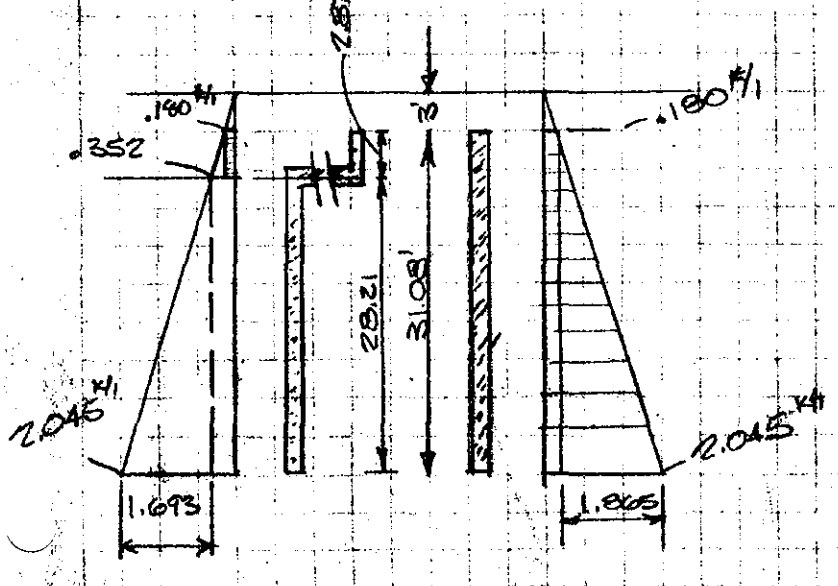
9 = 50^k + 4(.150) × 31' = 68.6^k

10 = 50^k + 4(.150 × 31' = 68.6^k

11 = 2.5(.150) × 31' = 11.63^k

Base Slab

5 × .150 = .750 k/'



$K_o \gamma_s = .06$ PSF

SUBJECT Park River Local Protection

COMPUTATION Typ. Computer Output

COMPUTED BY W. Rodger CHECKED BY DATE 11 Oct 74

ELEMENTS ON AN ELASTIC FOUNDATION

ELEMENT	SPRING PRESSURE I	SPRING PRESSURE J
11	.330718E+01	.384705E+01
12	.384705E+01	.440547E+01
13	.440547E+01	.443190E+01
14	.443190E+01	.432239E+01
15	.432239E+01	.365079E+01
16	.365079E+01	.347689E+01
17	.347689E+01	.369375E+01
18	.369375E+01	.306576E+01
19	.306576E+01	.262833E+01
20	.252833E+01	.277118E+01

NODAL POINT DISPLACEMENTS

NODE	X-DISPL.	Y-DISPL.	ROT-DISPL.
1	0.	-.19454E-01	-.26556E-03
2	.29500E-03	-.19978E-01	.74400E-04
3	-.43520E-04	-.27093E-01	.20018E-04
4	-.44047E-03	-.26725E-01	-.11019E-03
5	-.34036E-03	-.25426E-01	.18398E-03
6	-.26018E-03	-.26070E-01	.17543E-04
7	-.17994E-03	-.25915E-01	-.13151E-03
8	-.90007E-04	-.22630E-01	-.30595E-03
9	-.10178E-04	-.26839E-01	-.19633E-03
10	-.47625E-03	-.23269E-01	-.92722E-05
11	-.89912E-03	-.17003E-01	.47365E-03
12	-.91340E-03	-.16301E-01	-.82413E-04
13	-.81477E-03	-.15461E-01	.37605E-04
14	-.71614E-03	-.18034E-01	.33979E-03
15	-.43274E-03	-.21475E-01	.27107E-03
16	-.52513E-03	-.20452E-01	-.10080E-03
17	-.61751E-03	-.21728E-01	.74400E-04

SUBJECT Park River Local Protection

COMPUTATION Typ. Computer Output

COMPUTED BY W. Rodger CHECKED BY DATE 11 OCT 74

END ACTIONS WITH ELEMENT LOADS

ELEMENT	AXIAL I AXIAL J	SHEAR I SHEAR J	MOMENT I MOMENT J
1	13.048	15.035	72.588
	-13.048	11.917	-35.429
2	15.300	13.203	51.435
	-15.300	13.749	-57.940
3	87.573	-13.825	67.724
	-87.573	14.588	-108.615
4	14.588	18.973	108.615
	-14.588	20.492	-137.584
5	13.236	19.995	124.468
	-13.236	19.470	-114.459
6	25.535	20.762	69.114
	-25.535	13.048	-72.588
7	91.919	-2.252	-47.517
	-91.919	2.252	-16.006
8	101.321	1.475	51.391
	-101.321	-1.475	-9.784
9	109.087	1.352	28.909
	-109.087	-1.352	13.116
10	31.070	-21.340	-90.271
	-31.070	-13.236	114.459
11	20.765	-25.535	-69.114
	-20.765	-8.083	-41.202
12	20.765	8.083	41.202
	-20.765	-48.645	289.735
13	18.513	-43.275	-242.218
	-18.513	-.756	-11.296
14	18.513	.756	11.296
	-18.513	-44.290	258.024
15	19.988	-57.031	-309.415
	-19.988	15.695	-143.232
16	19.988	-15.695	143.232
	-19.988	-19.218	-118.525
17	19.988	19.218	118.525
	-19.988	-55.656	353.[illegible]
18	21.340	-53.430	-382.785
	-21.340	19.399	-71.105
19	21.340	-19.399	71.105
	-21.340	-6.563	-146.708
20	21.340	6.563	146.708
	-21.340	-31.070	90.271

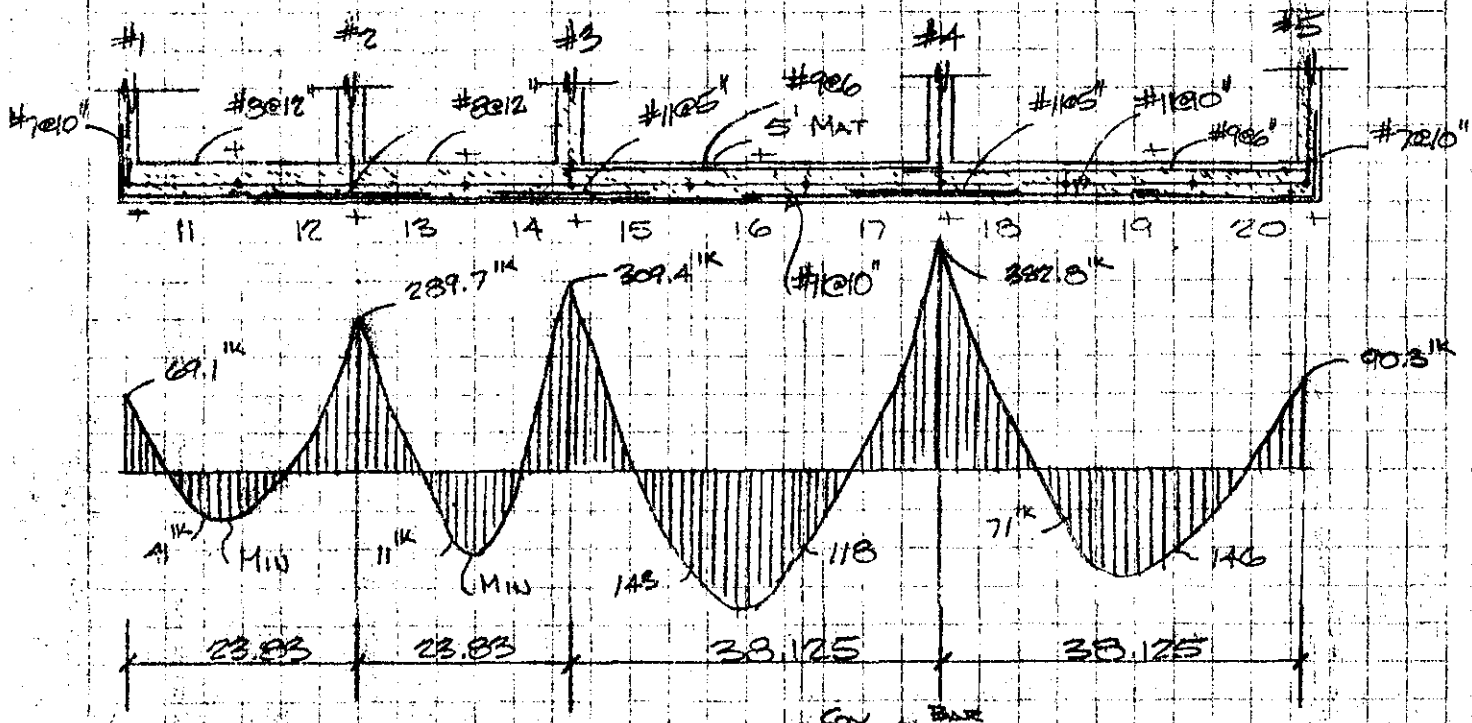
BASE SLAB

SUBJECT: PARK RIVER LOCAL PROTECTION

COMPUTATION: DESIGN BASE SLAB

COMPUTED BY: W. Rodger CHECKED BY: DATE: 11 OCT 74

FOR THIS SECTION (+) - TENSION FACE



CONSIDER $d_{(+)} = 60'' - 4.0'' - 1.4'' - .7^{\pm} = 54'' \pm$

$d_{(-)} = 60'' - 6.0 - 1.4 - .7^{\pm} = 52'' \pm$

MIN STEEL IS $= .002bt = .002(12)(60)$
$= 1.44 \text{ IN}^2$

BUT NOT > #8@12" - $.79 \text{ IN}^2$

$M_{CAP} = A_s \cdot a \cdot d = \frac{.79(1.44)(52)}{12} =$

$M_{(-)} = 382.8 - \frac{53.43(4)}{3} = 312$

$e = \frac{312(12)}{20^K} + 23'' = 210.2$ $E = 17.52$ $e/d = 4.0$

$f = 2.70$ $K = 324$ $= 874$ $NE = 351$ $i = 1.27$

$A_s = \frac{351}{1.44 \times 1.27 \times 52} = 3.69 \text{ IN}^2 = \underline{\#11@5'' \ (3.74)}$

USE @ PIER #3 & #4

SUBJECT: Park River Local Protection

COMPUTATION: Design Base Slab

COMPUTED BY: W. Rodger CHECKED BY: DATE: 11 Oct 74

Corner Steel Bottom

$M = 90.1 - \frac{31.07(8.5)}{3} = 64.21^{1K}$

$e = \frac{64.21(12)}{20} + 23 = 62$ $E = 5.17$ $e/d = 1.2$ $i = 3.53$

$A_s = \frac{5.17(20)}{1.44 \times 3.53 \times 52} = .39^{in^2} < (Min\ .79^{in^2})$

or #7@10" = $.72^{in^2}$)

Use @ Both Corners #1 & #5

Bottom Steel #2 Pier

$M_{DES} = 289.7 - \frac{48.65(4)}{3} = 225$ ← Governs

$242.2 - \frac{43.3(4)}{3} = 185$ $D = 19^K$ Avg

$e = \frac{225(12)}{19} + 23 = 165.1$ $E = 13.76$ $e/d = 3.2$

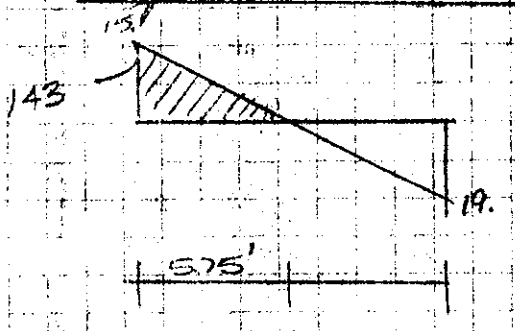
$i = 1.38$

$A_s = \frac{19(13.76)}{1.44 \times 1.38 \times 52} = 2.53^{in^2}$

#10@10 1.57

#9@10 Alt 1.20

2.77 ok



$M_D = 143 + \frac{15.7 \times 5.75}{2}$

$= 188$

$e = 136$ $E = 11.33$ $e/d\ 2.5$ $i = 1.52$

$A_s = \frac{20(11.33)}{1.44 \times 1.52 \times 54} = 1.92^{in^2}$

#9@6

SUBJECT PARK RIVER LOCAL PROTECTION 1/
COMPUTATION TEE TYPE "A" FLOODWALL STA. 42+92 - STA. 43+17
COMPUTED BY J.A.C. CHECKED BY W. Rodger DATE Sept. 74

STABILITY ANALYSIS

Top Elevation	54.5
Base El.	22.0
L.S. Grade (avg.)	39.0
R.S. Grade (avg.)	31.5

Soil Properties

$\phi = 25°$ $K_A = 0.40$, $K_P = 2.46$

$\gamma_{sub} = 60$ pcf
$\gamma_{sat} = 120$ pcf

DESIGN LOADINGS

LC 1 Water to top of wall
LC 2 Water to 3' below top of wall
LC 3 No water, 50 PSF wind on L.S. face, 3' surcharge L.S.
LC 4 Water to El. 21.0 on riverside, to El. 40.0 on L.S., 50 PSF wind on L.S.

Monolith Length 25' (@ STEM ℄ on curve w/ R = 69')

$\theta = 20.76°$ $\ell = \frac{\pi r \theta}{180}$

Mon. Length @ R.S. heel = 33.53
" @ L.S. toe = 19.77

Note: steel sheet cut-off (MA-22) driven to 10' depth on RIVERSIDE

- Extreme riverside pile driven first. MA-22 butted against H pile, using H pile as template, driven to 10' depth

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

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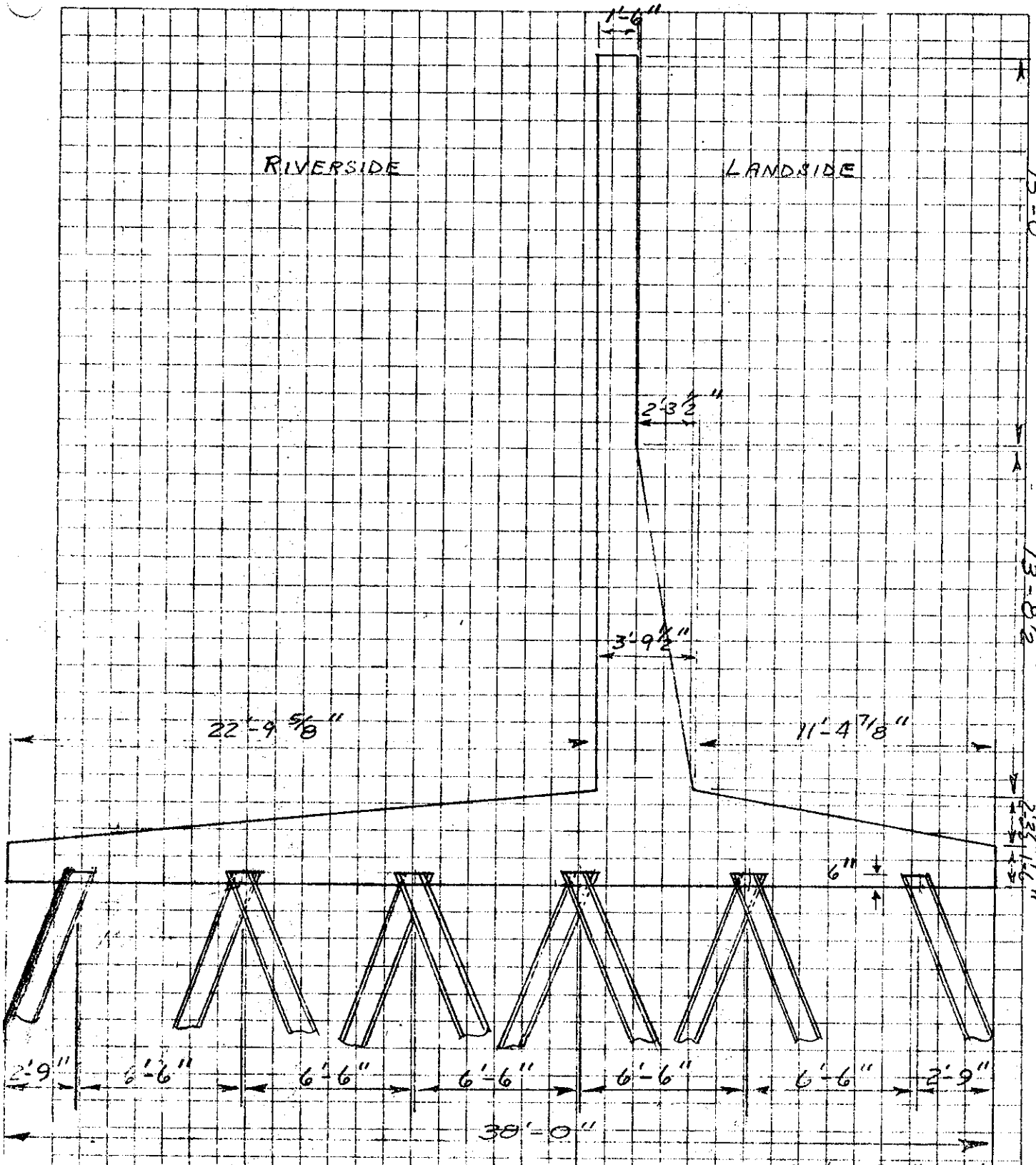
SUBJECT: PARK RIVER
COMPUTATION: TEE TYPE "A" FLOOD WALL @ STA. 43+92
COMPUTED BY: J.A.G. CHECKED BY: WHR DATE: Sept. 74

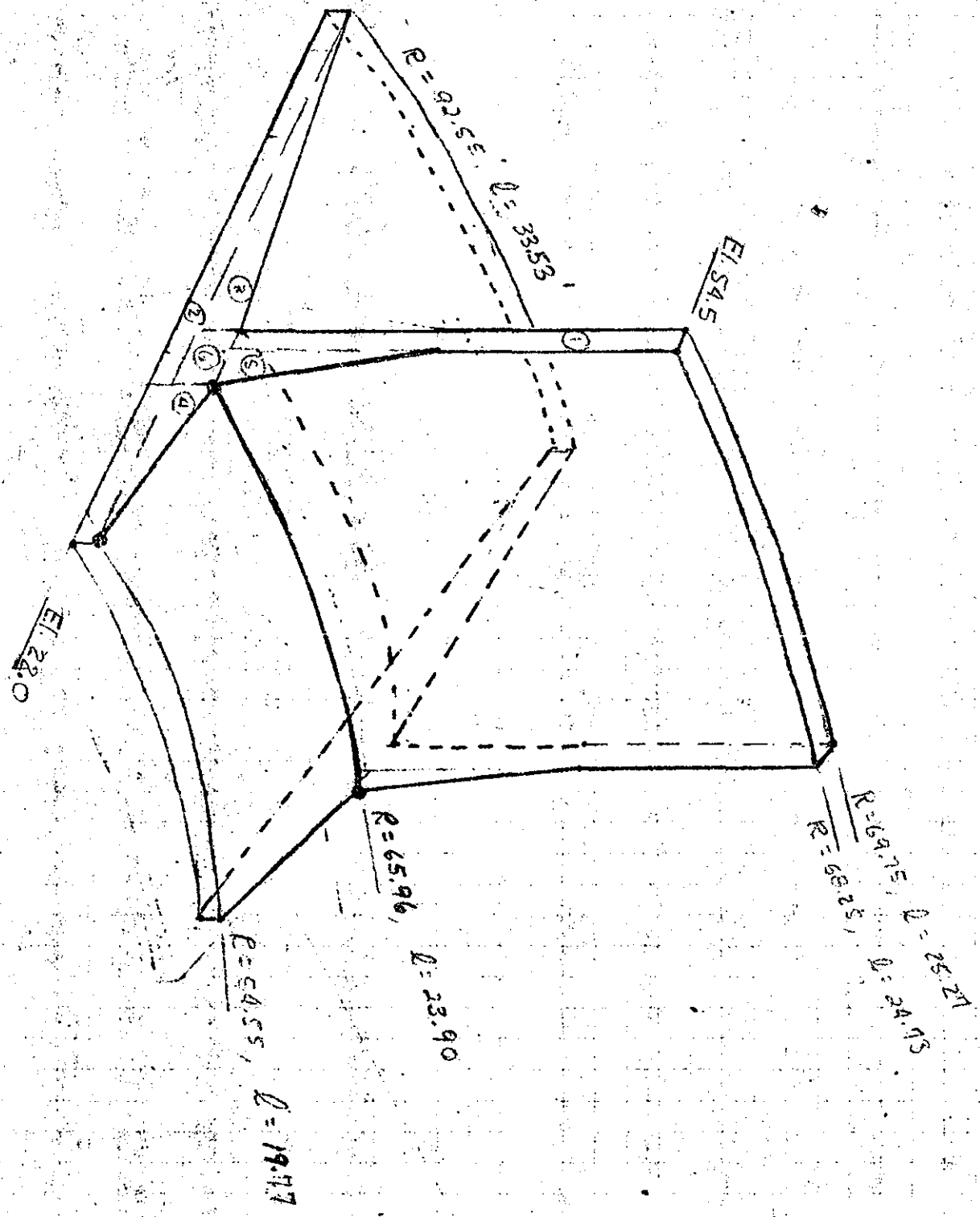


MA 22 Sheet Piling TYPICAL SECTION
Driven to 10' depth

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT PARK RIVER LOCAL PROTECTION
COMPUTATION TEE TYPE "A" FLOODWALL @ STA. 42+92
COMPUTED BY J.A.C. CHECKED BY DATE Sept 74



CORPS OF ENGINEERS, U.S. ARMY

SUBJECT PARK RIVER LOCAL PROTECTION

COMPUTATION TEE TYPE "A" FLOODWALL STA. 42+92 LOAD CASE I

COMPUTED BY J.A.C. CHECKED BY DATE Sept 74

PAGE 4/ DM7-124

LOADING CONDITION 1

LOAD	TYPE		V +↓	H +→	ARM	MOM toe +↺
C_1	I	V_1	$\pi(69.75)^2(20.76)/360\ (28.71)(.15)$ 3795.67		-8.30	-31,504.06
		$-V_2$	$-\pi(68.25)^2(20.76)/360\ (28.71)(.15)$ -3634.17		-9.30	33,797.78
C_2	I	V_1	$\pi(92.55)^2(20.76)/360\ (1.5)(.15)$ 349.15		6.81	2377.71
		$-V_2$	$-\pi(54.55)^2(20.76)/360(1.5)(.15)$ -121.30		-18.38	2229.49
C_3	III	V_1	$(.5)(22.8)(33.53)(2.29)(.15)$ 131.30		22.80	2993.64
		$-V_2$	$-(69.75)^2(.00394)(2.29)(.15)$ -6.58		14.48	-95.28
		$-2V_3$	$-2(4.13)(2.29)(22.8)/3\ (.15)$ -21.56		20.90	-450.68
C_4	II	V_1	$(.5)(2.29)(11.41)(19.77)(.15)$ 38.74		7.61	294.81
		V_2	$(65.96)^2(.00394)(2.29)(.15)$ 5.89		10.73	63.20
		$2V_3$	$2(2.065)(2.29)(11.41)/3\ (.15)$ 5.41		8.56	46.31
C_5	*		$\left(\frac{68.25}{65.96}\right)(.5)\left(\frac{20.76\pi}{360}\right)(68.25^2-65.96^2)(13.71)(.15)$ 59.23		12.99	769.40
C_6	I	V_1	$\pi(69.75)^2(20.76)/360\ (2.29)(.15)$ 302.75		-8.30	-2512.83
		$-V_2$	$-\pi(65.96)^2(20.76)/360(2.29)(.15)$ -270.75		-10.82	2929.52
Σ Concrete			633.77			10,939.01
E_{L1}	I	V_1	$\pi(65.96)^2(20.76)/360\ (13.21)(.12)$ 1249.46		-10.82	-13,519.16
		$-V_2$	$-\pi(54.55)^2(20.76)/360\ (13.21)(.12)$ -854.57		-18.38	15,707.00
E_{L2}	III	V_1	$(.5)(11.41)(23.9)(2.29)(.12)$ 37.47		3.80	142.38
		$-V_2$	$-(54.55)^2(.00394)(2.29)(.12)$ -3.22		-0.56	1.80
		$-2V_3$	$-2(2.065)(2.29)(11.41)/3\ (.12)$ -4.32		2.85	-12.30
E_{L3}	*		$\left(\frac{65.96}{68.25}\right)(.5)\frac{(\pi)(20.76)}{360}(68.25^2-65.96^2)(13.71)(.12)$ 44.27		12.15	537.88
E_{R2}	II	V_1	$(.5)(2.29)(22.8)(25.27)(.12)$ 79.16		30.40	2406.58
		V_2	$(92.55)^2(.00394)(2.29)(.12)$ 9.27		37.05	343.45
		$2V_3$	$2(4.13)(2.29)(22.8)/3\ (.12)$ 17.25		32.30	557.20
E_{R3}	I	V_1	$\pi(92.55)^2(20.76)/360\ (5.71)(.06)$ 531.64		6.81	3620.47
		$-V_2$	$-\pi(69.75)^2(20.76)/360\ (5.71)(.06)$ -301.96		-8.30	2506.27
Σ Earth			804.45			12,291.57

LOADING CONDITION 1 (cont'd)

LOAD	TYPE			V + ↓	H +	ARM	MOM toe +
WR1	I	V_1	$\pi(92.55)^2(20.76)/360\,(28.71)(.0625)$	2784.47		6.81	18,962.24
		$-V_2$	$-\pi(69.75)^2(20.76)/360\,(28.71)(.0625)$	-1581.53		-8.30	13,126.70
U_1	I	$-V_1$	$-\pi(92.55^2)(20.76)/360\,(1.32)$	-2048.34		6.81	-13,949.20
		V_2	$\pi(54.55)^2(20.76)/360\,(1.32)$	711.61		-18.38	-13,079.39
U_2	II	$-V_1$	$-(.5)(.71)(38.0)(19.77)$	-266.70		25.33	-6755.44
		$-V_2$	$-(92.55)^2(.00394)(.71)$	-23.96		37.05	-887.72
		$-2V_3$	$-2(6.88)(.71)(38.0)/3$	-123.75		28.5	-3526.83
Σ Concrete				633.77			10,939.01
Σ Earth				804.45			12,291.57
P WB			$-(32.5)^2(.0625)(.5)(25)$		-825.20	32.5/3	-8939.67
P WL			$(17)(21.085)(.0625)(.5)(25)$		280.04	17/3	1586.89
P AL			$(17)^2(.06)(.29)(.5)(25)$		62.80	17/3	356.19
M/V = 11.38' (from toe)				890.82	-482.30		10,124.38

Creep Dist. = 9.5 + 38.0 + 17.0 = 64.5

Hyd. Press. @ L.S. toe — Posit. Pot. 17.0; Seep. Pot. 17/64.5 (15.5) = 4.085 } 21.085(.0625) = 1.32 ksf

@ P.S. heel — 32.5(.0625) = 2.03 ksf

SUBJECT PARK RIVER LOCAL PROTECTION
COMPUTATION TEE TYPE "A" FLOODWALL STA 42+92 LOAD CASE 2
COMPUTED BY TAC CHECKED BY DATE Sept 74
CORPS OF ENGINEERS, U.S. ARMY
PAGE DM7-126

LOADING CONDITION C

LOAD	TYPE			V +↓	H ←+	ARM	MOM toe +↻
Σ Concrete				633.77			10,939.01
Σ Earth				804.45			12,291.57
WR1	I	V_1	$\pi(92.55)^2(20.76)/360\ (25.71)(.0625)$	2493.51		6.81	16,980.80
		$-V_2$	$-\pi(69.75)^2(20.76)/360\ (25.71)(.0625)$	-1416.27		-8.30	11,755.04
U1	I	$-V_1$	$-\pi(92.55)^2(20.76)/360\ (1.27)$	-1970.75		6.81	-13,420.81
		V_2	$\pi(54.55)^2(20.76)/360\ (1.27)$	684.65		-18.38	-12,583.87
U_2	II	$-V_1$	$-(.5)(.57)(38.0)(19.77)$	-214.11		25.33	-5423.38
		$-V_2$	$-(92.55)^2(.00394)(.57)$	-19.24		37.05	-712.84
		$-2V_3$	$-2(6.88)(.57)(38.0)/3$	-99.35		28.50	-2831.40
PWR			$-(29.5)^2(.0625)(.5)(25)$		-679.88	29.5/3	-6685.49
PWL			$(17)(20.29)(.0625)(.5)(25)$		269.47	17/3	1527.00
PAL			$(17)^2(.06)(.29)(.5)(25)$		62.86	17/3	356.19
M/V = 13.60' (from toe)				896.66	-347.55		12,191.28

Hyd. Press. @ LS toe — Posit Pot. 17.0; Seep Pot. (17/60.5)(12.5) 3.29 } 20.29(.0625) — 1.27 ksf

Hyd. Press @ RS heel — 29.5(.0625) — 1.84 ksf

1.5

LOADING CONDITION 3

LOAD TYPE				V+↓	H ⊣	ARM	MOM toe +↻
Σ Concrete				635.77			10,939.01
Σ Earth - ER3				574.77			6164.83
ER3	I	V_1	$\pi(92.55)^2(20.76)/360\ (5.71)(.12)$	1063.28		6.81	7240.94
		$-V_2$	$-\pi(69.75)^2(20.76)/360\ (5.71)(.12)$	-603.92		-8.30	5012.54
SUR	I	V_1	$\pi(68.25)^2(20.76)/360\ (3.0)(.12)$	303.80		-9.30	-2825.34
		$-V_2$	$-\pi(54.55)^2(20.76)/360\ (3.0)(.12)$	-194.07		-18.38	3567.01
P_{AEL}			$(20.0)^2(.12)(.29)(.5)(25)$		174.00	20/3	1160.00
P_{WIND}			$(12.5)(.05)(25)$		15.63	26.25	410.29
P_{AER}			$-(9.5)^2(.12)(.29)(.5)(25)$		-39.26	9.5/3	-124.32
			M/V = 17.75	1777.63	150.37		31,544.96

SUBJECT PARK RIVER LOCAL PROTECTION
COMPUTATION TEE TYPE "A" FLOODWALL STA 42+92 LOAD CASE 3
COMPUTED BY T.A.C. CHECKED BY DATE Sept 74
CORPS OF ENGINEERS, U.S. ARMY
PAGE DM 7-127

27 Sept 49

SUBJECT: PARK RIVER LOCAL PROTECTION

COMPUTATION: TEE TYPE "A" FLOODWALL @ STA. 42+92 LOAD CASE 4

COMPUTED BY: J.A.C. CHECKED BY: DATE: Sept 74

CORPS OF ENGINEERS, U.S. ARMY

NEW ENGLAND DIVISION

PAGE DMT-12.8

LOAD TYPE				V+↓	H ←	ARM	MOM toe +↺
Σ CONCRETE				633.77			10,939.01
Σ Earth (LC 3)				1034.13			18,418.31
U_1	I	$-V_1$	$-\pi(92.55)^2(20.76)(.379)/360$	-588.12		6.81	-4005.11
		V_2	$\pi(54.55)^2(20.76)(.379)/360$	204.32		-18.38	-3755.35
U_2	III	$-V_1$	$-(.5)(38.0)(.746)(33.39)$	-472.70		12.67	-5989.15
		V_2	$(54.55)^2(.00394)(.746)$	8.75		-0.56	-4.90
		$2V_3$	$2(6.845)(.746)(38.0)/3$	129.36		9.50	1228.93
Wat.	I	V_1	$\pi(68.25)^2(20.76)(.0625)(1)/360$	52.74		-9.30	-490.51
		$-V_2$	$-\pi(54.55)^2(20.76)(.0625)(1)/360$	-33.69		-18.38	619.29
P_{WN}			$(18)^2(.0625)(.5)(25)$		253.13	18/3	1518.75
P_{WR}			$(5)(6.066)(.0625)(.5)(25)$		-23.70	5/3	-39.49
P_{AL}			$(17)^2(.06)(.29)(.5)(25)$		62.86	17/3	356.21
P_{WND}			$(.05)(15.5)(25)$		19.38	25.75	498.91
M/V = 19.91' (from toe)				968.56	311.67		19,294.90

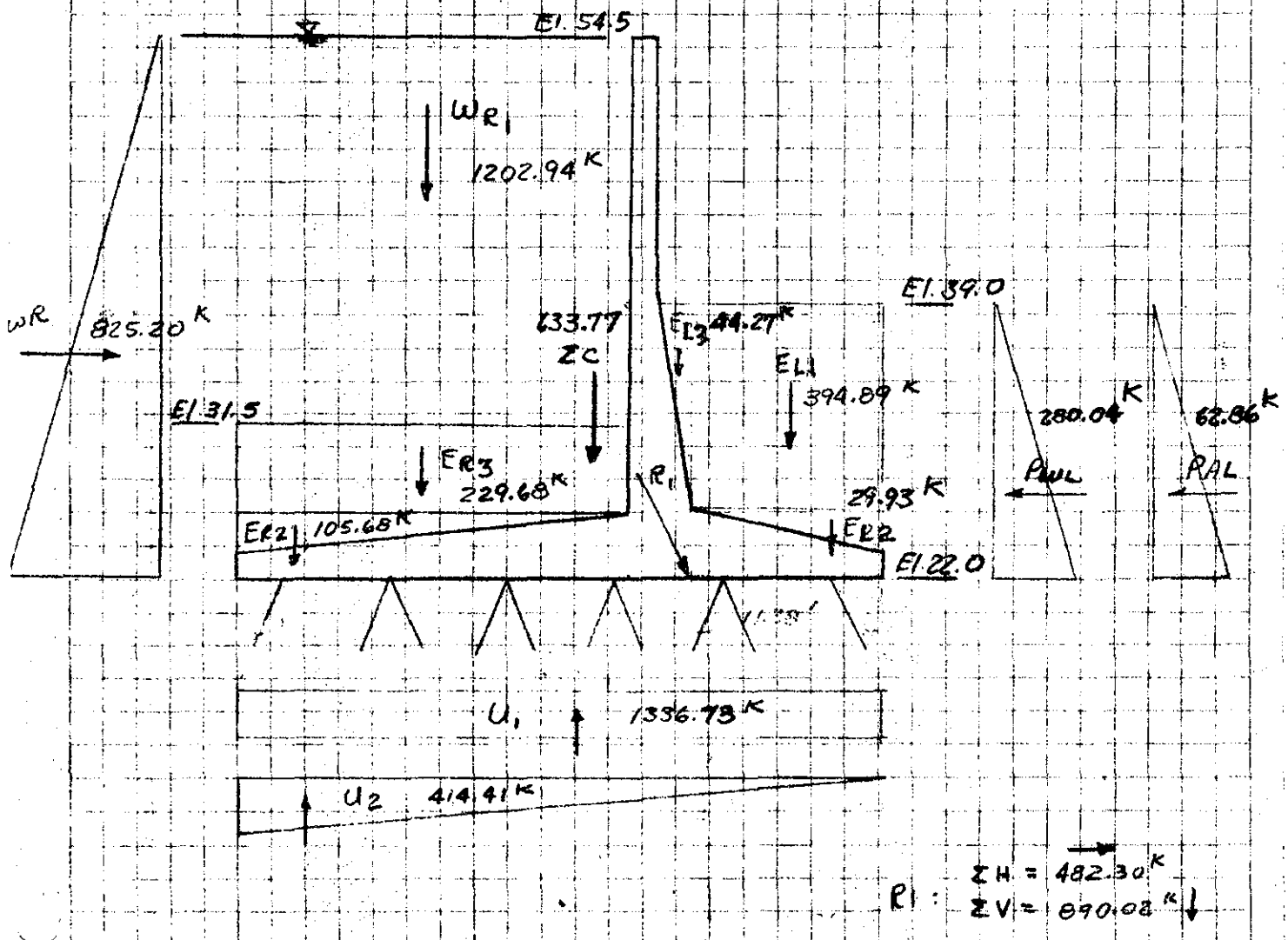
C.D. = 5 + 38 + 18 = 61

Hyd. Press. @ L.S. toe (18)(.0625) 1.125 ksf

@ R.S. heel Pos. Pot 5.0, Seep. Pot. $\frac{5}{61}(13) = 1.066$ } 6.066(.0625) = 0.379 ksf

SUBJECT: PARK RIVER
COMPUTATION: TEE TYPE "A" FLOODWALL @ STA 42+92
COMPUTED BY: J.A.C. CHECKED BY: DATE: Sept 74

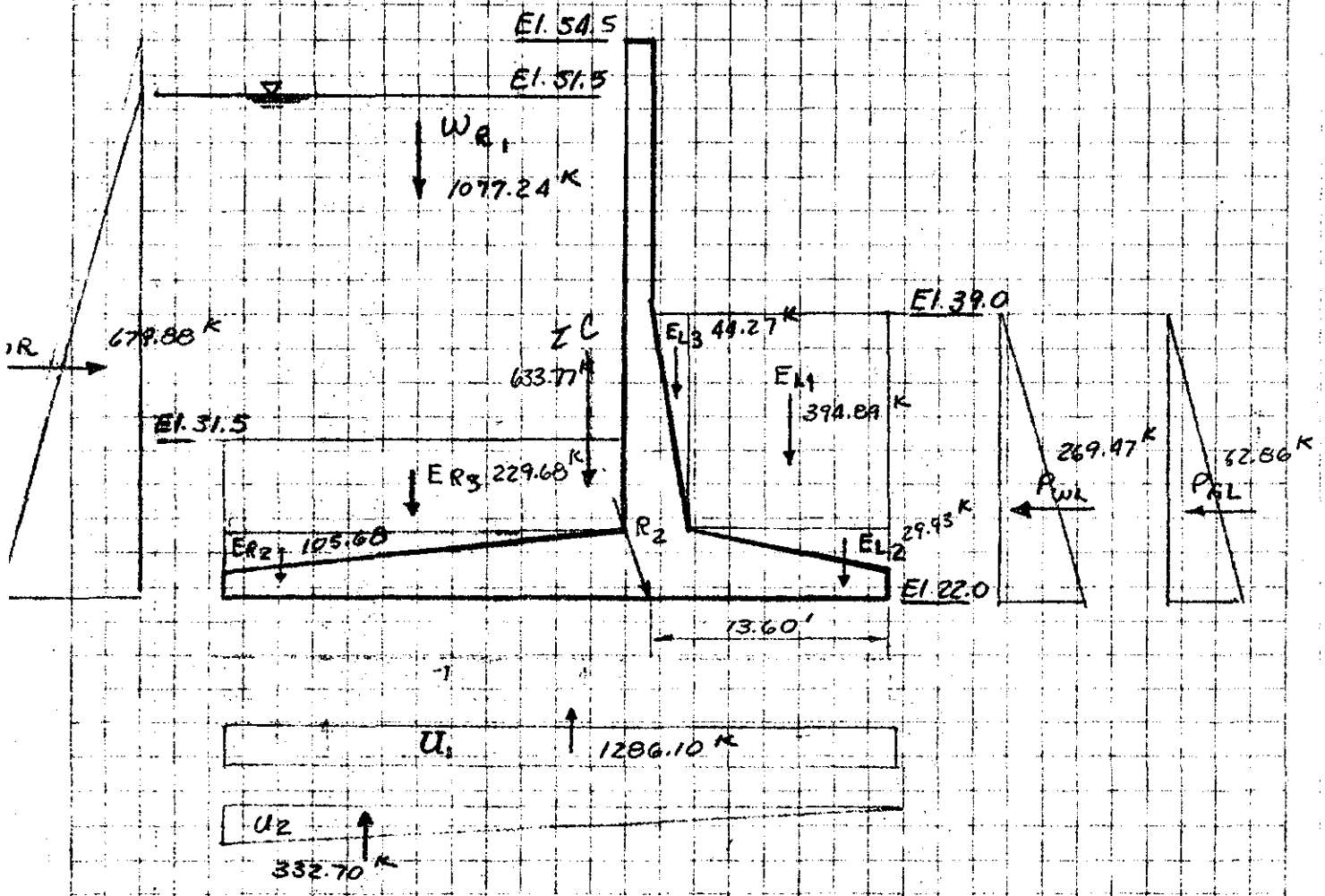
LOADING CONDITION 1



R_1: $\Sigma H = 482.30^K \rightarrow$
$\Sigma V = 890.02^K \downarrow$

SUBJECT PARK RIVER
COMPUTATION TEE TYPE "A" FLOODWALL @ STA. 42+92
COMPUTED BY J.A.C. CHECKED BY DATE Sept. 74

LOADING CONDITION 2



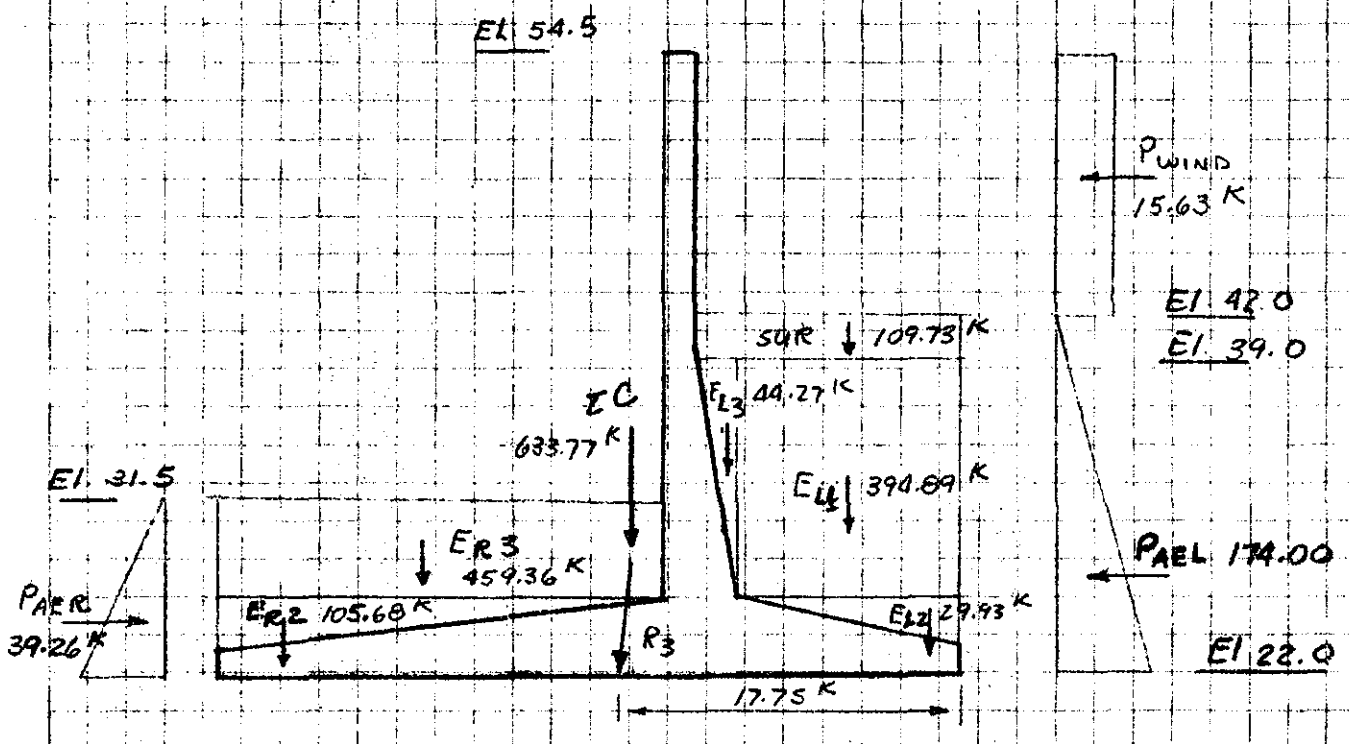
R_2: $\Sigma H = 347.55^{K} \rightarrow$
$\Sigma V = 894.66^{K} \downarrow$

SUBJECT PARK RIVER, "

COMPUTATION TEE TYPE "A" FLOODWALL @ STA. 42+92

COMPUTED BY J.A.C. CHECKED BY DATE Sept. 74

LOADING CONDITION 3



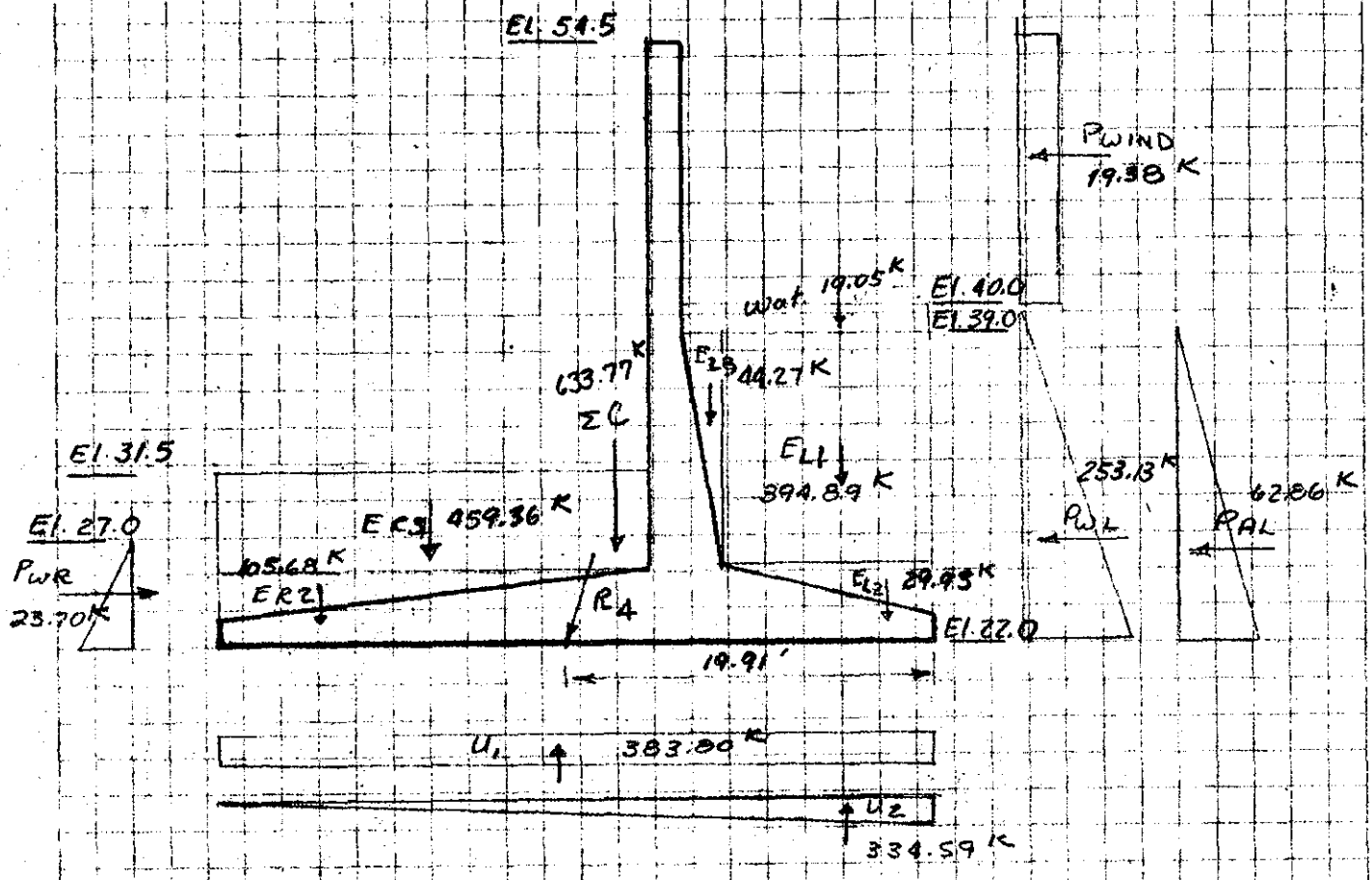
R_3: ΣH 150.37 K ←
ΣV 1777.63 K ↓

SUBJECT PARK RIVER

COMPUTATION TEE TYPE "A" FLOODWALL @ STA. 42+92

COMPUTED BY J.A.C. CHECKED BY DATE Sept. 74

LOADING CONDITION 4



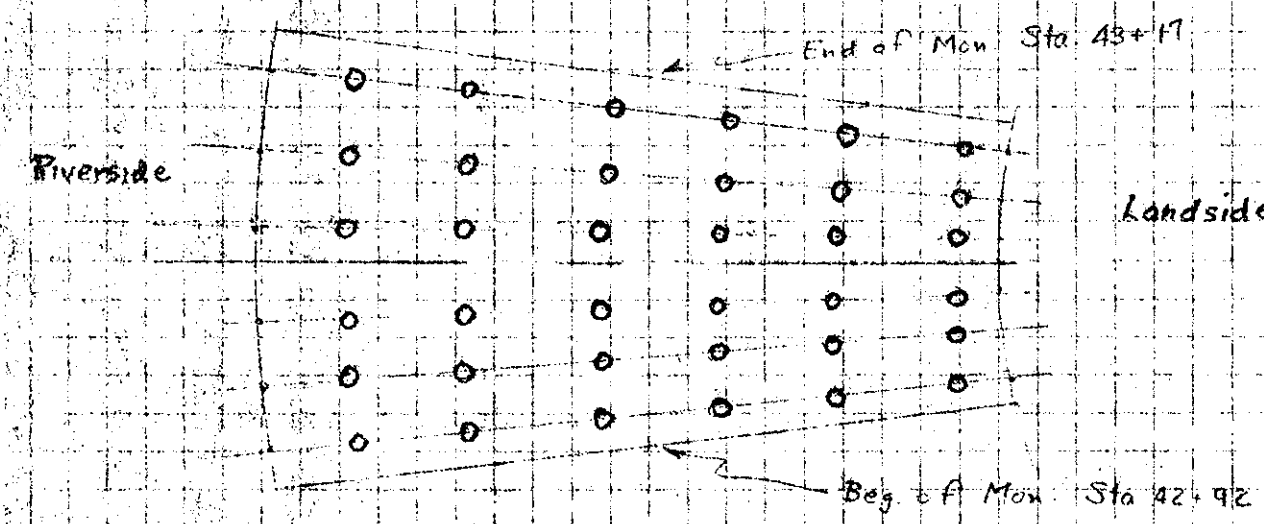
R_4: ΣH = 311.67 K ←

ΣV = 968.56 K

SUBJECT PARK RIVER LOCAL PROTECTION 1/1

COMPUTATION TEE TYPE "A" FLOODWALL @ STA. 42+92 - PILE SET-UP

COMPUTED BY J.A.C. CHECKED BY DATE Sept 74



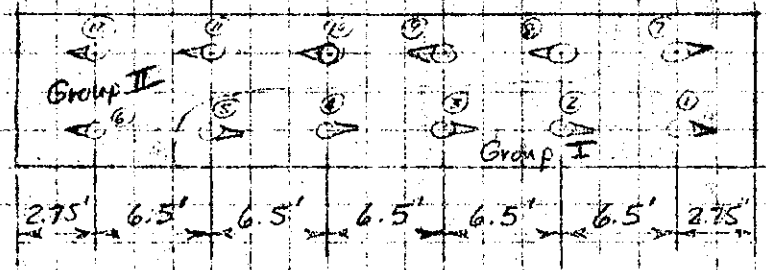
MONOLITH PLAN

length of Mon. @ stem ℄ = 25.00'

longitudinal pile spacing @ stem ℄ 4'-2"

dist. from outer pile row to edge of monolith 2'-1" } @ stem ℄

ALL PILE LOADS 50 TON - COMPRESSION
8 TON - TENSION



- try this arrangement (repetitive every two rows) of piles, with all piles battered 5 on 12 in the direction shown

long. pile spacing @ extreme R.S. piles = 5.67'
long. pile spacing @ extreme L.S. piles = 3.32'

SUBJECT PARK RIVER LOCAL PROTECTION

COMPUTATION TEE TYPE "A" FLOODWALL @ STA. 42+92

COMPUTED BY J.A.C. CHECKED BY DATE Sept. 74

$C.G._{I}$ toe $= 2(2.75) + 9.25 + 15.75 + 22.25 + 28.75 / 6 = 13.58'$

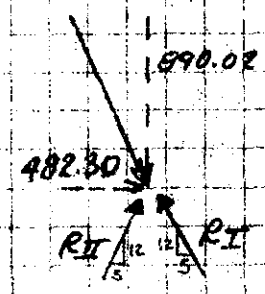
$C.G._{II}$ toe $= 13.58'$

Σr^2 (entire mon.) $= 6(2.00)^2 + 6(4.00)^2 + 6(8.00)^2 + 6(14.00)^2 + 12(10.00)^2 = 2880.0$

RESULTANT COMP. IN DIRECT. OF BATTER I, II

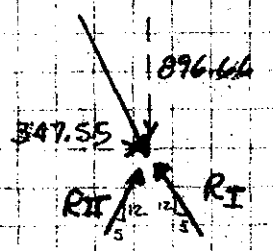
R_I: Resultant in direction of Group I piles
R_{II}: Resultant in direction of Group II piles

L.C. 1



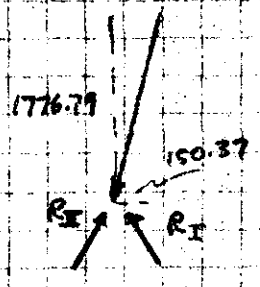
$R_I = 1109.08^K$
$R_{II} = -144.90^K$

LC 2



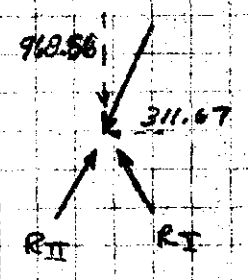
$R_I = 937.51^K$
$R_{II} = 33.88^K$

LC 3



$R_I = 766.95$
$R_{II} = 1157.91$

LC 4



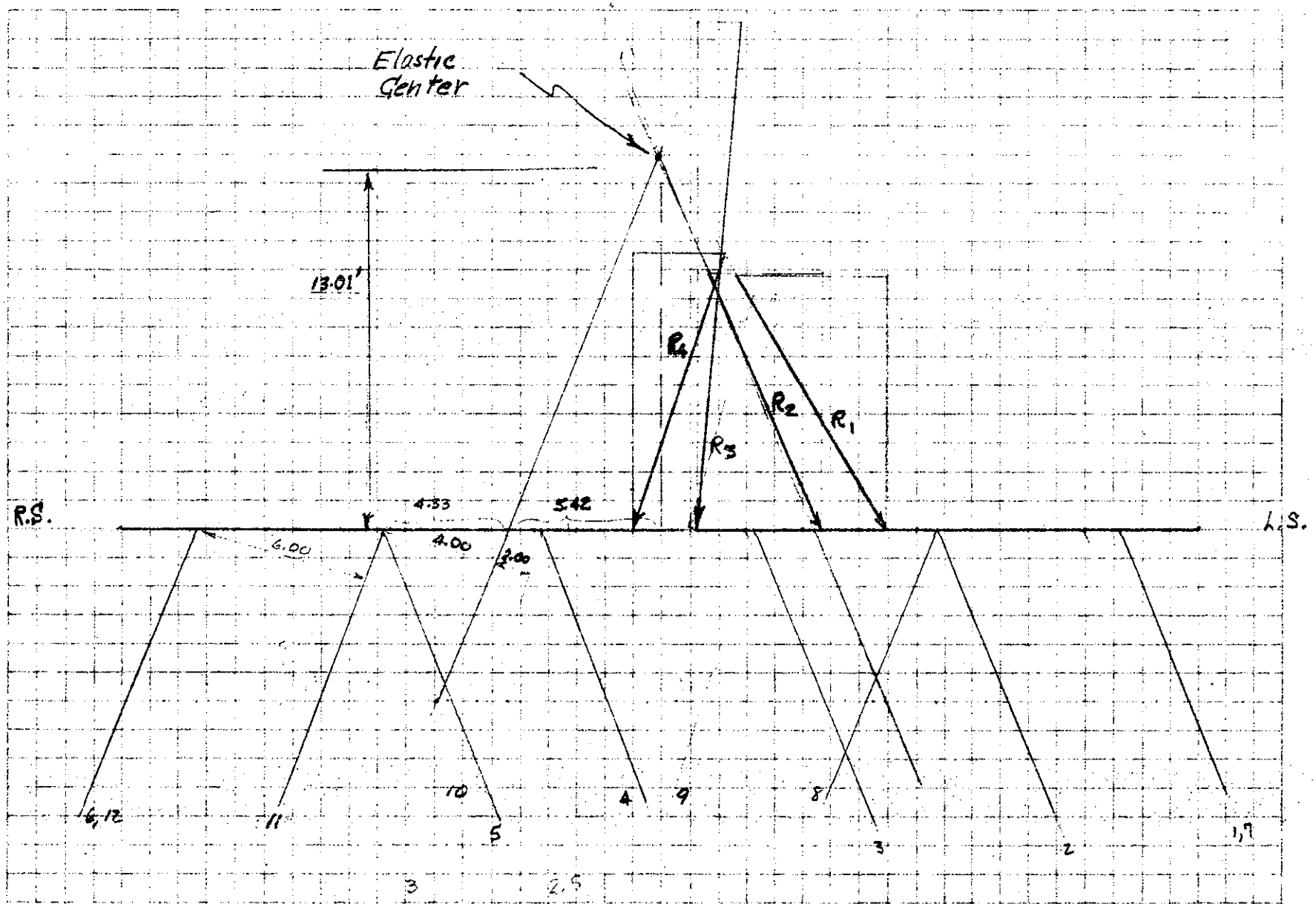
$R_I = 119.65$
$R_{II} = 929.99$

CORPS OF ENGINEERS, U. S. ARMY

SUBJECT PARK RIVER LOCAL PROTECTION

COMPUTATION TEE TYPE "A" FLOODWALL @ STA 42+92

COMPUTED BY J.A.C. CHECKED BY _____ DATE Sept 74



L.C.	M e.c. + ↻		
1	(890.02)(9.62) − (482.30)(13.01)	=	507.23 ^1K
2	(896.66)(5.40) − (347.55)(13.01)	=	−820.34 ^1K
3	(1776.79)(1.25) + (150.37)(13.01)	=	4177.30 ^1K
4	− (968.89)(0.91) + (311.67)(13.01)	=	3173.14 ^1K

LC 1 Max comp. load Pile # 1,7
Max tensile load Pile # 6,12

$$P_{1,7} = \frac{1109.08}{18} + \frac{(507.23)(10.00)}{2880.0} = 63.38^K (C)$$

$$P_{6,12} = -\frac{144.90}{18} - \frac{(507.23)(10.00)}{2880.0} = 9.81^K (T)$$

LC 2 Max comp. load Pile # 1,7
Max tensile load Pile # 6,12

$$P_{1,7} = \frac{937.51}{18} + \frac{(3203.4)(10.0)}{2880.0} = 53.20^K (C)$$

$$P_{6,12} = \frac{33.88}{18} - \frac{(3203.4)(10.0)}{2880.0} = 0.76^K (C)$$

LC 3 Max comp load Pile # 1,7, or Pile 8

$$P_{1,7} = \frac{766.95}{18} + \frac{(4177.30)(10)}{2880.0} = 57.11^K (C)$$

$$P_8 = \frac{1157.97}{18} + \frac{(4177.30)(14)}{2880} = 84.63^K (C)$$

LC 4 Max. comp. load Pile # 8
Max tens. load Pile # 5

$$P_5 = \frac{119.65}{18} - \frac{(3173.14)(14)}{2880.0} = -8.78^K (T)$$

$$P_8 = \frac{929.99}{18} + \frac{(3173.14)(14)}{2880} = 67.09^K (C)$$

Max. Tensile Load 9.81^K
Max. Comp. Load 84.63^K

(MAX. ALL. T = 16^K
MAX ALL C = 100^K)

SUBJECT PARK RIVER LOCAL PROTECTION

COMPUTATION TEE TYPE "A" FLOODWALL @ STA. 42+92

COMPUTED BY J.A.C. CHECKED BY DATE Sept 74

PILE REACTIONS

LC 1

$P_{1,7} = 63.38\ (C)$

$P_2 = 1109.08/18 + (507.23)(4)/2880 = 62.32^K\ (C)$

$P_3 = 1109.08/18 - (507.23)(2)/2880 = 61.26^K\ (C)$

$P_4 = 1109.08/18 - (507.23)(8)/2880 = 60.21^K\ (C)$

$P_5 = 1109.08/18 - (507.23)(14)/2880 = 59.15^K\ (C)$

$P_{6,12} = 9.81\ (T)$

$P_8 = 144.9/18 - (507.23)(14)/2880 = 5.58^K\ (T)$

$P_9 = 144.9/18 - (507.23)(8)/2880 = 16.64^K\ (T)$

$P_{10} = 144.9/18 - (507.23)(2)/2880 = 11.68^K\ (T)$

$P_{11} = 144.9/18 + (507.23)(4)/2880 = 8.75^K\ (T)$

LC 3

$P_{1,7} = 57.11^K\ (C)$

$P_2 = 42.61 + (4177.3)(4)/2880 = 48.41^K\ (C)$

$P_3 = 42.61 - (4177.3)(2)/2880 = 39.71^K\ (C)$

$P_4 = 42.61 - (4177.3)(8)/2880 = 31.00^K\ (C)$

$P_5 = 42.61 - (4177.3)(14)/2880 = 22.30^K\ (C)$

$P_{6,12} = 64.33 - (4177.3)(10)/2880 = 49.82^K\ (C)$

$P_8 = 84.63^K\ (C) = 84.63^K\ (C)$

$P_9 = 64.33 + (4177.3)(8)/2880 = 75.93^K\ (C)$

$P_{10} = 64.33 + (4177.3)(2)/2880 = 67.23^K\ (C)$

$P_{11} = 64.33 - (4177.3)(4)/2880 = 58.53^K\ (C)$

LC 4

$P_{1,7} = 119.65/18 + (3173.14)(10)/2880 = 17.67^K\ (C)$

$P_2 = 119.65/18 + (3173.14)(4)/2880 = 11.05^K\ (C)$

$P_3 = 119.65/18 - (3173.14)(2)/2880 = 4.44^K\ (C)$

$P_4 = 119.65/18 - (3173.14)(8)/2880 = -2.17^K\ (T)$

$P_5 = -8.78^K\ (T)$

$P_{6,12} = 929.99/18 - (3173.14)(10)/2880 = 40.65^K\ (C)$

$P_8 = 67.09^K\ (C)$

$P_9 = 929.99/18 + (3173.14)(8)/2880 = 60.48^K\ (C)$

$P_{10} = 929.99/18 + (3173.14)(2)/2880 = 53.87^K\ (C)$

$P_{11} = 929.99/18 - (3173.14)(4)/2880 = 47.26^K\ (C)$

SUBJECT

COMPUTATION

COMPUTED BY CHECKED BY DATE

AC 2

$P_{1,7} = 937.51/18 + (320.34)(10)/2880 = 53.20^K (C)$
$P_2 = \quad " \quad + (320.34)(4)/2880 = 52.53^K (C)$
$P_3 = \quad " \quad - (320.34)(2)/2880 = 51.86^K (C)$
$P_4 = \quad " \quad - (320.34)(8)/2880 = 51.19^K (C)$
$P_5 = \quad " \quad - (320.34)(14)/2880 = 50.53^K (C)$

$P_{6,12} = 0.76^K (C)$
$P_8 = 33.18/18 + (320.34)(14)/2880 = 3.40^K (C)$
$P_9 = 33.18/18 + (320.34)(8)/2880 = 2.73^K (C)$
$P_{10} = 33.18/18 + (320.34)(2)/2880 = 2.07^K (C)$
$P_{11} = 33.18/18 - (320.34)(4)/2880 = 1.40^K (C)$

SUBJECT Park River
COMPUTATION I-Wall Typical Section
COMPUTED BY JRF CHECKED BY RLH. DATE 9/24/74

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I-WALL STABILITY
FIND MINIMUM PENETRATION (X)
FIND Z DISTANCE
GIVEN WEIGHT OF SOIL,WATER,KP,KA,HEIGHT OF WALL ABOVE GRADE & T.W.ELEV.
BY C.M.TERZIAN                          8JULY66

LOADING:-------RIVERSIDE WATER TO TOP OF WALL
               LANDSIDE WATER AT ANY HEIGHT
               GRADE EQUAL BOTH SIDES
               DRY AND SUBMERGED PASSIVE PRESSURE CONSIDERED DUE TO T.W.

ALL UNITS POUND-FEET

WGT OF EARTH(SAT)      =120
KA                     =.41
KP                     =2.46
WGT OF WATER           =62.5
WALL HGT ABOVE GRADE   =10.0
T.W. WATER TO TOP WALL=10.0
GUESS PENETRATION (X),(>ACTUAL)=30.0
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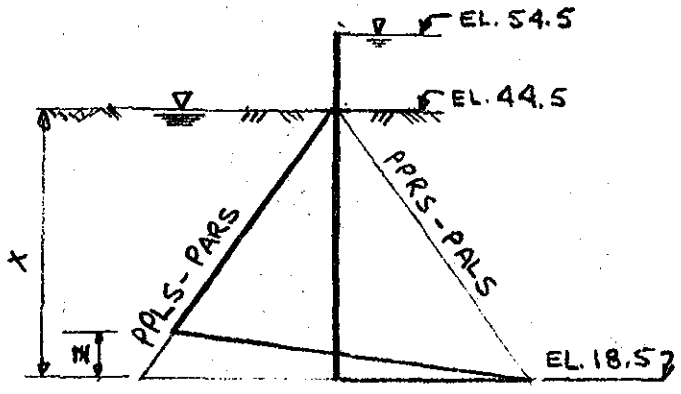


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X, Z, KP =     25.72        6.52        2.46


FIND KP AND Z WHEN PENETRATION (X) IS GIVEN (MUST BE > MINIMUM)
LOADING SAME AS ABOVE


GIVEN PENETRATION (X)=26.0

X, Z, KP =     26.0         6.67        2.46


FIND POINT OF ZERO SHEAR(DIST.DOWN FROM GRADE)     =XO    FT
FIND MAXIMUM MOMENT                                =M     FT-LBS
FIND SECTION MODULUS FOR FS=20000PSI               =SM    CU-IN

XO, M, SM =    14.50    61539.51    36.92
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USE PZ 32 S.M. = 38.3 IN.^{3}/FT.

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PLD=     295.20
PRS=     141.45
ALD=      49.20
ALS=      23.57

END
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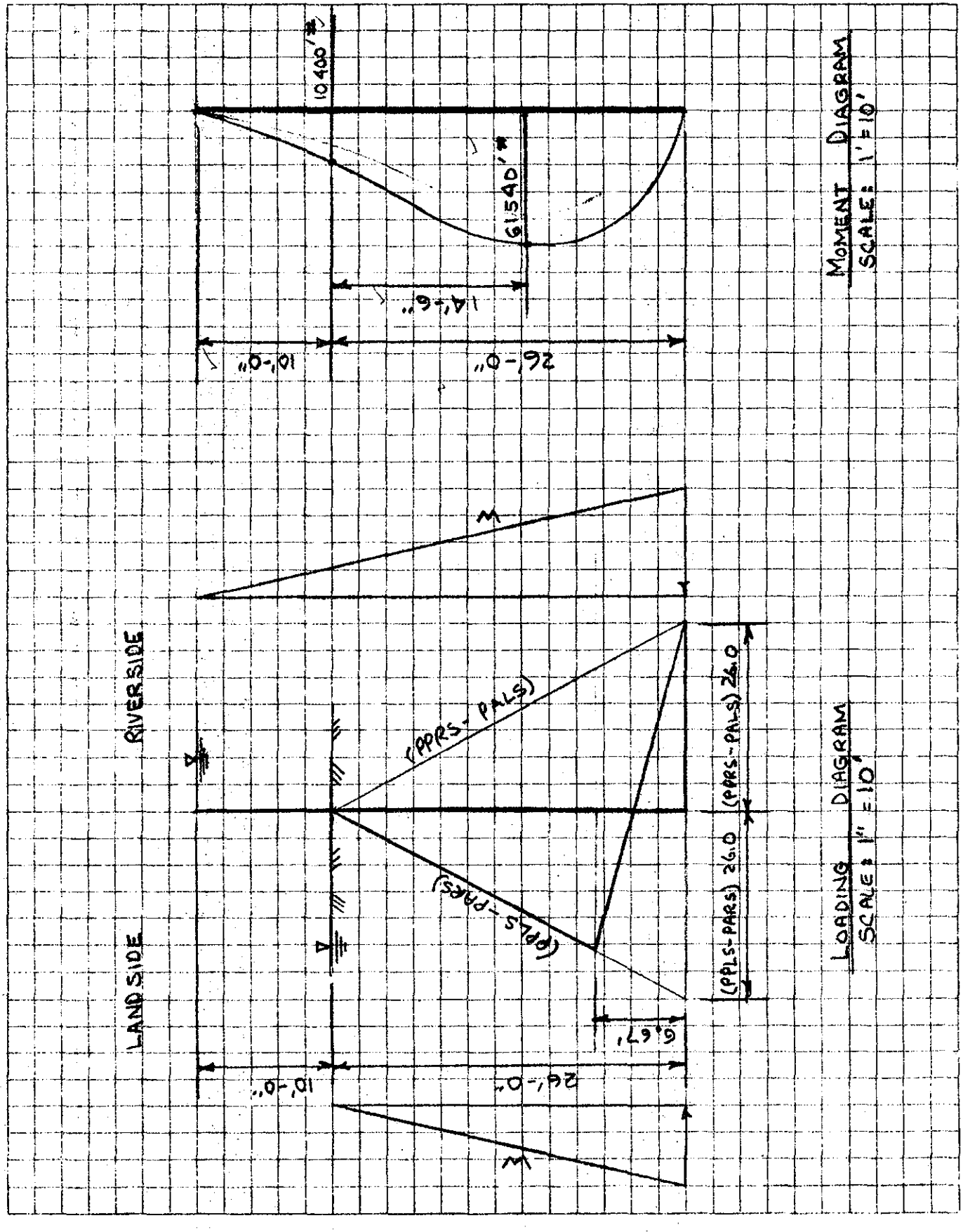
SUBJECT PARK RIVER
COMPUTATION I-Wall
COMPUTED BY JRF CHECKED BY RLH DATE 9/24/74



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REINFORCEMENT

Moment 2' BELOW GROUND LEVEL

$M = 0.0625 \times \frac{12^3}{6} = 18.0$ K·FT

$d = [1.5 + 2(\frac{10}{12})]12 - 4.5$

$d = 33.5''$

$A_s = \frac{18.0}{1.44 \times 33.5}$

$A_s = 0.37 \text{ in}^2$ USE #6@12" ($A_s = 0.44 \text{ in}^2$)

Moment @ GROUND LEVEL

$M = 0.0625 \times \frac{10^3}{6} = 10.4$ K·FT. ✓

$d = 33.5''$ ✓

$A_s = \frac{10.4}{1.44 \times 33.5} = .21 \text{ in}^2$ ✓ USE #5@12"

HORIZONTAL REINF.

$A_s = 0.001 \times 28 \times 12 = 0.34$ USE #5@12"

SUBJECT Park River
COMPUTATION "I"- Wall
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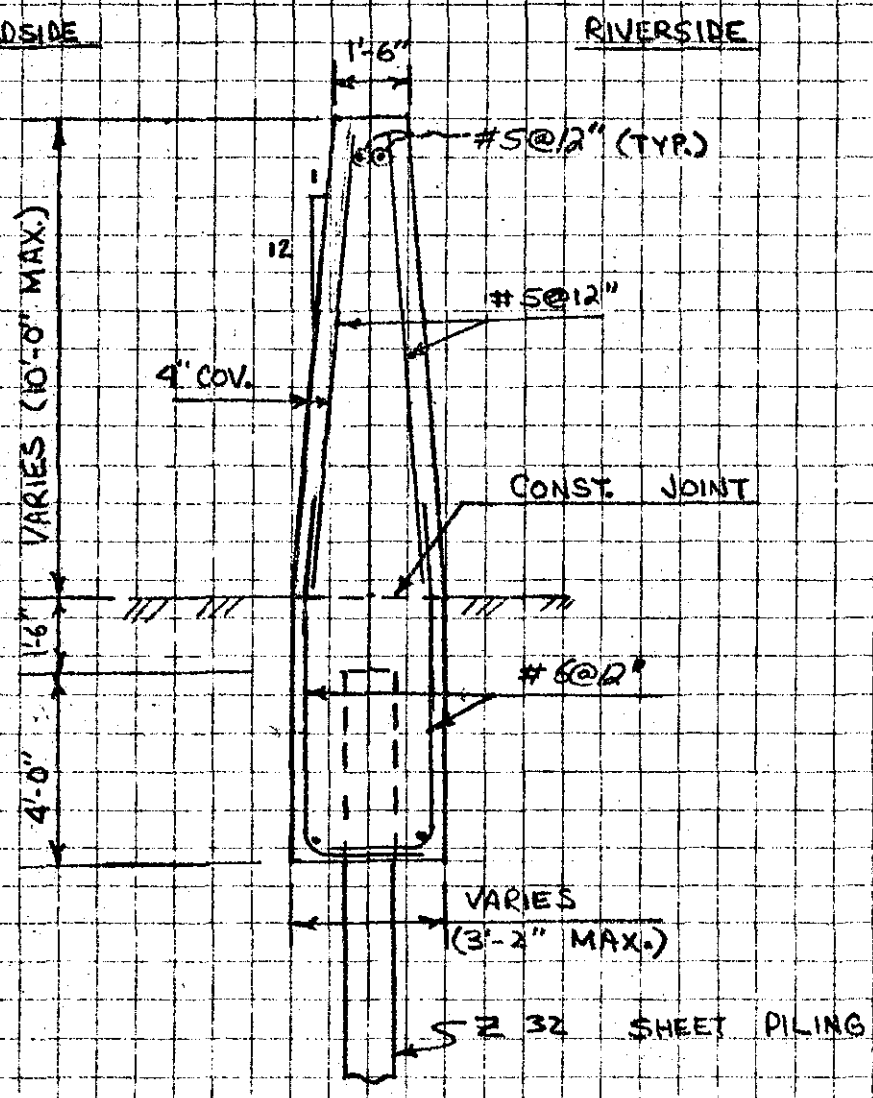

TYPICAL SECTION
SCALE: 1/4" = 1'-0"